Dimenzioniranje čelične konstrukcije objekta u nautičko turističkom kompleksu

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Diplomski rad

Rijeka, 2019.

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Sveučilišni diplomski studij
Smjer: konstrukcije
Čelične konstrukcije

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SAŽETAK

Naslov rada: Dimenzioniranje čelične konstrukcije objekta u nautičko turističkom kompleksu

Ime i prezime studenta: Tea Rojnić

Ime i prezime mentora: izv.prof.dr.sc. Mladen Bulić, dipl.ing.građ.

Ime i prezime komentorice: doc.dr.sc. Paulina Krolo, dipl.ing.građ.

Naziv studija: Sveučilišni diplomski studij Građevinarstvo – modul konstrukcije

Kolegij: Čelične konstrukcije

Tema ovog diplomskog rada je dimenzioniranje čelične konstrukcije hale. Građevina je smještena u Puli. Namijenjena je prodajno izložbenom prostoru za plovila sa ugostiteljskim dijelom, koji se sastoji od restorana i caffe bara. Objekt je pravokutnog tlocrtnog oblika sa dvostrešnim krovom koji u gornjem i donjem djelu ima različite nagibe.

Statička analiza provedena je u programskom paketu *Autodesk Robot Structural Analysis Professional 2019*, a model je oblikovan kao 3D štapni sustav. Analiza djelovanja provedena je sukladno Eurokodu i pripadnim Nacionalnim dodacima. Osim stalnog opterećenja, analizirana su djelovanja snijega i vjetra, dok je vlastita težina konstrukcije uzeta u obzir u softveru. Za sve elemente konstrukcije provedena je provjera graničnog stanja nosivosti, a u karakterističnim točkama provjera graničnog stanja uporabivosti. Također, dimenzionirani su i neki od spojeva: spoj glavnog stupa s temeljem, spoj glavnog stupa i gornjeg pojasa rešetke, te tri čvora rešetki.

Ključne riječi: čelična konstrukcija, statički proračun, dimenzioniranje, Eurokod

SUMMARY

Title: Design of steel structure of the building in nautical tourist complex

Name and surname of student: Tea Rojnić

Name and surname of mentor: Assoc. Prof. Mladen Bulić, dipl.ing.građ.

Name and surname of co-mentor: Asst. Prof. Paulina Krolo, dipl.ing.građ.

University programme: University Graduate Study Programme in Civil Engineering – Structural engineering module

Course: Steel structures

The graduate thesis subject is dimensioning of a steel hall. The building is located in Pula. It is intended to be used for selling vessels and as a showroom with restaurant and coffee shop. The building has a rectangular floor plan with a doublepitched roof with different slopes of the upper and lower surfaces.

Static analysis is performed in program package *Autodesk Robot Structural Analysis Professional 2019* and the model is shaped as a 3D wireframe. An action analysis is performed according to Eurocode and related National Annexes. Except for constant load, there are analyzed actions of snow and wind, while selfweight is considered in software. For all construction elements, Ultimate limit state is verified, and Serviceability limit state is verified at characteristic points of the building. Also, some of the connections are calculated: the main column base, frame knee, and three truss chord nodes.

Keywords: steel structure, static analysis, dimensioning, Eurocode

SADRŽAJ

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POPIS SLIKA

1. UVOD

U ovom diplomskom radu biti će proveden proračun nosive konstrukcije čelične hale namijenjene za prodajno izložbeni prostor za plovila sa ugostiteljskim sadržajima (caffe bar, restoran) smještene u Puli. U prvom djelu rada, konstrukcija građevine, kao i korišteni materijali biti će detaljno opisani, biti će navedene norme koje su korištene prilikom proračuna, način zaštite od korozije i požara, te program kontrole i osiguranje kvalitete. U drugom djelu rada biti će proveden proračun. Prvi korak kako bi se taj proračun mogao provesti je provođenje analize djelovanja. Djelovanja koja će se u ovom slučaju analizirati su stalna (vlastita težina konstrukcije, težina instalacija, pokrova i fasade) i promjenjiva (djelovanje snijega i djelovanje vjetra). Nakon provedene analize djelovanja, dobivene vrijednosti nanose se u 3D štapni model modeliran u programskom paketu *Autodesk Robot Structural Analysis Professional 2019* [1], te se provodi statički proračun. Potom se na osnovu vrijednosti reznih sila i momenata savijanja dobivenih statičkim proračunom provodi dimenzioniranje poprečnih presjeka elemenata konstrukcije, te na osnovu vrijednosti pomaka i progiba provjera graničnog stanja uporabivosti. Na kraju će biti provedeno dimenzioniranje spojeva elemenata konstrukcije – određivanje broja, položaja i kvalitete vijaka, te debljine zavara.

2. TEHNIČKI OPIS

2.1. Opis konstrukcije

Građevina je pravokutna, tlocrtnih dimenzija 22,00m x 80,12m, visine u sljemenu 12,67m. Krov je dvostrešni, u srednjem dijelu građevine na širini od 8m nagib krova iznosi 38,09°, dok u bočnim dijelovima, na širini od 7m sa svake strane nagib krova iznosi 18,45°. Glavna nosiva konstrukcija sastavljena je od 14 okvira na razmacima od 6m, međusobno povezanih sa po tri sekundarne rešetke. Kao pokrov odabrani su Izoforma Euro 5 sendvič paneli [2] debljine 200mm sa unutarnjom i vanjskom čeličnom oblogom od 0,4 mm, dok su za fasadu odabrani Izoforma Isopar ® Elegant sendvič paneli [3] debljine 120mm sa unutarnjom i vanjskom čeličnom oblogom od 0,4mm.

2.1.1. Glavni okviri

Glavne okvire čine vanjski stupovi poprečnog presjeka 2UPN180, visine 7,20m, unutarnji stupovi poprečnog presjeka 2UPN160 razmaknuti 100 mm, visine 10,15m, svi stupovi su upeti u temelje.

Stupovi su međusobno povezani glavnim rešetkama koje se sastoje od poprečnih presjeka 2UPN160, 2CAE70x7, 2CAE100x10 i 2CAE45x5.

2.1.2. Sekundarne rešetke

Sekundarne rešetke povezuju glavne okvire. Svaka dva glavna okvira povezana su sa po dvije bočne rešetke smještene na vrhu unutarnjih stupova i središnjom rešetkom smještenom u sljemenu. Bočne rešetke sastavljene su od poprečnih presjeka 2CAE120x10 i 2CAE70x7, a središnje rešetke od poprečnih presjeka 2UPN80, 2CAE120x10 i 2CAE70x7.

2.1.3. Podrožnice

Na središnjem dijelu krova nagiba 38,09° podrožnice poprečnog presjeka IPN 240 su postavljene okomito na uzdužnu os zgrade, dok su na bočnim dijelovima krova nagiba 18,45° podrožnice poprečnog presjeka IPN 180 postavljene paralelno sa uzdužnom osi zgrade, kao kontinuirani nosači.

2.1.4. Fasadni nosači

Fasadni nosači poprečnog presjeka IPN180 postavljeni su na međusobnom razmaku od 1,44m, a oslanjaju se na glavne stupove okvira poprečnog presjeka 2UPN180, i sekundarne fasadne stupove poprečnog presjeka IPN220 kao kontinuirani nosači.

2.1.5. Stabilizacijski vezovi

Horizontalni i vertikalni stabilizacijski vezovi postavljeni su u uzdužnom i poprečnom smjeru u obliku vlačnih dijagonala okruglog šupljeg poprečnog presjeka. Fasadni vezovi su poprečnog presjeka ϕ17, zabatni vezovi poprečnog presjeka ϕ28, a krovni ϕ15.

2.2. Opterećenja na konstrukciju

Djelovanja na konstrukciju koja je potrebno razmatrati dijele se na:

- stalna djelovanja
    - vlastita težina elemenata konstrukcije
    - težina instalacija, pokrova i obloge
- promjenjiva djelovanja
    - djelovanje snijega
    - djelovanje vjetra.

Sva opterećenja na konstrukciju uzimaju se u obzir sukladno s normom HRN EN 1991-1-1:2014 i pripadnim Nacionalnim dodacima.

2.3. Proračun konstrukcije

Proračun konstrukcije provodi se u programskom paketu *Autodesk Robot Structural Analysis Professional 2019* u skladu sa Eurokodom, te Nacionalnim dodacima. Konstrukcija je oblikovana na način da zadovolji uvjete graničnog stanja nosivosti i graničnog stanja uporabivosti.

2.4. Materijali za izradu konstrukcije

Svi elementi glavne nosive konstrukcije izvedeni su od čelika S355 koji ima granicu popuštanja f_y=355 N/mm^2 i čvrstoću f_u=510 N/mm^2. Za izvedbu svih priključaka korišteni su vijci kvalitete 4,8 koji imaju čvrstoću f_{ub}=400 N/mm^2 i 8,8 koji imaju čvrstoću f_{ub}=800 N/mm^2, te ankeri kvalitete 10,9 koji imaju čvrstoću f_{ub}=1000 N/mm^2. Za izvedbu temelja korišten je beton klase C25/30 i čelik za armiranje B500B.

2.5. Primijenjene norme

Pri proračunu konstrukcije korištene su sljedeće norme i nacionalni dodaci:

- norma HRN EN 1990:2011 i nacionalni dodatak HRN EN 1990:2011/NA:2011 (osnove projektiranja)
- norma HRN EN 1991-1-1:2012 i nacionalni dodatak HRN EN 1991-1-1:2012/NA:2012 (osnovna djelovanja)
- norma HRN EN 1991-1-3:2012 i nacionalni dodatak HRN EN 1991-1-3:2012/NA:2016 (djelovanje snijega)

- norma HRN EN 1991-1-4:2012 i nacionalni dodatak HRN EN 1991-1-4:2012/NA:2012 (djelovanje vjetra)
- norma HRN EN 1993-1-1:2014 i nacionalni dodatak HRN EN 1993-1-1:2014/NA:2015 (opća pravila za čelične konstrukcije)
-norma HRN EN 1993-1-8:2014 i nacionalni dodatak HRN EN 1993-1-8:2012/NA:2019 (priključci čeličnih konstrukcija).

2.6. Antikorozijska zaštita

Elemente čelične konstrukcije potrebno je zaštititi temeljnim premazom u radionici, a podlogu je prethodno potrebno opjeskariti do čistoće Sa $2^{1/2}$. Nakon toga elemente je potrebno odmastiti i nanijeti još jedan temeljni premaz. Debljina nanošenja temeljnih premaza iznosi 2x30µm. Završna obrada vrši se požarnim sredstvima s karakteristikom F60. Nakon što se konstrukcija montira sva eventualna oštećenja premaza potrebno je popraviti.

2.7. Protupožarna zaštita

Potrebna vatrootpornost elemenata konstrukcije iznosi F60, što predstavlja sposobnost elemenata da izdrže požar 60 min. Iz tog razloga nakon dva temeljna premaza, čelični elementi štite se protupožarnim premazom koji ima karakteristike vatrootpornosti F60 i kompatibilan je s temeljnim antikorozijskim premazima.

3. PROGRAM KONTROLE I OSIGURANJA KVALITETE [4]

3.1. Uvjeti za izradu čelične konstrukcije

Za izvođača mora se odabrati tvrtka koja ima prethodna iskustva u izvođenju takvih ili sličnih objekata. Kvaliteta i vrsta materijala od kojeg se izvodi konstrukcija definirana je projektnom dokumentacijom. Ukoliko dođe do odstupanja kvalitete materijala od projektom definirane, te promjene moraju biti odobrene od strane projektanta konstrukcije.

Prije započinjanja radova, nadzorni inženjer mora imati uvid u sljedeću dokumentaciju:

- uvjerenja o kvaliteti osnovnog i dodatnog materijala, sredstava za spajanje te sredstava za antikorozijsku zaštitu,
- uvjerenje o podobnosti pogona za izvođenje zavarivačkih radova
- uvjerenja zavarivača koji će raditi na izradi konstrukcije za vrstu zavarivačkih radova koja će se primjenjivati, za traženu debljinu, materijal i položaj zavarivanja,
- specifikacija postupaka zavarivanja i odobrenje o primjeni postupaka zavarivanja,
- uvjerenja o ispravnosti strojeva za izvođenje zavarivačkih radova
- plan izvođenja zavarivačkih radova
- uvjerenje o podobnosti izvođača za izvođenje antikorozijske zaštite
- ovlaštenja svih odgovornih osoba u sustavu interne kontrole izvođača,
- plan rada interne kontrole izvođača.

Prije početka radova montaže konstrukcije potrebno je ishodovati odobreni *Projekt montaže*.

Za obavljanje tehničkog pregleda konstrukcije potrebno je posjedovati svu prethodno navedenu dokumentaciju. Izvođač radova tijekom izvođenja i montaže konstrukcije mora voditi dnevnike koje propisuje zakon, a ovjerava nadzorni inženjer. Ukoliko se nabava materijala vrši za vrijeme izrade konstrukcije, izvođač je dužan posjedovati uvjerenja o kvaliteti materijala i predočiti ih nadzornom tijelu.

Konstrukcije koje se dostavljaju na gradilište moraju prethodno biti zaprimljene u radionici i imati sve potrebne certifikate o kvaliteti. Zapisnik o prijemu konstrukcije sastavljaju i ovjeravaju: investitor, izvođač radova u radionici, nadzorni inženjer te predstavnik izvođača radova na montaži konstrukcije.

3.2. Propisi

Svi sudionici moraju se držati propisa i normi koje su definirane projektom i poštivati pravila kako bi izvedba bila kvalitetna.

3.3. Opće napomene za izradu čelične konstrukcije u radionici

Pri rezanju materijala mora se paziti da ne dođe do lokalnih zareza, pogotovo kada se radi o vlačnim elementima. Ako ipak dođe do zareza, potrebno ih je izbrusiti i/ili dovariti.

Sva odstupanja elemenata moraju biti u dopuštenim granicama. Ako se dogode odstupanja koja premašuju dopuštene vrijednosti projektant mora bit suglasan sa izvedenim stanjem.

Zavarivačke radove potrebno je kontrolirati prije, za vrijeme i po završetku izvedbe. Prije početka zavarivanja potrebno je kvalitetno pripremiti površine koje se zavaruju – odmastiti ih, ukloniti hrđu i drugu prljavštinu. Nakon izvođenja radova mora se obaviti kontrola dimenzija kao i vizualna kontrola te sve ostale kontrole koje su projektom propisane. Ukoliko je to potrebno, provodi se probno sastavljanje koje mora biti popraćeno zapisnikom ovjerenim od strane nadzornog inženjera. Za vrijeme izvedbe zavarivačkih radova mora se paziti da ne dođe do deformiranja konstrukcije nakon hlađenja varova. Zavarivanje se ne smije vršiti na temperaturama ispod 0°C.

Za dio radova koji po završetku konstrukcije nije dostupan za pregled, zapisnik o preuzimanju sastavlja se u vrijeme dok su svi dijelovi konstrukcije vidljivi.

Prije transporta na gradilište elementi konstrukcije moraju se označiti i osigurati od oštećenja prije i tijekom transporta.

3.4. Elementi konstrukcije

Svi elementi konstrukcije moraju biti u potpunosti u skladu sa specifikacijama, nacrtima i naputcima za izradu.

3.5. Materijal za izradu konstrukcije

Materijali od kojih se izvodi konstrukcija definirani su u tehničkom opisu i statičkom proračunu. Za sav materijal korišten pri izradi potrebno je imati odgovarajuće certifikate o kvaliteti, a broj šarže i lima sa uvjerenja mora se podudarati sa onim na materijalu. Za važne elemente potrebno je prilikom rezanja i valjanja na manje dijelove naznačiti broj šarže i lima. Kod elemenata osjetljivih na zamor, broj šarže i lima mora se prenijeti bez utiskivanja oznaka (npr. bojom).

3.6. Antikorozijska zaštita

Zaštita od korozije mora biti u skladu sa projektom i propisima. Posebno treba paziti na temperaturu i vlažnost zraka. Nakon svakog premaza potrebno je izvršiti provjeru prionjivosti i debljine sloja.

3.7. Protupožarna zaštita

Zaštita od požara mora biti u skladu sa projektom i propisima, te uputama proizvođača. Posebno treba paziti na suhoću i čistoću površine. Nakon svakog premaza potrebno je izvršiti provjeru prionjivosti i debljine sloja.

3.8. Prijem elemenata čelične konstrukcije

Prijem čelične konstrukcije u radionicu vrši se prije isporuke na gradilište, a na temelju radioničkih crteža i specifikacije. Kod zaprimanja elemenata mora se uz prethodno navedenu dokumentaciju dostaviti i sljedeće:

- radioničke nacrte sa specifikacijama
- dnevnik izrade u radionici
- dnevnik zavarivačkih radova u radionici
- dnevnik izvođenja antikorozijske zaštite
- izvješće interne kontrole o kvaliteti izvedenih radova.

Prijem gotove čelične konstrukcije obavlja se na osnovu radioničkih crteža i projekta montaže. Prilikom prijema izvedene konstrukcije potrebno je staviti na uvid i sljedeće dokumente:

- kompletnu dokumentaciju sa primopredaje konstrukcije u radionici
- projekt montaže
- radioničke nacrte sa specifikacijama
- dnevnik izvođenja radova na montaži
- dnevnik zavarivačkih radova na montaži
- dnevnik izvođenja antikorozijske zaštite
- izvješće interne kontrole o kvaliteti izvedenih radova
- uvjerenja o kvaliteti dodatnog materijala, sredstava za spajanje te sredstava za antikorozijsku i protupožarnu zaštitu
- uvjerenje o podobnosti izvođača za izvođenje radova na montaži

- uvjerenja zavarivača koji će raditi na izradi i montaži konstrukcije za vrstu zavarivačkih radova koja će se primjenjivati, za traženu debljinu, materijal i položaj zavarivanja
- specifikacija postupaka zavarivanja i odobrenje o primjeni postupaka zavarivanja
- uvjerenja o ispravnosti strojeva za izvođenje zavarivačkih radova
- plan izvođenja zavarivačkih radova
- uvjerenje o podobnosti izvođača za izvođenje antikorozijske zaštite
- uvjerenje o podobnosti izvođača za izvođenje protupožarne zaštite
- ovlaštenja svih odgovornih osoba u sustavu interne kontrole izvođača
- plan rada interne kontrole izvođača.

4. LISTA PRISTANAKA

	LISTA PRISTANKA	**Odobrenje za:**	**Potpis**
		Tehnički dio	
		Kvaliteta	
		Tržište	

	Investitor	
	Projekt	
	Oznaka	
	Broj	
	Datum	

Br.	Metoda ili vrsta aktivnosti/ispitivanja	Kriterij pristanka	Način pristanka	Znak	Napomene
0.0.	JEZIK - OPĆENITO	Hrvatski		0	
1.0	PRORAČUN KONSTRUKCIJE				
1.1	Eurocode 3	EN 1993 - 1 - 1		0	
1.2	Eurocode 2	EN 1992 - 1 -1		0	
1.3	Eurocode 3	EN 1993 - 1 - 2		0	
2.0	DJELOVANJA				
2.1	Vjetra	Brzina vjetra 108 km/h	Po cijeloj visini	Y	
2.2	Snijeg	100 m n. M. I zona		Y	
2.3	Promjena temperaute	+/- 35 °C		Y	
3.0	OSNOVNI MATERIJAL				
3.1	Konstrukcijski čelik	EN 10025		0	
3.2	Kvalitetna grupa	S 355 JR G2		0	
4.0	VIJČANI PRIKLJUČCI				
4.1	Kvaliteta materijala za vijke	8.8 ISO 898 - 1		0	
4.2	Matica za vijak	8 ISO 898 - 2		0	
4.3	Podložna pločica			0	
4.4	AKZ - vruća galvanizacija	DIN 267 - Part 10		0	

Ref.			
		Y	pristanak naručioca
		N	bez pristanka
		P	djelomična sugl.
		0	podrazumjeva se ali bez navoda nar.

	LISTA PRISTANKA	Odobrenje za:	Potpis
		Tehnički dio	
		Kvaliteta	
		Tržište	

	Investitor	
	Projekt	
	Oznaka	
	Broj	
	Datum	

Br.	Metoda ili vrsta aktivnosti/ispitivanja	Kriterij pristanka	Način pristanka	Znak	Napomene
5.0	ZAVARENI PRIKLJUČCI				
5.1	Osiguranje kvalitete	EN 729 - 2	uvjerenje/izvještaj	0	
5.2	Kvaliteta - kriterij prihvaćanja	EN 25 817		0	
5.3	Kvaliteta za NDE/UT	EN 1 714	uvjerenje 10%	0	
5.4	Kvaliteta za NDE/MT	EN 1 290	uvjerenje 5%	0	
5.5	Kvaliteta - vizualni pregled	EN 970	100%	0	
5.6	Uvjerenje zavarivača	EN 287 - 1		0	
5.7	Kvaliteta za ispitivanje NDE/PT	EN 1 289		0	
5.8	Kvaliteta za NDE/RK	EN 1 435		0	
6.0	DIMENZIONALNA KONTROLA				
7.0	AKZ - GALVANIZACIJA				
7.1	Priprema površine	ISO 8501		0	
7.2	Metoda ispitivanja	KSB 10/5 - 2.0		0	
7.3	Način kontrole	KSB 10/5 - 2.1		0	
8.0	AKZ - VRUĆE CINČANJE	DIN 267		0	
9.0	DOKUMENTACIJA ZA				
9.1	Općenito	ISO 9001	izvještaj	0	
9.2	Zavarivanje	EN 729 - 2		0	
9.3	Odgovornost za zavarivanje	EN 719	ovlaštenje	0	
10.0	IZRADA I MONTAŽA	ENV 1090 - 1		0	

Ref.			
		Y	pristanak naručioca
		N	bez pristanka
		P	djelomična sugl.
		0	podrazumjeva se ali bez navoda nar.

5. ANALIZA DJELOVANJA

Analiza djelovanja provodi se prema normi HRN EN 1991 i pripadnim Nacionalnim dodacima. Djelovanja koja će se razmatrati su stalna (vlastita težina konstrukcije, težina pokrova, fasade i instalacija) i promjenjiva (djelovanje snijega i djelovanje vjetra).

5.1. Stalna djelovanja

- Vlastita težina konstrukcije – u proračun je uzeta u programu *Autodesk Robot Structural Analysis Professional 2019*

- Vlastita težina pokrova – odabrani su Izoforma Euro 5 sendvič paneli debljine 200mm, sa unutarnjom i vanjskom čeličnom oblogom od 0,4 mm [2]: g_p= 0,14 kN/m^2

- Težina instalacija ovješenih na krovište: g_i=0,50 kN/m^2

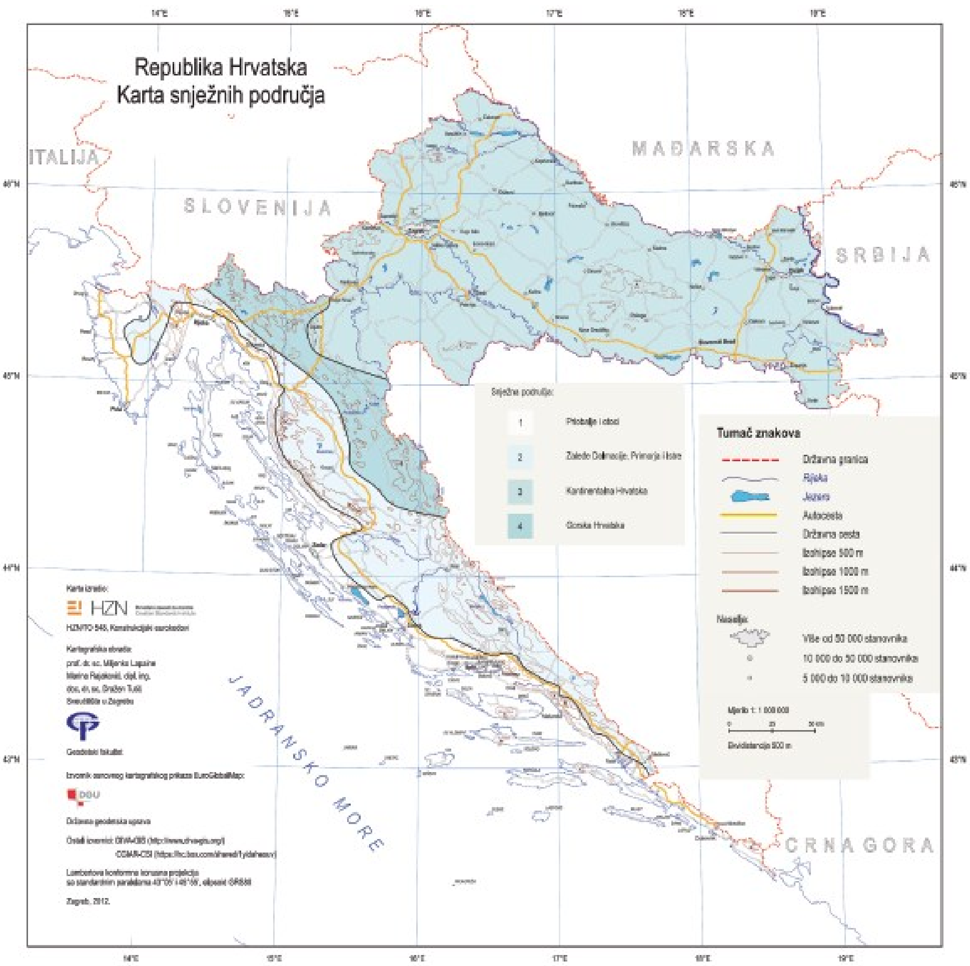
Ukupno stalno djelovanje na krov g_k=0,64 kN/m^2

-Vlastita težina fasade – odabrani su Izoforma Isopar ® elegant sendvič paneli debljine 120 mm, sa unutarnjom i vanjskom čeličnom oblogom od 0,4 mm [3]: g_f=0,11 kN/m^2

Ukupno stalno djelovanje na zidove g_z=0,11 kN/m^2

5.2. Djelovanje snijega

Analiza djelovanja snijega provodi se prema normi HRN EN 1991-1-3:2012 i Nacionalnom dodatku HRN EN 1991-1-3:2012/NA:2016.



Slika 1. Karta snježnih područja Republike Hrvatske [5]

Lokacija građevine: Pula

Snježno područje 1: priobalje i otoci

Nadmorska visina: 0 m

Tablica 1. Opterećenje snijegom za snježna područja i pripadajuće nadmorske visine [5]

Nadmorska visina do [m]	1. područje – priobalje i otoci [kN/m²]	2. područje – zaleđe Dalmacije, Primorja i Istre [kN/m²]	3. područje – kontinentalna Hrvatska [kN/m²]	4. područje – gorska Hrvatska [kN/m²]
100	0,50	0,75	1,00	1,25
200	0,50	0,75	1,25	1,50
300	0,50	0,75	1,50	1,75
400	0,50	1,00	1,75	2,00
500	0,50	1,25	2,00	2,50
600	0,50	1,50	2,25	3,00
700	0,50	2,00	2,50	3,50
800	0,50	2,50	2,75	4,00
900	1,00	3,00	3,00	4,50
1 000	2,00	4,00	3,50	5,00
1 100	3,00	5,00	4,00	5,50
1 200	4,00	6,00	4,50	6,00
1 300	5,00	7,00		7,00
1 400	6,00	8,00		8,00
1 500		9,00		9,00
1 600		10,00		10,00
1 700		11,00		11,00
1 800		12,00		

Karakteristična vrijednost opterećenja snijegom na tlu za 1. područje i nadmorsku visinu do 100 m iznosi: s_k=0,5 kN/m^2

Opterećenje snijegom na krovu: $s = s_k * \mu_i * c_e * c_t$

s_k=0,5 kN/m^2 karakteristična vrijednost opterećenja snijegom na tlu

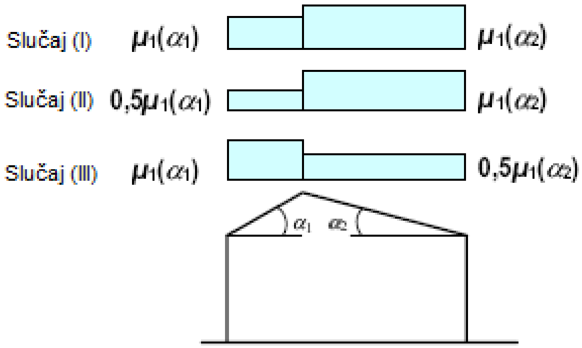
μ_i koeficijent oblika opterećenja snijegom na krovu

c_e=1,0 koeficijent izloženosti

c_t=1,0 toplinski koeficijent

Tablica 2. Koeficijent oblika opterećenja snijegom na krovu ovisno o kutu nagiba krova [6]

Kut nagiba krova α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	$0{,}8(60 - \alpha)/30$	0,0
μ_2	$0{,}8 + 0{,}8\ \alpha/30$	1,6	--



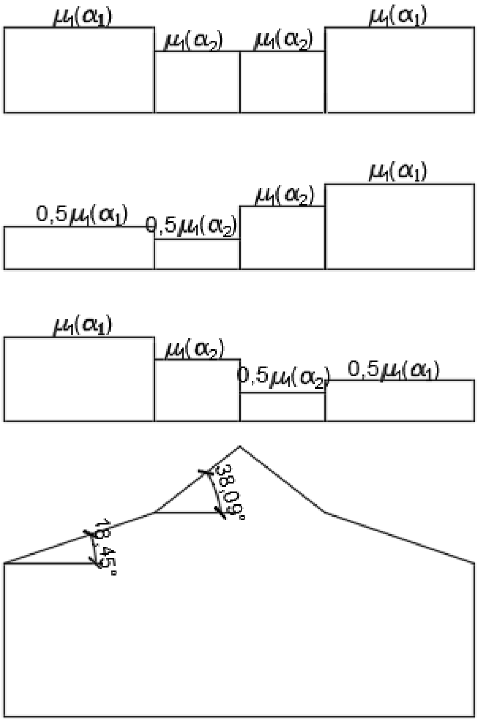
Slika 2. Slučajevi opterećenja snijegom na dvostrešnom krovu prema Eurokodu [6]

Gornji dio krova nagiba α_1=38,09°: 30°<α_1=38,09°≤60°

μ_1(38,09°)=0,8*(60-α)/30=0,8*(60-38,09)/30=0,584

Donji dio krova nagiba α_2=18,45°: 0°<α_2=18,45°≤30°

μ_1(18,45°)=0,8



Slika 3. Slučajevi opterećenja snijegom

$s = s_k * \mu_i * c_e * c_t$

$s_1(\alpha_1) = s_k * \mu_1(\alpha_1) * c_e * c_t = 0{,}5 * 0{,}584 * 1 * 1 = 0{,}29\ kN/m^2$

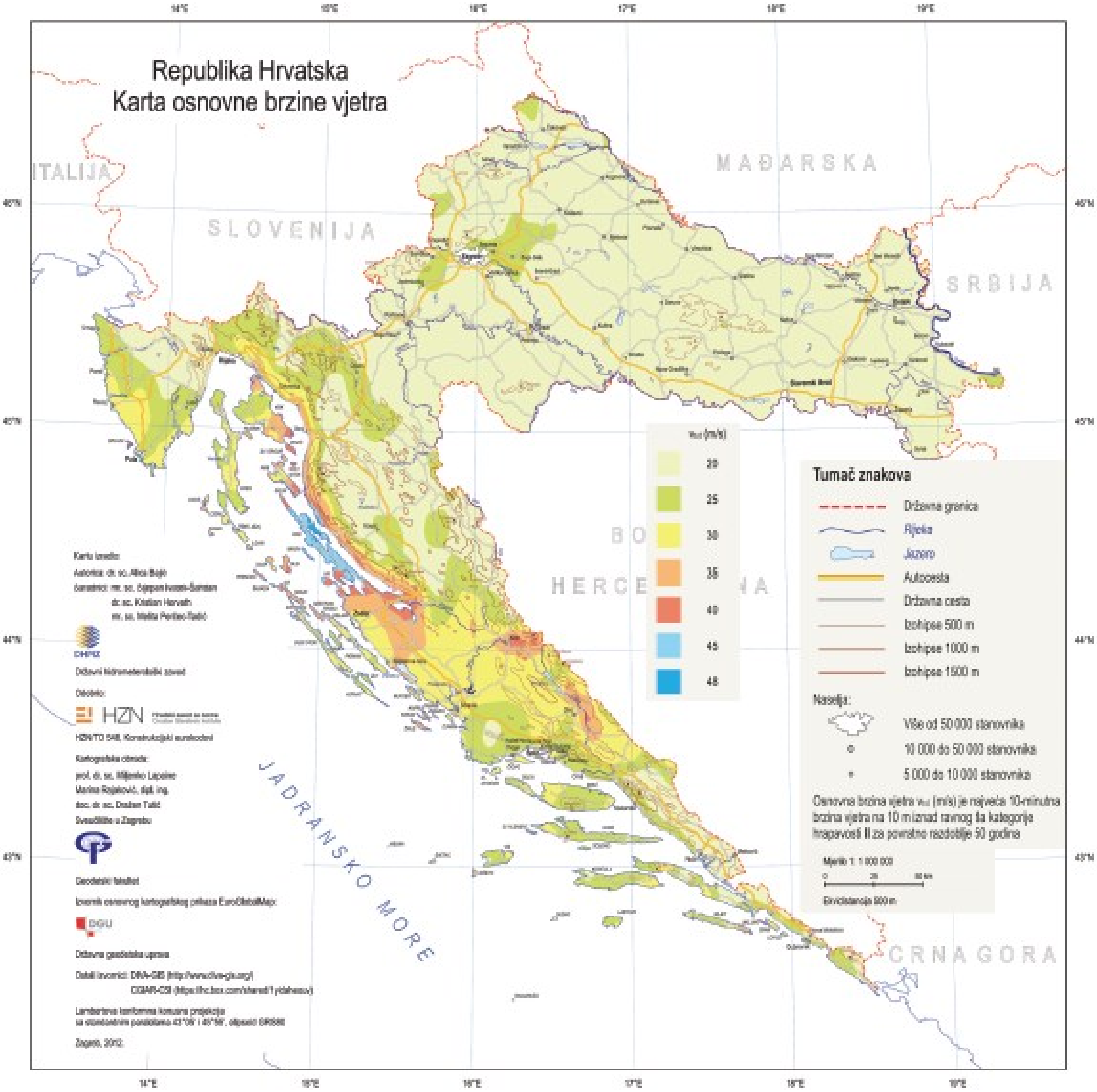
$s_2(\alpha_1) = 0{,}5 * s_k * \mu_1(\alpha_1) * c_e * c_t = 0{,}5 * 0{,}5 * 0{,}584 * 1 * 1 = 0{,}15\ kN/m^2$

$s_1(\alpha_2) = s_k * \mu_1(\alpha_2) * c_e * c_t = 0{,}5 * 0{,}8 * 1 * 1 = 0{,}40\ kN/m^2$

$s_2(\alpha_2) = 0{,}5 * s_k * \mu_1(\alpha_2) * c_e * c_t = 0{,}5 * 0{,}5 * 0{,}8 * 1 * 1 = 0{,}20\ kN/m^2$

5.3. Djelovanje vjetra

Analiza djelovanja snijega provodi se prema normi HRN EN 1991-1-4:2012 i Nacionalnom dodatku HRN EN 1991-1-4:2012/NA:2012.



Slika 4. Karta osnovnih brzina vjetra Republike Hrvatske [5]

Lokacija građevine: Pula

Kategorija terena: 0

Temeljna vrijednost osnovne brzine vjetra: $v_{b,0}$=30 m/s

Osnovna brzina vjetra: $v_b = v_{b,0} * c_{dir} * c_{season}$

c_{dir}=1,0	faktor smjera vjetra
c_{season}=1,0	faktor godišnjeg doba
ρ=1,25 kg/m^3	gustoća zraka

$$v_b = v_{b,0} * c_{dir} * c_{season} = 30 * 1 * 1 = 30\text{m/s}$$

Osnovni pritisak vjetra brzine v_b: $q_b = \frac{1}{2} * \rho * V_b^2$

ρ=1,25 kg/m^3 gustoća zraka

v_b=30 m/s osnovna brzina vjetra

$$q_b = \frac{1}{2} * \rho * V_b^2 = \frac{1}{2} * 1{,}25 * 30^2 = 562{,}5 \ \mathrm{N/m^2} = 0{,}56 \ \mathrm{kN/m^2}$$

Srednja brzina vjetra: $v_m(z) = c_r(z) * c_0(z) * v_b$

$c_r(z)$ koeficijent hrapavosti terena

$c_0(z)$=1,0 faktor orografije

v_b=30 m/s osnovna brzina vjetra

Koeficijent hrapavosti terena: $c_{r(z)} = k_r * \ln\left(\frac{z}{z_0}\right)$

k_r koeficijent terena ovisan o duljini hrapavosti z_0

z=12,67 m visina konstrukcije u sljemenu

z_0 duljina hrapavosti

Koeficijent terena ovisan o duljini hrapavosti z_0: $k_r = 0{,}19 * \left(\frac{z0}{z0{,}II}\right)^{0{,}07}$

z_0=0,003 m duljina hrapavosti (kategorija terena 0.)

z0,II=0,05 m duljina hrapavosti za II. kategoriju terena

Tablica 3. Kategorije i parametri terena [5]

	Kategorija terena	z_0 m	z_{min} m
0	More ili priobalna područja izložena otvorenom moru	0,003	1
I	Jezera ili ravna i horizontalno položena područja sa zanemarivom vegetacijom i bez prepreka	0,01	1
II	Područja s niskom vegetacijom, npr. travom, i izoliranim preprekama (drveće, zgrade) s razmakom najmanje 20 visina prepreke	0,05	2
III	Područja sa stalnim pokrovom od vegetacije ili zgrade ili područja s izoliranim preprekama s razmakom najviše 20 visina prepreke (npr. sela, predgrađa, stalna šuma)	0,3	5
IV	Područja s najmanje 15 % površine pokrivene zgradama čija prosječna visina premašuje 15 m	1,0	10
NAPOMENA: Kategorije terena prikazane su na slikama u točki A.1.			

$$k_r = 0{,}19 * \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07} = 0{,}19 * \left(\frac{0{,}003}{0{,}05}\right)^{0{,}07} = 0{,}156$$

Koeficijent hrapavosti terena:

$$c_{r(z)} = k_r * \ln\left(\frac{z}{z_0}\right) = 0{,}156 * \ln\left(\frac{12{,}67}{0{,}003}\right) = 1{,}3$$

Srednja brzina vjetra na visini z:

$$v_m(z) = c_r(z) * c_0(z) * v_b = 1{,}3 * 1{,}0 * 30 = 39 \text{ m/s}$$

Intenzitet turbulencije: $I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_0(z)*\ln\left(\frac{z}{z_0}\right)}$ za $z_{min} < z < z_{max}$

$I_v(z) = I_v(z_{min})$ za $z < z_{min}$

z_{min}=1 m minimalna visina (za 0. kategoriju terena)

z_{max}=200 m maksimalna visina

Uvjet: z_{min}=1 m < z=12,67m < z_{max}=200m

k_I=1,0 faktor turbulencije

$c_0(z)$=1,0 faktor orografije

z=12,67 m visina konstrukcije u sljemenu

z_0=0,003 m duljina hrapavosti (kategorija terena 0.)

$$I_v(z) = \frac{k_I}{c_0(z) * \ln\left(\frac{z}{z_0}\right)} = \frac{1}{1 * \ln\left(\frac{12{,}67}{0{,}003}\right)} = 0{,}12$$

Vršni pritisak brzine vjetra na visini z iznad terena: $q_p(z) = [1 + 7 * I_v(z)] * \frac{1}{2} * \rho * v_m^2(z)$

$I_v(z)$=0,12 intenzitet turbulencije

ρ=1,25 kg/m^3 gustoća zraka

$v_m(z)$=39 m/s srednja brzina vjetra na visini z

$$q_p(z) = [1 + 7 * I_v(z)] * \frac{1}{2} * \rho * v_m^2(z) = [1 + 7 * 0{,}12] * 0{,}5 * 1{,}25 * \frac{39^2}{1000}$$
$$= 1{,}75\ kN/m2$$

Koeficijent izloženosti $c_e(z)$ na visini z iznad terena: $C_e(z) = \frac{q_p(z)}{q_b}$

$q_p(z)$=1,75 kN/m^2 vršni pritisak brzine vjetra na visini z iznad terena

q_b=0,56 kN/m^2 osnovni pritisak vjetra brzine v_b

$$C_e(z) = \frac{q_p(z)}{q_b} = \frac{1{,}75}{0{,}56} = 3{,}13$$

5.3.1. Vanjski pritisak vjetra

Vanjski pritisak vjetra ovisi o vanjskim dimenzijama objekta a dobiva se prema sljedećem izrazu:

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

<u>Poprečno djelovanje vjetra</u>

Referentna visina: z_e=12,67m

Duljina zgrade: b=80,18 m

Širina zgrade: d=22m

Visina zgrade: h=12,67 m

Parametar e: e=min(b;2h)=min(80,18 m; 25,34 m)= 25,34 m

e=25,34 m>d=22 m (samo zone A i B)

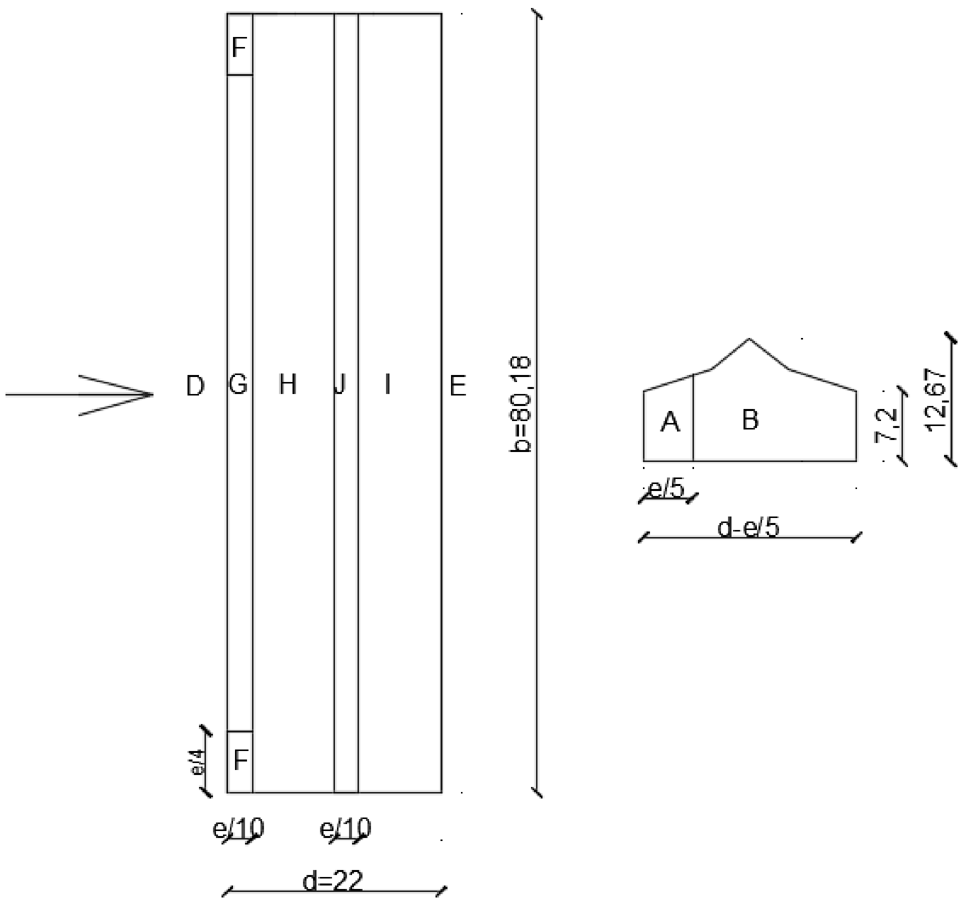
e/5=25,34/5=5,07 m

4e/5=4*25,34/5=20,27 m

e/4=25,34/4=6,34 m

e/10=25,34/10=2,53 m

Omjer h/d: h/d=12,67/22=0,58



Slika 5. Poprečno djelovanje vjetra

Djelovanje vjetra na vertikalne površine

Površine zona:

A_A=40,77 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_B=165,19 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_D=577,30 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_E=577,30 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

Tablica 4. Koeficijent vanjskog pritiska vjetra na zidove za poprečni smjer [6]

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

h/d=0,58 -1,2 -0,8 +0,744 -0,388

Vanjski tlak na vertikalne površine: $w_e = q_p(z_e) * c_{pe}$

$w_e^A = 1{,}75 * (-1{,}2) = -2{,}1 \text{ kN/m}^2$

$w_e^B = 1{,}75 * (-0{,}8) = -1{,}4 \text{ kN/m}^2$

$w_e^D = 1{,}75 * (0{,}744) = 1{,}30 \text{ kN/m}^2$

$w_e^E = 1{,}75 * (-0{,}388) = -0{,}68 \text{ kN/m}^2$

Djelovanje vjetra na krov

Površine zona:

A_F=16,05 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_G=191,34 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_H=746,53 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_I=181,54 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_J=746,53 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

Tablica 5. Koeficijenti vanjskog pritiska vjetra na dvostrešni krov za poprečni smjer[6]

Pitch Angle α	Zone for wind direction $\theta = 0°$									
	F		G		H		I		J	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-0,6		-0,6		-0,8		-0,7		-1,0	-1,5
-30°	-1,1	-2,0	-0,8	-1,5	-0,8		-0,6		-0,8	-1,4
-15°	-2,5	-2,8	-1,3	-2,0	-0,9	-1,2	-0,5		-0,7	-1,2
-5°	-2,3	-2,5	-1,2	-2,0	-0,8	-1,2	+0,2		+0,2	
							-0,6		-0,6	
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-0,6		+0,2	
	+0,0		+0,0		+0,0				-0,6	
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-0,4		-1,0	-1,5
	+0,2		+0,2		+0,2		+0,0		+0,0	+0,0
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-0,4		-0,5	
	+0,7		+0,7		+0,4		+0,0		+0,0	
45°	-0,0		-0,0		-0,0		-0,2		-0,3	
	+0,7		+0,7		+0,6		+0,0		+0,0	
60°	+0,7		+0,7		+0,7		-0,2		-0,3	
75°	+0,8		+0,8		+0,8		-0,2		-0,3	

Vanjski tlak na vertikalne površine: $w_e = q_p(z_e) * c_{pe}$

U zonama H i I dio krova ima nagib 38,09°, a dio 18,45°, u zonama F i G krov ima nagib 18,45°, a u zoni J 38,09°.

$w_e^F = 1{,}75 * (-0{,}808) = -1{,}41\ kN/m^2$ (18,45°)

$w_e^F = 1{,}75 * (0{,}315) = 0{,}55\ kN/m^2$ (18,45°)

$w_e^G = 1{,}75 * (-0{,}731) = -1{,}28\ kN/m^2$ (18,45°)

$w_e^G = 1{,}75 * (0{,}315) = 0{,}55\ kN/m^2$ (18,45°)

$w_e^H = 1{,}75 * (-0{,}277) = -0{,}49\ kN/m^2$ (18,45°)

$w_e^H = 1{,}75 * (0{,}246) = 0{,}43\ kN/m^2$ (18,45°)

$w_e^H = 1{,}75 * (-0{,}092) = -0{,}16\ kN/m^2$ (38,09°)

$w_e^H = 1{,}75 * (0{,}508) = 0{,}89 kN/m^2$ (38,09°)

$w_e^I = 1{,}75 * (-0{,}4) = -0{,}70\ kN/m^2$ (18,45°)

$w_e^I = 1{,}75 * (0{,}0) = 0{,}00\ kN/m^2$ (18,45°)

$w_e^I = 1{,}75 * (-0{,}292) = -0{,}51\ kN/m^2$ (38,09°)

$w_e^I = 1{,}75 * (0{,}0) = 0{,}00\ kN/m^2$ (38,09°)

$w_e^J = 1{,}75 * (-0{,}392) = -0{,}69\ kN/m^2$ (38,09°)

$w_e^J = 1{,}75 * (0{,}0) = 0{,}00 kN/m^2$ (38,09°)

Uzdužno djelovanje vjetra

Referentna visina: z_e=7,20 m

Duljina zgrade: b=22 m

Širina zgrade: d=80,18 m

Visina zgrade: h=7,20 m

Parametar e: e=min(b;2h)=min(22 m; 14,40 m)= 14,40 m

e=14,40 m<d=80,18 m (zone A, B i C)

e/5=14,40/5=2,88 m

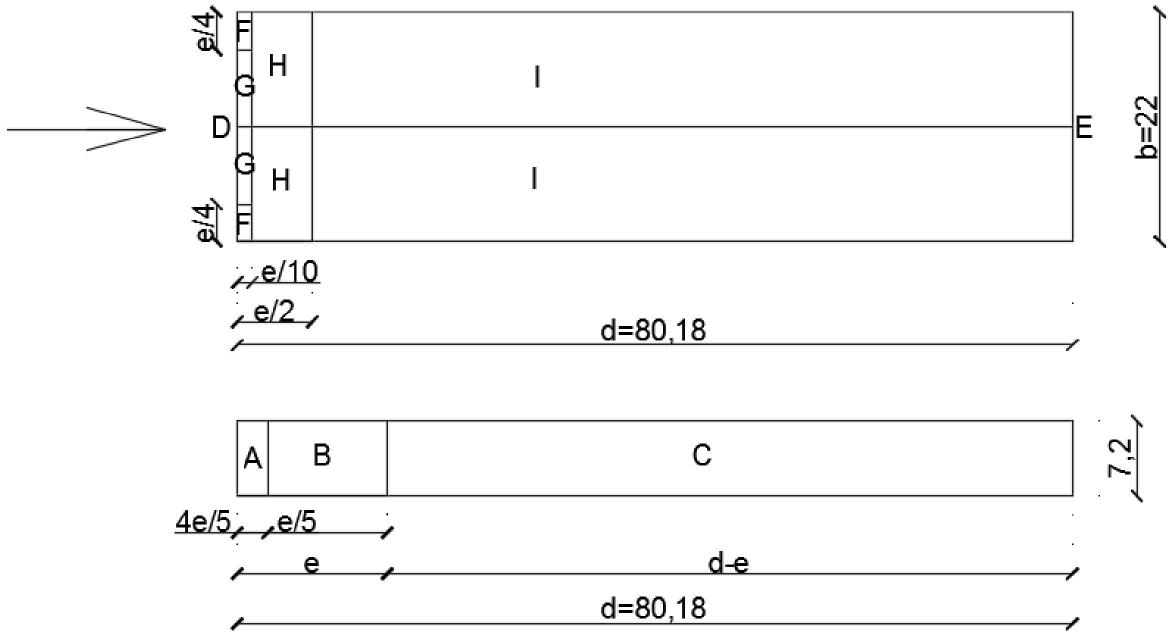
4e/5=4*14,40/5=11,52 m

e/2=14,40/2= 7,20 m

e/4=14,4/4=3,60 m

e/10=14,40/10=1,44 m

Omjer h/d: h/d=7,20/80,18=0,09



Slika 6. Uzdužno djelovanje vjetra

Djelovanje vjetra na vertikalne površine

Površine zona:

A_A=20,74 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_B=82,94 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_c=473,62 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_D=205,97 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

A_E=205,97 m^2 >10m^2 → $c_{pe}=c_{pe,10}$

Tablica 6. Koeficijenti vanjskog pritiska vjetra na zidove za uzdužni smjer[6]

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

h/d=0,09<0,25

Vanjski tlak na vertikalne površine: $w_e = q_p(z_e) * c_{pe}$

$w_e^A = 1{,}75 * (-1{,}2) = -2{,}1\ kN/m^2$

$w_e^B = 1{,}75 * (-0{,}8) = -1{,}4\ kN/m^2$

$w_e^C = 1{,}75 * (-0{,}5) = -0{,}88\ kN/m^2$

$w_e^D = 1{,}75 * (0{,}7) = 1{,}23kN/m^2$

$w_e^E = 1{,}75 * (-0{,}3) = -0{,}53\ kN/m^2$

<u>Djelovanje vjetra na krov</u>

Površine zona:

A_F=5,18 m^2 1m^2< 5,18 m^2<10m^2 → c_{pe}

A_G=10,66 m^2 >10m^2 → c_{pe}=$c_{pe,10}$

A_H=63,36 m^2 >10m^2 → c_{pe}=$c_{pe,10}$

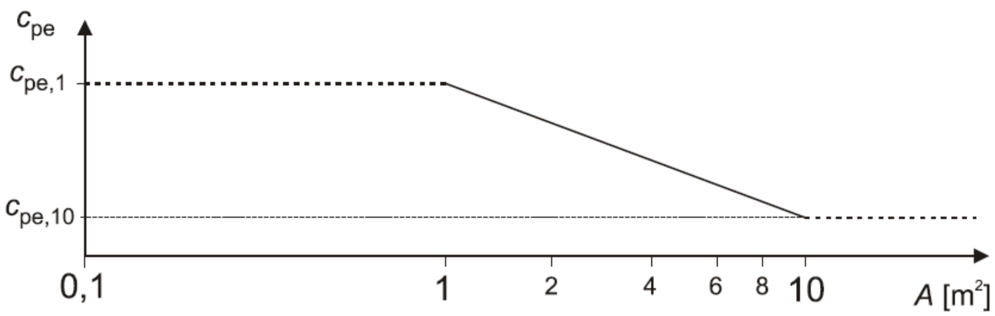
A_I=802,78 m^2 >10m^2 → c_{pe}=$c_{pe,10}$

Tablica 7. Koeficijenti vanjskog pritiska vjetra na dvostrešni krov za uzdužni smjer[6]

Pitch angle α	Zone for wind direction θ = 90°							
	F		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-1,4	-2,0	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-30°	-1,5	-2,1	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-15°	-1,9	-2,5	-1,2	-2,0	-0,8	-1,2	-0,8	-1,2
-5°	-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	-0,6	-1,2
5°	-1,6	-2,2	-1,3	-2,0	-0,7	-1,2	-0,6	
15°	-1,3	-2,0	-1,3	-2,0	-0,6	-1,2	-0,5	
30°	-1,1	-1,5	-1,4	-2,0	-0,8	-1,2	-0,5	
45°	-1,1	-1,5	-1,4	-2,0	-0,9	-1,2	-0,5	
60°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	
75°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	

Za površine $1m^2<A<10m^2$, koeficijent vanjskog pritiska vjetra može se odrediti prema grafu prikazanom na slici 7 ili prema formuli:

$$c_{pe} = c_{pe,1} - \left(c_{pe,1} - c_{pe,10}\right) \log_{10} A$$



Slika 7. Određivanje koeficijenta vanjskog pritiska vjetra za površine $1m^2<A<10m^2$ [6]

Zona F, nagib krova 18,45°

$c_{pe,1}$=-1,885

$c_{pe,10}$=-1,254

$c_{pe,5,18}$=-1,885-(-1,885+1,254)*$\log_{10}$5,18=-1,434

Vanjski tlak na vertikalne površine: $w_e = q_p(z_e) * c_{pe}$

U zonama G, H i I dio krova ima nagib 38,09°, a dio 18,45°, u zoni F krov ima nagib 18,45°.

$w_e^F = 1{,}75 * (-1{,}434) = -2{,}51\ kN/m^2$ (18,45°)

$w_e^G = 1{,}75 * (-1{,}323) = -2{,}32\ kN/m^2$ (18,45°)

$w_e^G = 1{,}75 * (-1{,}40) = -2{,}45\ kN/m^2$ (38,09°)

$w_e^H = 1{,}75 * (-0{,}646) = -1{,}13\ kN/m^2$ (18,45°)

$w_e^H = 1{,}75 * (-0{,}854) = -1{,}49\ kN/m^2$ (38,09°)

$w_e^I = 1{,}75 * (-0{,}50) = -0{,}88\ kN/m^2$ (18,45°)

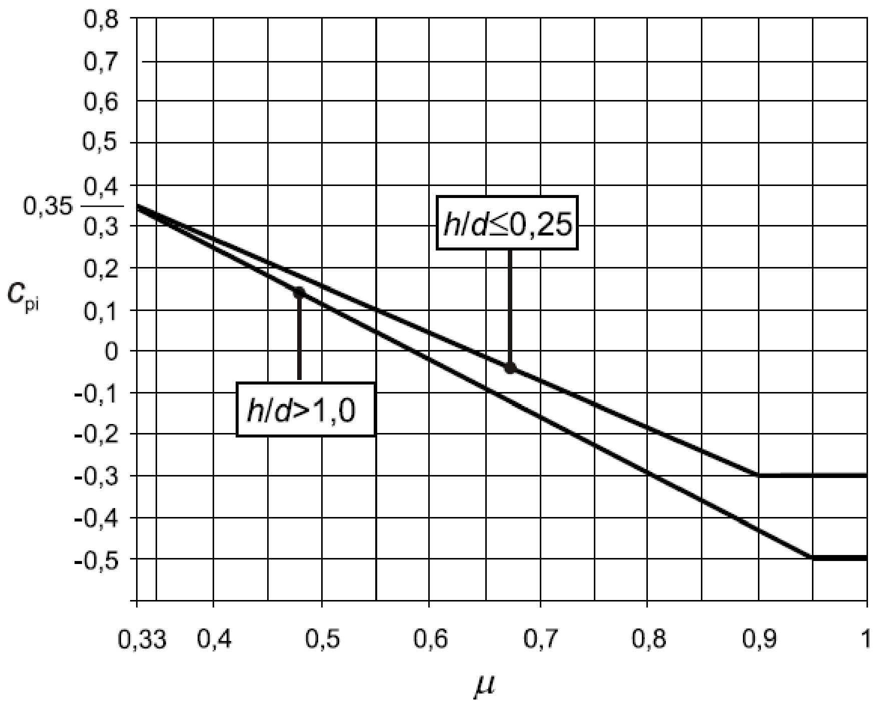
$w_e^I = 1{,}75 * (-0{,}50) = -0{,}88\ kN/m^2$ (38,09°)

5.3.2. Unutarnji pritisak vjetra

Unutarnji pritisak vjetra ovisi o veličini i položaju otvora, a dobiva se prema sljedećem izrazu:

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

Na sve četiri fasade površina otvora prelazi 30% površine fasade. Kako zgrada nema dominantnu fasadu, koeficijent unutarnjeg pritiska c_{pi} dobiva se preko grafa prikazanog na slici 8.



Slika 8. Graf za određivanje koeficijenta unutarnjeg pritiska vjetra [6]

Parametar μ može se izračunati prema sljedećem izrazu:

$$\mu = \frac{\sum površina\ otvora\ gdje\ je\ c_{pe}\ negativan\ ili - 0{,}0}{\sum površina\ svih\ otvora}$$

<u>Poprečno djelovanje vjetra</u>

Slučaj 1:

Površina svih otvora:

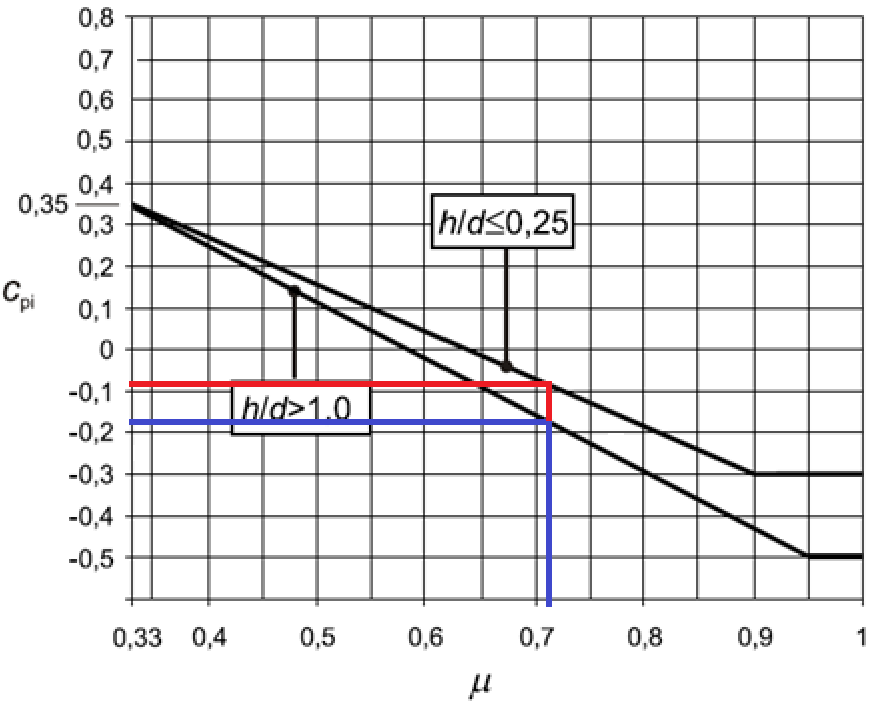
A=152,45 m^2 + 91,6 m^2 + 125,83 m^2 + 152,86 m^2 = 522,74 m^2

Površina otvora gdje je c_{pe} negativan ili -0,0:

A_1=91,6 m^2 + 125,83 m^2 + 152,86 m^2 =370,29 m^2

$$\mu = \frac{370{,}29\ m^2}{522{,}74\ m^2} = 0{,}71$$

$$\frac{h}{d} = \frac{12{,}67\ m}{22\ m} = 0{,}58$$



Slika 9. očitane vrijednosti za poprečni smjer , slučaj 1 [6]

h/d≤0,25 →c_{pi}=-0,09

h/d >1,0 →c_{pi}=-0,18

Linearnom interpolacijom, za h/d=0,58 dobije se c_{pi}=-0,13

Slučaj 2:

Površina svih otvora:

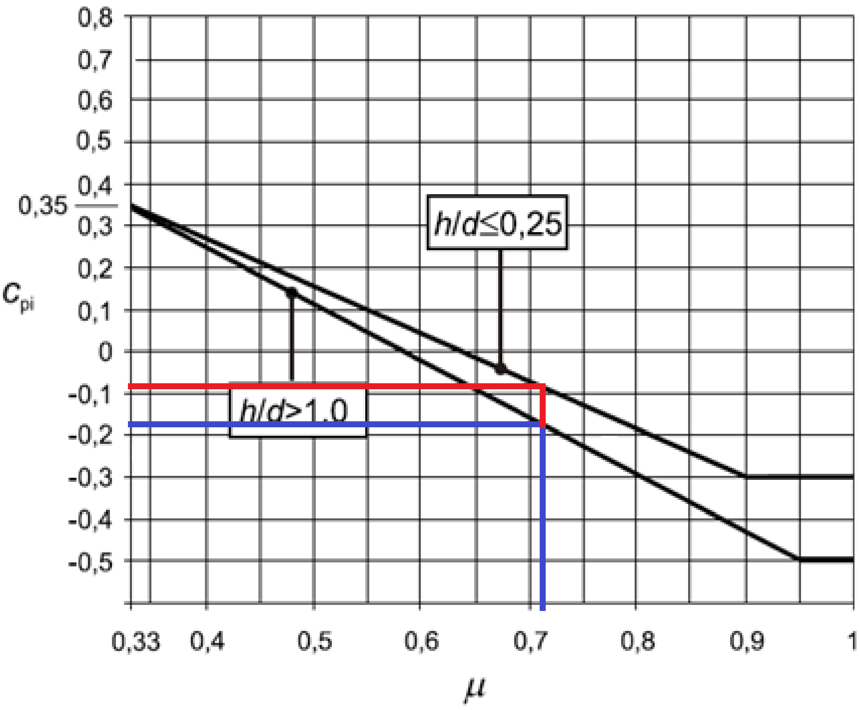
A=152,45 m^2 + 91,6 m^2 + 125,83 m^2 + 152,86 m^2 = 522,74 m^2

Površina otvora gdje je c_{pe} negativan ili -0,0:

A_1=152,45 m^2 + 91,6 m^2 + 125,83 m^2 =369,88 m^2

$$\mu = \frac{369{,}88\ m^2}{522{,}74\ m^2} = 0{,}71$$

$$\frac{h}{d} = \frac{12{,}67\ m}{22\ m} = 0{,}58$$



Slika 10. Očitane vrijednosti za poprečni smjer , slučaj 2 [6]

h/d≤0,25 →c_{pi}=-0,09

h/d >1,0 →c_{pi}=-0,18

Linearnom interpolacijom, za h/d=0,58 dobije se c_{pi}=-0,13

Za poprečno djelovanje mjerodavan je c_{pi}=-0,13

$$w_i = q_p(z_e) \cdot c_{pi} = 1{,}75 \cdot (-0{,}13) = -0{,}23\ kN/m^2$$

<u>Uzdužno djelovanje vjetra</u>

Slučaj 1:

Površina svih otvora:

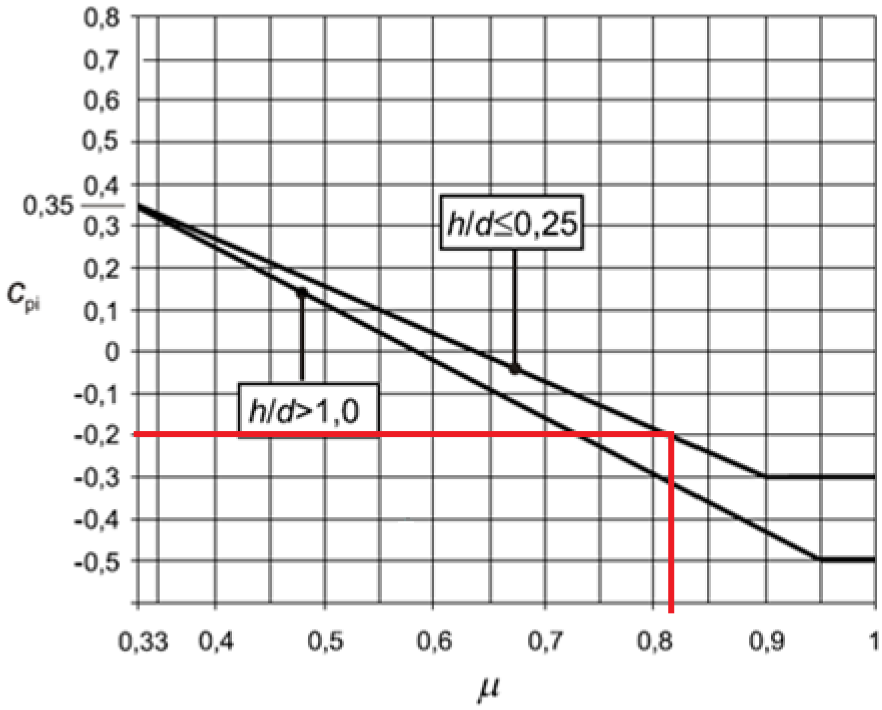
A=152,45 m^2 + 91,6 m^2 + 125,83 m^2 + 152,86 m^2 = 522,74 m^2

Površina otvora gdje je c_{pe} negativan ili -0,0:

A_1=152,45 m^2 + 125,83 m^2 + 152,86 m^2 = 431,14m^2

$$\mu = \frac{431,14\ m^2}{522,74\ m^2} = 0,82$$

$$\frac{h}{d} = \frac{7,2\ m}{80,18\ m} = 0,09$$



Slika 11. Učitane vrijednosti za uzdužni smjer , slučaj 1[6]

h/d=0,09<0,25→c_{pi}=-0,2

Slučaj 2:

Površina svih otvora:

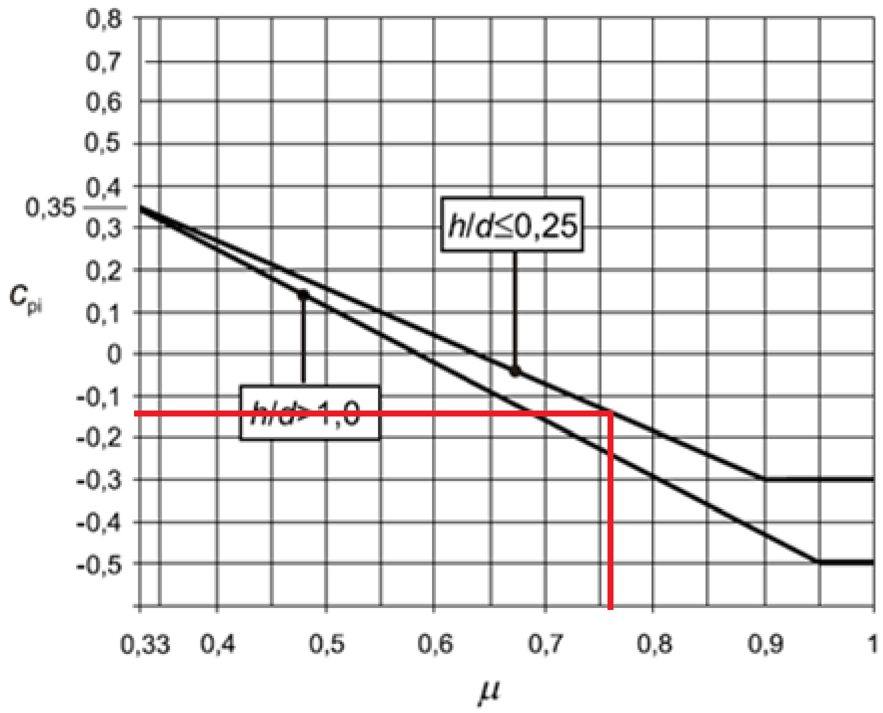
A=152,45 m^2 + 91,6 m^2 + 125,83 m^2 + 152,86 m^2 = 522,74 m^2

Površina otvora gdje je c_{pe} negativan ili -0,0:

A_1=152,45 m^2 + 91,6 m^2 + 152,86 m^2 = 396,91 m^2

$$\mu = \frac{396{,}91\ m^2}{522{,}74\ m^2} = 0{,}76$$

$$\frac{h}{d} = \frac{7{,}2\ m}{80{,}18\ m} = 0{,}09$$



Slika 12. Očitane vrijednosti za uzdužni smjer, slučaj 2 [6]

h/d=0,09<0,25→c_{pi}=-0,14

Za uzdužno djelovanje mjerodavan je c_{pi}=-0,20

$$w_i = q_p(z_e) \cdot c_{pi} = 1{,}75 \cdot (-0{,}20) = -0{,}35\ kN/m^2$$

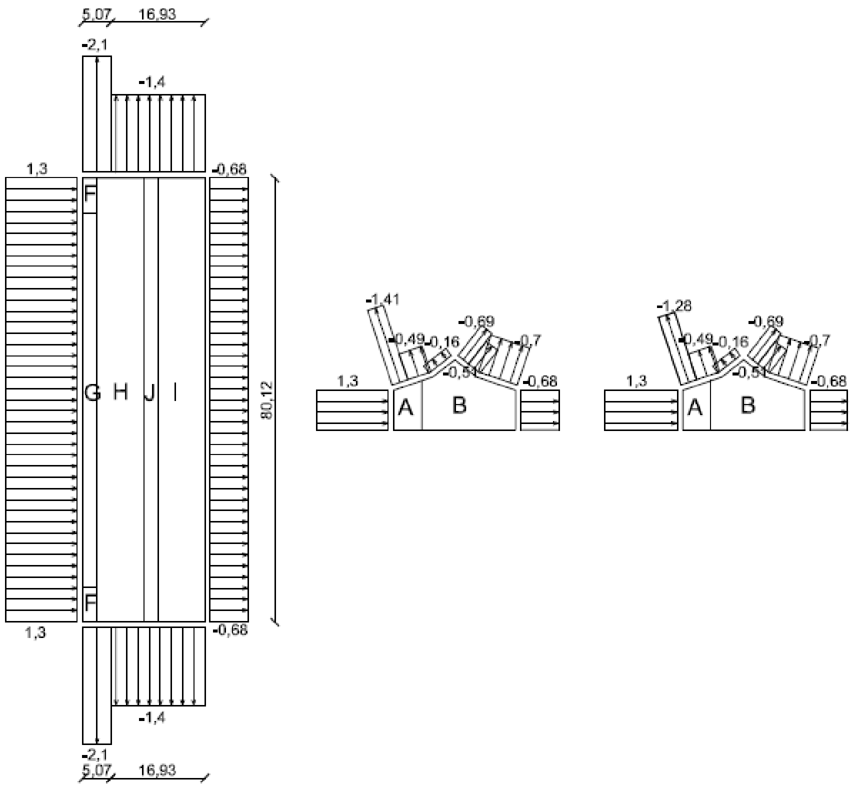
5.3.3. Rezultantni pritisak vjetra

Poprečni smjer

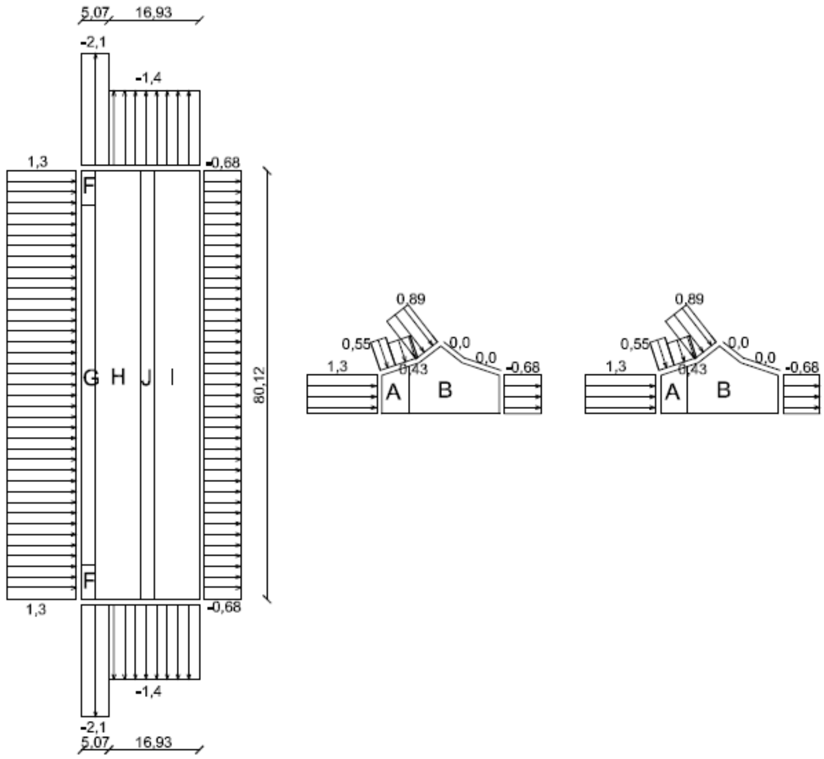
Zatvorena vrata→c_{pi}=0

Rezultantni pritisak vjetra na zidove					
Zona		A	B	D	E
we	kN/m²	-2,10	-1,40	1,30	-0,68
wi	kN/m²	0,00	0,00	0,00	0,00
we-wi	***kN/m²***	***-2,10***	***-1,40***	***1,30***	***-0,68***

Rezultantni pritisak vjetra na krov								
Zona		F (18,45°)	G (18,45°)	H (18,45°)	H (38,09 °)	I (18,45°)	I (38,09°)	J (38,09°)
we	kN/m²	-1,41	-1,28	-0,49	-0,16	-0,70	-0,51	-0,69
		0,55	0,55	0,43	0,89	0,00	0,00	0,00
wi	kN/m²	0,00	0,00	0,00	0,00	0,00	0,00	0,00
we-wi	***kN/m²***	***-1,41***	***-1,28***	***-0,49***	***-0,16***	***-0,70***	***-0,51***	***-0,69***
		0,55	***0,55***	***0,43***	***0,89***	***0,00***	***0,00***	***0,00***



Slika 13. Opterećenje vjetrom u poprečnom smjeru, slučaj sa zatvorenim vratima i negativnim vanjskim tlakom; djelovanje na zidove, krov kroz zonu F i krov kroz zonu G

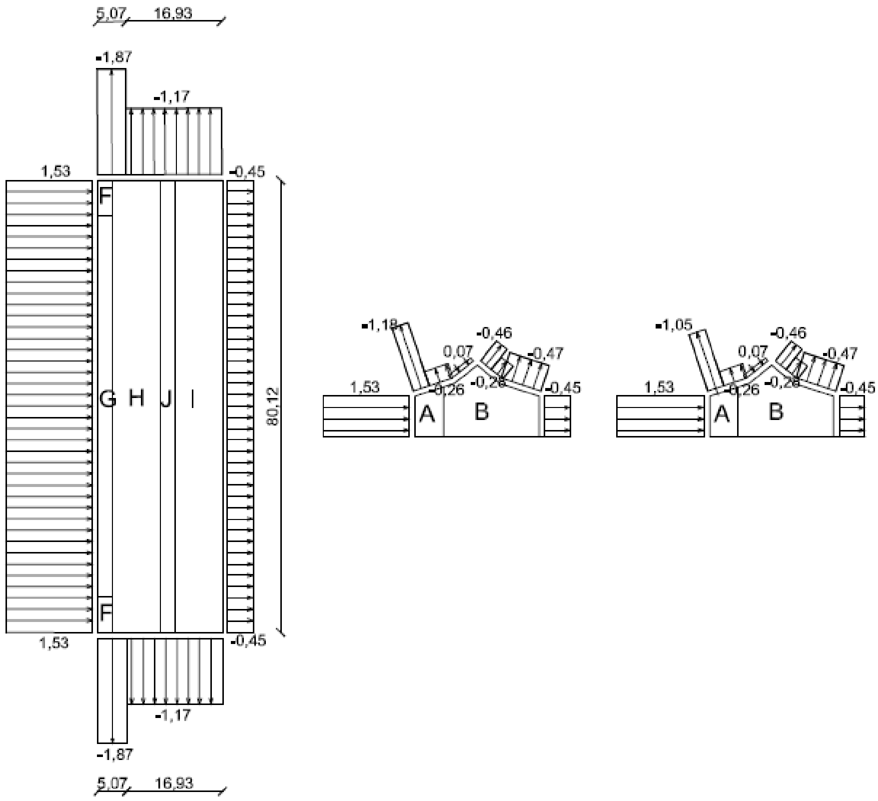


Slika 14. Opterećenje vjetrom u poprečnom smjeru, slučaj sa zatvorenim vratima i pozitivnim vanjskim tlakom; djelovanje na zidove, krov kroz zonu F i krov kroz zonu G

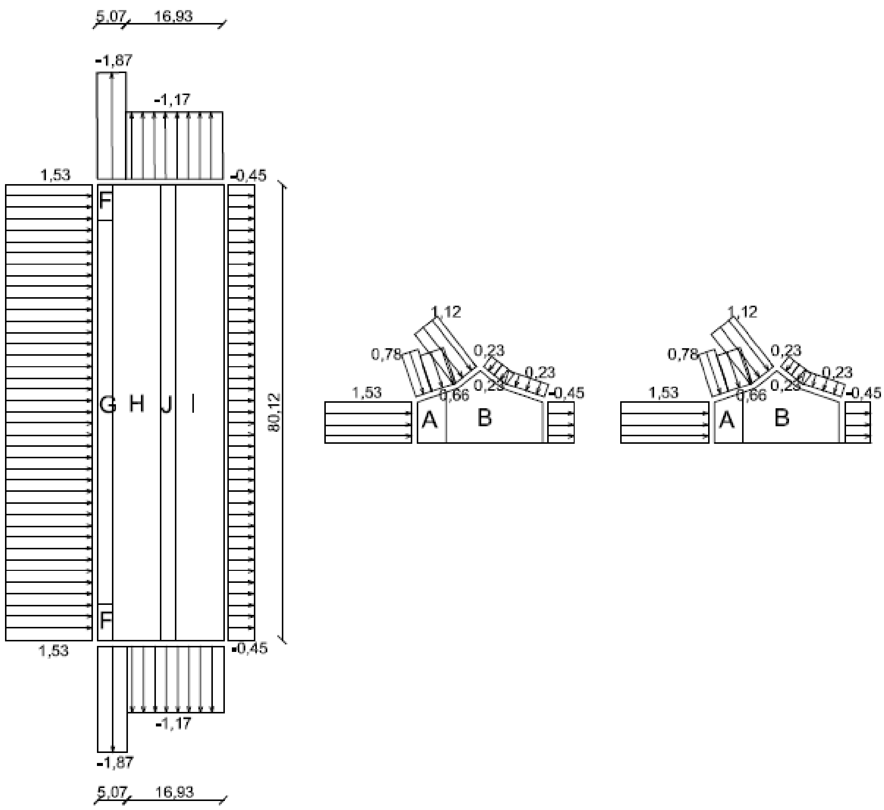
Otvorena vrata

Rezultantni pritisak vjetra na zidove					
Zona		A	B	D	E
we	kN/m²	-2,10	-1,40	1,30	-0,68
wi	kN/m²	-0,23	-0,23	-0,23	-0,23
we-wi	***kN/m²***	***-1,87***	***-1,17***	***1,53***	***-0,45***

Rezultantni pritisak vjetra na krov								
Zona		F (18,45°)	G (18,45°)	H (18,45°)	H (38,09 °)	I (18,45°)	I (38,09°)	J (38,09°)
we	kN/m²	-1,41	-1,28	-0,49	-0,16	-0,70	-0,51	-0,69
		0,55	0,55	0,43	0,89	0,00	0,00	0,00
wi	kN/m²	-0,23	-0,23	-0,23	-0,23	-0,23	-0,23	-0,23
we-wi	***kN/m²***	***-1,18***	***-1,05***	***-0,26***	***0,07***	***-0,47***	***-0,28***	***-0,46***
		0,78	***0,78***	***0,66***	***1,12***	***0,23***	***0,23***	***0,23***



Slika 15. Opterećenje vjetrom u poprečnom smjeru, slučaj sa otvorenim vratima i negativnim vanjskim tlakom; djelovanje na zidove, krov kroz zonu F i krov kroz zonu G



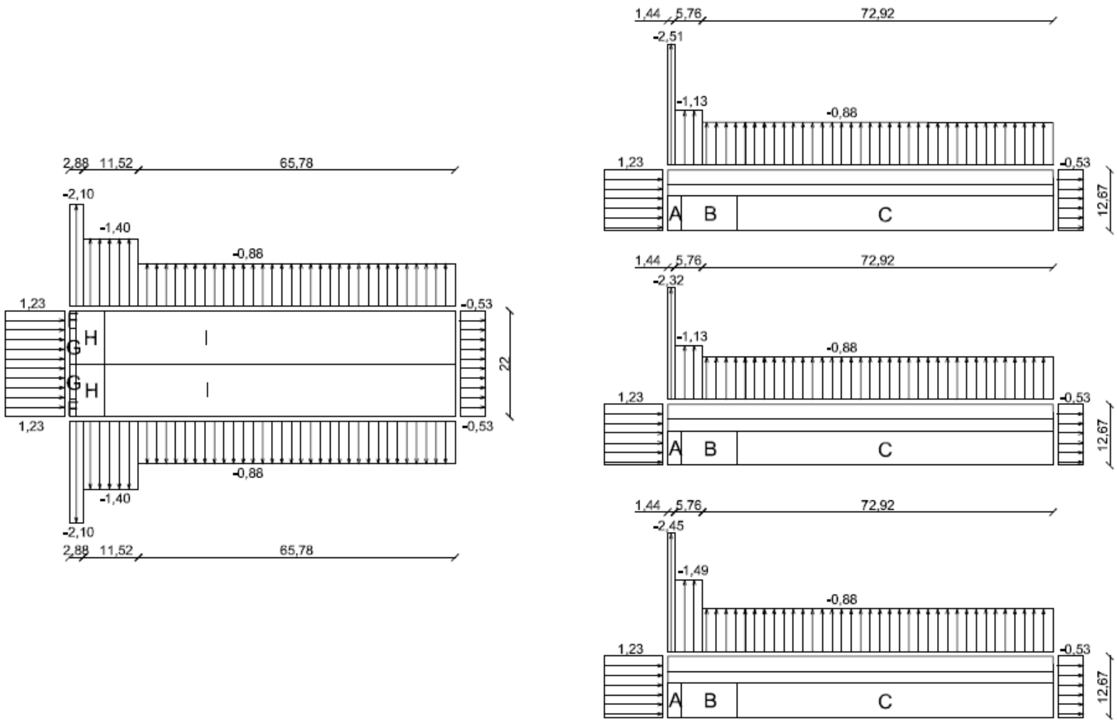
Slika 16. Opterećenje vjetrom u poprečnom smjeru, slučaj sa otvorenim vratima i pozitivnim vanjskim tlakom; djelovanje na zidove, krov kroz zonu F i krov kroz zonu G

Uzdužni smjer

Zatvorena vrata→c_{pi}=0

Rezultantni pritisak vjetra na zidove						
Zona		A	B	C	D	E
we	kN/m²	-2,10	-1,40	-0,88	1,23	-0,53
wi	kN/m²	0,00	0,00	0,00	0,00	0,00
we-wi	***kN/m²***	***-2,10***	***-1,40***	***-0,88***	***1,23***	***-0,53***

Rezultantni pritisak vjetra na krov								
Zona		F (18,45°)	G (18,45°)	G (38,09°)	H (18,45°)	H (38,09°)	I (18,45°)	I (38,09°)
we	kN/m²	-2,51	-2,32	-2,45	-1,13	-1,49	-0,88	-0,88
wi	kN/m²	0,00	0,00	0,00	0,00	0,00	0,00	0,00
we-wi	***kN/m²***	***-2,51***	***-2,32***	***-2,45***	***-1,13***	***-1,49***	***-0,88***	***-0,88***

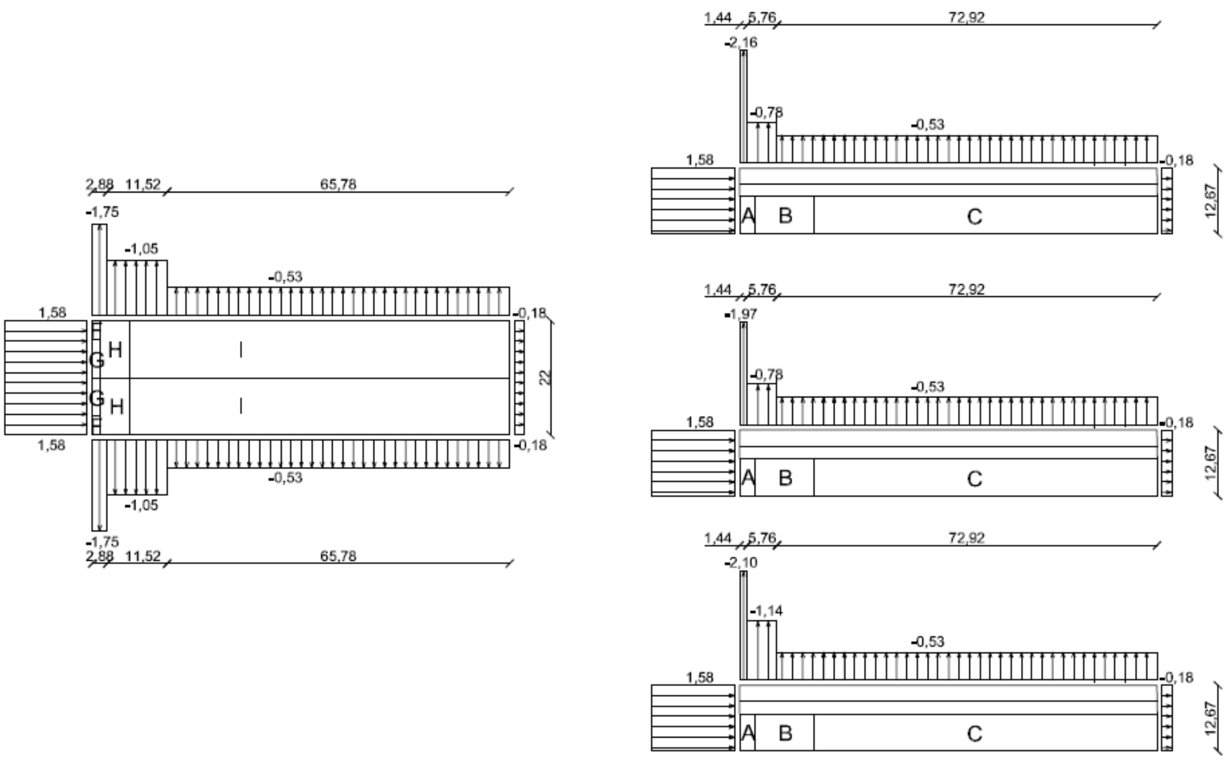


Slika 17. Opterećenje vjetrom u uzdužnom smjeru, slučaj sa zatvorenim vratima; djelovanje na zidove, donji dio krova kroz zonu F, donji dio krova kroz zonu G i gornji dio krova korz zonu G

Otvorena vrata

Rezultantni pritisak vjetra na zidove						
Zona		A	B	C	D	E
we	kN/m²	-2,10	-1,40	-0,88	1,23	-0,53
wi	kN/m²	-0,35	-0,35	-0,35	-0,35	-0,35
we-wi	***kN/m²***	***-1,75***	***-1,05***	***-0,53***	***1,58***	***-0,18***

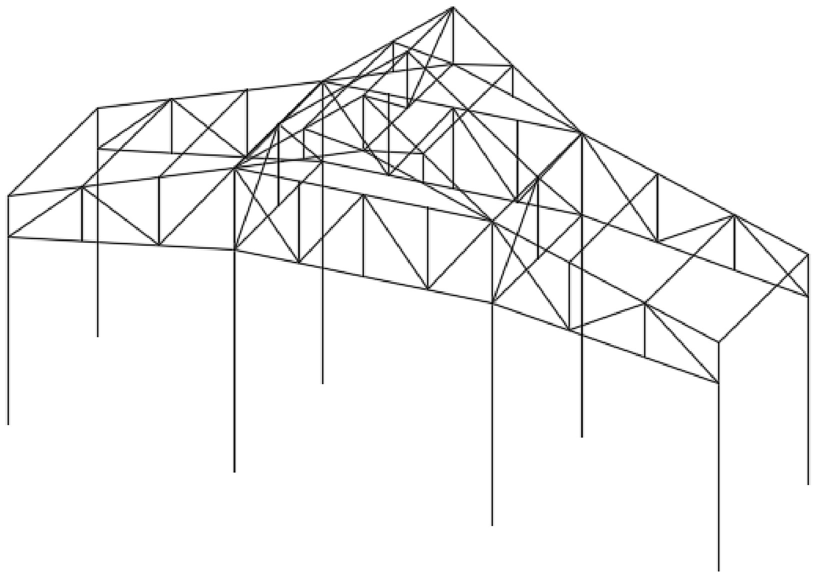
Rezultantni pritisak vjetra na krov								
Zona		F (18,45°)	G (18,45°)	G (38,09°)	H (18,45°)	H (38,09°)	I (18,45°)	I (38,09°)
we	kN/m²	-2,51	-2,32	-2,45	-1,13	-1,49	-0,88	-0,88
wi	kN/m²	-0,35	-0,35	-0,35	-0,35	-0,35	-0,35	-0,35
we-wi	***kN/m²***	***-2,16***	***-1,97***	***-2,10***	***-0,78***	***-1,14***	***-0,53***	***-0,53***



Slika 18. Opterećenje vjetrom u uzdužnom smjeru, slučaj sa otvorenim vratima; djelovanje na zidove, donji dio krova kroz zonu F, donji dio krova kroz zonu G i gornji dio krova korz zonu G

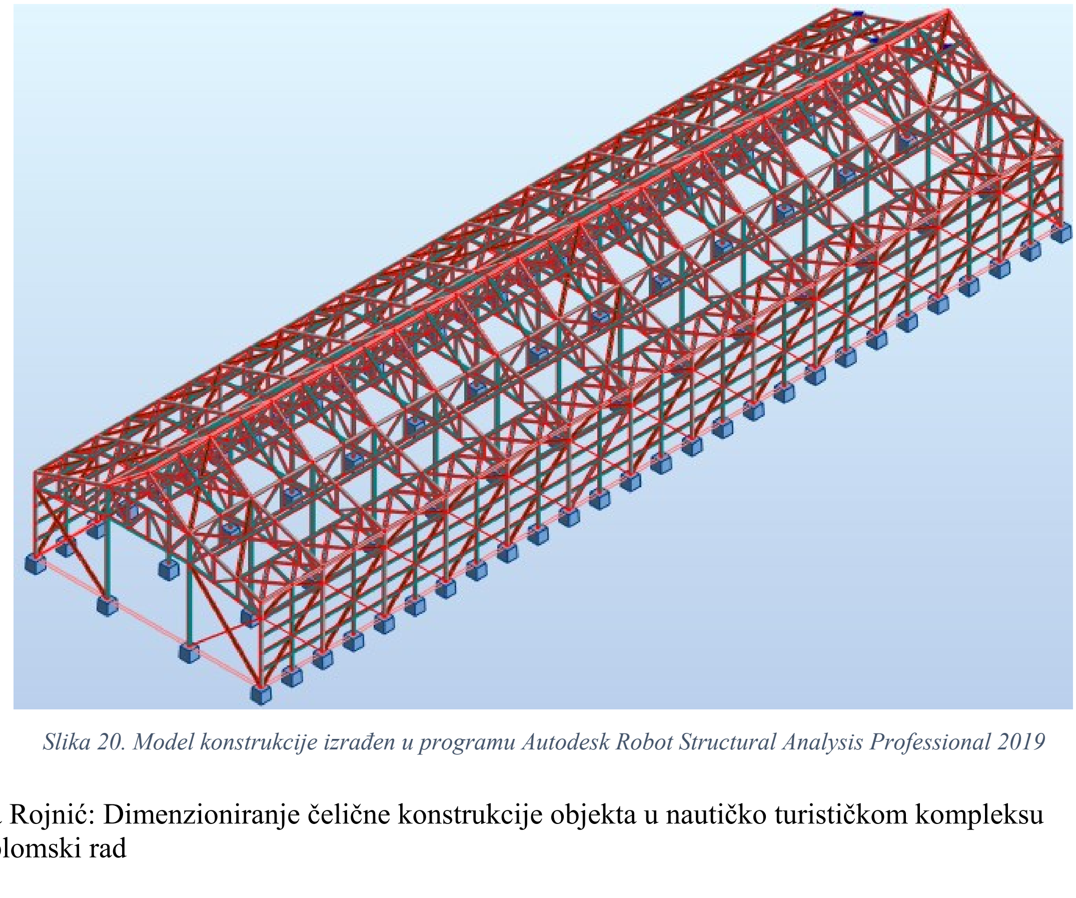
6. STATIČKI PRORAČUN

Prvi korak u izradi modela je izrada žičanog 3D modela dva karakteristična okvira u programskom paketu *AutoCAD 2019* [11] koji se potom uvozi u *Autodesk Robot Structural Analysis Professional 2019* [1]. Žičani model prikazan je na slici 19.



Slika 19. Žičani 3D model konstrukcije

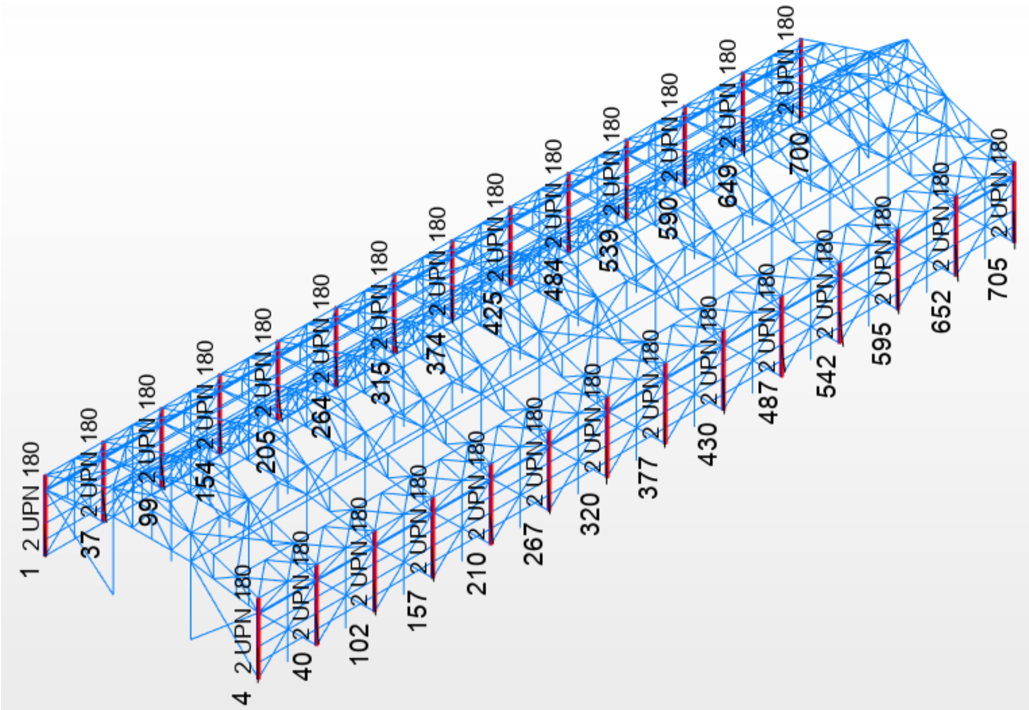
Svaka linija u *AutoCAD-u* prikazuje os jednog elementa kojem se dodjeljuje neki poprečni presjek, stupovima se definiraju oslonci, te se karakteristični okviri kopiraju, da se formira cijela konstrukcija, koja se potom opterećuje.



Slika 20. Model konstrukcije izrađen u programu Autodesk Robot Structural Analysis Professional 2019

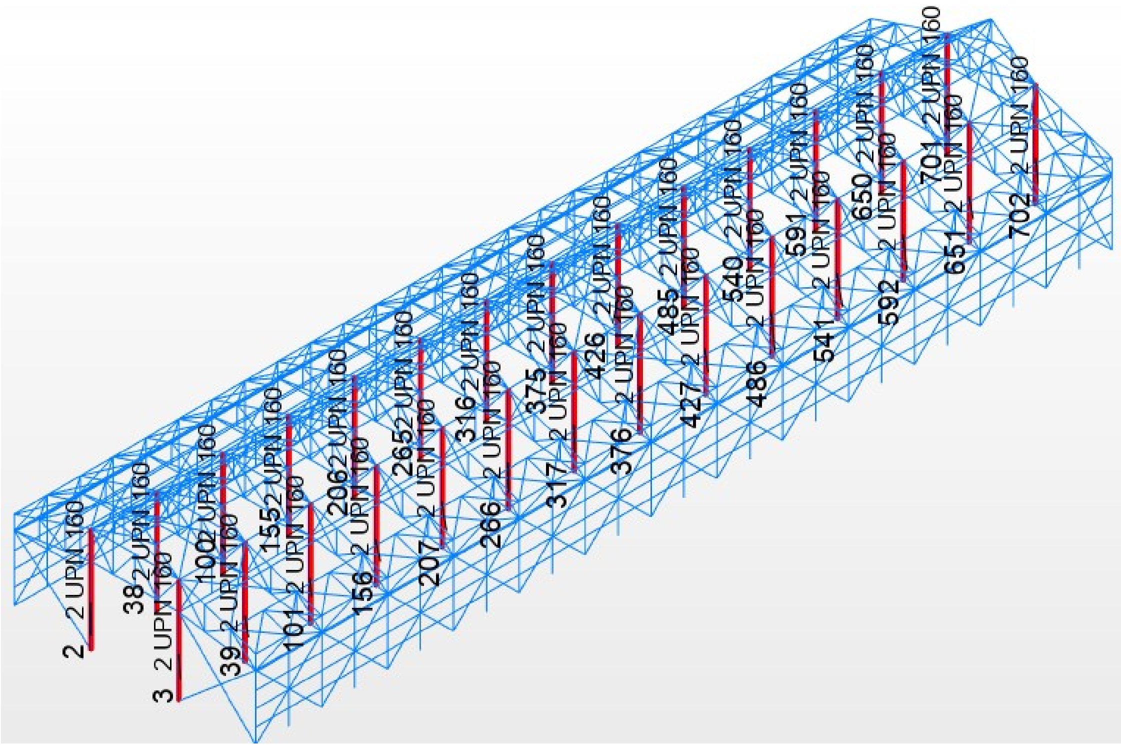
6.1. Plan pozicija

Pozicija S1 su glavni stupovi poprečnog presjeka sastavljenog od U profila 2UPN180, a njihov položaj prikazan je na slici 21.



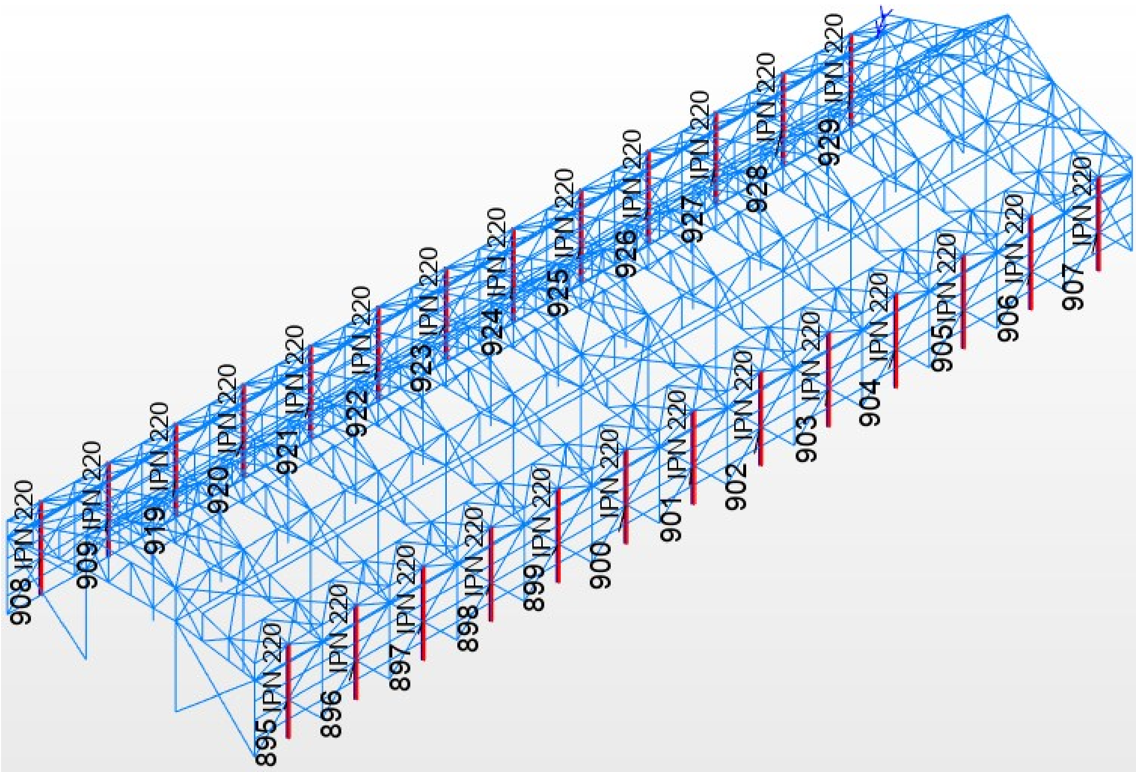
Slika 21. Pozicija S1

Pozicija S2 su unutarnji stupovi poprečnog presjeka sastavljenog od U profila 2UPN160 razmaknutih 100 mm, a njihov položaj prikazan je na slici 22.



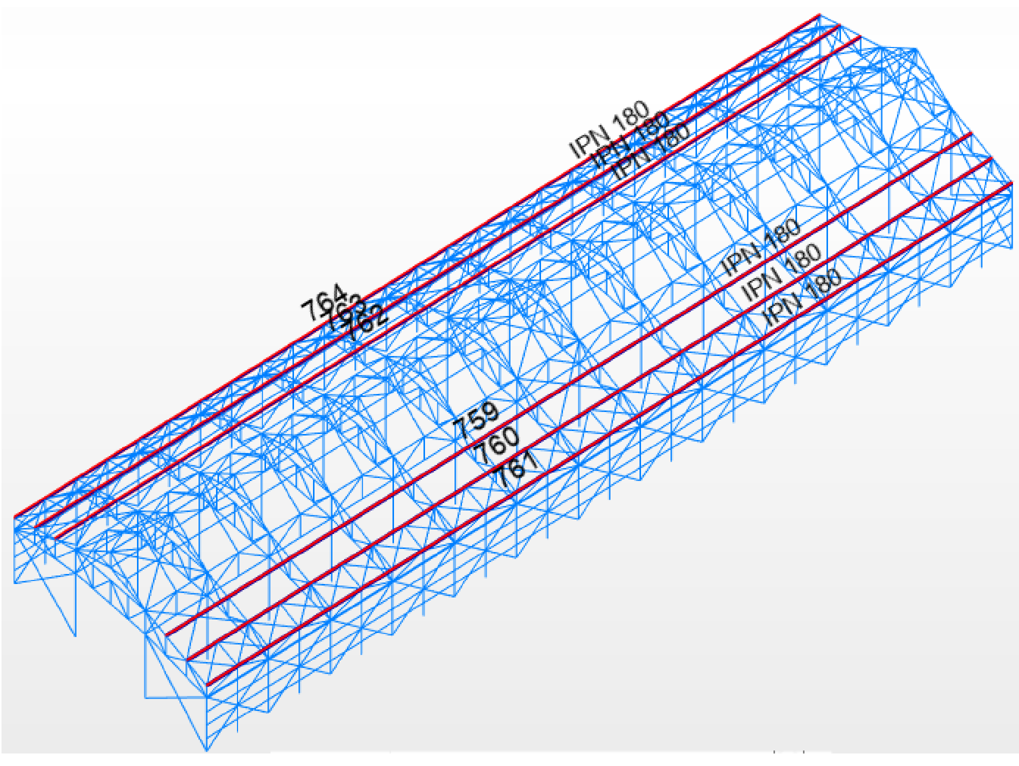
Slika 22. Pozicija S2

Pozicija S3 su sekundarni fasadni stupovi I poprečnog presjeka IPN220 , a njihov položaj prikazan je na slici 23.



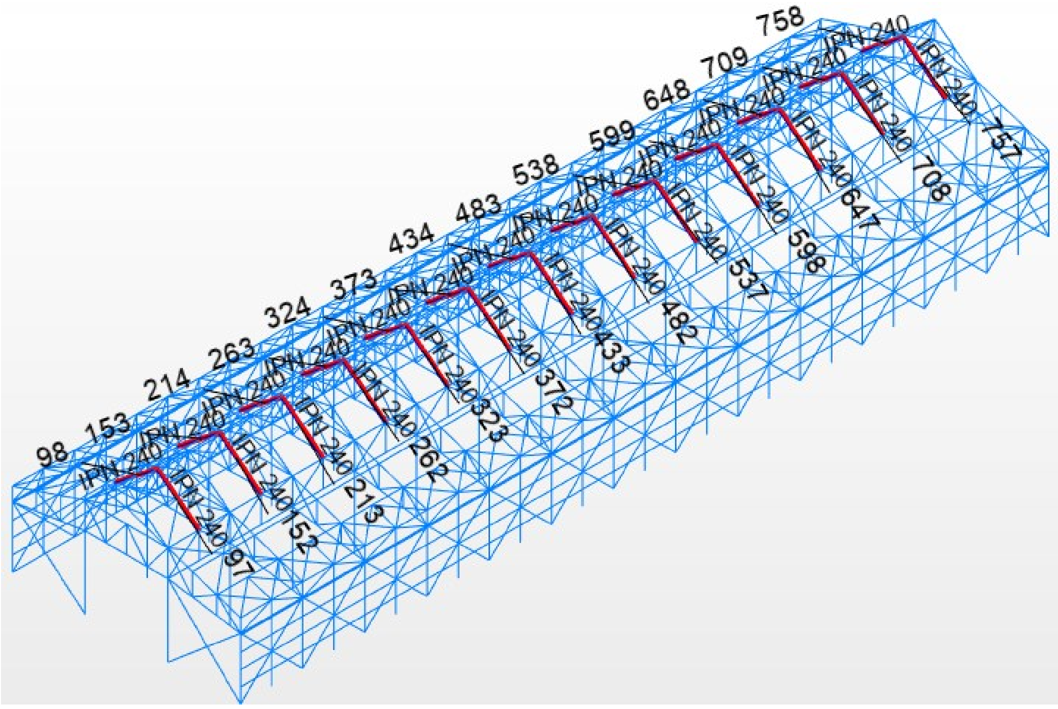
Slika 23. Pozicija S3

Pozicija P1 su podrožnice na donjem dijelu krova I poprečnog presjeka IPN180, a njihov položaj prikazan je na slici 24.



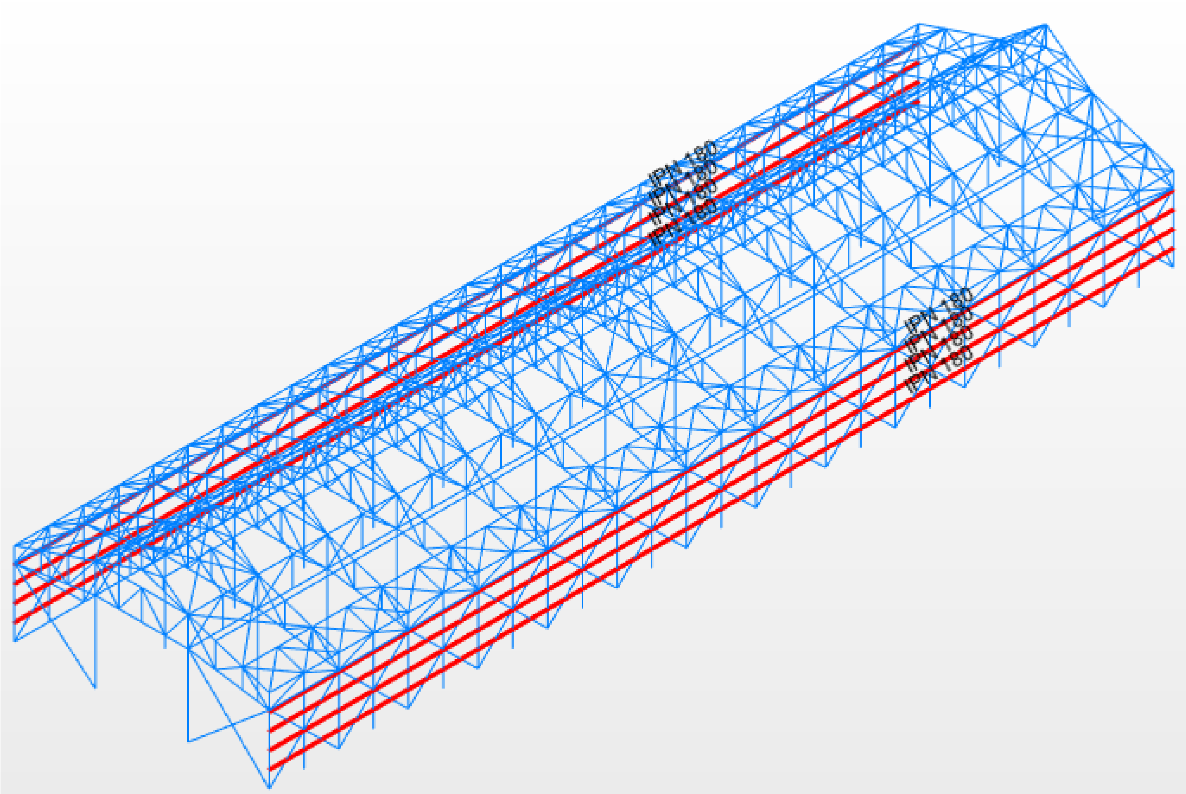
Slika 24. Pozicija P1

Pozicija P2 su podrožnice na gornjem dijelu krova I poprečnog presjeka IPN240, a njihov položaj prikazan je na slici 25.



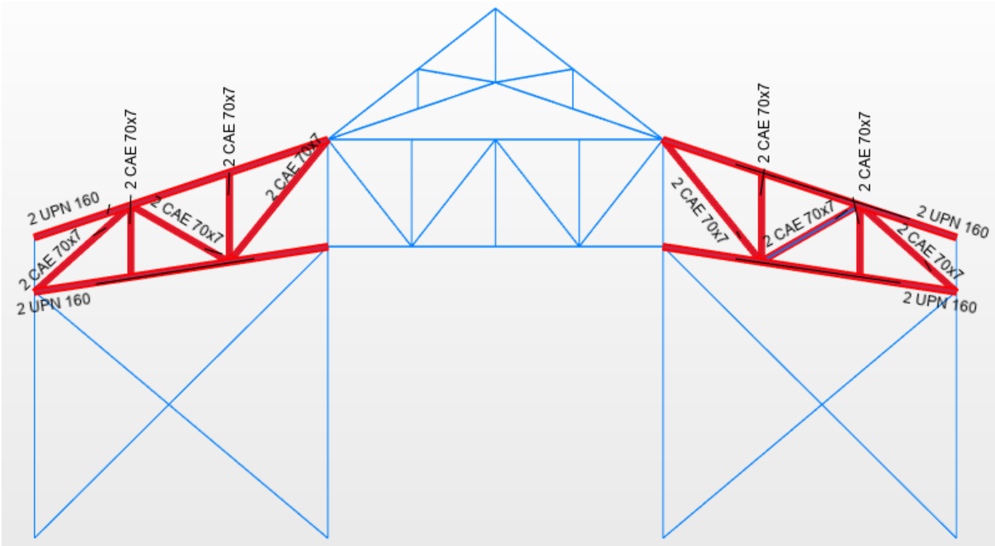
Slika 25. Pozicija P2

Pozicija P3 su nosači fasade I poprečnog presjeka IPN180, a njihov položaj prikazan je na slici 26.



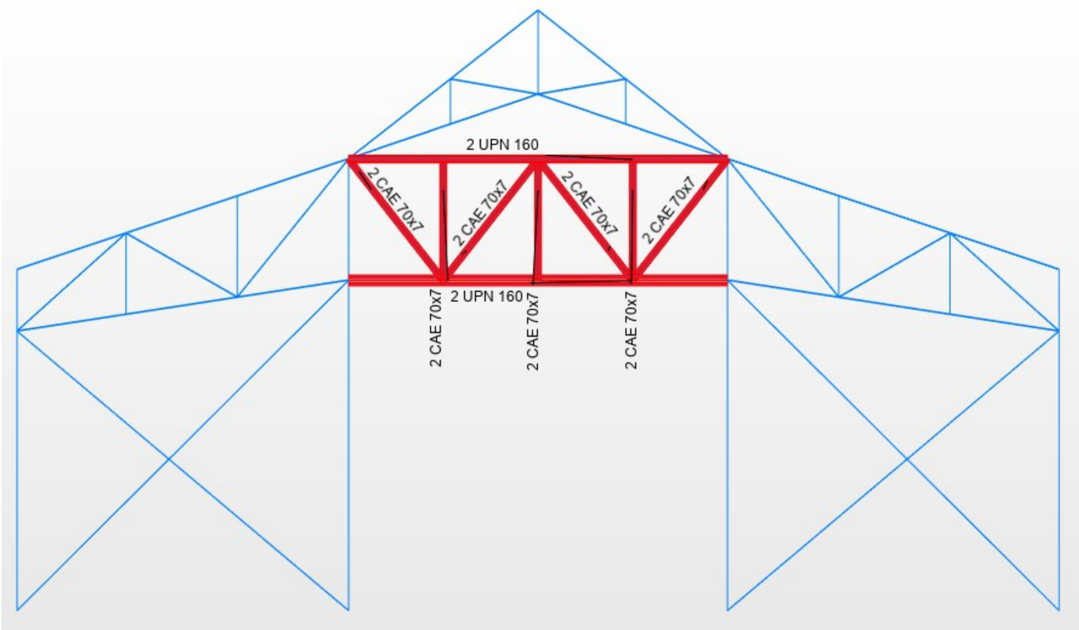
Slika 26. Pozicija P3

Pozicija R1 je bočni dio rešetke glavnog okvira koja se sastoji od gornjeg i donjeg pojasa poprečnog presjeka sastavljenog od U profila 2UPN160 i ispuna poprečnog presjeka sastavljenog od L profila 2CAE70x7, a njezin položaj prikazan je na slici 27.



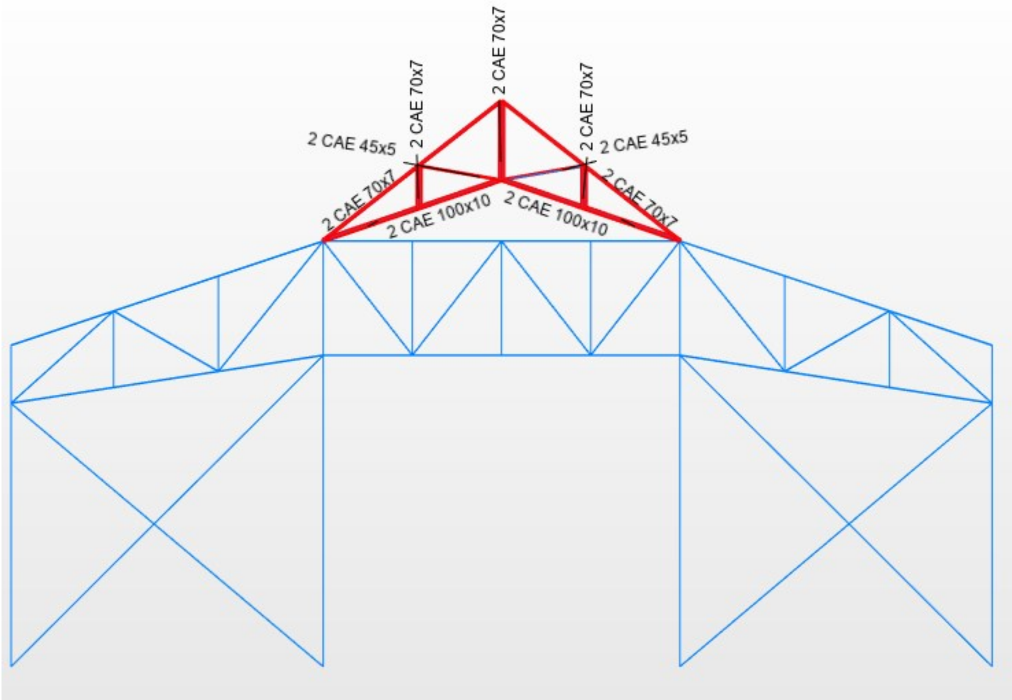
Slika 27. Pozicija R1

Pozicija R2 je donji središnji dio glavnog okvira koji se sastoji od gornjeg i donjeg pojasa poprečnog presjeka sastavljenog od U profila 2UPN160 i ispuna poprečnog presjeka sastavljenog od L profila 2CAE70x7, a njezin položaj prikazan je na slici 28.



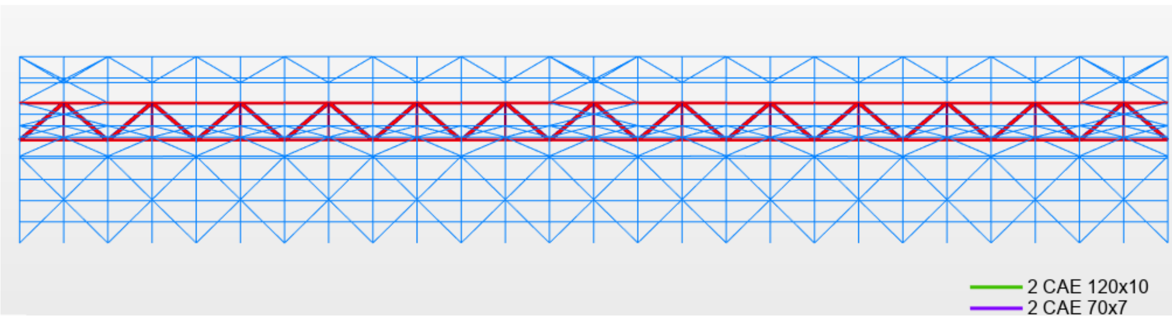
Slika 28. Pozicija R2

Pozicija R3 je gornji središnji dio glavnog okvira koja se sastoji od gornjeg pojasa poprečnog presjeka sastavljenog od L profila 2CAE70x7, donjeg pojasa poprečnog presjeka sastavljenog od L profila 2CAE100x10 i ispuna poprečnih presjeka sastavljenih od L profila 2CAE45x5 i 2CAE70x7, a njezin položaj prikazan je na slici 29.



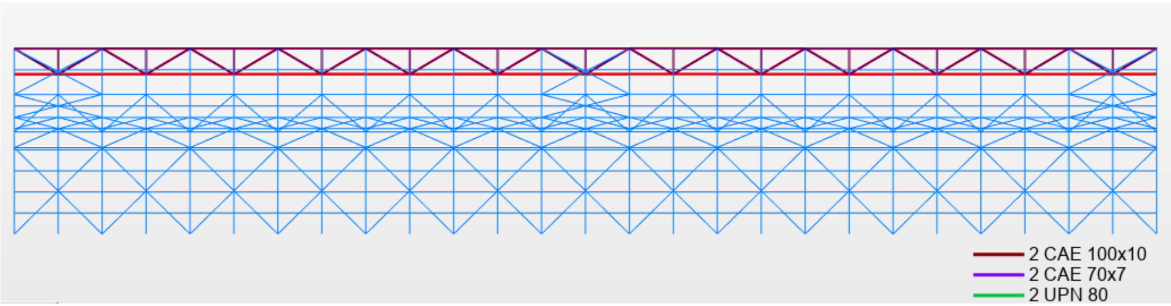
Slika 29. Pozicija R3

Pozicija R4 je bočna sekundarna rešetka koja se sastoji od gornjeg i donjeg pojasa poprečnog presjeka sastavljenog od L profila 2CAE120x10 i ispuna poprečnog presjeka sastavljenog od L profila 2CAE70x7, a njezin položaj prikazan je na slici 30.



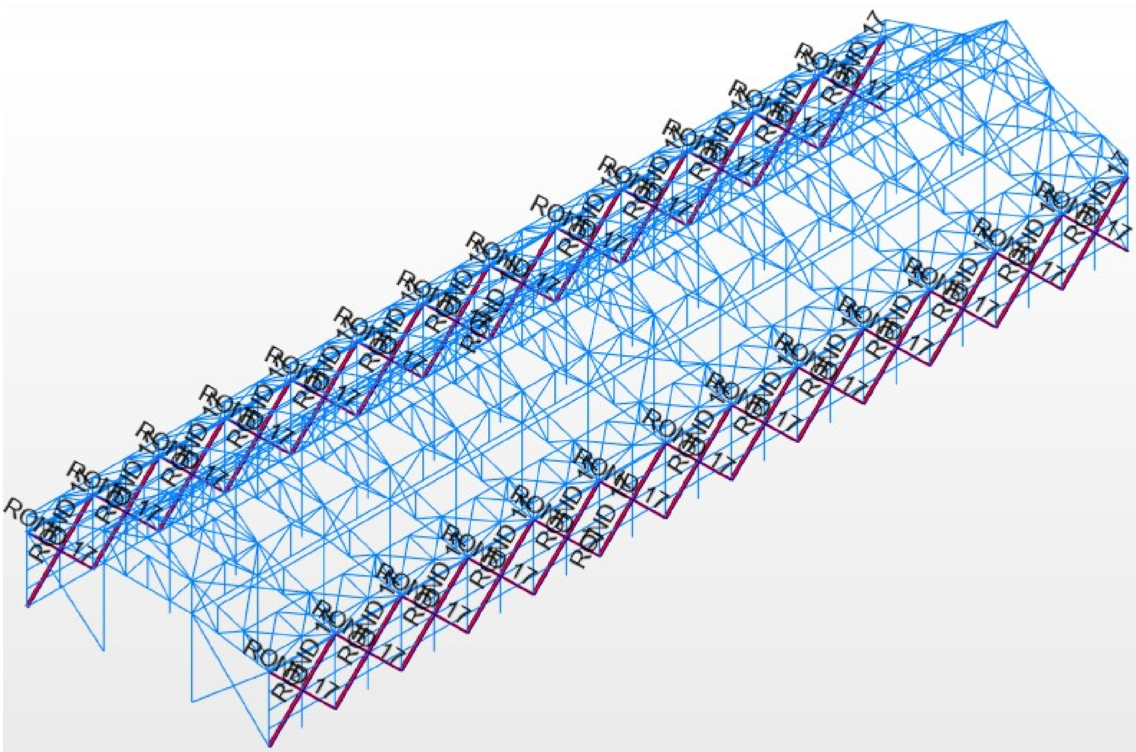
Slika 30. Pozicija R4

Pozicija R5 je središnja sekundarna rešetka koja se sastoji od gornjeg pojasa poprečnog presjeka sastavljenog od U profila 2UPN80, donjeg pojasa poprečnog presjeka sastavljenog od L profila 2CAE100x10 i ispuna poprečnog presjeka sastavljenog od L profila 2CAE70x7, a njezin položaj prikazan je na slici 31.



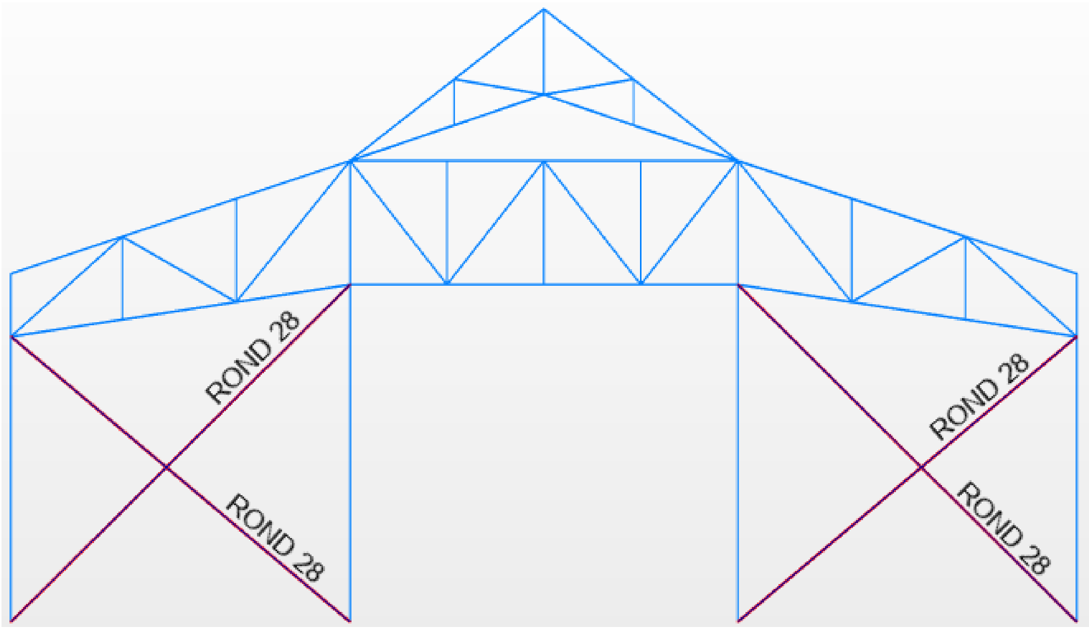
Slika 31. Pozicija R5

Pozicija V1 su fasadni stabilizacijski vezovi šupljeg cjevnog poprečnog presjeka ϕ17, a njihov položaj prikazan je na slici 32.



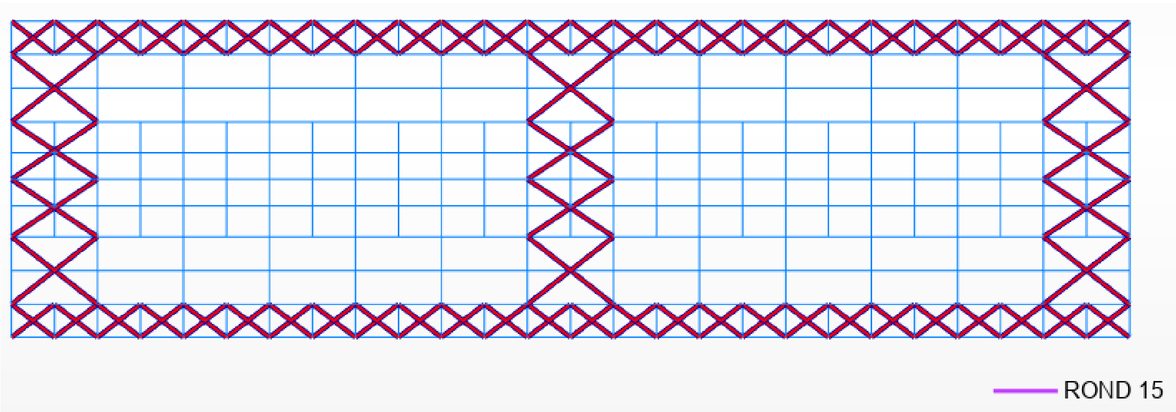
Slika 32. Pozicija V1

Pozicija V2 su zabatni stabilizacijski vezovi šupljeg cjevnog poprečnog presjeka ϕ28, a njihov položaj prikazan je na slici 33.



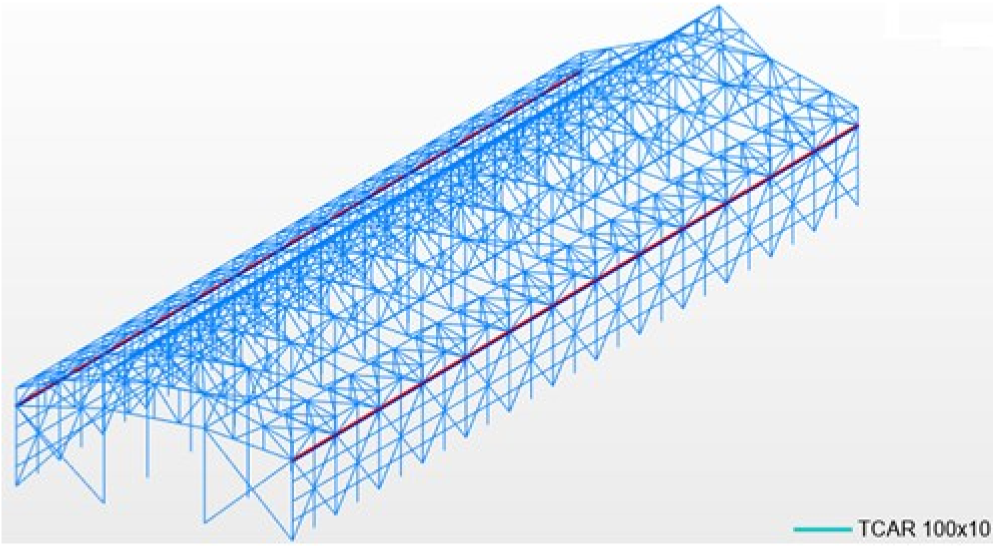
Slika 33. Pozicija V2

Pozicija V3 su krovni stabilizacijski vezovi šupljeg cjevnog poprečnog presjeka ϕ15, a njihov položaj prikazan je na slici 34.



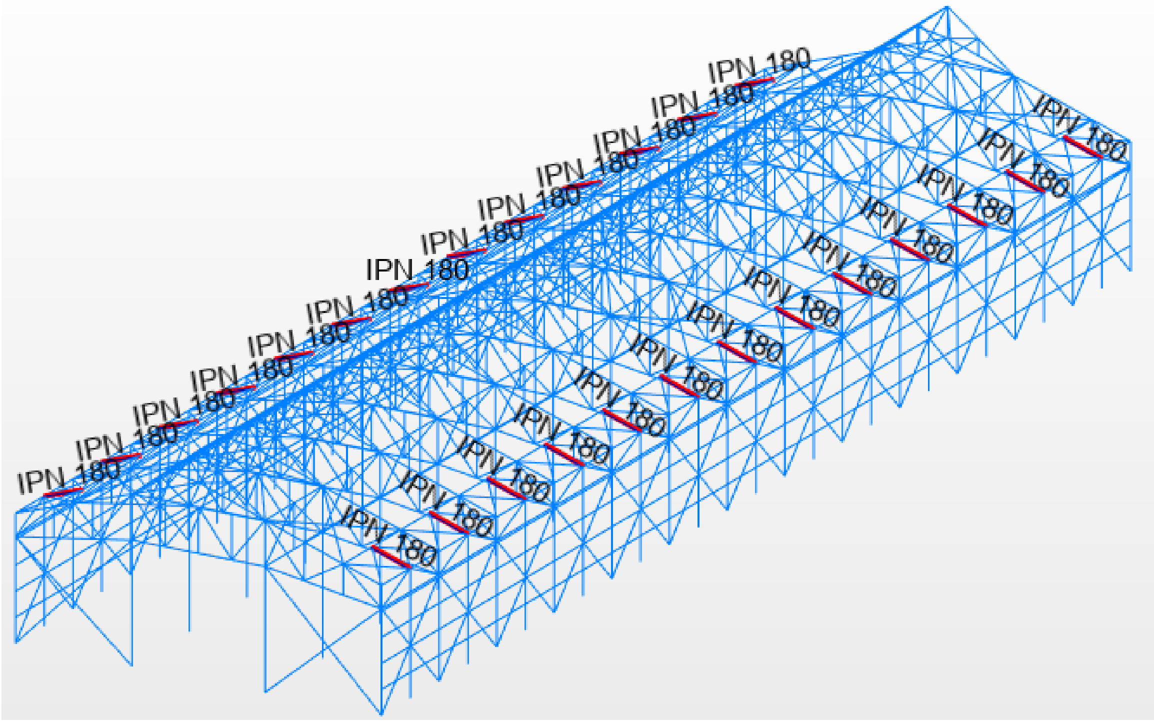
Slika 34. Pozicija V3

Pozicija E1 su fasadni elementi šupljeg kvadratnog poprečnog presjeka TCAR100x10 , a njihov položaj prikazan je na slici 35.



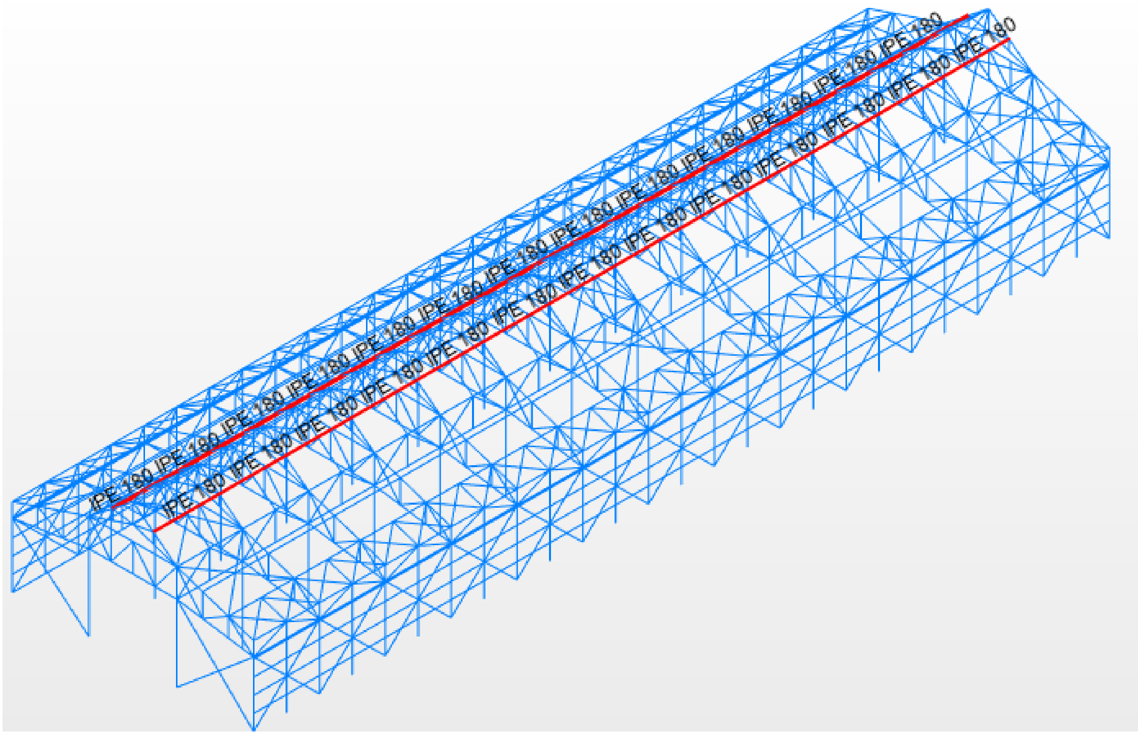
Slika 35. Pozicija E1

Pozicija E2 su krovni elementi I poprečnog presjeka IPN180, a njihov položaj prikazan je na slici 36.



Slika 36. Pozicija E2

Pozicija E3 su krovni elementi I poprečnog presjeka IPE180, a njihov položaj prikazan je na slici 37.



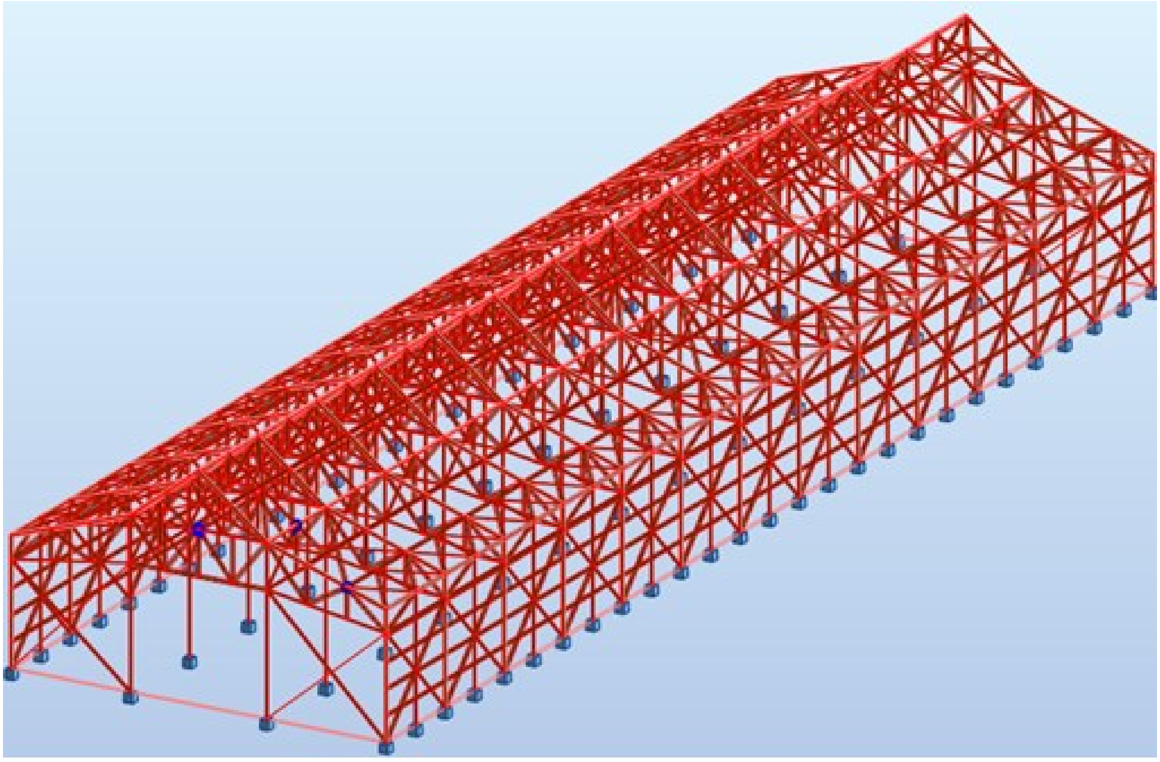
Slika 37. Pozicija E3

6.2. Opterećenja

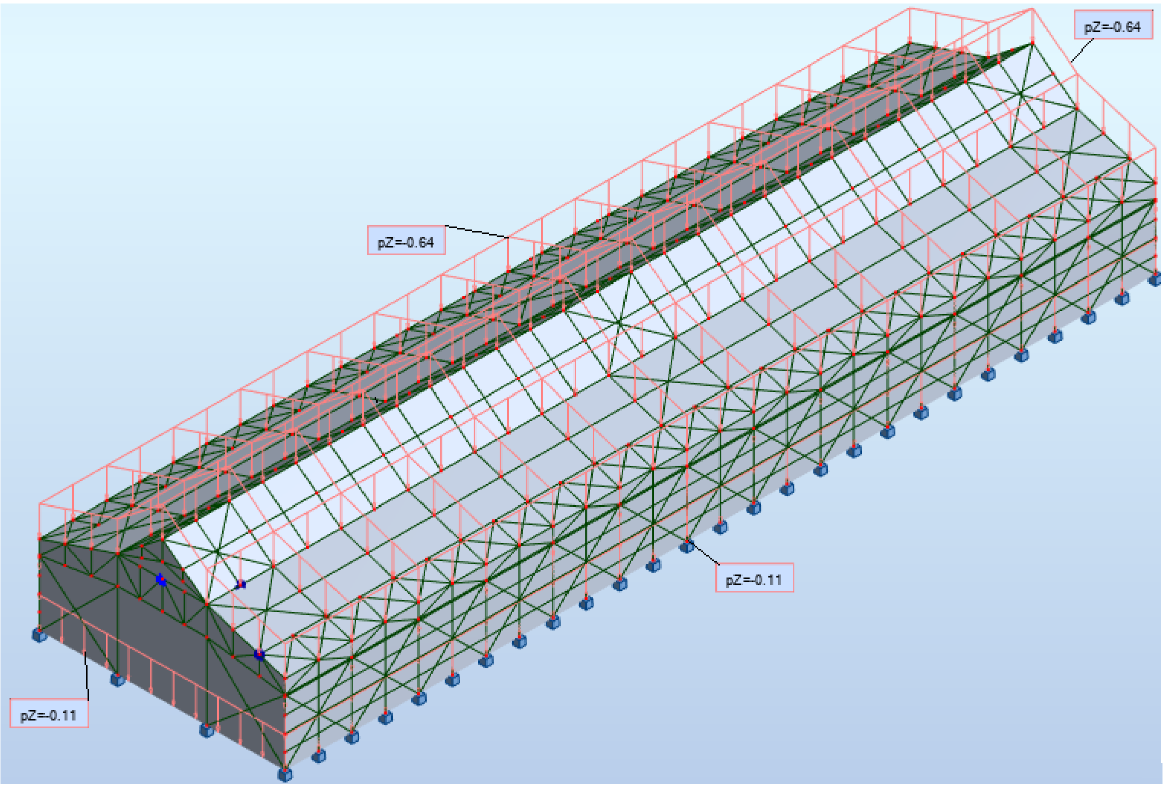
Analizom opterećenja dobivene vrijednosti unesene su u model preko ploha (claddings), a slučajevi opterećenja navedeni su u tablici 8, a grafički prikazani na slikama od 38 do 47.

Tablica 8. Slučajevi opterećenja

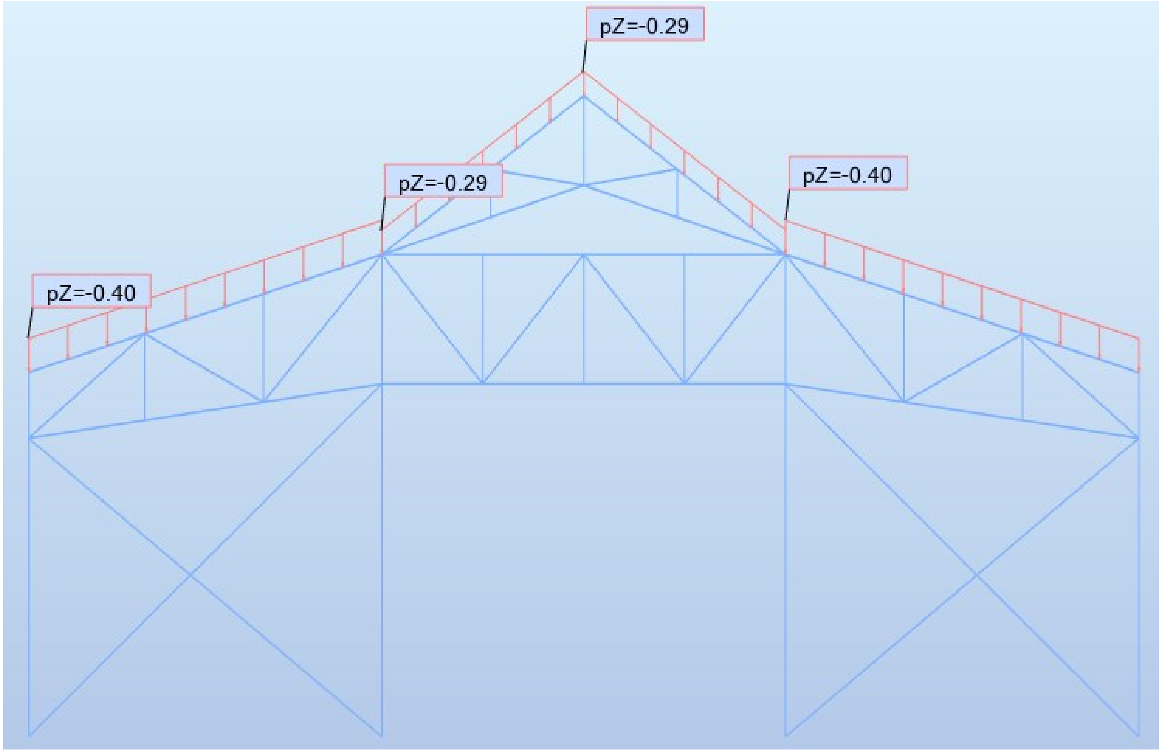
Redni broj	Opterećenje
1.	Vlastita težina
2.	Stalno opterećenje
3.	Simetrični snijeg
4.	Nesimetrični snijeg
5.	Vjetar-uzdužni, negativni unutarnji pritisak, otvorena vrata
6.	Vjetar-uzdužni, negativni unutarnji pritisak, zatvorena vrata
7.	Vjetar-poprečni, pozitivni unutarnji pritisak, otvorena vrata
8.	Vjetar-poprečni, negativni unutarnji pritisak, otvorena vrata
9.	Vjetar-poprečni, pozitivni unutarnji pritisak, zatvorena vrata
10.	Vjetar-poprečni, negativni unutarnji pritisak, zatvorena vrata



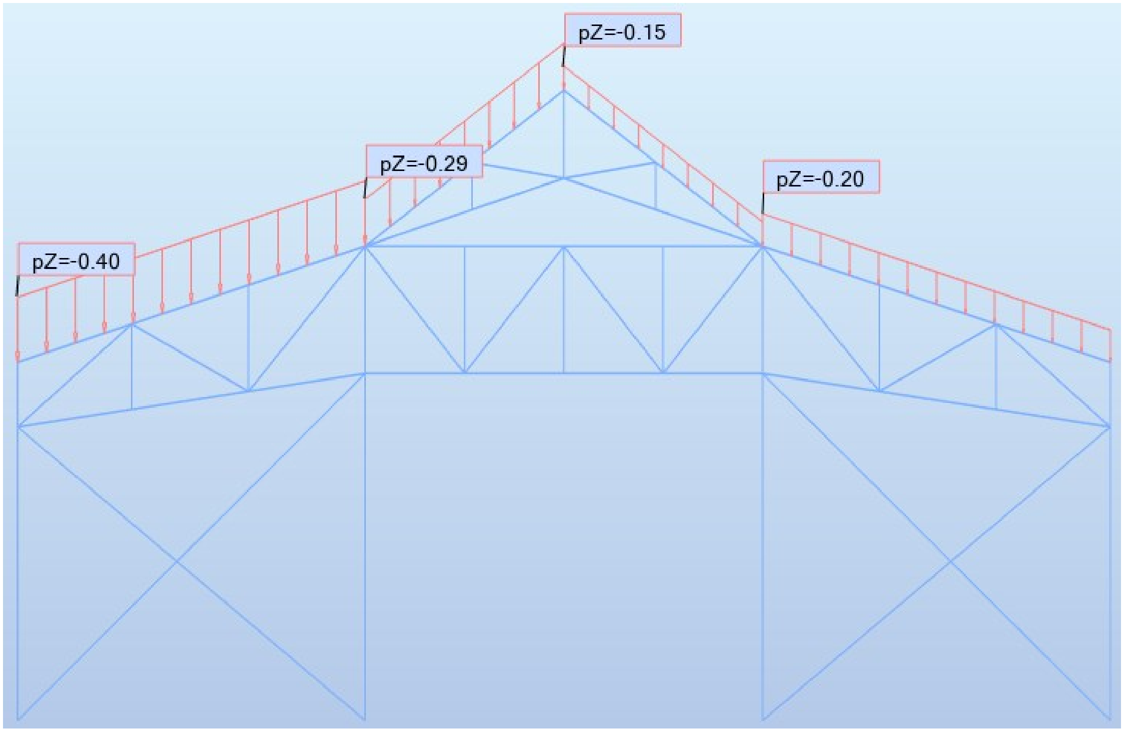
Slika 38. Vlastita težina



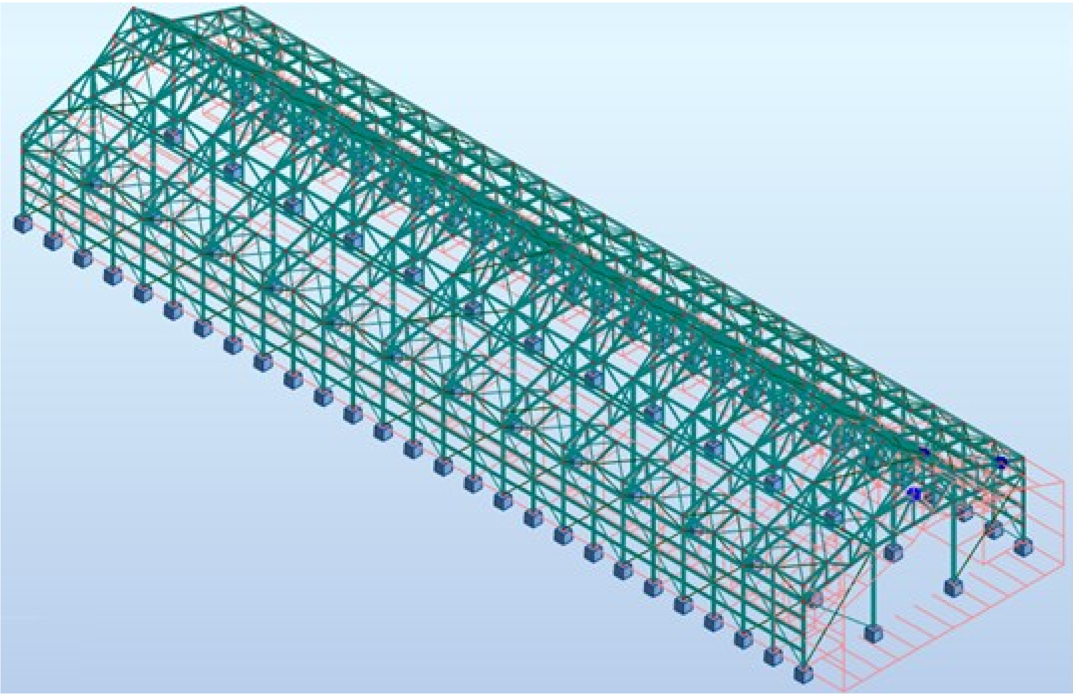
Slika 39. Stalno opterećenje



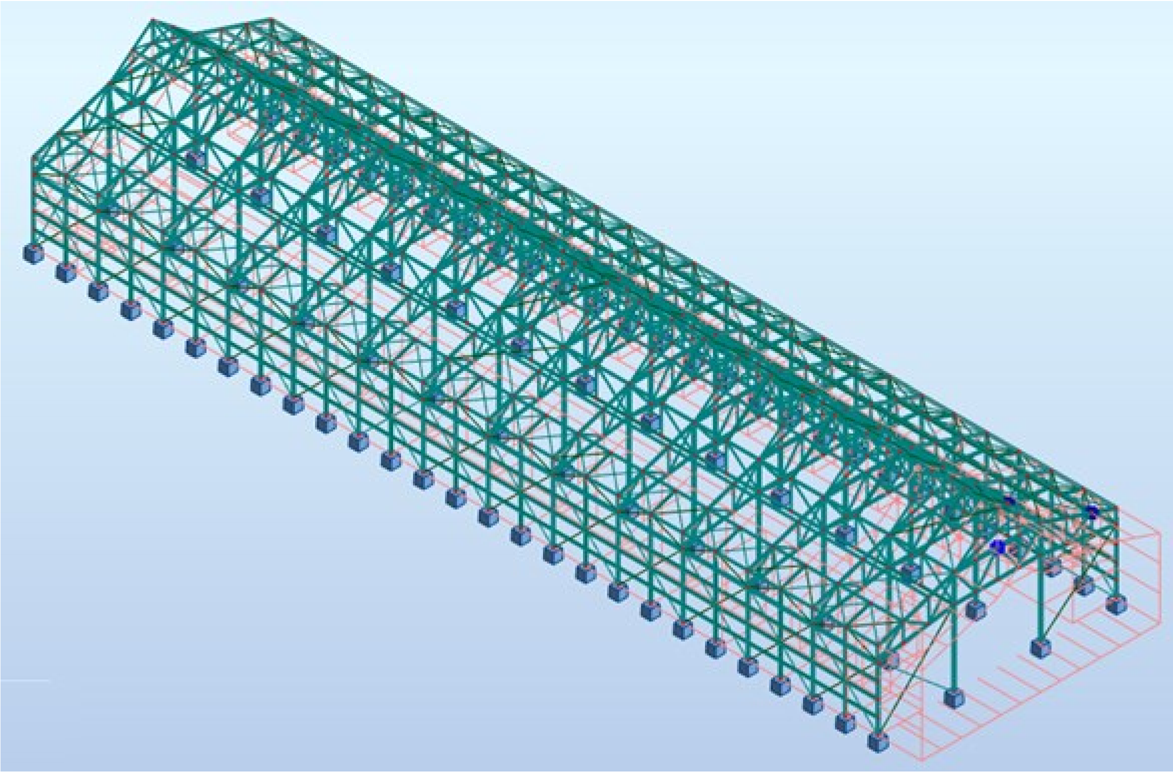
Slika 40. Simetrični snijeg



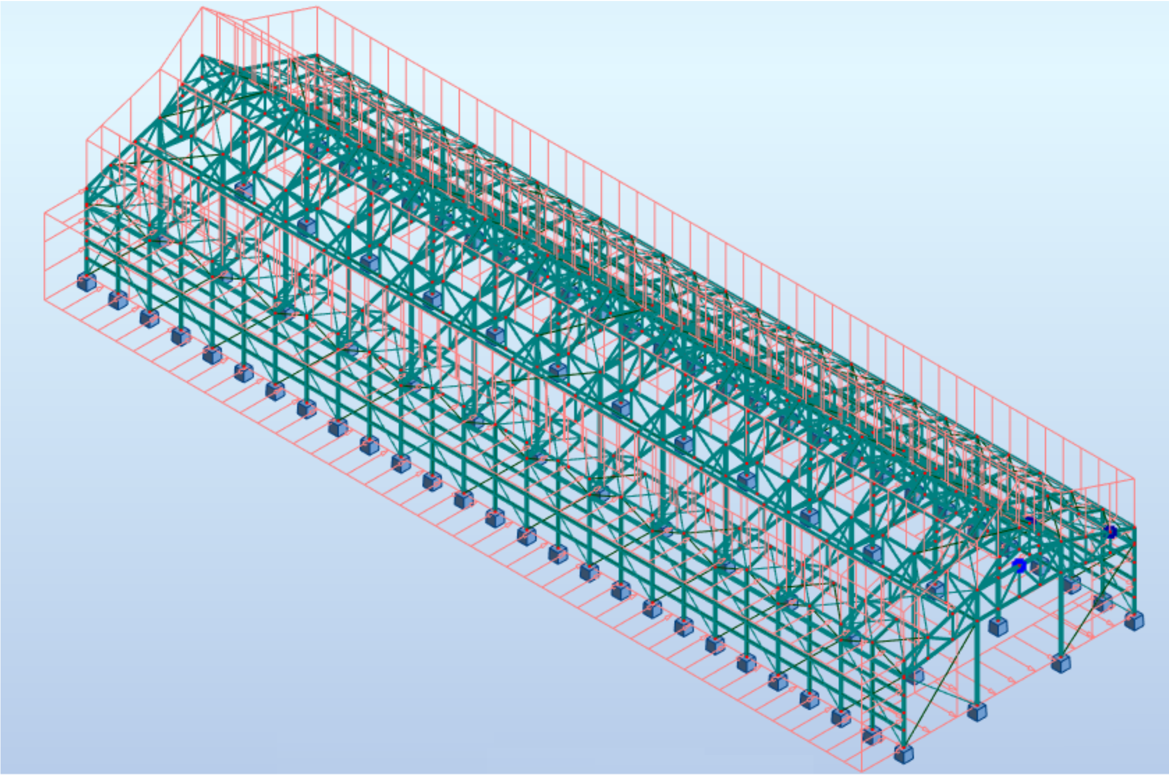
Slika 41. Nesimetrični snijeg



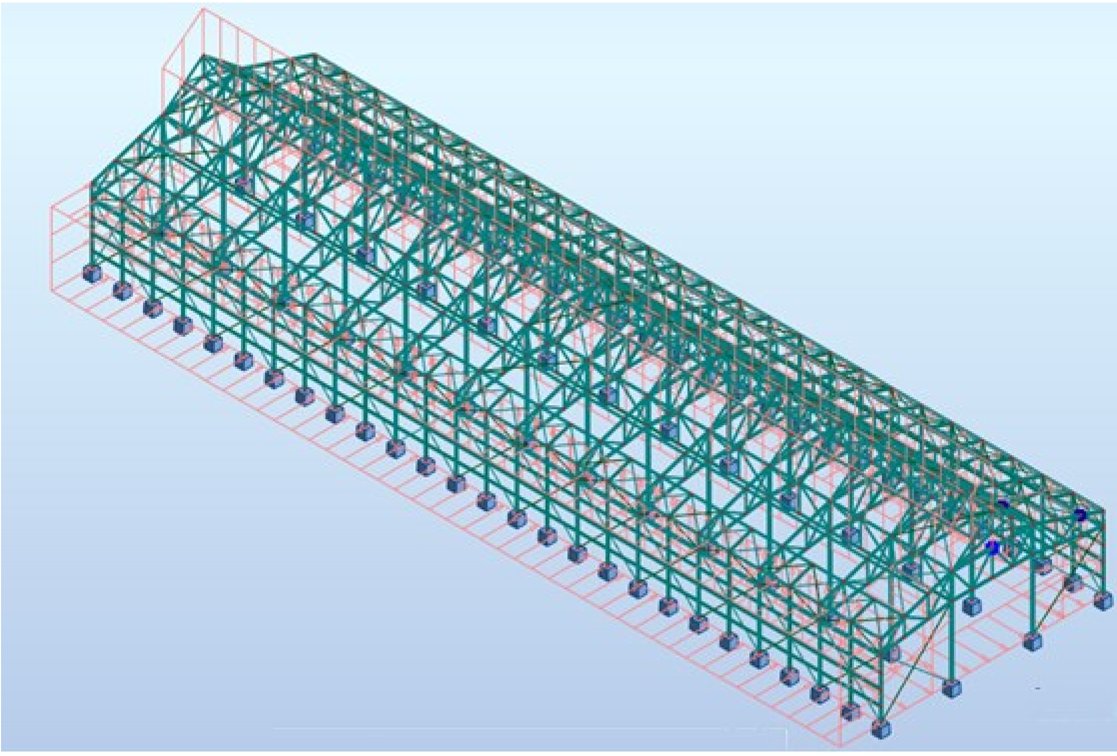
Slika 42. Vjetar-uzdužni, negativni unutarnji pritisak, otvorena vrata



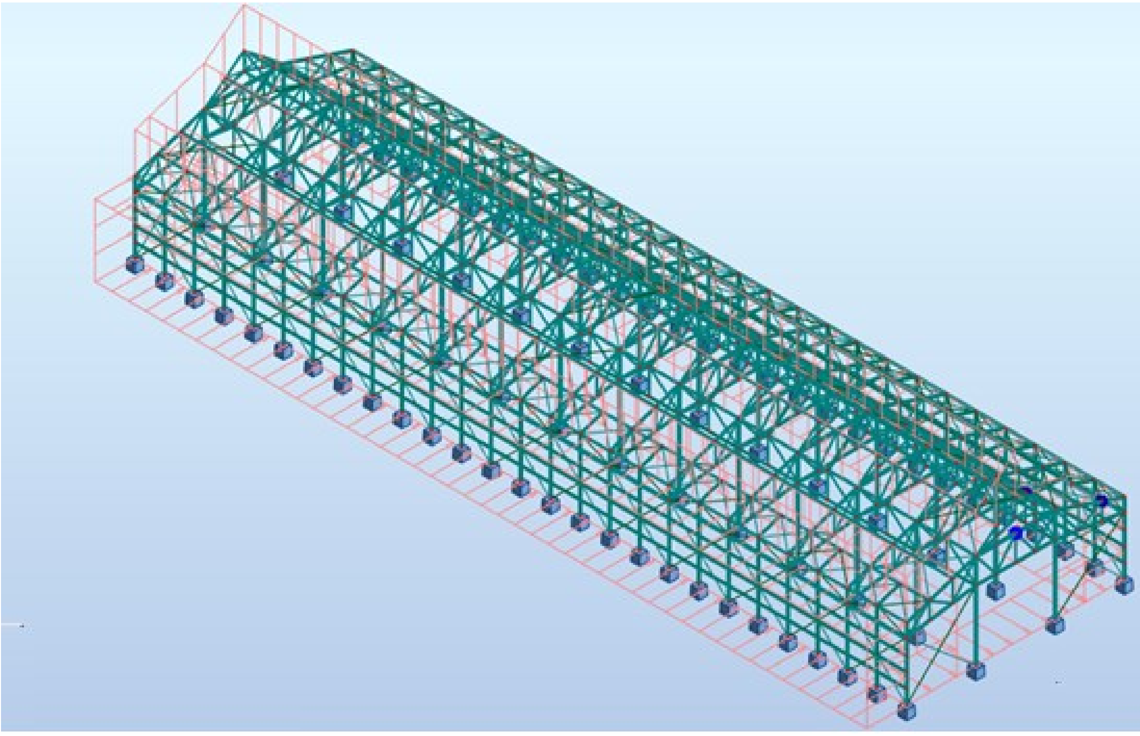
Slika 43. Vjetar-uzdužni, negativni unutarnji pritisak, zatvorena vrata



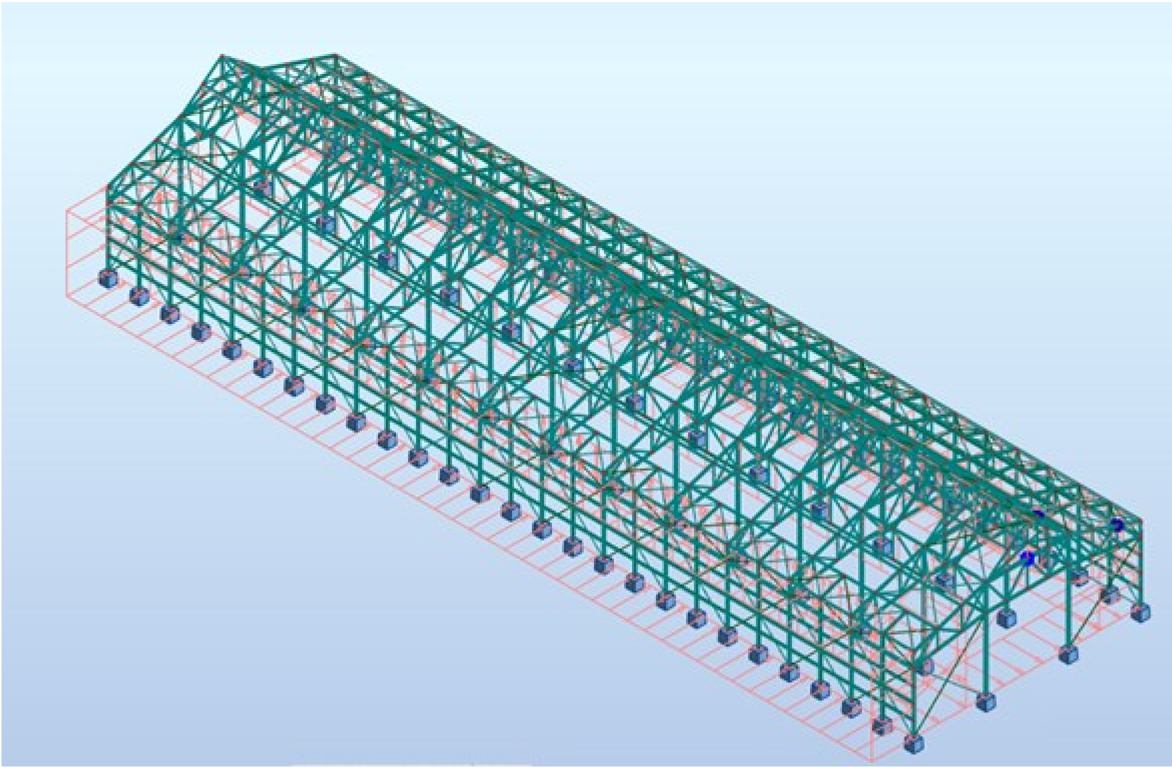
Slika 44. Vjetar-poprečni, pozitivni unutarnji pritisak, otvorena vrata



Slika 45. Vjetar-poprečni, negativni unutarnji pritisak, otvorena vrata



Slika 46. Vjetar-poprečni, pozitivni unutarnji pritisak, zatvorena vrata



Slika 47. Vjetar-poprečni, negativni unutarnji pritisak, zatvorena vrata

6.3. Kombinacije djelovanja

Prema [7] i [8] definirane su kombinacije djelovanja za provjeru graničnog stanja nosivosti (eng. Ultimate limit state) i graničnog stanja uporabivosti (eng. Serviceability limit state).

Za provjeru graničnog stanja nosivosti razmatra se osnovna proračunska situacija:

$$\sum_{j\geq 1} "\gamma_{G,j} G_{k,j}" + "\gamma_P P_k" + "\gamma_{Q,1} Q_{k,1}" + \sum_{i\geq 1} "\gamma_{Q,i} \Psi_{0,i} Q_{k,i}"$$

gdje su:

$G_{k,j}$ – karakteristična vrijednost stalnog djelovanja

P_k – karakteristična vrijednost djelovanja od prednapinjanja

$Q_{k,1}$ – karakteristična vrijednost vodećeg promjenjivog djelovanja

$Q_{k,i}$ – karakteristična vrijednost ostalih promjenjivih djelovanja

$\gamma_{G,j}$ – parcijalni faktor sigurnosti za stalna djelovanja

γ_P – parcijalni faktor sigurnosti za djelovanja od prednapinjanja

$\gamma_{Q,1}$ - parcijalni faktor sigurnosti za vodeće promjenjivo djelovanje

$\gamma_{Q,i}$ - parcijalni faktor sigurnosti za ostala promjenjiva djelovanja

$\Psi_{0,i}$ – faktor kombinacije opterećenja

Za provjeru graničnog stanja uporabivosti razmatra se karakteristična proračunska situacija:

$$\sum_{j\geq 1} "G_{k,j}" + "Q_{k,1}" + \sum_{i\geq 1} "\gamma_{Q,i} \Psi_{0,i} Q_{k,i}"$$

gdje su:

$G_{k,j}$ – karakteristična vrijednost stalnog djelovanja

$Q_{k,1}$ – karakteristična vrijednost vodećeg promjenjivog djelovanja

$Q_{k,i}$ – karakteristična vrijednost ostalih promjenjivih djelovanja

$\gamma_{Q,i}$ - parcijalni faktor sigurnosti za ostala promjenjiva djelovanja

$\Psi_{0,i}$ – faktor kombinacije opterećenja

Kombinacije za provjeru graničnog stanja nosivosti prikazane su u tablici 9.

Tablica 9. Kombinacije djelovanja za GSN

Redni broj	Kombinacija	Granično stanje	Kombinacija
11.	COMB1	GSN	(1+2)*1.35+3*1.50+5*0.90
12.	COMB2	GSN	(1+2)*1.35+3*1.50+6*0.90
13.	COMB3	GSN	(1+2)*1.35+3*1.50+7*0.90
14.	COMB4	GSN	(1+2)*1.35+3*1.50+8*0.90
15.	COMB5	GSN	(1+2)*1.35+3*1.50+9*0.90
16.	COMB6	GSN	(1+2)*1.35+3*1.50+10*0.90
17.	COMB7	GSN	(1+2)*1.35+4*1.50+5*0.90
18.	COMB8	GSN	(1+2)*1.35+4*1.50+6*0.90
19.	COMB9	GSN	(1+2)*1.35+4*1.50+7*0.90
20.	COMB10	GSN	(1+2)*1.35+4*1.50+8*0.90
21.	COMB11	GSN	(1+2)*1.35+4*1.50+9*0.90
22.	COMB12	GSN	(1+2)*1.35+4*1.50+10*0.90
23.	COMB13	GSN	(1+2)*1.35+5*1.50+3*0.75
24.	COMB14	GSN	(1+2)*1.35+6*1.50+3*0.75
25.	COMB15	GSN	(1+2)*1.35+7*1.50+3*0.75
26.	COMB16	GSN	(1+2)*1.35+8*1.50+3*0.75
27.	COMB17	GSN	(1+2)*1.35+9*1.50+3*0.75
28.	COMB18	GSN	(1+2)*1.35+10*1.50+3*0.75
29.	COMB19	GSN	(1+2)*1.35+5*1.50+4*0.75
30.	COMB20	GSN	(1+2)*1.35+6*1.50+4*0.75
31.	COMB21	GSN	(1+2)*1.35+7*1.50+4*0.75
32.	COMB22	GSN	(1+2)*1.35+8*1.50+4*0.75
33.	COMB23	GSN	(1+2)*1.35+9*1.50+4*0.75
34.	COMB24	GSN	(1+2)*1.35+10*1.50+4*0.75
35.	COMB25	GSN	(1+2)*1.35+3*1.50
36.	COMB26	GSN	(1+2)*1.35+4*1.50
37.	COMB27	GSN	(1+2)*1.35+5*1.50
38.	COMB28	GSN	(1+2)*1.35+6*1.50
39.	COMB29	GSN	(1+2)*1.35+7*1.50
40.	COMB30	GSN	(1+2)*1.35+8*1.50
41.	COMB31	GSN	(1+2)*1.35+9*1.50
42.	COMB32	GSN	(1+2)*1.35+10*1.50

Kombinacije djelovanja za provjeru graničnog stanja uporabivosti prikazane su u tablici 10.

Tablica 10. Kombinacije opterećenja za GSU

Redni broj	Kombinacija	Granično stanje	Kombinacija
43.	COMB33	GSU	(1+2+3)*1.00+5*0.60
44.	COMB34	GSU	(1+2+3)*1.00+6*0.60
45.	COMB35	GSU	(1+2+3)*1.00+7*0.60
46.	COMB36	GSU	(1+2+3)*1.00+8*0.60
47.	COMB37	GSU	(1+2+3)*1.00+9*0.60
48.	COMB38	GSU	(1+2+3)*1.00+10*0.60
49.	COMB39	GSU	(1+2+4)*1.00+5*0.60
50.	COMB40	GSU	(1+2+4)*1.00+6*0.60
51.	COMB41	GSU	(1+2+4)*1.00+7*0.60
52.	COMB42	GSU	(1+2+4)*1.00+8*0.60
53.	COMB43	GSU	(1+2+4)*1.00+9*0.60
54.	COMB44	GSU	(1+2+4)*1.00+10*0.60
55.	COMB45	GSU	(1+2+5)*1.00+3*0.50
56.	COMB46	GSU	(1+2+6)*1.00+3*0.50
57.	COMB47	GSU	(1+2+7)*1.00+3*0.50
58.	COMB48	GSU	(1+2+8)*1.00+3*0.50
59.	COMB49	GSU	(1+2+9)*1.00+3*0.50
60.	COMB50	GSU	(1+2+10)*1.00+3*0.50
61.	COMB51	GSU	(1+2+5)*1.00+4*0.50
62.	COMB52	GSU	(1+2+6)*1.00+4*0.50
63.	COMB53	GSU	(1+2+7)*1.00+4*0.50
64.	COMB54	GSU	(1+2+8)*1.00+4*0.50
65.	COMB55	GSU	(1+2+9)*1.00+4*0.50
66.	COMB56	GSU	(1+2+10)*1.00+4*0.50
67.	COMB57	GSU	(1+2+3)*1.00
68.	COMB58	GSU	(1+2+4)*1.00
69.	COMB59	GSU	(1+2+5)*1.00
70.	COMB60	GSU	(1+2+6)*1.00
71.	COMB61	GSU	(1+2+7)*1.00
72.	COMB62	GSU	(1+2+8)*1.00
73.	COMB63	GSU	(1+2+9)*1.00
74.	COMB64	GSU	(1+2+10)*1.00

6.4. Rezultati statičkog proračuna

U nastavku će biti prikazani rezultati statičkog proračuna – prvo vrijednosti unutarnjih sila potrebne za provjeru GSN, a potom vrijednosti pomaka u karakterističnim točkama potrebne za provjeru GSU.

6.4.1. Vrijednosti unutarnjih sila

Pozicija S1

Tablica 11. Unutarnje sile - pozicija S1

S1 2UPN180	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	175,02	7,23	13,69	0,3	20,96	2,25
Bar	705	102	40	484	1	1
Node	597	94	41	408	1	1
Case	25	39	24	23	25	31
MIN	-12,15	-9,24	-13,7	-0,3	-23,06	-10,14
Bar	4	37	37	487	700	37
Node	7	35	35	414	591	35
Case	38	23	24	23	39	23

Pozicija S2

Tablica 12. Unutarnje sile - pozicija S2

S2 razmaknuti 2UPN160	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	158,09	27,69	29,48	1,68	31,52	32,44
Bar	701	2	701	485	2	2
Node	593	3	593	410	3	3
Case	25	33	31	35	39	33
MIN	-19,04	-34,09	-24,12	-1,87	-40,47	-47,6
Bar	39	2	2	486	701	2
Node	39	3	3	412	593	3
Case	10	23	39	25	31	23

Pozicija S3

Tablica 13. Unutarnje sile - pozicija S3

S3 IPN220	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	32,29	4,25	18,95	0,03	23,52	1,61
Bar	908	908	895	908	908	908
Node	1819	1819	1793	1820	1819	1819
Case	33	31	24	42	24	31
MIN	-7,8	-7,27	-18,95	-0,03	-26,31	-4,89
Bar	895	908	908	908	908	908
Node	1794	1819	1819	1820	1819	1819
Case	38	23	24	29	40	23

Pozicija P1

Tablica 14. Unutarnje sile - pozicija P1

P1 IPN180	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	41,4	3,34	5,17	0,01	5,5	1,77
Bar	760	762	763	763	763	763
Node	34	636	30	637	637	637
Case	38	35	31	13	42	42
MIN	-11,78	-3,35	-9,42	-0,01	-2,09	-1,75
Bar	763	759	759	763	759	762
Node	637	633	633	30	633	636
Case	42	13	13	25	13	35

Pozicija P2

Tablica 15. Unutarnje sile - pozicija P2

P2 IPN240	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	21,54	2,3	5	0,07	44,39	5,5
Bar	757	757	599	757	758	757
Node	627	624	488	624	624	624
Case	25	25	16	25	13	25
MIN	-13,61	-0,66	-9,7	-0,06	-4,21	-6,17
Bar	98	758	757	758	599	757
Node	70	624	627	624	488	627
Case	29	27	13	15	16	25

Pozicija P3

Tablica 16. Unutarnje sile - pozicija P3

P3 IPN180	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	14,57	0,8	4,52	0	0,9	2,07
Bar	1200	1201	1203	1202	1204	1201
Node	1906	1908	1912	1910	1914	1909
Case	23	31	30	24	37	23
MIN	-7,17	-1,72	-4,52	0	-1,13	-1,64
Bar	1204	1201	1199	1202	1204	1201
Node	1915	1909	1904	1910	1914	1908
Case	42	23	24	40	33	23

Pozicija R1

Tablica 17. Unutarnje sile - pozicija R1 - gornji pojas

R1-gornji pojas 2UPN160	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	49,31	4,63	5,42	0,22	0,83	3,26
Bar	707	706	544	706	42	6
Node	594	596	458	596	38	4
Case	31	25	16	15	38	23
MIN	-25,75	-2,2	-5,23	-0,25	-3,63	-9,02
Bar	42	5	543	707	42	706
Node	38	6	460	594	38	596
Case	38	23	13	27	13	25

Tablica 18. Unutarnje sile - pozicija R1 - donji pojas

R1-donji pojas 2UPN160	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	117,88	15,41	4,56	0,01	4,99	27,49
Bar	713	712	658	713	45	9
Node	602	599	554	602	47	11
Case	39	33	25	27	38	42
MIN	-52,82	-16,68	-3,52	-0,01	-7,07	-25,04
Bar	657	9	713	712	658	712
Node	553	11	600	601	554	599
Case	31	42	25	27	25	33

Tablica 19. Unutarnje sile - pozicija R1 - ispuna

R1-ispuna 2CAE70x7	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	57,56	4,13	9,61	1,15	0	0
Bar	67	677	733	732	735	735
Node	47	572	622	620	615	615
Case	19	37	39	15	11	27
MIN	-48,08	-4,13	-22,58	-1,23	0	0
Bar	681	675	346	730	737	737
Node	546	568	298	616	619	619
Case	19	37	24	15	27	39

Pozicija R2

Tablica 20. Unutarnje sile - pozicija R2 - gornji pojas

R2-gornji pojas 2UPN160	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	26,57	11,98	1,34	0	0,6	16,67
Bar	7	7	7	710	710	7
Node	6	6	4	596	594	6
Case	38	23	39	31	39	34
MIN	-59,54	-11,97	-1,59	0	-1,75	-18,53
Bar	325	7	710	7	710	7
Node	274	4	596	6	596	6
Case	35	29	25	31	25	37

Tablica 21. Unutarnje sile - pozicija R2 - donji pojas

R2-donji pojas 2UPN160	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	48,41	1,37	33,16	0	38,19	1,36
Bar	711	711	8	601	8	711
Node	600	599	11	511	11	599
Case	31	25	29	39	42	25
MIN	-54,14	-1,18	-33,21	0	-39,73	-0,54
Bar	8	8	8	711	8	711
Node	12	12	12	599	11	600
Case	25	25	23	25	29	39

Tablica 22. Unutarnje sile - pozicija R2 - ispuna

R2-ispuna 2CAE70x7	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	22,19	2,06	2,53	0	0	0
Bar	717	716	15	720	720	718
Node	605	608	6	594	594	596
Case	31	28	23	28	27	13
MIN	-26,79	-2,06	-2,52	0	0	0
Bar	718	714	14	718	718	720
Node	605	603	16	596	596	594
Case	31	34	23	28	42	39

Pozicija R3

Tablica 23. Unutarnje sile - pozicija R3 - gornji pojas

R3-gornji pojas 2CAE70x7	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	59,84	2,97	2,36	1,9	1,19	4,19
Bar	336	721	55	722	55	721
Node	276	609	38	609	38	609
Case	13	39	38	38	38	25
MIN	-15,74	-2,1	-3,61	-1,64	-1,85	-1,59
Bar	18	721	557	722	557	55
Node	6	596	458	594	458	55
Case	38	36	13	15	13	27

Tablica 24. Unutarnje sile - pozicija R3 - donji pojas

R3- donji pojas 2CAE100x10	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	36,98	6,64	0,99	0,09	0,14	5,08
Bar	339	724	725	724	21	724
Node	276	596	610	596	6	610
Case	15	25	25	13	38	25
MIN	-14,58	-5,96	-3,49	-0,04	-1,21	-14,16
Bar	57	724	339	725	339	724
Node	56	610	276	610	276	596
Case	38	42	13	27	13	25

Tablica 25. Unutarnje sile - pozicija R3 - ispuna 1

R3-ispuna 1 2CAE70x7	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	13,35	5,66	39,26	1,38	1,15	0,39
Bar	338	727	283	231	506	59
Node	290	613	242	195	427	57
Case	15	25	28	11	38	28
MIN	-17,96	-5	-39,45	-1,6	-4,75	-0,26
Bar	672	122	228	506	506	59
Node	565	109	193	427	427	57
Case	33	29	28	23	13	37

Tablica 26. Unutarnje sile - pozicija R3 - ispuna 2

R3-ispuna 2 2CAE45x5	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	23,95	1,35	0,88	0	0	0
Bar	728	729	508	729	728	728
Node	610	611	429	611	610	610
Case	31	28	35	28	38	27
MIN	-8,13	-1,32	-0,88	0	0	0
Bar	61	728	509	509	728	729
Node	59	613	427	427	610	611
Case	38	28	35	23	25	37

Pozicija R4

Tablica 27. Unutarnje sile - pozicija R4 - donji pojas

R4-gornji pojas 2CAE120x10	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	30,11	9,86	3,65	0,05	2,6	21,97
Bar	80	745	582	747	580	747
Node	38	596	458	594	447	594
Case	38	25	19	13	15	27
MIN	-23,95	-7,89	-4,87	-0,05	-1,56	-24,68
Bar	582	747	580	745	747	745
Node	448	594	447	596	594	596
Case	41	27	15	13	39	25

Tablica 28. Unutarnje sile - pozicija R4 - donji pojas

R4-donji pojas 2CAE120x10	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	89,6	3,53	1,54	0	3,26	14,5
Bar	79	81	81	746	748	746
Node	46	45	45	586	599	600
Case	29	29	29	25	42	42
MIN	-114,27	-3,54	-1,54	0	-2,16	-16,51
Bar	748	79	79	693	81	79
Node	599	12	46	511	45	12
Case	27	37	23	39	23	37

Tablica 29. Unutarnje sile - pozicija R4 - ispuna

R4-ispuna 2CAE70x7	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	42,57	7,53	3,23	0	0	0
Bar	751	751	751	479	754	148
Node	585	625	625	395	627	123
Case	25	27	31	29	39	27
MIN	-12,49	-7,53	-1,96	0	0	0
Bar	83	750	699	699	149	643
Node	71	625	579	579	123	539
Case	38	27	29	23	11	15

Pozicija R5

Tablica 30. Unutarnje sile - pozicija R5 - gornji pojas

R5-gornji pojas 2UPN80	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	26,44	0,23	4,19	0	1,34	2,35
Bar	73	73	575	73	73	740
Node	21	55	451	21	55	609
Case	11	31	13	39	38	31
MIN	-23,36	-1,82	-2,43	0	-1,86	-0,36
Bar	520	740	575	73	73	465
Node	403	587	473	21	55	377
Case	28	39	16	34	27	38

Tablica 31. Unutarnje sile - pozicija R5 - donji pojas

R5-donji pojas 2CAE100x10	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	23,43	0,46	5,25	0	2,77	1,37
Bar	136	686	741	741	74	686
Node	106	522	588	588	22	522
Case	23	39	13	25	28	39
MIN	-40,12	-0,12	-4,96	0	-4,04	-1,38
Bar	576	741	741	136	741	686
Node	474	610	610	84	588	562
Case	28	31	13	25	13	39

Tablica 32. Unutarnje sile - pozicija R5 - ispuna

R5-ispuna 2CAE70x7	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	18,93	6,09	5,59	0	0	0
Bar	522	742	743	193	194	137
Node	439	624	587	159	145	120
Case	15	39	27	29	16	39
MIN	-47,7	-1,82	-5,59	0	0	0
Bar	359	744	744	744	139	742
Node	289	609	609	609	105	624
Case	15	39	27	25	17	25

Pozicija V1

Tablica 33. Unutarnje sile - pozicija V1

V1 ϕ17	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	0	N/A	N/A	N/A	N/A	N/A
Bar	993	956	956	956	956	956
Node	277	35	35	35	35	35
Case	41	10	10	10	10	10
MIN	-56,83	N/A	N/A	N/A	N/A	N/A
Bar	957	956	956	956	956	956
Node	1	35	35	35	35	35
Case	37	10	10	10	10	10

Pozicija V2

Tablica 34. Unutarnje sile - pozicija V2

V2 ϕ28	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	-0,02	N/A	N/A	N/A	N/A	N/A
Bar	1011	1008	1008	1008	1008	1008
Node	3	5	5	5	5	5
Case	35	10	10	10	10	10
MIN	-202,8	N/A	N/A	N/A	N/A	N/A
Bar	1013	1008	1008	1008	1008	1008
Node	595	5	5	5	5	5
Case	31	10	10	10	10	10

Pozicija V3

Tablica 35. Unutarnje sile - pozicija V3

V3 ϕ15	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	0	N/A	N/A	N/A	N/A	N/A
Bar	1195	1018	1018	1018	1018	1018
Node	274	550	550	550	550	550
Case	37	10	10	10	10	10
MIN	-40,79	N/A	N/A	N/A	N/A	N/A
Bar	1185	1018	1018	1018	1018	1018
Node	30	550	550	550	550	550
Case	38	10	10	10	10	10

Pozicija E1

Tablica 36. Unutarnje sile - pozicija E1

E1 TCAR100x10	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	36,61	4,97	1,41	0,31	0,75	13,31
Bar	894	882	877	882	881	894
Node	47	601	370	601	602	13
Case	37	27	23	31	37	33
MIN	-22,34	-4,68	-1,41	-0,31	-1,7	-10,64
Bar	882	894	886	894	877	894
Node	601	13	369	47	370	13
Case	27	33	29	40	23	37

Pozicija E2

Tablica 37. Unutarnje sile - pozicija E2

E2 IPN180	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	21,82	0,25	7,22	0	7,58	0,35
Bar	943	955	955	955	943	942
Node	1820	1870	1870	1870	1820	1883
Case	33	33	15	33	38	27
MIN	-1,57	-0,19	-3,35	0	-11,68	-0,46
Bar	930	942	943	942	943	955
Node	1794	1818	1820	1818	1820	1896
Case	38	28	38	28	25	33

Pozicija E3

Tablica 38. Unutarnje sile - pozicija E3

E3 IPE180	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	2,32	4,77	6,45	0	1,11	4,71
Bar	92	594	594	593	594	593
Node	23	453	453	477	453	477
Case	23	13	16	16	38	17
MIN	-2,03	-4,29	-6,45	0	-4,57	-2,12
Bar	92	593	593	91	594	92
Node	23	454	454	59	453	23
Case	33	36	22	38	16	38

6.4.2. Vrijednosti pomaka

Pozicija R1

Tablica 39. Pomaci za poziciju R1

R1	UX(mm)	UY(mm)	UZ(mm)
MAX	13	35	0
Bar	2090	2090	2077
Case	63	59	70
MIN	-9	-3	-1
Bar	2092	2092	2077
Case	57	59	45

Pozicija R2

Tablica 40. Pomaci za poziciju R2

R2	UX(mm)	UY(mm)	UZ(mm)
MAX	13	32	0
Bar	603	603	49
Case	63	74	70
MIN	-9	-2	-1
Bar	49	99	555
Case	57	59	45

Pozicija R3

Tablica 41. Pomaci za poziciju R3

R3	UX(mm)	UY(mm)	UZ(mm)
MAX	13	20	0
Bar	610	22	22
Case	63	55	70
MIN	-12	-6	-26
Bar	22	426	623
Case	71	45	45

Pozicija R4

Tablica 42. Pomaci za poziciju R4

R4	UX(mm)	UY(mm)	UZ(mm)
MAX	19	25	1
Bar	628	72	72
Case	25	23	6
MIN	-16	-3	-2
Bar	72	72	626
Case	25	33	27

Pozicija R5

Tablica 43. Pomaci za poziciju R5

R5	UX(mm)	UY(mm)	UZ(mm)
MAX	17	30	5
Bar	623	119	623
Case	31	23	6
MIN	-15	-8	-37
Bar	69	439	623
Case	39	13	13

Pozicija P1

Tablica 44. Pomaci za poziciju P1

P1	UX(mm)	UY(mm)	UZ(mm)
MAX	0	35	2
Bar	763	762	764
Case	57	48	57
MIN	0	-34	-8
Bar	760	759	762
Case	70	48	47

Pozicija P2

Tablica 45. Pomaci za poziciju P2

P2	UX(mm)	UY(mm)	UZ(mm)
MAX	0	3	0
Bar	97	758	97
Case	70	59	70
MIN	0	-5	-6
Bar	599	757	758
Case	47	57	45

Pozicija P3

Tablica 46. Pomaci za poziciju P3

P3	UX(mm)	UY(mm)	UZ(mm)
MAX	0	3	0
Bar	97	758	97
Case	70	59	70
MIN	0	-5	-6
Bar	599	757	758
Case	47	57	45

Pozicija S1

Tablica 47. Pomaci za poziciju S1

S1	UX(mm)	UY(mm)	UZ(mm)
MAX	13	9	0
Node	592	2	8
Case	63	61	70
MIN	-12	-1	-1
Node	8	2	598
Case	71	57	57

Pozicija S2

Tablica 48. Pomaci za poziciju S2

S2	UX(mm)	UY(mm)	UZ(mm)
MAX	13	17	0
Node	594	4	6
Case	63	55	70
MIN	-12	-2	-1
Node	4	38	6
Case	57	73	57

Pozicija S3

Tablica 49. Pomaci za poziciju S3

S3	UX(mm)	UY(mm)	UZ(mm)
MAX	12	9	0
Node	1870	1820	1794
Case	63	61	70
MIN	-10	-1	0
Node	1794	1820	1820
Case	57	57	65

7. DIMENZIONIRANJE ELEMENATA

Provjere GSN i GSU provode se u programskom paketu *Autodesk Robot Structural Analysis Professional 2019.* [1], a prema [9] i [10].

7.1. Provjera GSN

Pozicija S1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 10
MEMBER: 99 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

LOADS:
Governing Load Case: 23 COMB13 (1+2)*1.35+5*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 UPN 180

h=180 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=3080 mm2	Az=2880 mm2	Ax=5565 mm2
tw=8 mm	Iy=27072200 mm4	Iz=4341486 mm4	Ix=174820 mm4
tf=11 mm	Wply=358158 mm3	Wplz=107350 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 61.49 kN	My,Ed = 0.33 kN*m	Mz,Ed = -10.11 kN*m	Vy,Ed = -9.23 kN
Nc,Rd = 1975.67 kN	My,Ed,max = 0.33 kN*m	Mz,Ed,max = -10.11 kN*m	Vy,T,Rd = 631.27 kN
Nb,Rd = 1452.57 kN	My,c,Rd = 127.15 kN*m	Mz,c,Rd = 38.11 kN*m	Vz,Ed = -0.12 kN
	MN,y,Rd = 127.02 kN*m	MN,z,Rd = 38.07 kN*m	Vz,T,Rd = 590.28 kN
			Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 7.20 m	Lam_y = 0.27	Lz = 7.20 m	Lam_z = 0.68
Lcr,y = 1.44 m	Xy = 0.96	Lcr,z = 1.44 m	Xz = 0.74
Lamy = 20.65	kzy = 0.54	Lamz = 51.56	kzz = 0.93

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=5.90 m	fi,T=1.08	Ncr,y=26415.09 kN	fi,TF=1.08
Ncr,T=2441.96 kN	X,T=0.60	Ncr,TF=2441.96 kN	X,TF=0.60
Lam_T=0.90	Nb,T,Rd=1185.71 kN	Lam_TF=0.90	Nb,TF,Rd=1185.71 kN

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.03 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.27 < 1.00 (6.2.9.1.(6))

Vy,Ed/Vy,c,Rd = 0.01 < 1.00   (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00   (6.2.6.(1))

Global stability check of member:
Lambda,y = 20.65 < Lambda,max = 210.00   Lambda,z = 51.56 < Lambda,max = 210.00   STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.05 < 1.00   (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.18 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.29 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija S2

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 9
MEMBER: 701   **POINT:** 1   **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355)   fy = 355.00 MPa

SECTION PARAMETERS: 2 UPN 160

h=160 mm	gM0=1.00	gM1=1.00	
b=230 mm	Ay=2730 mm2	Az=2400 mm2	Ax=4779 mm2
tw=8 mm	Iy=18490920 mm4	Iz=24060351 mm4	Ix=136580 mm4
tf=11 mm	Wply=275020 mm3	Wplz=326876 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 158.09 kN	My,Ed = -40.47 kN*m	Mz,Ed = 0.73 kN*m	Vy,Ed = 0.17 kN
Nc,Rd = 1696.47 kN	My,Ed,max = -40.47 kN*m		Mz,Ed,max = 0.73 kN*m
	Vy,T,Rd = 559.54 kN		
Nb,Rd = 542.36 kN	My,c,Rd = 97.63 kN*m	Mz,c,Rd = 116.04 kN*m	Vz,Ed = 29.48 kN
	MN,y,Rd = 96.78 kN*m	MN,z,Rd = 115.03 kN*m	Vz,T,Rd = 491.90 kN
			Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 9.54 m	Lam_y = 1.48	Lz = 9.54 m	Lam_z = 1.30
Lcr,y = 6.97 m	Xy = 0.32	Lcr,z = 6.97 m	Xz = 0.39
Lamy = 112.05	kyy = 1.11	Lamz = 98.23	kyz = 0.72

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=6.97 m	fi,T=1.44	Ncr,y=770.10 kN	fi,TF=1.44
Ncr,T=1209.41 kN	X,T=0.44	Ncr,TF=1209.41 kN	X,TF=0.44
Lam_T=1.18	Nb,T,Rd=748.65 kN	Lam_TF=1.18	Nb,TF,Rd=748.65 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.09 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.42 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.06 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 112.05 < Lambda,max = 210.00 Lambda,z = 98.23 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.29 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.76 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.52 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija S3

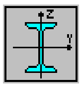
STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 17
MEMBER: 908 **POINT:** 3 **COORDINATE:** x = 1.00 L = 7.20 m

LOADS:
Governing Load Case: 40 COMB30 (1+2)*1.35+8*1.50

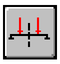
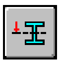
MATERIAL:
S355 (S355) fy = 355.00 MPa


SECTION PARAMETERS: IPN 220

h=220 mm	gM0=1.00	gM1=1.00	
b=98 mm	Ay=2499 mm2	Az=1857 mm2	Ax=3952 mm2
tw=8 mm	Iy=30535000 mm4	Iz=1622060 mm4	Ix=192000 mm4
tf=12 mm	Wply=322771 mm3	Wplz=55738 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 9.57 kN	My,Ed = -5.16 kN*m	Mz,Ed = 0.17 kN*m	Vy,Ed = -0.27 kN
Nc,Rd = 1402.87 kN	My,Ed,max = -26.31 kN*m		Mz,Ed,max = -0.45 kN*m
	Vy,T,Rd = 510.02 kN		
Nb,Rd = 890.09 kN	My,c,Rd = 114.58 kN*m	Mz,c,Rd = 19.79 kN*m	Vz,Ed = -12.01 kN
	MN,y,Rd = 114.58 kN*m	MN,z,Rd = 19.79 kN*m	Vz,T,Rd = 379.58 kN
	Mb,Rd = 33.92 kN*m		Tt,Ed = 0.03 kN*m
			Class of section = 1

  **LATERAL BUCKLING PARAMETERS:**

z = 0.00	Mcr = 35.67 kN*m	Curve,LT - c	XLT = 0.30
Lcr,low=7.20 m	Lam_LT = 1.79	fi,LT = 2.05	XLT,mod = 0.30

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 7.20 m	Lam_y = 0.22	Lz = 7.20 m	Lam_z = 0.94
Lcr,y = 1.44 m	Xy = 1.00	Lcr,z = 1.44 m	Xz = 0.63
Lamy = 16.38	kzy = 1.00	Lamz = 71.08	kzz = 0.91

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34
Lt=7.20 m	fi,T=0.97	Ncr,y=29793.84 kN	fi,TF=0.97
Ncr,T=1944.33 kN	X,T=0.69	Ncr,TF=1944.33 kN	X,TF=0.69
Lam_T=0.85	Nb,T,Rd=972.90 kN	Lam_TF=0.85	Nb,TF,Rd=972.90 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.01 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.03 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)

Global stability check of member:

Lambda,y = 16.38 < Lambda,max = 210.00 Lambda,z = 71.08 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.01 < 1.00 (6.3.1)
My,Ed,max/Mb,Rd = 0.78 < 1.00 (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.72 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.81 < 1.00 (6.3.3.(4))

Section OK !!

Pozicija P1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 14
MEMBER: 762 **POINT:** 1 **COORDINATE:** x = 0.92 L = 72.00 m

---

LOADS:
Governing Load Case: 19 COMB9 (1+2)*1.35+4*1.50+7*0.90

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: IPN 180

h=180 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1784 mm2	Az=1297 mm2	Ax=2787 mm2

tw=7 mm	Iy=14434400 mm4	Iz=812871 mm4	Ix=98000 mm4
tf=10 mm	Wply=186578 mm3	Wplz=33318 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 0.75 kN	My,Ed = -13.96 kN*m	Mz,Ed = -4.54 kN*m	Vy,Ed = -4.27 kN
Nc,Rd = 989.43 kN	My,Ed,max = -13.96 kN*m		Mz,Ed,max = -4.54 kN*m
	Vy,T,Rd = 365.41 kN		
Nb,Rd = 146.75 kN	My,c,Rd = 66.24 kN*m	Mz,c,Rd = 11.83 kN*m	Vz,Ed = 13.40 kN
	MN,y,Rd = 66.24 kN*m	MN,z,Rd = 11.83 kN*m	Vz,T,Rd = 265.69 kN
	Mb,Rd = 34.37 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 43.80 kN*m	Curve,LT - c	XLT = 0.51
Lcr,low=3.12 m	Lam_LT = 1.23	fi,LT = 1.27	XLT,mod = 0.52

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 78.00 m	Lam_y = 0.57	Lz = 78.00 m	Lam_z = 2.42
Lcr,y = 3.12 m	Xy = 0.90	Lcr,z = 3.12 m	Xz = 0.15
Lamy = 43.35	kzy = 1.00	Lamz = 182.69	kzz = 0.91

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.00 < 1.00   (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.43 < 1.00   (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.01 < 1.00   (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.05 < 1.00   (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)

Global stability check of member:

Lambda,y = 43.35 < Lambda,max = 210.00   Lambda,z = 182.69 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.41 < 1.00   (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.58 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.76 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija P2

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 21
MEMBER: 757   **POINT:** 1   **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: IPN 240

h=240 mm	gM0=1.00	gM1=1.00	
b=106 mm	Ay=2901 mm2	Az=2175 mm2	Ax=4610 mm2
tw=9 mm	Iy=42374500 mm4	Iz=2202600 mm4	Ix=257000 mm4
tf=13 mm	Wply=410517 mm3	Wplz=70008 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 20.04 kN	My,Ed = 42.53 kN*m	Mz,Ed = 5.50 kN*m	Vy,Ed = 2.30 kN
Nc,Rd = 1636.55 kN	My,Ed,max = 42.53 kN*m	Mz,Ed,max = 5.50 kN*m	Vy,T,Rd = 590.61 kN
Nb,Rd = 608.29 kN	My,c,Rd = 145.73 kN*m	Mz,c,Rd = 24.85 kN*m	Vz,Ed = -7.42 kN
	MN,y,Rd = 145.73 kN*m	MN,z,Rd = 24.85 kN*m	Vz,T,Rd = 443.72 kN
	Mb,Rd = 104.49 kN*m		Tt,Ed = 0.07 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 177.24 kN*m	Curve,LT - c	XLT = 0.70
Lcr,upp=2.35 m	Lam_LT = 0.91	fi,LT = 0.93	XLT,mod = 0.72

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 5.08 m	Lam_y = 0.32	Lz = 5.08 m	Lam_z = 1.42
Lcr,y = 2.35 m	Xy = 0.97	Lcr,z = 2.35 m	Xz = 0.37
Lamy = 24.51	kzy = 0.99	Lamz = 107.51	kzz = 0.94

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.31 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.02 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)

Global stability check of member:

Lambda,y = 24.51 < Lambda,max = 210.00 Lambda,z = 107.51 < Lambda,max = 210.00 STABLE
My,Ed,max/Mb,Rd = 0.41 < 1.00 (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.50 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.65 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija P3

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 19
MEMBER: 1204 **POINT:** 3 **COORDINATE:** x = 0.08 L = 6.00 m

----

LOADS:
Governing Load Case: 24 COMB14 (1+2)*1.35+6*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: IPN 180

h=180 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1784 mm2	Az=1297 mm2	Ax=2787 mm2
tw=7 mm	Iy=14434400 mm4	Iz=812871 mm4	Ix=98000 mm4
tf=10 mm	Wply=186578 mm3	Wplz=33318 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 11.07 kN	My,Ed = -7.29 kN*m	Mz,Ed = 0.37 kN*m	Vy,Ed = -0.59 kN
Nc,Rd = 989.43 kN	My,Ed,max = -7.29 kN*m	Mz,Ed,max = 0.37 kN*m	Vy,T,Rd = 365.51 kN
Nb,Rd = 146.75 kN	My,c,Rd = 66.24 kN*m	Mz,c,Rd = 11.83 kN*m	Vz,Ed = -5.33 kN
	MN,y,Rd = 66.24 kN*m	MN,z,Rd = 11.83 kN*m	Vz,T,Rd = 265.74 kN
	Mb,Rd = 19.71 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 20.76 kN*m	Curve,LT - c	XLT = 0.30
Lcr,low=6.24 m	Lam_LT = 1.79	fi,LT = 2.04	XLT,mod = 0.30

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 78.00 m	Lam_y = 0.57	Lz = 78.00 m	Lam_z = 2.42
Lcr,y = 3.12 m	Xy = 0.90	Lcr,z = 3.12 m	Xz = 0.15
Lamy = 43.35	kzy = 0.99	Lamz = 182.69	kzz = 1.00

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.04 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
Lambda,y = 43.35 < Lambda,max = 210.00 Lambda,z = 182.69 < Lambda,max = 210.00 STABLE
My,Ed,max/Mb,Rd = 0.37 < 1.00 (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.37 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.47 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R1 - gornji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 8
MEMBER: 706 **POINT:** 2 **COORDINATE:** x = 0.83 L = 6.14 m

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 UPN 160 spojeni

h=160 mm	gM0=1.00	gM1=1.00	
b=130 mm	Ay=2730 mm2	Az=2400 mm2	Ax=4779 mm2
tw=8 mm	Iy=18490920 mm4	Iz=3319786 mm4	Ix=136580 mm4
tf=11 mm	Wply=275020 mm3	Wplz=87936 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 32.08 kN	My,Ed = 0.14 kN*m	Mz,Ed = -3.29 kN*m	Vy,Ed = 4.63 kN
Nc,Rd = 1696.47 kN	My,Ed,max = -0.82 kN*m	Mz,Ed,max = -9.02 kN*m	Vy,T,Rd = 559.54 kN
Nb,Rd = 723.21 kN	My,c,Rd = 97.63 kN*m	Mz,c,Rd = 31.22 kN*m	Vz,Ed = -0.48 kN
	MN,y,Rd = 97.60 kN*m	MN,z,Rd = 31.21 kN*m	Vz,T,Rd = 491.90 kN
			Tt,Ed = 0.14 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 7.38 m	Lam_y = 0.52	Lz = 7.38 m	Lam_z = 1.22
Lcr,y = 2.42 m	Xy = 0.83	Lcr,z = 2.42 m	Xz = 0.43
Lamy = 38.89	kzy = 0.54	Lamz = 91.79	kzz = 0.96

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=2.42 m	fi,T=1.02	Ncr,y=6392.60 kN	fi,TF=1.02
Ncr,T=2359.48 kN	X,T=0.63	Ncr,TF=2359.48 kN	X,TF=0.63
Lam_T=0.85	Nb,T,Rd=1072.36 kN	Lam_TF=0.85	Nb,TF,Rd=1072.36 kN

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.02 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.11 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.01 < 1.00 (6.2.6.(1))

Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Global stability check of member:
Lambda,y = 38.89 < Lambda,max = 210.00 Lambda,z = 91.79 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.04 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.20 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.33 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R1 - donji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 7
MEMBER: 9 **POINT:** 3 **COORDINATE:** x = 1.00 L = 7.08 m

LOADS:
Governing Load Case: 27 COMB17 (1+2)*1.35+9*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 UPN 160 spojeni

h=160 mm	gM0=1.00	gM1=1.00	
b=130 mm	Ay=2730 mm2	Az=2400 mm2	Ax=4779 mm2
tw=8 mm	Iy=18490920 mm4	Iz=3319786 mm4	Ix=136580 mm4
tf=11 mm	Wply=275020 mm3	Wplz=87936 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 11.25 kN	My,Ed = -1.77 kN*m	Mz,Ed = 27.41 kN*m	Vy,Ed = -16.65 kN
Nc,Rd = 1696.47 kN	My,Ed,max = -2.22 kN*m	Mz,Ed,max = 27.41 kN*m	Vy,T,Rd = 559.54 kN
Nb,Rd = 529.18 kN	My,c,Rd = 97.63 kN*m	Mz,c,Rd = 31.22 kN*m	Vz,Ed = -1.62 kN
	MN,y,Rd = 97.63 kN*m	MN,z,Rd = 31.22 kN*m	Vz,T,Rd = 491.90 kN
			Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 7.08 m	Lam_y = 1.51	Lz = 7.08 m	Lam_z = 1.19
Lcr,y = 7.08 m	Xy = 0.31	Lcr,z = 2.37 m	Xz = 0.44
Lamy = 113.85	kzy = 0.55	Lamz = 90.00	kzz = 0.92

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=7.08 m	fi,T=1.02	Ncr,y=745.92 kN	fi,TF=1.02
Ncr,T=2359.48 kN	X,T=0.63	Ncr,TF=2359.48 kN	X,TF=0.63

Lam_T=0.85 Nb,T,Rd=1072.36 kN Lam_TF=0.85 Nb,TF,Rd=1072.36 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.90 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.03 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 113.85 < Lambda,max = 210.00 Lambda,z = 90.00 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.02 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.53 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.83 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R1 - ispuna

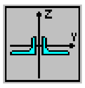
STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 4
MEMBER: 737 **POINT:** 2 **COORDINATE:** x = 0.50 L = 1.88 m

LOADS:
Governing Load Case: 28 COMB18 (1+2)*1.35+10*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa



SECTION PARAMETERS: 2 CAE 70x7

h=70 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=980 mm2	Az=882 mm2	Ax=1879 mm2
tw=7 mm	Iy=846000 mm4	Iz=1575369 mm4	Ix=30412 mm4
tf=7 mm	Wely=16819 mm3	Welz=22505 mm3	
	Weff,y=16819 mm3	Weff,z=22505 mm3	

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = -18.02 kN	My,Ed = -3.71 kN*m	Mz,Ed = 0.38 kN*m	Vy,Ed = 0.01 kN
Nt,Rd = 667.18 kN	My,el,Rd = 5.97 kN*m	Mz,el,Rd = 7.99 kN*m	Vy,c,Rd = 200.86 kN
	My,c,Rd = 5.97 kN*m	Mz,c,Rd = 7.99 kN*m	Vz,Ed = 0.15 kN
			Vz,c,Rd = 180.77 kN
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nt,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.70 < 1.00   (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*Tau,y,Ed^2)/(fy/gM0) = 0.69 < 1.00   (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00   (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00   (6.2.6.(1))

Section OK !!!

Pozicija R2 - gornji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 12
MEMBER: 7   **POINT:** 3   **COORDINATE:** x = 1.00 L = 8.00 m

---

LOADS:
Governing Load Case: 37 COMB27 (1+2)*1.35+5*1.50

MATERIAL:
S355 (S355)   fy = 355.00 MPa

SECTION PARAMETERS: 2 UPN 160 spojeni

h=160 mm	gM0=1.00	gM1=1.00	
b=130 mm	Ay=2730 mm2	Az=2400 mm2	Ax=4779 mm2
tw=8 mm	Iy=18490920 mm4	Iz=3319786 mm4	Ix=136580 mm4
tf=11 mm	Wply=275020 mm3	Wplz=87936 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 11.59 kN	My,Ed = -0.55 kN*m	Mz,Ed = -18.53 kN*m	Vy,Ed = 11.98 kN
Nc,Rd = 1696.47 kN	My,Ed,max = -0.55 kN*m	Mz,Ed,max = -18.53 kN*m	Vy,T,Rd = 559.54 kN
Nb,Rd = 435.72 kN	My,c,Rd = 97.63 kN*m	Mz,c,Rd = 31.22 kN*m	Vz,Ed = -0.88 kN
	MN,y,Rd = 97.63 kN*m	MN,z,Rd = 31.22 kN*m	Vz,T,Rd = 491.90 kN
			Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 8.00 m	Lam_y = 1.70	Lz = 8.00 m	Lam_z = 1.01
Lcr,y = 8.00 m	Xy = 0.26	Lcr,z = 2.00 m	Xz = 0.54
Lamy = 128.61	kzy = 0.55	Lamz = 75.88	kzz = 0.92

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=8.00 m	fi,T=1.02	Ncr,y=584.56 kN	fi,TF=1.02
Ncr,T=2359.48 kN	X,T=0.63	Ncr,TF=2359.48 kN	X,TF=0.63
Lam_T=0.85	Nb,T,Rd=1072.36 kN	Lam_TF=0.85	Nb,TF,Rd=1072.36 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.01 < 1.00   (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.60 < 1.00   (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.02 < 1.00   (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00   (6.2.6.(1))

Global stability check of member:

Lambda,y = 128.61 < Lambda,max = 210.00   Lambda,z = 75.88 < Lambda,max = 210.00   STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.03 < 1.00   (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.36 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.56 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R2 - donji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 28
MEMBER: 8   **POINT:** 1   **COORDINATE:** x = 0.00 L = 0.00 m

----

LOADS:
Governing Load Case: 29 COMB19 (1+2)*1.35+5*1.50+4*0.75

MATERIAL:
S355 (S355)   fy = 355.00 MPa

SECTION PARAMETERS: 2 UPN 160 spojeni

h=160 mm	gM0=1.00	gM1=1.00	
b=130 mm	Ay=2730 mm2	Az=2400 mm2	Ax=4779 mm2
tw=8 mm	Iy=18490920 mm4	Iz=3319786 mm4	Ix=136580 mm4
tf=11 mm	Wply=275020 mm3	Wplz=87936 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 9.89 kN	My,Ed = -39.73 kN*m	Mz,Ed = 0.28 kN*m	Vy,Ed = 0.67 kN
Nc,Rd = 1696.47 kN	My,Ed,max = -39.73 kN*m		Mz,Ed,max = 0.28 kN*m
	Vy,T,Rd = 559.54 kN		
Nb,Rd = 435.72 kN	My,c,Rd = 97.63 kN*m	Mz,c,Rd = 31.22 kN*m	Vz,Ed = 33.16 kN
	MN,y,Rd = 97.63 kN*m	MN,z,Rd = 31.22 kN*m	Vz,T,Rd = 491.90 kN
			Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 8.00 m	Lam_y = 1.70	Lz = 8.00 m	Lam_z = 1.01
Lcr,y = 8.00 m	Xy = 0.26	Lcr,z = 2.00 m	Xz = 0.54

Lamy = 128.61	kyy = 0.92	Lamz = 75.88	kyz = 0.55

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=8.00 m	fi,T=1.02	Ncr,y=584.56 kN	fi,TF=1.02
Ncr,T=2359.48 kN	X,T=0.63	Ncr,TF=2359.48 kN	X,TF=0.63
Lam_T=0.85	Nb,T,Rd=1072.36 kN	Lam_TF=0.85	Nb,TF,Rd=1072.36 kN

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.42 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.07 < 1.00 (6.2.6.(1))
Global stability check of member:
Lambda,y = 128.61 < Lambda,max = 210.00   Lambda,z = 75.88 < Lambda,max = 210.00   STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.02 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.40 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.24 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R2 - ispuna

STEEL DESIGN

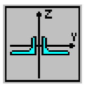
CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 25
MEMBER: 15   **POINT:** 2   **COORDINATE:** x = 0.50 L = 1.62 m

---

LOADS:
Governing Load Case: 29 COMB19 (1+2)*1.35+5*1.50+4*0.75

MATERIAL:
S355 (S355)   fy = 355.00 MPa



SECTION PARAMETERS: 2 CAE 70x7

h=70 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=980 mm2	Az=882 mm2	Ax=1879 mm2
tw=7 mm	Iy=846000 mm4	Iz=1575369 mm4	Ix=30412 mm4
tf=7 mm	Wely=16819 mm3	Welz=22505 mm3	
	Weff,y=16819 mm3	Weff,z=22505 mm3	

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = -8.97 kN	My,Ed = -2.66 kN*m	Mz,Ed = 0.26 kN*m	Vy,Ed = -0.00 kN
Nt,Rd = 667.18 kN	My,el,Rd = 5.97 kN*m	Mz,el,Rd = 7.99 kN*m	Vy,c,Rd = 200.86 kN
	My,c,Rd = 5.97 kN*m	Mz,c,Rd = 7.99 kN*m	Vz,Ed = -0.06 kN
			Vz,c,Rd = 180.77 kN

Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nt,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.49 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*Tau,y,Ed^2)/(fy/gM0) = 0.49 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Section OK !!!

Pozicija R3 - Gornji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

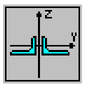
CODE GROUP: 5
MEMBER: 557 **POINT:** 1 **COORDINATE:** x = 0.46 L = 2.35 m

LOADS:

Governing Load Case: 19 COMB9 (1+2)*1.35+4*1.50+7*0.90

MATERIAL:

S355 (S355) fy = 355.00 MPa



SECTION PARAMETERS: 2 CAE 70x7

h=70 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=980 mm2	Az=882 mm2	Ax=1879 mm2
tw=7 mm	Iy=846000 mm4	Iz=1575369 mm4	Ix=30412 mm4
tf=7 mm	Wely=16819 mm3	Welz=22505 mm3	
	Weff,y=16819 mm3	Weff,z=22505 mm3	Aeff=1879 mm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = 25.33 kN	My,Ed = -1.44 kN*m	Mz,Ed = -3.36 kN*m	Vy,Ed = -1.97 kN
Nc,Rd = 667.18 kN	My,Ed,max = -1.44 kN*m	Mz,Ed,max = -3.36 kN*m	Vy,T,Rd = 200.86 kN
Nb,Rd = 184.70 kN	My,c,Rd = 5.97 kN*m	Mz,c,Rd = 7.99 kN*m	Vz,Ed = 3.84 kN
			Vz,T,Rd = 180.77 kN
			Tt,Ed = -0.00 kN*m
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 5.08 m	Lam_y = 1.70	Lz = 5.08 m	Lam_z = 1.25
Lcr,y = 2.73 m	Xy = 0.28	Lcr,z = 2.73 m	Xz = 0.45
Lamy = 128.67	kyy = 1.03	Lamz = 94.29	kyz = 0.96

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34
Lt=2.73 m	fi,T=0.79	Ncr,y=427.67 kN	fi,TF=1.51
Ncr,T=1546.19 kN	X,T=0.81	Ncr,TF=403.55 kN	X,TF=0.43
Lam_T=0.66	Nb,T,Rd=538.74 kN	Lam_TF=1.29	Nb,TF,Rd=289.42 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.62 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.67 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.01 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.02 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 128.67 < Lambda,max = 210.00 Lambda,z = 94.29 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.14 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.64 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.62 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R3 - donji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 27
MEMBER: 724 **POINT:** 3 **COORDINATE:** x = 1.00 L = 4.23 m

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: **2 CAE 100x10**

h=100 mm	gM0=1.00	gM1=1.00	
b=200 mm	Ay=2000 mm2	Az=1800 mm2	Ax=3831 mm2
tw=10 mm	Iy=3534000 mm4	Iz=6580485 mm4	Ix=126666 mm4
tf=10 mm	Wely=49220 mm3	Welz=65805 mm3	
	Weff,y=49220 mm3	Weff,z=65805 mm3	Aeff=3831 mm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

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INTERNAL FORCES AND CAPACITIES:

N,Ed = 31.17 kN	My,Ed = -0.44 kN*m	Mz,Ed = -14.16 kN*m	Vy,Ed = 6.64 kN
Nc,Rd = 1359.97 kN	My,Ed,max = -0.44 kN*m	Mz,Ed,max = -14.16 kN*m	Vy,T,Rd = 409.92 kN
Nb,Rd = 328.93 kN	My,c,Rd = 17.47 kN*m	Mz,c,Rd = 23.36 kN*m	Vz,Ed = -1.78 kN
			Vz,T,Rd = 368.93 kN
			Tt,Ed = 0.09 kN*m
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 4.23 m	Lam_y = 1.84	Lz = 4.23 m	Lam_z = 0.73
Lcr,y = 4.23 m	Xy = 0.24	Lcr,z = 2.28 m	Xz = 0.77
Lamy = 139.15	kyy = 0.99	Lamz = 54.93	kyz = 0.91

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34
Lt=4.23 m	fi,T=0.79	Ncr,y=2569.09 kN	fi,TF=0.95
Ncr,T=3142.11 kN	X,T=0.81	Ncr,TF=1988.58 kN	X,TF=0.71
Lam_T=0.66	Nb,T,Rd=1097.42 kN	Lam_TF=0.83	Nb,TF,Rd=962.40 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.64 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.64 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.02 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 139.15 < Lambda,max = 210.00 Lambda,z = 54.93 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.09 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.66 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.66 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R3 - ispuna 1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 23
MEMBER: 506 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

LOADS:
Governing Load Case: 13 COMB3 (1+2)*1.35+3*1.50+7*0.90

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 CAE 70x7

h=70 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=980 mm2	Az=882 mm2	Ax=1879 mm2
tw=7 mm	Iy=846000 mm4	Iz=1575369 mm4	Ix=30412 mm4
tf=7 mm	Wely=16819 mm3	Welz=22505 mm3	
	Weff,y=16819 mm3	Weff,z=22505 mm3	

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = -14.62 kN	My,Ed = -4.75 kN*m	Mz,Ed = -0.06 kN*m	Vy,Ed = -0.43 kN
Nt,Rd = 667.18 kN	My,el,Rd = 5.97 kN*m	Mz,el,Rd = 7.99 kN*m	Vy,T,Rd = 200.86 kN
	My,c,Rd = 5.97 kN*m	Mz,c,Rd = 7.99 kN*m	Vz,Ed = 0.34 kN
			Vz,T,Rd = 180.77 kN
			Tt,Ed = -0.71 kN*m
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nt,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.82 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*Tau,y,Ed^2)/(fy/gM0) = 0.82 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Section OK !!!

Pozicija R3 - spuna 2

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 26
MEMBER: 728 **POINT:** 2 **COORDINATE:** x = 0.50 L = 0.94 m

LOADS:

Governing Load Case: 33 COMB23 (1+2)*1.35+9*1.50+4*0.75

MATERIAL:

S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 CAE 45x5

h=45 mm	gM0=1.00	gM1=1.00	
b=90 mm	Ay=450 mm2	Az=405 mm2	Ax=861 mm2
tw=5 mm	Iy=156800 mm4	Iz=297787 mm4	Ix=7083 mm4
tf=5 mm	Wely=4870 mm3	Welz=6617 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 16.04 kN	My,Ed = 0.09 kN*m	Mz,Ed = 0.76 kN*m	Vy,Ed = 0.13 kN
Nc,Rd = 305.48 kN	My,Ed,max = 0.09 kN*m	Mz,Ed,max = 0.76 kN*m	Vy,c,Rd = 92.23 kN
Nb,Rd = 73.95 kN	My,c,Rd = 1.73 kN*m	Mz,c,Rd = 2.35 kN*m	Vz,Ed = -0.01 kN
			Vz,c,Rd = 83.01 kN
			Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.88 m	Lam_y = 1.84	Lz = 1.88 m	Lam_z = 1.34
Lcr,y = 1.88 m	Xy = 0.24	Lcr,z = 1.88 m	Xz = 0.41
Lamy = 139.09	kzy = 0.89	Lamz = 100.93	kzz = 0.99

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34
Lt=1.88 m	fi,T=0.74	Ncr,y=170.93 kN	fi,TF=1.63
Ncr,T=880.41 kN	X,T=0.84	Ncr,TF=164.60 kN	X,TF=0.40
Lam_T=0.59	Nb,T,Rd=257.37 kN	Lam_TF=1.36	Nb,TF,Rd=121.59 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.36 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.42 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 139.09 < Lambda,max = 210.00 Lambda,z = 100.93 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.22 < 1.00 (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.52 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.52 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R4 - gornji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 1
MEMBER: 745 **POINT:** 3 **COORDINATE:** x = 1.00 L = 6.00 m

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 CAE 120x10

h=120 mm	gM0=1.00	gM1=1.00	
b=240 mm	Ay=2400 mm2	Az=2160 mm2	Ax=4636 mm2
tw=10 mm	Iy=6258000 mm4	Iz=11337533 mm4	Ix=153334 mm4
tf=10 mm	Wely=72014 mm3	Welz=94479 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 18.94 kN	My,Ed = 2.25 kN*m	Mz,Ed = -24.68 kN*m	Vy,Ed = 9.86 kN
Nc,Rd = 1645.87 kN	My,Ed,max = 2.25 kN*m	Mz,Ed,max = -24.68 kN*m	Vy,T,Rd = 491.90 kN
Nb,Rd = 899.11 kN	My,c,Rd = 25.56 kN*m	Mz,c,Rd = 33.54 kN*m	Vz,Ed = 2.00 kN
			Vz,T,Rd = 442.71 kN
			Tt,Ed = -0.05 kN*m
			Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.00 m	Lam_y = 1.08	Lz = 6.00 m	Lam_z = 0.80
Lcr,y = 3.00 m	Xy = 0.55	Lcr,z = 3.00 m	Xz = 0.72
Lamy = 81.66	kyy = 0.91	Lamz = 60.67	kyz = 0.91

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.76 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.83 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.02 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 81.66 < Lambda,max = 210.00 Lambda,z = 60.67 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.68 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.67 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R4 - donji pojas

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 20
MEMBER: 79 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

LOADS:

Governing Load Case: 29 COMB19 (1+2)*1.35+5*1.50+4*0.75

MATERIAL:

S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 CAE 120x10

h=120 mm	gM0=1.00	gM1=1.00	
b=240 mm	Ay=2400 mm2	Az=2160 mm2	Ax=4636 mm2
tw=10 mm	Iy=6258000 mm4	Iz=11337533 mm4	Ix=153334 mm4
tf=10 mm	Wely=72014 mm3	Welz=94479 mm3	
	Weff,y=72014 mm3	Weff,z=94479 mm3	Aeff=4636 mm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = 89.60 kN	My,Ed = -1.77 kN*m	Mz,Ed = -16.50 kN*m	Vy,Ed = -3.54 kN
Nc,Rd = 1645.87 kN	My,Ed,max = -2.15 kN*m	Mz,Ed,max = -16.50 kN*m	Vy,T,Rd = 491.90 kN
Nb,Rd = 299.49 kN	My,c,Rd = 25.56 kN*m	Mz,c,Rd = 33.54 kN*m	Vz,Ed = 1.41 kN
			Vz,T,Rd = 442.71 kN
			Tt,Ed = -0.00 kN*m
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.00 m	Lam_y = 2.16	Lz = 6.00 m	Lam_z = 0.80
Lcr,y = 6.00 m	Xy = 0.18	Lcr,z = 3.00 m	Xz = 0.72
Lamy = 163.31	kyy = 1.25	Lamz = 60.67	kyz = 0.93

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.57 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.57 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.01 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 163.31 < Lambda,max = 210.00 Lambda,z = 60.67 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.80 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.79 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R4 - ispuna

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 3
MEMBER: 751 **POINT:** 2 **COORDINATE:** x = 0.50 L = 1.98 m

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 CAE 70x7

h=70 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=980 mm2	Az=882 mm2	Ax=1879 mm2
tw=7 mm	Iy=846000 mm4	Iz=1575369 mm4	Ix=30412 mm4
tf=7 mm	Wely=16819 mm3	Welz=22505 mm3	
	Weff,y=16819 mm3	Weff,z=22505 mm3	Aeff=1879 mm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = 42.32 kN	My,Ed = 0.02 kN*m	Mz,Ed = -0.34 kN*m	Vy,Ed = -0.03 kN
Nc,Rd = 667.18 kN	My,Ed,max = 0.05 kN*m	Mz,Ed,max = -0.35 kN*m	Vy,c,Rd = 200.86 kN
Nb,Rd = 95.85 kN	My,c,Rd = 5.97 kN*m	Mz,c,Rd = 7.99 kN*m	Vz,Ed = -0.01 kN
			Vz,c,Rd = 180.77 kN
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 3.94 m	Lam_y = 2.46	Lz = 3.94 m	Lam_z = 1.80
Lcr,y = 3.94 m	Xy = 0.14	Lcr,z = 3.94 m	Xz = 0.25
Lamy = 185.88	kzy = 1.19	Lamz = 136.22	kzz = 1.15

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.11 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.11 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 185.88 < Lambda,max = 210.00 Lambda,z = 136.22 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.49 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.49 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R5 - gornji pojas

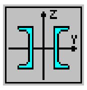
STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 11
MEMBER: 740 **POINT:** 3 **COORDINATE:** x = 0.50 L = 3.00 m

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355)   fy = 355.00 MPa



SECTION PARAMETERS: 2 UPN 80

h=80 mm	gM0=1.00	gM1=1.00	
b=90 mm	Ay=1440 mm2	Az=960 mm2	Ax=2191 mm2
tw=6 mm	Iy=2118140 mm4	Iz=849465 mm4	Ix=39040 mm4
tf=8 mm	Wply=63782 mm3	Wplz=31820 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -11.09 kN	My,Ed = -0.82 kN*m	Mz,Ed = 5.47 kN*m	Vy,Ed = -1.82 kN
Nt,Rd = 777.75 kN	My,pl,Rd = 22.64 kN*m	Mz,pl,Rd = 11.30 kN*m	Vy,c,Rd = 295.14 kN
	My,c,Rd = 22.64 kN*m	Mz,c,Rd = 11.30 kN*m	Vz,Ed = -0.61 kN
	MN,y,Rd = 22.64 kN*m	MN,z,Rd = 11.29 kN*m	Vz,c,Rd = 196.76 kN
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nt,Rd = 0.01 < 1.00   (6.2.3.(1))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.52 < 1.00   (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.01 < 1.00   (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00   (6.2.6.(1))

Section OK !!!

Pozicija R5 - donji pojas

STEEL DESIGN

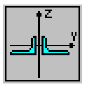
CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 22
MEMBER: 741   **POINT:** 3   **COORDINATE:** x = 0.50 L = 3.00 m

---

LOADS:
Governing Load Case: 13 COMB3 (1+2)*1.35+3*1.50+7*0.90

MATERIAL:
S355 (S355)   fy = 355.00 MPa



SECTION PARAMETERS: 2 CAE 100x10

h=100 mm	gM0=1.00	gM1=1.00	
b=200 mm	Ay=2000 mm2	Az=1800 mm2	Ax=3831 mm2
tw=10 mm	Iy=3534000 mm4	Iz=6580485 mm4	Ix=126666 mm4
tf=10 mm	Wely=49220 mm3	Welz=65805 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 7.58 kN	My,Ed = 9.92 kN*m	Mz,Ed = 0.06 kN*m	Vy,Ed = 0.00 kN
Nc,Rd = 1359.97 kN	My,Ed,max = 9.92 kN*m	Mz,Ed,max = 0.06 kN*m	Vy,T,Rd = 409.92 kN
Nb,Rd = 174.58 kN	My,c,Rd = 17.47 kN*m	Mz,c,Rd = 23.36 kN*m	Vz,Ed = 4.05 kN
			Vz,T,Rd = 368.93 kN
			Tt,Ed = 0.00 kN*m
			Class of section = 3

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.00 m	Lam_y = 2.62	Lz = 6.00 m	Lam_z = 0.96
Lcr,y = 6.00 m	Xy = 0.13	Lcr,z = 3.00 m	Xz = 0.62
Lamy = 197.55	kyy = 0.96	Lamz = 72.38	kyz = 0.90

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.57 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*Tau,y,Ed^2)/(fy/gM0) = 0.58 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.01 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 197.55 < Lambda,max = 210.00 Lambda,z = 72.38 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.59 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.48 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija R5 - ispuna

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 2
MEMBER: 744 **POINT:** 2 **COORDINATE:** x = 0.50 L = 1.73 m

LOADS:
Governing Load Case: 25 COMB15 (1+2)*1.35+7*1.50+3*0.75

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: 2 CAE 70x7

h=70 mm	gM0=1.00	gM1=1.00	
b=140 mm	Ay=980 mm2	Az=882 mm2	Ax=1879 mm2
tw=7 mm	Iy=846000 mm4	Iz=1575369 mm4	Ix=30412 mm4
tf=7 mm	Wely=16819 mm3	Welz=22505 mm3	

	Weff,y=16819 mm3	Weff,z=22505 mm3	Aeff=1879 mm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = 17.98 kN	My,Ed = 0.31 kN*m	Mz,Ed = -0.02 kN*m	Vy,Ed = 0.01 kN
Nc,Rd = 667.18 kN	My,Ed,max = 0.31 kN*m	Mz,Ed,max = -0.03 kN*m	Vy,c,Rd = 200.86 kN
Nb,Rd = 120.25 kN	My,c,Rd = 5.97 kN*m	Mz,c,Rd = 7.99 kN*m	Vz,Ed = 0.03 kN
			Vz,c,Rd = 180.77 kN
			Class of section = 4

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 3.48 m	Lam_y = 2.17	Lz = 3.48 m	Lam_z = 1.59
Lcr,y = 3.48 m	Xy = 0.18	Lcr,z = 3.48 m	Xz = 0.31
Lamy = 164.17	kyy = 1.08	Lamz = 120.31	kyz = 0.97

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.08 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*Tau,y,Ed^2)/(fy/gM0) = 0.08 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Global stability check of member:

Lambda,y = 164.17 < Lambda,max = 210.00 Lambda,z = 120.31 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.21 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.19 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija V1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 15
MEMBER: 957 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

LOADS:
Governing Load Case: 37 COMB27 (1+2)*1.35+5*1.50

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: ROND 17

h=17 mm	gM0=1.00	gM1=1.00	
	Ay=144 mm2	Az=144 mm2	Ax=227 mm2
tw=9 mm	Iy=4100 mm4	Iz=4100 mm4	Ix=8200 mm4

Wply=819 mm3 Wplz=819 mm3

INTERNAL FORCES AND CAPACITIES:
N,Ed = -56.83 kN
Nt,Rd = 80.58 kN

Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.71 < 1.00 (6.2.3.(1))

Section OK !!!

Pozicija V2

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 29
MEMBER: 1013 **POINT:** 3 **COORDINATE:** x = 1.00 L = 9.15 m

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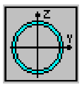
LOADS:
Governing Load Case: 31 COMB21 (1+2)*1.35+7*1.50+4*0.75

MATERIAL:
S355 (S355) fy = 345.00 MPa



SECTION PARAMETERS: ROND 28

h=28 mm	gM0=1.00	gM1=1.00	
	Ay=392 mm2	Az=392 mm2	Ax=616 mm2
tw=14 mm	Iy=30172 mm4	Iz=30172 mm4	Ix=60344 mm4
	Wply=3659 mm3	Wplz=3659 mm3	

INTERNAL FORCES AND CAPACITIES:
N,Ed = -202.80 kN
Nt,Rd = 212.43 kN

Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nt,Rd = 0.95 < 1.00   (6.2.3.(1))

Section OK !!!

Pozicija V3

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 30
MEMBER: 1185   **POINT:** 1   **COORDINATE:** x = 0.00 L = 0.00 m

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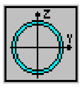
LOADS:

Governing Load Case: 38 COMB28 (1+2)*1.35+6*1.50

MATERIAL:

S355 (S355)   fy = 355.00 MPa



SECTION PARAMETERS: ROND 15

h=15 mm	gM0=1.00	gM1=1.00	
	Ay=113 mm2	Az=113 mm2	Ax=177 mm2
tw=8 mm	Iy=2485 mm4	Iz=2485 mm4	Ix=4970 mm4
	Wply=563 mm3	Wplz=563 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -40.79 kN
Nt,Rd = 62.73 kN

Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nt,Rd = 0.65 < 1.00   (6.2.3.(1))

Section OK !!!

Pozicija E1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 16
MEMBER: 894 **POINT:** 1 **COORDINATE:** x = 0.50 L = 3.00 m

LOADS:
Governing Load Case: 37 COMB27 (1+2)*1.35+5*1.50

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: TCAR 100x10

h=100 mm	gM0=1.00	gM1=1.00	
b=100 mm	Ay=1776 mm2	Az=1776 mm2	Ax=3552 mm2
tw=10 mm	Iy=4743000 mm4	Iz=4743000 mm4	Ix=7612000 mm4
tf=10 mm	Wply=122000 mm3	Wplz=122000 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 34.49 kN	My,Ed = -0.26 kN*m	Mz,Ed = 2.35 kN*m	Vy,Ed = 4.33 kN
Nc,Rd = 1260.96 kN	My,Ed,max = -0.84 kN*m	Mz,Ed,max = -10.64 kN*m	Vy,T,Rd = 363.78 kN
Nb,Rd = 762.39 kN	My,c,Rd = 43.31 kN*m	Mz,c,Rd = 43.31 kN*m	Vz,Ed = 0.68 kN
	MN,y,Rd = 43.31 kN*m	MN,z,Rd = 43.31 kN*m	Vz,T,Rd = 363.78 kN
			Tt,Ed = 0.02 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.00 m	Lam_y = 1.09	Lz = 6.00 m	Lam_z = 1.09
Lcr,y = 3.00 m	Xy = 0.60	Lcr,z = 3.00 m	Xz = 0.60
Lamy = 82.10	kzy = 0.56	Lamz = 82.10	kzz = 0.93

Torsional buckling:		Flexural-torsional buckling	
Curve,T=a	alfa,T=0.21	Curve,TF=a	alfa,TF=0.21
Lt=6.00 m	fi,T=0.49	Ncr,y=1066.26 kN	fi,TF=0.49
Ncr,T=218770.40 kN	X,T=1.00	Ncr,TF=218770.40 kN	X,TF=1.00
Lam_T=0.08	Nb,T,Rd=1260.96 kN	Lam_TF=0.08	Nb,TF,Rd=1260.96 kN

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.03 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.66 + (Mz,Ed/MN,z,Rd)^1.66 = 0.01 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.01 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)

Global stability check of member:

Lambda,y = 82.10 < Lambda,max = 210.00   Lambda,z = 82.10 < Lambda,max = 210.00   STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.05 < 1.00   (6.3.1)
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.20 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.29 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija E2

STEEL DESIGN

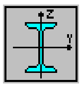
CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 18
MEMBER: 943   **POINT:** 1   **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 27 COMB17 (1+2)*1.35+9*1.50+3*0.75

MATERIAL:
S355 (S355)   fy = 355.00 MPa

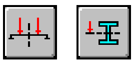


SECTION PARAMETERS: IPN 180

h=180 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1784 mm2	Az=1297 mm2	Ax=2787 mm2
tw=7 mm	Iy=14434400 mm4	Iz=812871 mm4	Ix=98000 mm4
tf=10 mm	Wply=186578 mm3	Wplz=33318 mm3	

INTERNAL FORCES AND CAPACITIES:

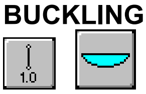
N,Ed = 21.82 kN	My,Ed = -11.62 kN*m	Mz,Ed = -0.08 kN*m	Vy,Ed = -0.08 kN
Nc,Rd = 989.43 kN	My,Ed,max = -11.62 kN*m		Mz,Ed,max = 0.11 kN*m
	Vy,T,Rd = 365.49 kN		
Nb,Rd = 231.06 kN	My,c,Rd = 66.24 kN*m	Mz,c,Rd = 11.83 kN*m	Vz,Ed = 6.44 kN
	MN,y,Rd = 66.24 kN*m	MN,z,Rd = 11.83 kN*m	Vz,T,Rd = 265.73 kN
	Mb,Rd = 40.94 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1



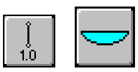
LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 58.83 kN*m	Curve,LT - c	XLT = 0.60
Lcr,low=2.42 m	Lam_LT = 1.06	fi,LT = 1.08	XLT,mod = 0.62

BUCKLING PARAMETERS:



About y axis:



About z axis:

Ly = 2.42 m	Lam_y = 0.45	Lz = 2.42 m	Lam_z = 1.88
Lcr,y = 2.42 m	Xy = 0.94	Lcr,z = 2.42 m	Xz = 0.23
Lamy = 33.69	kzy = 0.99	Lamz = 141.97	kzz = 1.02

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34

Lt=2.42 m	fi,T=0.87	Ncr,y=4967.97 kN	fi,TF=0.87
Ncr,T=1780.19 kN	X,T=0.76	Ncr,TF=1780.19 kN	X,TF=0.76
Lam_T=0.75	Nb,T,Rd=749.39 kN	Lam_TF=0.75	Nb,TF,Rd=749.39 kN

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.02 < 1.00   (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.04 < 1.00   (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00   (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.02 < 1.00   (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)
Global stability check of member:
Lambda,y = 33.69 < Lambda,max = 210.00   Lambda,z = 141.97 < Lambda,max = 210.00   STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.09 < 1.00   (6.3.1)
My,Ed,max/Mb,Rd = 0.28 < 1.00   (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.29 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.38 < 1.00 (6.3.3.(4))

Section OK !!!

Pozicija E3

STEEL DESIGN

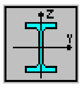
CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 13
MEMBER: 593   **POINT:** 2   **COORDINATE:** x = 0.50 L = 3.00 m

---

LOADS:
Governing Load Case: 19 COMB9 (1+2)*1.35+4*1.50+7*0.90

MATERIAL:
S355 (S355)   fy = 355.00 MPa

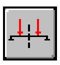
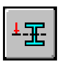


SECTION PARAMETERS: IPE 180

h=180 mm	gM0=1.00	gM1=1.00	
b=91 mm	Ay=1621 mm2	Az=1125 mm2	Ax=2395 mm2
tw=5 mm	Iy=13169600 mm4	Iz=1008500 mm4	Ix=49000 mm4
tf=8 mm	Wply=166424 mm3	Wplz=34600 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 1.33 kN	My,Ed = 6.42 kN*m	Mz,Ed = -2.69 kN*m	Vy,Ed = 0.43 kN
Nc,Rd = 850.13 kN	My,Ed,max = 6.53 kN*m	Mz,Ed,max = 4.65 kN*m	Vy,T,Rd = 332.16 kN
Nb,Rd = 188.57 kN	My,c,Rd = 59.08 kN*m	Mz,c,Rd = 12.28 kN*m	Vz,Ed = -0.43 kN
	MN,y,Rd = 59.08 kN*m	MN,z,Rd = 12.28 kN*m	Vz,T,Rd = 230.58 kN
	Mb,Rd = 34.32 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 39.94 kN*m	Curve,LT - b	XLT = 0.57
Lcr,upp=3.00 m	Lam_LT = 1.22	fi,LT = 1.19	XLT,mod = 0.58

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.00 m	Lam_y = 0.54	Lz = 6.00 m	Lam_z = 1.94
Lcr,y = 3.00 m	Xy = 0.91	Lcr,z = 3.00 m	Xz = 0.22
Lamy = 40.45	kzy = 1.00	Lamz = 146.19	kzz = 0.91

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.00 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.23 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

Lambda,y = 40.45 < Lambda,max = 210.00 Lambda,z = 146.19 < Lambda,max = 210.00 STABLE
My,Ed,max/Mb,Rd = 0.19 < 1.00 (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.38 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.54 < 1.00 (6.3.3.(4))

Section OK !!!

7.2. Provjera GSU

Pozicija R1

Progib od promjenjivog opterećenja

$$\delta_p = 2{,}1cm \leq \delta_{dop} = \frac{L}{250} = \frac{700}{250} = 2{,}8cm$$

Ukupan progib

$$d_{uk} = 3{,}5cm \leq \delta_{max} = \frac{L}{200} = \frac{700}{200} = 3{,}5cm$$

ZADOVOLJAVA!

Pozicija R2

Progib od promjenjivog opterećenja

$$\delta_p = 2{,}6cm \leq \delta_{dop} = \frac{L}{250} = \frac{800}{250} = 3{,}2cm$$

Ukupan progib

$$d_{uk}3{,}2cm =\leq \delta_{max} = \frac{L}{200} = \frac{800}{200} = 4{,}0cm$$

ZADOVOLJAVA!

Pozicija R3

Progib od promjenjivog opterećenja

$$\delta_p = 2{,}6cm \leq \delta_{dop} = \frac{L}{250} = \frac{800}{250} = 3{,}2cm$$

Ukupan progib

$$d_{uk} = 2{,}0cm =\leq \delta_{max} = \frac{L}{200} = \frac{800}{200} = 4{,}0cm$$

ZADOVOLJAVA!

Pozicija R4

Progib od promjenjivog opterećenja

$$\delta_p 1{,}6cm =\leq \delta_{dop} = \frac{L}{250} = \frac{600}{250} = 2{,}4cm$$

Ukupan progib

$$d_{uk} = 2{,}5cm \leq \delta_{max} = \frac{L}{200} = \frac{600}{200} = 3{,}0cm$$

ZADOVOLJAVA!

Pozicija R5

Progib od promjenjivog opterećenja

$$\delta_p = 1{,}7cm \leq \delta_{dop} = \frac{L}{250} = \frac{600}{250} = 2{,}4cm$$

Ukupan progib

$$d_{uk} = 3{,}0cm \leq \delta_{max} = \frac{L}{200} = \frac{600}{200} = 3{,}0cm$$

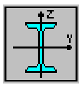
ZADOVOLJAVA!

Pozicija P1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 14
MEMBER: 762 **POINT:** **COORDINATE:**

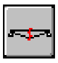


SECTION PARAMETERS: IPN 180

ht=180 mm

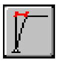
bf=82 mm	Ay=1706 mm2	Az=1242 mm2	Ax=2787 mm2
tw=7 mm	Iy=14434400 mm4	Iz=812871 mm4	Ix=98000 mm4
tf=10 mm	Wely=160382 mm3	Welz=19826 mm3	

LIMIT DISPLACEMENTS



Deflections (LOCAL SYSTEM):

uy = 35 mm < uy max = L/200.00 = 390 mm Verified
Governing Load Case: 48 COMB38 (1+2+3)*1.00+10*0.60
uz = 8 mm < uz max = L/200.00 = 390 mm Verified
Governing Load Case: 47 COMB37 (1+2+3)*1.00+9*0.60
u inst,y = 14 mm < u inst,max,y = L/250.00 = 312 mm Verified
Governing Load Case: 1*6
u inst,z = 3 mm < u inst,max,z = L/250.00 = 312 mm Verified
Governing Load Case: 1*6



Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

Pozicija P2

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 21
MEMBER: 758 **POINT:** **COORDINATE:**

SECTION PARAMETERS: IPN 240

ht=240 mm			
bf=106 mm	Ay=2777 mm2	Az=2088 mm2	Ax=4610 mm2
tw=9 mm	Iy=42374500 mm4	Iz=2202600 mm4	Ix=257000 mm4
tf=13 mm	Wely=353121 mm3	Welz=41558 mm3	

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 3 mm < uy max = L/200.00 = 25 mm Verified
Governing Load Case: 59 COMB49 (1+2+9)*1.00+3*0.50
uz = 6 mm < uz max = L/200.00 = 25 mm Verified
Governing Load Case: 45 COMB35 (1+2+3)*1.00+7*0.60
u inst,y = 3 mm < u inst,max,y = L/250.00 = 20 mm Verified
Governing Load Case: 1*7
u inst,z = 2 mm < u inst,max,z = L/250.00 = 20 mm Verified
Governing Load Case: 1*3 + 0.6*7

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

Pozicija P3

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 19
MEMBER: 1204 **POINT:** **COORDINATE:**

SECTION PARAMETERS: IPN 180

ht=180 mm			
bf=82 mm	Ay=1706 mm2	Az=1242 mm2	Ax=2787 mm2
tw=7 mm	Iy=14434400 mm4	Iz=812871 mm4	Ix=98000 mm4
tf=10 mm	Wely=160382 mm3	Welz=19826 mm3	

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 1 mm < uy max = L/200.00 = 390 mm Verified
Governing Load Case: 55 COMB45 (1+2+5)*1.00+3*0.50

uz = 5 mm < uz max = L/200.00 = 390 mm Verified
Governing Load Case: 57 COMB47 (1+2+7)*1.00+3*0.50
u inst,y = 0 mm < u inst,max,y = L/250.00 = 312 mm Verified
Governing Load Case: 1*7
u inst,z = 5 mm < u inst,max,z = L/250.00 = 312 mm Verified
Governing Load Case: 0.5*3 + 1*7

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

Pozicija S1

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 10
MEMBER: 700 **POINT:** **COORDINATE:**

SECTION PARAMETERS: 2 UPN 180

ht=180 mm			
bf=140 mm	Ay=3080 mm2	Az=2880 mm2	Ax=5565 mm2
tw=8 mm	Iy=27072200 mm4	Iz=4341486 mm4	Ix=174820 mm4
tf=11 mm	Wely=300802 mm3	Welz=62021 mm3	

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM): *Not analyzed*

Displacements (GLOBAL SYSTEM):
vx = 13 mm < vx max = L/150.00 = 48 mm Verified
Governing Load Case: 63 COMB53 (1+2+7)*1.00+4*0.50
vy = 8 mm < vy max = L/150.00 = 48 mm Verified
Governing Load Case: 55 COMB45 (1+2+5)*1.00+3*0.50

Section OK !!!

Pozicija S2

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 9
MEMBER: 2 **POINT:** **COORDINATE:**

SECTION PARAMETERS: 2 UPN 160

ht=160 mm			
bf=230 mm	Ay=2730 mm2	Az=2400 mm2	Ax=4779 mm2

tw=8 mm	Iy=18490920 mm4	Iz=24060351 mm4	Ix=136580 mm4
tf=11 mm	Wely=231136 mm3	Welz=209220 mm3	

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM): *Not analyzed*

Displacements (GLOBAL SYSTEM):
vx = 12 mm < vx max = L/150.00 = 64 mm Verified
Governing Load Case: 57 COMB47 (1+2+7)*1.00+3*0.50
vy = 17 mm < vy max = L/150.00 = 64 mm Verified
Governing Load Case: 55 COMB45 (1+2+5)*1.00+3*0.50

Section OK !!!

Pozicija S3

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 17
MEMBER: 929 **POINT:** **COORDINATE:**

SECTION PARAMETERS: IPN 220

ht=220 mm			
bf=98 mm	Ay=2391 mm2	Az=1782 mm2	Ax=3952 mm2
tw=8 mm	Iy=30535000 mm4	Iz=1622060 mm4	Ix=192000 mm4
tf=12 mm	Wely=277591 mm3	Welz=33103 mm3	

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM): *Not analyzed*

Displacements (GLOBAL SYSTEM):
vx = 12 mm < vx max = L/150.00 = 48 mm Verified
Governing Load Case: 63 COMB53 (1+2+7)*1.00+4*0.50
vy = 8 mm < vy max = L/150.00 = 48 mm Verified
Governing Load Case: 55 COMB45 (1+2+5)*1.00+3*0.50

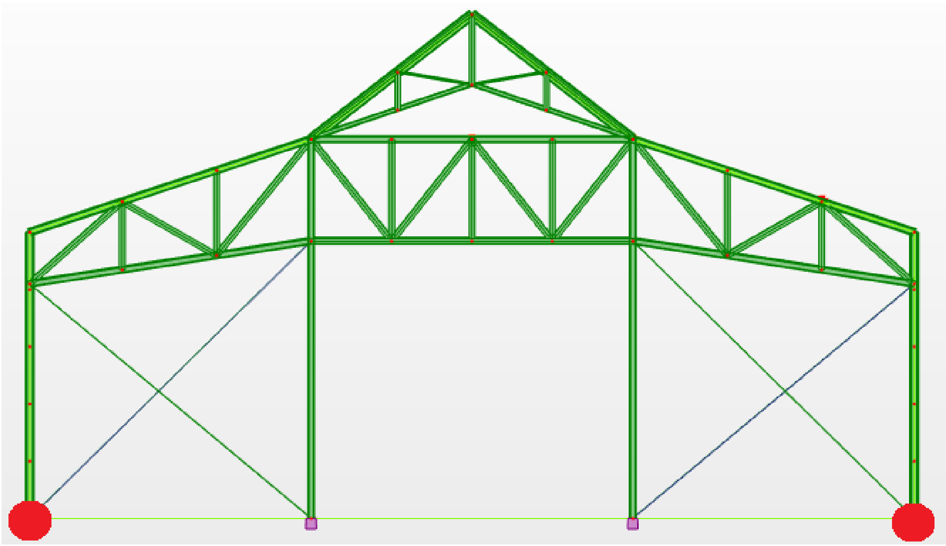
Section OK !!!

8. DIMENZIONIRANJE SPOJEVA

Dimenzioniranje spojeva provedeno je u programskom paketu *Autodesk Robot Structural Analysis Professional 2019* [1], a prema [9] i [10].

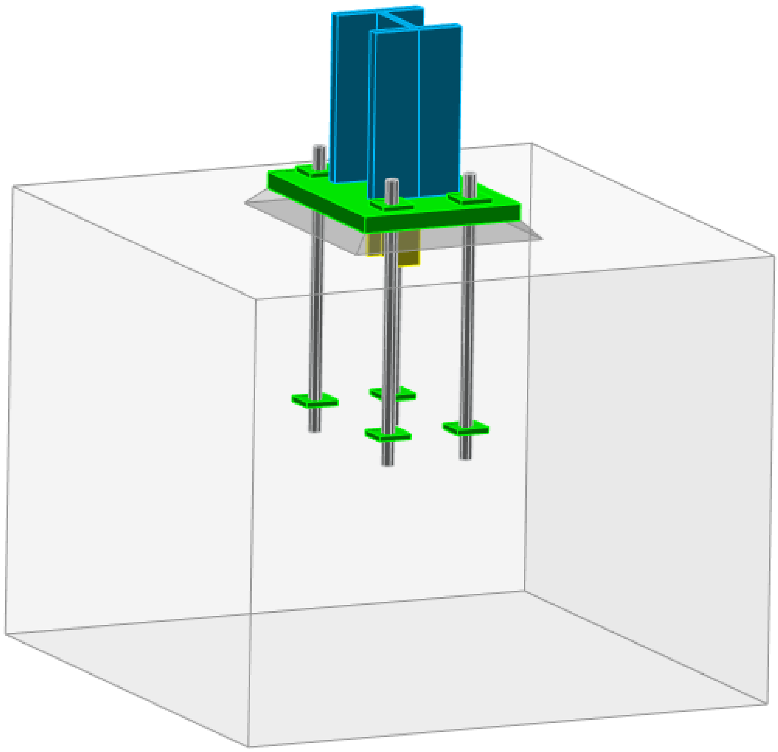
8.1. Spoj glavnog stupa s temeljem

Položaj spoja na konstrukciji prikazan je na slici_crvenom bojom.



Slika 48. Položaj spoja glavnog stupa s temeljem na konstrukciji

Na slici_ prikazan je spoj glavnog stupa s temeljem u 3D-u.



Slika 49. 3D prikaz spoja glavnog stupa s temeljem

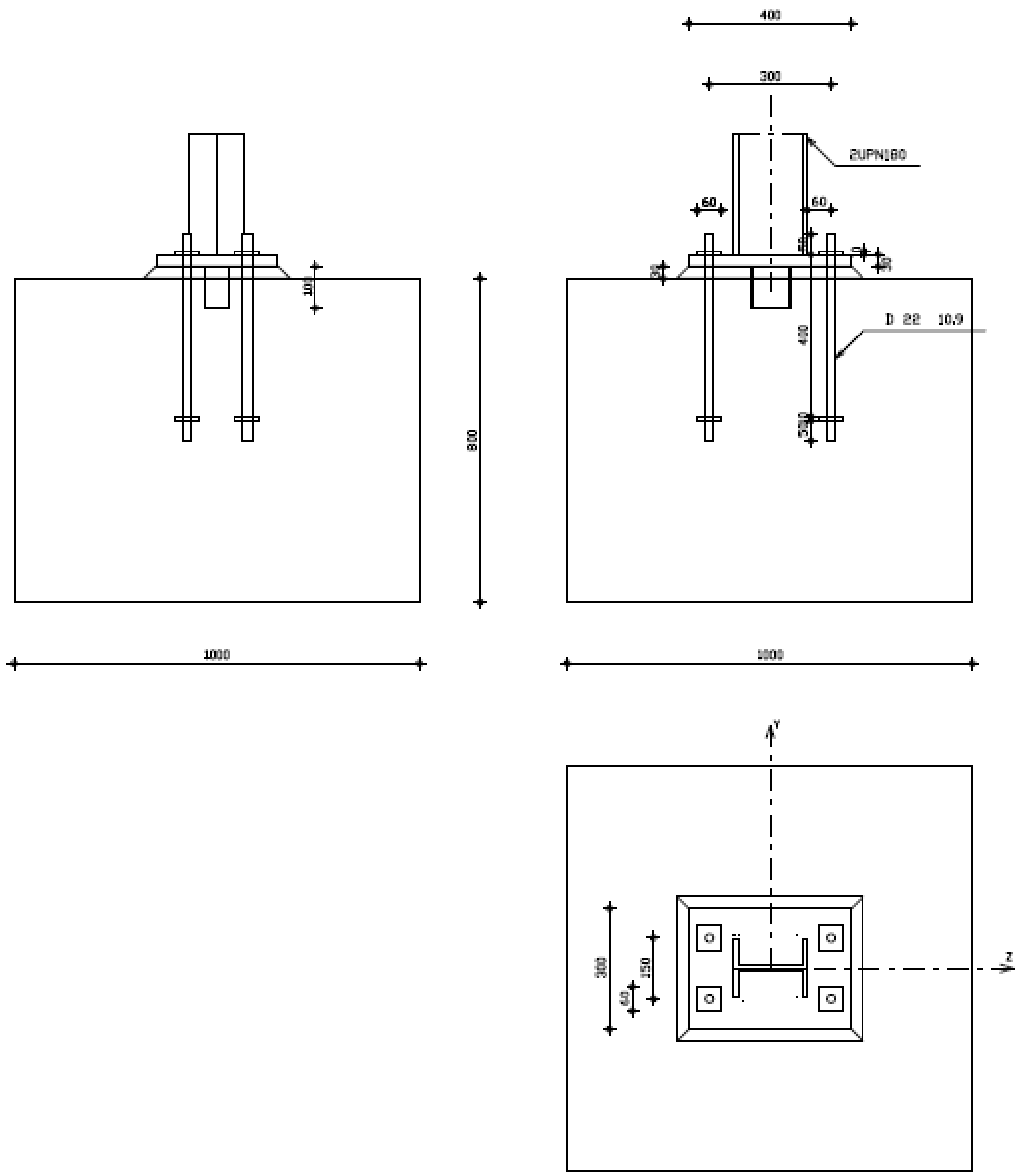
Autodesk Robot Structural Analysis Professional 2019

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide: Design of fastenings in concrete

OK

Ratio
0,80



General

Connection no.: `5`

Connection name: `Fixed column base`

Geometry

Column

Section: `2UPN180`

L_c =	7,20	[m]	Column length
α =	0,0	[Deg]	Inclination angle
h_c =	180	[mm]	Height of column section
b_{fc} =	140	[mm]	Width of column section
t_{wc} =	16	[mm]	Thickness of the web of column section
t_{fc} =	11	[mm]	Thickness of the flange of column section
r_c =	11	[mm]	Radius of column section fillet
A_c =	5600	[mm^2]	Cross-sectional area of a column
I_{yc} =	27000000	[mm^4]	Moment of inertia of the column section

Material: S355

f_{yc} =	355,00	[MPa]	Resistance
f_{uc} =	490,00	[MPa]	Yield strength of a material

Column base

l_{pd} =	400	[mm]	Length
b_{pd} =	300	[mm]	Width
t_{pd} =	30	[mm]	Thickness

Material: S355

f_{ypd} =	355,00	[MPa]	Resistance
f_{upd} =	490,00	[MPa]	Yield strength of a material

Anchorage

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	10.9		Anchor class
f_{yb} =	900,00	[MPa]	Yield strength of the anchor material
f_{ub} =	1000,00	[MPa]	Tensile strength of the anchor material
d =	22	[mm]	Bolt diameter
A_s =	303	[mm^2]	Effective section area of a bolt
A_v =	380	[mm^2]	Area of bolt section
n_H =	2		Number of bolt columns
n_V =	2		Number of bolt rows

Horizontal spacing e_{Hi} = 300 [mm]

Vertical spacing e_{Vi} = 150 [mm]

Anchor dimensions

L_1 =	50	[mm]
L_2 =	400	[mm]
L_3 =	50	[mm]

Anchor plate

l_p =	60	[mm]	Length
b_p =	60	[mm]	Width
t_p =	10	[mm]	Thickness

l_p = 60 [mm] Length

Material: S355

f_y = 355,00 [MPa] Resistance

Washer

l_{wd} = 60 [mm] Length

b_{wd} = 60 [mm] Width

t_{wd} = 10 [mm] Thickness

Wedge

Section: IPE 100

l_w = 100 [mm] Length

Material: S355

f_{yw} = 355,00 [MPa] Resistance

Material factors

γ_{M0} = 1,00 Partial safety factor

γ_{M2} = 1,25 Partial safety factor

γ_C = 1,50 Partial safety factor

Spread footing

L = 1000 [mm] Spread footing length

B = 1000 [mm] Spread footing width

H = 800 [mm] Spread footing height

Concrete

Class C25

f_{ck} = 25,00 [MPa] Characteristic resistance for compression

Grout layer

t_g = 30 [mm] Thickness of leveling layer (grout)

$f_{ck,g}$ = 12,00 [MPa] Characteristic resistance for compression

$C_{f,d}$ = 0,30 Coeff. of friction between the base plate and concrete

Welds

a_p = 4 [mm] Footing plate of the column base

a_w = 4 [mm] Wedge

Loads

Case: Manual calculations.

$N_{j,Ed}$ = 61,51 [kN] Axial force

$V_{j,Ed,y}$ = -9,23 [kN] Shear force

$V_{j,Ed,z}$ = 0,12 [kN] Shear force

$M_{j,Ed,y}$ = -0,33 [kN*m] Bending moment

$M_{j,Ed,z}$ = -10,11 [kN*m] Bending moment

Results

Compression zone

COMPRESSION OF CONCRETE

f_{cd} = 16,67 [MPa] Design compressive resistance EN 1992-1:[3.1.6.(1)]

f_j = 30,43 [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

$c = t_p \sqrt{(f_{yp}/(3 * f_j * \gamma_{M0}))}$

c = 59 [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

b_{eff} = 141 [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

l_{eff} = 284 [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

A_{c0} = 40180 [mm^2] Area of the joint between the base plate and the foundation EN 1992-1:[6.7.(3)]

A_{c1} = 361624 [mm^2] Maximum design area of load distribution EN 1992-1:[6.7.(3)]

$F_{rdu} = A_{c0} * f_{cd} * \sqrt{(A_{c1}/A_{c0})} \leq 3 * A_{c0} * f_{cd}$

F_{rdu} = 2009,02 [kN] Bearing resistance of concrete EN 1992-1:[6.7.(3)]

β_j = 0,67 Reduction factor for compression [6.2.5.(7)]

$f_{jd} = \beta_j * F_{rdu}/(b_{eff} * l_{eff})$

f_{jd} = 33,33 [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,y}$ = 40180 [mm^2] Bearing area for bending My [6.2.8.3.(1)]

$A_{c,z}$ = 40180 [mm^2] Bearing area for bending Mz [6.2.8.3.(1)]

$F_{c,Rd,i} = A_{C,i} * f_{jd}$

$F_{c,Rd,y}$ = 1339,35 [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

$F_{c,Rd,z}$ = 1339,35 [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

CL = 1,00 Section class EN 1993-1-1:[5.5.2]

$W_{pl,y}$ = 675000 [mm^3] Plastic section modulus EN1993-1-1:[6.2.5.(2)]

$M_{c,Rd,y}$ = 239,63 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

$h_{f,y}$ = 157 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$

$F_{c,fc,Rd,y}$ = 1526,27 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

$W_{pl,z}$ = 325000 [mm^3] Plastic section modulus EN1993-1-1:[6.2.5.(2)]

$M_{c,Rd,z}$ = 115,38 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

$h_{f,z}$ = 142 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$

$F_{c,fc,Rd,z}$ = 811,58 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$

$F_{C,Rd,y}$ = 1339,35 [kN] Resistance of spread footing in the compression zone [6.2.8.3]

$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$

$F_{C,Rd,z}$ = 811,58 [kN] Resistance of spread footing in the compression zone [6.2.8.3]

Tension zone

STEEL FAILURE

A_b =	303	[mm^2]	Effective anchor area	[Table 3.4]
f_{ub} =	1000,00	[MPa]	Tensile strength of the anchor material	[Table 3.4]
Beta =	0,85		Reduction factor of anchor resistance	[3.6.1.(3)]

$F_{t,Rd,s1}$ = beta*0.9*f_{ub}*A_b/γ_{M2}

$F_{t,Rd,s1}$ =	185,44	[kN]	Anchor resistance to steel failure	[Table 3.4]
γ_{Ms} =	1,20		Partial safety factor	CEB [3.2.3.2]
f_{yb} =	900,00	[MPa]	Yield strength of the anchor material	CEB [9.2.2]

$F_{t,Rd,s2}$ = f_{yb}*A_b/γ_{Ms}

$F_{t,Rd,s2}$ =	227,25	[kN]	Anchor resistance to steel failure	CEB [9.2.2]

$F_{t,Rd,s}$ = min($F_{t,Rd,s1}$,$F_{t,Rd,s2}$)

$F_{t,Rd,s}$ =	185,44	[kN]	Anchor resistance to steel failure	

PULL-OUT FAILURE

f_{ck} =	25,00	[MPa]	Characteristic compressive strength of concrete	EN 1992-1:[3.1.2]
A_h =	3220	[mm^2]	Bearing area of the head	CEB [15.1.2.3]
p_k =	175,00	[MPa]	Characteristic strength of concrete (pull-out)	CEB [15.1.2.3]
γ_{Mp} =	2,16		Partial safety factor	CEB [3.2.3.1]

$F_{t,Rd,p}$ = p_k*A_h/γ_{Mp}

$F_{t,Rd,p}$ =	279,50	[kN]	Design uplift capacity	CEB [9.2.3]

CONCRETE CONE FAILURE

h_{ef} =	283	[mm]	Effective anchorage depth	CEB [9.2.4]

$N_{Rk,c}^0$ = 9.0[$N^{0.5}$/$mm^{0.5}$]*f_{ck}*$h_{ef}^{1.5}$

$N_{Rk,c}^0$ =	214,61	[kN]	Characteristic resistance of an anchor	CEB [9.2.4]
$s_{cr,N}$ =	850	[mm]	Critical width of the concrete cone	CEB [9.2.4]
$c_{cr,N}$ =	425	[mm]	Critical edge distance	CEB [9.2.4]
$A_{c,N0}$ =	1150000	[mm^2]	Maximum area of concrete cone	CEB [9.2.4]
$A_{c,N}$ =	1000000	[mm^2]	Actual area of concrete cone	CEB [9.2.4]

$\psi_{A,N}$ = $A_{c,N}$/$A_{c,N0}$

$\psi_{A,N}$ =	0,87		Factor related to anchor spacing and edge distance	CEB [9.2.4]
c =	350	[mm]	Minimum edge distance from an anchor	CEB [9.2.4]

$\psi_{s,N}$ = 0.7 + 0.3*c/$c_{cr.N}$ ≤ 1.0

$\psi_{s,N}$ =	0,95		Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete	CEB [9.2.4]
$\psi_{ec,N}$ =	1,00		Factor related to distribution of tensile forces acting on anchors	CEB [9.2.4]

$\psi_{re,N}$ = 0.5 + h_{ef}[mm]/200 ≤ 1.0

$\psi_{re,N}$ =	1,00		Shell spalling factor	CEB [9.2.4]
$\psi_{ucr,N}$ =	1,00		Factor taking into account whether the anchorage is in cracked or non-cracked concrete	CEB [9.2.4]

$\psi_{re,N}$ = 1,00 Shell spalling factor CEB [9.2.4]

γ_{Mc} = 2,16 Partial safety factor CEB [3.2.3.1]

$F_{t,Rd,c} = N_{Rk,c}^{0} * \psi_{A,N} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{ucr,N} / \gamma_{Mc}$

$F_{t,Rd,c}$ = 81,82 [kN] Design anchor resistance to concrete cone failure EN 1992-1:[8.4.2.(2)]

SPLITTING FAILURE

h_{ef} = 400 [mm] Effective anchorage depth CEB [9.2.5]

$N_{Rk,c}^{0} = 9.0[N^{0.5}/mm^{0.5}] * f_{ck} * h_{ef}^{1.5}$

$N_{Rk,c}^{0}$ = 360,00 [kN] Design uplift capacity CEB [9.2.5]

$s_{cr,N}$ = 800 [mm] Critical width of the concrete cone CEB [9.2.5]

$c_{cr,N}$ = 400 [mm] Critical edge distance CEB [9.2.5]

$A_{c,N0}$ = 1045000 [mm^2] Maximum area of concrete cone CEB [9.2.5]

$A_{c,N}$ = 950000 [mm^2] Actual area of concrete cone CEB [9.2.5]

$\psi_{A,N} = A_{c,N}/A_{c,N0}$

$\psi_{A,N}$ = 0,91 Factor related to anchor spacing and edge distance CEB [9.2.5]

c = 350 [mm] Minimum edge distance from an anchor CEB [9.2.5]

$\psi_{s,N} = 0.7 + 0.3 * c/c_{cr.N} \leq 1.0$

$\psi_{s,N}$ = 0,96 Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete CEB [9.2.5]

$\psi_{ec,N}$ = 1,00 Factor related to distribution of tensile forces acting on anchors CEB [9.2.5]

$\psi_{re,N} = 0.5 + h_{ef}[mm]/200 \leq 1.0$

$\psi_{re,N}$ = 1,00 Shell spalling factor CEB [9.2.5]

$\psi_{ucr,N}$ = 1,00 Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]

$\psi_{h,N} = (h/(2*h_{ef}))^{2/3} \leq 1.2$

$\psi_{h,N}$ = 1,00 Coeff. related to the foundation height CEB [9.2.5]

$\gamma_{M,sp}$ = 2,16 Partial safety factor CEB [3.2.3.1]

$F_{t,Rd,sp} = N_{Rk,c}^{0} * \psi_{A,N} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{ucr,N} * \psi_{h,N} / \gamma_{M,sp}$

$F_{t,Rd,sp}$ = 145,83 [kN] Design anchor resistance to splitting of concrete CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$

$F_{t,Rd}$ = 81,82 [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE

Bending moment $M_{j,Ed,y}$

$l_{eff,1}$ = 150 [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2}$ = 150 [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

m = 55 [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd}$ = 11,98 [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd}$ = 11,98 [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd}$ = 863,91 [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd}$ = 304,77 [kN] Resistance of a plate for mode 2 [6.2.4]

Bending moment $M_{j,Ed,y}$

$l_{eff,1}$ =	150	[mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$F_{T,3,Rd}$ =	163,65	[kN]	Resistance of a plate for mode 3	[6.2.4]

$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,y}$ =	163,65	[kN]	Tension resistance of a plate	[6.2.4]

Bending moment $M_{j,Ed,z}$

$l_{eff,1}$ =	200	[mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$l_{eff,2}$ =	200	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
m =	55	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$M_{pl,1,Rd}$ =	15,97	[kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd}$ =	15,97	[kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd}$ =	1151,88	[kN]	Resistance of a plate for mode 1	[6.2.4]
$F_{T,2,Rd}$ =	380,49	[kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd}$ =	163,65	[kN]	Resistance of a plate for mode 3	[6.2.4]

$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,z}$ =	163,65	[kN]	Tension resistance of a plate	[6.2.4]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$N_{j,Rd}$ =	327,30	[kN]	Resistance of a spread footing for axial tension	[6.2.8.3]

$F_{T,Rd,y} = F_{t,pl,Rd,y}$

$F_{T,Rd,y}$ =	163,65	[kN]	Resistance of a column base in the tension zone	[6.2.8.3]

$F_{T,Rd,z} = F_{t,pl,Rd,z}$

$F_{T,Rd,z}$ =	163,65	[kN]	Resistance of a column base in the tension zone	[6.2.8.3]

Connection capacity check

$N_{j,Ed} / N_{j,Rd} \leq 1,0$ (6.24)		0,19 < 1,00	verified	(0,19)
e_y =	5	[mm]	Axial force eccentricity	[6.2.8.3]
$z_{c,y}$ =	79	[mm]	Lever arm $F_{C,Rd,y}$	[6.2.8.1.(2)]
$z_{t,y}$ =	150	[mm]	Lever arm $F_{T,Rd,y}$	[6.2.8.1.(3)]
$M_{j,Rd,y}$ =	1,70	[kN*m]	Connection resistance for bending	[6.2.8.3]
$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0$ (6.23)		0,19 < 1,00	verified	(0,19)
e_z =	164	[mm]	Axial force eccentricity	[6.2.8.3]
$z_{c,z}$ =	71	[mm]	Lever arm $F_{C,Rd,z}$	[6.2.8.1.(2)]
$z_{t,z}$ =	75	[mm]	Lever arm $F_{T,Rd,z}$	[6.2.8.1.(3)]
$M_{j,Rd,z}$ =	16,69	[kN*m]	Connection resistance for bending	[6.2.8.3]
$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0$ (6.23)		0,61 < 1,00	verified	(0,61)
$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0$		0,80 < 1,00	verified	(0,80)

Shear

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

Shear force $V_{j,Ed,y}$

$\alpha_{d,y}$ = 1,04	Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$\alpha_{b,y}$ = 1,00	Coeff. for resistance calculation $F_{1,vb,Rd}$	[Table 3.4]
$k_{1,y}$ = 2,50	Coeff. taking account of the bolt position - perpendicularly to the direction of shear	[Table 3.4]

$F_{1,vb,Rd,y} = k_{1,y}*\alpha_{b,y}*f_{up}*d*t_p / \gamma_{M2}$

$F_{1,vb,Rd,y}$ = 646,80 [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

Shear force $V_{j,Ed,z}$

$\alpha_{d,z}$ = 0,69	Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$\alpha_{b,z}$ = 0,69	Coeff. for resistance calculation $F_{1,vb,Rd}$	[Table 3.4]
$k_{1,z}$ = 2,50	Coeff. taking account of the bolt position - perpendicularly to the direction of shear	[Table 3.4]

$F_{1,vb,Rd,z} = k_{1,z}*\alpha_{b,z}*f_{up}*d*t_p / \gamma_{M2}$

$F_{1,vb,Rd,z}$ = 449,17 [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

SHEAR OF AN ANCHOR BOLT

α_b =	0,25		Coeff. for resistance calculation $F_{2,vb,Rd}$	[6.2.2.(7)]
A_{vb} =	380	[mm^2]	Area of bolt section	[6.2.2.(7)]
f_{ub} =	1000,00	[MPa]	Tensile strength of the anchor material	[6.2.2.(7)]
γ_{M2} =	1,25		Partial safety factor	[6.2.2.(7)]

$F_{2,vb,Rd} = \alpha_b*f_{ub}*A_{vb}/\gamma_{M2}$

$F_{2,vb,Rd}$ = 75,42 [kN] Shear resistance of a bolt - without lever arm [6.2.2.(7)]

α_M =	2,00		Factor related to the fastening of an anchor in the foundation	CEB [9.3.2.2]
$M_{Rk,s}$ =	1,10	[kN*m]	Characteristic bending resistance of an anchor	CEB [9.3.2.2]
l_{sm} =	56	[mm]	Lever arm length	CEB [9.3.2.2]
γ_{Ms} =	1,20		Partial safety factor	CEB [3.2.3.2]

$F_{v,Rd,sm} = \alpha_M*M_{Rk,s}/(l_{sm}*\gamma_{Ms})$

$F_{v,Rd,sm}$ = 32,74 [kN] Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c}$ =	176,74	[kN]	Design uplift capacity	CEB [9.2.4]
k_3 =	2,00		Factor related to the anchor length	CEB [9.3.3]
γ_{Mc} =	2,16		Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,cp} = k_3*N_{Rk,c}/\gamma_{Mc}$

$F_{v,Rd,cp}$ = 163,65 [kN] Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0$ =	710,44	[kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$\psi_{A,V,y}$ =	0,55		Factor related to anchor spacing and edge distance	CEB [9.3.4]
$\psi_{h,V,y}$ =	1,00		Factor related to the foundation thickness	CEB [9.3.4.(c)]
$\psi_{s,V,y}$ =	0,86		Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(d)]

$V_{Rk,c,y}^0$ =	710,44 [kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$\psi_{ec,V,y}$ =	1,00	Factor taking account a group effect when different shear loads are acting on the individual anchors in a group	CEB [9.3.4.(e)]
$\psi_{\alpha,V,y}$ =	1,00	Factor related to the angle at which the shear load is applied	CEB [9.3.4.(f)]
$\psi_{ucr,V,y}$ =	1,00	Factor related to the type of edge reinforcement used	CEB [9.3.4.(g)]
γ_{Mc} =	2,16	Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,c,y} = V_{Rk,c,y}^0 * \psi_{A,V,y} * \psi_{h,V,y} * \psi_{s,V,y} * \psi_{ec,V,y} * \psi_{\alpha,V,y} * \psi_{ucr,V,y} / \gamma_{Mc}$

$F_{v,Rd,c,y}$ = 156,15 [kN] Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}^0$ =	530,94 [kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$\psi_{A,V,z}$ =	0,81	Factor related to anchor spacing and edge distance	CEB [9.3.4]
$\psi_{h,V,z}$ =	1,00	Factor related to the foundation thickness	CEB [9.3.4.(c)]
$\psi_{s,V,z}$ =	0,94	Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(d)]
$\psi_{ec,V,z}$ =	1,00	Factor taking account a group effect when different shear loads are acting on the individual anchors in a group	CEB [9.3.4.(e)]
$\psi_{\alpha,V,z}$ =	1,00	Factor related to the angle at which the shear load is applied	CEB [9.3.4.(f)]
$\psi_{ucr,V,z}$ =	1,00	Factor related to the type of edge reinforcement used	CEB [9.3.4.(g)]
γ_{Mc} =	2,16	Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 * \psi_{A,V,z} * \psi_{h,V,z} * \psi_{s,V,z} * \psi_{ec,V,z} * \psi_{\alpha,V,z} * \psi_{ucr,V,z} / \gamma_{Mc}$

$F_{v,Rd,c,z}$ = 187,62 [kN] Concrete resistance for edge failure CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d}$ =	0,30	Coeff. of friction between the base plate and concrete	[6.2.2.(6)]
$N_{c,Ed}$ =	0,00 [kN]	Compressive force	[6.2.2.(6)]

$F_{f,Rd} = C_{f,d} * N_{c,Ed}$

$F_{f,Rd}$ = 0,00 [kN] Slip resistance [6.2.2.(6)]

BEARING PRESSURE OF THE WEDGE ONTO CONCRETE

$F_{v,Rd,wg,y} = 1.4 * l_w * b_{wy} * f_{ck} / \gamma_c$

$F_{v,Rd,wg,y}$ = 233,33 [kN] Resistance for bearing pressure of the wedge onto concrete

$F_{v,Rd,wg,z} = 1.4 * l_w * b_{wz} * f_{ck} / \gamma_c$

$F_{v,Rd,wg,z}$ = 128,33 [kN] Resistance for bearing pressure of the wedge onto concrete

SHEAR CHECK

$V_{j,Rd,y} = n_b * \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{v,Rd,wg,y} + F_{f,Rd}$

$V_{j,Rd,y}$ = 364,29 [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$	0,03 < 1,00	verified	(0,03)

$V_{j,Rd,z} = n_b * \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{v,Rd,wg,z} + F_{f,Rd}$

$V_{j,Rd,z}$ =	259,29	[kN]	Connection resistance for shear	CEB [9.3.1]
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0,00 < 1,00		verified	(0,00)
$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0,03 < 1,00		verified	(0,03)

Welds between the column and the base plate

$\sigma_\perp$ =	113,80	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_\perp$ =	113,80	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
τ_{yII} =	-3,63	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
τ_{zII} =	0,11	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
β_W =	0,90		Resistance-dependent coefficient	[4.5.3.(7)]

$\sigma_\perp / (0.9 {*} f_u/\gamma_{M2}) \leq 1.0$ (4.1)	0,32 < 1,00	verified	(0,32)
$\sqrt{(\sigma_\perp^2 + 3.0 (\tau_{yII}^2 + \tau_\perp^2))} / (f_u/(\beta_W {*} \gamma_{M2})) \leq 1.0$ (4.1)	0,52 < 1,00	verified	(0,52)
$\sqrt{(\sigma_\perp^2 + 3.0 (\tau_{zII}^2 + \tau_\perp^2))} / (f_u/(\beta_W {*} \gamma_{M2})) \leq 1.0$ (4.1)	0,06 < 1,00	verified	(0,06)

Connection stiffness

Bending moment $M_{j,Ed,y}$

b_{eff} =	141	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
l_{eff} =	284	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]

$k_{13,y} = E_c {*} \sqrt{(b_{eff} {*} l_{eff})}/(1.275 {*} E)$

$k_{13,y}$ =	19	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]
l_{eff} =	150	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
m =	55	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]

$k_{15,y} = 0.425 {*} l_{eff} {*} t_p^3/(m^3)$

$k_{15,y}$ =	10	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
L_b =	257	[mm]	Effective anchorage depth	[Table 6.11]

$k_{16,y} = 1.6 {*} A_b/L_b$

$k_{16,y}$ =	2	[mm]	Stiffness coeff. of an anchor subjected to tension	[Table 6.11]
$\lambda_{0,y}$ =	1,32		Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,y}$ =	14659,18	[kN*m]	Initial rotational stiffness	[Table 6.12]
$S_{j,rig,y}$ =	43545,42	[kN*m]	Stiffness of a rigid connection	[5.2.2.5]

$S_{j,ini,y} < S_{j,rig,y}$ SEMI-RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$k_{13,z} = E_c {*} \sqrt{(A_{c,z})}/(1.275 {*} E)$

$k_{13,z}$ =	19	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]
l_{eff} =	200	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
m =	55	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]

$k_{15,z} = 0.425 {*} l_{eff} {*} t_p^3/(m^3)$

$k_{15,z}$ =	13	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
L_b =	257	[mm]	Effective anchorage depth	[Table 6.11]

$k_{16,z} = 1.6 {*} A_b/L_b$

$k_{16,z}$ =	2	[mm]	Stiffness coeff. of an anchor subjected to tension	[Table 6.11]

$\lambda_{0,z}$ =	2,24		Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,z}$ =	4891,72	[kN*m]	Initial rotational stiffness	[6.3.1.(4)]
$S_{j,rig,z}$ =	15024,79	[kN*m]	Stiffness of a rigid connection	[5.2.2.5]
$S_{j,ini,z} < S_{j,rig,z}$ SEMI-RIGID				[5.2.2.5.(2)]

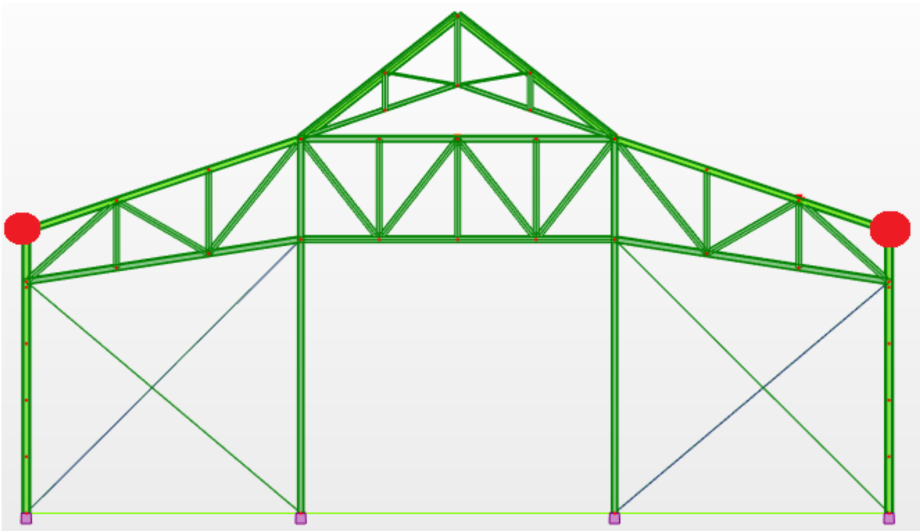
Weakest component:

FOUNDATION - CONCRETE CONE PULL-OUT FAILURE

Connection conforms to the code	Ratio	0,80

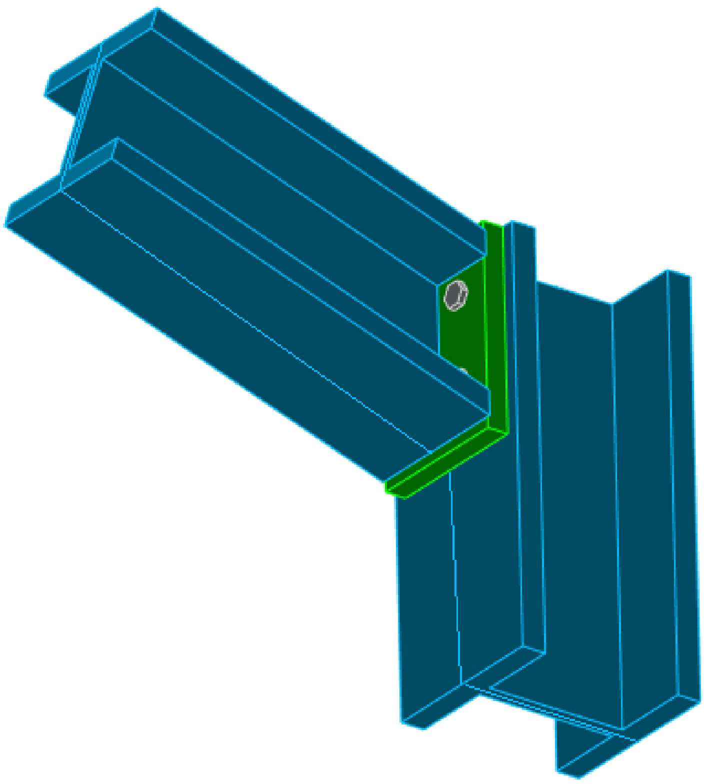
8.2. Spog glavnog stupa i gornjeg pojasa rešetke

Položaj spoja na konstrukciji prikazan je na slici_ crvenom bojom.



Slika 50. Položaj spoja glavnog stupa s gornjim pojasom rešetke

Na slici_ prikazan je spoj glavnog stupa s gornjim pojasom rešetke u 3D-u.



Slika 51. 3D prikaz spoja glavnog stupa s gornjim pojasom rešetke

Autodesk Robot Structural Analysis Professional 2019

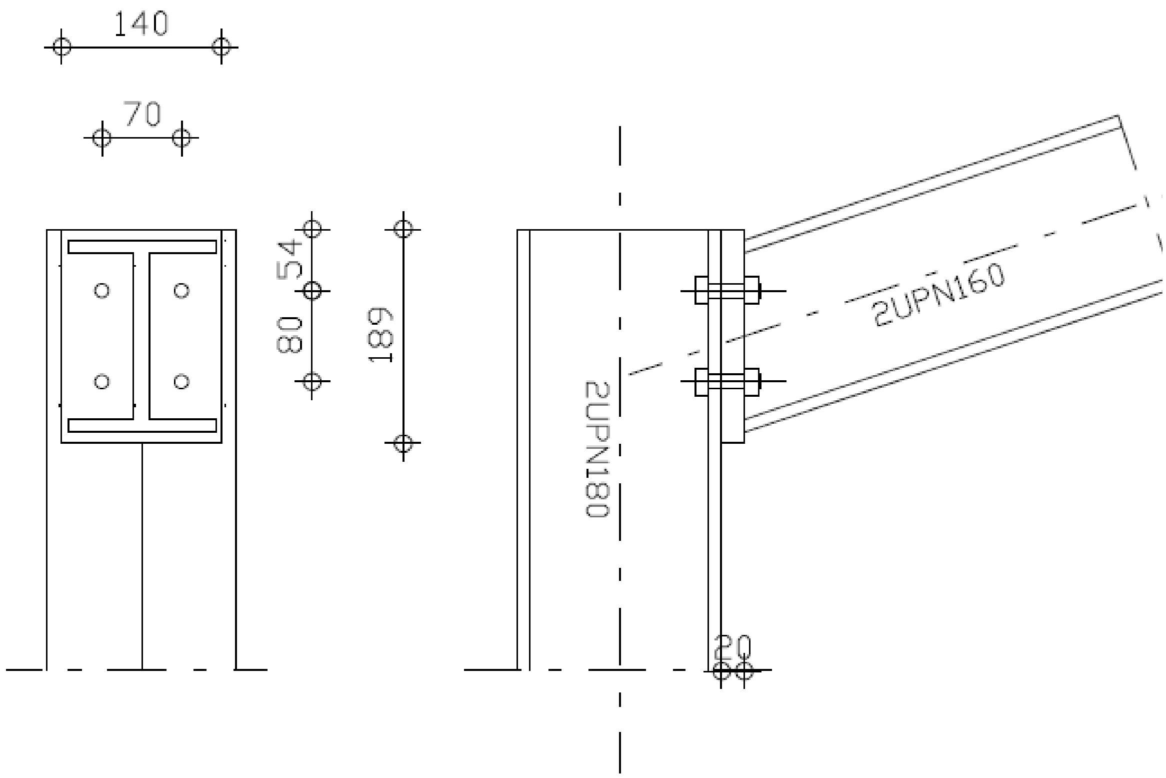
Design of fixed beam-to-column connection

EN 1993-1-8:2005/AC:2009

OK

Ratio

0,17



General

Connection no.: 4

Connection name: Frame knee

Geometry

Column

Section: 2UPN180

α =	-90,0	[Deg]	Inclination angle
h_c =	180	[mm]	Height of column section
b_{fc} =	140	[mm]	Width of column section
t_{wc} =	16	[mm]	Thickness of the web of column section
t_{fc} =	21	[mm]	Thickness of the flange of column section
r_c =	11	[mm]	Radius of column section fillet
A_c =	5600	[mm^2]	Cross-sectional area of a column
I_{xc} =	2280000	[mm^4]	Moment of inertia of the column section

Material: S355

f_{yc} =	355,00	[MPa]	Resistance

Beam

Section: 2UPN160

α =	18,4	[Deg]	Inclination angle
h_b =	160	[mm]	Height of beam section
b_f =	130	[mm]	Width of beam section
t_{wb} =	15	[mm]	Thickness of the web of beam section
t_{fb} =	10,5	[mm]	Thickness of the flange of beam section
r_b =	10,5	[mm]	Radius of beam section fillet
r_b =	10,5	[mm]	Radius of beam section fillet
A_b =	4800	[mm^2]	Cross-sectional area of a beam
I_{xb} =	1706000	[mm^4]	Moment of inertia of the beam section

Material: S355

f_{yb} =	355,00	[MPa]	Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d =	12	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	48,56	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	2		Number of bolt rows
h_1 =	54	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 70 [mm]

Vertical spacing p_i = 80 [mm]

Plate

h_p =	189	[mm]	Plate height
b_p =	140	[mm]	Plate width
t_p =	20	[mm]	Plate thickness

Material: S355

f_{yp} =	355,00	[MPa]	Resistance

Fillet welds

a_w =	5	[mm]	Web weld
a_f =	5	[mm]	Flange weld

Material factors

γ_{M0} =	1,00	Partial safety factor	[2.2]
γ_{M1} =	1,00	Partial safety factor	[2.2]
γ_{M2} =	1,25	Partial safety factor	[2.2]
γ_{M3} =	1,25	Partial safety factor	[2.2]

Loads

Ultimate limit state

Case: Manual calculations.

$M_{b1,Ed}$ =	-1,95	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	2,88	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	32,07	[kN]	Axial force in the right beam
$M_{c1,Ed}$ =	-10,11	[kN*m]	Bending moment in the lower column
$V_{c1,Ed}$ =	-9,23	[kN]	Shear force in the lower column
$N_{c1,Ed}$ =	61,51	[kN]	Axial force in the lower column

Results

Beam resistances

TENSION

A_b =	8056	[mm²]	Area	EN1993-1-1:[6.2.3]

$N_{tb,Rd} = A_b f_{yb} / \gamma_{M0}$

$N_{tb,Rd}$ =	2859,74	[kN]	Design tensile resistance of the section	EN1993-1-1:[6.2.3]

SHEAR

A_{vb} =	2446	[mm²]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ =	501,25	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0,01 < 1,00	verified	(0,01)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} =	493837	[mm³]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ =	175,31	[kN*m]	Plastic resistance of the section for bending (without stiffeners)	EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W_{pl} =	541321	[mm³]	Plastic section modulus	EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ =	192,17	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ =	192,17	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
h_f =	145	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ =	1320,95	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	-1,95	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$M_{b2,Ed}$ =	0,00	[kN*m]	Bending moment (left beam)	[5.3.(3)]
$V_{c1,Ed}$ =	-9,23	[kN]	Shear force (lower column)	[5.3.(3)]
$V_{c2,Ed}$ =	0,00	[kN]	Shear force (upper column)	[5.3.(3)]

WEB PANEL - SHEAR

$M_{b1,Ed}$ = -1,95 [kN*m] Bending moment (right beam) [5.3.(3)]

z = 113 [mm] Lever arm [6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ = -12,68 [kN] Shear force acting on the web panel [5.3.(3)]

A_{vs} = 3086 [mm^2] Shear area of the column web EN1993-1-1:[6.2.6.(3)]

A_{vc} = 3086 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$V_{wp,Rd} = 0.9*(f_{y,wc}*A_{vc}+f_{y,wp}*A_{vp}+f_{ys}*A_{vd}) / (\sqrt{3}\ \gamma_{M0})$

$V_{wp,Rd}$ = 569,25 [kN] Resistance of the column web panel for shear [6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \leq 1,0$ 0,02 < 1,00 verified (0,02)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

t_{wc} = 14 [mm] Effective thickness of the column web [6.2.6.2.(6)]

$b_{eff,c,wc}$ = 250 [mm] Effective width of the web for compression [6.2.6.2.(1)]

A_{vc} = 3086 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

ω = 0,61 Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed}$ = 3,98 [MPa] Maximum compressive stress in web [6.2.6.2.(2)]

k_{wc} = 1,00 Reduction factor conditioned by compressive stresses [6.2.6.2.(2)]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wbc}\ t_{wc}\ f_{yc} / \gamma_{M0}$

$F_{c,wc,Rd1}$ = 760,38 [kN] Column web resistance [6.2.6.2.(1)]

Buckling:

d_{wc} = 104 [mm] Height of compressed web [6.2.6.2.(1)]

λ_p = 0,45 Plate slenderness of an element [6.2.6.2.(1)]

ρ = 1,00 Reduction factor for element buckling [6.2.6.2.(1)]

$F_{c,wb,Rd2} = \omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 760,38 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,upp} = Min (F_{c,wc,Rd1} , F_{c,wc,Rd2})$

$F_{c,wc,Rd,upp}$ = 760,38 [kN] Column web resistance [6.2.6.2.(1)]

Geometrical parameters of a connection

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	16	-	48	-	80	101	116	101	116	130	94	94	94
2	16	-	48	-	80	101	116	101	116	130	94	94	94

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	23	-	35	-	80	144	170	144	170	152	142	142	142
2	23	-	35	-	80	144	170	144	170	152	142	142	142

m – Bolt distance from the web

m_x – Bolt distance from the beam flange

e – Bolt distance from the outer edge

e_x – Bolt distance from the horizontal outer edge

p – Distance between bolts

$l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt for mode 1

$l_{eff,2}$ – Effective length for a single bolt for mode 2

$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode

$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode

$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1

$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

Connection resistance for tension

$F_{t,Rd}$ = 48,56 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 266,00 [kN] Punching shear resistance of a bolt [Table 3.4]

$N_{j,Rd} = \mathrm{Min}\,(N_{tb,Rd}\ ,\ n_v\, n_h\, F_{t,Rd}\ ,\ n_v\, n_h\, B_{p,Rd})$

$N_{j,Rd}$ = 194,23 [kN] Connection resistance for tension [6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1{,}0$ 0,17 < 1,00 verified (0,17)

Connection resistance for bending

$F_{t,Rd}$ = 48,56 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 266,00 [kN] Punching shear resistance of a bolt [Table 3.4]

$F_{t,fc,Rd}$ – column flange resistance due to bending

$F_{t,wc,Rd}$ – column web resistance due to tension

$F_{t,ep,Rd}$ – resistance of the front plate due to bending

$F_{t,wb,Rd}$ – resistance of the web in tension

$F_{t,fc,Rd} = \mathrm{Min}\,(F_{T,1,fc,Rd}\ ,\ F_{T,2,fc,Rd}\ ,\ F_{T,3,fc,Rd})$ [6.2.6.4] , [Tab.6.2]

$F_{t,wc,Rd} = \omega\, b_{eff,t,wc}\, t_{wc}\, f_{yc} / \gamma_{M0}$ [6.2.6.3.(1)]

$F_{t,ep,Rd} = \mathrm{Min}\,(F_{T,1,ep,Rd}\ ,\ F_{T,2,ep,Rd}\ ,\ F_{T,3,ep,Rd})$ [6.2.6.5] , [Tab.6.2]

$F_{t,wb,Rd} = b_{eff,t,wb}\, t_{wb}\, f_{yb} / \gamma_{M0}$ [6.2.6.8.(1)]

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - **Formula**	$F_{t1,Rd,comp}$	**Component**
$F_{t1,Rd} = \mathrm{Min}\,(F_{t1,Rd,comp})$	97,11	Bolt row resistance
$F_{t,fc,Rd(1)}$ = 97,11	97,11	Column flange - tension
$F_{t,wc,Rd(1)}$ = 443,29	443,29	Column web - tension
$F_{t,ep,Rd(1)}$ = 97,11	97,11	Front plate - tension
$F_{t,wb,Rd(1)}$ = 662,38	662,38	Beam web - tension
$B_{p,Rd}$ = 532,01	532,01	Bolts due to shear punching

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$V_{wp,Rd}/\beta$ = 569,25	569,25	Web panel - shear
$F_{c,wc,Rd}$ = 760,38	760,38	Column web - compression
$F_{c,fb,Rd}$ = 1320,95	1320,95	Beam flange - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd}$ = Min ($F_{t2,Rd,comp}$)	97,11	Bolt row resistance
$F_{t,fc,Rd(2)}$ = 97,11	97,11	Column flange - tension
$F_{t,wc,Rd(2)}$ = 443,29	443,29	Column web - tension
$F_{t,ep,Rd(2)}$ = 97,11	97,11	Front plate - tension
$F_{t,wb,Rd(2)}$ = 662,38	662,38	Beam web - tension
$B_{p,Rd}$ = 532,01	532,01	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd}$ = 569,25 - 97,11	472,14	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd}$ = 760,38 - 97,11	663,27	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd}$ = 1320,95 - 97,11	1223,84	Beam flange - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{tj,Rd}$ = 194,23 - 97,11	97,11	Column flange - tension - group
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{tj,Rd}$ = 671,08 - 97,11	573,97	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd}$ = 194,23 - 97,11	97,11	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd}$ = 1312,29 - 97,11	1215,17	Beam web - tension - group

Additional reduction of the bolt row resistance

$F_{t2,Rd} = F_{t1,Rd}\, h_2/h_1$

$F_{t2,Rd}$ = 28,20 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	113	97,11	97,11	443,29	97,11	662,38	97,11	532,01
2	33	28,20	97,11	443,29	97,11	662,38	97,11	532,01

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd}$ = 11,87 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \le 1,0$ 0,16 < 1,00 verified (0,16)

Connection resistance for shear

α_v = 0,60 Coefficient for calculation of $F_{v,Rd}$ [Table 3.4]

$F_{v,Rd}$ = 43,43 [kN] Shear resistance of a single bolt [Table 3.4]

$F_{t,Rd,max}$ = 48,56 [kN] Tensile resistance of a single bolt [Table 3.4]

$F_{b,Rd,int}$ = 235,20 [kN] Bearing resistance of an intermediate bolt [Table 3.4]

$F_{b,Rd,ext}$ = 235,20 [kN] Bearing resistance of an outermost bolt [Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	97,11	16,04	97,11	15,95	31,99	66,42

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
2	97,11	16,04	28,20	4,63	20,67	73,66

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension

$F_{tj,Ed,N}$ – Force due to axial force in a bolt row

$F_{tj,Rd,M}$ – Bolt row resistance for simple bending

$F_{tj,Ed,M}$ – Force due to moment in a bolt row

$F_{tj,Ed}$ – Maximum tensile force in a bolt row

$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed}\ F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed}\ F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \text{Min}\ (n_h\ F_{v,Ed}\ (1 - F_{tj,Ed}/\ (1.4\ n_h\ F_{t,Rd,max}),\ n_h\ F_{v,Rd}\ ,\ n_h\ F_{b,Rd}))$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd} =$	140,08	[kN]	Connection resistance for shear	[Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \le 1,0$	0,02 < 1,00	verified	(0,02)

Weld resistance

$A_w =$	3400	[mm^2]	Area of all welds	[4.5.3.2(2)]
$A_{wy} =$	2430	[mm^2]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} =$	970	[mm^2]	Area of vertical welds	[4.5.3.2(2)]
$I_{wy} =$	14863802	[mm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max} =$	14,73	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_\perp=\tau_\perp =$	11,17	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
$\tau_{II} =$	2,97	[MPa]	Tangent stress	[4.5.3.2(5)]
$\beta_w =$	0,90		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	29,45 < 435,56	verified	(0,07)
$\sqrt{[\sigma_\perp^2 + 3*(\tau_\perp^2+\tau_{II}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	22,92 < 435,56	verified	(0,05)
$\sigma_\perp \le 0.9*f_u/\gamma_{M2}$	14,73 < 352,80	verified	(0,04)

Connection stiffness

$t_{wash} =$	3	[mm]	Washer thickness	[6.2.6.3.(2)]
$h_{head} =$	9	[mm]	Bolt head height	[6.2.6.3.(2)]
$h_{nut} =$	12	[mm]	Bolt nut height	[6.2.6.3.(2)]
$L_b =$	60	[mm]	Bolt length	[6.2.6.3.(2)]

Connection stiffness

t_{wash} = 3 [mm] Washer thickness [6.2.6.3.(2)]

k_{10} = 2 [mm] Stiffness coefficient of bolts [6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}\ h_j$	$k_{eff,j}\ h_j^2$
					Sum	242	22905
1	113	7	252	86	2	187	21124
2	33	7	252	86	2	54	1781

$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \sum_j k_{eff,j}\ h_j^2 / \sum_j k_{eff,j}\ h_j$

z_{eq} = 95 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \sum_j k_{eff,j}\ h_j / z_{eq}$

k_{eq} = 3 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

A_{vc} = 3086 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

β = 1,00 Transformation parameter [5.3.(7)]

z = 113 [mm] Lever arm [6.2.5]

k_1 = 10 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

$b_{eff,c,wc}$ = 250 [mm] Effective width of the web for compression [6.2.6.2.(1)]

t_{wc} = 14 [mm] Effective thickness of the column web [6.2.6.2.(6)]

d_c = 134 [mm] Height of compressed web [6.2.6.2.(1)]

k_2 = 18 [mm] Stiffness coefficient of the compressed column web [6.3.2.(1)]

$S_{j,ini} = E\ z_{eq}^2 / \sum_i (1 / k_1 + 1 / k_2 + 1 / k_{eq})$ [6.3.1.(4)]

$S_{j,ini}$ = 3390,80 [kN*m] Initial rotational stiffness [6.3.1.(4)]

μ = 1,00 Stiffness coefficient of a connection [6.3.1.(6)]

$S_j = S_{j,ini} / \mu$ [6.3.1.(4)]

S_j = 3390,80 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig}$ = 7496,99 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin}$ = 468,56 [kN*m] Stiffness of a pinned connection [5.2.2.5]

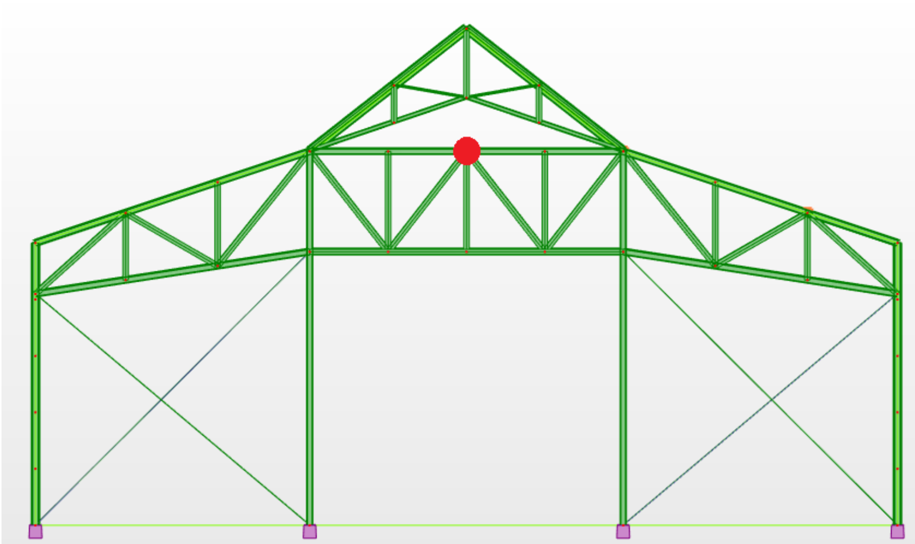
$S_{j,pin} \le S_{j,ini} < S_{j,rig}$ SEMI-RIGID

Weakest component:

Connection conforms to the code	Ratio	0,17

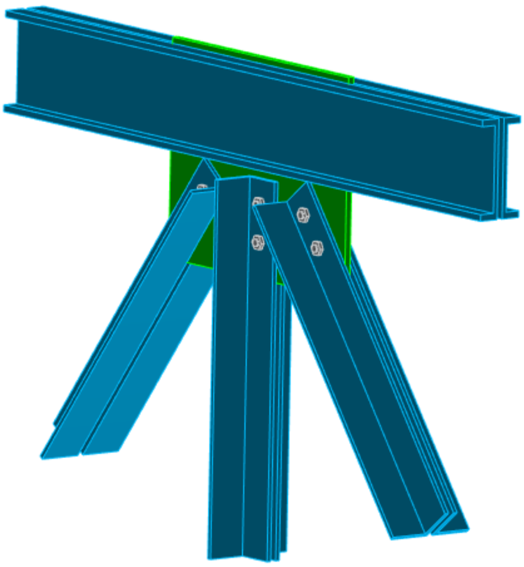
8.3. Čvor rešetke 1

Položaj spoja na konstrukciji prikazan je na slici_ crvenom bojom.



Slika 52. Položaj čvora rešetke 1 na konstrukciji

Na slici _ prikazan je čvor rešetke 1 u 3D-u.



Slika 53. 3D prikaz čvora rešetke 1

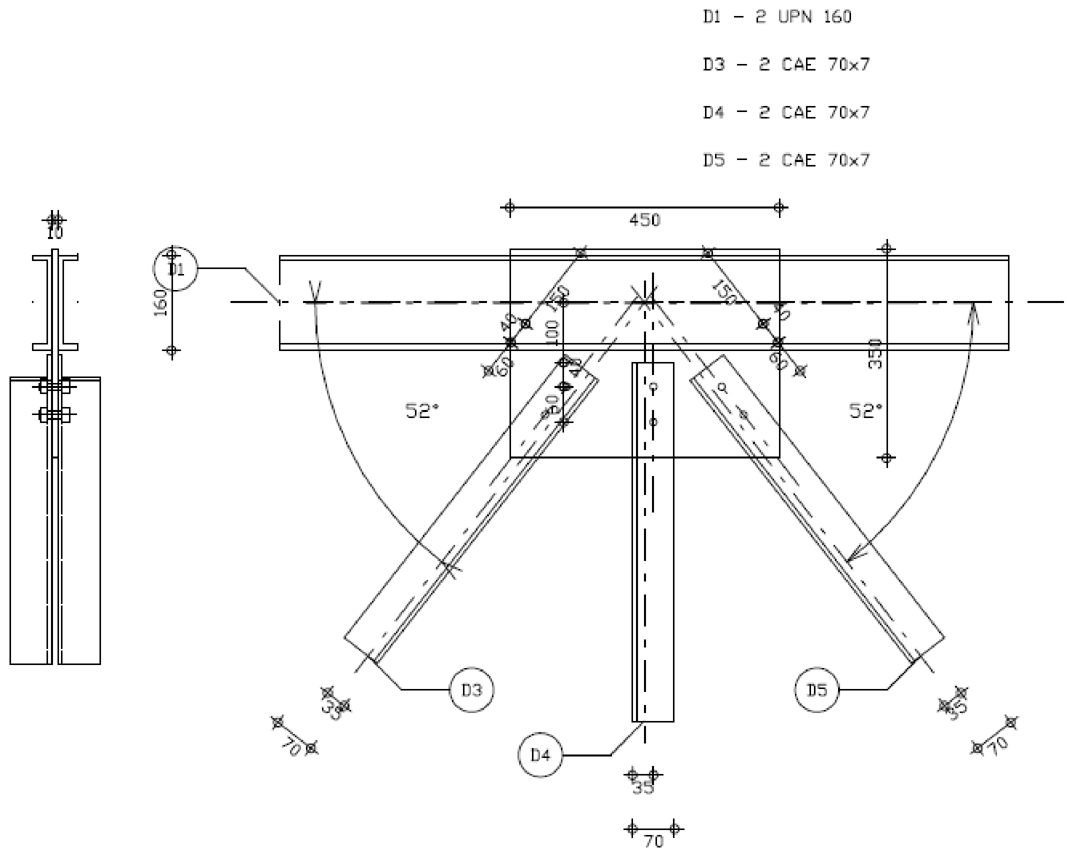
Autodesk Robot Structural Analysis Professional 2019

Calculations of the connection with the gusset plate

EN 1993-1-8:2005/AC:2009

OK

Ratio **0,16**



General

Connection no.: 1

Connection name: Gusset plate: truss chord node

Structure node: 16

Structure bars: 7, 7, 14, 11, 16,

Geometry

Bars

		Bar 1-2		Bar 3	Bar 4	Bar 5	
Bar no.:		7		14	11	16	
Section:		2 UPN 160 spojeni		2 CAE 70x7	2 CAE 70x7	2 CAE 70x7	
	h	160		70	70	70	mm
	b_f	65		70	70	70	mm
	t_w	8		7	7	7	mm

		Bar 1-2		Bar 3	Bar 4	Bar 5	
	t_f	11		7	7	7	mm
	r	11		9	9	9	mm
	A	4779		1879	1879	1879	mm2
Material:		S355		S355	S355	S355	
	f_y	355,00		355,00	355,00	355,00	MPa
	f_u	490,00		490,00	490,00	490,00	MPa
Angle	α	0,0		52,0	90,0	52,0	Deg
Length	l	8,00		3,25	2,56	3,25	m

Bolts

Bar 3

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	4.8		Bolt class
d =	12	[mm]	Bolt diameter
d_0 =	13	[mm]	Bolt opening diameter
A_s =	84	[mm^2]	Effective section area of a bolt
A_v =	113	[mm^2]	Area of bolt section
f_{yb} =	320,00	[MPa]	Yield point
f_{ub} =	400,00	[MPa]	Bolt tensile resistance
n =	2		Number of bolt columns

Bolt spacing 60 [mm]

e_1 =	40	[mm]	Distance of the center of gravity of first bolt from the member end
e_2 =	35	[mm]	Distance of the axis of bolts from the member edge
e_c =	150	[mm]	Distance of the member end from the point of intersection of member axes

Bar 4

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	4.8		Bolt class
d =	12	[mm]	Bolt diameter
d_0 =	13	[mm]	Bolt opening diameter
A_s =	84	[mm^2]	Effective section area of a bolt
A_v =	113	[mm^2]	Area of bolt section
f_{yb} =	320,00	[MPa]	Yield point
f_{ub} =	400,00	[MPa]	Bolt tensile resistance
n =	2		Number of bolt columns

Bolt spacing 60 [mm]

e_1 =	40	[mm]	Distance of the center of gravity of first bolt from the member end
e_2 =	35	[mm]	Distance of the axis of bolts from the member edge
e_c =	100	[mm]	Distance of the member end from the point of intersection of member axes

Bar 5

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	4.8		Bolt class
d =	12	[mm]	Bolt diameter
d_0 =	13	[mm]	Bolt opening diameter
A_s =	84	[mm^2]	Effective section area of a bolt
A_v =	113	[mm^2]	Area of bolt section
f_{yb} =	320,00	[MPa]	Yield point
f_{ub} =	400,00	[MPa]	Bolt tensile resistance
n =	2		Number of bolt columns

Bolt spacing 60 [mm]

e_1 =	40	[mm]	Distance of the center of gravity of first bolt from the member end
e_2 =	35	[mm]	Distance of the axis of bolts from the member edge
e_c =	150	[mm]	Distance of the member end from the point of intersection of member axes

Welds

Member welds

Bar 1-2

l_1 =	100	[mm]	Length 1 of longitudinal fillet weld
l_2 =	100	[mm]	Length 2 of longitudinal fillet weld
a =	3	[mm]	Thickness of longitudinal fillet welds

Gusset plate

l_p =	450	[mm]	Plate length
h_p =	350	[mm]	Plate height
t_p =	10	[mm]	Plate thickness

Parameters

h_1 =	0	[mm]	Cut
v_1 =	0	[mm]	Cut
h_2 =	0	[mm]	Cut
v_2 =	0	[mm]	Cut
h_3 =	0	[mm]	Cut
v_3 =	0	[mm]	Cut
h_4 =	0	[mm]	Cut
v_4 =	0	[mm]	Cut

Center of gravity of the plate with respect to the center of gravity of bars (0;85)

e_V =	90	[mm]	Vertical distance of the plate edge from the point of intersection of member axes
e_H =	225	[mm]	Horizontal distance of the plate edge from the point of intersection of member axes
e_0 =	0	[mm]	Distance to chord axis (horiz.)

Material: S355

f_y =	355,00	[MPa]	Resistance

Material factors

γ_{M0} =	1,00	Partial safety factor	[2.2]
γ_{M2} =	1,25	Partial safety factor	[2.2]

Loads

Case: 39: COMB29 (1+2)*1.35+7*1.50

$N_{b1,Ed}$ =	5,73	[kN]	Axial force
$N_{b2,Ed}$ =	-2,61	[kN]	Axial force
$N_{b3,Ed}$ =	7,25	[kN]	Axial force
$N_{b4,Ed}$ =	2,60	[kN]	Axial force
$N_{b5,Ed}$ =	-12,73	[kN]	Axial force

Results

Bar 1-2

Verification of welds

e =	0	[mm]	Axial force eccentricity relative to the centroid of a weld group	
M_0 =	0,00	[kN*m]	Real bending moment	$M_0 = 0.5*N_{b1,Ed}*e$
A_w =	1080	[mm^2]	Area of welds	
I_0 =	6217427	[mm^4]	Polar moment of inertia of welds	
τ_N =	3,86	[MPa]	Component stress due to influence of the longitudinal force	$\tau_N = 0.5*N_{b1,Ed}/A_s$
τ_{Mx} =	0,00	[MPa]	Component stress due to influence of the moment on the x direction	$\tau_{Mx}=M_0*z/I_0$
τ_{Mz} =	0,00	[MPa]	Component stress due to influence of the moment on the z direction	$\tau_{Mz}=M_0*x/I_0$
τ =	3,86	[MPa]	Resultant stress	$\tau =\sqrt{[(\tau_N+\tau_{Mx})^2+\tau_{Mz}^2]}$
β_w =	0,90		Correlation coefficient	[Table 4.1]
$f_{vw,d}$ =	251,47	[MPa]		$f_{vw,d} = f_u/(\sqrt{3}*\beta_w*\gamma_{M2})$

$\tau \le f_{vRd}$	3,86 < 251,47	verified	(0,02)

Section resistance

A =	2389	[mm^2]	Cross-sectional area	
N_{plRd} =	848,24	[kN]	Design plastic resistance of the gross section	$N_{plRd}=A*f_{y1}/\gamma_{M0}$

$\lvert 0.5*N_{b1,Ed}\rvert \le N_{pl,Rd}$	|4,17| < 848,24	verified	(0,00)

Bar 3

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0		2,50 > 0,00	verified	
α_{bx} =	0,82		Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0		0,82 > 0,00	verified	
$F_{b,Rd1x}$ =	134,40	[kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50		Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0		2,50 > 0,00	verified	
α_{bz} =	0,82		Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0		0,82 > 0,00	verified	
$F_{b,Rd1z}$ =	134,40	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2,50		Coefficient for calculation of $F_{b,Rd}$	k_1=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0		2,50 > 0,00	verified	
α_{bx} =	0,82		Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0		0,82 > 0,00	verified	
$F_{b,Rd2x}$ =	96,00	[kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd2x}$=k_1*α_b*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50		Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0		2,50 > 0,00	verified	
α_{bz} =	0,82		Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0		0,82 > 0,00	verified	
$F_{b,Rd2z}$ =	96,00	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	0,11	[kN*m]	Real bending moment	M_0=$N_{b3,Ed}$*e
F_{NSd} =	3,63	[kN]	Component force in a bolt due to influence of the longitudinal force	F_{NSd} = $N_{b3,Ed}$/n
F_{MSd} =	1,85	[kN]	Component force in a bolt due to influence of the moment	F_{MSd}=M_0*x_{max}/Σx_i^2
$F_{x,Ed}$ =	3,63	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = F_{NSd}
$F_{z,Ed}$ =	1,85	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = F_{MSd}

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
F_{Ed} =	4,07	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx}=\min(F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz}=\min(F_{bRd1z}, F_{bRd2z})$

$\|F_{x,Ed}\| \le F_{Rdx}$	|3,63| < 96,00	verified	(0,04)
$\|F_{z,Ed}\| \le F_{Rdz}$	|1,85| < 96,00	verified	(0,02)
$F_{Ed} \le F_{vRd}$	4,07 < 43,43	verified	(0,09)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm^2]	Cross-sectional area of an angle	
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0*t_{f3}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2*A_{net}*f_{u3})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9*A*f_{y3})/\gamma_{M2}$

$\|0.5*N_{b3,Ed}\| \le N_{u,Rd}$	|3,63| < 217,53	verified	(0,02)
$\|0.5*N_{b3,Ed}\| \le N_{pl,Rd}$	|3,63| < 300,23	verified	(0,01)

Bar verification - block tearing

A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

$\|0.5*N_{b3,Ed}\| \le V_{effRd}$	|3,63| < 154,60	verified	(0,02)

Bar 4

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	$\alpha_{bx}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$

$\alpha_{bx} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd1x} =$ 134,40 [kN]	Design capacity in the limit state of plastification of the opening wall		$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

$k_{1z} =$ 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00	verified	
$\alpha_{bz} =$ 0,82	Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd1z} =$ 134,40 [kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

$k_{1x} =$ 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_1=min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00	verified	
$\alpha_{bx} =$ 0,82	Coefficient determined by bolt spacing		$\alpha_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd2x} =$ 96,00 [kN]	Design capacity in the limit state of plastification of the opening wall		$F_{b,Rd2x}=k_1*\alpha_b*f_u*d*t_i/\gamma_{M2}$

Direction z

$k_{1z} =$ 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00	verified	
$\alpha_{bz} =$ 0,82	Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd2z} =$ 96,00 [kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Verification of a connection due to forces acting on bolts

Bolt shear

$e =$	15	[mm]	Axial force eccentricity relative to the bolt axis	
$M_0 =$	0,04	[kN*m]	Real bending moment	$M_0=N_{b4,Ed}*e$
$F_{NSd} =$	1,30	[kN]	Component force in a bolt due to influence of the longitudinal force	$F_{NSd} = N_{b4,Ed}/n$
$F_{MSd} =$	0,66	[kN]	Component force in a bolt due to influence of the moment	$F_{MSd}=M_0*x_{max}/\Sigma x_i^2$
$F_{x,Ed} =$	1,30	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed} = F_{NSd}$
$F_{z,Ed} =$	0,66	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed} = F_{MSd}$
$F_{Ed} =$	1,46	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{F_{x,Ed}^2 + F_{z,Ed}^2}$
$F_{Rdx} =$	96,00	[kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx}=min(F_{bRd1x}, F_{bRd2x})$
$F_{Rdz} =$	96,00	[kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz}=min(F_{bRd1z}, F_{bRd2z})$

$\lvert F_{x,Ed}\rvert \le F_{Rdx}$	|1,30| < 96,00	verified	(0,01)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$	|0,66| < 96,00	verified	(0,01)
$F_{Ed} \le F_{vRd}$	1,46 < 43,43	verified	(0,03)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm^2]	Cross-sectional area of an angle	
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0*t_{f4}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2*A_{net}*f_{u4})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9*A*f_{y4})/\gamma_{M2}$

$\lvert 0.5*N_{b4,Ed}\rvert \le N_{u,Rd}$	|1,30| < 217,53	verified	(0,01)
$\lvert 0.5*N_{b4,Ed}\rvert \le N_{pl,Rd}$	|1,30| < 300,23	verified	(0,00)

Bar verification - block tearing

A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2}$ + $(1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

$\lvert 0.5*N_{b4,Ed}\rvert \le V_{effRd}$	|1,30| < 154,60	verified	(0,01)

Bar 5

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50		Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00		verified	
α_{bx} =	0,82		Coefficient determined by bolt spacing	$\alpha_{bx}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$	0,82 > 0,00		verified	
$F_{b,Rd1x}$ =	134,40	[kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} =	2,50		Coefficient for calculation of $F_{b,Rd}$	$k_{1z}=\min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00		verified	
α_{bz} =	0,82		Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bz}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00		verified	
$F_{b,Rd1z}$ =	134,40	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_1=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2x}$ =	96,00 [kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd2x}$=k_1*α_b*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0	2,50 > 0,00	verified	
α_{bz} =	0,82	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2z}$ =	96,00 [kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	-0,19	[kN*m]	Real bending moment	M_0=$N_{b5,Ed}$*e
F_{NSd} =	-6,37	[kN]	Component force in a bolt due to influence of the longitudinal force	F_{NSd} = $N_{b5,Ed}$/n
F_{MSd} =	-3,24	[kN]	Component force in a bolt due to influence of the moment	F_{MSd}=M_0*x_{max}/Σx_i^2
$F_{x,Ed}$ =	-6,37	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = F_{NSd}
$F_{z,Ed}$ =	-3,24	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = F_{MSd}
F_{Ed} =	7,14	[kN]	Resultant shear force in a bolt	F_{Ed} = $\sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	F_{Rdx}=min(F_{bRd1x}, F_{bRd2x})
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	F_{Rdz}=min(F_{bRd1z}, F_{bRd2z})

$\lvert F_{x,Ed}\rvert \le F_{Rdx}$	|-6,37| < 96,00	verified	(0,07)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$	|-3,24| < 96,00	verified	(0,03)
$F_{Ed} \le F_{vRd}$	7,14 < 43,43	verified	(0,16)

Verification of a section weakened by openings

β_2 =	0,65	Reduction coefficient	[Table 3.8]
A =	940 [mm^2]	Cross-sectional area of an angle	

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0 * t_{f5}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2 * A_{net} * f_{u5})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9 * A * f_{y5})/\gamma_{M2}$

$\lvert 0.5 * N_{b5,Ed} \rvert \leq N_{u,Rd}$	|-6,37| < 217,53	verified	(0,03)
$\lvert 0.5 * N_{b5,Ed} \rvert \leq N_{pl,Rd}$	|-6,37| < 300,23	verified	(0,02)

Bar verification - block tearing

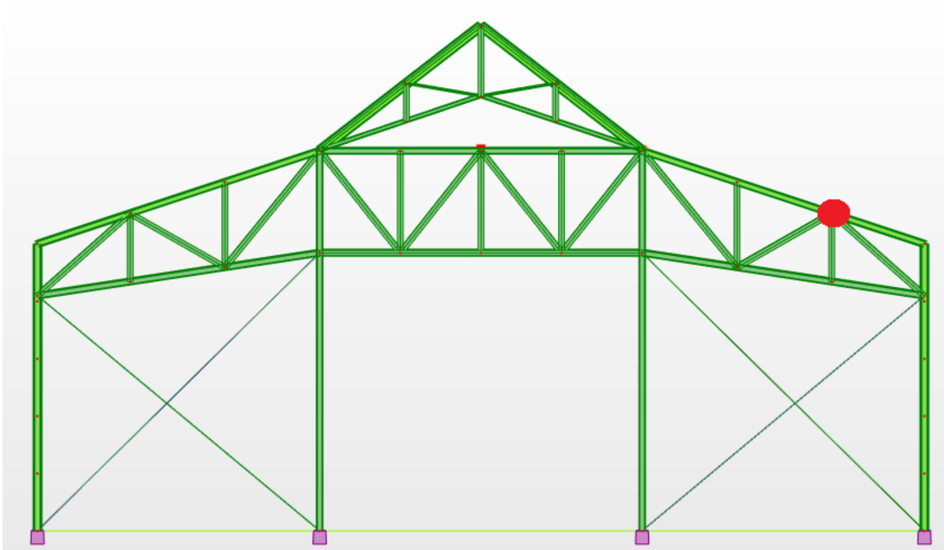
A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd} = 0.5 * f_u * A_{nt}/\gamma_{M2} + (1/\sqrt{3}) * f_y * A_{nv}/\gamma_{M0}$

$\lvert 0.5 * N_{b5,Ed} \rvert \leq V_{effRd}$	|-6,37| < 154,60	verified	(0,04)

Connection conforms to the code	Ratio	0,16

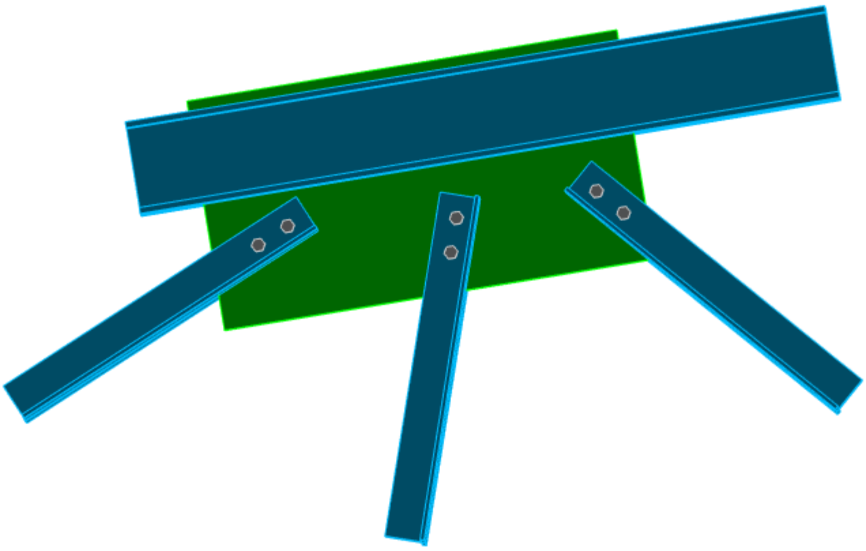
8.4. Čvor rešetke 2

Položaj spoja na konstrukciji prikazan je na slici_ crvenom bojom.¸



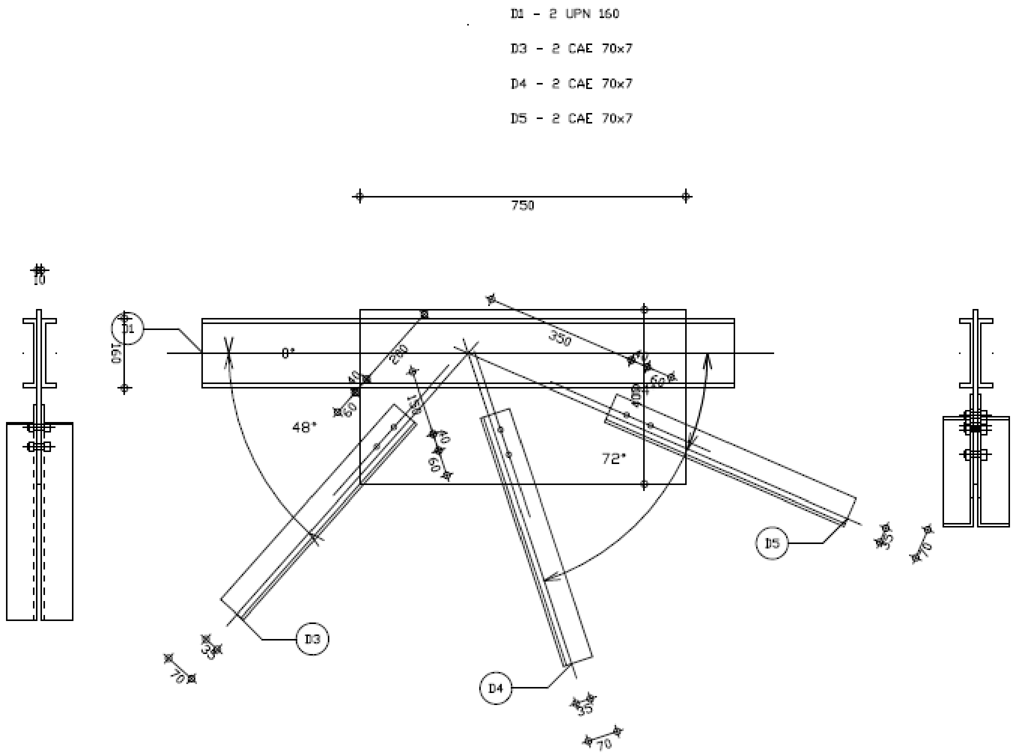
Slika 54. Položaj čvora rešetke 2 na konstrukciji

Na slici _ prikazan je čvor rešetke 2 u 3D-u.



Slika 55. 3D prikaz čvora rešetke 2

	Autodesk Robot Structural Analysis Professional 2019	OK
	Calculations of the connection with the gusset plate	Ratio
	EN 1993-1-8:2005/AC:2009	**0,45**



General

Connection no.: 2

Connection name: Gusset plate: truss chord node

Structure node: 34

Structure bars: 5, 5, 36, 30, 35,

Geometry

Bars

		Bar 1-2		Bar 3	Bar 4	Bar 5	
Bar no.:		5		36	30	35	
Section:		2 UPN 160 spojeni		2 CAE 70x7	2 CAE 70x7	2 CAE 70x7	
	h	160		70	70	70	mm
	b_f	65		70	70	70	mm
	t_w	8		7	7	7	mm
	t_f	11		7	7	7	mm
	r	11		9	9	9	mm

		Bar 1-2		Bar 3	Bar 4	Bar 5	
	A	4779		1879	1879	1879	mm2
Material:		S355		S355	S355	S355	
	f_y	355,00		355,00	355,00	355,00	MPa
	f_u	490,00		490,00	490,00	490,00	MPa
Angle	α	0,0		48,4	71,6	23,5	Deg
Length	l	7,38		3,09	1,71	2,71	m

Bolts

Bar 3

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 4.8 Bolt class

d = 12 [mm] Bolt diameter

d_0 = 13 [mm] Bolt opening diameter

A_s = 84 [mm^2] Effective section area of a bolt

A_v = 113 [mm^2] Area of bolt section

f_{yb} = 320,00 [MPa] Yield point

f_{ub} = 400,00 [MPa] Bolt tensile resistance

n = 2 Number of bolt columns

Bolt spacing 60 [mm]

e_1 = 40 [mm] Distance of the center of gravity of first bolt from the member end

e_2 = 35 [mm] Distance of the axis of bolts from the member edge

e_c = 200 [mm] Distance of the member end from the point of intersection of member axes

Bar 4

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 4.8 Bolt class

d = 12 [mm] Bolt diameter

d_0 = 13 [mm] Bolt opening diameter

A_s = 84 [mm^2] Effective section area of a bolt

A_v = 113 [mm^2] Area of bolt section

f_{yb} = 320,00 [MPa] Yield point

f_{ub} = 400,00 [MPa] Bolt tensile resistance

n = 2 Number of bolt columns

Bolt spacing 60 [mm]

e_1 = 40 [mm] Distance of the center of gravity of first bolt from the member end

e_2 = 35 [mm] Distance of the axis of bolts from the member edge

e_c = 150 [mm] Distance of the member end from the point of intersection of member axes

Bar 5

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	4.8		Bolt class
d =	12	[mm]	Bolt diameter
d_0 =	13	[mm]	Bolt opening diameter
A_s =	84	[mm^2]	Effective section area of a bolt
A_v =	113	[mm^2]	Area of bolt section
f_{yb} =	320,00	[MPa]	Yield point
f_{ub} =	400,00	[MPa]	Bolt tensile resistance
n =	2		Number of bolt columns

Bolt spacing 60 [mm]

e_1 =	40	[mm]	Distance of the center of gravity of first bolt from the member end
e_2 =	35	[mm]	Distance of the axis of bolts from the member edge
e_c =	350	[mm]	Distance of the member end from the point of intersection of member axes

Welds

Member welds

Bar 1-2

l_1 =	100	[mm]	Length 1 of longitudinal fillet weld
l_2 =	100	[mm]	Length 2 of longitudinal fillet weld
a =	3	[mm]	Thickness of longitudinal fillet welds

Gusset plate

l_p =	750	[mm]	Plate length
h_p =	400	[mm]	Plate height
t_p =	10	[mm]	Plate thickness

Parameters

h_1 =	0	[mm]	Cut
v_1 =	0	[mm]	Cut
h_2 =	0	[mm]	Cut
v_2 =	0	[mm]	Cut
h_3 =	0	[mm]	Cut
v_3 =	0	[mm]	Cut
h_4 =	0	[mm]	Cut
v_4 =	0	[mm]	Cut

Center of gravity of the plate with respect to the center of gravity of bars (125;100)

e_V =	100	[mm]	Vertical distance of the plate edge from the point of intersection of member axes
e_H =	250	[mm]	Horizontal distance of the plate edge from the point of intersection of member axes
e_0 =	0	[mm]	Distance to chord axis (horiz.)

Material: S355

f_y =	355,00	[MPa]	Resistance

Material factors

γ_{M0} =	1,00	Partial safety factor	[2.2]
γ_{M2} =	1,25	Partial safety factor	[2.2]

Loads

Case: 25: COMB15 (1+2)*1.35+7*1.50+3*0.75

$N_{b1,Ed}$ =	-38,66	[kN]	Axial force
$N_{b2,Ed}$ =	-11,77	[kN]	Axial force
$N_{b3,Ed}$ =	-34,72	[kN]	Axial force
$N_{b4,Ed}$ =	3,76	[kN]	Axial force
$N_{b5,Ed}$ =	-4,11	[kN]	Axial force

Results

Bar 1-2

Verification of welds

e =	0	[mm]	Axial force eccentricity relative to the centroid of a weld group	
M_0 =	0,00	[kN*m]	Real bending moment	$M_0 = 0.5*N_{b1,Ed}*e$
A_w =	1080	[mm^2]	Area of welds	
I_0 =	6217427	[mm^4]	Polar moment of inertia of welds	
τ_N =	-12,45	[MPa]	Component stress due to influence of the longitudinal force	$\tau_N = 0.5*N_{b1,Ed}/A_s$
τ_{Mx} =	0,00	[MPa]	Component stress due to influence of the moment on the x direction	$\tau_{Mx}=M_0*z/I_0$
τ_{Mz} =	0,00	[MPa]	Component stress due to influence of the moment on the z direction	$\tau_{Mz}=M_0*x/I_0$
τ =	12,45	[MPa]	Resultant stress	$\tau=\sqrt{[(\tau_N+\tau_{Mx})^2+\tau_{Mz}^2]}$
β_w =	0,90		Correlation coefficient	[Table 4.1]
$f_{vw,d}$ =	251,47	[MPa]		$f_{vw,d} = f_u/(\sqrt{3}*\beta_w*\gamma_{M2})$

$\tau \le f_{vRd}$	12,45 < 251,47	verified	(0,05)

Section resistance

A =	2389	[mm^2]	Cross-sectional area	
N_{plRd} =	848,24	[kN]	Design plastic resistance of the gross section	$N_{plRd}=A*f_{y1}/\gamma_{M0}$

$\lvert 0.5*N_{b1,Ed}\rvert \le N_{pl,Rd}$	|-13,45| < 848,24	verified	(0,02)

Bar 3

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd1x}$ =	134,40 [kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0	2,50 > 0,00	verified	
α_{bz} =	0,82	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd1z}$ =	134,40 [kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_1=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2x}$ =	96,00 [kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd2x}$=k_1*α_b*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0	2,50 > 0,00	verified	
α_{bz} =	0,82	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2z}$ =	96,00 [kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	-0,53	[kN*m]	Real bending moment	M_0=$N_{b3,Ed}$*e
F_{NSd} =	–17,36	[kN]	Component force in a bolt due to influence of the longitudinal force	F_{NSd} = $N_{b3,Ed}$/n
F_{MSd} =	-8,85	[kN]	Component force in a bolt due to influence of the moment	F_{MSd}=M_0*x_{max}/Σx_i^2
$F_{x,Ed}$ =	–17,36	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = F_{NSd}
$F_{z,Ed}$ =	-8,85	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = F_{MSd}

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
F_{Ed} =	19,48	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx}=\min(F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz}=\min(F_{bRd1z}, F_{bRd2z})$

$\lvert F_{x,Ed}\rvert \le F_{Rdx}$	|-17,36| < 96,00	verified	(0,18)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$	|-8,85| < 96,00	verified	(0,09)
$F_{Ed} \le F_{vRd}$	19,48 < 43,43	verified	(0,45)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm^2]	Cross-sectional area of an angle	
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0 * t_{f3}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2 * A_{net} * f_{u3})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9 * A * f_{y3})/\gamma_{M2}$

$\lvert 0.5*N_{b3,Ed}\rvert \le N_{u,Rd}$	|-17,36| < 217,53	verified	(0,08)
$\lvert 0.5*N_{b3,Ed}\rvert \le N_{pl,Rd}$	|-17,36| < 300,23	verified	(0,06)

Bar verification - block tearing

A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

$\lvert 0.5*N_{b3,Ed}\rvert \le V_{effRd}$	|-17,36| < 154,60	verified	(0,11)

Bar 4

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	$\alpha_{bx}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$

$\alpha_{bx} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd1x}$ = 134,40 [kN]	Design capacity in the limit state of plastification of the opening wall		$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} = 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00	verified	
α_{bz} = 0,82	Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd1z}$ = 134,40 [kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} = 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_1=min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00	verified	
α_{bx} = 0,82	Coefficient determined by bolt spacing		$\alpha_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd2x}$ = 96,00 [kN]	Design capacity in the limit state of plastification of the opening wall		$F_{b,Rd2x}=k_1*\alpha_b*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} = 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00	verified	
α_{bz} = 0,82	Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd2z}$ = 96,00 [kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	0,06	[kN*m]	Real bending moment	$M_0=N_{b4,Ed}*e$
F_{NSd} =	1,88	[kN]	Component force in a bolt due to influence of the longitudinal force	$F_{NSd} = N_{b4,Ed}/n$
F_{MSd} =	0,96	[kN]	Component force in a bolt due to influence of the moment	$F_{MSd}=M_0*x_{max}/\Sigma x_i^2$
$F_{x,Ed}$ =	1,88	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed} = F_{NSd}$
$F_{z,Ed}$ =	0,96	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed} = F_{MSd}$
F_{Ed} =	2,11	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx}=min(F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz}=min(F_{bRd1z}, F_{bRd2z})$

$\lvert F_{x,Ed}\rvert \le F_{Rdx}$	|1,88| < 96,00	verified	(0,02)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$	|0,96| < 96,00	verified	(0,01)
$F_{Ed} \le F_{vRd}$	2,11 < 43,43	verified	(0,05)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm^2]	Cross-sectional area of an angle	
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0*t_{f4}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2*A_{net}*f_{u4})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9*A*f_{y4})/\gamma_{M2}$

$\lvert 0.5*N_{b4,Ed}\rvert \le N_{u,Rd}$	|1,88| < 217,53	verified	(0,01)
$\lvert 0.5*N_{b4,Ed}\rvert \le N_{pl,Rd}$	|1,88| < 300,23	verified	(0,01)

Bar verification - block tearing

A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

$\lvert 0.5*N_{b4,Ed}\rvert \le V_{effRd}$	|1,88| < 154,60	verified	(0,01)

Bar 5

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50		Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00		verified	
α_{bx} =	0,82		Coefficient determined by bolt spacing	$\alpha_{bx}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$	0,82 > 0,00		verified	
$F_{b,Rd1x}$ =	134,40	[kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} =	2,50		Coefficient for calculation of $F_{b,Rd}$	$k_{1z}=\min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00		verified	
α_{bz} =	0,82		Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bz}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00		verified	
$F_{b,Rd1z}$ =	134,40	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_1=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2x}$ =	96,00 [kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd2x}$=k_1*α_b*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0	2,50 > 0,00	verified	
α_{bz} =	0,82	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2z}$ =	96,00 [kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	–0,06	[kN*m]	Real bending moment	M_0=$N_{b5,Ed}$*e
F_{NSd} =	–2,05	[kN]	Component force in a bolt due to influence of the longitudinal force	F_{NSd} = $N_{b5,Ed}$/n
F_{MSd} =	–1,05	[kN]	Component force in a bolt due to influence of the moment	F_{MSd}=M_0*x_{max}/Σx_i^2
$F_{x,Ed}$ =	–2,05	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = F_{NSd}
$F_{z,Ed}$ =	–1,05	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = F_{MSd}
F_{Ed} =	2,31	[kN]	Resultant shear force in a bolt	F_{Ed} = √($F_{x,Ed}^2$ + $F_{z,Ed}^2$)
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	F_{Rdx}=min(F_{bRd1x}, F_{bRd2x})
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	F_{Rdz}=min(F_{bRd1z}, F_{bRd2z})

$\lvert F_{x,Ed}\rvert \le F_{Rdx}$	|-2,05| < 96,00	verified	(0,02)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$	|-1,05| < 96,00	verified	(0,01)
$F_{Ed} \le F_{vRd}$	2,31 < 43,43	verified	(0,05)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm^2]	Cross-sectional area of an angle	

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0 * t_{f5}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2 * A_{net} * f_{u5})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9 * A * f_{y5})/\gamma_{M2}$

$\lvert 0.5 * N_{b5,Ed} \rvert \leq N_{u,Rd}$	|-2,05| < 217,53	verified	(0,01)
$\lvert 0.5 * N_{b5,Ed} \rvert \leq N_{pl,Rd}$	|-2,05| < 300,23	verified	(0,01)

Bar verification - block tearing

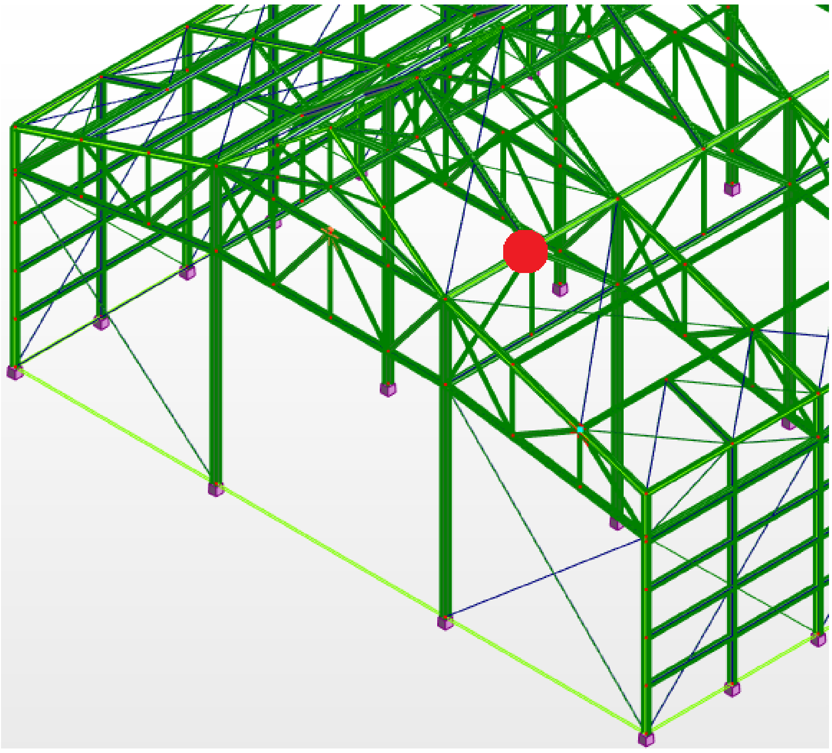
A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd} = 0.5 * f_u * A_{nt}/\gamma_{M2} + (1/\sqrt{3}) * f_y * A_{nv}/\gamma_{M0}$

$\lvert 0.5 * N_{b5,Ed} \rvert \leq V_{effRd}$	|-2,05| < 154,60	verified	(0,01)

Connection conforms to the code	Ratio	0,45

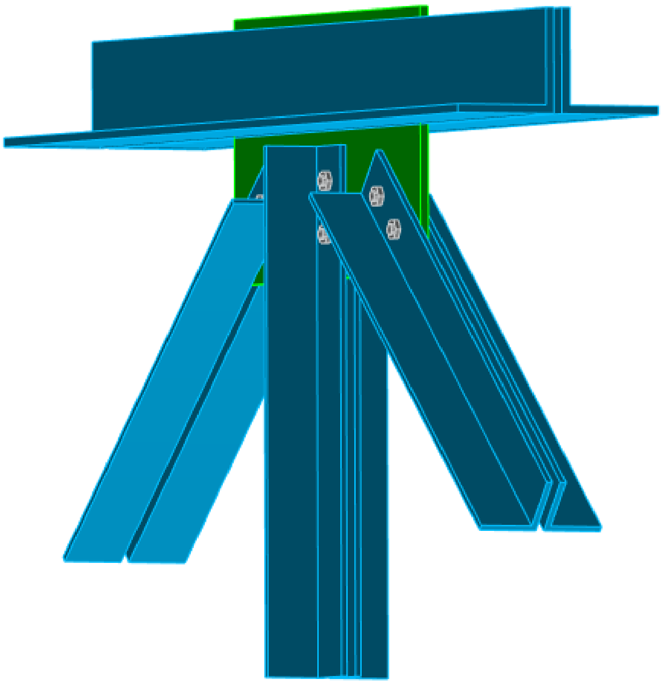
8.5. Čvor rešetke 3

Položaj spoja na konstrukciji prikazan je na slici_ crvenom bojom.



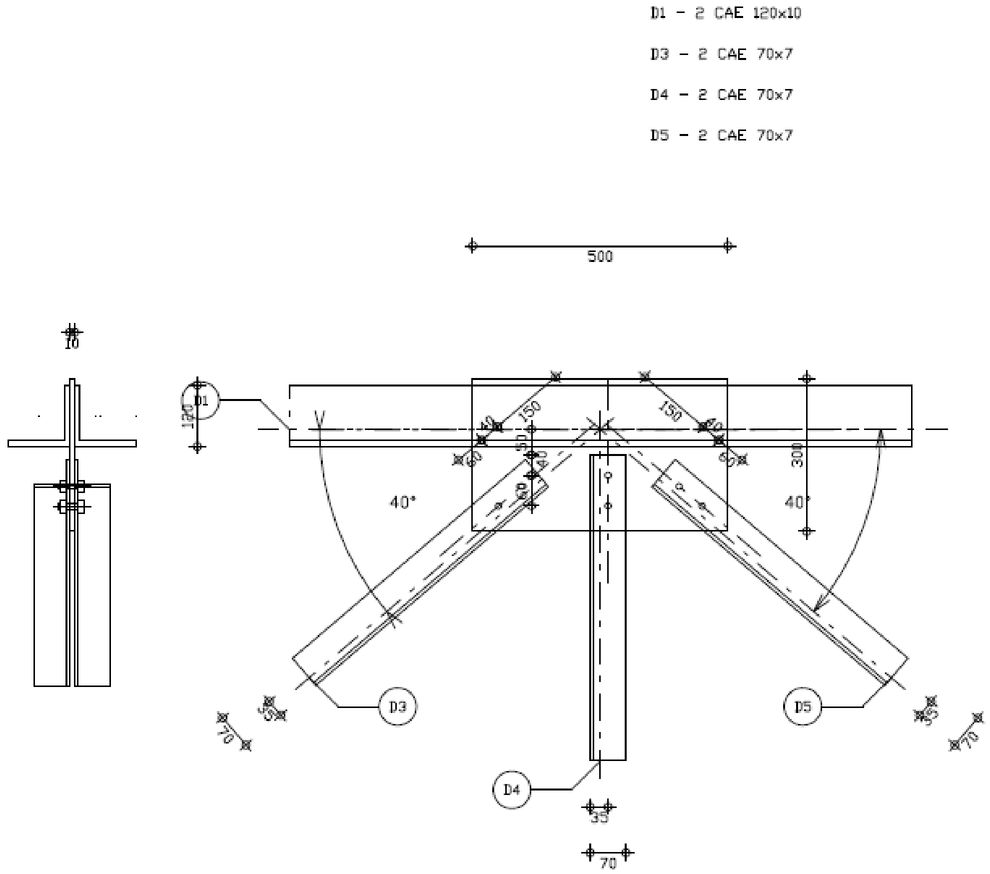
Slika 56. Položaj čvora rešetke 3 na konstrukciji

Na slici _ prikazan je čvor rešetke 3 u 3D-u.



Slika 57. 3D prikaz čvora rešetke 3

	Autodesk Robot Structural Analysis Professional 2019 **Calculations of the connection with the gusset plate** EN 1993-1-8:2005/AC:2009	OK
		Ratio **0,23**



General

Connection no.: 3

Connection name: Gusset plate: truss chord node

Structure node: 73

Structure bars: 78, 78, 86, 85, 87,

Geometry

Bars

		Bar 1-2		Bar 3	Bar 4	Bar 5	
Bar no.:		78		86	85	87	
Section:		2 CAE 120x10		2 CAE 70x7	2 CAE 70x7	2 CAE 70x7	
	h	120		70	70	70	mm
	b_f	120		70	70	70	mm
	t_w	10		7	7	7	mm

		Bar 1-2		Bar 3	Bar 4	Bar 5	
	t_f	10		7	7	7	mm
	r	13		9	9	9	mm
	A	4636		1879	1879	1879	mm2
Material:		S355		S355	S355	S355	
	f_y	355,00		355,00	355,00	355,00	MPa
	f_u	490,00		490,00	490,00	490,00	MPa
Angle	α	0,0		40,5	90,0	40,5	Deg
Length	l	6,00		3,94	2,56	3,94	m

Bolts

Bar 3

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 4.8 Bolt class

d = 12 [mm] Bolt diameter

d_0 = 13 [mm] Bolt opening diameter

A_s = 84 [mm^2] Effective section area of a bolt

A_v = 113 [mm^2] Area of bolt section

f_{yb} = 320,00 [MPa] Yield point

f_{ub} = 400,00 [MPa] Bolt tensile resistance

n = 2 Number of bolt columns

Bolt spacing 60 [mm]

e_1 = 40 [mm] Distance of the center of gravity of first bolt from the member end

e_2 = 35 [mm] Distance of the axis of bolts from the member edge

e_c = 150 [mm] Distance of the member end from the point of intersection of member axes

Bar 4

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 4.8 Bolt class

d = 12 [mm] Bolt diameter

d_0 = 13 [mm] Bolt opening diameter

A_s = 84 [mm^2] Effective section area of a bolt

A_v = 113 [mm^2] Area of bolt section

f_{yb} = 320,00 [MPa] Yield point

f_{ub} = 400,00 [MPa] Bolt tensile resistance

n = 2 Number of bolt columns

Bolt spacing 60 [mm]

e_1 = 40 [mm] Distance of the center of gravity of first bolt from the member end

e_2 = 35 [mm] Distance of the axis of bolts from the member edge

e_c = 50 [mm] Distance of the member end from the point of intersection of member axes

Bar 5

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	4.8		Bolt class
d =	12	[mm]	Bolt diameter
d_0 =	13	[mm]	Bolt opening diameter
A_s =	84	[mm^2]	Effective section area of a bolt
A_v =	113	[mm^2]	Area of bolt section
f_{yb} =	320,00	[MPa]	Yield point
f_{ub} =	400,00	[MPa]	Bolt tensile resistance
n =	2		Number of bolt columns

Bolt spacing 60 [mm]

e_1 =	40	[mm]	Distance of the center of gravity of first bolt from the member end
e_2 =	35	[mm]	Distance of the axis of bolts from the member edge
e_c =	150	[mm]	Distance of the member end from the point of intersection of member axes

Welds

Member welds

Bar 1-2

l_1 =	100	[mm]	Length 1 of longitudinal fillet weld
l_2 =	100	[mm]	Length 2 of longitudinal fillet weld
a =	3	[mm]	Thickness of longitudinal fillet welds

Gusset plate

l_p =	500	[mm]	Plate length
h_p =	300	[mm]	Plate height
t_p =	10	[mm]	Plate thickness

Parameters

h_1 =	0	[mm]	Cut
v_1 =	0	[mm]	Cut
h_2 =	0	[mm]	Cut
v_2 =	0	[mm]	Cut
h_3 =	0	[mm]	Cut
v_3 =	0	[mm]	Cut
h_4 =	0	[mm]	Cut
v_4 =	0	[mm]	Cut

Center of gravity of the plate with respect to the center of gravity of bars (0;50)

e_V =	100	[mm]	Vertical distance of the plate edge from the point of intersection of member axes
e_H =	250	[mm]	Horizontal distance of the plate edge from the point of intersection of member axes
e_0 =	0	[mm]	Distance to chord axis (horiz.)

Material: S355

f_y =	355,00	[MPa]	Resistance

Material factors

γ_{M0} =	1,00	Partial safety factor	[2.2]
γ_{M2} =	1,25	Partial safety factor	[2.2]

Loads

Case: 13: COMB3 (1+2)*1.35+3*1.50+7*0.90

$N_{b1,Ed}$ =	-11,66	[kN]	Axial force
$N_{b2,Ed}$ =	-0,31	[kN]	Axial force
$N_{b3,Ed}$ =	-18,10	[kN]	Axial force
$N_{b4,Ed}$ =	1,73	[kN]	Axial force
$N_{b5,Ed}$ =	-3,01	[kN]	Axial force

Results

Bar 1-2

Verification of welds

e =	27	[mm]	Axial force eccentricity relative to the centroid of a weld group	
M_0 =	-0,15	[kN*m]	Real bending moment	$M_0 = 0.5*N_{b1,Ed}*e$
A_w =	960	[mm^2]	Area of welds	
I_0 =	3798826	[mm^4]	Polar moment of inertia of welds	
τ_N =	-5,92	[MPa]	Component stress due to influence of the longitudinal force	$\tau_N = 0.5*N_{b1,Ed}/A_s$
τ_{Mx} =	-2,47	[MPa]	Component stress due to influence of the moment on the x direction	$\tau_{Mx}=M_0*z/I_0$
τ_{Mz} =	-2,78	[MPa]	Component stress due to influence of the moment on the z direction	$\tau_{Mz}=M_0*x/I_0$
τ =	8,84	[MPa]	Resultant stress	$\tau=\sqrt{[(\tau_N+\tau_{Mx})^2+\tau_{Mz}^2]}$
β_w =	0,90		Correlation coefficient	[Table 4.1]
$f_{vw,d}$ =	251,47	[MPa]		$f_{vw,d} = f_u/(\sqrt{3}*\beta_w*\gamma_{M2})$

$\tau \le f_{vRd}$	8,84 < 251,47	verified	(0,04)

Section resistance

A =	2318	[mm^2]	Cross-sectional area	
N_{plRd} =	822,94	[kN]	Design plastic resistance of the gross section	$N_{plRd}=A*f_{y1}/\gamma_{M0}$

$\lvert 0.5*N_{b1,Ed}\rvert \le N_{pl,Rd}$	|-5,68| < 822,94	verified	(0,01)

Bar 3

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd1x}$ =	134,40 [kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0	2,50 > 0,00	verified	
α_{bz} =	0,82	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd1z}$ =	134,40 [kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_1=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2x}$ =	96,00 [kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd2x}$=k_1*α_b*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50	Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0	2,50 > 0,00	verified	
α_{bz} =	0,82	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0	0,82 > 0,00	verified	
$F_{b,Rd2z}$ =	96,00 [kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	-0,28	[kN*m]	Real bending moment	M_0=$N_{b3,Ed}$*e
F_{NSd} =	-9,05	[kN]	Component force in a bolt due to influence of the longitudinal force	F_{NSd} = $N_{b3,Ed}$/n
F_{MSd} =	-4,61	[kN]	Component force in a bolt due to influence of the moment	F_{MSd}=M_0*x_{max}/Σx_i^2
$F_{x,Ed}$ =	-9,05	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = F_{NSd}
$F_{z,Ed}$ =	-4,61	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = F_{MSd}

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
F_{Ed} =	10,15	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx}=\min(F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz}=\min(F_{bRd1z}, F_{bRd2z})$

$\lvert F_{x,Ed}\rvert \leq F_{Rdx}$	|-9,05| < 96,00	verified	(0,09)
$\lvert F_{z,Ed}\rvert \leq F_{Rdz}$	|-4,61| < 96,00	verified	(0,05)
$F_{Ed} \leq F_{vRd}$	10,15 < 43,43	verified	(0,23)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm²]	Cross-sectional area of an angle	
A_{net} =	849	[mm²]	Net cross-sectional area	$A_{net} = A - d_0*t_{f3}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2*A_{net}*f_{u3})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9*A*f_{y3})/\gamma_{M2}$

$\lvert 0.5*N_{b3,Ed}\rvert \leq N_{u,Rd}$	|-9,05| < 217,53	verified	(0,04)
$\lvert 0.5*N_{b3,Ed}\rvert \leq N_{pl,Rd}$	|-9,05| < 300,23	verified	(0,03)

Bar verification - block tearing

A_{nt} =	200	[mm²]	Net area of the section in tension	
A_{nv} =	564	[mm²]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

$\lvert 0.5*N_{b3,Ed}\rvert \leq V_{effRd}$	|-9,05| < 154,60	verified	(0,06)

Bar 4

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50	Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00	verified	
α_{bx} =	0,82	Coefficient determined by bolt spacing	$\alpha_{bx}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$

$\alpha_{bx} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd1x}$ = 134,40 [kN]	Design capacity in the limit state of plastification of the opening wall		$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} = 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=\min[2.8*(e_1/d_0)-1.7,\ 1.4*(p_1/d_0)-1.7,\ 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00	verified	
α_{bz} = 0,82	Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_2/(3*d_0),\ f_{ub}/f_u,\ 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd1z}$ = 134,40 [kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} = 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_1=\min[2.8*(e_2/d_0)-1.7,\ 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00	verified	
α_{bx} = 0,82	Coefficient determined by bolt spacing		$\alpha_{bx}=\min[e_1/(3*d_0),\ p_1/(3*d_0)-0.25,\ f_{ub}/f_u,\ 1]$
$\alpha_{bx} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd2x}$ = 96,00 [kN]	Design capacity in the limit state of plastification of the opening wall		$F_{b,Rd2x}=k_1*\alpha_b*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} = 2,50	Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=\min[2.8*(e_1/d_0)-1.7,\ 1.4*(p_1/d_0)-1.7,\ 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00	verified	
α_{bz} = 0,82	Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_2/(3*d_0),\ f_{ub}/f_u,\ 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00	verified	
$F_{b,Rd2z}$ = 96,00 [kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	0,03	[kN*m]	Real bending moment	$M_0=N_{b4,Ed}*e$
F_{NSd} =	0,86	[kN]	Component force in a bolt due to influence of the longitudinal force	$F_{NSd} = N_{b4,Ed}/n$
F_{MSd} =	0,44	[kN]	Component force in a bolt due to influence of the moment	$F_{MSd}=M_0*x_{max}/\Sigma x_i^2$
$F_{x,Ed}$ =	0,86	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed} = F_{NSd}$
$F_{z,Ed}$ =	0,44	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed} = F_{MSd}$
F_{Ed} =	0,97	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{F_{x,Ed}^2 + F_{z,Ed}^2}$
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx}=\min(F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz}=\min(F_{bRd1z}, F_{bRd2z})$

$\lvert F_{x,Ed}\rvert \le F_{Rdx}$	|0,86| < 96,00	verified	(0,01)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$	|0,44| < 96,00	verified	(0,00)
$F_{Ed} \le F_{vRd}$	0,97 < 43,43	verified	(0,02)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm^2]	Cross-sectional area of an angle	
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0 * t_{f4}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2 * A_{net} * f_{u4})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9 * A * f_{y4})/\gamma_{M2}$

$\lvert 0.5 * N_{b4,Ed}\rvert \le N_{u,Rd}$	|0,86| < 217,53	verified	(0,00)
$\lvert 0.5 * N_{b4,Ed}\rvert \le N_{pl,Rd}$	|0,86| < 300,23	verified	(0,00)

Bar verification - block tearing

A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5 * f_u * A_{nt}/\gamma_{M2}$ + $(1/\sqrt{3}) * f_y * A_{nv}/\gamma_{M0}$

$\lvert 0.5 * N_{b4,Ed}\rvert \le V_{effRd}$	|0,86| < 154,60	verified	(0,01)

Bar 5

Bolt capacities

$F_{v,Rd}$ =	43,43	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd} = 0.6 * f_{ub} * A_v * m/\gamma_{M2}$

Bolt bearing on the bar

Direction x

k_{1x} =	2,50		Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8 * (e_2/d_0)-1.7,\ 2.5]$
$k_{1x} > 0.0$	2,50 > 0,00		verified	
α_{bx} =	0,82		Coefficient determined by bolt spacing	$\alpha_{bx}=\min[e_1/(3 * d_0),\ p_1/(3 * d_0)-0.25,\ f_{ub}/f_u,\ 1]$
$\alpha_{bx} > 0.0$	0,82 > 0,00		verified	
$F_{b,Rd1x}$ =	134,40	[kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd1x}=k_{1x} * \alpha_{bx} * f_u * d * t_i/\gamma_{M2}$

Direction z

k_{1z} =	2,50		Coefficient for calculation of $F_{b,Rd}$	$k_{1z}=\min[2.8 * (e_1/d_0)-1.7,\ 1.4 * (p_1/d_0)-1.7,\ 2.5]$
$k_{1z} > 0.0$	2,50 > 0,00		verified	
α_{bz} =	0,82		Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bz}=\min[e_2/(3 * d_0),\ f_{ub}/f_u,\ 1]$
$\alpha_{bz} > 0.0$	0,82 > 0,00		verified	
$F_{b,Rd1z}$ =	134,40	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}=k_{1z} * \alpha_{bz} * f_u * d * t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} =	2,50		Coefficient for calculation of $F_{b,Rd}$	k_1=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0		2,50 > 0,00	verified	
α_{bx} =	0,82		Coefficient determined by bolt spacing	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0		0,82 > 0,00	verified	
$F_{b,Rd2x}$ =	96,00	[kN]	Design capacity in the limit state of plastification of the opening wall	$F_{b,Rd2x}$=k_1*α_b*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2,50		Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1z} > 0.0		2,50 > 0,00	verified	
α_{bz} =	0,82		Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bz} > 0.0		0,82 > 0,00	verified	
$F_{b,Rd2z}$ =	96,00	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Verification of a connection due to forces acting on bolts

Bolt shear

e =	15	[mm]	Axial force eccentricity relative to the bolt axis	
M_0 =	–0,05	[kN*m]	Real bending moment	M_0=$N_{b5,Ed}$*e
F_{NSd} =	–1,51	[kN]	Component force in a bolt due to influence of the longitudinal force	F_{NSd} = $N_{b5,Ed}$/n
F_{MSd} =	–0,77	[kN]	Component force in a bolt due to influence of the moment	F_{MSd}=M_0*x_{max}/Σx_i^2
$F_{x,Ed}$ =	–1,51	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = F_{NSd}
$F_{z,Ed}$ =	–0,77	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = F_{MSd}
F_{Ed} =	1,69	[kN]	Resultant shear force in a bolt	F_{Ed} = √($F_{x,Ed}^2$ + $F_{z,Ed}^2$)
F_{Rdx} =	96,00	[kN]	Effective design capacity of a bolt on the direction x	F_{Rdx}=min(F_{bRd1x}, F_{bRd2x})
F_{Rdz} =	96,00	[kN]	Effective design capacity of a bolt on the direction z	F_{Rdz}=min(F_{bRd1z}, F_{bRd2z})

|$F_{x,Ed}$| ≤ F_{Rdx}	|-1,51| < 96,00	verified	(0,02)
|$F_{z,Ed}$| ≤ F_{Rdz}	|-0,77| < 96,00	verified	(0,01)
F_{Ed} ≤ F_{vRd}	1,69 < 43,43	verified	(0,04)

Verification of a section weakened by openings

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A =	940	[mm²]	Cross-sectional area of an angle	

β_2 =	0,65		Reduction coefficient	[Table 3.8]
A_{net} =	849	[mm^2]	Net cross-sectional area	$A_{net} = A - d_0 \cdot t_{f5}$
$N_{u,Rd}$ =	217,53	[kN]	Design plastic resistance of the net section	$N_{u,Rd} = (\beta_2 \cdot A_{net} \cdot f_{u5})/\gamma_{M2}$
$N_{pl,Rd}$ =	300,23	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd} = (0.9 \cdot A \cdot f_{y5})/\gamma_{M2}$

$\lvert 0.5 \cdot N_{b5,Ed} \rvert \le N_{u,Rd}$	|-1,51| < 217,53	verified	(0,01)
$\lvert 0.5 \cdot N_{b5,Ed} \rvert \le N_{pl,Rd}$	|-1,51| < 300,23	verified	(0,01)

Bar verification - block tearing

A_{nt} =	200	[mm^2]	Net area of the section in tension	
A_{nv} =	564	[mm^2]	Area of the section in shear	
V_{effRd} =	154,60	[kN]	Design capacity of a section weakened by openings	$V_{effRd}=0.5 \cdot f_u \cdot A_{nt}/\gamma_{M2} + (1/\sqrt{3}) \cdot f_y \cdot A_{nv}/\gamma_{M0}$

$\lvert 0.5 \cdot N_{b5,Ed} \rvert \le V_{effRd}$	|-1,51| < 154,60	verified	(0,01)

Connection conforms to the code	Ratio	0,23

9. ZAKLJUČAK

Dimenzioniranje konstrukcije provedeno je u skladu sa Eurokodom i pripadnim Nacionalnim dodacima. Proračun je proveden u programskom paketu *Autodesk Robot Structural Analysis Professional 2019* [1] što omogućava vrlo jednostavnu optimizaciju poprečnih presjeka.

Nosiva konstrukcija ima relativno velike raspone zbog čega su glavni i sekundarni nosači izvedeni u obliku rešetki, a dimenzije poprečnih presjeka prilično velike. Svi elementi dimenzionirani su tako da zadovolje uvjete graničnog stanja nosivosti i graničnog stanja uporabivosti. Nastojalo se da iskoristivost poprečnih presjeka bude što veća, te ista na nekim mjestima premašuje 90%. Unatoč tome na nekim mjestima je iskoristivost presjeka svega oko 30%, što je posljedica velikih raspona koji utječu na progibe, odnosno na provjeru graničnog stanja uporabivosti. Razmatranjam rezultata može se zaključiti da provjera graničnog stanja uporabivosti može uvelike utjecati na dimenzioniranje elemenata, te je često upravo ona mjerodavna.

LITERATURA

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