Dimenzioniranje čelične konstrukcije namjene sportsko rekreativnog centra

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Diplomski rad

Rijeka, 2024.

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GRAĐEVINSKI FAKULTET

Sveučilišni diplomski studij Građevinarstvo

Smjer: konstrukcije

Naziv kolegija: Čelične konstrukcije

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JMBAG: 0114031419

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IZJAVA

Završni/Diplomski rad izradio/izradila sam samostalno, u suradnji s mentorom/mentoricom i uz poštivanje pozitivnih građevinskih propisa i znanstvenih dostignuća iz područja građevinarstva. Građevinski fakultet u Rijeci je nositelj prava intelektualnog vlasništva u odnosu na ovaj rad.

Roko Požgaj

U Rijeci, srpanj 2024.

ZAHVALA

Želim izraziti iskrenu zahvalnost svom mentoru izv.prof.dr.sc. Mladenu Buliću mag.ing.aedif. za sveobuhvatnu podršku, vodstvo i stručne savjete tijekom izrade diplomskog rada. Također, zahvaljujem svim profesorima koji su mi svojim znanjem i voljom pomogli da postignem svoje akademske ciljeve.

SAŽETAK

Ovaj diplomski rad bavi se analizom i dimenzioniranjem elemenata čelične konstrukcije prema Eurocode-u 3, HRN EN 1993. za sportsko rekreativni centar u Novom Vinodolskom. Građevina ima vanjske gabarite širine 8,0 m i dužine 15,0 m. Krovna streha je jednostrešna s nagibom krovne konstrukcije od 2,0°. Visina zidne plohe varira od 6,0 m do 6,3 m. Nadmorska visina lokacije građevine iznosi 0,00 metara nadmorske visine.

Primarni ciljevi rada uključuju detaljnu analizu konstrukcije po Eurocode-u 1 te opisivanje građevinskih elemenata objekta koji će se koristiti u izgradnji, a da pritom objekt bude izveden u određenom roku i budžetu.

Statička analiza konstrukcije izvedena je korištenjem softvera Autodesk Robot structural Analysis professional.

Ovaj projekt dokazuje da projektiranje čeličnih konstrukcija s odabranim elementima zadovoljava sve zakonske i tehničke uvjete, a predloženi materijali osiguravaju dugovječnost i stabilnost objekta.

Ključne riječi: čelična konstrukcija, dimenzioniranje, statička analiza, građevinski elementi, Autodesk Robot structural Analysis professional

SUMMARY

This thesis deals with the analysis and dimensioning of steel structure elements according to Eurocode 3, HRN EN 1993, for a sports and recreational center in Novi Vinodolski. The building has external dimensions of 8.0 m in width and 15.0 m in length. The roof is a single-pitch roof with a slope of 2.0°. The height of the wall surface varies from 6.0 m to 6.3 m. The altitude of the building's location is 0.00 meters above sea level.

The primary objectives of the thesis include a detailed structural analysis according to Eurocode 1, describing the construction elements of the building that will be used in the construction, while ensuring the project is completed within a specified timeframe and budget. The static analysis of the structure was performed using Autodesk Robot Structural Analysis Professional software.

This project demonstrates that the designed steel structure with the selected elements meets all legal and technical requirements, and the proposed materials ensure the durability and stability of the building.

Keywords: steel structure, dimensioning, static analysis, construction elements, Autodesk Robot Structural Analysis Professional

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1. TEHNIČKI OPIS

1.1. Opis građevine

Sportsko rekreativni centar nalazi se u Novom Vinodolskom i ima vanjske gabarite širine 8,0 m i dužine 15,0 m. Krovna streha je jednostrešna s nagibom od 2,0°, a visina zidne plohe varira od 6,0 m do 6,3 m. Građevina je smještena na nadmorskoj visini od 0,00 metara.

1.2. Konstrukcijski elementi

Glavna nosiva konstrukcija:

Glavni nosivi elementi koji obuhvaćaju gredne nosače i glavne stupove izrađeni su od čeličnih profila HEA 160 prema Eurocode 3 (HRN EN 1993).

Statička analiza konstrukcije izvedena je korištenjem softvera Autodesk Robot structural Analysis professional.

Sekundarni stupovi:

Sekundarni stupovi izvedeni su od profila IPE 240, odabranih zbog svojih mehaničkih svojstava i sposobnosti da podrže dodatna opterećenja.

Podrožnice:

Za podrožnice je odabran IPE 160 profil kao ključni element u raspodjeli opterećenja s krova na glavne nosače.

Tetive stubišta i sekundarni nosači stubišta:

Ovi elementi, također izvedeni od profila IPE 160, omogućuju sigurnu i stabilnu upotrebu stubišta čak i pod velikim opterećenjima.

Vjetrovni vezovi:

Za dodatnu stabilizaciju konstrukcije koriste se vjetrovni vezovi od šipke okruglog punog profila promjera 10 mm.

Fasadni nosači:

Fasadni nosači izvedeni od profila UAP 80 pružaju čvrstu osnovu za pričvršćivanje fasadnih elemenata.

Temelji:

Stupovi su zglobno oslonjeni na plitke temelje samce izvedene od armiranog betona i dimenzionirane prema Eurocode-u 2 (HRN EN 1992). Povezani su armiranobetonskom pločom debljine 10 cm kako bi se ostvarila monolitnost konstrukcije, a projektirani su tako da osiguraju stabilnost i nosivost cijele konstrukcije.

Krovna konstrukcija:

Krov je jednostrešni s nagibom od 2,0°, a izrađen je od čeličnih nosača i pokriven sendvič panelima debljine 8 cm.

1.3. Materijali

Koristi se kvalitetni konstrukcijski čelik prema normi EN 10025. Svi čelični elementi izrađeni su od čelika S275. Kod priključka temelja i stupa koriste se 2 ankera M16 k.v. 4.6 i beton klase C 25/30 s armaturnim košom klase B500B. Za ostale priključke se koriste vijci M16 k.v. 10.9. Na temelju se nalazi nadozid također klase C 25/30, visine 30 cm kao oslonac fasadnih elemenata koji ujedno prihvaća težinu same fasade.

1.4. Opterećenja

Stalna opterećenja uključuju težinu konstrukcije, krovnog pokrova i instalacija. Promjenjiva opterećenja obuhvaćaju korisna opterećenja (ljudi, oprema) te snijeg i vjetar prema Eurocode-u 1 (HRN EN 1991).

1.5. Konstrukcijska analiza

Statička analiza izvedena je korištenjem softvera Autodesk Robot Structural Analysis Professional, čime je osigurana preciznost i pouzdanost proračuna. Analiza je obuhvatila provjeru svih čeličnih elemenata na savijanje, smicanje, pritisak i torziju, dok je ručna analiza obuhvatila provjeru pojedinih elemenata.

2. PROGRAM KONTROLE I OSIGURANJA PROJEKTA

Izrada čelične konstrukcije mora se povjeriti onom izvođaču koji ima odgovarajuće reference već izvedenih sličnih objekata. U tehničkoj dokumentaciji predviđena je vrsta i kvaliteta materijala od kojeg treba izraditi konstrukciju. Odstupanja u kvaliteti materijala može odobriti jedino projektant konstrukcije, a izvođač radova dužan je prije početka radova predočiti nadzornom inženjeru sljedeću važeću dokumentaciju:

- Uvjerenja o kvaliteti osnovnog i dodatnog materijala, sredstava za spajanje te sredstva za antikorozijsku zaštitu
- Uvjerenje o podobnosti pogona za izvođenje zavarivačkih radova
- Uvjerenja zavarivača koji će raditi na izradi konstrukcije za vrstu zavarivačkih radova koji će se primjenjivati za traženu debljinu, materijal i položaj zavarivanja
- Specifikacije postupka zavarivanja i odobrenje o primjeni postupka 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 pristupanja radovima na montaži potrebno je predočiti odobreni Projekt montaže.

Navedena dokumentacija dio je dokumentacije za tehnički pregled konstrukcije. Tijekom izrade i montaže konstrukcije izvođač radova dužan je voditi zakonom propisane dnevnike, koje je uz internu kontrolu izvođača dužan ovjeriti i nadzorni inženjer. Ako se materijal za izradu konstrukcije nabavlja i tijekom izrade čelične konstrukcije, potrebno je nadzornom organu staviti na uvid odgovarajuća uvjerenja o kvaliteti. Prije isporuke konstrukcije na gradilište vrši se prijem konstrukcije u radionici uz pribavljenu dokumentaciju o kvaliteti. O prijemu konstrukcije sastavlja se zapisnik koji ovjeravaju svi sudionici izgradnje, a to su investitor, izvođač radova u radionici, nadzorni inženjer te predstavnik izvođača radova na montaži konstrukcije.

Potrebno je pridržavati se normi i propisa navedenih u projektu te poštivati pravila izvedbe.

Prilikom rezanja materijala treba paziti na mogućnost pojave lokalnih zareza, naročito kod vlačno napregnutih elemenata. Svaki uočeni zarez je potrebno izbrusiti i/ili dovariti.

Elemente konstrukcije potrebno je izraditi u skladu sa specifikacijama, crtežima i uputama iz ovog dijela projekta.

Materijali za izradu konstrukcije navedeni su u statičkom proračunu.

Antikorozijsku zaštitu smije se nanositi strogo prema zahtjevima projekta i propisa, a posebnu važnost treba obratiti na vlažnost zraka i temperaturu.

Protupožarnu zaštitu smije se nanositi strogo prema zahtjevima projekta, propisa i uputama proizvođača.

Prijem elemenata čelične konstrukcije u radionici obavlja se prije isporuke na gradilište na temelju radioničkih nacrta i specifikacija.

LISTA PRISTANKA - COMPLIANCE LIST

	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	ENV 1993-1-1		0	
1.2	Eurocode 4	prEN 1994-1-1		0	
1.3	Eurocode 8	prEN 1998-1		0	
2.0	DJELOVANJA				
2.1	Vjetar	Brzina vjetra 120 km/h	Po cijeloj visini	Y	
2.2	Snijeg	200 m.n.v. II. zona		Y	
2.3	Promjena temperature	+/– 35°C		Y	
2.4	Potres	VIII. zona		Y	
3.0	OSNOVNI MATERIJAL				
3.1	Konstrukcijski čelik	EN 10025		0	
3.2	Kvalitetna grupa	S 235 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	podrazumijeva se ali bez navoda nar.

		Odobrenje za:	Potpis
	LISTA PRISTANKA	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 – kriteriji 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 - vizuelni 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	ISO 13929 klasa C		0	
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	

Ref.			
		Y	pristanak naručioca
		N	bez pristanka
		P	djelomična sugl.
		0	podrazumijeva 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
9.0	DOKUMENTACIJA ZA OSIGURANJE KVALITETE				
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	podrazumijeva se ali bez navoda nar.

3. ANALIZA OPTEREĆENJA

3.1. Opterećenja

3.1.1. Stalno opterećenje na GNK

-sendvič paneli (aluminij – 80mm) [1] ....0,26 kN/m^2

-instalacije...0,10 kN/m^2

$\sum = 0{,}36\ kN/m^2$

3.1.2. Stalno opterećenje za stubište

-gazišta koja se oslanjaju na tetive..................0,20 kN/m^2

3.1.3. Stalno opterećenje za galeriju

-slojevi poda...0,50 kN/m^2

3.1.4. Uporabno opterećenje za stubište i galeriju

-slojevi poda...2,0 kN/m^2

3.1.5. Opterećenje snijegom

Učinak snijega na konstrukciju utvrđen je na temelju:

Slika 1: Karta snježnih zona[2]

U našem slučaju, imamo samo jedno opterećenje snijegom na krov [2]:

$$s = s_k * C_e * C_t * \mu_i$$

Odabrane vrijednosti za naše područje:

$$\mu_i = 0{,}8$$

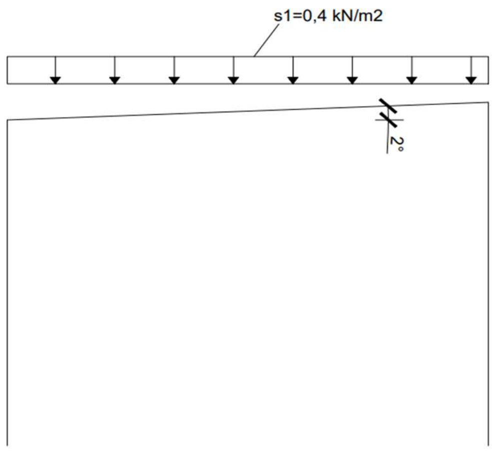
$$s_k = 0{,}5\ kN/m^2$$

$$C_e = 1$$

$$C_t = 1$$

Iz jednadžbe slijedi:

$$s = 0{,}5 * 1 * 1 * 0{,}8 = \mathbf{0{,}40}\ kN/m^2$$



Slika 2: Vrijednosti opterećenja snijega na krov [izradio autor]

3.1.6. Djelovanje vjetrom

Tlak vjetra na vanjske površine [3]:

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

Tlak vjetra na unutarnje površine [3]:

$$w_i = q_p(z_i) * c_{pi}$$

Slika 3: Karta osnovne brzine vjetra $v_{b,0}$[3]

Tlak pri osnovnoj brzini [3]:

$$q_b = \frac{1}{2} * \rho * v_b^2$$

$v_b = c_{season} * c_{dir} * v_{b0}$(m/s)

$c_{dir} = 1$

c_{season}=1

$v_{b0} = 35\ m/s$ => $v_b = 35\ m/s$

Usvajamo da je gustoća zraka $\rho = 1{,}25\ kg/m^3$

$$q_b = \frac{1}{2} * 1{,}25 * 35^2 = 0{,}77\ kN/m^2$$

Iz Tablice 1 određujemo kategoriju i parametre terena.

Tablica 1: Kategorije i parametri terena[3]

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 prepreka	0,05	2
III	Područja sa stalnim pokrovom od vegetacije ili zgrada ili područja s izoliranim preprekama s razmakom najviše 20 visina prepreka (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

$$k_r = 0{,}19 * \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07} = 0{,}19 * \left(\frac{0{,}3}{0{,}005}\right)^{0{,}07} = 0{,}215$$

Za teren III. kategorije vrijedi: $z_{min} < z < z_{max}$; $5\ m < 6{,}3\ m < 200\ m$

$$c_r(z) = k_r * \ln\left(\frac{z}{z_0}\right) = 0{,}215 * ln\left(\frac{6{,}3}{0{,}3}\right) = 0{,}65$$

$$v_m(z) = c_r(z) * c_0(z) * v_b = 0{,}65 * 1 * 35 = 22{,}75\ m/s$$

$$I_v(z) = \frac{k_1}{c_0(z) * \ln\left(\frac{z}{z_0}\right)} = \frac{1}{1 * ln\left(\frac{6{,}3}{0{,}3}\right)} = 0{,}33$$

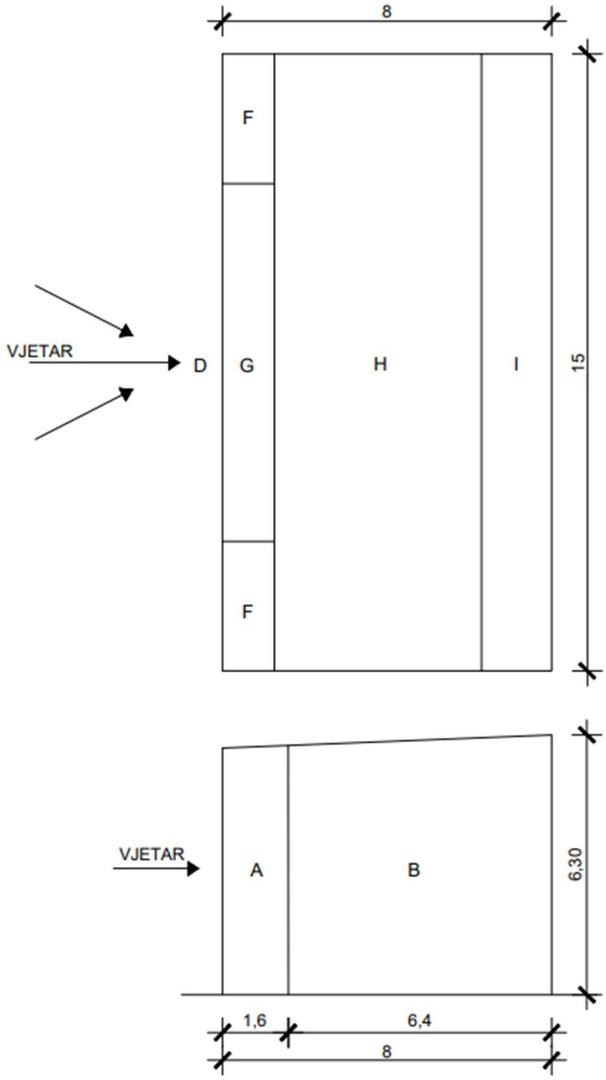
Vršni pritisak brzine [3]:

$$q_p = (1 + 7 * I_v(z)) * q_b$$

$$q_p = (1 + 7 * 0{,}33) * 0{,}77 = \mathbf{1{,}07}\ kN/m^2$$

3.1.6.1. Određivanje vanjskog transverzalnog pritiska vjetra sa zatvorenim otvorima (Θ=0)

<u>Vertikalne površine:</u>



Slika 4: Prikaz zona na vertikalnim površinama pri poprečnom djelovanju vjetra [izradio autor]

Površine vjetrovnih zona:

$$P_A = 6{,}30 * 1{,}6 = 10{,}08\ m^2$$

$$P_B = 6{,}3 * 6{,}4 = 40{,}32\ m^2$$

$$P_D = 6{,}3 * 15 = 94{,}5\ m^2$$

$$P_E = 6{,}3 * 15 = 94{,}5\ m^2$$

Linearnom interpolacijom, koeficijenti vanjskog pritiska c_{pe} na vertikalne zidove za:

$$\frac{h}{d} = \frac{6,3}{8} = 0,79$$

Iznose:

PODRUČJE	A	B	D	E
$c_{pe,10}$	-1,2	-0,8	+0,77	-0,44

Jednadžba opterećenja vjetra na vanjske površine glasi [3]:

$$w = w_{e,i} - w_i = q_p(z) * (c_{pe} - c_{pi})$$

Unutrašnji koeficijent pritiska pri zatvorenim otvorima; $c_{pi} = 0$

Opterećenje na vertikalne plohe uslijed djelovanja vjetra:

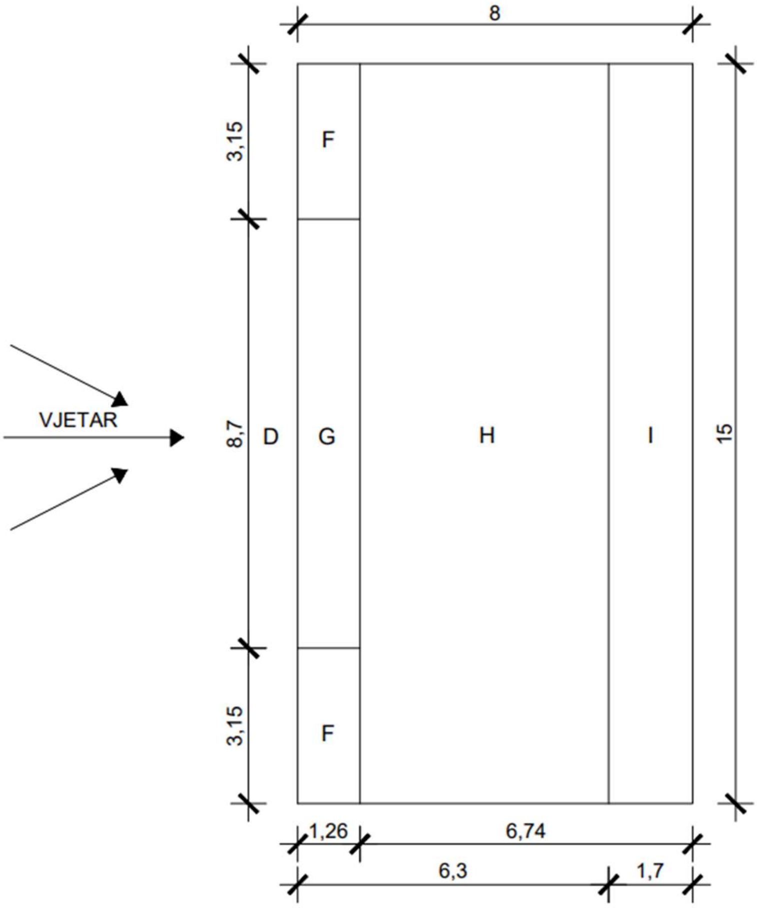
$w_A = w_{e,A} - w_i = 1,07 * (-1,2 - 0) = -1,28$

$w_B = w_{e,B} - w_i = 1,07 * (-0,8 - 0) = -0,86$

$w_D = w_{e,D} - w_i = 1,07 * (0,77 - 0) = +0,82$

$w_E = w_{e,E} - w_i = 1,07 * (-0,44 - 0) = -0,47$

Krovne površine:



Slika 5: Raspodjela zona na krovu pri poprečnom djelovanju vjetra [izradio autor]

Površine vjetrovnih zona:

$$P_F = 1{,}26 * 3{,}15 = 3{,}97\ m^2$$

$$P_G = 1{,}26 * 8{,}7 = 10{,}96\ m^2$$

$$P_H = 5{,}04 * 15 = 75{,}60\ m^2$$

$$P_I = 1{,}7 * 15 = 25{,}50\ m^2$$

Linearnom interpolacijom, koeficijenti vanjskog pritiska c_{pe} na vertikalne zidove za h/d=0,79 iznose:

PODRUČJE	F	G	H	I
c_{pe}	-1,80	-1,20	-0,7	-0,2

Jednadžba opterećenja vjetra na vanjske površine glasi [3]:

$$w = w_{e,i} - w_i = q_p(z) * (c_{pe} - c_{pi})$$

Unutrašnji koeficijent pritiska pri zatvorenim otvorima; $c_{pi} = 0$

Opterećenje na krovne plohe uslijed djelovanja vjetra:

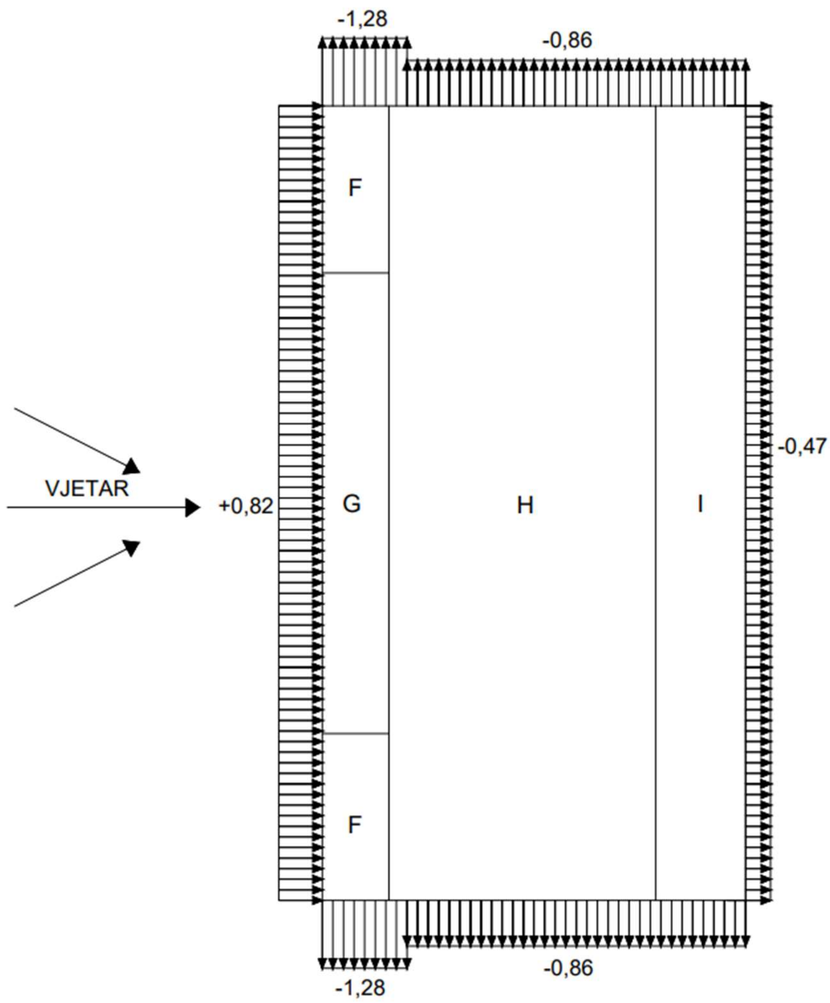
$w_F = w_{e,A} - w_i = 1{,}07 * (-1{,}8 - 0) = -1{,}93$

$w_G = w_{e,B} - w_i = 1{,}07 * (-1{,}2 - 0) = -1{,}28$

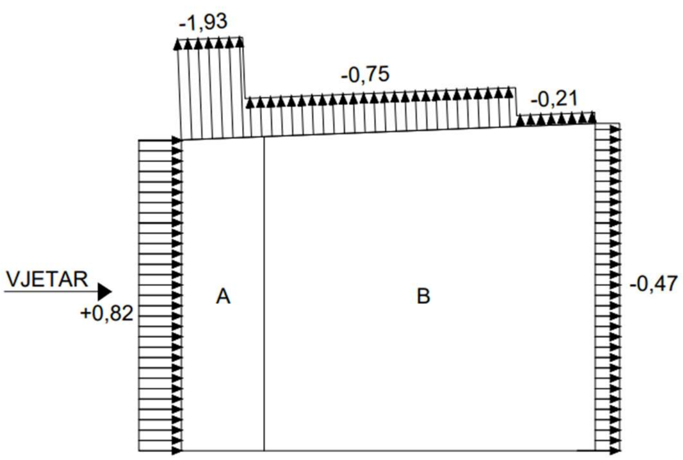
$w_H = w_{e,D} - w_i = 1{,}07 * (-0{,}7 - 0) = -0{,}75$

$w_I = w_{e,E} - w_i = 1{,}07 * (-0{,}2 - 0) = -0{,}21$

Shematski prikaz opterećenja vjetra (Θ=0):



Slika 6: Djelovanje vjetra za slučaj 1[izradio autor]



Slika 7: Mjerodavni presjek [izradio autor]

3.1.6.2. Određivanje vanjskog transverzalnog pritiska vjetra s otvorenim otvorima (Θ=0)

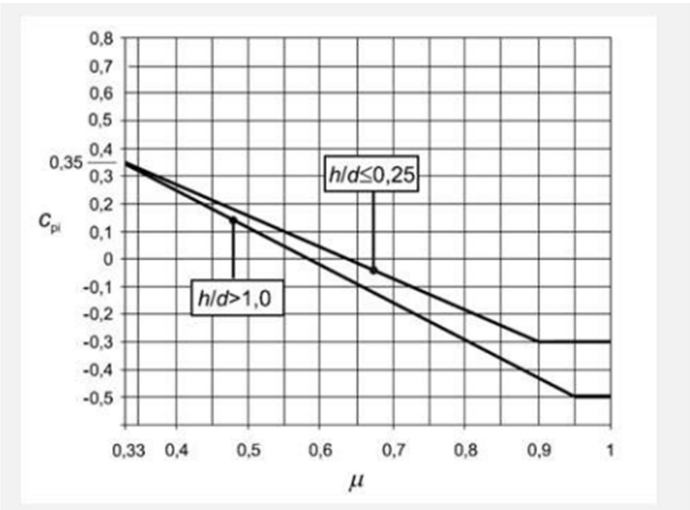
Dimenzije otvora:

Kraće stranice: 1 otvor za ulaz/izlaz ljudi (2,0 x 1,0 m), 5 prozora (1,6 x 2,0m)

Duže stranice: 16 prozora (1,4 x 1,6 m)

Koeficijent pritiska na unutarnje plohe dobivamo jednadžbom [3]:

$$\mu = \frac{\sum ploština\ svih\ otvora\ gdje\ je\ c_{pe}\ negativan\ ili\ 0}{\sum ploština\ svih\ otvora} = \frac{2 * 1 + 5 * (1{,}6 * 2) + 8 * (1{,}4 * 1{,}6)}{2 * 1 + 5 * (1{,}6 * 2) + 16 * (1{,}4 * 1{,}6)} = 0{,}67$$



Slika 8: c_{pi} unutarnjeg pritiska vjetra [3]

Iščitavanjem iz grafa dobivamo:

$$c_{pi} = -0{,}15$$

Jednadžba za opterećenje vjetrom na vanjske površine glasi [3]:

$$w = w_{e,i} - w_i = q_p(z) * (c_{pe} - c_{pi})$$

Za vertikalne površine:

$$w_A = w_{e,A} - w_i = 1{,}07 * (-1{,}2 + 0{,}15) = -1{,}12$$

$$w_B = w_{e,B} - w_i = 1{,}07 * (-0{,}8 + 0{,}15) = -0{,}70$$

$$w_D = w_{e,D} - w_i = 1{,}07 * (0{,}77 + 0{,}15) = +0{,}98$$

$$w_E = w_{e,E} - w_i = 1{,}07 * (-0{,}44 + 0{,}15) = -0{,}31$$

Za krovne površine:

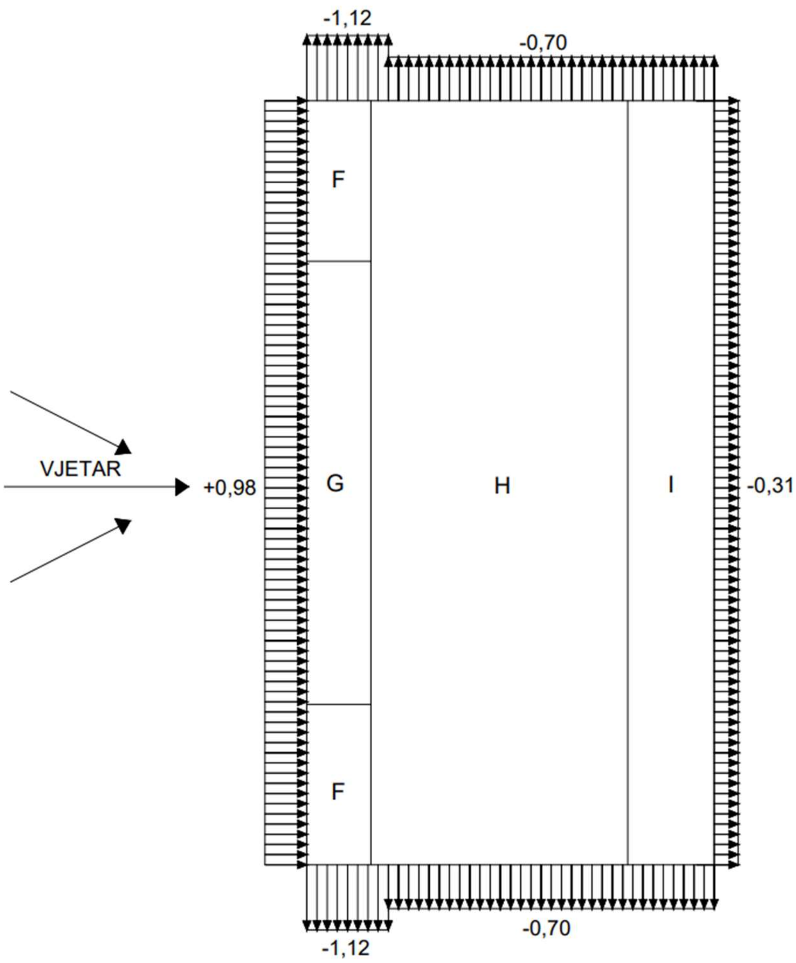
$$w_F = w_{e,A} - w_i = 1{,}07 * (-1{,}8 + 0{,}15) = -1{,}77$$

$$w_G = w_{e,B} - w_i = 1{,}07 * (-1{,}2 + 0{,}15) = -1{,}12$$

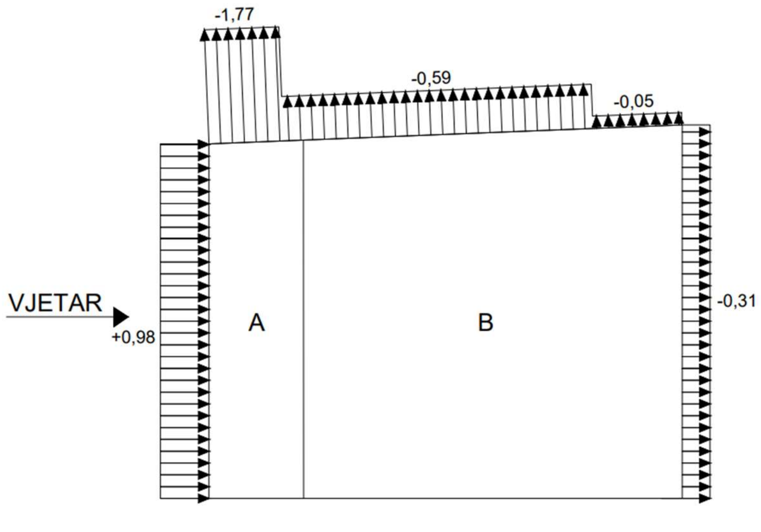
$$w_H = w_{e,D} - w_i = 1{,}07 * (-0{,}7 + 0{,}15) = -0{,}59$$

$$w_I = w_{e,E} - w_i = 1{,}07 * (-0{,}2 + 0{,}15) = -0{,}05$$

Shematski prikaz opterećenja (Θ=0)



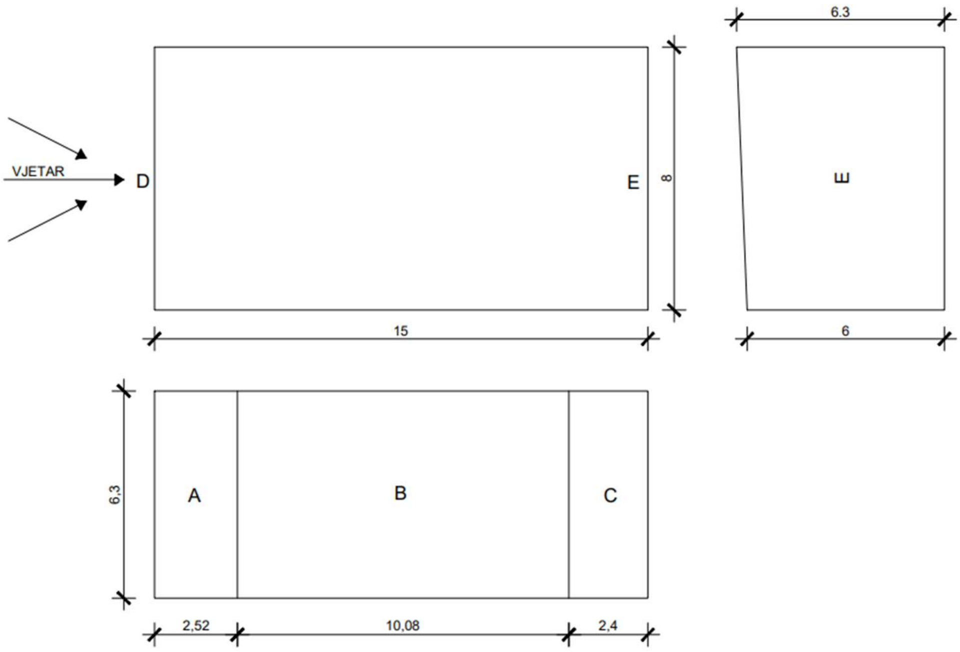
Slika 9: Djelovanje vjetra za slučaj 2 [izradio autor]



Slika 10: Mjerodavni presjek [izradio autor]

3.1.6.3. Određivanje vanjskog longitudinalnog pritiska vjetra sa zatvorenim otvorima (Θ=90)

Vertikalne površine:



Slika 11: Prikaz zona na vertikalnim površinama pri uzdužnom djelovanju vjetra [izradio autor]

Površine vjetrovnih zona:

$$P_A = 2{,}52 * 6{,}3 = 15{,}88\ m^2$$

$$P_B = 10{,}08 * 6{,}3 = 63{,}50\ m^2$$

$$P_C = 2{,}4 * 6{,}3 = 15{,}12\ m^2$$

$$P_{D,E} = 49{,}28\ m^2$$

Linearnom interpolacijom, koeficijenti vanjskog pritiska c_{pe} na vertikalne zidove za h/d=0,79 iznose:

PODRUČJE	A	B	C	D	E
$c_{pe,10}$	-1,2	-0,8	-0,5	+0,8	-0,42

Jednadžba opterećenja vjetra na vanjske površine glasi [3]:

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

Unutrašnji koeficijent pritiska pri zatvorenim otvorima; $c_{pi} = 0$

Opterećenje na vertikalne plohe uslijed djelovanja vjetra:

$w_A = w_{e,A} - w_i = 1{,}07 * (-1{,}2 - 0) = -1{,}28$

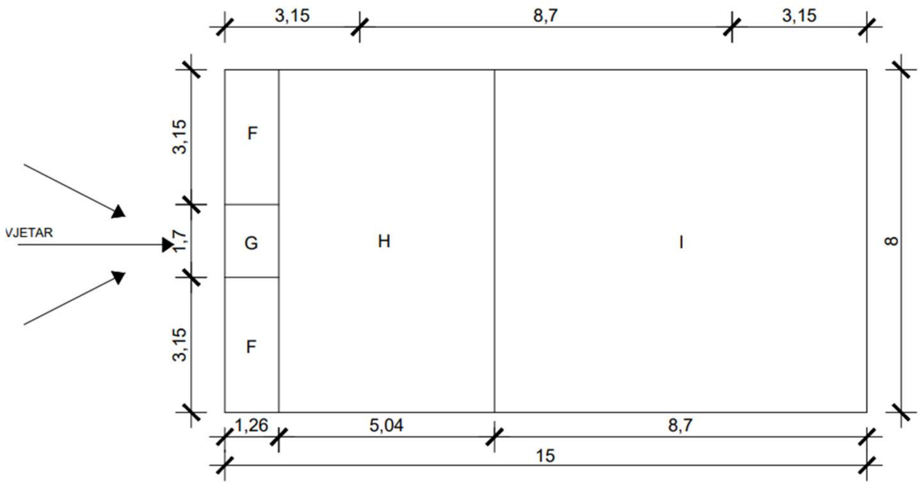
$w_B = w_{e,B} - w_i = 1{,}07 * (-0{,}8 - 0) = -0{,}86$

$w_C = w_{e,D} - w_i = 1{,}07 * (-0{,}5 - 0) = -0{,}54$

$w_D = w_{e,E} - w_i = 1{,}07 * (+0{,}8 - 0) = +0{,}86$

$w_E = w_{e,E} - w_i = 1{,}07 * (-0{,}42 - 0) = -0{,}45$

<u>Krovne površine:</u>



Slika 12: Raspodjela zona na krovu pri uzdužnom djelovanju vjetra [izradio autor]

Površine vjetrovnih zona:

$$P_F = 1{,}26 * 3{,}15 = 3{,}97\ m^2$$

$$P_G = 1{,}26 * 1{,}7 = 2{,}14\ m^2$$

$$P_H = 5{,}04 * 8 = 40{,}32\ m^2$$

$$P_I = 8{,}7 * 8 = 69{,}6\ m^2$$

Linearnom interpolacijom, koeficijenti vanjskog pritiska c_{pe} na vertikalne zidove za h/d=0,79 iznose:

PODRUČJE	F	G	H	I
c_{pe}	-2,08	-1,74	-0,7	-0,2

Jednadžba opterećenja vjetra na vanjske površine glasi [3]:

$$w = w_{e,i} - w_i = q_p(z) * (c_{pe} - c_{pi})$$

Unutrašnji koeficijent pritiska pri zatvorenim otvorima; $c_{pi} = 0$

Opterećenje na krovne plohe uslijed djelovanja vjetra:

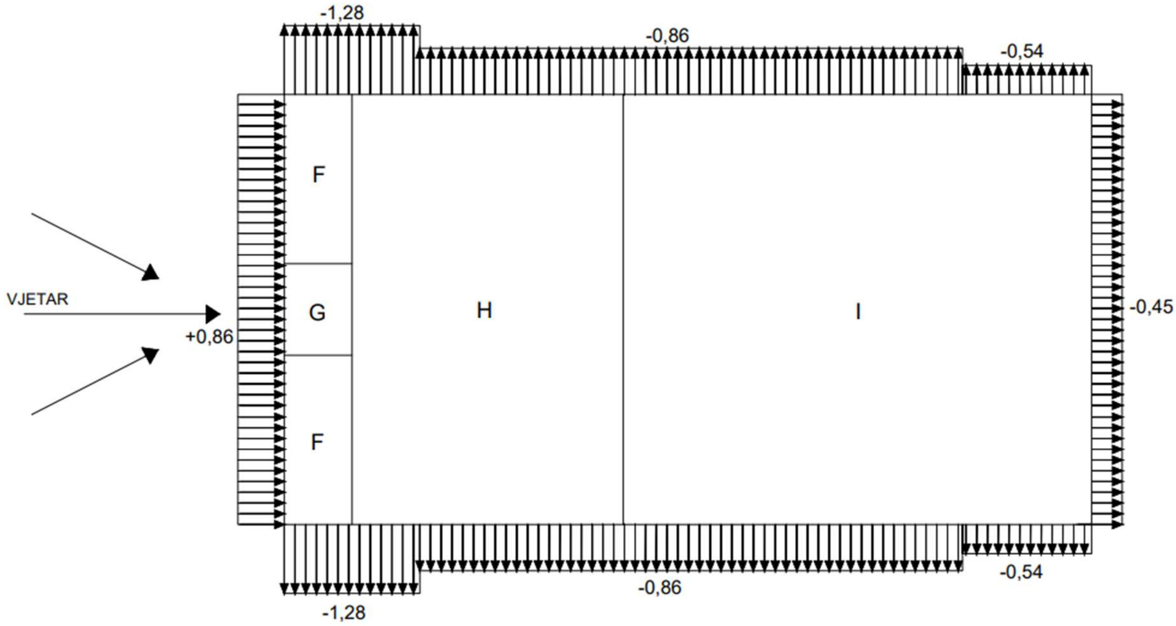
$w_F = w_{e,A} - w_i = 1{,}07 * (-2{,}08 - 0) = -2{,}23$

$w_G = w_{e,B} - w_i = 1{,}07 * (-1{,}74 - 0) = -1{,}86$

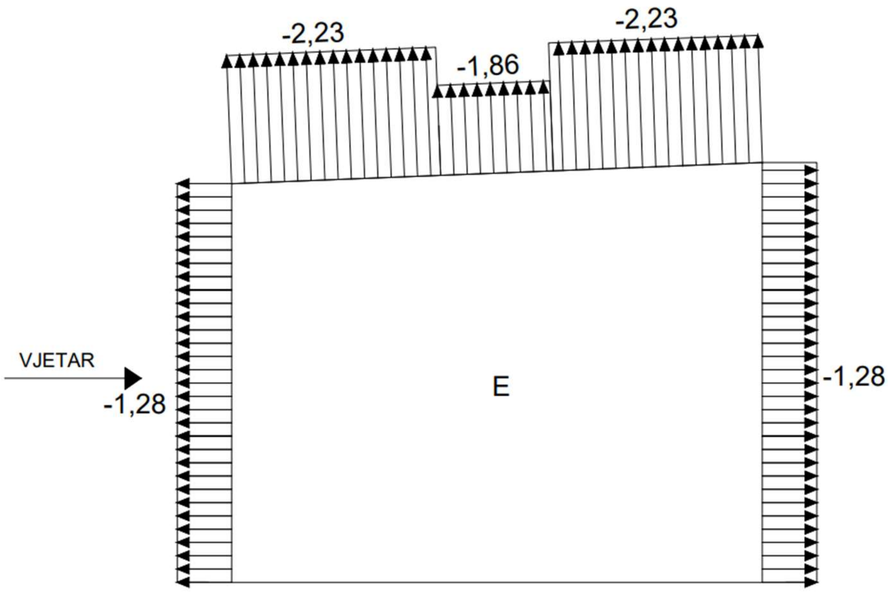
$w_H = w_{e,D} - w_i = 1{,}07 * (-0{,}7 - 0) = -0{,}75$

$w_I = w_{e,E} - w_i = 1{,}07 * (-0{,}2 - 0) = -0{,}21$

Shematski prikaz opterećenja (Θ=90)



Slika 13: Djelovanje vjetra za slučaj 3 [izradio autor]



Slika 14: Mjerodavni presjek [izradio autor]

3.1.6.4. Određivanje vanjskog longitudinalnog pritiska vjetra s otvorenim otvorima (Θ=90)

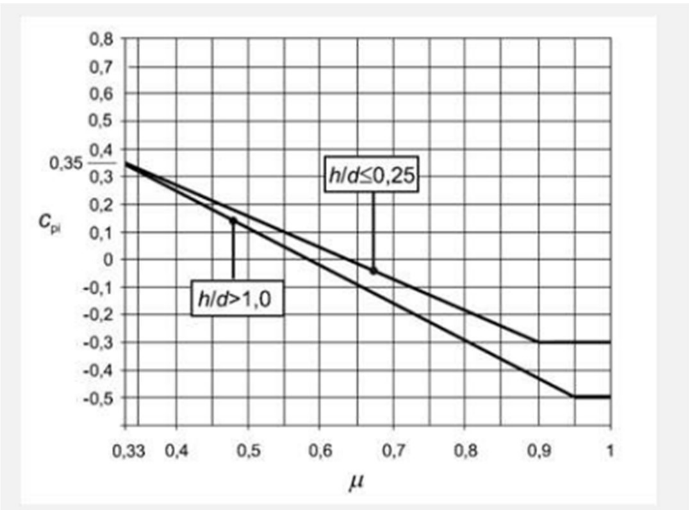
Dimenzije otvora:

Kraća stranica: 1 otvor za ulaz/izlaz ljudi (2,0 x 1,0 m), 5 prozora (1,6 x 2,0 m)

Duža stranica: 16 prozora (1,4 x 1,6 m) od kojih se 8 ne otvaraju

Koeficijent pritiska na unutarnje plohe dobivamo na sljedeći način [3]:

$$\mu = \frac{\sum plo\check{s}tina\ svih\ otvora\ gdje\ je\ c_{pe}\ negativan\ ili\ 0}{\sum plo\check{s}tina\ svih\ otvora} = \frac{4*(1{,}6*2)+8*(1{,}4*1{,}6)}{2*1+5*(1{,}6*2)+16*(1{,}4*1{,}6)} = 0{,}57$$



Slika 15: cpi unutarnjeg pritiska vjetra [3]

Iščitavanjem iz grafa dobivamo:

$$c_{pi} = +0{,}05$$

Jednadžba opterećenja vjetra na vanjske površine glasi [3]:

$$w = w_{e,i} - w_i = q_p(z) * (c_{pe} - c_{pi})$$

Za vertikalne površine:

$$w_A = w_{e,A} - w_i = 1{,}07 * (-1{,}2 - 0{,}05) = -1{,}33$$

$$w_B = w_{e,B} - w_i = 1{,}07 * (-0{,}8 - 0{,}05) = -0{,}91$$

$$w_C = w_{e,D} - w_i = 1{,}07 * (-0{,}5 - 0{,}05) = -0{,}59$$

$$w_D = w_{e,E} - w_i = 1{,}07 * (+0{,}80 - 0{,}05) = +0{,}80$$

$$w_E = w_{e,E} - w_i = 1{,}07 * (-0{,}42 - 0{,}05) = -0{,}50$$

Za krovne površine:

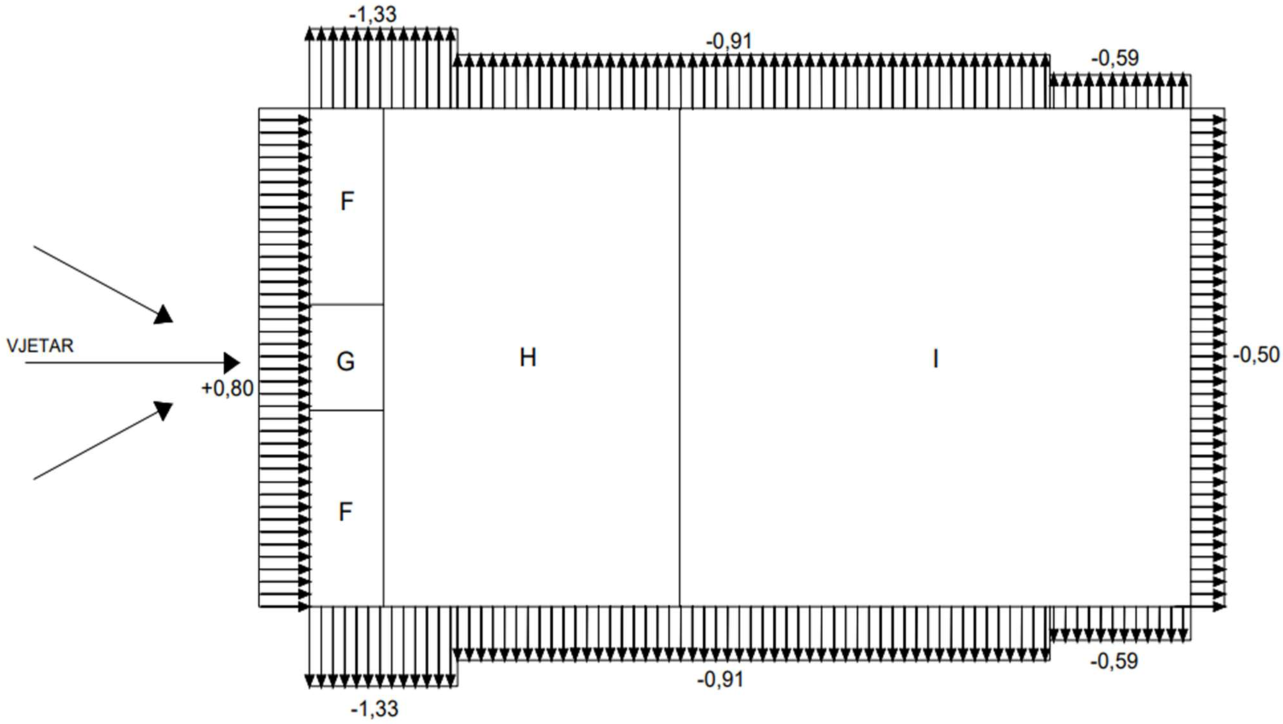
$$w_F = w_{e,A} - w_i = 1{,}07 * (-2{,}08 - 0{,}05) = -2{,}27$$

$$w_G = w_{e,B} - w_i = 1{,}07 * (-1{,}74 - 0{,}05) = -1{,}92$$

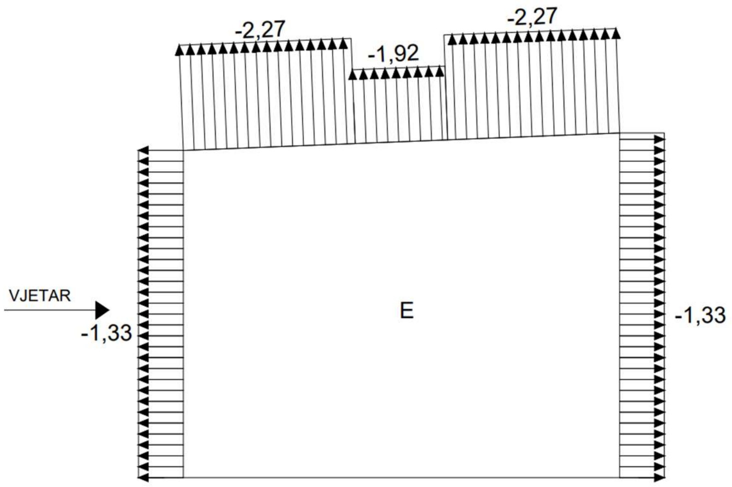
$$w_H = w_{e,D} - w_i = 1{,}07 * (-0{,}7 - 0{,}05) = -0{,}80$$

$$w_I = w_{e,E} - w_i = 1{,}07 * (-0{,}2 - 0{,}05) = -0{,}27$$

Shematski prikaz opterećenja (Θ=90)



Slika 16: Djelovanje vjetra za slučaj 4 [izradio autor]



Slika 17: Mjerodavni presjek [izradio autor]

4. MEHANIČKA OTPORNOST I STABILNOST

Vlastitu težinu je program Robot structural Analysis professional sam proračunao.
Proračunski model konstrukcije je 3D štapni model.

Oslonci stupova su nepomični s onemogućenim pomacima, a omogućenom rotacijom.

4.1. Stalno opterećenje glavne nosive konstrukcije

Opterećenje koje nose podrožnice (e= 2,0 m)

Opterećenje na krajnje podrožnice $G = g * \frac{1}{2} e = 0{,}36 * \frac{1}{2} * 2 = 0{,}36 kN/m'$

Opterećenje na srednje podrožnice $G = g * e = 0{,}36 * 2 = 0{,}72 kN/m'$

4.2. Stalno opterećenje za stubište

Opterećenje na tetive stubišta (e= 1,0 m):

$$G = g * \frac{1}{2} e = 0{,}20 * \frac{1}{2} * 1 = 0{,}10\ kN/m'$$

4.3. Stalno opterećenje za galeriju

Opterećenje na sekundarne nosače (e= 2,0 m):

Opterećenje na krajnje nosače $G = g * \frac{1}{2} e = 0{,}50 * \frac{1}{2} * 2 = 0{,}50\ kN/m'$

Opterećenje na srednje nosače $G = g * \frac{1}{2} e = 0{,}50 * 2 = 1{,}0\ kN/m'$

4.4. Uporabno opterećenje za stubište

Opterećenje na tetive stubišta (e= 1,0 m):

$$G = g * \frac{1}{2} e = 2{,}0 * \frac{1}{2} * 1 = 0{,}50\ kN/m'$$

4.5. Uporabno opterećenje za galeriju

Opterećenje na krajnje nosače: $G = g * \frac{1}{2} e = 2{,}0 * \frac{1}{2} * 2 = 2{,}0\ kN/m'$

Opterećenje na srednje nosače: $G = g * e = 2{,}0 * 2 = 4{,}0\ kN/m'$

4.6. Opterećenje snijegom

$s = 0{,}4\ kN/m^2$

Opterećenje na krajnje podrožnice: $G = g * e = 0{,}4 * 1 = 0{,}40\ kN/m'$

Opterećenje na srednje podrožnice: $G = g * e = 0{,}4 * 2 = 0{,}80\ kN/m'$

4.7. Opterećenje vjetrom

<u>Za prvi slučaj:</u>

Opterećenje na podrožnice (s lijeva na desno 1 – 5):

1. podrožnica -D i F $G = g * \frac{1}{2} e = 1{,}93 * \frac{1}{2} * 2 = 1{,}93\ kN/m'$

   -D i G $G = g * \frac{1}{2} e = 1{,}28 * \frac{1}{2} * 2 = 1{,}28\ kN/m'$
2. podrožnica -H i F $G = g * e = 0{,}75 * 1{,}74 + 1{,}93 * 0{,}26 = 1{,}81\ kN/m'$
3. podrožnica -H i G $G = g * e = 0{,}75 * 1{,}74 + 1{,}28 * 0{,}26 = 1{,}64\ kN/m'$
4. podrožnica $G = g * e = 0{,}75 * 2 = 1{,}50\ kN/m'$
5. podrožnica $G = g * e = 0{,}21 * 1{,}3 + 0{,}75 * 0{,}7 = 0{,}80\ kN/m'$
6. podrožnica $G = g * e = 0{,}21 * 1 = 0{,}21\ kN/m'$

8. i 12. podrožnica $G = g * e = 0{,}75 * 1{,}74 + 1{,}28 * 0{,}26 = 1{,}64\ kN/m'$

Opterećenje na zidove

D zona -najniži fasadni nosač: $G = g * e = 0{,}82 * 1{,}69 = 1{,}39\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}82 * 2 = 1{,}64\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}82 * 1{,}32 = 1{,}06\ kN/m'$

E zona -najniži fasadni nosač: $G = g * e = 0{,}47 * 1{,}69 = 0{,}79\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}47 * 2 = 0{,}94\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}47 * 1{,}32 = 0{,}62\ kN/m'$

A zona -najniži fasadni nosač: $G = g * e = 1{,}28 * 1{,}69 = 2{,}16\ kN/m'$

-srednji fasadni nosač: $G = g * e = 1{,}28 * 2 = 2{,}56\ kN/m'$

-najviši fasadni nosač: $G = g * e = 1{,}28 * 1{,}32 = 1{,}69\ kN/m'$

B zona -najniži fasadni nosač: $G = g * e = 0{,}86 * 1{,}69 = 1{,}45\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}86 * 2 = 1{,}72\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}86 * 1{,}32 = 1{,}14\ kN/m'$

<u>Za drugi slučaj:</u>

Opterećenje na podrožnice(s lijeva na desno 1 – 5)

1. podrožnica -D i F $G = g * \frac{1}{2}e = 1{,}77 * \frac{1}{2} * 2 = 1{,}77\ kN/m'$

   -D i G $G = g * \frac{1}{2}e = 1{,}12 * \frac{1}{2} * 2 = 1{,}12\ kN/m'$

2. podrožnica $G = g * e = 0{,}59 * 1{,}74 + 1{,}77 * 0{,}26 = 1{,}49\ kN/m'$
3. podrožnica $G = g * e = 0{,}59 * 2 = 1{,}18\ kN/m'$
4. podrožnica $G = g * e = 0{,}05 * 1{,}3 + 0{,}59 * 0{,}7 = 0{,}48\ kN/m'$
5. podrožnica $G = g * e = 0{,}05 * 1 = 0{,}05\ kN/m'$
7. i 12. podrožnica $G = g * e = 0{,}59 * 1{,}74 + 1{,}12 * 0{,}26 = 1{,}32\ kN/m'$

Opterećenje na zidove:

D zona -najniži fasadni nosač: $G = g * e = 0,98 * 1,69 = 1,39\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0,98 * 2 = 1,96\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0,98 * 1,32 = 1,29\ kN/m'$

E zona -najniži fasadni nosač: $G = g * e = 0,31 * 1,69 = 0,52\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0,31 * 2 = 0,62\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0,31 * 1,32 = 0,41\ kN/m'$

A zona -najniži fasadni nosač: $G = g * e = 1,12 * 1,69 = 1,89\ kN/m'$

-srednji fasadni nosač: $G = g * e = 1,12 * 2 = 2,24\ kN/m'$

-najviši fasadni nosač: $G = g * e = 1,12 * 1,32 = 1,48\ kN/m'$

B zona -najniži fasadni nosač: $G = g * e = 0,70 * 1,69 = 1,18\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0,70 * 2 = 1,40\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0,70 * 1,32 = 0,92\ kN/m'$

<u>Za treći slučaj:</u>

Opterećenje na podrožnice (s lijeva na desno 1 – 5):

F i G zone podrožnica

1. podrožnica $G = g * \frac{1}{2} e = 2,23 * \frac{1}{2} * 2 = 2,23\ kN/m'$
2. podrožnica $G = g * e = 2,23 * 2 = 4,46\ kN/m'$
3. podrožnica $G = g * e = 1,86 * 1,7 + 2,23 * 0,3 = 3,83\ kN/m'$
4. podrožnica $G = g * e = 2,23 * 2 = 4,46\ kN/m'$
5. podrožnica $G = g * e = 2,23 * \frac{1}{2} * 2 = 2,23\ kN/m'$

H zona podrožnica

6,10. podrožnica $G = g * e = 0,75 * 1 = 0,75\ kN/m'$

7,8,9. podrožnica $G = g * e = 0,75 * 2 = 1,50\ kN/m'$

I zona podrožnica

krajnje $G = g * e = 0{,}21 * 1 = 0{,}21\ kN/m'$

srednje $G = g * e = 0{,}21 * 2 = 0{,}42\ kN/m'$

Opterećenje na zidove

D zona -najniži fasadni nosač: $G = g * e = 0{,}86 * 1{,}69 = 1{,}45\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}86 * 2 = 1{,}72\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}86 * 1{,}32 = 1{,}14\ kN/m'$

E zona -najniži fasadni nosač: $G = g * e = 0{,}45 * 1{,}69 = 0{,}76\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}45 * 2 = 0{,}90\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}45 * 1{,}32 = 0{,}59\ kN/m'$

A zona -najniži fasadni nosač: $G = g * e = 1{,}28 * 1{,}69 = 2{,}16\ kN/m'$

-srednji fasadni nosač: $G = g * e = 1{,}28 * 2 = 2{,}56\ kN/m'$

-najviši fasadni nosač: $G = g * e = 1{,}28 * 1{,}32 = 1{,}69\ kN/m'$

B zona -najniži fasadni nosač: $G = g * e = 0{,}86 * 1{,}69 = 1{,}45\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}86 * 2 = 1{,}72\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}86 * 1{,}32 = 1{,}14\ kN/m'$

C zona -najniži fasadni nosač: $G = g * e = 0{,}54 * 1{,}69 = 0{,}91\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}54 * 2 = 1{,}08\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}54 * 1{,}32 = 0{,}72\ kN/m'$

Za četvrti slučaj:

Opterećenje na podrožnice(s lijeva na desno 1 – 5)

F i G zone podrožnica

1. podrožnica $G = g * \frac{1}{2}e = 2{,}27 * \frac{1}{2} * 2 = 2{,}27\ kN/m'$
2. podrožnica $G = g * e = 2{,}27 * 2 = 4{,}54\ kN/m'$
3. podrožnica $G = g * e = 1{,}92 * 1{,}7 + 2{,}27 * 0{,}3 = 3{,}95\ kN/m'$
4. podrožnica $G = g * e = 2{,}27 * 2 = 4{,}54\ kN/m'$
5. podrožnica $G = g * e = 2{,}27 * \frac{1}{2} * 2 = 2{,}27\ kN/m'$

H zona podrožnica

6,10. podrožnica $G = g * e = 0{,}80 * 1 = 0{,}80\ kN/m'$

7,8,9. podrožnica $G = g * e = 0{,}80 * 2 = 1{,}60\ kN/m'$

I zona podrožnica

krajnje $G = g * e = 0{,}27 * 1 = 0{,}27\ kN/m'$

srednje $G = g * e = 0{,}27 * 2 = 0{,}54\ kN/m'$

Opterećenje na zidove

D zona -najniži fasadni nosač: $G = g * e = 0{,}80 * 1{,}69 = 1{,}35\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}80 * 2 = 1{,}60\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}80 * 1{,}32 = 1{,}06\ kN/m'$

E zona -najniži fasadni nosač: $G = g * e = 0{,}50 * 1{,}69 = 0{,}85\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0{,}50 * 2 = 1{,}0\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0{,}50 * 1{,}32 = 0{,}66\ kN/m'$

A zona -najniži fasadni nosač: $G = g * e = 1{,}33 * 1{,}69 = 2{,}25\ kN/m'$

-srednji fasadni nosač: $G = g * e = 1{,}33 * 2 = 2{,}66\ kN/m'$

-najviši fasadni nosač: $G = g * e = 1{,}33 * 1{,}32 = 1{,}76\ kN/m'$

B zona -najniži fasadni nosač: $G = g * e = 0,91 * 1,69 = 1,54\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0,91 * 2 = 1,82\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0,91 * 1,32 = 1,20\ kN/m'$

C zona -najniži fasadni nosač: $G = g * e = 0,59 * 1,69 = 1,0\ kN/m'$

-srednji fasadni nosač: $G = g * e = 0,59 * 2 = 1,18\ kN/m'$

-najviši fasadni nosač: $G = g * e = 0,59 * 1,32 = 0,78\ kN/m'$

4.8. Imperfekcije okvira

$\emptyset = k_c * k_s * \emptyset_0$

$k_c = (0,5 + \frac{1}{nc})^{0.5} = (0,5 + \frac{1}{2})^{0.5} = 1$

$k_s = (0,2 + \frac{1}{ns})^{0.5} = (0,2 + \frac{1}{2})^{0.5} = 0,84$

$\emptyset = 1 * 0,84 * \frac{1}{200} = 0,0042$

$V1 = v + g = 22,15\ kN$ – ukupno vertikalno opterećenje od vlastite težine i stalnog djelovanja

$V2 = s = 6,01\ kN$ – ukupno vertikalno opterećenje od snijega

$V3 = w1 = 42,59\ kN$ - ukupno vertikalno opterećenje od vjetra 1

$V4 = w2 = 42,57\ kN$ - ukupno vertikalno opterećenje od vjetra 2

$V5 = w3 = -33,12\ kN$ - ukupno vertikalno opterećenje od vjetra 3

$V6 = w4 = -32,31\ kN$ - ukupno vertikalno opterećenje od vjetra 4

$\Delta H = \emptyset * V$

$\Delta H1 = \emptyset * V1 = 0,0042 * 22,15 = 0,093\ kN$

$\Delta H2 = \emptyset * V2 = 0,0042 * 6,01 = 0,025\ kN$

$\Delta H3 = \emptyset * V3 = 0,0042 * 42,59 = 0,18\ kN$

$\Delta H4 = \emptyset * V4 = 0,0042 * 42,57 = 0,18\ kN$

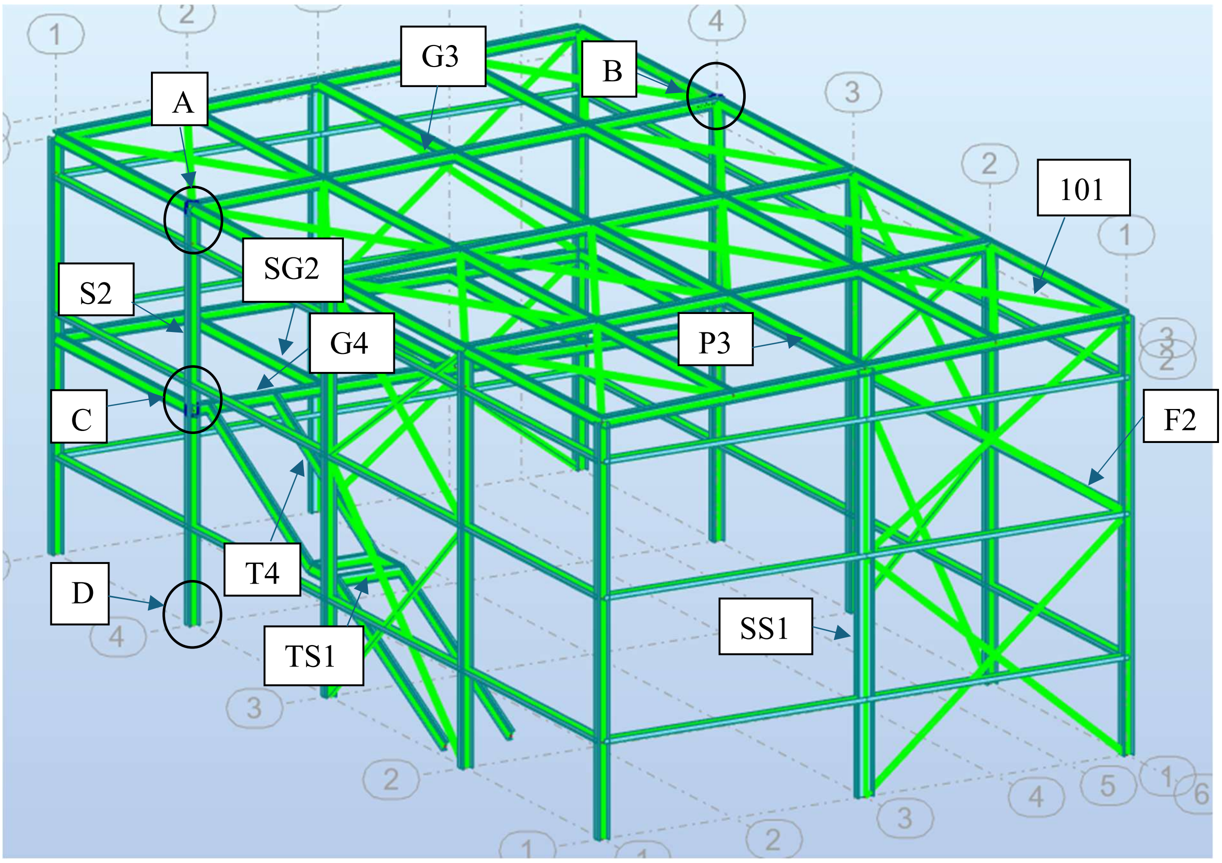
$\Delta H5 = \emptyset * V5 = 0,0042 * (-33,12) = -0,14\ kN$

$\Delta H6 = \emptyset * V6 = 0,0042 * (-32,31) = -0,14\ kN$

5. PLAN POZICIJA

Prikaz modela s označenim mjerodavnim elementima s najvećim reznim silama, koji se koristi za daljnji proračun dimenzioniranja, prikazan je na slici 18.

Ostale pozicije elemenata prikazane su u dispozicijskim nacrtima.



Slika 18: Prikaz modela s označenim mjerodavnim pozicijama elemenata i detalja [izradio autor]

6. KOMBINACIJE OPTEREĆENJA

Za granično stanje nosivosti – GSN(ULS) [4, 5, 6]**:**

$$\sum_{j\geq1}\gamma_{k,j} * G_{k,j}'' + ''\gamma_{Q,1} * Q_{k,1}'' + ''\sum_{i\geq1}\gamma_{Q,i} * \psi_{0,1} * Q_{k,i}''$$

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

$\gamma_{k,j}$- parcijalni koeficijent sigurnosti stalnih djelovanja

$\gamma_{Q,1}$- parcijalni koeficijent sigurnosti vodećeg promjenjivog djelovanja

$\gamma_{Q,i}$- parcijalni koeficijent sigurnosti ostalih promjenjivih djelovanja

$\psi_{0,1}$- koeficijent kombinacija opterećenja

Tablica 2: Vrijednosti koeficijenta ψ [4]

Djelovanje	ψ_0	ψ_1	ψ_2
Snijeg	0,5	0,2	0
Vjetar	0,6	0,2	0

Za granično stanje uporabljivosti – GSU (SLS) [4, 5, 6]**:**

$$\sum_{j\geq 1} G''_{k,j} + Q''_{k,1} +'' \sum_{i\geq 1} \gamma_{Q,1} * \psi_{0,1} * Q''_{k,i}$$

Djelovanja:

DL1 - vlastita težina

DL2 - stalno opterećenje

LL1 - uporabno opterećenje

SN1 - opterećenje snijegom

WIND1 - opterećenje vjetrom (slučaj 1)

WIND2 - opterećenje vjetrom (slučaj 2)

WIND3 - opterećenje vjetrom (slučaj 3)

WIND4 - opterećenje vjetrom (slučaj 4)

Load Combination - Cases: 9to26 29to38 40
Values
1

- Cases: 9to26 29to38 40

Filtering	Combinations
Full list	9to26 29to38 40
Selection	9to26 29to38 40
Total number	77
Selected number	77

- Cases: 9to26 29to38 40

Combinations	Name	Analysis type	Combination type	Case nature
9 (C)	COMB1	Linear Combinatio	ULS	Structural
10 (C)	COMB2	Linear Combinatio	ULS	Structural
11 (C)	COMB3	Linear Combinatio	ULS	Structural
12 (C)	COMB5	Linear Combinatio	ULS	Structural
13 (C)	COMB6	Linear Combinatio	ULS	Structural
14 (C)	COMB7	Linear Combinatio	ULS	Structural
15 (C)	COMB8	Linear Combinatio	ULS	Structural
16 (C)	COMB9	Linear Combinatio	ULS	Structural
17 (C)	COMB10	Linear Combinatio	ULS	Structural
18 (C)	COMB11	Linear Combinatio	ULS	Structural
19 (C)	COMB12	Linear Combinatio	ULS	Structural
20 (C)	COMB13	Linear Combinatio	ULS	Structural
21 (C)	COMB13	Linear Combinatio	SLS	Structural
22 (C)	COMB14	Linear Combinatio	SLS	Structural
23 (C)	COMB15	Linear Combinatio	SLS	Structural
24 (C)	COMB16	Linear Combinatio	SLS	Structural
25 (C)	COMB28	Linear Combinatio	ULS	Structural
26 (C)	COMB29	Linear Combinatio	ULS	Structural
29 (C)	COMB17	Linear Combinatio	SLS	Structural
30 (C)	COMB18	Linear Combinatio	SLS	Structural
31 (C)	COMB19	Linear Combinatio	SLS	Structural
32 (C)	COMB20	Linear Combinatio	SLS	Structural
33 (C)	COMB21	Linear Combinatio	SLS	Structural
34 (C)	COMB22	Linear Combinatio	SLS	Structural
35 (C)	COMB23	Linear Combinatio	SLS	Structural
36 (C)	COMB24	Linear Combinatio	SLS	Structural
37 (C)	COMB25	Linear Combinatio	SLS	Structural
38 (C)	COMB4	Linear Combinatio	ULS	Structural
40 (C)	COMB28	Linear Combinatio	ULS	Structural

Combinations	Definition
9 (C)	(1+2)*1.35+4*1.50
10 (C)	(1+2)*1.00+5*1.50
11 (C)	(1+2)*1.00+6*1.50
12 (C)	(1+2)*1.00+8*1.50
13 (C)	(1+2)*1.35+4*1.50+5*0.90
14 (C)	(1+2)*1.35+4*1.50+6*0.90
15 (C)	(1+2)*1.35+4*1.50+7*0.90
16 (C)	(1+2)*1.35+4*1.50+8*0.90
17 (C)	(1+2)*1.35+4*0.75+5*1.50
18 (C)	(1+2)*1.35+4*0.75+6*1.50
19 (C)	(1+2)*1.35+4*0.75+7*1.50
20 (C)	(1+2)*1.35+4*0.75+8*1.50
21 (C)	(1+2+4)*1.00
22 (C)	(1+2+5)*1.00
23 (C)	(1+2+6)*1.00
24 (C)	(1+2+7)*1.00
25 (C)	3*1.50+2*1.35
26 (C)	(1+2)*1.00
29 (C)	(1+2+8)*1.00
30 (C)	(1+2+4)*1.00+5*0.60
31 (C)	(1+2+4)*1.00+6*0.60
32 (C)	(1+2+4)*1.00+7*0.60
33 (C)	(1+2+4)*1.00+8*0.60
34 (C)	(1+2+5)*1.00+4*0.50
35 (C)	(1+2+6)*1.00+4*0.50
36 (C)	(1+2+7)*1.00+4*0.50
37 (C)	(1+2+8)*1.00+4*0.50
38 (C)	(1+2)*1.00+7*1.50
40 (C)	1*1.35+3*1.50

7. REZULTATI

7.1. Rezne sile

U sljedećim tablicama su prikazni globalni (maksimalni) rezultati reznih sila dobivenih kombiniranjem gore navedenih kombinacija za pojedine elemente.

Tablica 3: Maksimalne rezne sile grednog nosača [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	13,59	21,85	29,83	0,05	17,24	0,71
Member	35	209	209	210	29	35
Node	49	296	296	364	46	49
Case	14 (C)	38 (C)	25 (C)	25 (C)	38 (C)	10 (C)
MIN	-28,45	-1,57	-18,93	-0,45	-30,98	-1,87
Member	209	210	209	209	209	209
Node	296	363	303	296	296	303
Case	20 (C)	25 (C)	25 (C)	40 (C)	25 (C)	17 (C)

Tablica 4: Maksimalne rezne sile glavnih stupova [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	51,05	6,30	21,96	0,06	20,37	2,84
Member	34	37	28	24	33	24
Node	19	51	45	34	48	34
Case	40 (C)	10 (C)	12 (C)	38 (C)	9 (C)	10 (C)
MIN	-21,30	-6,61	-19,44	-0,51	-23,03	-2,84
Member	30	24	30	30	34	37
Node	41	34	46	46	49	51
Case	7	10 (C)	12 (C)	17 (C)	9 (C)	10 (C)

Tablica 5: Maksimalne rezne sile sekundarnih stupova [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	40,19	10,85	6,14	0,02	0,60	10,31
Member	113	112	113	112	113	112
Node	305	201	305	201	213	201
Case	40 (C)	10 (C)	17 (C)	18 (C)	12 (C)	10 (C)
MIN	-34,31	-12,13	-2,59	-0,01	-7,98	-11,18
Member	112	112	112	113	112	112
Node	304	201	201	213	201	201
Case	10 (C)	19 (C)	10 (C)	18 (C)	17 (C)	19 (C)

Tablica 6: Maksimalne rezne sile podrožnica [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	27,68	0,26	7,22	0,01	10,27	0,66
Member	54	53	48	57	48	51
Node	205	202	201	222	201	213
Case	17 (C)	17 (C)	9 (C)	16 (C)	10 (C)	17 (C)
MIN	-28,64	-0,35	-6,78	-0,01	-11,12	-0,66
Member	128	51	53	129	48	51
Node	206	210	202	203	201	210
Case	17 (C)	17 (C)	12 (C)	9 (C)	19 (C)	17 (C)

Tablica 7: Maksimalne rezne sile sekundarnih nosača galerije [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	2,70	0,33	13,82	0,01	0,06	0,62
Member	211	208	211	211	212	208
Node	365	296	365	365	367	296
Case	25 (C)	25 (C)	25 (C)	25 (C)	25 (C)	25 (C)
MIN	-7,82	-0,12	-16,91	-0,01	-11,68	-0,60
Member	208	213	212	213	212	208
Node	296	369	368	369	368	363
Case	25 (C)	25 (C)	25 (C)	25 (C)	25 (C)	25 (C)

Tablica 8: Maksimalne rezne sile tetiva stubišta [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	11,49	0,26	6,33	0,00	12,25	0,34
Member	98	99	61	214	100	215
Node	282	230	229	284	285	285
Case	40 (C)	10 (C)	40 (C)	19 (C)	40 (C)	10 (C)
MIN	-25,91	-0,15	-10,92	-0,01	-3,05	-0,68
Member	214	100	99	99	99	98
Node	371	283	284	230	284	283
Case	19 (C)	38 (C)	19 (C)	25 (C)	7	17 (C)

Tablica 9: Maksimalne rezne sile sekundarnih nosača stubišta [izradio autor]

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	0,21	0,15	0,71	0,01	0,30	0,62
Member	162	163	163	163	163	163
Node	284	283	283	283	230	230
Case	25 (C)	38 (C)	17 (C)	25 (C)	17 (C)	17 (C)
MIN	-0,24	-1,24	-0,38	-0,00	-0,30	-0,61
Member	162	163	162	163	163	163
Node	284	283	285	283	283	283
Case	12 (C)	17 (C)	17 (C)	5	17 (C)	17 (C)

Tablica 10: Maksimalne rezne sile vjetrovnih vezova [izradio autor]

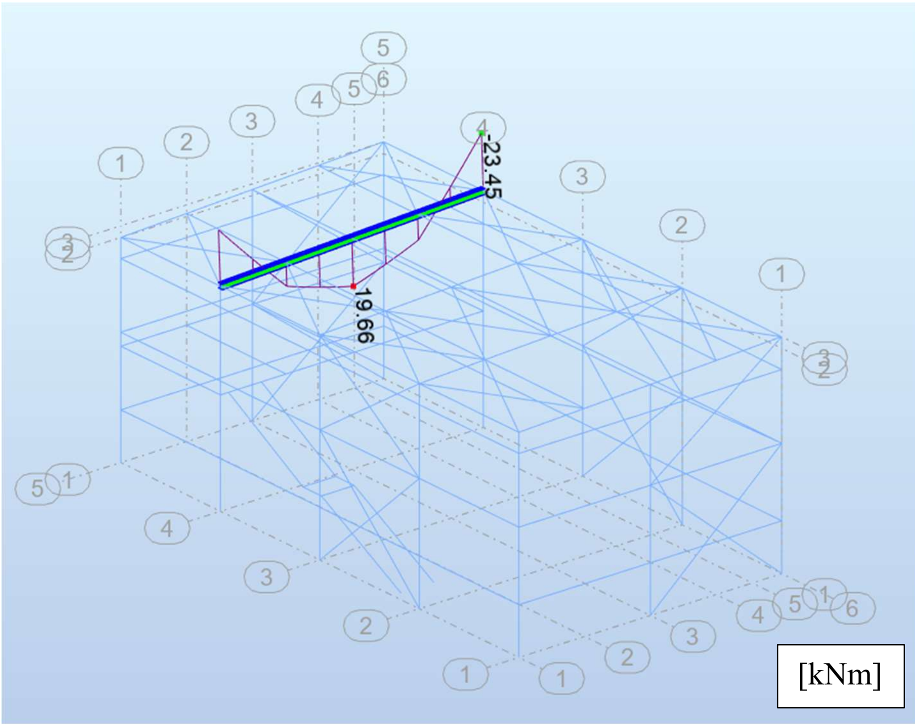
	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	14,85	0,00	0,02	0,00	0,00	0,00
Member	226	218	220	227	219	222
Node	205	45	49	202	34	209
Case	17 (C)	10 (C)	15 (C)	20 (C)	5	17 (C)
MIN	-14,74	-0,00	-0,02	-0,00	-0,01	-0,00
Member	227	223	229	218	220	223
Node	223	212	225	45	49	212
Case	17 (C)	17 (C)	15 (C)	20 (C)	16 (C)	10 (C)

Tablica 11: Maksimalne rezne sile fasadnih nosača [izradio autor]

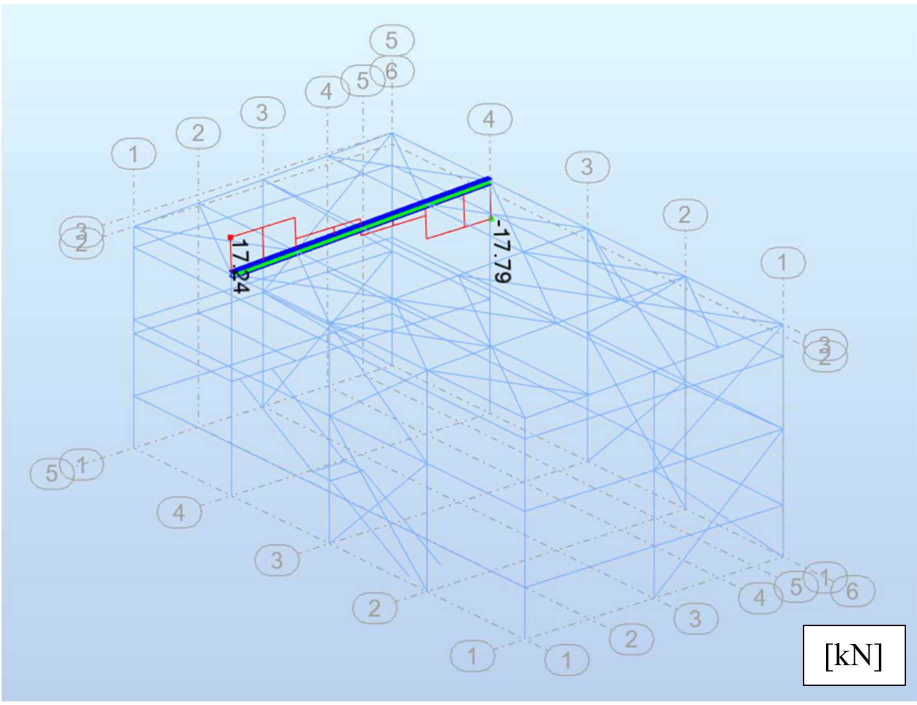
	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	15,76	0,65	9,69	0,02	11,65	1,41
Member	170	173	200	188	200	188
Node	332	309	314	306	314	351
Case	38 (C)	19 (C)	17 (C)	19 (C)	17 (C)	17 (C)
MIN	-10,68	-1,02	-9,44	-0,02	-11,63	-0,86
Member	182	188	189	190	176	188
Node	320	351	308	310	314	306
Case	17 (C)	17 (C)	17 (C)	38 (C)	17 (C)	10 (C)

8. DIMENZIONIRANJE ELEMENATA KONSTRUKCIJE

8.1. Gredni nosač POZ G3



Slika 19: Vrijednost My za krovni gredni nosač [izradio autor]



Slika 20:Vrijednost Fz za krovni gredni nosač [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 35 Beam_35 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

----

LOADS:
Governing Load Case: 9 COMB1 (1+2)*1.35+4*1.50

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: HEA 160

h=152 mm	gM0=1.00	gM1=1.00	
b=160 mm	Ay=3253 mm2	Az=1321 mm2	Ax=3877 mm2
tw=6 mm	Iy=16729800 mm4	Iz=6155730 mm4	Ix=109000 mm4
tf=9 mm	Wply=245167 mm3	Wplz=117635 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 12.51 kN	My,Ed = -23.02 kN*m	Mz,Ed = -0.01 kN*m	Vy,Ed = -0.02 kN
Nc,Rd = 1066.21 kN	My,Ed,max = -23.45 kN*m		Mz,Ed,max = 0.03 kN*m
	Vy,T,Rd = 515.05 kN		
Nb,Rd = 397.70 kN	My,c,Rd = 67.42 kN*m	Mz,c,Rd = 32.35 kN*m	Vz,Ed = 17.24 kN
	MN,y,Rd = 67.42 kN*m	MN,z,Rd = 32.35 kN*m	Vz,T,Rd = 209.36 kN
	Mb,Rd = 40.79 kN*m		Tt,Ed = -0.01 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 48.65 kN*m	Curve,LT - b	XLT = 0.59
Lcr,low=8.01 m	Lam_LT = 1.18	fi,LT = 1.15	XLT,mod = 0.61

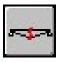
BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 8.01 m	Lam_y = 1.42	Lz = 8.01 m	Lam_z = 0.59
Lcr,y = 8.01 m	Xy = 0.37	Lcr,z = 2.00 m	Xz = 0.79
Lamy = 121.88	kyy = 1.01	Lamz = 50.23	kyz = 1.51

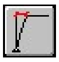
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.34 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.00 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.12 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)

Vz,Ed/Vz,T,Rd = 0.08 < 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)) = 0.00 < 1.00   (6.2.6)
Global stability check of member:
Lambda,y = 121.88 < Lambda,max = 210.00   Lambda,z = 50.23 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.57 < 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) = 0.62 < 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))

LIMIT DISPLACEMENTS



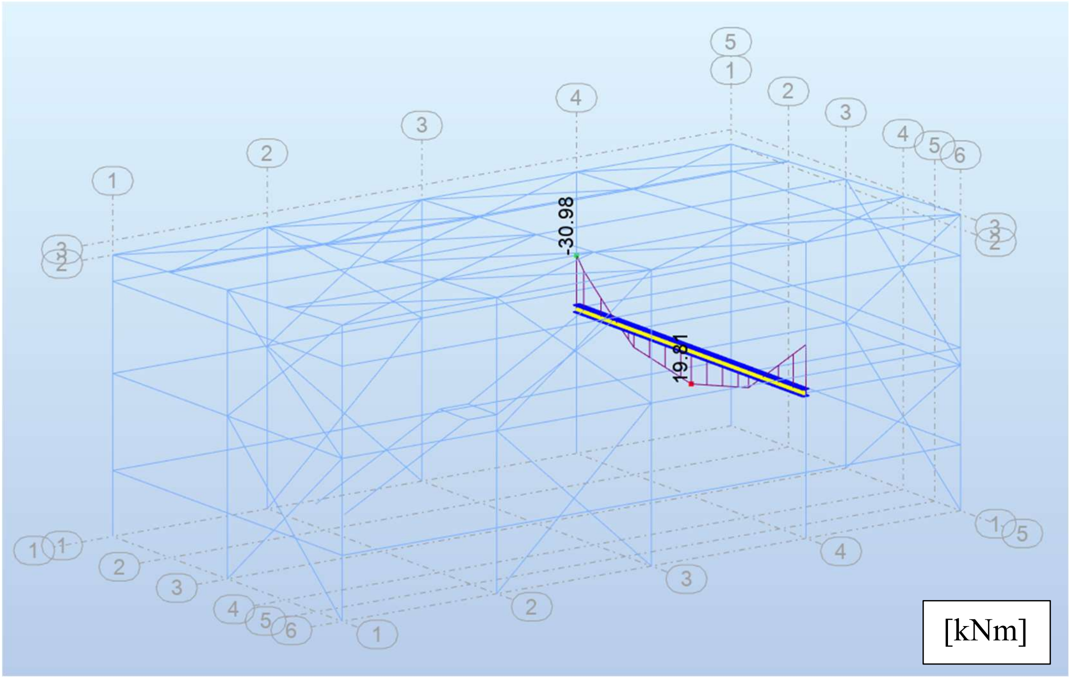
Deflections (LOCAL SYSTEM):
uy = 1 mm < uy max = L/200.00 = 40 mm   Verified
Governing Load Case: 22 COMB14 (1+2+5)*1.00
uz = 19 mm < uz max = L/200.00 = 40 mm   Verified
Governing Load Case: 21 COMB13 (1+2+4)*1.00
u inst,y = 1 mm < u inst,max,y = L/250.00 = 32 mm   Verified
Governing Load Case: 1*5
u inst,z = 12 mm < u inst,max,z = L/250.00 = 32 mm   Verified
Governing Load Case: 1*5



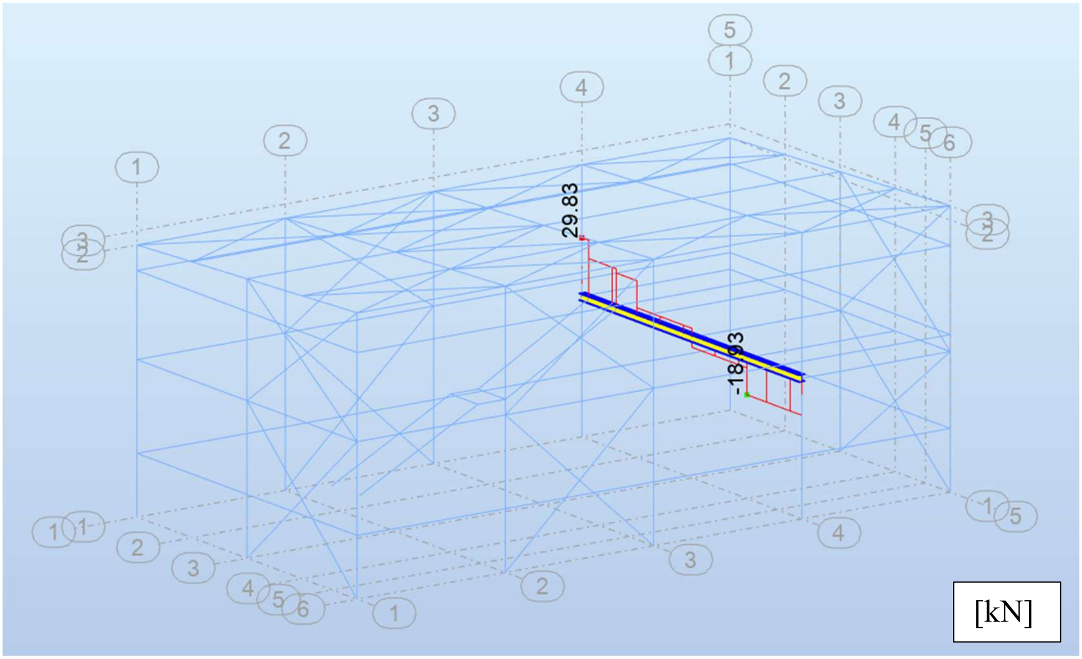
Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

8.2. Gredni nosač POZ G4



Slika 21: Mjerodavna vrijednost My za gredni nosač galerije [izradio autor]



Slika 22: Mjerodavna vrijednost Fz za gredni nosač galerije [izradio autor]

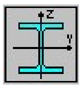
STEEL DESIGN

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

CODE GROUP:
MEMBER: 209 gl. nosači galerija_209 **POINT:** 1
COORDINATE: x = 0.00 L = 0.00 m

LOADS:
Governing Load Case: 25 COMB28 3*1.50+2*1.35

MATERIAL:
S275 (S275) fy = 275.00 MPa

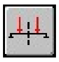
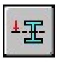


SECTION PARAMETERS: HEA 160

h=152 mm	gM0=1.00	gM1=1.00	
b=160 mm	Ay=3253 mm2	Az=1321 mm2	Ax=3877 mm2
tw=6 mm	Iy=16729800 mm4	Iz=6155730 mm4	Ix=109000 mm4
tf=9 mm	Wply=245167 mm3	Wplz=117635 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -5.00 kN	My,Ed = -30.98 kN*m	Mz,Ed = -0.70 kN*m	Vy,Ed = 2.19 kN
Nt,Rd = 1066.21 kN	My,pl,Rd = 67.42 kN*m	Mz,pl,Rd = 32.35 kN*m	Vy,T,Rd = 471.63 kN
	My,c,Rd = 67.42 kN*m	Mz,c,Rd = 32.35 kN*m	Vz,Ed = 29.83 kN
	MN,y,Rd = 67.42 kN*m	MN,z,Rd = 32.35 kN*m	Vz,T,Rd = 197.80 kN
	Mb,Rd = 67.20 kN*m		Tt,Ed = -0.40 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 311.35 kN*m	Curve,LT - b	XLT = 0.97
Lcr,low=2.00 m	Lam_LT = 0.47	fi,LT = 0.59	XLT,mod = 1.00

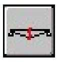
BUCKLING PARAMETERS:

About y axis: About z axis:

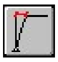
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
My,Ed/MN,y,Rd = 0.46 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.02 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.23 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.15 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.21 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.14 < 1.00 (6.2.6)

Global stability check of member:
My,Ed/Mb,Rd = 0.46 < 1.00   (6.3.2.1.(1))

LIMIT DISPLACEMENTS



Deflections (LOCAL SYSTEM):
uy = 8 mm < uy max = L/200.00 = 40 mm   Verified
Governing Load Case: 34 COMB22 (1+2+5)*1.00+4*0.50
uz = 15 mm < uz max = L/200.00 = 40 mm   Verified
Governing Load Case: 3 LL1
u inst,y = 6 mm < u inst,max,y = L/250.00 = 32 mm   Verified
Governing Load Case: 0.5*4 + 1*5
u inst,z = 3 mm < u inst,max,z = L/250.00 = 32 mm   Verified
Governing Load Case: 1*5



Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

Za pojedine elemente koristimo detaljnu provjeru proračuna, što uključuje temeljitu analizu i izračune dimenzioniranja elementa [4, 5, 6].

Profil: HEA 160

Materijal: S275

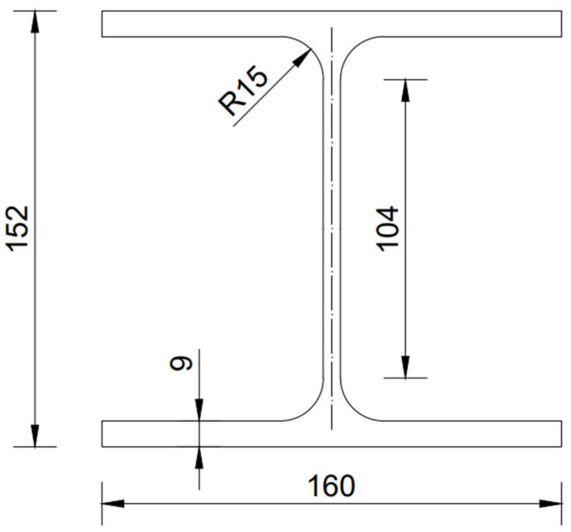
$N_{c,Ed} = 5{,}08\ kN$

$V_{c,Ed} = 29{,}83\ kN$

$M_{y,Ed} = 30{,}98\ kNm$

Tablica 12: Karakteristike profila HEA 160 [izradio autor]

Profil	G	A	h	b	tw	tf	r	I_t	$I_{w*}10^{-3}$
	kg/m	cm^2	mm	mm	mm	mm	mm	cm^4	cm^6
HEA 160	30,4	38,8	152	160	6	9	15	12,19	31,41



Slika 23: Prikaz profila HEA 160 [izradio autor]

Klasifikacija poprečnog presjeka:

HRBAT

$$\frac{c}{t}=\frac{d}{t_w}=\frac{h-2t_f-2r}{t_w}=\frac{152-2\cdot 9{,}0-2\cdot 15}{6{,}0}=17{,}33$$

$$a=\frac{N_{Ed}}{2*t_w*\frac{f_y}{\gamma_{m0}}}=\frac{5{,}08}{2*0{,}6*\frac{27{,}5}{1}}=0{,}15$$

$$\alpha=\frac{1}{d}*\left(\frac{d}{2}+a\right)=\frac{1}{10{,}4}*\left(\frac{10{,}4}{2}+0{,}15\right)=0{,}51;\ \alpha>0{,}5$$

$$\frac{c}{t}=\frac{396\varepsilon}{13\alpha-1};17{,}33<64{,}71$$

Hrbat je klasa 1.

POJASNICA

$$\frac{c}{t}=\frac{c}{t_f}=\left(\frac{\frac{b}{2}-\frac{t_w}{2}-r}{t_f}\right)=\frac{\frac{160}{2}-\frac{6}{2}-15}{9{,}0}=6{,}88$$

$$\frac{c}{t}=9\varepsilon;\ 6{,}88<8{,}28$$

Pojasnica je klasa 1.

Poprečni presjek je **KLASA 1.**

Otpornost poprečnog presjeka na tlačnu silu:

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{m0}} = \frac{38{,}8 * 27{,}5}{1} = 1067\ kN$$

$N_{c,Ed} < N_{c,Rd}$; $5{,}06\ kN < 1067\ kN$ **ZADOVOLJAVA** (1%)

Otpornost poprečnog presjeka na savijanje:

$$M_{c,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{245{,}1 * 27{,}5}{1} = 67{,}42\ kNm$$

$M_{y,Ed} < M_{c,Rd}$; $30{,}98\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (46%)

Otpornost poprečnog presjeka na posmik:

Provjera izbočivanja hrpta na posmik

$$\frac{h_w}{t_w} \leq 72\frac{\varepsilon}{\eta}$$

$$\frac{h_w}{t_w} \leq \frac{h - 2t_f}{t_w} = \frac{152 - 2 * 9{,}0}{6{,}0} = 22{,}33$$

$22{,}33 \leq 72 * \frac{0{,}92}{1{,}2} = 79{,}49$ Ne treba provjera izbočavanja hrpta na posmik

Posmična otpornost:

$$V_{pl,rd} = \frac{A_{v,z} * \frac{f_y}{\sqrt{3}}}{\gamma_{m0}} = \frac{13{,}21 * \frac{27{,}5}{\sqrt{3}}}{1{,}0} = 209{,}74\ kN$$

$V_{z,Ed} \leq V_{pl,Rd}$; $29{,}83\ kN \leq 209{,}74\ kN$ **ZADOVOLJAVA** (14%)

Interakcija M-N:

$N_{Ed} \leq 0{,}25 N_{c,Rd}$

$5{,}06 < 0{,}25 * 1067 = 266{,}75\ kN$ **ZADOVOLJAVA** (2%)

$$N_{Ed} \leq \frac{0{,}5 * h_w * t_w * f_y}{\gamma_{m0}} = \frac{0{,}5 * 13{,}4 * 0{,}6 * 27{,}5}{1} = 110{,}55$$

$5{,}06\ kN < 110{,}55\ kN$ **ZADOVOLJAVA** (5%)

Uzdužna sila ima nisku razinu utjecaja na otpornost presjeka na savijanje

$M_{y,Ed} \leq M_{V,N,y,Rd} = M_{c,Rd}$

$30{,}98\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (46%)

Interakcija M-V:

$V_{z,Ed} \leq 0{,}5 V_{pl,Rd}$

$29{,}83\ kN < 0{,}5 * 209{,}74 = 104{,}87\ kN$

Ne trebamo redukciju plastične otpornosti na savijanje

$M_{y,Ed} \leq M_{y,V,Rd}$

$30{,}98\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (46%)

Otpornost elemenata na izvijanje:

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

Oko osi y:

$$N_{cr} = \frac{\pi^2 * E * I}{l_{cr}^2} = \frac{\pi^2 * 21000 * 1673}{800^2} = 541{,}80\ kN$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{38{,}8 * 27{,}5}{541{,}80}} = 1{,}40$$

$$\frac{h}{b} = \frac{152}{160} = 0{,}95 < 1{,}2 \rightarrow \alpha = b = 0{,}34$$

$$\Phi = 0{,}5 * [1 + \alpha * (\lambda - 0{,}2) + \lambda^2] = 0{,}5 * [1 + 0{,}34 * (1{,}4 - 0{,}2) + 1{,}4^2] = 1{,}68$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{1{,}68 + \sqrt{1{,}68^2 - 1{,}40^2}} = 0{,}38$$

Oko osi z:

$$N_{cr} = \frac{\pi^2 * E * I}{l_{cr}^2} = \frac{\pi^2 * 21000 * 1673}{200^2} = 8668{,}72\ kN$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{38{,}8 * 27{,}5}{8668{,}72}} = 0{,}35$$

$$\frac{h}{b} = \frac{152}{160} = 0{,}95 < 1{,}2 \rightarrow \alpha = c = 0{,}49$$

$$\Phi = 0{,}5 * [1 + \alpha * (\lambda - 0{,}2) + \lambda^2] = 0{,}5 * [1 + 0{,}49 * (0{,}35 - 0{,}2) + 0{,}35^2] = 0{,}60$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{0{,}60 + \sqrt{0{,}60^2 - 0{,}35^2}} = 0{,}92$$

$$N_{b,Rd} = \chi * \frac{A * f_y}{\gamma_{M1}} = 0{,}38 * \frac{38{,}8 * 27{,}5}{1} = 405{,}46\ kN$$

$N_{c,Ed} \leq N_{b,Rd};\ 5{,}06\ kN < 405{,}46\ kN$ **ZADOVOLJAVA** (1%)

Otpornost elementa na bočno torzijsko izvijanje:

$$M_{cr} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{(k \cdot L)^2} \cdot \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k \cdot L)^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z}}\right]$$

$$= 2{,}58 * \frac{\pi^2 * 21000 * 615{,}5}{(1 * 200)^2} * [\sqrt{\left(\frac{1}{1}\right)^2 * \frac{31{,}41}{615{,}5} + \frac{(1 * 200)^2 * 8076 * 12{,}19}{\pi^2 * 21000 * 615{,}5}}]$$

$$M_{cr} = 45753{,}25\ kNcm$$

Vitkost:

$$\overline{\lambda_{LT}} = \sqrt{\frac{W_{pl,y} * f_y}{M_{cr}}} = \sqrt{\frac{245{,}2 * 27{,}5}{45753{,}25}} = 0{,}38 < 0{,}4\ (zdepasti\ nosač)$$

Mjerodavna linija izvijanja:

$$\frac{h}{b} = \frac{152}{160} = 0{,}95 < 2 \rightarrow \alpha_{LT} = a = 0{,}21$$

Koeficijent redukcije:

$$\chi_{LT} = 1$$

$$M_{y,Ed} \leq M_{b,Rd}$$

$30{,}98\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (46%)

Provjera progiba:

Maksimalni progib od ukupnog djelovanja:

$\delta_{inst,z} = 2{,}7\ cm$

Dopušteni progib:

$\delta_{max,inst,z} = \frac{800}{200} = 4{,}0\ cm$

$\delta_{inst,z} < \delta_{max,inst,z}$; $2{,}7\ cm < 4{,}0\ cm$ **ZADOVOLJAVA**

Maksimalni progib od promjenjivog djelovanja:

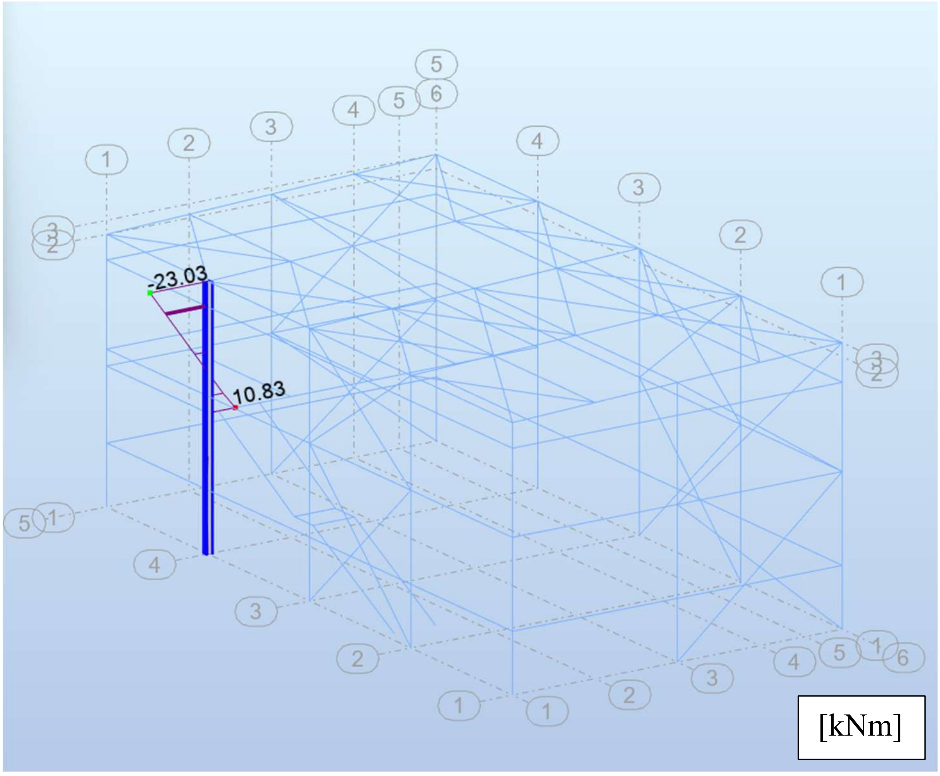
$\delta_{inst,z} = 2{,}3\ cm$

Dopušteni progib:

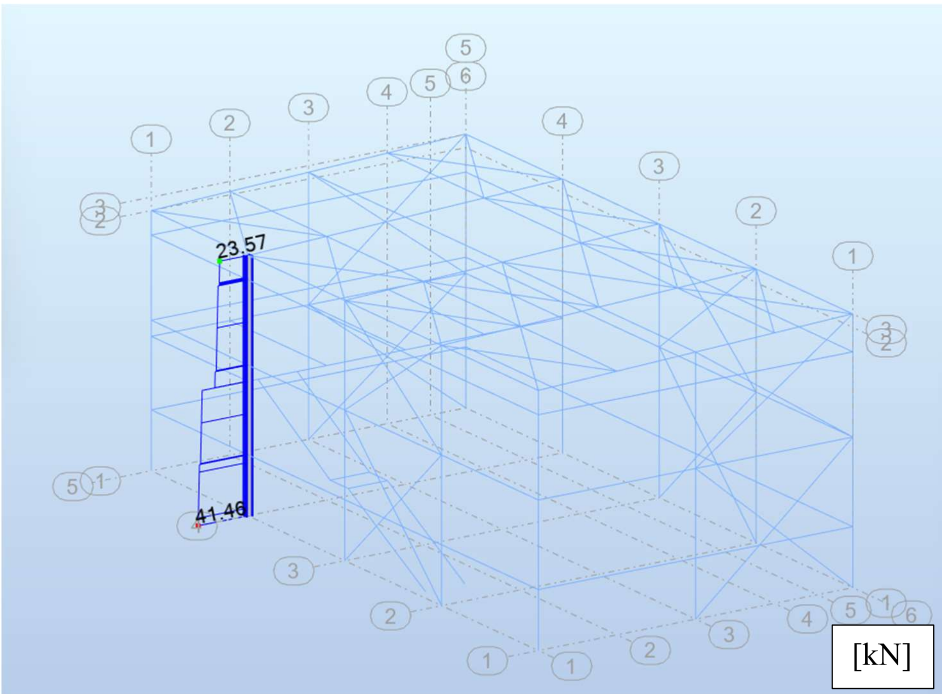
$\delta_{max,inst,z} = \frac{800}{250} = 3{,}2\ cm$

$\delta_{inst,z} < \delta_{max,inst,z}$; $2{,}3\ cm < 3{,}2\ cm$ **ZADOVOLJAVA**

8.3. Stupovi POZ S2



Slika 24: Mjerodavna vrijednost My za glavne stupove [izradio autor]



Slika 25: Mjerodavna vrijednost Fx za glavne stupove [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 34 gl. stupovi_34 **POINT:** 3 **COORDINATE:** x = 0.91 L = 5.26 m

----

LOADS:
Governing Load Case: 9 COMB1 (1+2)*1.35+4*1.50

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: HEA 160

h=152 mm	gM0=1.00	gM1=1.00	
b=160 mm	Ay=3253 mm2	Az=1321 mm2	Ax=3877 mm2
tw=6 mm	Iy=16729800 mm4	Iz=6155730 mm4	Ix=109000 mm4
tf=9 mm	Wply=245167 mm3	Wplz=117635 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 24.89 kN	My,Ed = -16.53 kN*m	Mz,Ed = 0.14 kN*m	Vy,Ed = -0.12 kN
Nc,Rd = 1066.21 kN	My,Ed,max = -23.03 kN*m		Mz,Ed,max = -0.28 kN*m
	Vy,T,Rd = 516.05 kN		
Nb,Rd = 275.19 kN	My,c,Rd = 67.42 kN*m	Mz,c,Rd = 32.35 kN*m	Vz,Ed = -11.84 kN
	MN,y,Rd = 67.42 kN*m	MN,z,Rd = 32.35 kN*m	Vz,T,Rd = 209.64 kN
	Mb,Rd = 67.42 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 1828.75 kN*m	Curve,LT - b	XLT = 1.00
Lcr,low=0.75 m	Lam_LT = 0.19	fi,LT = 0.48	XLT,mod = 1.00

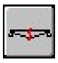
BUCKLING PARAMETERS:

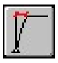
About y axis:		About z axis:	
Ly = 5.81 m	Lam_y = 1.03	Lz = 5.81 m	Lam_z = 1.70
Lcr,y = 5.81 m	Xy = 0.58	Lcr,z = 5.81 m	Xz = 0.26
Lamy = 88.37	kyy = 1.03	Lamz = 145.69	kyz = 0.77

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.02 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.25 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.00 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.06 < 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 = 0.06 < 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 = 88.37 < Lambda,max = 210.00   Lambda,z = 145.69 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.34 < 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) = 0.40 < 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) = 0.28 < 1.00   (6.3.3.(4))

LIMIT DISPLACEMENTS


Deflections (LOCAL SYSTEM): *Not analyzed*


Displacements (GLOBAL SYSTEM):
vx = 16 mm < vx max = L/150.00 = 39 mm   Verified
Governing Load Case: 34 COMB22 (1+2+5)*1.00+4*0.50
vy = 3 mm < vy max = L/150.00 = 39 mm   Verified
Governing Load Case: 7 WIND3

Section OK !!!

Profil: HEA 160

Materijal: S275

$N_{c,Ed} = 41{,}46\ kN$

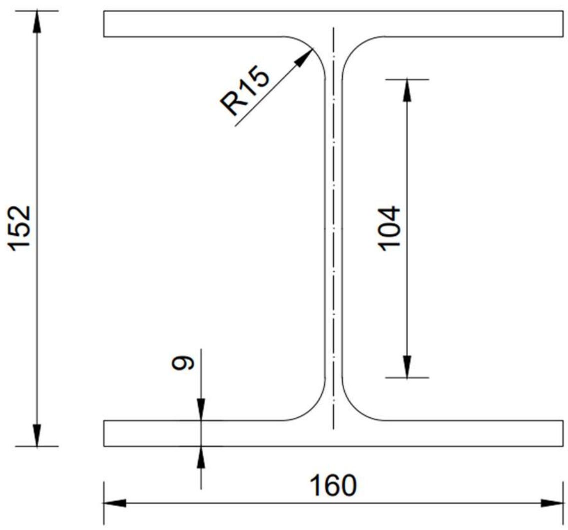
$V_{z,Ed} = 11{,}84\ kN$

$M_{y,Ed} = 23{,}03\ kNm$

$M_{z,Ed} = 0{,}14\ kNm$

Tablica 13: Karakteristike profila HEA 160 [izradio autor]

Profil	G	A	h	b	tw	tf	r	I_t	$I_w \cdot 10^{-3}$
	kg/m	cm^2	mm	mm	mm	mm	mm	cm^4	cm^6
HEA 160	30,4	38,8	152	160	6	9	15	12,19	31,41



Slika 26: Prikaz profila HEA 160 [izradio autor]

Klasifikacija poprečnog presjeka:

HRBAT

$$\frac{c}{t}=\frac{d}{t_w}=\frac{h-2t_f-2r}{t_w}=\frac{152-2\cdot 9{,}0-2\cdot 15}{6{,}0}=17{,}33$$

$$a=\frac{N_{Ed}}{2*t_w*\frac{f_y}{\gamma_{m0}}}=\frac{51{,}01}{2*0{,}6*\frac{27{,}5}{1}}=1{,}55$$

$$\alpha=\frac{1}{d}*\left(\frac{d}{2}+a\right)=\frac{1}{10{,}4}*\left(\frac{10{,}4}{2}+1{,}55\right)=0{,}65;\ \alpha>0{,}5$$

$$\frac{c}{t}=\frac{396\varepsilon}{13\alpha-1};17{,}33<49{,}14$$

Hrbat je <u>klasa 1.</u>

POJASNICA

$$\frac{c}{t}=\frac{c}{t_f}=\left(\frac{\frac{b}{2}-\frac{t_w}{2}-r}{t_f}\right)=\frac{\frac{160}{2}-\frac{6}{2}-15}{9{,}0}=6{,}88$$

$$\frac{c}{t}=9\varepsilon;\ 6{,}88<8{,}28$$

Pojasnica je <u>klasa 1.</u>

Poprečni presjek je **KLASA 1.**

Otpornost poprečnog presjeka na tlačnu silu:

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{m0}} = \frac{38{,}8 * 27{,}5}{1} = 1067\ kN$$

$N_{c,Ed} < N_{c,Rd}$; $41{,}46\ kN < 1067\ kN$ **ZADOVOLJAVA** (4%)

Otpornost poprečnog presjeka na savijanje:

$$M_{c,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{245{,}1 * 27{,}5}{1} = 67{,}42\ kNm$$

$M_{y,Ed} < M_{c,Rd}$; $23{,}03\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (34%)

Otpornost poprečnog presjeka na posmik:

Provjera izbočivanja hrpta na posmik

$$\frac{h_w}{t_w} \leq 72 \frac{\varepsilon}{\eta}$$

$$\frac{h_w}{t_w} \leq \frac{h - 2t_f}{t_w} = \frac{152 - 2 * 9{,}0}{6{,}0} = 22{,}33$$

$22{,}33 \ \leq 72 * \frac{0{,}92}{1{,}2} = 79{,}49$ Ne treba provjera izbočavanja hrpta na posmik

Posmična otpornost:

$$V_{pl,rd} = \frac{A_{v,z} * \frac{f_y}{\sqrt{3}}}{\gamma_{m0}} = \frac{13{,}21 * \frac{27{,}5}{\sqrt{3}}}{1{,}0} = 209{,}74\ kN$$

$V_{z,Ed} \leq V_{pl,Rd}$; $11{,}84\ kN \leq 209{,}74\ kN$ **ZADOVOLJAVA** (6%)

Interakcija M-N:

$N_{Ed} \leq 0{,}25 N_{c,Rd}$

$41{,}46 < 0{,}25 * 1067 = 266{,}75\ kN$ **ZADOVOLJAVA** (15%)

$$N_{Ed} \leq \frac{0{,}5 * h_w * t_w * f_y}{\gamma_{m0}} = \frac{0{,}5 * 13{,}4 * 0{,}6 * 27{,}5}{1} = 110{,}55$$

$41{,}46\ kN < 110{,}55\ kN$ **ZADOVOLJAVA** (38%)

Uzdužna sila ima nisku razinu utjecaja na otpornost presjeka na savijanje

$M_{y,Ed} \leq M_{V,N,y,Rd} = M_{c,Rd}$

$23{,}03\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (34%)

Interakcija M-V:

$V_{z,Ed} \leq 0{,}5 V_{pl,Rd}$

$11{,}84\ kN < 0{,}5 * 209{,}74 = 104{,}87\ kN$

Ne trebamo redukciju plastične otpornosti na savijanje

$M_{y,Ed} \leq M_{y,V,Rd}$

$23{,}03\ kNm < 67{,}42\ kNm$ **ZADOVOLJAVA** (34%)

Otpornost elemenata na izvijanje:

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

Oko osi y:

$$N_{cr} = \frac{\pi^2 * E * I}{l_{cr}^2} = \frac{\pi^2 * 21000 * 1673}{600^2} = 963{,}19\ kN$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{38{,}8 * 27{,}5}{963{,}19}} = 1{,}05$$

$$\frac{h}{b} = \frac{152}{160} = 0,95 < 1,2 \rightarrow \alpha = b = 0,34$$

$$\Phi = 0,5 * [1 + \alpha * (\lambda - 0,2) + \lambda^2] = 0,5 * [1 + 0,34 * (1,05 - 0,2) + 1,05^2] = 1,20$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{1,20 + \sqrt{1,20^2 - 1,05^2}} = 0,56$$

Oko osi z:

$$N_{cr} = \frac{\pi^2 * E * I}{l_{cr}^2} = \frac{\pi^2 * 21000 * 1673}{300^2} = 3852,76\ kN$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{38,8 * 27,5}{3852,76}} = 0,53$$

$$\frac{h}{b} = \frac{152}{160} = 0,95 < 1,2 \rightarrow \alpha = c = 0,49$$

$$\Phi = 0,5 * [1 + \alpha * (\lambda - 0,2) + \lambda^2] = 0,5 * [1 + 0,49 * (0,53 - 0,2) + 0,53^2] = 0,72$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{0,72 + \sqrt{0,72^2 - 0,53^2}} = 0,83$$

$$N_{b,Rd} = \chi * \frac{A * f_y}{\gamma_{M1}} = 0,56 * \frac{38,8 * 27,5}{1} = 597,52\ kN$$

$N_{c,Ed} \leq N_{b,Rd};\ 41,46\ kN < 597,52\ kN$ **ZADOVOLJAVA** (9%)

<u>Otpornost elementa na bočno torzijsko izvijanje:</u>

$C_1 = 1,31$

$L_{cr} = 200\ cm$

$$M_{cr} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{(k \cdot Lcr)^2} \cdot \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k \cdot L)^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z}} \right]$$

$$= 1,31 * \frac{\pi^2 * 21000 * 615,5}{(1 * 200)^2} * [\sqrt{\left(\frac{1}{1}\right)^2 * \frac{31,41}{615,5} + \frac{(1 * 200)^2 * 8076 * 12,19}{\pi^2 * 21000 * 615,5}}]$$

$$M_{cr} = 23231,30\ kNcm$$

Vitkost:

$$\overline{\lambda_{LT}} = \sqrt{\frac{W_{pl,y} * f_y}{M_{cr}}} = \sqrt{\frac{245,2 * 27,5}{23231,30}} = 0,54$$

Mjerodavna linija izvijanja:

$$\frac{h}{b} = \frac{152}{160} = 0,95 < 2 \rightarrow \alpha_{LT} = a = 0,21$$

Koeficijent redukcije:

$$\Phi = 0,5 * \left[1 + \alpha * \left(\overline{\lambda_{LT}} - 0,2\right) + \overline{\lambda_{LT}}^2\right] = 0,5 * [1 + 0,21 * (0,54 - 0,2) + 0,54^2]$$
$$= 0,68$$

$$\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{0,68 + \sqrt{0,68^2 - 0,54^2}} = 0,91$$

$$M_{b,Rd} = \chi_{LT} * \frac{W_{pl,y} * f_y}{\gamma_{M1}} = 0,91 * \frac{245,2 * 275}{1} = 61361,3\ kNcm = 613,6\ kNm$$

$$M_{y,Ed} \leq M_{b,Rd}$$

$23,03\ kNm < 613,6\ kNm$ **ZADOVOLJAVA** (4%)

<u>Interakcija izvijanja i bočno-torzijskog izvijanja (N-M):</u>

Uvjeti:

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

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

$$N_{Rk} = A * f_y = 38,8 * 27,5 = 1067\ kN$$

$$M_{y,Rk} = W_{pl,y} * f_y = 245,2 * 27,5 = 6743\ kNcm = 67,43\ kNm$$

Potrebne vrijednosti za daljnju interakciju:

$\lambda_y = 1{,}05$

$\lambda_z = 0{,}53$

$\Phi_y = 1{,}20$

$\Phi_z = 0{,}72$

$\chi_y = 0{,}56$

$\chi_z = 0{,}83$

$C_{my} = 0{,}6 + 0{,}4 * 0{,}47 = 0{,}79$

$$k_{yy} = C_{my} * \left[1 + \left(\overline{\lambda_Y} - 0{,}2\right) * \frac{N_{Ed}}{\phi_y * \frac{N_{Rk}}{\gamma_{M1}}}\right] \leq C_{my} * \left(1 + 0{,}8 * \frac{N_{Ed}}{\chi_y * \frac{N_{Rk}}{\gamma_{M1}}}\right)$$

$$k_{yy} = 0{,}79 * \left[1 + (1{,}05 - 0{,}2) * \frac{51{,}05}{1{,}20 * \frac{1067}{1{,}1}}\right] \leq 0{,}79 * \left(1 + 0{,}8 * \frac{51{,}05}{0{,}56 * \frac{1067}{1{,}1}}\right)$$

$k_{yy} = 0{,}82 < 0{,}85$

$k_{Zy} = 0{,}60 * 0{,}82 = 0{,}49$

$$\frac{N_{Ed}}{\chi_y * \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} * \frac{M_{y,Ed}}{\chi_{LT} * \frac{M_{y,Rk}}{\gamma_{M1}}} \leq 1; \frac{41{,}46}{0{,}56 * \frac{1067}{1{,}1}} + 0{,}82 * \frac{23{,}03}{1 * \frac{67{,}43}{1{,}1}} \leq 1\,; 0{,}39 < 1$$

ZADOVOLJAVA (39%)

$$\frac{N_{Ed}}{\chi_z * \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} * \frac{M_{y,Ed}}{\chi_{LT} * \frac{M_{y,Rk}}{\gamma_{M1}}} \leq 1; \frac{41{,}46}{0{,}83 * \frac{1067}{1{,}1}} + 0{,}49 * \frac{23{,}03}{1 * \frac{67{,}43}{1{,}1}} \leq 1\,; 0{,}24 < 1$$

ZADOVOLJAVA (24%)

Provjera progiba:

Maksimalni progib od ukupnog djelovanja:

$\delta_{inst,x} = 1{,}6\ cm$

Dopušteni progib:

$$\delta_{max,inst,x} = \frac{600}{150} = 4\ cm$$

$\delta_{inst,x} < \delta_{max,inst,x}$; 1,6 cm < 4,0 cm **ZADOVOLJAVA**

Maksimalni progib od promjenjivog djelovanja:

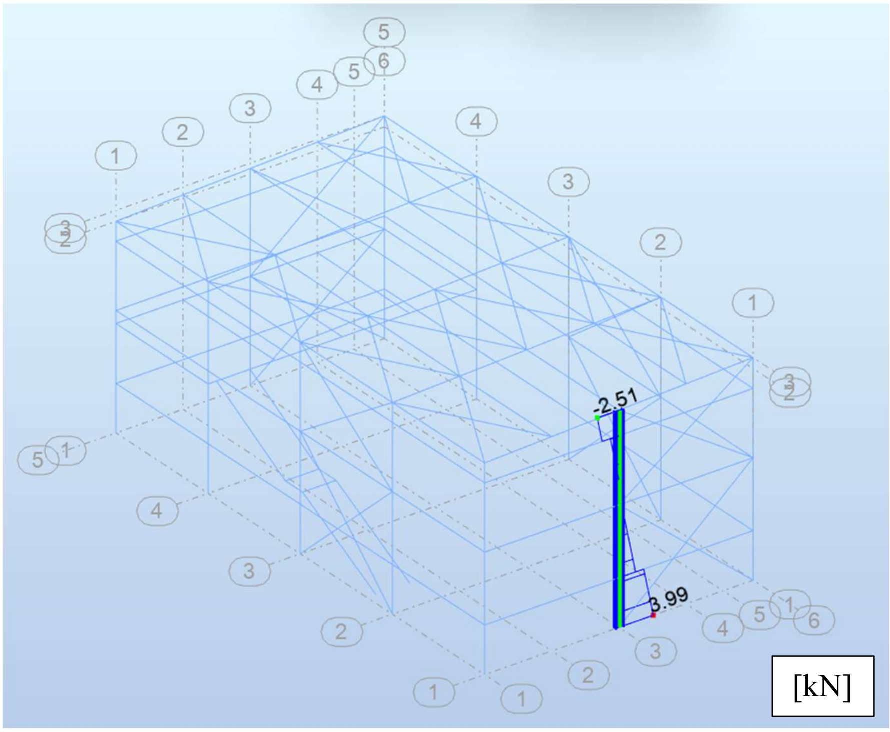
$\delta_{inst,z} = 0{,}5\ cm$

Dopušteni progib:

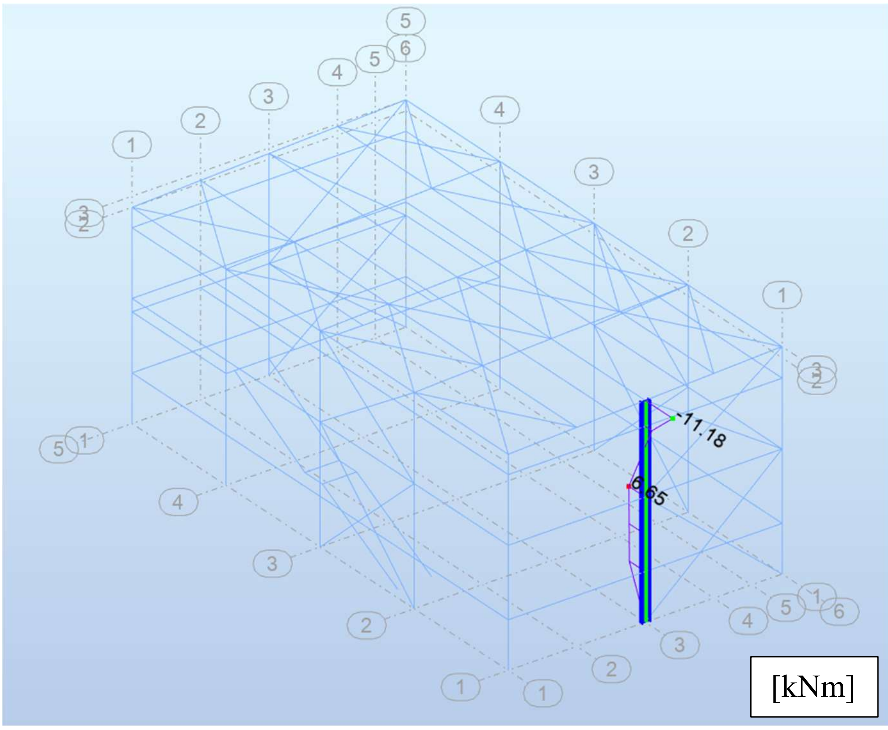
$$\delta_{max,inst,z} = \frac{375}{250} = 1{,}5\ cm$$

$\delta_{inst,z} < \delta_{max,inst,z}$; 0,50 cm < 3,2 cm **ZADOVOLJAVA**

8.4. Sekundarni stupovi POZ SS1



Slika 27: Mjerodavna vrijednost Fx za sekundarne stupove [izradio autor]



Slika 28: Mjerodavna vrijednost Mz za sekundarne stupove [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 112 sekundarni stup_112 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

-----

LOADS:
Governing Load Case: 19 COMB12 (1+2)*1.35+4*0.75+7*1.50

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: IPE 240

h=240 mm	gM0=1.00	gM1=1.00	
b=120 mm	Ay=2731 mm2	Az=1914 mm2	Ax=3912 mm2
tw=6 mm	Iy=38916300 mm4	Iz=2836340 mm4	Ix=116000 mm4
tf=10 mm	Wply=366679 mm3	Wplz=73927 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -2.51 kN	My,Ed = -0.13 kN*m	Mz,Ed = -11.18 kN*m	Vy,Ed = -12.13 kN
Nt,Rd = 1075.70 kN	My,pl,Rd = 100.84 kN*m	Mz,pl,Rd = 20.33 kN*m	Vy,T,Rd = 433.48 kN
	My,c,Rd = 100.84 kN*m	Mz,c,Rd = 20.33 kN*m	Vz,Ed = -0.64 kN
	MN,y,Rd = 100.84 kN*m	MN,z,Rd = 20.33 kN*m	Vz,T,Rd = 303.89 kN
	Mb,Rd = 45.66 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

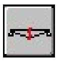
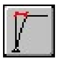
z = 0.00	Mcr = 48.15 kN*m	Curve,LT - b	XLT = 0.45
Lcr,low=5.96 m	Lam_LT = 1.45	fi,LT = 1.46	XLT,mod = 0.45

BUCKLING PARAMETERS:

About y axis: About z axis:

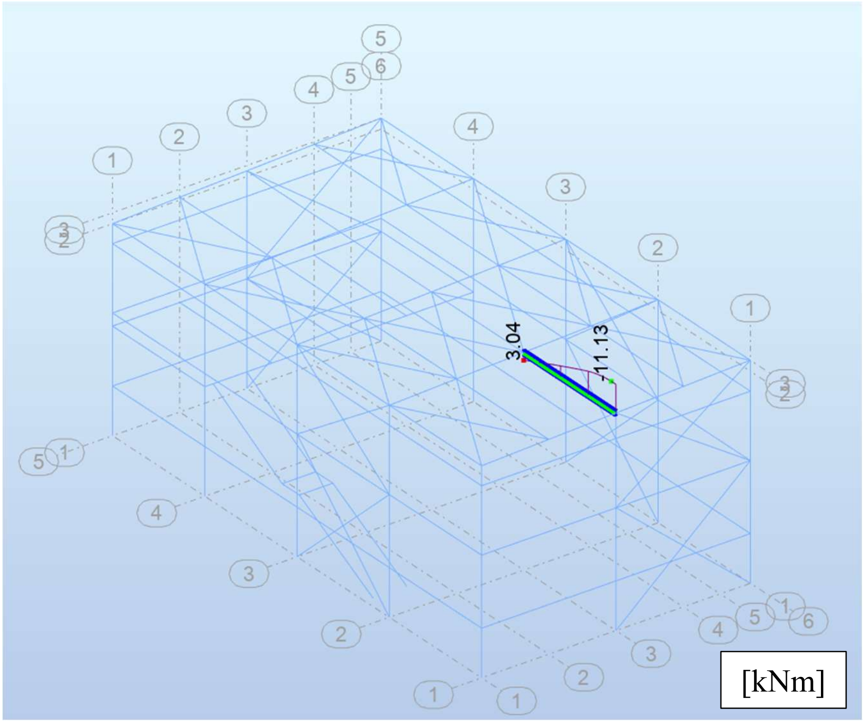
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
My,Ed/MN,y,Rd = 0.00 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.55 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.55 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.03 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 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:
My,Ed/Mb,Rd = 0.01 < 1.00   (6.3.2.1.(1))

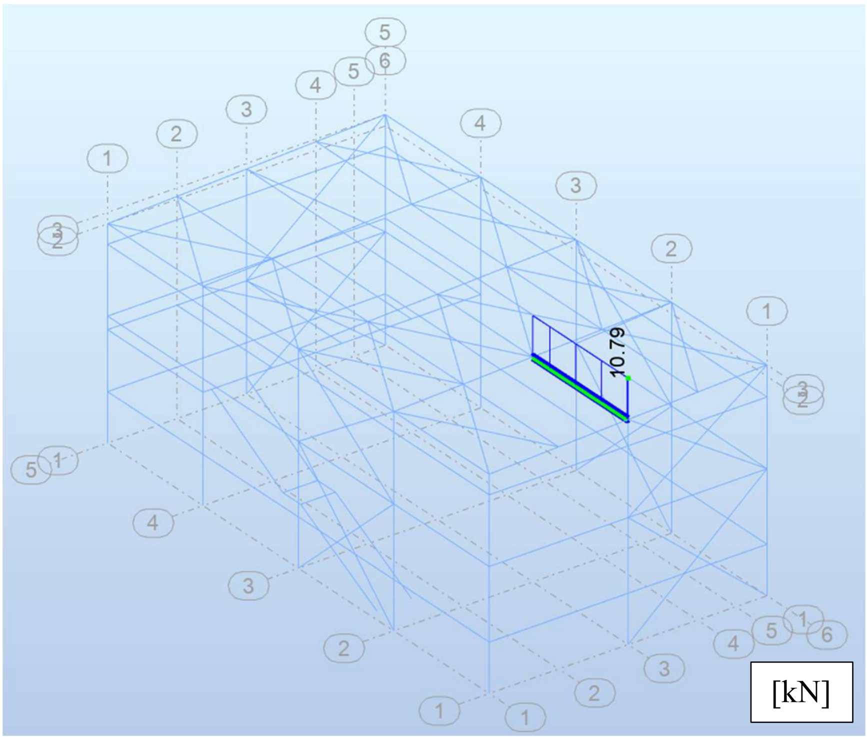
LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM): *Not analyzed*

Displacements (GLOBAL SYSTEM):
vx = 12 mm < vx max = L/150.00 = 40 mm   Verified
Governing Load Case: 34 COMB22 (1+2+5)*1.00+4*0.50
vy = 5 mm < vy max = L/150.00 = 40 mm   Verified
Governing Load Case: 7 WIND3

Section OK !!!

8.5. Podrožnice POZ P3



Slika 29: Vrijednost My za mjerodavni element [izradio autor]



Slika 30: Vrijednost Fx za mjerodavni element [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 48 podrožnice_48 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

----

LOADS:
Governing Load Case: 19 COMB12 (1+2)*1.35+4*0.75+7*1.50

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: IPE 160

h=160 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1373 mm2	Az=966 mm2	Ax=2009 mm2
tw=5 mm	Iy=8692930 mm4	Iz=683145 mm4	Ix=35300 mm4
tf=7 mm	Wply=123868 mm3	Wplz=26101 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 10.79 kN	My,Ed = -11.12 kN*m	Mz,Ed = 0.12 kN*m	Vy,Ed = 0.06 kN
Nc,Rd = 552.51 kN	My,Ed,max = -11.12 kN*m		Mz,Ed,max = 0.12 kN*m
	Vy,T,Rd = 218.00 kN		
Nb,Rd = 85.08 kN	My,c,Rd = 34.06 kN*m	Mz,c,Rd = 7.18 kN*m	Vz,Ed = -0.38 kN
	MN,y,Rd = 34.06 kN*m	MN,z,Rd = 7.18 kN*m	Vz,T,Rd = 153.32 kN
	Mb,Rd = 18.29 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 20.47 kN*m	Curve,LT - b	XLT = 0.53
Lcr,low=3.75 m	Lam_LT = 1.29	fi,LT = 1.28	XLT,mod = 0.54

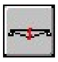
BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 3.75 m	Lam_y = 0.66	Lz = 3.75 m	Lam_z = 2.37
Lcr,y = 3.75 m	Xy = 0.86	Lcr,z = 3.75 m	Xz = 0.15
Lamy = 57.01	kyy = 1.10	Lamz = 203.37	kyz = 1.02

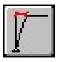
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.02 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.33 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.02 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.12 < 1.00 (6.2.9.1.(6))

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 = 57.01 < Lambda,max = 210.00   Lambda,z = 203.37 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.61 < 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) = 0.71 < 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))

LIMIT DISPLACEMENTS



Deflections (LOCAL SYSTEM):
uy = 1 mm < uy max = L/200.00 = 19 mm   Verified
Governing Load Case: 34 COMB22 (1+2+5)*1.00+4*0.50
uz = 5 mm < uz max = L/200.00 = 19 mm   Verified
Governing Load Case: 8 WIND4
u inst,y = 0 mm < u inst,max,y = L/250.00 = 15 mm   Verified
Governing Load Case: 0.5*4 + 1*5
u inst,z = 5 mm < u inst,max,z = L/250.00 = 15 mm   Verified
Governing Load Case: 1*8



Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

Profil: IPE 160

Materijal: S275

$N_{c,Ed} = 10{,}79\ kN$

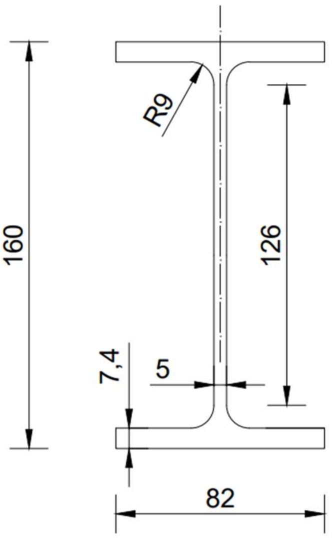
$V_{z,Ed} = 0{,}38\ kN$

$M_{y,Ed} = 11{,}12\ kNm$

$M_{z,Ed} = 0{,}12\ kNm$

Tablica 14: Karakteristike profila IPE 160 [izradio autor]

Profil	G	A	h	b	tw	tf	r	I_t	$I_{w*}10^{-3}$
	kg/m	cm^2	mm	mm	mm	mm	mm	cm^4	cm^6
IPE 160	15,8	20,09	160	82	5	7,4	9	3,60	3,96



Slika 31: Prikaz profila IPE 160 [izradio autor]

Klasifikacija poprečnog presjeka:

HRBAT

$$\frac{c}{t} = \frac{d}{t_w} = \frac{h - 2t_f - 2r}{t_w} = \frac{160 - 2 \cdot 7{,}4 - 2 \cdot 9}{5{,}0} = 25{,}44$$

$$a = \frac{N_{Ed}}{2 * t_w * \frac{f_y}{\gamma_{m0}}} = \frac{10{,}79}{2 * 0{,}5 * \frac{27{,}5}{1}} = 0{,}39$$

$$\alpha = \frac{1}{d} * \left(\frac{d}{2} + a\right) = \frac{1}{12{,}6} * \left(\frac{12{,}6}{2} + 0{,}39\right) = 0{,}53;\ \alpha > 0{,}5$$

$$\frac{c}{t} \leq \frac{396\varepsilon}{13\alpha - 1}; 25{,}44 < 61{,}85$$

Hrbat je klasa 1.

POJASNICA

$$\frac{c}{t} = \frac{c}{t_f} = \left(\frac{\frac{b}{2} - \frac{t_w}{2} - r}{t_f}\right) = \frac{\frac{82}{2} - \frac{5}{2} - 9}{7,4} = 3,99$$

$$\frac{c}{t} \leq 9\varepsilon;\ 3,99 < 8,28$$

Pojasnica je <u>klasa 1.</u>

Poprečni presjek je **KLASA 1.**

<u>Otpornost poprečnog presjeka na tlačnu silu:</u>

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{m0}} = \frac{20,09 * 27,5}{1} = 552,48\ kN$$

$N_{c,Ed} < N_{c,Rd};\ \ 10,79\ kN < 552,48\ kN$ **ZADOVOLJAVA** (2%)

<u>Otpornost poprečnog presjeka na savijanje:</u>

$$M_{c,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{123,9 * 27,5}{1} = 34,07\ kNm$$

$M_{y,Ed} < M_{c,Rd};\ \ 11,12\ kNm < 34,07\ kNm$ **ZADOVOLJAVA** (33%)

<u>Otpornost poprečnog presjeka na posmik:</u>

Provjera izbočivanja hrpta na posmik

$$\frac{h_w}{t_w} \leq 72\frac{\varepsilon}{\eta}$$

$$\frac{h_w}{t_w} \leq \frac{h - 2t_f}{t_w} = \frac{160 - 2 * 7,4}{5,0} = 29,04$$

$29,04\ \leq 72 * \frac{0,92}{1,2} = 79,49$ Ne treba provjera izbočavanja hrpta na posmik

Posmična otpornost:

$$V_{pl,rd} = \frac{A_{v,z} * \frac{f_y}{\sqrt{3}}}{\gamma_{m0}} = \frac{9,66 * \frac{27,5}{\sqrt{3}}}{1,0} = 153,37\ kN$$

$V_{z,Ed} \leq V_{pl,Rd};\ 0,38\ kN \leq 153,37\ kN$ **ZADOVOLJAVA** (1%)

Interakcija M-N:

$N_{Ed} \leq 0,25 N_{c,Rd}$

$10,79 < 0,25 * 552,48 = 138,12\ kN$ **ZADOVOLJAVA** (8%)

$$N_{Ed} \leq \frac{0,5 * h_w * t_w * f_y}{\gamma_{m0}} = \frac{0,5 * 14,5 * 0,5 * 27,5}{1} = 99,68\ kN$$

$10,79\ kN < 99,68\ kN$ **ZADOVOLJAVA** (11%)

Uzdužna sila ima nisku razinu utjecaja na otpornost presjeka na savijanje

$M_{y,Ed} \leq M_{V,N,y,Rd} = M_{c,Rd}$

$11,12\ kNm < 34,07\ kNm$ **ZADOVOLJAVA** (33%)

Interakcija M-V:

$V_{z,Ed} \leq 0,5 V_{pl,Rd}$

$0,38\ kN < 0,5 * 153,37 = 76,69\ kN$

Ne trebamo redukciju plastične otpornosti na savijanje

$M_{y,Ed} \leq M_{y,V,Rd}$

$11,12\ kNm < 34,07\ kNm$ **ZADOVOLJAVA** (33%)

<u>Otpornost elemenata na izvijanje:</u>

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

Oko osi y:

$$N_{cr} = \frac{\pi^2 * E * I}{l_{cr}^2} = \frac{\pi^2 * 21000 * 869{,}3}{375^2} = 1281{,}23\ kN$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{20{,}09 * 27{,}5}{1281{,}23}} = 0{,}66$$

$$\frac{h}{b} = \frac{160}{82} = 1{,}95\ >\ 1{,}2 \rightarrow \alpha = b = 0{,}34$$

$$\Phi = 0{,}5 * [1 + \alpha * (\lambda - 0{,}2) + \lambda^2] = 0{,}5 * [1 + 0{,}34 * (0{,}66 - 0{,}2) + 0{,}66^2] = 0{,}80$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{0{,}80 + \sqrt{0{,}80^2 - 0{,}66^2}} = 0{,}80$$

Oko osi z:

$$N_{cr} = \frac{\pi^2 * E * I}{l_{cr}^2} = \frac{\pi^2 * 21000 * 68{,}31}{93{,}72^2} = 1611{,}90\ kN$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{20{,}09 * 27{,}5}{1611{,}90}} = 0{,}59$$

$$\frac{h}{b} = \frac{160}{82} = 1{,}95\ >\ 1{,}2 \rightarrow \alpha = c = 0{,}49$$

$$\Phi = 0{,}5 * [1 + \alpha * (\lambda - 0{,}2) + \lambda^2] = 0{,}5 * [1 + 0{,}49 * (0{,}59 - 0{,}2) + 0{,}59^2] = 0{,}77$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{0{,}77 + \sqrt{0{,}77^2 - 0{,}59^2}} = 0{,}79$$

$$N_{b,Rd} = \chi * \frac{A * f_y}{\gamma_{M1}} = 0{,}79 * \frac{20{,}09 * 27{,}5}{1} = 436{,}46\ kN$$

$N_{c,Ed} \leq N_{b,Rd};\ 10{,}79\ kN < 436{,}46\ kN$ **ZADOVOLJAVA** (3%)

Otpornost elementa na bočno torzijsko izvijanje:

$C_1 = 2{,}05$

$L_{cr} = 93{,}75\ cm$

$$M_{cr} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{(k \cdot Lcr)^2} \cdot \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k \cdot L)^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z}}\right]$$

$$= 2{,}05 * \frac{\pi^2 * 21000 * 68{,}31}{(1 * 93{,}75)^2} * [\sqrt{\left(\frac{1}{1}\right)^2 * \frac{3{,}96}{68{,}31} + \frac{(1 * 93{,}75)^2 * 8076 * 3{,}60}{\pi^2 * 21000 * 68{,}31}}]$$

$M_{cr} = 14051{,}74\ kNcm$

Vitkost:

$$\overline{\lambda_{LT}} = \sqrt{\frac{W_{pl,y} * f_y}{M_{cr}}} = \sqrt{\frac{123{,}9 * 27{,}5}{14051{,}74}} = 0{,}49$$

Mjerodavna linija izvijanja:

$$\frac{h}{b} = \frac{160}{82} = 1{,}95 < 2 \rightarrow \alpha_{LT} = a = 0{,}21$$

Koeficijent redukcije:

$$\Phi = 0{,}5 * \left[1 + \alpha * \left(\overline{\lambda_{LT}} - 0{,}2\right) + \overline{\lambda_{LT}}^2\right] = 0{,}5 * [1 + 0{,}21 * (0{,}49 - 0{,}2) + 0{,}49^2]$$
$$= 0{,}65$$

$$\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{0{,}65 + \sqrt{0{,}65^2 - 0{,}49^2}} = 0{,}93$$

$$M_{b,Rd} = \chi_{LT} * \frac{W_{pl,y} * f_y}{\gamma_{M1}} = 0{,}93 * \frac{123{,}9 * 27{,}5}{1} = 31687{,}43\ kNcm = 31{,}69\ kNm$$

$M_{y,Ed} \leq M_{b,Rd}$

$11{,}12\ kNm < 31{,}69\ kNm$ **ZADOVOLJAVA** (35%)

Interakcija izvijanja i bočno-torzijskog izvijanja (N-M):

Uvjeti:

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

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

$$N_{Rk} = A * f_y = 20{,}09 * 27{,}5 = 552{,}48\ kN$$

$$M_{y,Rk} = W_{pl,y} * f_y = 123{,}9 * 27{,}5 = 3407{,}25\ kNcm = 34{,}07\ kNm$$

Potrebne vrijednosti za daljnju interakciju:

$\lambda_y = 0{,}66$

$\lambda_z = 0{,}59$

$\Phi_y = 0{,}80$

$\Phi_z = 0{,}77$

$\chi_y = 0{,}80$

$\chi_z = 0{,}79$

$C_{my} = 0{,}6 + 0{,}4 * 0{,}27 = 0{,}71$

$$k_{yy} = C_{my} * \left[1 + \left(\overline{\lambda_Y} - 0{,}2\right) * \frac{N_{Ed}}{\phi_y * \frac{N_{Rk}}{\gamma_{M1}}}\right] \leq C_{my} * \left(1 + 0{,}8 * \frac{N_{Ed}}{\chi_y * \frac{N_{Rk}}{\gamma_{M1}}}\right)$$

$$k_{yy} = 0{,}71 * \left[1 + (0{,}66 - 0{,}2) * \frac{10{,}79}{0{,}80 * \frac{552{,}48}{1{,}1}}\right] \leq 0{,}71 * \left(1 + 0{,}8 * \frac{10{,}79}{0{,}80 * \frac{552{,}48}{1{,}1}}\right)$$

$k_{yy} = 0{,}72 < 0{,}73$

$k_{Zy} = 0,60 * 0,73 = 0,44$

$$\frac{N_{Ed}}{\chi_y * \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} * \frac{M_{y,Ed}}{\chi_{LT} * \frac{M_{y,Rk}}{\gamma_{M1}}} \leq 1; \frac{10,79}{0,80 * \frac{552,48}{1,1}} + 0,72 * \frac{11,12}{1 * \frac{34,07}{1,1}} \leq 1\,; 0,29 < 1$$

ZADOVOLJAVA (29%)

$$\frac{N_{Ed}}{\chi_z * \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} * \frac{M_{y,Ed}}{\chi_{LT} * \frac{M_{y,Rk}}{\gamma_{M1}}} \leq 1; \frac{10,79}{0,79 * \frac{552,48}{1,1}} + 0,44 * \frac{11,12}{1 * \frac{34,07}{1,1}} \leq 1\,; 0,19 < 1$$

ZADOVOLJAVA (19%)

<u>Provjera progiba:</u>

Maksimalni progib od ukupnog djelovanja:

$\delta_{inst,z} = 0,7\ cm$

Dopušteni progib:

$$\delta_{max,inst,z} = \frac{375}{200} = 1,9\ cm$$

$\delta_{inst,z} < \delta_{max,inst,z}$; $0,7\ cm < 1,9\ cm$ **ZADOVOLJAVA**

Maksimalni progib od promjenjivog djelovanja:

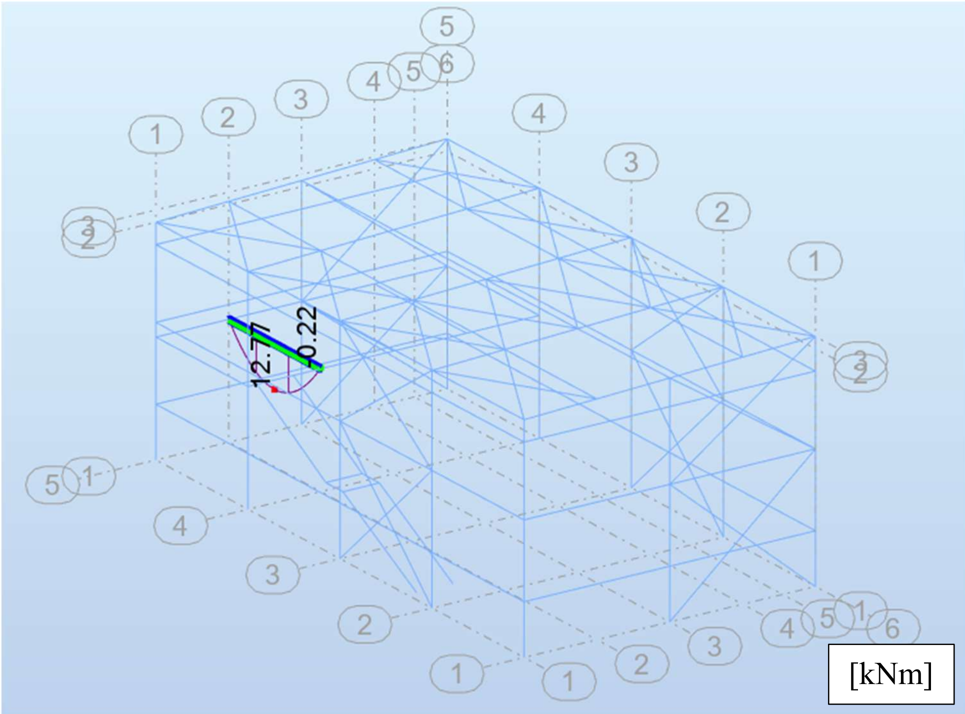
$\delta_{inst,z} = 0,5\ cm$

Dopušteni progib:

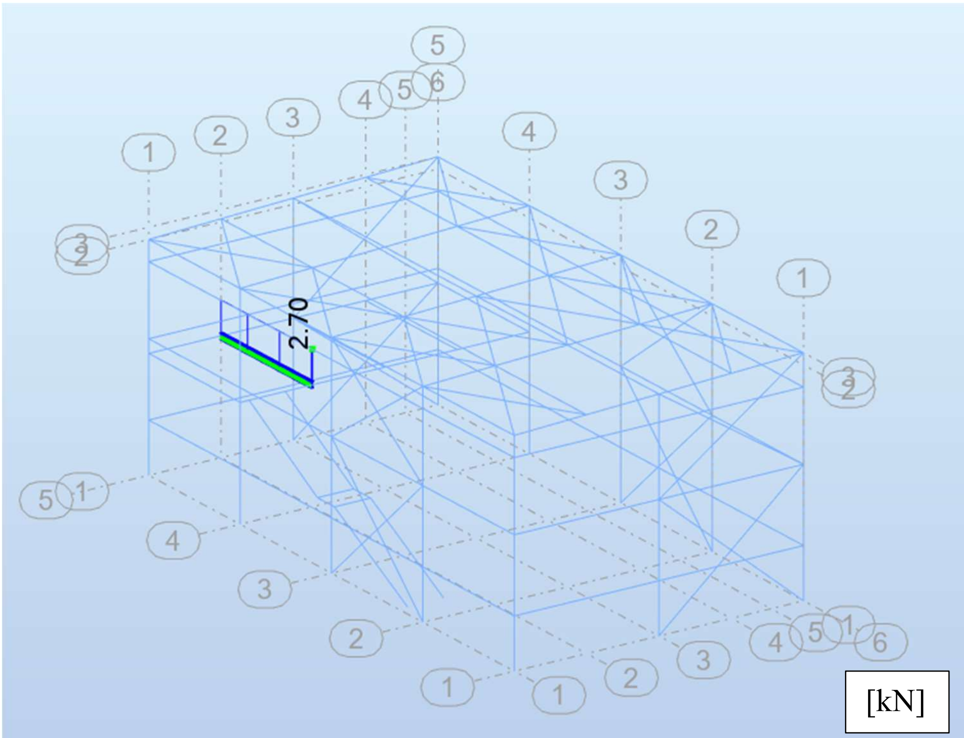
$$\delta_{max,inst,z} = \frac{375}{250} = 1,5\ cm$$

$\delta_{inst,z} < \delta_{max,inst,z}$; $0,50\ cm < 1,5\ cm$ **ZADOVOLJAVA**

8.6. Sekundarni nosači galerije POZ SG2



Slika 32: Mjerodavna vrijednost My za sekundarni nosač galerije [izradio autor]



Slika 33: Mjerodavna vrijednost Fx za sekundarni nosač galerije [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 211 sekundarni nosači_211 **POINT:** 2
COORDINATE: x = 0.50 L = 1.88 m

LOADS:
Governing Load Case: 25 COMB28 3*1.50+2*1.35

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: IPE 160

h=160 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1373 mm2	Az=966 mm2	Ax=2009 mm2
tw=5 mm	Iy=8692930 mm4	Iz=683145 mm4	Ix=35300 mm4
tf=7 mm	Wply=123868 mm3	Wplz=26101 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 2.70 kN	My,Ed = 12.77 kN*m	Mz,Ed = -0.02 kN*m	Vy,Ed = 0.11 kN
Nc,Rd = 552.51 kN	My,Ed,max = 12.77 kN*m	Mz,Ed,max = -0.22 kN*m	Vy,T,Rd = 217.24 kN
Nb,Rd = 85.08 kN	My,c,Rd = 34.06 kN*m	Mz,c,Rd = 7.18 kN*m	Vz,Ed = 0.04 kN
	MN,y,Rd = 34.06 kN*m	MN,z,Rd = 7.18 kN*m	Vz,T,Rd = 152.96 kN
	Mb,Rd = 33.90 kN*m		Tt,Ed = 0.01 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 154.42 kN*m	Curve,LT - b	XLT = 0.97
Lcr,upp=0.94 m	Lam_LT = 0.47	fi,LT = 0.59	XLT,mod = 1.00

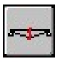
BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 3.75 m	Lam_y = 0.66	Lz = 3.75 m	Lam_z = 2.37
Lcr,y = 3.75 m	Xy = 0.86	Lcr,z = 3.75 m	Xz = 0.15
Lamy = 57.01	kyy = 1.02	Lamz = 203.37	kyz = 0.73

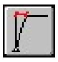
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.00 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.37 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.00 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.14 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)

Vz,Ed/Vz,T,Rd = 0.00 < 1.00   (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 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 = 57.01 < Lambda,max = 210.00   Lambda,z = 203.37 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.38 < 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) = 0.41 < 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) = 0.26 < 1.00   (6.3.3.(4))

LIMIT DISPLACEMENTS



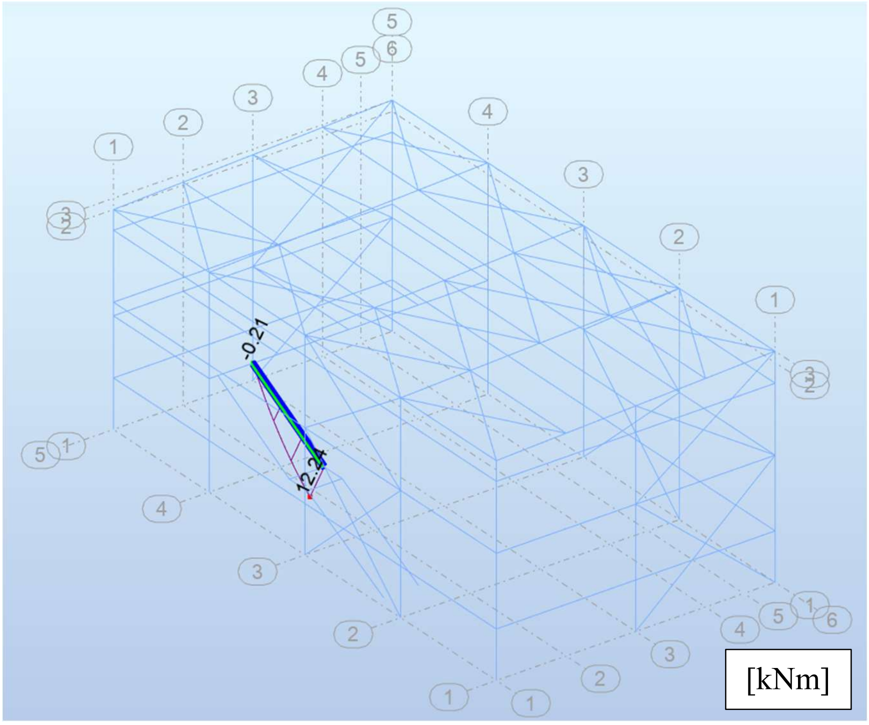
Deflections (LOCAL SYSTEM):
uy = 1 mm < uy max = L/200.00 = 19 mm   Verified
Governing Load Case: 37 COMB25 (1+2+8)*1.00+4*0.50
uz = 6 mm < uz max = L/200.00 = 19 mm   Verified
Governing Load Case: 3 LL1
u inst,y = 1 mm < u inst,max,y = L/250.00 = 15 mm   Verified
Governing Load Case: 0.5*4 + 1*8
u inst,z = 0 mm < u inst,max,z = L/250.00 = 15 mm   Verified
Governing Load Case: 1*5



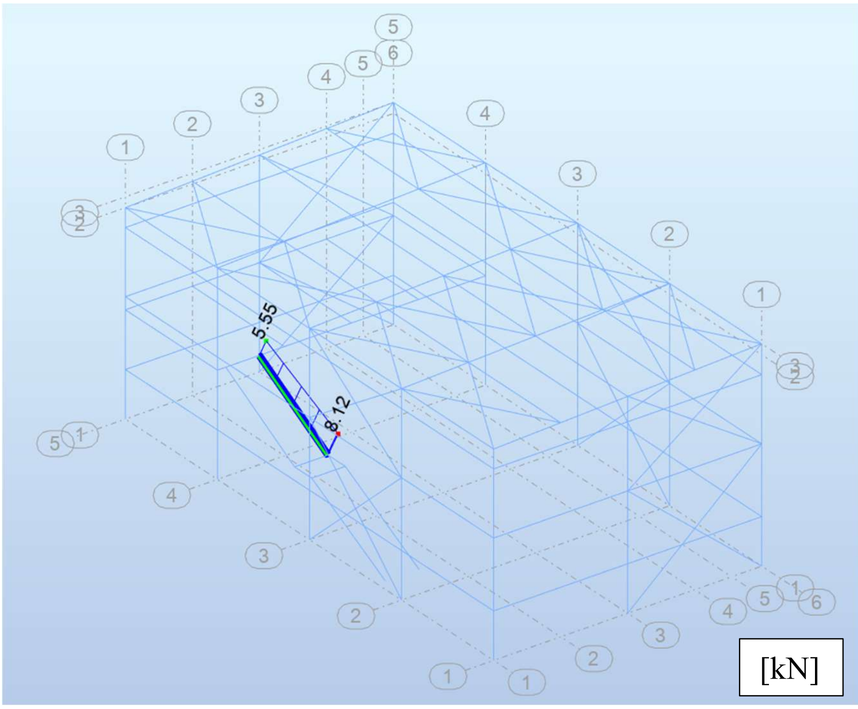
Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

8.7. Tetive stubišta POZ T4



Slika 34: Mjerodavna vrijednost My za tetive stubišta [izradio autor]



Slika 35: Mjerodavna vrijednost Fx za tetive stubišta [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 215 tetive_215 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

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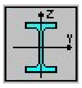
LOADS:
Governing Load Case: 40 COMB28 1*1.35+3*1.50

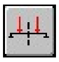
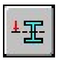
MATERIAL:
S275 (S275) fy = 275.00 MPa


SECTION PARAMETERS: IPE 160

h=160 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1373 mm2	Az=966 mm2	Ax=2009 mm2
tw=5 mm	Iy=8692930 mm4	Iz=683145 mm4	Ix=35300 mm4
tf=7 mm	Wply=123868 mm3	Wplz=26101 mm3	

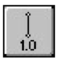
INTERNAL FORCES AND CAPACITIES:

N,Ed = 8.12 kN	My,Ed = 12.24 kN*m	Mz,Ed = -0.12 kN*m	Vy,Ed = -0.11 kN
Nc,Rd = 552.51 kN	My,Ed,max = 12.24 kN*m	Mz,Ed,max = 0.25 kN*m	Vy,T,Rd = 217.85 kN
Nb,Rd = 114.88 kN	My,c,Rd = 34.06 kN*m	Mz,c,Rd = 7.18 kN*m	Vz,Ed = -1.53 kN
	MN,y,Rd = 34.06 kN*m	MN,z,Rd = 7.18 kN*m	Vz,T,Rd = 153.25 kN
	Mb,Rd = 34.06 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

 
LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 208.01 kN*m	Curve,LT - b	XLT = 1.00
Lcr,upp=0.79 m	Lam_LT = 0.40	fi,LT = 0.56	XLT,mod = 1.00

BUCKLING PARAMETERS:

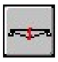
 About y axis:   About z axis:

Ly = 3.18 m	Lam_y = 0.56	Lz = 3.18 m	Lam_z = 2.01
Lcr,y = 3.18 m	Xy = 0.90	Lcr,z = 3.18 m	Xz = 0.21
Lamy = 48.29	kyy = 1.02	Lamz = 172.26	kyz = 0.76

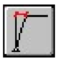
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.36 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.02 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.15 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)

Vz,Ed/Vz,T,Rd = 0.01 < 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 = 48.29 < Lambda,max = 210.00   Lambda,z = 172.26 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.36 < 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) = 0.41 < 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) = 0.30 < 1.00   (6.3.3.(4))

LIMIT DISPLACEMENTS



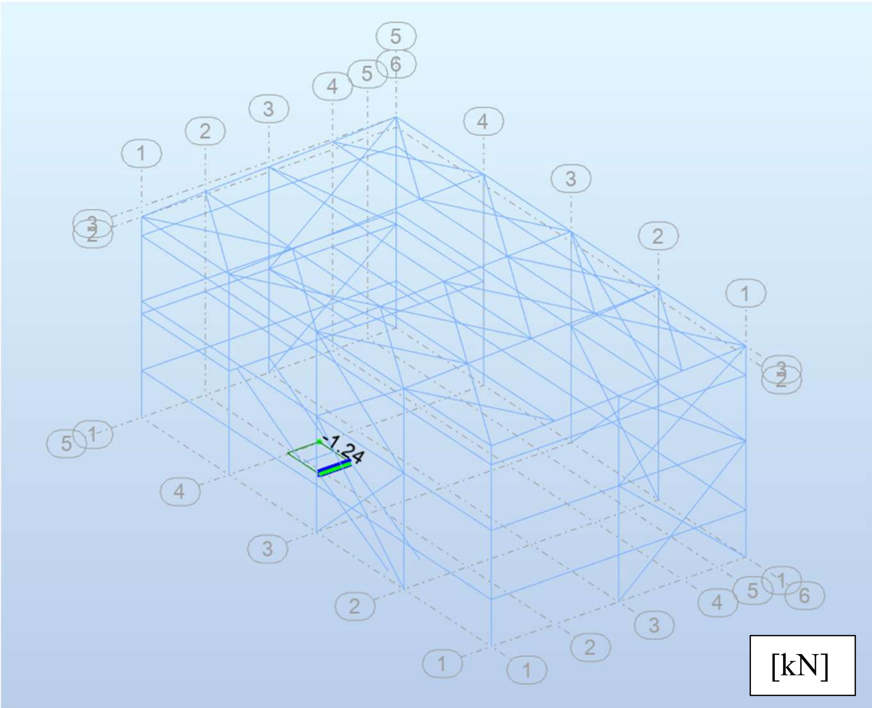
Deflections (LOCAL SYSTEM):
uy = 1 mm < uy max = L/200.00 = 16 mm   Verified
Governing Load Case: 5 WIND1
uz = 3 mm < uz max = L/200.00 = 16 mm   Verified
Governing Load Case: 3 LL1
u inst,y = 1 mm < u inst,max,y = L/250.00 = 13 mm   Verified
Governing Load Case: 0.5*4 + 1*5
u inst,z = 1 mm < u inst,max,z = L/250.00 = 13 mm   Verified
Governing Load Case: 0.5*4 + 1*5



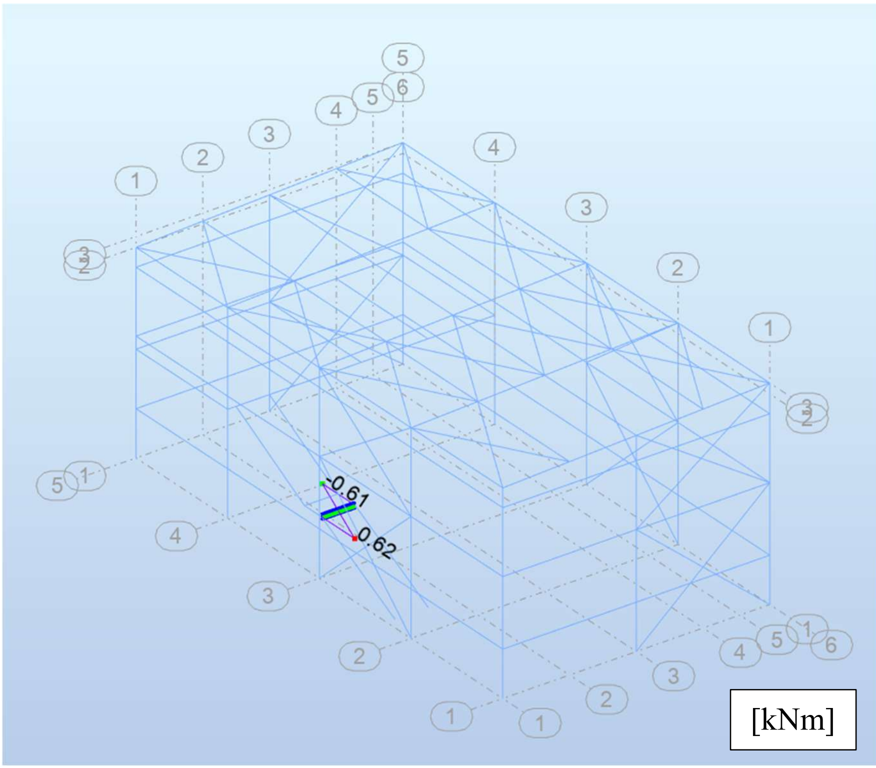
Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

8.8. Sekundarni nosači stubišta POZ TS1



Slika 36: Mjerodavna vrijednost Fy sekundarnog nosača stubišta [izradio autor]



Slika 37: Mjerodavna vrijednost Mz za sekundarni nosač stubišta [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 163 sekundarni nosači_163 **POINT:** 1
COORDINATE: x = 0.00 L = 0.00 m

LOADS:
Governing Load Case: 17 COMB10 (1+2)*1.35+4*0.75+5*1.50

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: IPE 160

h=160 mm	gM0=1.00	gM1=1.00	
b=82 mm	Ay=1373 mm2	Az=966 mm2	Ax=2009 mm2
tw=5 mm	Iy=8692930 mm4	Iz=683145 mm4	Ix=35300 mm4
tf=7 mm	Wply=123868 mm3	Wplz=26101 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 0.05 kN	My,Ed = -0.30 kN*m	Mz,Ed = -0.61 kN*m	Vy,Ed = -1.24 kN
Nc,Rd = 552.51 kN	My,Ed,max = -0.30 kN*m	Mz,Ed,max = 0.62 kN*m	Vy,T,Rd = 217.85 kN
Nb,Rd = 453.36 kN	My,c,Rd = 34.06 kN*m	Mz,c,Rd = 7.18 kN*m	Vz,Ed = 0.71 kN
	MN,y,Rd = 34.06 kN*m	MN,z,Rd = 7.18 kN*m	Vz,T,Rd = 153.25 kN
	Mb,Rd = 34.06 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 1922.11 kN*m	Curve,LT - b	XLT = 1.00
Lcr,low=0.25 m	Lam_LT = 0.13	fi,LT = 0.46	XLT,mod = 1.00

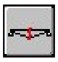
BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 1.00 m	Lam_y = 0.18	Lz = 1.00 m	Lam_z = 0.63
Lcr,y = 1.00 m	Xy = 1.00	Lcr,z = 1.00 m	Xz = 0.82
Lamy = 15.20	kzy = 0.52	Lamz = 54.23	kzz = 1.00

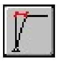
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.00 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.01 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.09 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.09 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.01 < 1.00 (6.2.6-7)

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 = 15.20 < Lambda,max = 210.00   Lambda,z = 54.23 < Lambda,max = 210.00   STABLE
My,Ed,max/Mb,Rd = 0.01 < 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) = 0.07 < 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) = 0.09 < 1.00   (6.3.3.(4))

LIMIT DISPLACEMENTS



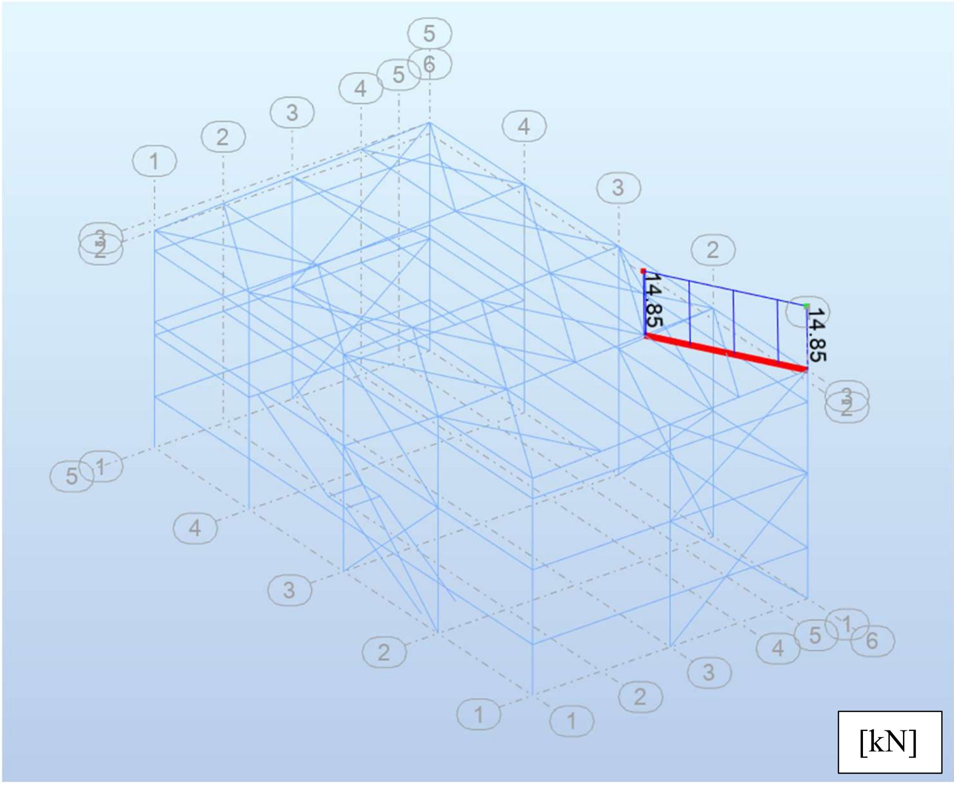
Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 5 mm   Verified
Governing Load Case: 5 WIND1
uz = 0 mm < uz max = L/200.00 = 5 mm   Verified
Governing Load Case: 34 COMB22 (1+2+5)*1.00+4*0.50
u inst,y = 0 mm < u inst,max,y = L/250.00 = 4 mm   Verified
Governing Load Case: 0.5*4 + 1*5
u inst,z = 0 mm < u inst,max,z = L/250.00 = 4 mm   Verified
Governing Load Case: 0.5*4 + 1*5



Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

8.9. Vjetrovni vezovi POZ 101



Slika 38: Mjerodavna vrijednost Fx za vjetrovni vez [izradio autor]

STEEL DESIGN

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

CODE GROUP:
MEMBER: 226 vezovi_226 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 17 COMB10 (1+2)*1.35+4*0.75+5*1.50

MATERIAL:
S275 (S275) fy = 275.00 MPa

SECTION PARAMETERS: šipka fi 10

h=10 mm	gM0=1.00	gM1=1.00	
	Ay=50 mm2	Az=50 mm2	Ax=79 mm2
tw=5 mm	Iy=491 mm4	Iz=491 mm4	Ix=982 mm4
	Wply=167 mm3	Wplz=167 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 14.85 kN	My,Ed = -0.01 kN*m	Mz,Ed = 0.00 kN*m	Vy,Ed = 0.00 kN
Nc,Rd = 21.60 kN	My,pl,Rd = 0.05 kN*m	Mz,pl,Rd = 0.05 kN*m	Vy,T,Rd = 7.93 kN
Nb,Rd = 21.60 kN	My,c,Rd = 0.05 kN*m	Mz,c,Rd = 0.05 kN*m	Vz,Ed = 0.02 kN
	MN,y,Rd = 0.02 kN*m	MN,z,Rd = 0.02 kN*m	Vz,T,Rd = 7.93 kN
			Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

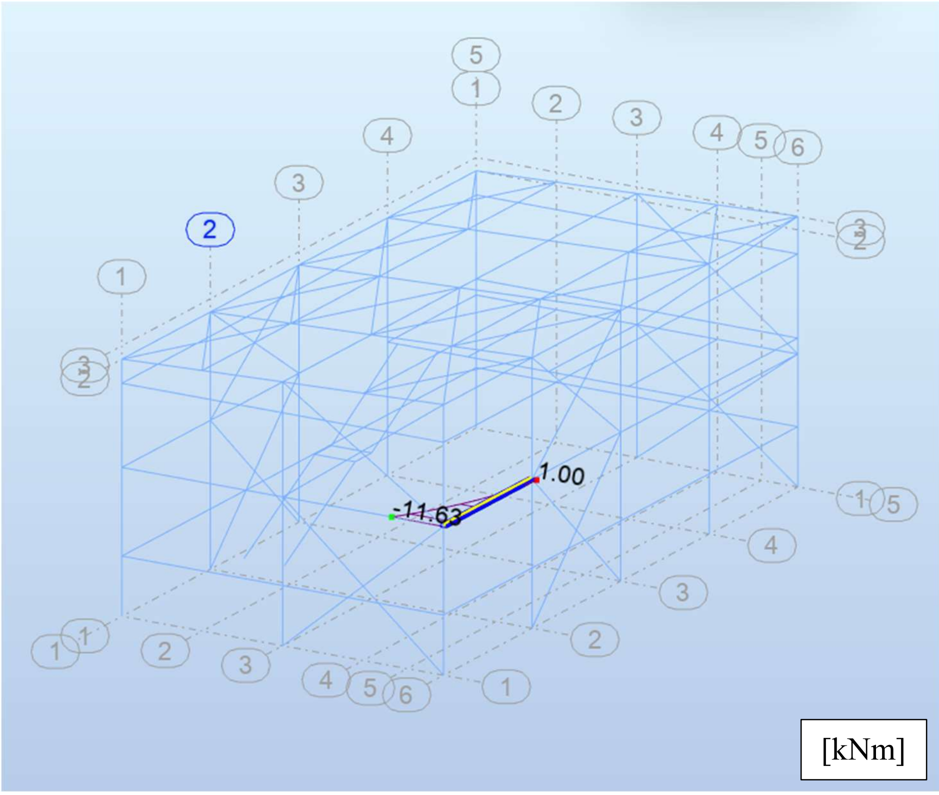
BUCKLING PARAMETERS:

About y axis: About z axis:

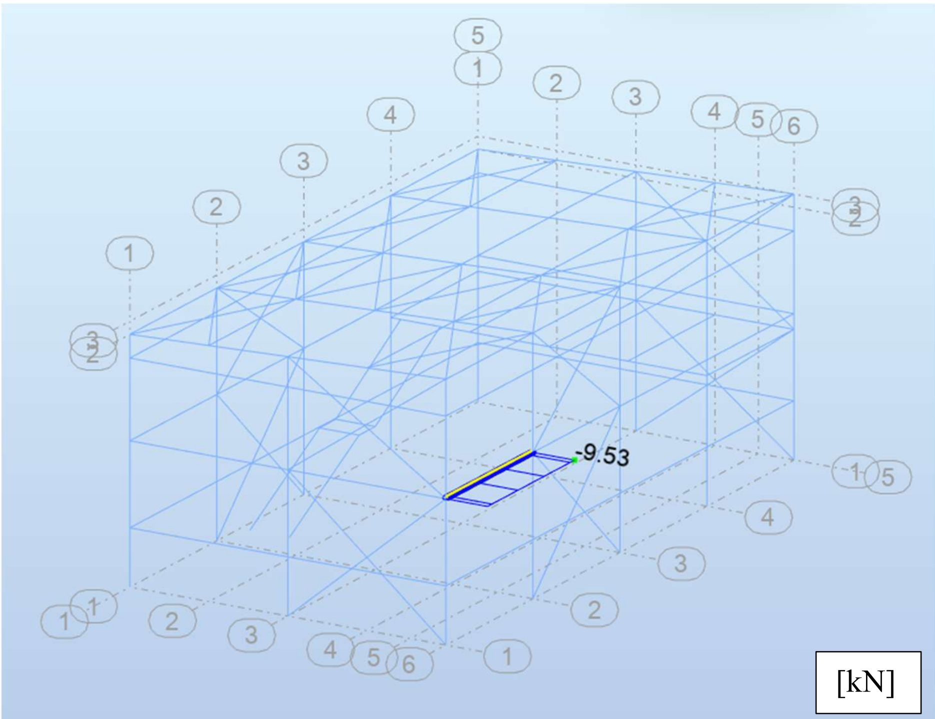
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.69 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.57 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.01 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^2.00 = 0.32 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 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)

Section OK !!

8.10. Fasadni nosači POZ F2



Slika 39: Mjerodavna vrijednost My za fasadni nosač [izradio autor]



Slika 40: Mjerodavna vrijednost Fx za fasadni nosač [izradio autor]

STEEL DESIGN

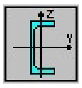
CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 176 fasadni nosači_176 **POINT:** 3 **COORDINATE:** x = 1.00 L = 3.75 m

----

LOADS:
Governing Load Case: 17 COMB10 (1+2)*1.35+4*0.75+5*1.50

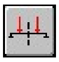
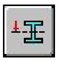
MATERIAL:
S275 (S275) fy = 275.00 MPa


SECTION PARAMETERS: UAP 150

h=150 mm	gM0=1.00	gM1=1.00	
b=65 mm	Ay=1521 mm2	Az=1128 mm2	Ax=2284 mm2
tw=7 mm	Iy=7960560 mm4	Iz=932542 mm4	Ix=67600 mm4
tf=10 mm	Wply=125278 mm3	Wplz=37937 mm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -9.53 kN	My,Ed = -11.63 kN*m	Mz,Ed = 0.28 kN*m	Vy,Ed = -0.45 kN
Nt,Rd = 628.12 kN	My,pl,Rd = 34.45 kN*m	Mz,pl,Rd = 10.43 kN*m	Vy,T,Rd = 241.36 kN
	My,c,Rd = 34.45 kN*m	Mz,c,Rd = 10.43 kN*m	Vz,Ed = -6.01 kN
	MN,y,Rd = 34.44 kN*m	MN,z,Rd = 10.43 kN*m	Vz,T,Rd = 179.09 kN
	Mb,Rd = 18.91 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

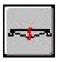
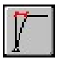
 
LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 31.61 kN*m	Curve,LT - d	XLT = 0.53
Lcr,low=3.75 m	Lam_LT = 1.04	fi,LT = 1.15	XLT,mod = 0.55

BUCKLING PARAMETERS:
 About y axis:  About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.02 < 1.00 (6.2.3.(1))
My,Ed/MN,y,Rd = 0.34 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.03 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.36 < 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)) = 0.00 < 1.00 (6.2.6)

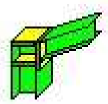
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.62 < 1.00   (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 1 mm < uy max = L/200.00 = 19 mm   Verified
Governing Load Case: 36 COMB24 (1+2+7)*1.00+4*0.50
uz = 5 mm < uz max = L/200.00 = 19 mm   Verified
Governing Load Case: 7 WIND3
u inst,y = 0 mm < u inst,max,y = L/250.00 = 15 mm   Verified
Governing Load Case: 0.5*4 + 1*7
u inst,z = 4 mm < u inst,max,z = L/250.00 = 15 mm   Verified
Governing Load Case: 1*7

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

9. DIMENZIONIRANJE PRIKLJUČAKA KONSTRUKCIJE

9.1. Stup i gredni nosač (DETALJ A)



Robot Structural Analysis Professional 2025

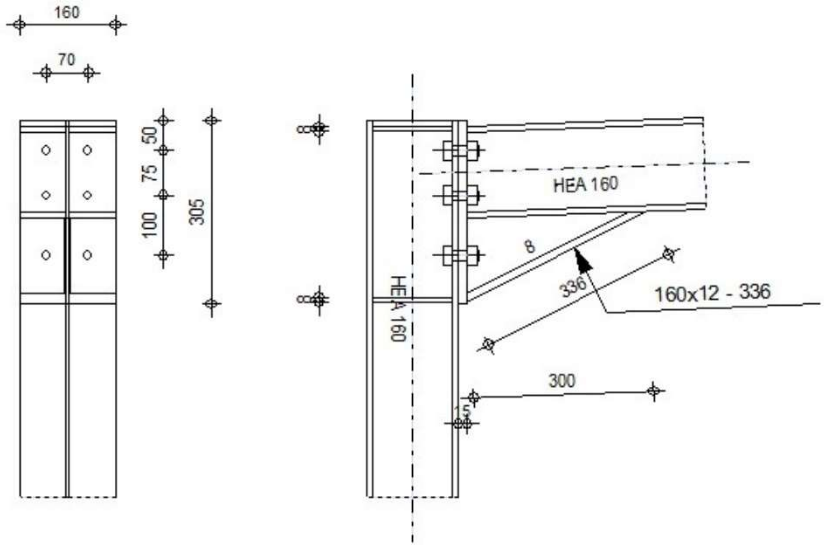
Design of fixed beam-to-column connection

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

OK

Ratio

0,54



General

Connection no.: 8

Connection name: Frame knee

Structure node: 50

Structure members: 36, 35

Geometry

Column

Section: HEA 160

Member no.: 36

$\alpha =$	-90,0	[Deg]	Inclination angle
$h_c =$	152	[mm]	Height of column section
$b_{fc} =$	160	[mm]	Width of column section

Section: HEA 160

t_{wc} = 6 [mm] Thickness of the web of column section

t_{fc} = 9 [mm] Thickness of the flange of column section

r_c = 15 [mm] Radius of column section fillet

A_c = 3877 [mm^2] Cross-sectional area of a column

I_{xc} = 16729800 [mm^4] Moment of inertia of the column section

Material: S275

f_{yc} = 275,00 [MPa] Resistance

Beam

Section: HEA 160

Member no.: 35

α = 2,2 [Deg] Inclination angle

h_b = 152 [mm] Height of beam section

b_f = 160 [mm] Width of beam section

t_{wb} = 6 [mm] Thickness of the web of beam section

t_{fb} = 9 [mm] Thickness of the flange of beam section

r_b = 15 [mm] Radius of beam section fillet

r_b = 15 [mm] Radius of beam section fillet

A_b = 3877 [mm^2] Cross-sectional area of a beam

I_{xb} = 16729800 [mm^4] Moment of inertia of the beam section

Material: S275

f_{yb} = 275,00 [MPa] Resistance

Bolts

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

d = 16 [mm] Bolt diameter

Class = 10.9 Bolt class

F_{tRd} = 113,04 [kN] Tensile resistance of a bolt

n_h = 2 Number of bolt columns

n_v = 3 Number of bolt rows

h_1 = 50 [mm] Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 70 [mm]

Vertical spacing p_i = 75;100 [mm]

Plate

$h_p =$ 305 [mm] Plate height

$b_p =$ 160 [mm] Plate width

$t_p =$ 15 [mm] Plate thickness

Material: S275

$f_{yp} =$ 275,00 [MPa] Resistance

Lower stiffener

$w_d =$ 160 [mm] Plate width

$t_{fd} =$ 12 [mm] Flange thickness

$h_d =$ 140 [mm] Plate height

$t_{wd} =$ 8 [mm] Web thickness

$l_d =$ 300 [mm] Plate length

$\alpha =$ 26,8 [Deg] Inclination angle

Material: STEEL 43-245

$f_{ybu} =$ 245,00 [MPa] Resistance

Column stiffener

Upper

$h_{su} =$ 134 [mm] Stiffener height

$b_{su} =$ 77 [mm] Stiffener width

$t_{hu} =$ 8 [mm] Stiffener thickness

Material: S275

$f_{ysu} =$ 275,00 [MPa] Resistance

Lower

$h_{sd} =$ 134 [mm] Stiffener height

$b_{sd} =$ 77 [mm] Stiffener width

$t_{hd} =$ 8 [mm] Stiffener thickness

Material: S275

$f_{ysu} =$ 275,00 [MPa] Resistance

Fillet welds

$a_w =$ 8 [mm] Web weld

$a_f =$ 8 [mm] Flange weld

$a_s =$ 8 [mm] Stiffener weld

$a_{fd} =$ 5 [mm] Horizontal 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: 9: COMB1 (1+2)*1.35+4*1.50

$M_{b1,Ed}$ =	23,45	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	18,18	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	-9,87	[kN]	Axial force in the right beam
$M_{c1,Ed}$ =	23,46	[kN*m]	Bending moment in the lower column
$V_{c1,Ed}$ =	11,23	[kN]	Shear force in the lower column
$N_{c1,Ed}$ =	-20,19	[kN]	Axial force in the lower column

Results

Beam resistances

COMPRESSION

A_b =	3877	[mm^2]	Area	EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b\ f_{yb} / \gamma_{M0}$

$N_{cb,Rd}$ =	1066,21	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

SHEAR

A_{vb} =	2441	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb}\ (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ =	387,58	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0,05 < 1,00	verified	(0,05)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} =	245167	[mm^3]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb}\ f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ =	67,42	[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} =	593341	[mm^3]	Plastic section modulus	EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl}\ f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ = 163,17 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 163,17 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

h_f = 281 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ = 580,91 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

β = 2,2 [Deg] Angle between the front plate and the beam

γ = 26,8 [Deg] Inclination angle of the bracket plate

$b_{eff,c,wb}$ = 171 [mm] Effective width of the web for compression [6.2.6.2.(1)]

A_{vb} = 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

ω = 0,75 Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed}$ = 75,44 [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)]

$F_{c,wb,Rd1} = [\omega\, k_{wc}\, b_{eff,c,wb}\, t_{wb}\, f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd1}$ = 453,30 [kN] Beam web resistance [6.2.6.2.(1)]

Buckling:

d_{wb} = 104 [mm] Height of compressed web [6.2.6.2.(1)]

λ_p = 0,76 Plate slenderness of an element [6.2.6.2.(1)]

ρ = 0,97 Reduction factor for element buckling [6.2.6.2.(1)]

$F_{c,wb,Rd2} = [\omega\, k_{wc}\, \rho\, b_{eff,c,wb}\, t_{wb}\, f_{yb} / \gamma_{M1}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd2}$ = 439,62 [kN] Beam web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wb,Rd,low} = \mathrm{Min}\,(F_{c,wb,Rd1}\,,\, F_{c,wb,Rd2})$

$F_{c,wb,Rd,low}$ = 439,62 [kN] Beam web resistance [6.2.6.2.(1)]

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed}$ = 23,45 [kN*m] Bending moment (right beam) [5.3.(3)]

$M_{b2,Ed}$ = 0,00 [kN*m] Bending moment (left beam) [5.3.(3)]

$V_{c1,Ed}$ = 11,23 [kN] Shear force (lower column) [5.3.(3)]

$V_{c2,Ed}$ = 0,00 [kN] Shear force (upper column) [5.3.(3)]

WEB PANEL - SHEAR

$M_{b1,Ed}$ = 23,45 [kN*m] Bending moment (right beam) [5.3.(3)]

z = 208 [mm] Lever arm [6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ = 107,19 [kN] Shear force acting on the web panel [5.3.(3)]

A_{vs} =	1321	[mm^2]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
d_s =	277	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	0,89	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	0,70	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	0,70	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9 (A_{vs}*f_{y,wc}) / (\sqrt{3} \gamma_{M0}) + Min(4 M_{pl,fc,Rd} / d_s , (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ = 200,30 [kN] Resistance of the column web panel for shear [6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \leq 1,0$ 0,54 < 1,00 verified (0,54)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	6	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	186	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0,72		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	78,11	[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)]
A_s =	1232	[mm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M0} + A_s\ f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ = 559,89 [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 =	0,79		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	0,94		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2,21		Stiffener slenderness	EN1993-1-1:[6.3.1.2]

Buckling:

d_{wc} =	104	[mm]	Height of compressed web	[6.2.6.2.(1)]
χ_s =	1,00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M1} + A_s\, \chi_s\, f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 547,56 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \mathrm{Min}\,(F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd}$ = 547,56 [kN] Column web resistance [6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

t_{wc} =	6	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	182	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0,73		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	78,11	[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)]
A_s =	1232	[mm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M0} + A_s\, f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ = 557,11 [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 =	0,78		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	0,95		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2,21		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
χ_s =	1,00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M1} + A_s\, \chi_s\, f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 546,58 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,upp} = \mathrm{Min}\,(F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd,upp}$ = 546,58 [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	20	-	45	-	75	126	154	126	154	138	124	124	124
2	20	-	45	-	88	126	136	126	136	175	88	88	88
3	20	-	45	-	100	126	136	126	136	163	118	118	118

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	-	45	-	75	144	172	144	172	147	136	136	136
2	23	-	45	-	88	144	148	144	148	175	88	88	88
3	23	-	45	-	100	144	148	144	148	172	124	124	124

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 row in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt row in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt row for mode 1

$l_{eff,2}$ – Effective length for a single bolt row 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 compression

$N_{j,Rd} = \text{Min} (N_{cb,Rd} 2 F_{c,wb,Rd,low} , 2 F_{c,wc,Rd,low} , 2 F_{c,wc,Rd,upp})$

$N_{j,Rd} =$ 879,23 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1{,}0$	0,01 < 1,00	verified	(0,01)

Connection resistance for bending

$F_{t,Rd} =$ 113,04 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd} =$ 140,06 [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

$F_{t,fc,Rd}$ – column flange resistance 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**
$F_{t1,Rd} = \mathrm{Min}\ (F_{t1,Rd,comp})$	139,96	Bolt row resistance
$F_{t,fc,Rd(1)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(1)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(1)} = 226{,}08$	226,08	Front plate - tension
$F_{t,wb,Rd(1)} = 237{,}92$	237,92	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta = 200{,}30$	200,30	Web panel - shear
$F_{c,wc,Rd} = 547{,}56$	547,56	Column web - compression
$F_{c,fb,Rd} = 580{,}91$	580,91	Beam flange - compression
$F_{c,wb,Rd} = 439{,}62$	439,62	Beam web - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ **- Formula**	$F_{t2,Rd,comp}$	**Component**
$F_{t2,Rd} = \mathrm{Min}\ (F_{t2,Rd,comp})$	60,34	Bolt row resistance
$F_{t,fc,Rd(2)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(2)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(2)} = 214{,}30$	214,30	Front plate - tension
$F_{t,wb,Rd(2)} = 237{,}92$	237,92	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 200{,}30 - 139{,}96$	60,34	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 547{,}56 - 139{,}96$	407,60	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 580{,}91 - 139{,}96$	440,95	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^1 F_{tj,Rd} = 439{,}62 - 139{,}96$	299,66	Beam web - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 235{,}41 - 139{,}96$	95,45	Column flange - tension - group

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 235{,}24 - 139{,}96$	95,28	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 385{,}02 - 139{,}96$	245,07	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 368{,}54 - 139{,}96$	228,58	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t3,Rd} = \mathrm{Min}\ (F_{t3,Rd,comp})$	0,00	Bolt row resistance
$F_{t,fc,Rd(3)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(3)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(3)} = 214{,}30$	214,30	Front plate - tension
$F_{t,wb,Rd(3)} = 237{,}92$	237,92	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^2 F_{ti,Rd} = 200{,}30 - 200{,}30$	0,00	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 547{,}56 - 200{,}30$	347,26	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 580{,}91 - 200{,}30$	380,61	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 439{,}62 - 200{,}30$	239,32	Beam web - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 228{,}46 - 60{,}34$	168,12	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 232{,}01 - 60{,}34$	171,67	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 366{,}42 - 200{,}30$	166,12	Column flange - tension - group
$F_{t,wc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 274{,}80 - 200{,}30$	74,50	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 377{,}93 - 60{,}34$	317,59	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 349{,}01 - 60{,}34$	288,67	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 584{,}93 - 200{,}30$	384,63	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 573{,}17 - 200{,}30$	372,87	Beam web - tension - group

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	245	139,96	139,96	173,79	226,08	237,92	226,08	280,12
2	170	60,34	139,96	173,79	214,30	237,92	226,08	280,12
3	70	-	139,96	173,79	214,30	237,92	226,08	280,12

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd} =$ 44,63 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1,0$	0,53 < 1,00	verified	(0,53)

Connection resistance for shear

$\alpha_v =$ 0,60 Coefficient for calculation of $F_{v,Rd}$ [Table 3.4]

$F_{v,Rd} =$ 96,51 [kN] Shear resistance of a single bolt [Table 3.4]

$F_{t,Rd,max} =$ 113,04 [kN] Tensile resistance of a single bolt [Table 3.4]

$F_{b,Rd,int} =$ 123,84 [kN] Bearing resistance of an intermediate bolt [Table 3.4]

$F_{b,Rd,ext} =$ 114,67 [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	226,08	-3,29	139,96	73,55	70,26	150,17
2	226,08	-3,29	60,34	31,71	28,42	175,69
3	226,08	-3,29	0,00	0,00	-3,29	193,02

$F_{tj,Rd,N}$ – 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

$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \mathrm{Min}\,(n_h F_{v,Ed} / (1 - F_{tj,Ed} / (1.4\, n_h F_{t,Rd,max})\,),\ n_h F_{v,Rd}\ ,\ n_h F_{b,Rd})$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd} =$ 518,88 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \leq 1,0$	0,04 < 1,00	verified	(0,04)

Weld resistance

$A_w =$	9130	[mm^2]	Area of all welds	[4.5.3.2(2)]
$A_{wy} =$	5440	[mm^2]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} =$	3690	[mm^2]	Area of vertical welds	[4.5.3.2(2)]
$I_{wy} =$	83989863	[mm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]

Weld resistance

$A_w =$	9130	[mm^2]	Area of all welds	[4.5.3.2(2)]
$\sigma_{\perp max}=\tau_{\perp max} =$	-30,98	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_\perp=\tau_\perp =$	-30,98	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
$\tau_{II} =$	4,93	[MPa]	Tangent stress	[4.5.3.2(5)]
$\beta_w =$	0,85		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	61,97 < 404,71	verified	(0,15)
$\sqrt{[\sigma_\perp^2 + 3*(\tau_\perp^2+\tau_{II}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	62,55 < 404,71	verified	(0,15)
$\sigma_\perp \leq 0.9*f_u/\gamma_{M2}$	30,98 < 309,60	verified	(0,10)

Connection stiffness

$t_{wash} =$	4	[mm]	Washer thickness	[6.2.6.3.(2)]
$h_{head} =$	12	[mm]	Bolt head height	[6.2.6.3.(2)]
$h_{nut} =$	16	[mm]	Bolt nut height	[6.2.6.3.(2)]
$L_b =$	46	[mm]	Bolt length	[6.2.6.3.(2)]
$k_{10} =$	5	[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	784	153959
1	245	4	10	34	2	432	105933
2	170	3	7	22	1	233	39630
3	70	4	10	31	2	119	8396

$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$

$z_{eq} =$	196	[mm]	Equivalent force arm	[6.3.3.1.(3)]

$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$

$k_{eq} =$	4	[mm]	Equivalent stiffness coefficient of a bolt arrangement	[6.3.3.1.(1)]
$A_{vc} =$	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
$\beta =$	1,00		Transformation parameter	[5.3.(7)]
$z =$	196	[mm]	Lever arm	[6.2.5]

A_{vc} = 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

k_1 = 3 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

k_2 = ∞ 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)]

$S_{j,ini}$ = 12325,00 [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 = 12325,00 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig}$ = 3427,12 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin}$ = 214,19 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,ini} \geq S_{j,rig}$ RIGID

Weakest component:

COLUMN WEB PANEL - SHEAR

Connection conforms to the code Ratio 0,54

Na temelju sljedećih mjerodavnih vrijednosti dobivenih iz Robot Structural Analysis za glavni okvir radimo provjeru proračuna priključka stup – gredni nosač [7].

$M_{y,Ed} = 23{,}46\ kNm$

$V_{Ed} = 18{,}18\ kN$

$N_{Ed} = 20{,}19\ kN$

Karakteristike vijaka i ploče:

Vijci: M16 – k.v. 10.9.

Granica popuštanja: $f_{yb} = 900\ N/mm^2$

Vlačna čvrstoća: $f_{ub} = 1000\ N/mm^2$

Promjer jezgre vijka: $d = 16\ mm$

Promjer rupe za vijak: $d_0 = 18\ mm$

Vlačna površina poprečnog presjeka vijka: $A_s = 157\ mm^2$

Čelična ploča: $S\ 275 \quad t = 15mm$

Vlačna čvrstoća: $f_{ub} = 430\ N/mm^2$

<u>Otpornost vijaka na vlak:</u>

$N_{Rd,u,1} = n * F_{t,Rd,u}$

n – ukupni broj vijaka

$$F_{t,Rd} = \frac{k_2 * f_{ub} * A_s}{\gamma_{M2}} = \frac{0{,}9 * 1000 * 157}{1{,}25} = 113{,}04\ kN$$

$N_{Rd,u,1} = 6 * 113{,}04 = 678{,}24\ kN$

Možemo pretpostaviti da prvi red vijaka preuzima maksimalno opterećenje, dok ostali redovi preuzimaju opterećenje prema raspodjeli.

Red vijaka	h_j	$F_{tj,Rd}$
1	245	139,96
2	170	60,34
3	70	-

$h_j -$ vrijednost udaljenosti određenog vijka od tlačnog ruba

$M_{t,Rd} = \sum h_j * F_{tj,Rd} = 44{,}55\ kNm$

$M_{y,Ed} \leq M_{t,Rd};\ 23{,}46 < 44{,}55$ **ZADOVOLJAVA**(53%)

Otpornost vijaka na posmik:

Otpornost jednog vijka na posmik

$$F_{v,Rd} = \frac{\alpha_v * f_{ub} * A_s}{\gamma_{M2}}$$

$\alpha_v = 0{,}5$

$$F_{v,Rd} = \frac{0{,}5 * 1000 * 157}{1{,}25} = 62{,}8\ kN (robot\ je\ druge\ parametre\ odabrao..)$$

$F_{v,Rd} = 62{,}8 * 6 = 376{,}8\ kN$

$F_{v,Ed} \leq F_{v,Rd}; 18{,}18 < 376{,}8$ **ZADOVOLJAVA**(5%)

Otpornost na pritisak po omotaču rupe osnovnog materijala:

$$e_1 = \binom{50}{180} mm \begin{matrix} od\ gornjeg\ ruba \\ od\ doneg\ ruba \end{matrix}$$

$$e_2 = \binom{45}{45} mm \begin{matrix} od\ lijevog\ ruba \\ od\ desnog\ ruba \end{matrix}$$

$p_1 = 75\ mm$

$p_2 = 100\ mm$

Za krajnje vijke

$$\alpha_d = \frac{e_1}{3 * d_0} = \frac{50}{3 * 18} = 0{,}93$$

Za unutarnje vijke

$$\alpha_d = \frac{p_1}{3 * d_0} - \frac{1}{4} = \frac{75}{3 * 18} - \frac{1}{4} = 1{,}14$$

$$\alpha_b = \min\left(\alpha_b; \frac{f_{ub}}{f_u}; 1\right) = \min\left(0{,}93; \frac{1000}{430}; 1\right) = 0{,}93$$

Za krajnje vijke

$$k_1 = \min\left(2{,}8 * \frac{e_2}{d_0} - 1{,}7 ; 2{,}5\right) = min(5{,}3 ; 2{,}5) = 2{,}5$$

Za unutarnje vijke:

$$k_1 = \min\left(1{,}4 * \frac{p_2}{d_0} - 1{,}7\,;2{,}5\right) = min(6{,}07\,;2{,}5) = 2{,}5$$

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_{ub} * d * t}{\gamma_{M2}} = \frac{2{,}5 * 0{,}93 * 1000 * 16 * 15}{1{,}25} = 446{,}4$$

$F_{v,Ed} \leq F_{b,Rd}$; $18{,}18 < 191{,}95$ **ZADOVOLJAVA**(4%)

<u>Interakcija posmika i vlaka:</u>

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{M_{y,Ed}}{1{,}40 * M_{t,Rd}} \leq 1\,;\ \frac{18{,}18}{376{,}8} + \frac{23{,}46}{1{,}4 * 44{,}55} \leq 1\ ;0{,}42\ < 1$$

ZADOVOLJAVA(42%)

<u>Proračun zavara:</u>

$$F_{t,Ed} = \frac{23{,}46}{0{,}85} = 27{,}6\,kN$$

Djelovanje na zavar:

$$F_{w,Ed} = \sqrt{(0{,}5 * V_{z,Ed})^2 + F_{t,Ed}^2} = \sqrt{9{,}09^2 + 27{,}6^2} = 29{,}06\,kN$$

Duljina zavara:

$$L = 2 * (2b - tw - 2r) = 2 * (2 * 160 - 6 - 2 * 15) = 568\,mm$$

Debljina zavara iznosi 8 mm.

$$F_{w,Rd} = \frac{\frac{f_u}{\sqrt{3} * \beta_w} * a * L}{\gamma_{M2}} = \frac{\frac{43{,}0}{\sqrt{3} * 0{,}85} * 0{,}8 * 56{,}8}{1{,}25} = 1061{,}74\,kN$$

$F_{w,Ed} \leq F_{w,Rd}$; $29{,}06\ < 1061{,}74$ **ZADOVOLJAVA**(1%)

9.2. Stup i gredni nosač (DETALJ B)

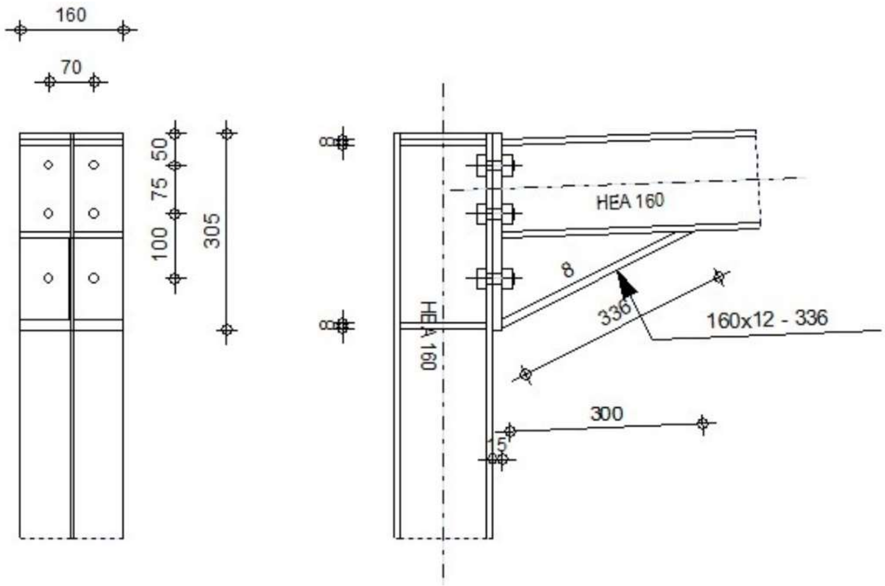
Robot Structural Analysis Professional 2025

Design of fixed beam-to-column connection

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

OK

Ratio
0,52



General

Connection no.: 7

Connection name: Frame knee

Structure node: 49

Structure members: 34, 35

Geometry

Column

Section: HEA 160

Member no.: 34

α =	-90,0	[Deg]	Inclination angle
h_c =	152	[mm]	Height of column section
b_{fc} =	160	[mm]	Width of column section
t_{wc} =	6	[mm]	Thickness of the web of column section

Section: HEA 160

t_{fc} = 9 [mm] Thickness of the flange of column section

r_c = 15 [mm] Radius of column section fillet

A_c = 3877 [mm^2] Cross-sectional area of a column

I_{xc} = 16729800 [mm^4] Moment of inertia of the column section

Material: S275

f_{yc} = 275,00 [MPa] Resistance

Beam

Section: HEA 160

Member no.: 35

α = 2,2 [Deg] Inclination angle

h_b = 152 [mm] Height of beam section

b_f = 160 [mm] Width of beam section

t_{wb} = 6 [mm] Thickness of the web of beam section

t_{fb} = 9 [mm] Thickness of the flange of beam section

r_b = 15 [mm] Radius of beam section fillet

r_b = 15 [mm] Radius of beam section fillet

A_b = 3877 [mm^2] Cross-sectional area of a beam

I_{xb} = 16729800 [mm^4] Moment of inertia of the beam section

Material: S275

f_{yb} = 275,00 [MPa] Resistance

Bolts

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

d = 16 [mm] Bolt diameter

Class = 10.9 Bolt class

F_{tRd} = 113,04 [kN] Tensile resistance of a bolt

n_h = 2 Number of bolt columns

n_v = 3 Number of bolt rows

h_1 = 50 [mm] Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 70 [mm]

Vertical spacing p_i = 75;100 [mm]

Plate

h_p = 305 [mm] Plate height

b_p = 160 [mm] Plate width

t_p = 15 [mm] Plate thickness

Material: S275

f_{yp} = 275,00 [MPa] Resistance

Lower stiffener

w_d = 160 [mm] Plate width

t_{fd} = 12 [mm] Flange thickness

h_d = 140 [mm] Plate height

t_{wd} = 8 [mm] Web thickness

l_d = 300 [mm] Plate length

α = 26,8 [Deg] Inclination angle

Material: STEEL 43-245

f_{ybu} = 245,00 [MPa] Resistance

Column stiffener

Upper

h_{su} = 134 [mm] Stiffener height

b_{su} = 77 [mm] Stiffener width

t_{hu} = 8 [mm] Stiffener thickness

Material: S275

f_{ysu} = 275,00 [MPa] Resistance

Lower

h_{sd} = 134 [mm] Stiffener height

b_{sd} = 77 [mm] Stiffener width

t_{hd} = 8 [mm] Stiffener thickness

Material: S275

f_{ysu} = 275,00 [MPa] Resistance

Fillet welds

a_w = 8 [mm] Web weld

a_f = 8 [mm] Flange weld

a_s = 8 [mm] Stiffener weld

a_{fd} = 5 [mm] Horizontal weld

Material factors

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

Loads

Ultimate limit state

Case: 9: COMB1 (1+2)*1.35+4*1.50

$M_{b1,Ed} =$	23,02	[kN*m]	Bending moment in the right beam
$V_{b1,Ed} =$	17,71	[kN]	Shear force in the right beam
$N_{b1,Ed} =$	-11,84	[kN]	Axial force in the right beam
$M_{c1,Ed} =$	23,03	[kN*m]	Bending moment in the lower column
$V_{c1,Ed} =$	11,94	[kN]	Shear force in the lower column
$N_{c1,Ed} =$	-23,57	[kN]	Axial force in the lower column

Results

Beam resistances

COMPRESSION

$A_b =$	3877	[mm^2]	Area	EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b\ f_{yb} / \gamma_{M0}$

$N_{cb,Rd} =$	1066,21	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

SHEAR

$A_{vb} =$	2441	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb}\ (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd} =$	387,58	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0,05 < 1,00	verified	(0,05)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

$W_{plb} =$	245167	[mm^3]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb}\ f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd} =$	67,42	[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

$W_{pl} =$	593341	[mm^3]	Plastic section modulus	EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl}\ f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ = 163,17 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 163,17 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

h_f = 281 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ = 580,91 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

β = 2,2 [Deg] Angle between the front plate and the beam

γ = 26,8 [Deg] Inclination angle of the bracket plate

$b_{eff,c,wb}$ = 171 [mm] Effective width of the web for compression [6.2.6.2.(1)]

A_{vb} = 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

ω = 0,75 Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed}$ = 74,60 [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)]

$F_{c,wb,Rd1} = [\omega\ k_{wc}\ b_{eff,c,wb}\ t_{wb}\ f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd1}$ = 453,30 [kN] Beam web resistance [6.2.6.2.(1)]

Buckling:

d_{wb} = 104 [mm] Height of compressed web [6.2.6.2.(1)]

λ_p = 0,76 Plate slenderness of an element [6.2.6.2.(1)]

ρ = 0,97 Reduction factor for element buckling [6.2.6.2.(1)]

$F_{c,wb,Rd2} = [\omega\ k_{wc}\ \rho\ b_{eff,c,wb}\ t_{wb}\ f_{yb} / \gamma_{M1}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd2}$ = 439,62 [kN] Beam web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wb,Rd,low} = \text{Min}(F_{c,wb,Rd1}, F_{c,wb,Rd2})$

$F_{c,wb,Rd,low}$ = 439,62 [kN] Beam web resistance [6.2.6.2.(1)]

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed}$ = 23,02 [kN*m] Bending moment (right beam) [5.3.(3)]

$M_{b2,Ed}$ = 0,00 [kN*m] Bending moment (left beam) [5.3.(3)]

$V_{c1,Ed}$ = 11,94 [kN] Shear force (lower column) [5.3.(3)]

$V_{c2,Ed}$ = 0,00 [kN] Shear force (upper column) [5.3.(3)]

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	23,02	[kN*m]	Bending moment (right beam)	[5.3.(3)]
z =	208	[mm]	Lever arm	[6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ =	104,76	[kN]	Shear force acting on the web panel	[5.3.(3)]
A_{vs} =	1321	[mm^2]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
d_s =	277	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	0,89	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	0,70	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	0,70	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9 (A_{vs}*f_{y,wc}) / (\sqrt{3}\ \gamma_{M0}) + Min(4 M_{pl,fc,Rd} / d_s , (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ =	200,30	[kN]	Resistance of the column web panel for shear	[6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \le 1,0$ 0,52 < 1,00 verified (0,52)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	6	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	186	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0,72		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	77,66	[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)]
A_s =	1232	[mm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M0} + A_s\ f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ =	559,89	[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 =	0,79		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	0,94		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2,21		Stiffener slenderness	EN1993-1-1:[6.3.1.2]

Buckling:

$d_{wc} =$ 104 [mm] Height of compressed web [6.2.6.2.(1)]

$\chi_s =$ 1,00 Buckling coefficient of the stiffener EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M1} + A_s\, \chi_s\, f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2} =$ 547,56 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \mathrm{Min}\,(F_{c,wc,Rd1}\, ,\, F_{c,wc,Rd2})$

$F_{c,wc,Rd} =$ 547,56 [kN] Column web resistance [6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

$t_{wc} =$ 6 [mm] Effective thickness of the column web [6.2.6.2.(6)]

$b_{eff,c,wc} =$ 182 [mm] Effective width of the web for compression [6.2.6.2.(1)]

$A_{vc} =$ 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega =$ 0,73 Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed} =$ 77,66 [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)]

$A_s =$ 1232 [mm^2] Area of the web stiffener EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M0} + A_s\, f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1} =$ 557,11 [kN] Column web resistance [6.2.6.2.(1)]

Buckling:

$d_{wc} =$ 104 [mm] Height of compressed web [6.2.6.2.(1)]

$\lambda_p =$ 0,78 Plate slenderness of an element [6.2.6.2.(1)]

$\rho =$ 0,95 Reduction factor for element buckling [6.2.6.2.(1)]

$\lambda_s =$ 2,21 Stiffener slenderness EN1993-1-1:[6.3.1.2]

$\chi_s =$ 1,00 Buckling coefficient of the stiffener EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M1} + A_s\, \chi_s\, f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2} =$ 546,58 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,upp} = \mathrm{Min}\,(F_{c,wc,Rd1}\, ,\, F_{c,wc,Rd2})$

$F_{c,wc,Rd,upp} =$ 546,58 [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	20	-	45	-	75	126	154	126	154	138	124	124	124
2	20	-	45	-	88	126	136	126	136	175	88	88	88
3	20	-	45	-	100	126	136	126	136	163	118	118	118

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	-	45	-	75	144	172	144	172	147	136	136	136
2	23	-	45	-	88	144	148	144	148	175	88	88	88
3	23	-	45	-	100	144	148	144	148	172	124	124	124

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 row in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt row in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt row for mode 1

$l_{eff,2}$ – Effective length for a single bolt row 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 compression

$N_{j,Rd}$ = Min ($N_{cb,Rd}$2 $F_{c,wb,Rd,low}$, 2 $F_{c,wc,Rd,low}$, 2 $F_{c,wc,Rd,upp}$)

$N_{j,Rd}$ = 879,23 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1,0$ 0,01 < 1,00 verified (0,01)

Connection resistance for bending

$F_{t,Rd}$ = 113,04 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 140,06 [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

$F_{t,fc,Rd}$ – column flange resistance 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
$F_{t1,Rd} = \mathrm{Min}\,(F_{t1,Rd,comp})$	139,96	Bolt row resistance
$F_{t,fc,Rd(1)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(1)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(1)} = 226{,}08$	226,08	Front plate - tension
$F_{t,wb,Rd(1)} = 237{,}92$	237,92	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta = 200{,}30$	200,30	Web panel - shear
$F_{c,wc,Rd} = 547{,}56$	547,56	Column web - compression
$F_{c,fb,Rd} = 580{,}91$	580,91	Beam flange - compression
$F_{c,wb,Rd} = 439{,}62$	439,62	Beam web - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd} = \mathrm{Min}\,(F_{t2,Rd,comp})$	60,34	Bolt row resistance
$F_{t,fc,Rd(2)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(2)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(2)} = 214{,}30$	214,30	Front plate - tension
$F_{t,wb,Rd(2)} = 237{,}92$	237,92	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 200{,}30 - 139{,}96$	60,34	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 547{,}56 - 139{,}96$	407,60	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 580{,}91 - 139{,}96$	440,95	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^1 F_{tj,Rd} = 439{,}62 - 139{,}96$	299,66	Beam web - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 235{,}41 - 139{,}96$	95,45	Column flange - tension - group

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 235{,}24 - 139{,}96$	95,28	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 385{,}02 - 139{,}96$	245,07	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 368{,}54 - 139{,}96$	228,58	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t3,Rd} = \mathrm{Min}\ (F_{t3,Rd,comp})$	0,00	Bolt row resistance
$F_{t,fc,Rd(3)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(3)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(3)} = 214{,}30$	214,30	Front plate - tension
$F_{t,wb,Rd(3)} = 237{,}92$	237,92	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^2 F_{ti,Rd} = 200{,}30 - 200{,}30$	0,00	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 547{,}56 - 200{,}30$	347,26	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 580{,}91 - 200{,}30$	380,61	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 439{,}62 - 200{,}30$	239,32	Beam web - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 228{,}46 - 60{,}34$	168,12	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 232{,}01 - 60{,}34$	171,67	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 366{,}42 - 200{,}30$	166,12	Column flange - tension - group
$F_{t,wc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 274{,}80 - 200{,}30$	74,50	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 377{,}93 - 60{,}34$	317,59	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 349{,}01 - 60{,}34$	288,67	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 584{,}93 - 200{,}30$	384,63	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 573{,}17 - 200{,}30$	372,87	Beam web - tension - group

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	245	139,96	139,96	173,79	226,08	237,92	226,08	280,12
2	170	60,34	139,96	173,79	214,30	237,92	226,08	280,12
3	70	-	139,96	173,79	214,30	237,92	226,08	280,12

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd}$ = 44,63 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1{,}0$	0,52 < 1,00	verified	(0,52)

Connection resistance for shear

α_v =	0,60		Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
$F_{v,Rd}$ =	96,51	[kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max}$ =	113,04	[kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int}$ =	123,84	[kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext}$ =	114,67	[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	226,08	-3,95	139,96	72,19	68,25	151,40
2	226,08	-3,95	60,34	31,13	27,18	176,45
3	226,08	-3,95	0,00	0,00	-3,95	193,02

$F_{tj,Rd,N}$ – 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

$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \mathrm{Min}\, (n_h F_{v,Ed} / (1 - F_{tj,Ed} / (1.4\, n_h F_{t,Rd,max})\,),\ n_h F_{v,Rd}\ ,\ n_h F_{b,Rd})$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd}$ = 520,87 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \leq 1{,}0$	0,03 < 1,00	verified	(0,03)

Weld resistance

A_w =	9130	[mm^2]	Area of all welds	[4.5.3.2(2)]
A_{wy} =	5440	[mm^2]	Area of horizontal welds	[4.5.3.2(2)]
A_{wz} =	3690	[mm^2]	Area of vertical welds	[4.5.3.2(2)]
I_{wy} =	83989863	[mm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]

Weld resistance

$A_w =$	9130	[mm^2]	Area of all welds	[4.5.3.2(2)]
$\sigma_{\perp max}=\tau_{\perp max} =$	-30,72	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_{\perp}=\tau_{\perp} =$	-30,72	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
$\tau_{II} =$	4,80	[MPa]	Tangent stress	[4.5.3.2(5)]
$\beta_w =$	0,85		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	61,44 < 404,71	verified	(0,15)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	62,00 < 404,71	verified	(0,15)
$\sigma_{\perp} \leq 0.9*f_u/\gamma_{M2}$	30,72 < 309,60	verified	(0,10)

Connection stiffness

$t_{wash} =$	4	[mm]	Washer thickness	[6.2.6.3.(2)]
$h_{head} =$	12	[mm]	Bolt head height	[6.2.6.3.(2)]
$h_{nut} =$	16	[mm]	Bolt nut height	[6.2.6.3.(2)]
$L_b =$	46	[mm]	Bolt length	[6.2.6.3.(2)]
$k_{10} =$	5	[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	784	153959
1	245	4	10	34	2	432	105933
2	170	3	7	22	1	233	39630
3	70	4	10	31	2	119	8396

$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$

$z_{eq} =$	196	[mm]	Equivalent force arm	[6.3.3.1.(3)]

$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$

$k_{eq} =$	4	[mm]	Equivalent stiffness coefficient of a bolt arrangement	[6.3.3.1.(1)]
$A_{vc} =$	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
$\beta =$	1,00		Transformation parameter	[5.3.(7)]
$z =$	196	[mm]	Lever arm	[6.2.5]

$A_{vc} =$ 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$k_1 =$ 3 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

$k_2 = \infty$ 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)]

$S_{j,ini} =$ 12325,00 [kN*m] Initial rotational stiffness [6.3.1.(4)]

$\mu =$ 1,00 Stiffness coefficient of a connection [6.3.1.(6)]

$S_j = S_{j,ini} / \mu$ [6.3.1.(4)]

$S_j =$ 12325,00 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig} =$ 3427,12 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin} =$ 214,19 [kN*m] Stiffness of a pinned connection [5.2.2.5]

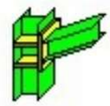
$S_{j,ini} \geq S_{j,rig}$ RIGID

Weakest component:

COLUMN WEB PANEL - SHEAR

Connection conforms to the code Ratio 0,52

9.3. Stup i gredni nosač galerije (DETALJ C)



Robot Structural Analysis Professional 2025

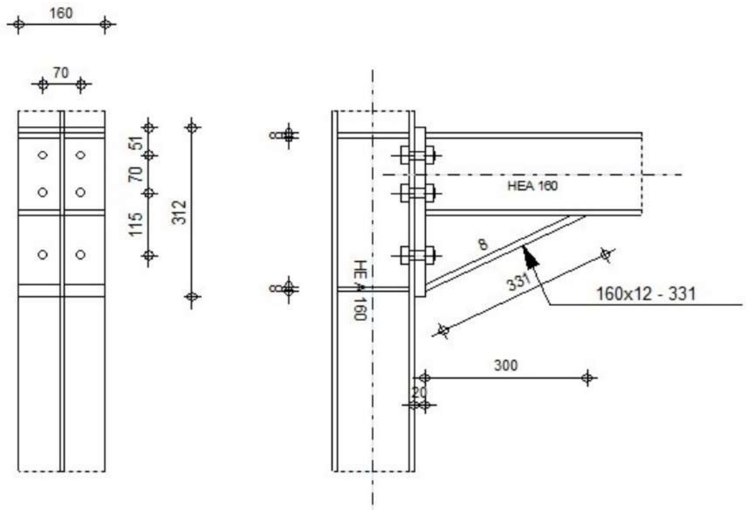
Design of fixed beam-to-column connection

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

OK

Ratio

0,72



General

Connection no.: 10

Connection name: Column-Beam

Structure node: 296

Structure members: 34, 209

Geometry

Column

Section: HEA 160

Member no.: 34

$\alpha =$	-90,0	[Deg]	Inclination angle
$h_c =$	152	[mm]	Height of column section
$b_{fc} =$	160	[mm]	Width of column section
$t_{wc} =$	6	[mm]	Thickness of the web of column section
$t_{fc} =$	9	[mm]	Thickness of the flange of column section
$r_c =$	15	[mm]	Radius of column section fillet

Section: HEA 160

A_c = 3877 [mm^2] Cross-sectional area of a column

I_{xc} = 16729800 [mm^4] Moment of inertia of the column section

Material: S275

f_{yc} = 275,00 [MPa] Resistance

Beam

Section: HEA 160

Member no.: 209

α =	0,0	[Deg]	Inclination angle
h_b =	152	[mm]	Height of beam section
b_f =	160	[mm]	Width of beam section
t_{wb} =	6	[mm]	Thickness of the web of beam section
t_{fb} =	9	[mm]	Thickness of the flange of beam section
r_b =	15	[mm]	Radius of beam section fillet
r_b =	15	[mm]	Radius of beam section fillet
A_b =	3877	[mm^2]	Cross-sectional area of a beam
I_{xb} =	16729800	[mm^4]	Moment of inertia of the beam section

Material: S275

f_{yb} = 275,00 [MPa] Resistance

Bolts

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

d =	16	[mm]	Bolt diameter
Class =	10.9		Bolt class
F_{tRd} =	113,04	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	3		Number of bolt rows
h_1 =	51	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 70 [mm]

Vertical spacing p_i = 70;115 [mm]

Plate

h_p =	312	[mm]	Plate height
b_p =	160	[mm]	Plate width
t_p =	20	[mm]	Plate thickness

Material: STEEL 43-245

f_{yp} = 245,00 [MPa] Resistance

Lower stiffener

w_d =	160	[mm]	Plate width
t_{fd} =	12	[mm]	Flange thickness
h_d =	140	[mm]	Plate height
t_{wd} =	8	[mm]	Web thickness
l_d =	300	[mm]	Plate length
α =	25,0	[Deg]	Inclination angle

Material: S275

f_{ybu} = 275,00 [MPa] Resistance

Column stiffener

Upper

h_{su} =	134	[mm]	Stiffener height
b_{su} =	77	[mm]	Stiffener width
t_{hu} =	8	[mm]	Stiffener thickness

Material: S275

f_{ysu} = 275,00 [MPa] Resistance

Lower

h_{sd} =	134	[mm]	Stiffener height
b_{sd} =	77	[mm]	Stiffener width
t_{hd} =	8	[mm]	Stiffener thickness

Material: S275

f_{ysu} = 275,00 [MPa] Resistance

Fillet welds

a_w =	5	[mm]	Web weld
a_f =	7	[mm]	Flange weld
a_s =	5	[mm]	Stiffener weld
a_{fd} =	5	[mm]	Horizontal 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]

Material factors

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

Loads

Ultimate limit state

Case: 25: COMB28 3*1.50+2*1.35

$M_{b1,Ed}$ =	31,88	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	30,48	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	5,04	[kN]	Axial force in the right beam
$M_{c1,Ed}$ =	13,84	[kN*m]	Bending moment in the lower column
$V_{c1,Ed}$ =	5,17	[kN]	Shear force in the lower column
$N_{c1,Ed}$ =	-45,62	[kN]	Axial force in the lower column
$M_{c2,Ed}$ =	-18,04	[kN*m]	Bending moment in the upper column
$V_{c2,Ed}$ =	-10,54	[kN]	Shear force in the upper column
$N_{c2,Ed}$ =	-8,13	[kN]	Axial force in the upper column

Results

Beam resistances

TENSION

A_b =	3877	[mm^2]	Area	EN1993-1-1:[6.2.3]

$N_{tb,Rd} = A_b\ f_{yb} / \gamma_{M0}$

$N_{tb,Rd}$ =	1066,21	[kN]	Design tensile resistance of the section	EN1993-1-1:[6.2.3]

SHEAR

A_{vb} =	2441	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb}\ (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ =	387,58	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0,08 < 1,00	verified	(0,08)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} =	245167	[mm^3]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb}\ f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ =	67,42	[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

$W_{pl} =$ 590469 [mm^3] Plastic section modulus EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl}\, f_{yb} / \gamma_{M0}$

$M_{cb,Rd} =$ 162,38 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd} =$ 162,38 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

$h_f =$ 281 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd} =$ 578,11 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

$\beta =$ 0,0 [Deg] Angle between the front plate and the beam

$\gamma =$ 25,0 [Deg] Inclination angle of the bracket plate

$b_{eff,c,wb} =$ 168 [mm] Effective width of the web for compression [6.2.6.2.(1)]

$A_{vb} =$ 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega =$ 0,75 Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed} =$ 97,80 [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)]

$F_{c,wb,Rd1} = [\omega\, k_{wc}\, b_{eff,c,wb}\, t_{wb}\, f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd1} =$ 448,42 [kN] Beam web resistance [6.2.6.2.(1)]

Buckling:

$d_{wb} =$ 104 [mm] Height of compressed web [6.2.6.2.(1)]

$\lambda_p =$ 0,75 Plate slenderness of an element [6.2.6.2.(1)]

$\rho =$ 0,98 Reduction factor for element buckling [6.2.6.2.(1)]

$F_{c,wb,Rd2} = [\omega\, k_{wc}\, \rho\, b_{eff,c,wb}\, t_{wb}\, f_{yb} / \gamma_{M1}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd2} =$ 437,56 [kN] Beam web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wb,Rd,low} = \mathrm{Min}\,(F_{c,wb,Rd1}\, , F_{c,wb,Rd2})$

$F_{c,wb,Rd,low} =$ 437,56 [kN] Beam web resistance [6.2.6.2.(1)]

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed} =$ 31,88 [kN*m] Bending moment (right beam) [5.3.(3)]

$M_{b2,Ed} =$ 0,00 [kN*m] Bending moment (left beam) [5.3.(3)]

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	31,88	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$V_{c1,Ed}$ =	5,17	[kN]	Shear force (lower column)	[5.3.(3)]
$V_{c2,Ed}$ =	-10,54	[kN]	Shear force (upper column)	[5.3.(3)]
z =	209	[mm]	Lever arm	[6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ =	144,42	[kN]	Shear force acting on the web panel	[5.3.(3)]
A_{vs} =	1321	[mm^2]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
d_s =	284	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	0,89	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	0,70	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	0,70	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9 (A_{vs}*f_{y,wc}) / (\sqrt{3}\ \gamma_{M0}) + Min(4 M_{pl,fc,Rd} / d_s , (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ =	200,02	[kN]	Resistance of the column web panel for shear	[6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \le 1,0$	0,72 < 1,00	verified	(0,72)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	6	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	193	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	1321	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0,71		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	58,17	[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)]
A_s =	1232	[mm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M0} + A_s\ f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ =	564,07	[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 =	0,81		Plate slenderness of an element	[6.2.6.2.(1)]

Buckling:

d_{wc} =	104	[mm]	Height of compressed web	[6.2.6.2.(1)]
ρ =	0,93		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2,21		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
χ_s =	1,00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M1} + A_s\, \chi_s\, f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 548,92 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \text{Min}\,(F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd}$ = 548,92 [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	20	-	45	-	70	126	151	126	151	133	117	117	117
2	20	-	45	-	92	126	136	126	136	185	92	92	92
3	20	-	45	-	115	126	136	126	136	178	126	126	126

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	26	-	45	-	70	166	188	166	188	153	142	142	142
2	26	-	45	-	92	166	162	162	162	185	92	92	92
3	26	-	45	-	115	166	162	162	162	198	138	138	138

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 row in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt row in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt row for mode 1

$l_{eff,2}$ – Effective length for a single bolt row 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

m – Bolt distance from the web

$l_{eff,1,g}$ – 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

$F_{t,Rd}$ = 113,04 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 140,06 [kN] Punching shear resistance of a bolt [Table 3.4]

$N_{j,Rd} = \mathrm{Min}\ (N_{tb,Rd}\ ,\ n_v\ n_h\ F_{t,Rd}\ ,\ n_v\ n_h\ B_{p,Rd})$

$N_{j,Rd}$ = 678,24 [kN] Connection resistance for tension [6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1{,}0$ 0,01 < 1,00 verified (0,01)

Connection resistance for bending

$F_{t,Rd}$ = 113,04 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 140,06 [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**
$F_{t1,Rd} = \mathrm{Min}\ (F_{t1,Rd,comp})$	139,96	Bolt row resistance
$F_{t,fc,Rd(1)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(1)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(1)} = 226{,}08$	226,08	Front plate - tension
$F_{t,wb,Rd(1)} = 273{,}11$	273,11	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta = 200{,}02$	200,02	Web panel - shear
$F_{c,wc,Rd} = 548{,}92$	548,92	Column web - compression
$F_{c,fb,Rd} = 578{,}11$	578,11	Beam flange - compression
$F_{c,wb,Rd} = 437{,}56$	437,56	Beam web - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd} = \text{Min}(F_{t2,Rd,comp})$	60,06	Bolt row resistance
$F_{t,fc,Rd(2)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(2)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(2)} = 226{,}08$	226,08	Front plate - tension
$F_{t,wb,Rd(2)} = 266{,}68$	266,68	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 200{,}02 - 139{,}96$	60,06	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 548{,}92 - 139{,}96$	408,96	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 578{,}11 - 139{,}96$	438,15	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^1 F_{tj,Rd} = 437{,}56 - 139{,}96$	297,60	Beam web - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 233{,}87 - 139{,}96$	93,92	Column flange - tension - group
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 234{,}54 - 139{,}96$	94,58	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 445{,}09 - 139{,}96$	305,14	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 386{,}99 - 139{,}96$	247,04	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t3,Rd} = \text{Min}(F_{t3,Rd,comp})$	0,00	Bolt row resistance
$F_{t,fc,Rd(3)} = 139{,}96$	139,96	Column flange - tension
$F_{t,wc,Rd(3)} = 173{,}79$	173,79	Column web - tension
$F_{t,ep,Rd(3)} = 226{,}08$	226,08	Front plate - tension
$F_{t,wb,Rd(3)} = 266{,}68$	266,68	Beam web - tension
$B_{p,Rd} = 280{,}12$	280,12	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^2 F_{ti,Rd} = 200{,}02 - 200{,}02$	0,00	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 548{,}92 - 200{,}02$	348,91	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 578{,}11 - 200{,}02$	378,09	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 437{,}56 - 200{,}02$	237,55	Beam web - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 243{,}10 - 60{,}06$	183,05	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 238{,}65 - 60{,}06$	178,59	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 373{,}95 - 200{,}02$	173,94	Column flange - tension - group

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t,wc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 276{,}22 - 200{,}02$	76,20	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 442{,}01 - 60{,}06$	381,95	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 380{,}84 - 60{,}06$	320,78	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 678{,}24 - 200{,}02$	478,22	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 615{,}21 - 200{,}02$	415,19	Beam web - tension - group

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	244	139,96	139,96	173,79	226,08	273,11	226,08	280,12
2	174	60,06	139,96	173,79	226,08	266,68	226,08	280,12
3	59	-	139,96	173,79	226,08	266,68	226,08	280,12

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd} =$ 44,68 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \le 1{,}0$ 0,71 < 1,00 verified (0,71)

Connection resistance for shear

$\alpha_v =$ 0,60 Coefficient for calculation of $F_{v,Rd}$ [Table 3.4]

$F_{v,Rd} =$ 96,51 [kN] Shear resistance of a single bolt [Table 3.4]

$F_{t,Rd,max} =$ 113,04 [kN] Tensile resistance of a single bolt [Table 3.4]

$F_{b,Rd,int} =$ 123,84 [kN] Bearing resistance of an intermediate bolt [Table 3.4]

$F_{b,Rd,ext} =$ 123,84 [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	226,08	1,68	139,96	99,88	101,56	131,08
2	226,08	1,68	60,06	42,86	44,54	165,86
3	226,08	1,68	0,00	0,00	1,68	191,99

$F_{tj,Rd,N}$ – 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]

$V_{j,Rd} =$	488,94	[kN]	Connection resistance for shear	[Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \le 1,0$	0,06 < 1,00	verified	(0,06)

Weld resistance

$A_w =$	8272	[mm²]	Area of all welds	[4.5.3.2(2)]
$A_{wy} =$	5964	[mm²]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} =$	2308	[mm²]	Area of vertical welds	[4.5.3.2(2)]
$I_{wy} =$	96003973	[mm⁴]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max} =$	36,01	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_{\perp}=\tau_{\perp} =$	29,63	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
$\tau_{II} =$	13,21	[MPa]	Tangent stress	[4.5.3.2(5)]
$\beta_w =$	0,85		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	72,03 < 404,71	verified	(0,18)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	63,53 < 404,71	verified	(0,16)
$\sigma_{\perp} \le 0.9*f_u/\gamma_{M2}$	36,01 < 309,60	verified	(0,12)

Connection stiffness

$t_{wash} =$	4	[mm]	Washer thickness	[6.2.6.3.(2)]
$h_{head} =$	12	[mm]	Bolt head height	[6.2.6.3.(2)]
$h_{nut} =$	16	[mm]	Bolt nut height	[6.2.6.3.(2)]
$L_b =$	51	[mm]	Bolt length	[6.2.6.3.(2)]
$k_{10} =$	5	[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	760	149320

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}$ hj	$k_{eff,j}$ h_j^2
1	244	4	10	56	2	410	100162
2	174	3	8	36	1	247	42997
3	59	4	10	54	2	104	6161

$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$

$z_{eq} =$ 196 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$

$k_{eq} =$ 4 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

$A_{vc} =$ 1321 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$\beta =$ 1,00 Transformation parameter [5.3.(7)]

$z =$ 196 [mm] Lever arm [6.2.5]

$k_1 =$ 3 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

$k_2 =$ ∞ 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)]

$S_{j,ini} =$ 12174,96 [kN*m] Initial rotational stiffness [6.3.1.(4)]

$\mu =$ 1,20 Stiffness coefficient of a connection [6.3.1.(6)]

$S_j = S_{j,ini} / \mu$ [6.3.1.(4)]

$S_j =$ 10129,94 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig} =$ 3429,61 [kN*m] Stiffness of a rigid connection [5.2.2.5]

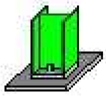
$S_{j,pin} =$ 214,35 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,ini} \geq S_{j,rig}$ RIGID

Weakest component:

Connection conforms to the code Ratio 0,72

9.4. Stup i temelj (DETALJ D)



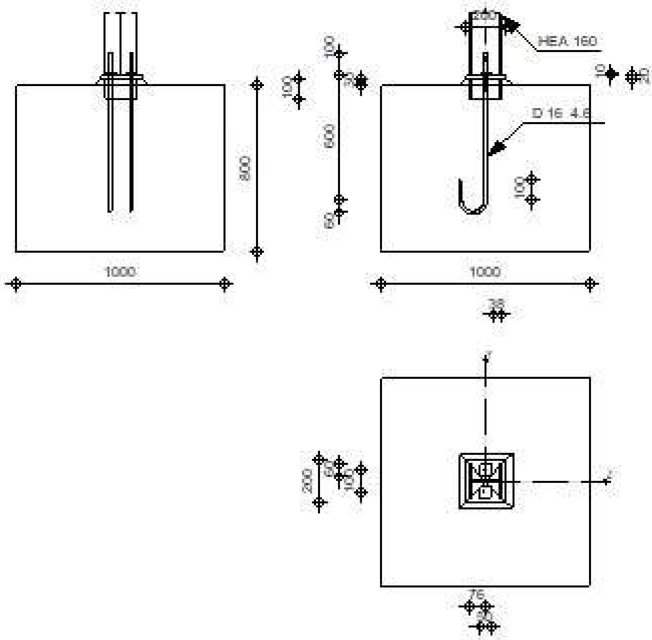
Robot Structural Analysis Professional 2025

Pinned column base design

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

OK

Ratio
0,81



General

Connection no.: 9

Connection name: Pinned column base

Geometry

Column

Section: HEA 160

L_c =	6,00	[m]	Column length
a =	0,0	[Deg]	Inclination angle
h_c =	152	[mm]	Height of column section
b_{fc} =	160	[mm]	Width of column section
t_{wc} =	6	[mm]	Thickness of the web of column section
t_{fc} =	9	[mm]	Thickness of the flange of column section
r_c =	15	[mm]	Radius of column section fillet
A_c =	3880	[mm^2]	Cross-sectional area of a column

L_c = 6,00 [m] Column length

I_{yc} = 16730000 [mm^4] Moment of inertia of the column section

Material: S275

f_{yc} = 275,00 [MPa] Resistance

f_{uc} = 430,00 [MPa] Yield strength of a material

Column base

l_{pd} = 200 [mm] Length

b_{pd} = 200 [mm] Width

t_{pd} = 20 [mm] Thickness

Material: S275

f_{ypd} = 275,00 [MPa] Resistance

f_{upd} = 430,00 [MPa] Yield strength of a material

Anchorage

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

Class = 4.6 Anchor class

f_{yb} = 240,00 [MPa] Yield strength of the anchor material

f_{ub} = 400,00 [MPa] Tensile strength of the anchor material

d = 16 [mm] Bolt diameter

A_s = 157 [mm^2] Effective section area of a bolt

A_v = 201 [mm^2] Area of bolt section

n = 2 Number of bolt rows

ev = 100 [mm] Vertical spacing

Anchor dimensions

L_1 = 100 [mm]

L_2 = 600 [mm]

L_3 = 120 [mm]

L_4 = 100 [mm]

Washer

l_{wd} = 50 [mm] Length

b_{wd} = 60 [mm] Width

t_{wd} = 10 [mm] Thickness

Wedge

Section: HEA 160

l_w = 100 [mm] Length

Material: S275

f_{yw} = 275,00 [MPa] Resistance

Material factors

g_{M0} = 1,00 Partial safety factor

g_{M2} = 1,25 Partial safety factor

g_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.

$N_{j,Ed}$ = 51,05 [kN] Axial force

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

$V_{j,Ed,z}$ = 11,84 [kN] Shear force

Results

Tension zone

STEEL FAILURE

A_b = 157 [mm^2] Effective anchor area [Table 3.4]

f_{ub} = 400,00 [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/g_{M2}

$F_{t,Rd,s1}$ = 38,43 [kN] Anchor resistance to steel failure [Table 3.4]

g_{Ms} = 1,20 Partial safety factor CEB [3.2.3.2]

f_{yb} = 240,00 [MPa] Yield strength of the anchor material CEB [9.2.2]

$F_{t,Rd,s2} = f_{yb}*A_b/g_{Ms}$

$F_{t,Rd,s2}$ = 31,40 [kN] Anchor resistance to steel failure CEB [9.2.2]

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

$F_{t,Rd,s}$ = 31,40 [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]

$f_{ctd} = 0.7*0.3*f_{ck}^{2/3}/g_C$

f_{ctd} = 1,20 [MPa] Design tensile resistance EN 1992-1:[8.4.2.(2)]

h_1 = 1,00 Coeff. related to the quality of the bond conditions and concreting conditions EN 1992-1:[8.4.2.(2)]

h_2 = 1,00 Coeff. related to the bar diameter EN 1992-1:[8.4.2.(2)]

$f_{bd} = 2.25*h_1*h_2*f_{ctd}$

f_{bd} = 2,69 [MPa] Design value of the ultimate bond stress EN 1992-1:[8.4.2.(2)]

h_{ef} = 570 [mm] Effective anchorage depth EN 1992-1:[8.4.2.(2)]

$F_{t,Rd,p} = p*d*h_{ef}*f_{bd}$

$F_{t,Rd,p}$ = 77,16 [kN] Design uplift capacity EN 1992-1:[8.4.2.(2)]

CONCRETE CONE FAILURE

h_{ef} = 333 [mm] Effective anchorage depth CEB [9.2.4]

$N_{Rk,c}^0 = 7.5[N^{0.5}/mm^{0.5}]*f_{ck}^{0.5}*h_{ef}^{1.5}$

$N_{Rk,c}^0$ = 228,22 [kN] Characteristic resistance of an anchor CEB [9.2.4]

$s_{cr,N}$ = 1000 [mm] Critical width of the concrete cone CEB [9.2.4]

$c_{cr,N}$ = 500 [mm] Critical edge distance CEB [9.2.4]

$A_{c,N0}$ = 1000000 [mm^2] Maximum area of concrete cone CEB [9.2.4]

$A_{c,N}$ = 500000 [mm^2] Actual area of concrete cone CEB [9.2.4]

$y_{A,N} = A_{c,N}/A_{c,N0}$

$y_{A,N}$ = 0,50 Factor related to anchor spacing and edge distance CEB [9.2.4]

c = 450 [mm] Minimum edge distance from an anchor CEB [9.2.4]

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

$y_{s,N} =$ 0,97 | Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete | CEB [9.2.4]

$y_{ec,N} =$ 1,00 | Factor related to distribution of tensile forces acting on anchors | CEB [9.2.4]

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

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

$y_{ucr,N} =$ 1,00 | Factor taking into account whether the anchorage is in cracked or non-cracked concrete | CEB [9.2.4]

$g_{Mc} =$ 2,16 | Partial safety factor | CEB [3.2.3.1]

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

$F_{t,Rd,c} =$ 51,24 [kN] | Design anchor resistance to concrete cone failure | EN 1992-1:[8.4.2.(2)]

SPLITTING FAILURE

$h_{ef} =$	570	[mm]	Effective anchorage depth	CEB [9.2.5]

$N_{Rk,c}^0 = 7.5[N^{0.5}/mm^{0.5}] * f_{ck}^{0.5} * h_{ef}^{1.5}$

$N_{Rk,c}^0 =$	510,32	[kN]	Design uplift capacity	CEB [9.2.5]
$s_{cr,N} =$	1140	[mm]	Critical width of the concrete cone	CEB [9.2.5]
$c_{cr,N} =$	570	[mm]	Critical edge distance	CEB [9.2.5]
$A_{c,N0} =$	1299600	[mm^2]	Maximum area of concrete cone	CEB [9.2.5]
$A_{c,N} =$	500000	[mm^2]	Actual area of concrete cone	CEB [9.2.5]

$y_{A,N} = A_{c,N}/A_{c,N0}$

$y_{A,N} =$	0,38		Factor related to anchor spacing and edge distance	CEB [9.2.5]
$c =$	450	[mm]	Minimum edge distance from an anchor	CEB [9.2.5]

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

$y_{s,N} =$ 0,94 | Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete | CEB [9.2.5]

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

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

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

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

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

$y_{h,N} =$	0,79	Coeff. related to the foundation height	CEB [9.2.5]
$g_{M,sp} =$	2,16	Partial safety factor	CEB [3.2.3.1]

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

$F_{t,Rd,sp} =$ 67,25 [kN] | Design anchor resistance to splitting of concrete | CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

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

$F_{t,Rd}$ =	31,40	[kN]	Tensile resistance of an anchor	

BENDING OF THE BASE PLATE

$l_{eff,1}$ =	207	[mm]	Effective length for a single bolt row for mode 1	[6.2.6.5]
$l_{eff,2}$ =	207	[mm]	Effective length for a single bolt row for mode 2	[6.2.6.5]
m =	42	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$M_{pl,1,Rd}$ =	5,70	[kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd}$ =	5,70	[kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd}$ =	537,12	[kN]	Resistance of a plate for mode 1	[6.2.4]
$F_{T,2,Rd}$ =	157,31	[kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd}$ =	62,80	[kN]	Resistance of a plate for mode 3	[6.2.4]

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

$F_{t,pl,Rd}$ =	62,80	[kN]	Tension resistance of a plate	[6.2.4]

TENSILE RESISTANCE OF A COLUMN WEB

t_{wc} =	6	[mm]	Effective thickness of the column web	[6.2.6.3.(8)]
$b_{eff,t,wc}$ =	207	[mm]	Effective width of the web for tension	[6.2.6.3.(2)]
A_{vc} =	1324	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
w =	0,68		Reduction factor for interaction with shear	[6.2.6.3.(4)]

$F_{t,wc,Rd} = w\ b_{eff,t,wc}\ t_{wc}\ f_{yc} / g_{M0}$

$F_{t,wc,Rd}$ =	233,47	[kN]	Column web resistance	[6.2.6.3.(1)]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$N_{j,Rd}$ =	62,80	[kN]	Resistance of a spread footing for axial tension	[6.2.8.3]

Connection capacity check

$N_{j,Ed} / N_{j,Rd} \leq 1,0$ (6.24)	0,81 < 1,00	verified	(0,81)

Shear

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

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

$a_{d,y}$ = 0,93	Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$a_{b,y}$ = 0,93	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]

$F_{1,vb,Rd,y} = k_{1,y} * a_{b,y} * f_{up} * d * t_p / g_{M2}$

$F_{1,vb,Rd,y}$ = 254,81 [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

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

$a_{d,z}$ = 1,85	Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$a_{b,z}$ = 0,93	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]

$F_{1,vb,Rd,z} = k_{1,z}*a_{b,z}*f_{up}*d*t_p / g_{M2}$

$F_{1,vb,Rd,z}$ = 256,00 [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

SHEAR OF AN ANCHOR BOLT

a_b =	0,37		Coeff. for resistance calculation $F_{2,vb,Rd}$	[6.2.2.(7)]
A_{vb} =	201	[mm^2]	Area of bolt section	[6.2.2.(7)]
f_{ub} =	400,00	[MPa]	Tensile strength of the anchor material	[6.2.2.(7)]
g_{M2} =	1,25		Partial safety factor	[6.2.2.(7)]

$F_{2,vb,Rd} = a_b*f_{ub}*A_{vb}/g_{M2}$

$F_{2,vb,Rd}$ =	23,68	[kN]	Shear resistance of a bolt - without lever arm	[6.2.2.(7)]

a_M =	2,00		Factor related to the fastening of an anchor in the foundation	CEB [9.3.2.2]
$M_{Rk,s}$ =	0,03	[kN*m]	Characteristic bending resistance of an anchor	CEB [9.3.2.2]
l_{sm} =	48	[mm]	Lever arm length	CEB [9.3.2.2]
g_{Ms} =	1,20		Partial safety factor	CEB [3.2.3.2]

$F_{v,Rd,sm} = a_M*M_{Rk,s}/(l_{sm}*g_{Ms})$

$F_{v,Rd,sm}$ =	0,94	[kN]	Shear resistance of a bolt - with lever arm	CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c}$ =	110,69	[kN]	Design uplift capacity	CEB [9.2.4]
k_3 =	2,00		Factor related to the anchor length	CEB [9.3.3]
g_{Mc} =	2,16		Partial safety factor	CEB [3.2.3.1]

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

$F_{v,Rd,cp}$ =	102,49	[kN]	Concrete resistance for pry-out failure	CEB [9.3.1]

CONCRETE EDGE FAILURE

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

$V_{Rk,c,y}^0$ =	224,07	[kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$y_{A,V,y}$ =	0,52		Factor related to anchor spacing and edge distance	CEB [9.3.4]
$y_{h,V,y}$ =	1,00		Factor related to the foundation thickness	CEB [9.3.4.(c)]
$y_{s,V,y}$ =	0,86		Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(d)]

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

$V_{Rk,c,y}{}^{0}$ =	224,07	[kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$y_{ec,V,y}$ =	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)]
$y_{a,V,y}$ =	1,00		Factor related to the angle at which the shear load is applied	CEB [9.3.4.(f)]
$y_{ucr,V,y}$ =	1,00		Factor related to the type of edge reinforcement used	CEB [9.3.4.(g)]
g_{Mc} =	2,16		Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,c,y} = V_{Rk,c,y}{}^{0}*y_{A,V,y}*y_{h,V,y}*y_{s,V,y}*y_{ec,V,y}*y_{a,V,y}*y_{ucr,V,y}/g_{Mc}$

$F_{v,Rd,c,y}$ =	46,02	[kN]	Concrete resistance for edge failure	CEB [9.3.1]

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

$V_{Rk,c,z}{}^{0}$ =	153,69	[kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$y_{A,V,z}$ =	0,86		Factor related to anchor spacing and edge distance	CEB [9.3.4]
$y_{h,V,z}$ =	1,00		Factor related to the foundation thickness	CEB [9.3.4.(c)]
$y_{s,V,z}$ =	0,96		Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(d)]
$y_{ec,V,z}$ =	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)]
$y_{a,V,z}$ =	1,00		Factor related to the angle at which the shear load is applied	CEB [9.3.4.(f)]
$y_{ucr,V,z}$ =	1,00		Factor related to the type of edge reinforcement used	CEB [9.3.4.(g)]
g_{Mc} =	2,16		Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,c,z} = V_{Rk,c,z}{}^{0}*y_{A,V,z}*y_{h,V,z}*y_{s,V,z}*y_{ec,V,z}*y_{a,V,z}*y_{ucr,V,z}/g_{Mc}$

$F_{v,Rd,c,z}$ =	58,38	[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

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

$F_{v,Rd,wg,y} = 354{,}67$ [kN]   Resistance for bearing pressure of the wedge onto concrete

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

$F_{v,Rd,wg,z} = 373{,}33$ [kN]   Resistance for bearing pressure of the wedge onto concrete

SHEAR CHECK

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

$V_{j,Rd,y} =$	356,55	[kN]	Connection resistance for shear	CEB [9.3.1]

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

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

$V_{j,Rd,z} =$	375,21	[kN]	Connection resistance for shear	CEB [9.3.1]

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

Welds between the column and the base plate

$s_{\wedge} =$	10,07	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\wedge} =$	10,07	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{yII} =$	0,05	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$t_{zII} =$	11,04	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
$b_W =$	0,85		Resistance-dependent coefficient	[4.5.3.(7)]

$s_{\wedge} / (0.9*f_u/g_{M2})) \leq 1.0$ (4.1)	$0{,}03 < 1{,}00$	verified	(0,03)
$Ö(s_{\wedge}^2 + 3.0\ (t_{yII}^2 + t_{\wedge}^2)) / (f_u/(b_W*g_{M2}))) \leq 1.0$ (4.1)	$0{,}05 < 1{,}00$	verified	(0,05)
$Ö(s_{\wedge}^2 + 3.0\ (t_{zII}^2 + t_{\wedge}^2)) / (f_u/(b_W*g_{M2}))) \leq 1.0$ (4.1)	$0{,}07 < 1{,}00$	verified	(0,07)

Weakest component:

ANCHOR BOLT - RUPTURE

Connection conforms to the code   Ratio   0,81

$V_{Ed} = 11{,}84\ kN$

$N_{Ed} = 51{,}05\ kN$

Karakteristike vijaka i ploče:

Vijci: M16 – k.v. 4.6.

Granica popuštanja: $f_{yb} = 240\ N/mm^2$

Vlačna čvrstoća: $f_{ub} = 400\ N/mm^2$

Promjer jezgre vijka: $d = 16\ mm$

Promjer rupe za vijak: $d_0 = 18\ mm$

Vlačna površina poprečnog presjeka vijka: $A_s = 157\ mm^2$

Čelična ploča: $S\ 275 \quad t = 20\ mm$

Vlačna čvrstoća: $f_u = 430\ N/mm^2$

Otpornost vijaka na vlak:

$N_{Rd,u,1} = n * F_{t,Rd,u}$

n – ukupni broj vijaka

$$F_{t,Rd} = \frac{k_2 * f_{ub} * A_s}{\gamma_{M2}} = \frac{0,9 * 400 * 157}{1,25} = 45,22\ kN$$

$N_{Rd,u,1} = 2 * 45,22 = 90,4\ kN$

$N_{Ed} \leq N_{Rd,u}$; $51,05 < 90,4\ kN$ **ZADOVOLJAVA**(56%)

Otpornost vijaka na posmik:

Otpornost jednog vijka na posmik:

$$F_{v,Rd} = \frac{\alpha_v * f_{ub} * A_s}{\gamma_{M2}}$$

$\alpha_v = 0,37$

$$F_{v,Rd} = \frac{0,37 * 400 * 157}{1,25} = 18,59\ kN$$

$F_{v,Rd} = 18,59 * 2 = 37,18\ kN$

$F_{v,Ed} \leq F_{v,Rd}$; $11,84 < 37,18\ kN$ **ZADOVOLJAVA**(32%)

Otpornost na pritisak po omotaču rupe osnovnog materijala:

$$e_1 = \frac{l_{pd}}{2} = \frac{500}{2} = 250\,mm$$

$$e_2 = \frac{b_{pd} - e_v}{2} = \frac{300 - 100}{2} = 100\,mm$$

$$\alpha_d = \frac{e_1}{3 * d_0} = \frac{250}{3 * 18} = 4{,}63$$

$$\alpha_b = \min\left(\alpha_b; \frac{f_{ub}}{f_u}; 1\right) = \min\left(4{,}63; \frac{400}{430}; 1\right) = 0{,}93$$

$$k_1 = \min\left(2{,}8 * \frac{e_2}{d_0} - 1{,}7\,; 2{,}5\right) = min(15{,}56\,; 2{,}5) = 2{,}5$$

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_{ub} * d * t}{\gamma_{M2}} = \frac{2{,}5 * 0{,}93 * 400 * 16 * 20}{1{,}25} = 238{,}08\,kN$$

$F_{v,Ed} \leq F_{b,Rd}$; $51{,}05 < 238{,}08$ **ZADOVOLJAVA**(21%)

Proračun zavara:

Djelovanje na zavar:

$$F_{w,Ed} = V_{Ed} = 11{,}84\,kN$$

Duljina zavara:

$$L = 2 * (2b - tw - 2r + h - 2tf - 2r)$$

$$L = 2 * (2 * 160 - 6 - 2 * 15 + 152 - 2 * 9 - 2 * 15) = 776\,mm$$

Debljina zavara iznosi 4 mm.

$$F_{w,Rd} = \frac{\frac{f_u}{\sqrt{3} * \beta_w} * a * L}{\gamma_{M2}} = \frac{\frac{43{,}0}{\sqrt{3} * 0{,}85} * 0{,}4 * 77{,}6}{1{,}25} = 725{,}27\,kN$$

$F_{w,Ed} \leq F_{w,Rd}$; $11{,}84 < 725{,}27$ **ZADOVOLJAVA**(2%)

10. ZAKLJUČAK

U ovom diplomskom radu detaljno je analizirano projektiranje čelične hale za sportsko-rekreativni centar, a fokus je na dimenzioniranju elemenata i priključaka. Glavni nosač hale odabran je kao HEA 160 dok su podrožnice dimenzionirane kao IPE 160, sekundarni stupovi kao IPE 240, vjetrovne veze promjera fi 10 mm te fasadni nosači kao UAP 80.

Proces dimenzioniranja uključio je proračune opterećenja kao što su vjetrovna i snježna opterećenja primjenom softvera Robot Autodesk za modeliranje i analizu. Korištenjem naprednih alata omogućeno je precizno prilagođavanje konstrukcije specifičnim zahtjevima sportskih objekata.

Posebna pažnja posvećena je konstrukcijskim vezama između elemenata, pri čemu su provedeni detaljni proračuni čvrstoće i stabilnosti kako bi se osigurala određena otpornost. Integracija galerije sa stubištem od profila IPE 160 dodatno je obogatila funkcionalnost hale, pružajući prostor za gledatelje ili dodatne aktivnosti.

Ovaj rad pruža dublje razumijevanje procesa projektiranja čeličnih konstrukcija za sportske objekte te predstavlja vrijedan doprinos praksi inženjerskog projektiranja. Metodologija i zaključci ovog rada poslužiti će kao temelj za buduće projekte slične namjene, kombinirajući teorijsko znanje s praktičnom primjenom u izgradnji modernih sportskih objekata.

U tablici 15. prikazane su iskoristivosti elemenata i priključaka:

Tablica 15: Prikaz iskoristivosti elemenata i priključaka [izradio autor]

Element	Profil	Materijal	Iskoristivost
Gredni nosači	HEA 160	S275	**62%**
Stupovi	HEA 160	S275	**40%**
Sekundarni stupovi	IPE 240	S275	**55%**
Podrožnice	IPE 160	S275	**71%**
Sekundarni nosači galerije	IPE 160	S275	**41%**
Tetive stubišta	IPE 160	S275	**41%**
Sekundarni nosači stubišta	IPE 160	S275	**9%**
Vjetrovni vezovi	Okrugli puni profil 10 mm	S275	**69%**
Fasadni nosači	UAP 80	S275	**62%**
Detalj A, spoj stupa i grede			**54%**
Detalj B, spoj stupa i grede			**52%**
Detalj C, spoj stupa i grede galerije			**72%**
Detalj D, spoj stupa i temelja			**81%**

11. LITERATURA

[1] https://www.trimo-group.com/hr/proizvodi/krovovi/trimoterm-1

[2] HRN EN 1991 - 1 - 3: 2012/NA, Eurokod 1: Djelovanja na konstrukcije - Dio 1 - 3: Opća djelovanja – Opterećenje snijegom

[3] HRN EN 1991-1-4:2012/NA, Eurokod 1: Djelovanja na konstrukcije - Dio 1 - 4: Opća djelovanja – Djelovanja vjetra

[4] Dujmović, D., Androić, B., Džeba, I., Modeliranje konstrukcija prema Eurocode 3, IA Projektiranje, Zagreb, 20024

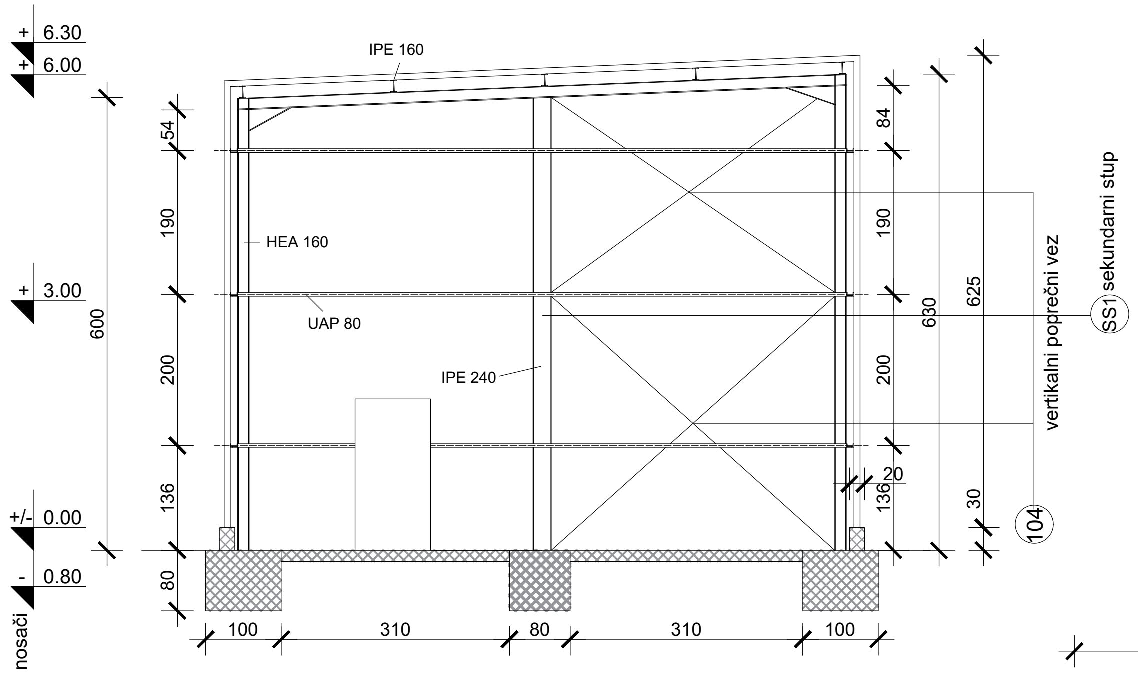
[5] Androic , B., Dujmovic , D., *Čelične konstrukcije - 1. dio*, IA Projektiranje, Zagreb, 2021.

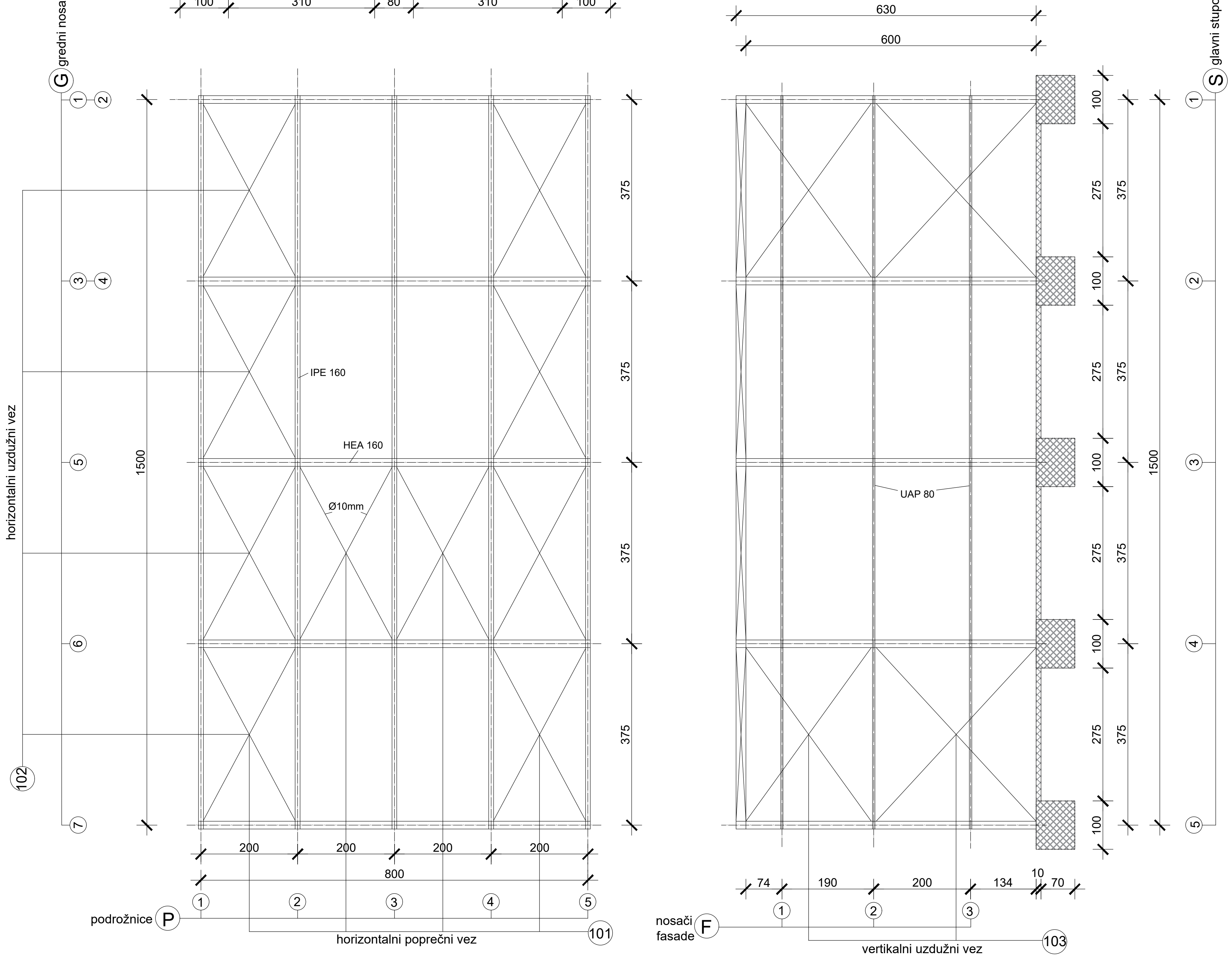
[6] Androic , B., Dujmovic , D., *Čelične konstrukcije - 2. dio*, IA Projektiranje, Zagreb, 2021.

[7] Androic , B., Dujmovic , D., Dzeba, Bulić M., I., *Čelične konstrukcije - 3.* dio, IA Projektiranje, Zagreb, 2024.

NACRTI

DISPOZICIJA VANJSKIH ELEMENATA
M 1:100





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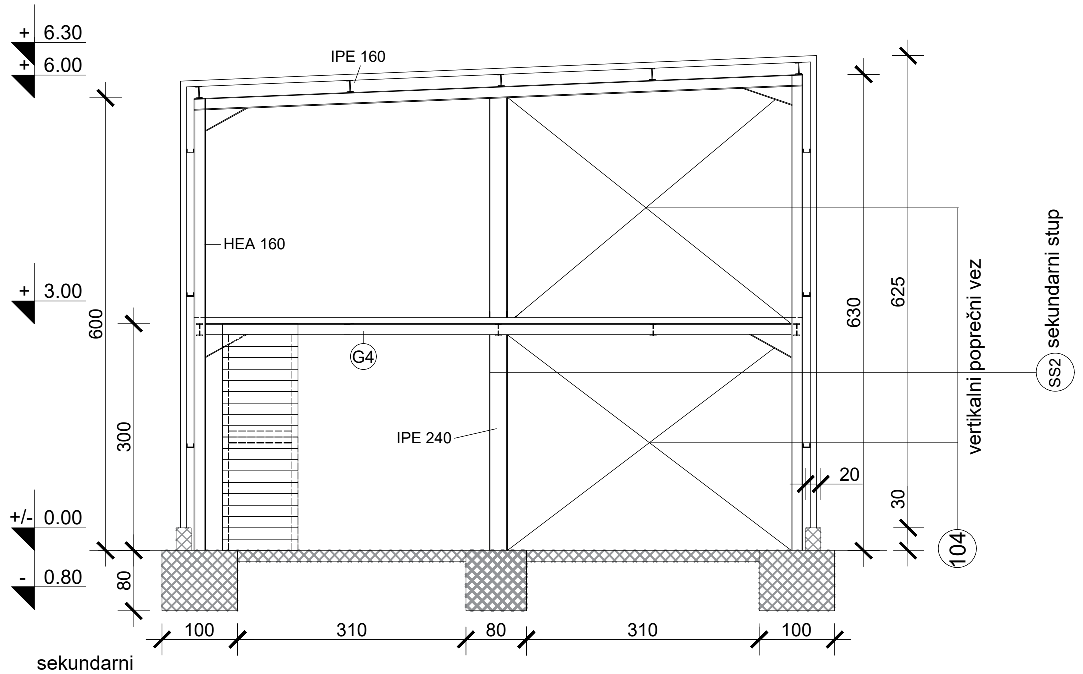
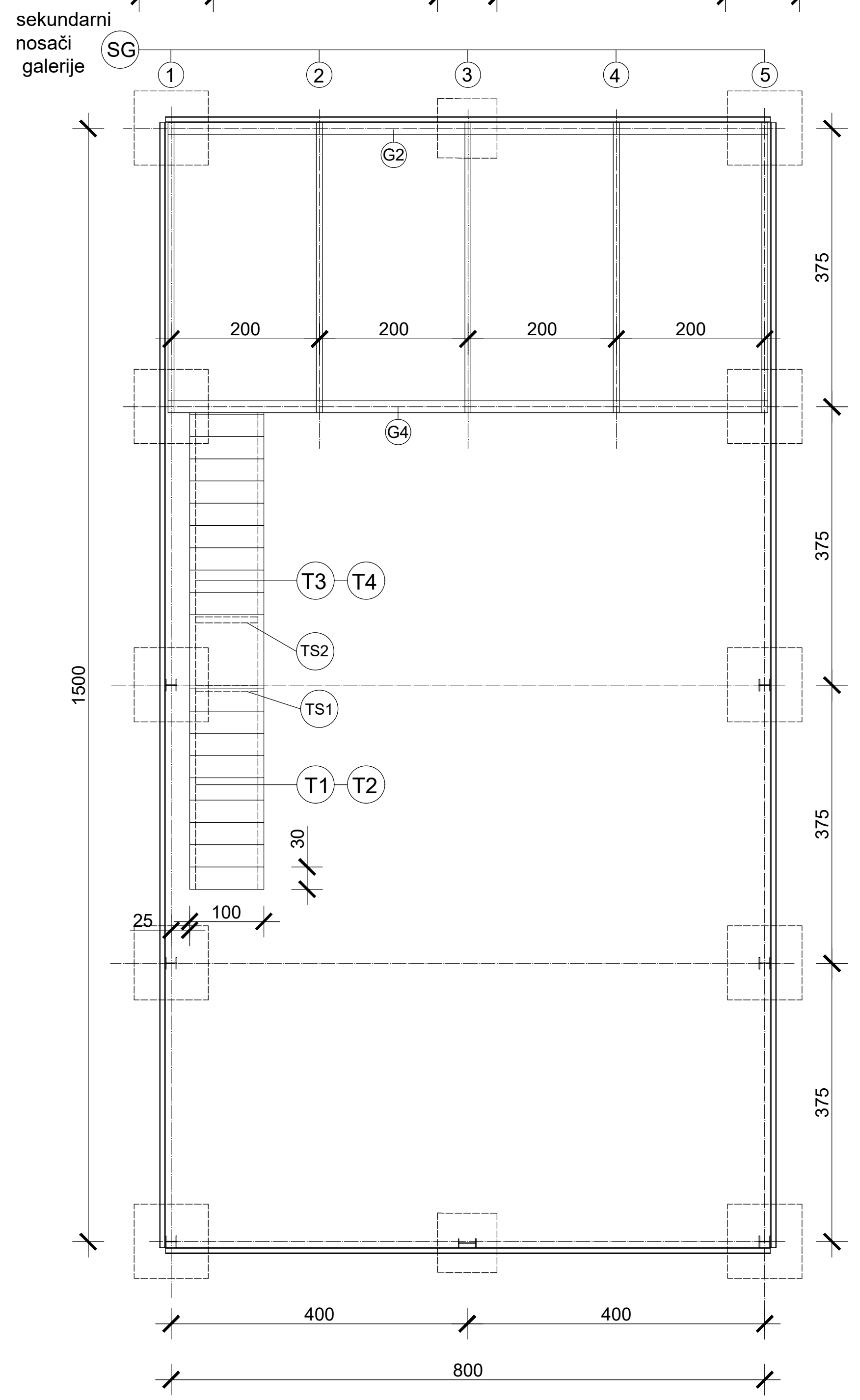
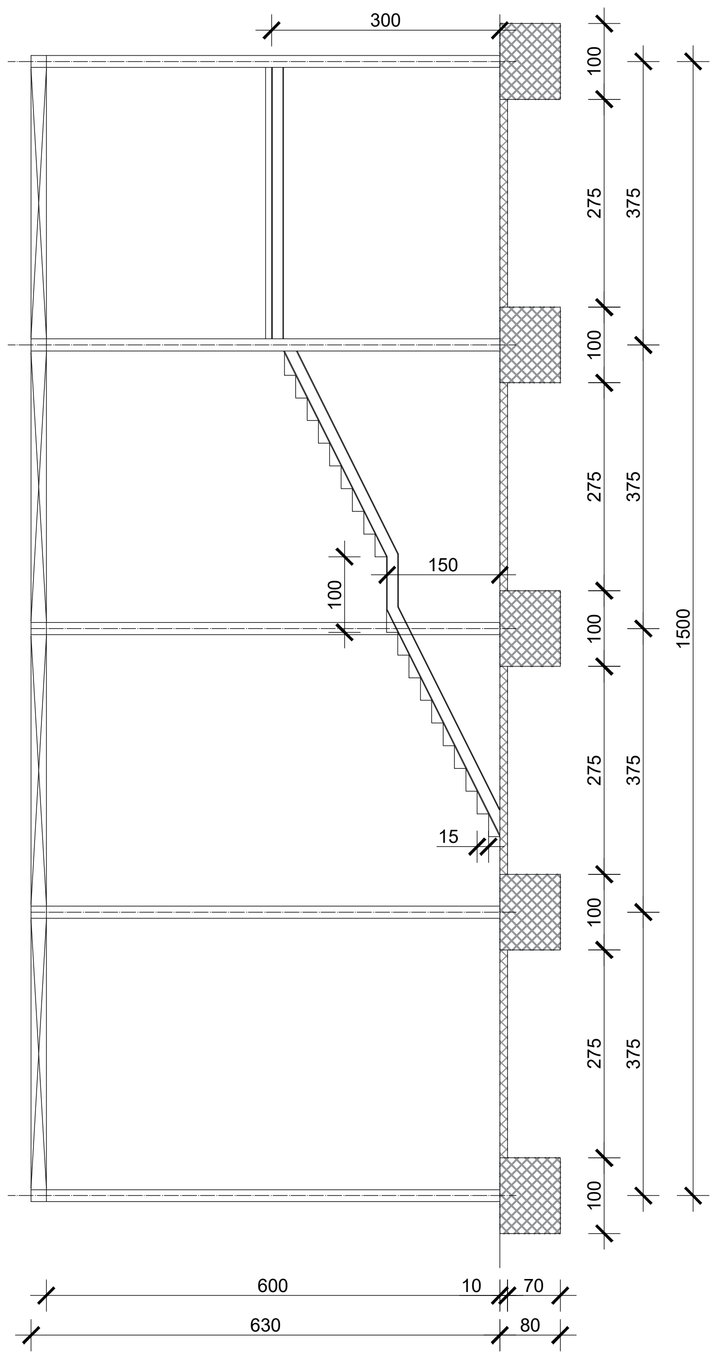
Diplomski rad: **Dimenzioniranje čelične konstrukcije namjene sportsko rekreativnog centra**	Sadržaj nacrta: **Dispozicija vanjskih elemenata**		
Ime i prezime: **Roko Požgaj**	Kolegij: **Čelične konstrukcije**		
Mentor: **Izv.prof.dr.sc. Mladen Bulić dipl.ing.građ**	Datum: 05.07.2024.	Mjerilo: M 1:100	List: 1

DISPOZICIJA UNUTARNJIH ELEMENATA
M 1:100







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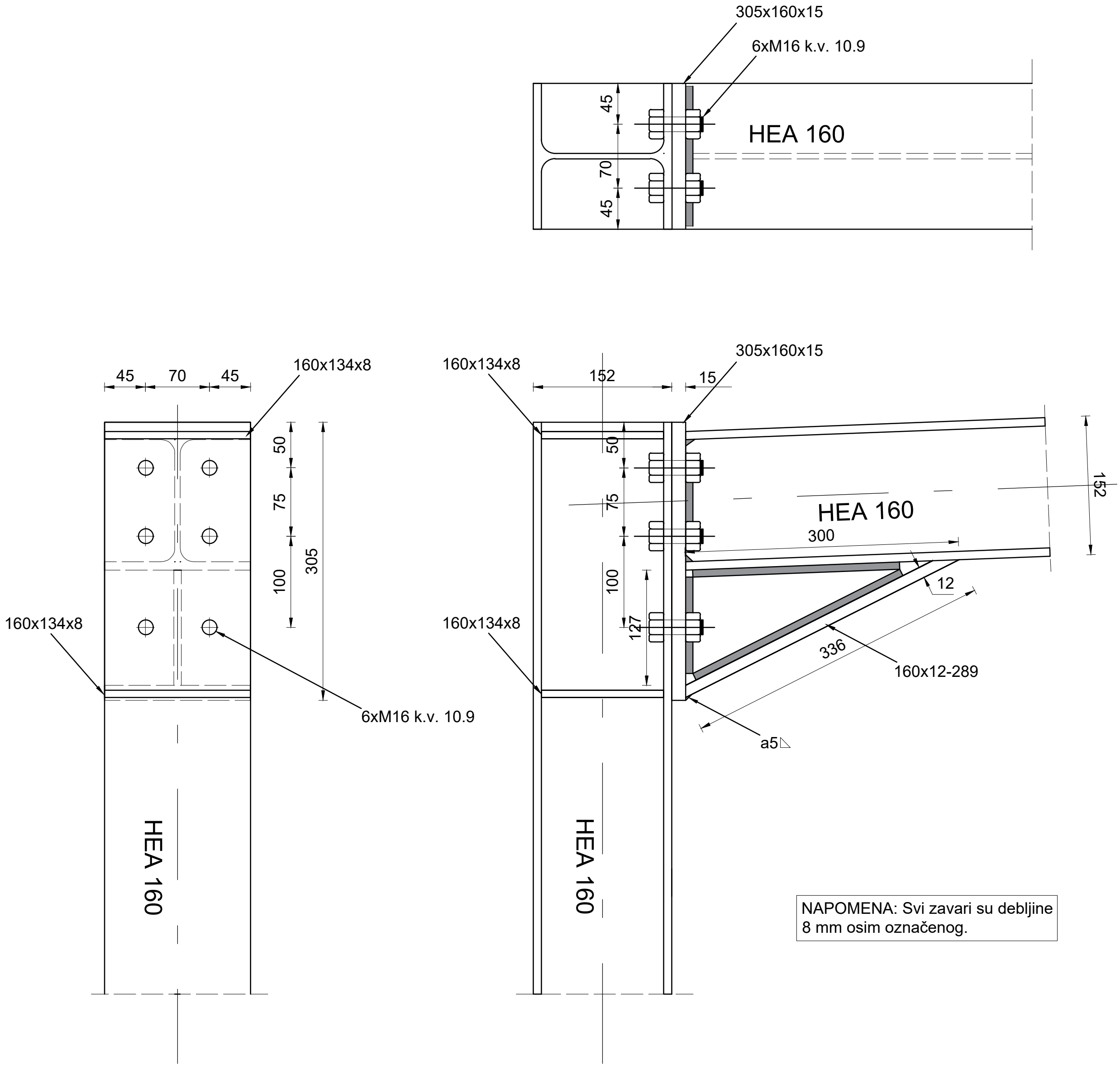
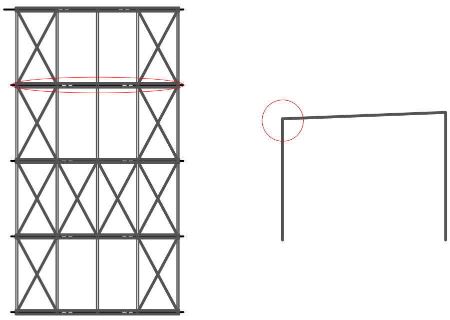
Ime i prezime: **Roko Požgaj**	Kolegij: **Čelične konstrukcije**
Mentor: **Izv.prof.dr.sc. Mladen Bulić dipl.ing.građ**	Datum: 05.07.2024. / Mjerilo: M 1:100 / List: 2

PRIKLJUČAK STUPA I GREDNOG NOSAČA (DETALJ A)
M 1:7,5



NAPOMENA: Svi zavari su debljine 8 mm osim označenog.



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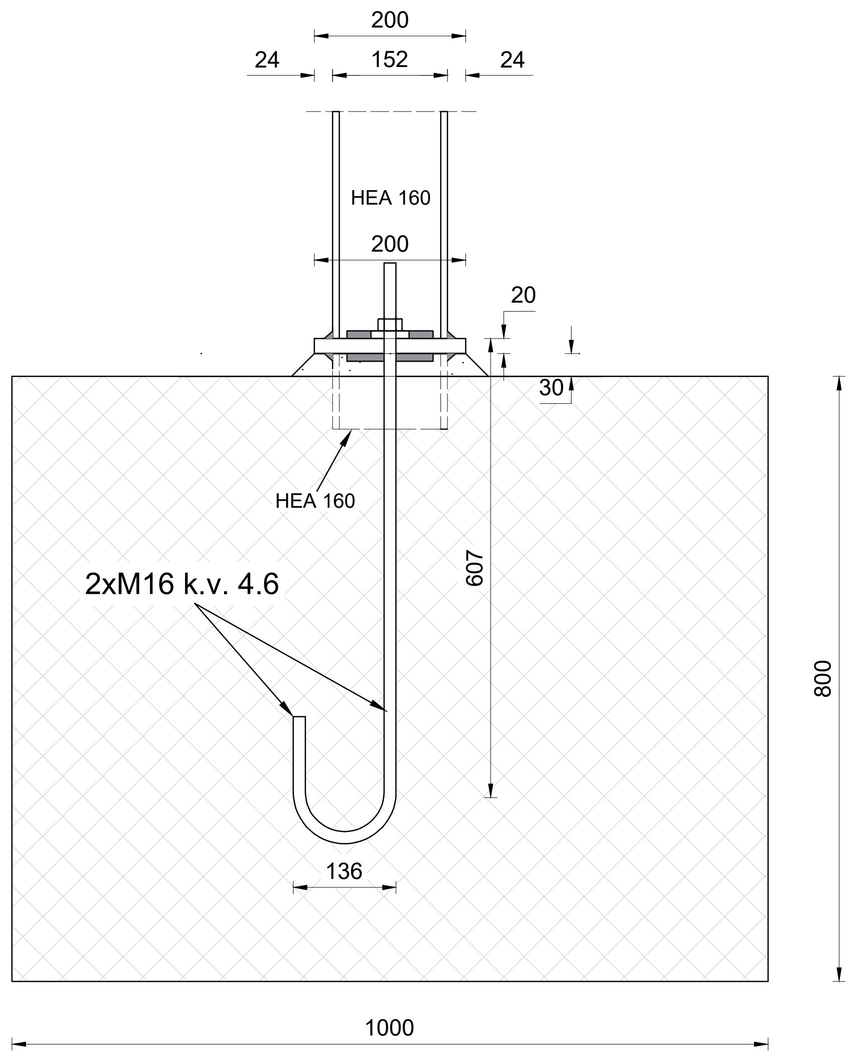
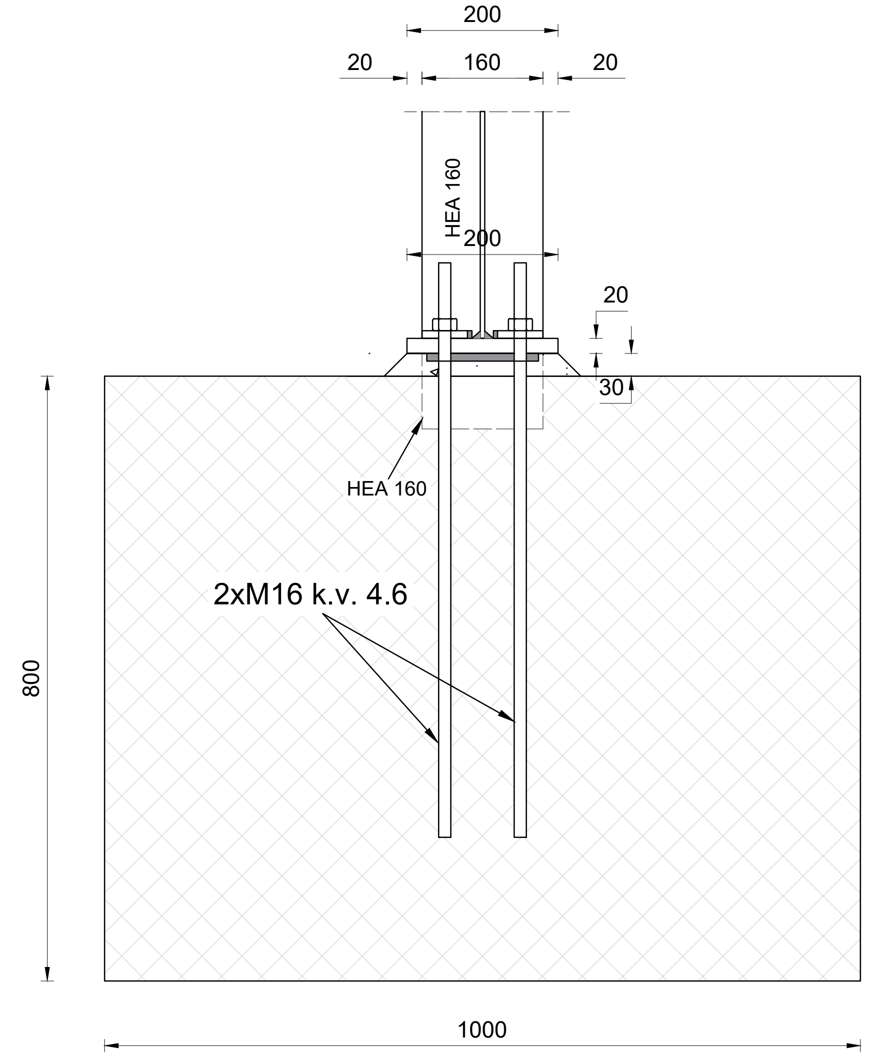
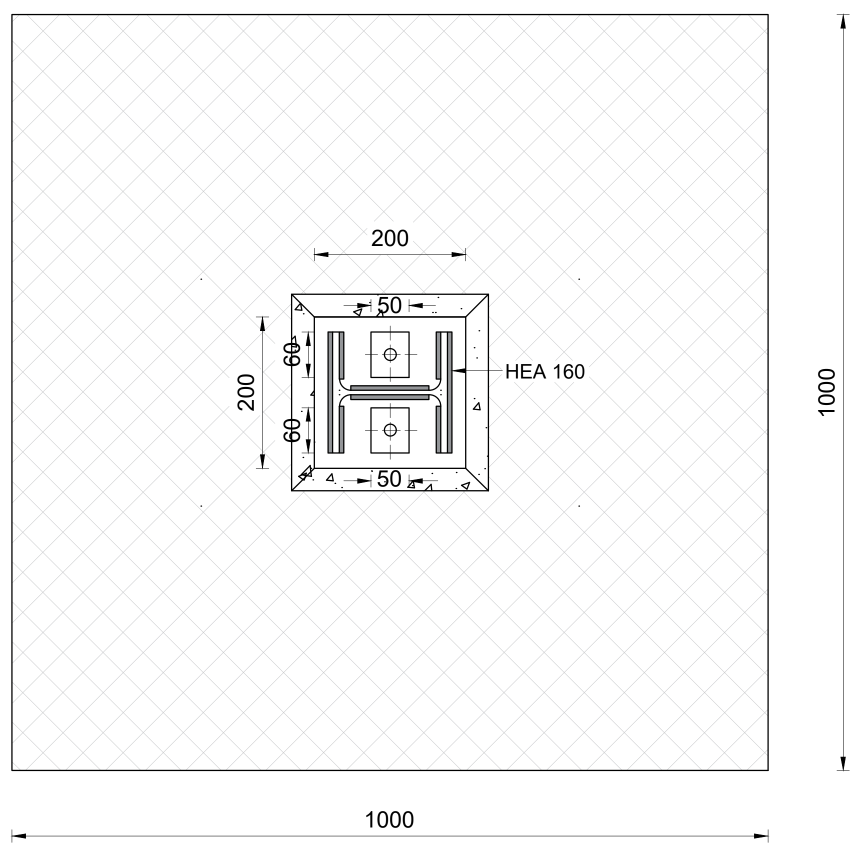
Diplomski rad: **Dimenzioniranje čelične konstrukcije namjene sportsko rekreativnog centra**	Sadržaj nacrta: **Priključak stupa i grednog nosača**		
Ime i prezime: **Roko Požgaj**	Kolegij: **Čelične konstrukcije**		
Mentor: **Izv.prof.dr.sc. Mladen Bulić dipl.ing.građ**	Datum: 05.07.2024.	Mjerilo: M 1:7,5	List: 3

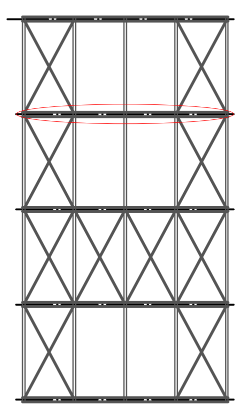
PRIKLJUČAK STUPA NA TEMELJ (DETALJ D)
M 1:10

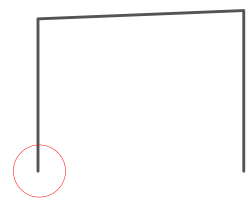






NAPOMENA: Svi zavari su debljine 4 mm.





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Ime i prezime: **Roko Požgaj**	Kolegij: **Čelične konstrukcije**
Mentor: **Izv.prof.dr.sc. Mladen Bulić dipl.ing.građ**	Datum: 05.07.2024. Mjerilo: M 1:10 List: 4