INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

NASLOVNA STRAN NAČRTA

PODATKI O GRADNJI

naziv gradnje: P-BGM – BRUNARICA S PARKIRNIMI POVRŠINAMI - GM

kratek opis gradnje: Novogradnja brunarice s parkirišči in pripadajočo zunanjo ureditvijo

vrste gradnje:

- ☒ novogradnja - novozgrajen objekt
- ☐ novogradnja - prizidava
- ☐ rekonstrukcija
- ☐ sprememba namembnosti
- ☐ odstranitev celotnega objekta
- ☐ legalizacija
- ☐ manjša rekonstrukcija

PODATKI O PROJEKTNI DOKUMENTACIJI

vrsta dokumentacije: PZI

številka projekta: 190-25 PZI

PODATKI O NAČRTU

strokovno področje načrta: NAČRT S PODROČJA GRADBENIŠTVA

naziv načrta: GRADBENE KONSTRUKCIJE

številka načrta: 33/25-K

datum izdelave: oktober 2025

datum spremembe:

PODATKI O PROJEKTANTU NAČRTA

projektant načrta (družba): Inženiring biro Armatura d.o.o.

naslov: Zrkovska cesta 75, 2000 Maribor

odg. oseba projektanta načrta: Uroš Žvan

podpis odg. osebe projektanta načrta:

INŽENIRING BIRO ARMATURA D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

PODATKI O IZDELOVALCU NAČRTA

ime in priimek poobl. inženirja: Uroš Žvan, univ. dipl. inž. grad.

identifikacijska številka: IZS G-0028

podpis pooblaščenega inženirja:

UROŠ ŽVAN
univ. dipl. inž. grad.
IZS G-0028

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

KAZALO VSEBINE NAČRTA

NASLOVNA STRAN NAČRTA....1

KAZALO VSEBINE NAČRTA....2

IZJAVA PROJEKTANTA NAČRTA IN POOBLAŠČENEGA STROKOVNJAKA, KI JE IZDELAL NAČRT V PZI IN PID....3

TEHNIČNO POROČILO....4

STATIČNI RAČUN....6

RISBE....57

OPAŽNO ARMATURNA PLOŠČA TEMELJNE PLOŠČE....list 1

KOSOVNICA ARMATURE

JEKLENI OKVIR....list 2

POZICIJSKA RISBA - TLORIS NOSILCEV IN STEBROV STENAH....list 3

POZICIJSKA RISBA STROPA....list 4

POZICIJSKA RISBA DETAJLA 1 IN 2....list 5

POZICIJSKA RISBA DETAJLA 3....list 6

POZICIJSKA RISBA SIDER V ETAŽI....list 7a

POZICIJSKA RISBA SIDER V PRITLIČJU....list 7b

POZICIJSKA RISBA OSTREŠJA....8

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

IZJAVA PROJEKTANTA NAČRTA IN POOBLAŠČENEGA STROKOVNJAKA KI JE IZDELAL NAČRT V PZI

PROJEKTANT NAČRTA

projektant načrta (družba):	Inženiring biro Armatura d.o.o.
naslov:	Zrkovska cesta 75, 2000 Maribor
odg. oseba projektanta načrta:	Uroš Žvan

IN POOBLAŠČEN STROKOVNJAK, KI JE IZDELAL NAČRT

pooblaščen strokovnjak:	Uroš Žvan, univ. dipl. inž. grad.

IZJAVLJATA:

da načrt

vrsta dokumentacije:	PZI
strokovno področje načrta:	načrt s področja gradbeništva
naziv načrta:	načrt gradbenih konstrukcij
številka načrta:	69/21-K
datum izdelave:	april 2024

upošteva relevantne predpise in druge normativne dokumente ter, da so upoštevane ustrezne bistvene in druge zahteve

pooblaščen strokovnjak:	Uroš Žvan, univ. dipl. inž. grad.
identifikacijska številka:	IZS G-0028
podpis pooblaščenega strokovnjaka:	UROŠ ŽVAN univ. dipl. inž. grad. IZS G-0028
odg. oseba projektanta načrta:	Uroš Žvan
podpis odg. oseba projektanta načrta:	INŽENIRING BIRO ARMATURA D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

TEHNIČNO POROČILO

1 PODATKI O INVESTITORJU IN OBJEKTU

Investitor Občina Kranjska gora ima namen zgraditi novo leseno montažno hišo imenovana P-BGM – brunarica s parkirnimi površinami – GM.

Pritlična hiša tlorisno obsega površino 12×6,34 m. Preko teh gabaritov segajo veliki napušči, ki znašajo na čelnih fasadah 2,40 in 1,60 m, na vzdolžnih pa 0,75 m.

2 KONSTRUKCIJSKA ZASNOVA OBJEKTA

TEMELJI

Stavba bo temeljena na temeljni plošči debeline 30 cm, ki bo po obodu zaključena z protizmrzlinskim zobom globine 50 cm pod temeljno ploščo, ki služi tudi za povezavo stavbe s temeljnimi tlemi.

PRITLIČJE IN STREHA

Objekt se od temeljne plošče navzgor izvede z leseno konstrukcijo.

V statiki smo za posamezne razpone in glede na povečane zahteve po nosilnosti (karakteristična snežna obtežba s_k=3,97 kN/m^2) določili dimenzije špirovcev in njihov razmik.

Za nosilne stenske elemente so določene minimalne dimenzije in max. razmiki med stenskimi pokončniki, kakor tudi dimenzije zg. in sp. venca v steni.

Stropovi se, vsak stropnik med njimi, direktno vijačijo v nosilni stenski element oz. njegov zg. venec (praviloma lesni vijaki premera d=6 ali 8mm, min. globina vijačenja v venec je 8d).

Za zagotavljanje togosti in zavetrovanja stropa kot celote (t.i. diafragma) služi z zgornje strani nameščena in na stropnike vijačena obloga (praviloma iz OSB plošč) ali pa se kot alternativa strop izvede iz KLH/CLT plošč deb. min 16 cm.

Z istim namenom se na vsakem nosilnem stenskem elementu nahaja obojestranska obloga (iz MVP plošč in OSB plošč), ki se na stropnike pritrjuje z jeklenimi sponkami – t.i. klamficami (max. razmik med veznimi sredstvi je določen v statiki). Tudi tu je alternativa lahko KLH/CLT plošča.

Nad okenskimi in vratnimi odprtinami so predvidne preklade iz lepljenega lesa in v enem primeru jeklen (glej pozicijske risbe), ki ob straneh nalegajo na stenske pokončnike.

Strešna konstrukcija se delno podpira z jeklenim okvirjem v osi 2 izdelanim iz jeklenega profila HEB 200 in sidranim v AB ploščo.

<u>vezna sredstva</u>

V statiki izbrana vezna sredstva za spoje med lesenimi elementi z oblogami in med seboj, kakor tudi za sidranje na AB ploščo, so tipizirana (atestirana) vezna sredstva proizvajalca Wuerth

Dovoljena je uporaba atestiranih in certificiranih veznih sredstev drugih proizvajalcev (npr. Strong-Tie Simpson, SHERPA, SPAX, Pitzl….)

Zanje izvajalec pred izvedbo obvezno predloži ustrezen tehnični list s primerljivimi vrednostni glede potrebnih nosilnosti in jih posreduje odgovornemu projektantu gradbenih konstrukcij v potrditev.

3 UPORABLJENI BODO NASLEDNJI KONSTRUKCIJSKI MATERIALI

- lesena konstrukcija ostrešja, sten in stropov: masivni les iglavcev C24 (vlažnost ob vgradnji max. 20%)
- okenske preklade, ojačani stebri v stenah, nosilci v stropu: lepljen les GL24h (vlažnost ob vgradnji max. 20%)
- beton temeljev: C25/30 XC2
- jeklena konstrukcija jeklo S235 JR
- vijačni material kvalitete 8.8
- lepljena sidra Hilti HUS4-H 16×165 za sidranje v beton
- armaturno jeklo S500 B

Razred izvedbe jeklene konstrukcije je EXC2.

Protikorozisjka zaščita jeklene konstrukcije se izvede za razred trajnosti H (15 do 25 let) skladno s SIST EN ISO 12944-1:2017 in za razred korozivnosti okolja C1 skladno s SIST EN ISO 12944-2:2017.

4 VPLIVI NA KONSTRUKCIJO

Na konstrukcijo deluje naslednji stalni vplivi: lastna teža konstrukcije, teža sekundarnih elementov, teža oblog in stavbnega pohištva ter drugih obdelav.

Spremenljive vplive na konstrukcijo predstavljajo: koristna obtežba (kat. A) v objektu, koristna obtežba (kat. H) na strehi, vplivi vetra, snega in potresa.

Vplivi vetra so določeni za vetrno cono 1, kategorijo terena III. in nadmorsko višino pod 1.000 m.

Vplivi snega so določeni za alpsko cono 3 z nadmorsko višino 747 m.

Na osnovi lokacije stavbe in njenega namena zanjo veljajo naslednji vhodni podatki za izračun seizmičnih vplivov:
- kategorija pomena II.
- pospešek temeljnih tal 0,250×g
- faktor obnašanja q=5 (velikostenski panelni sistem)

5 TEMELJENJE

Temeljenje konstrukcija bo izvedeno na temeljni plošči in pasovnih temeljih.

Geomehanske preiskave niso bile narejena, je pa bil opravljen ogled izkopa v kleti, ki jasno kaže na temeljenje na prodno peščenih slojih.

6 UPORABLJENI STANDARDI IN IZRAČUN KONSTRUKCIJE

Izračun konstrukcije temelji na standardih družine Evrokod in sicer SIST EN 1990 do SIST EN 1999.

Izračun konstrukcije je bil opravljen s programsko opremo Frilo. Tower 8.5, Graitec Advance Steel Design in Hilti Profis Anchor.

7 ZAKLJUČEK

Stavba zgrajena skladno s tem načrtom gradbene konstrukcije bo, ob predpostavki solidne izvedbe, izpolnjevala bistveni zahtevi mehanske odpornosti in stabilnosti.

v Mariboru, oktobra 2025

sestavil:

Uroš Žvan, u.d.i.g.

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

STATIČNI RAČUN

ANALIZA OBTEŽB....7
STREHA: ŠPIROVCI IN ŠKARJE - POZICIJA 100A....14
SLEMENSKA LEGA POZICIJA 101....20
KAPNE LEGE POZICIJA 102....25
STEBER 101 A (POD 101) V STENI ALI KOT SOHA A....30
STEBER 101 C (POD 101) V STENI ALI KOT SOHA C....32
STEBER 101 D (POD 101) V STENI ALI KOT SOHA D....34
STROP P – POZICIJA 200A....36
STROP P – POZICIJA 200B....38
NOSILEC POZICIJA 201 NAD OKNOM V ČELNI STENI 1....40
NOSILEC POZICIJA 202 VRATI V OSI 4 – D....44
STEBER SP1 V PRITLIČJU – VOGALNI STEBER KAPNE STENE....47
SHERPA SPOJNI ELEMENTI....51
POTRESNA ANALIZA....53
JEKLEN OKVIR....68
SPOJ JEKLENEGA OKVIRJA....73
SIDRANJE JEKLENEGA OKVIRJA....80
TEMELJNA PLOŠČA....87

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Analiza obtežb

1.1 Analiza lastne teže konstrukcije

a) **STREHA**
Dvokapnica , naklon 45,0°

strešna kritina in letve 5/5cm	0,550	kN/m^2
prezračevalne letve 5/5 (e=95cm)	0,011	
paroprepustna folija		
špirovci	0,092	
termo izolacija	0,062	
letve -podkonstrukcija	0,028	
parna zapora PE folija 0,2mm		
mavčno-kartonska plošča 12,5mm	0,137	
Stalna obtežba strehe skupaj g =	**0,880**	kN/m^2

b1) **STROP MANSARDE** **elementi**

deske	0,105	
škarje	0,092	
termo izolacija	0,103	
letve -podkonstrukcija	0,022	
parna zapora PE folija 0,2mm		
mavčno-kartonska plošča 12,5mm	0,137	
Stalna obtežba stropa skupaj =	**0,459**	kN/m^2

b2) **STROP NAD PRITLIČJEM - leseni**

OSB plošča min. 18mm	0,150	kN/m^2
stropniki	0,148	kN/m^2
termo izolacija	0,030	
podkonstrukcija	0,022	
mavčno-kartonska plošča 12,5mm	0,137	
Stalna obtežba stropa skupaj =	**0,487**	kN/m^2

c) **ESTRIH in termoizolacija (stiropor SGP20)**

estrih 5,5cm	1,210	
parna zapora PE folija 0,2mm		
termoizolacija Stiropor SGP20 (7cm)	0,021	
	1,231	kN/m^2
Finalni pod: parket 22mm	0,092	
	~ **0,100**	kN/m^2

1.2 Koristna obtežba

a-Streha je nepohodna oz. pohodna le za vzdrževalna dela, po EC1 spada v ktg.H.

Naklon strehe je α = 45,0°

* pri naklonu strehe $\alpha \leq 20°$ → q_k=0,75kN/m^2 in Q_k=1,5kN/m^2

* pri naklonu strehe $20,1° < \alpha \leq 40°$ → linearno interpoliramo

* pri naklonu strehe $\alpha > 40,1°$ → q_k=0,0 in Q_k=0,0kN/m^2

V našem primeru je α > 40° → q_k=0,0kN/m^2 in Q_k=0,0kN/m^2

Obremenitev na škarje upoštevamo po predpisih DIN 1052 Teil 1-3, ki določajo, da se v podstrešjih

* do višine 2,0m predpostavi q_k=1,0kN/m^2
* nad višino 2,0m predpostavi q_k=2,0kN/m^2

V našem primeru je h = 1,50 < 2,00 → q_k=1,0kN/m^2

b-Koristna obtežba stanovanjskih prostorov, po EC ktg. A = 2,000 kN/m^2

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

2. Analiza obtežbe zaradi snega

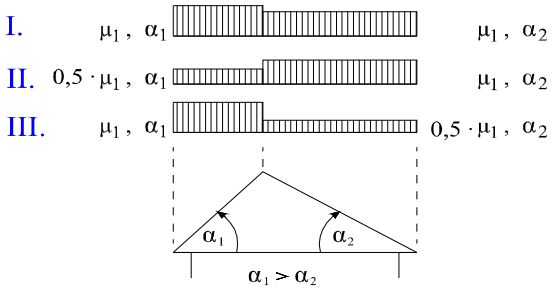
Podatki za obtežbo snega so vzeti iz standarda SIST EN 1991-1-3 : 2004

$s = \mu_i \times c_e \times c_t \times s_k$ (SIST EN 1991-1-3 : 2004 (5.1))

Objekt se nahaja v coni: A3 na nadmorski višini: 747 m

s_k =	3,97	kN/m^2	karakteristična obtežba snega na tleh
c_e =	1,00		koeficient izpostavljenosti
c_t =	1,00		termični koeficient

α_1 = 45,0 ° α_2 = 45,0 °



1) Naklon strehe med 0° in 30°: (SIST EN 1991-1-3 : 2004 (5.3.3))
μ_1 = 0,8

2) Naklon strehe med 30° in 60°:
μ_1 = 0,8 (60-α)/30

3) Naklon strehe več kot 60°:
μ_1 = 0,0

Iz pogoja 2 → $\mu_{1,L}$ = 0,40 → = **0,80** μ . . . Ne sme biti manj kot 0,8!
1 → $\mu_{1,D}$ = 0,80 → = **0,80**

$s_{I,L} = \mu_{1,L} C_e C_t s_k$ =	**3,18**	kN/m^2
$s_{I,D} = \mu_{1,D} C_e C_t s_k$ =	**3,18**	
$s_{II,L} = 0,5\ \mu_{1,L} C_e C_t s_k$ =	1,59	
$s_{II,D} = \mu_{1,D} C_e C_t s_k$ =	3,18	
$s_{III,L} = \mu_1 C_e C_t s_k$ =	3,18	
$s_{III,D} = 0,5\ \mu_{1,D} C_e C_t s_k$ =	1,588928	

3. Analiza obtežbe zaradi vetra

Podatki za obtežbo vetra so vzeti iz standarda SIST EN 1991-1-4 : 2005

GozdMartu → cona **1** → hitrost zračnih mas: $v_{b,0}$ = **20,0** m/s

$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}$ =	20,00	m/s	Enačba (4.1)
$q_b = 0{,}5 \cdot \rho \cdot (v_b)^2$ =	0,250	kN/m^2	Enačba (4.10)
Koeficient hribovitosti	$c_0(z)$ =	1,00	
Kategorija terena		III	
Koeficient kategorije objekta	k_r =	0,22	Enačba (4.5)
Višina hrapavosti	z_0 =	0,30	
Višina enakomerne hitrosti	z_{min} =	5,00 m	
Višina objekta nad nivojem terena	z =	7,00 m	

Koeficient hrapavosti terena — Poglavje (4.3.2)

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) = 0{,}68 \qquad c_r(z) = c_r(z_{min}) \text{ za } z \le z_{min}$$ Enačba (4.4)

Koeficient izpostavljenosti objekta

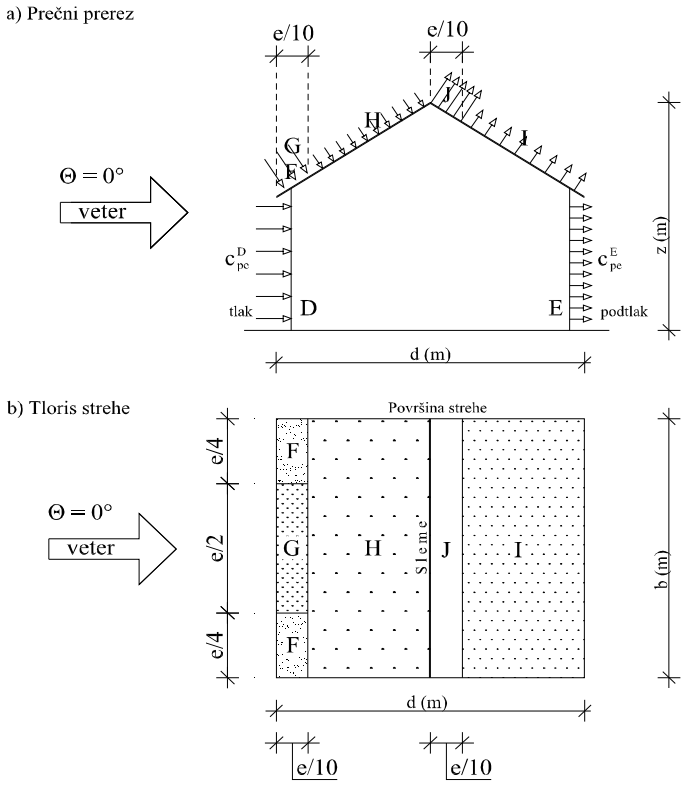
$$c_e(z) = \frac{q_p(z)}{q_b} = 1{,}48$$ Enačba (4.9)

$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b$ = 13,57 m/s — Enačba (4.3)

$$l_v = \frac{k_l}{c_0(z) \cdot \ln(z/z_o)} = 0{,}32 \qquad l_v(z) = l_v(z_{min}) \text{ za } z \le z_{min}$$ Enačba (4.7)

$$q_p(z) = [1 + 7 \cdot l_v(z)] \cdot 1/2 \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b = 0{,}37$$ Enačba (4.8)

Pritisk vetra na STREHO / pravokotno na sleme θ = 0° z LEVE strani



Zunanji pritisk

$e = \min\begin{cases} d = \\ b = \\ 2 \cdot h = \end{cases}$	7,60	m	e =	14,00
	16	m	e/2 =	7,00
	14	m	e/4 =	3,50
			e/10 =	1,40

$c_{pe,10}$ za dani naklon strehe

F	G	H	I	J
0,7	**0,7**	**0,6**	**-0,2**	**-0,3**

Vrednosti c_{pe} lahko poenostavimo, s tem ko predvidimo kontinuiran vpliv po dolžini strešine.

$$c_{pe,p,L} \cdot d/2 = c_{pe,F} \cdot e/10 + c_{pe,H} \cdot (d/2 - e/10)$$

$c_{pe,p,L}$ = 0,64 . . . leva stran

$$w_{prav,e,L} = q_p(z_e) \cdot c_{pe,p,L} =$$ **0,24** kN/m^2 . . . tlak na streho

$$c_{pe,p,D} \cdot d/2 = c_{pe,J} \cdot e/10 + c_{pe,I} \cdot (d/2 - e/10)$$

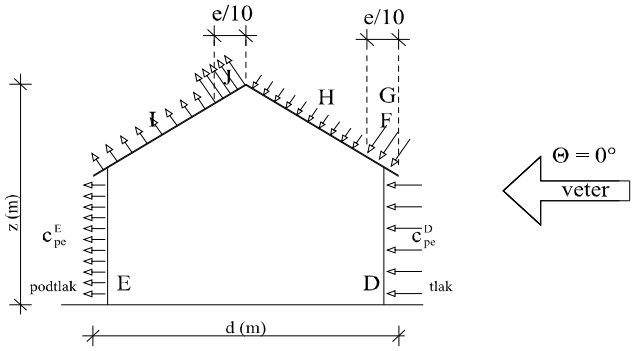
$c_{pe,p,D}$ = -0,24 . . . desna stran

$$w_{prav,e,D} = q_p(z_e) \cdot c_{pe,p,D} =$$ **-0,09** kN/m^2 . . . srk strehe

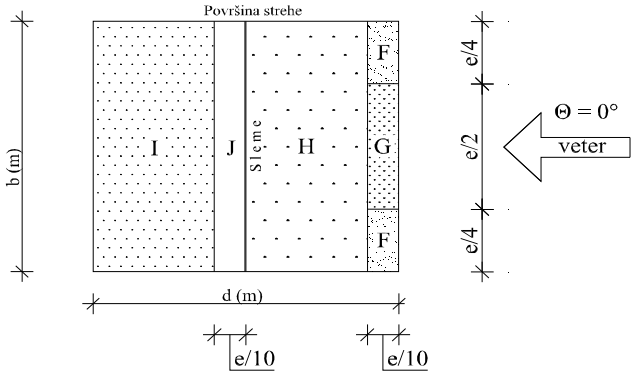
INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Pritisk vetra na STREHO / pravokotno na sleme θ = 0° z DESNE strani

a) Prečni prerez



b) Tloris strehe



Zunanji pritisk

$$e = \min\begin{cases} d = \\ b = \\ 2 \cdot h = \end{cases}$$

d =	7,60	m	e =	14,00
b =	16,00	m	e/2 =	7,00
2 · h =	14,00	m	e/4 =	3,50
			e/10 =	1,40

$c_{pe,10}$ za dani naklon strehe

F	G	H	I	J
0,7	**0,7**	**0,6**	**-0,2**	**-0,3**

Vrednosti c_{pe} lahko poenostavimo, s tem ko predvidimo kontinuiran vpliv po dolžini strešine.

$$c_{pe,p,L} \cdot d/2 = c_{pe,J} \cdot e/10 + c_{pe,I} \cdot (d/2 - e/10)$$

$c_{pe,p,L}$ = -0,24 . . . leva stran

$w_{prav,e,L} = q_p(z_e) \cdot c_{pe,p,L}$ = **-0,09** kN/m² . . . srk strehe

$$c_{pe,p,D} \cdot d/2 = c_{pe,F} \cdot e/10 + c_{pe,H} \cdot (d/2 - e/10)$$

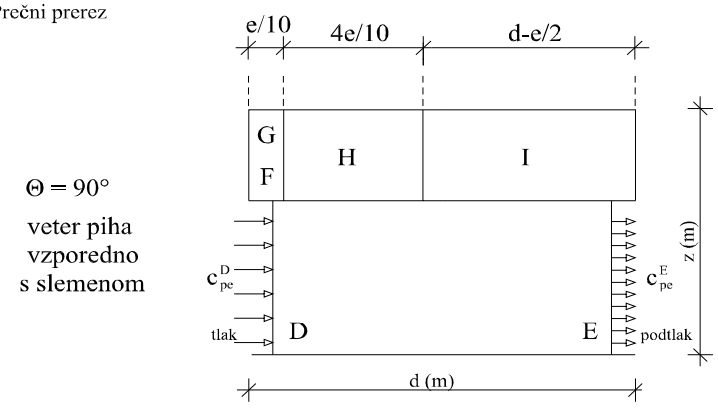
$c_{pe,p,D}$ = 0,64 . . . desna stran

$w_{prav,e,D} = q_p(z_e) \cdot c_{pe,p,D}$ = **0,24** kN/m² . . . tlak na streho

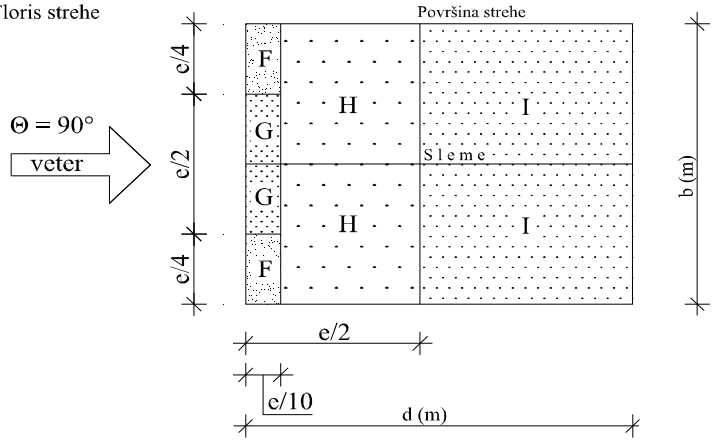
INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Pritisk vetra na STREHO / vzporedno s slemenom θ = 90°

a) Prečni prerez



b) Tloris strehe



Zunanji pritisk

$$e = \min \begin{cases} d = & 16{,}00 & \text{m} \\ b = & 7{,}60 & \text{m} \\ 2 \cdot h = & 14 & \text{m} \end{cases}$$

e =	7,60
e/2 =	3,80
e/4 =	1,90
e/10 =	0,76

$c_{pe,10}$ za dani naklon strehe

F	G	H	I
-1,1	**-1,4**	**-0,9**	**-0,5**

Vrednosti c_{pe} lahko poenostavimo, s tem ko predvidimo kontinuiran vpliv po dolžini strešine.

$$c_{pe,p,L} \cdot b/2 = c_{pe,F} \cdot e/4 + c_{pe,G} \cdot e/4$$

$c_{pe,p,L} =$ -1,25 . . . leva stran

$w_{vzp,e,L} = q_p(z_e) \cdot c_{pe,p,L} =$ **-0,46** kN/m^2 . . . srk strehe

$c_{pe,p,D} = c_{pe,p,L}$

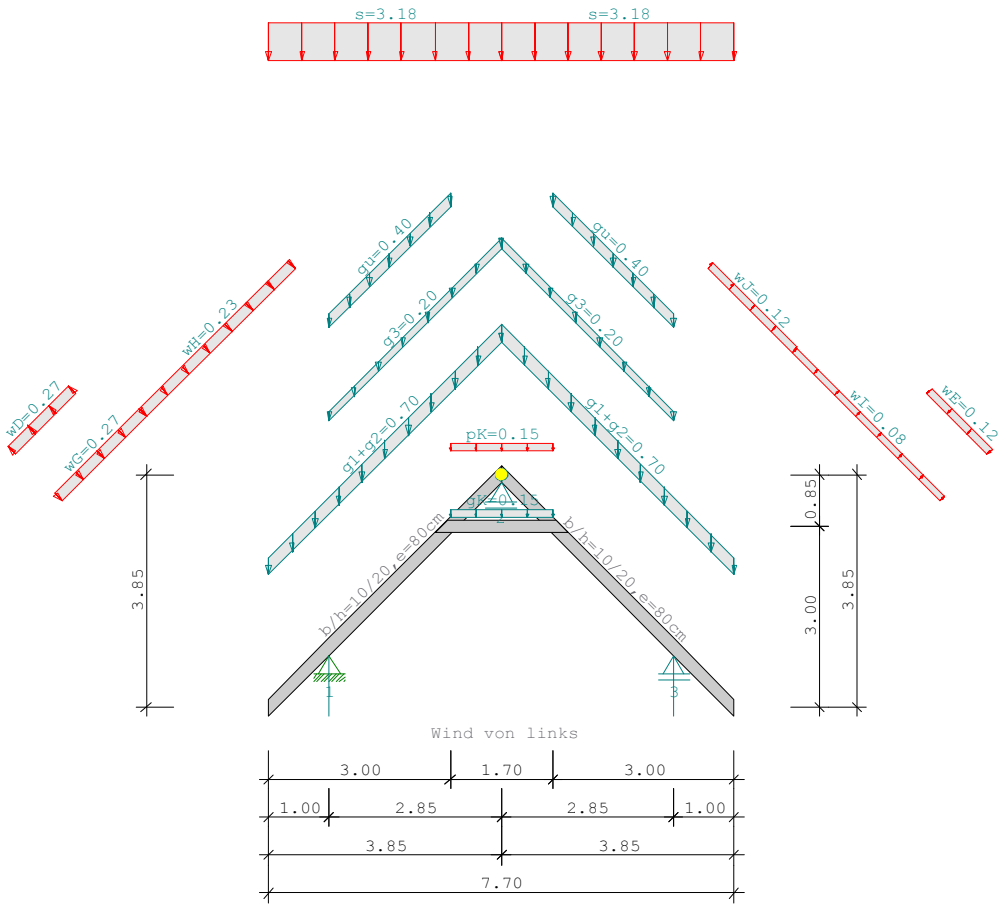
$c_{pe,p,D} =$ -1,25 . . . desna stran

$w_{vzp,e,D} = w_{vzp,e,L}$ **-0,46** kN/m^2 . . . srk strehe

INŽENIRING BIRO
ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

1. Item: 100A_streha 45 spere in skarje C24_10/20/90cm+enostr. škarje 6/20cm

Collar Beam Roof D12 02/2020/H (FRILO R-2025-2/P06)
Material Softwood C24 (EN 338:2016)
class 1



SYSTEM collar beam roof, displacable
b.ar. = base area , r.ar. = area of roof

rafter span	length b.ar.	length r.ar. (m)			
ca li	1.00	1.41	left	45.0 degree	10/20
ea.ex	1.00	1.41			
1	2.00	2.83	left	45.0 degree	10/20
2	0.85	1.20	left	45.0 degree	10/20
3	0.85	1.20	right	45.0 degree	10/20
4	2.00	2.83	right	45.0 degree	10/20
ca ri	1.00	1.41	right	45.0 degree	10/20
ea.ex	1.00	1.41			
with hinge at ridge					

collar beam span	length b.ac.	Ho (m)	
1	1.70	0.85	6/20

definitions of supports of rafter No	Cx[kN/cm]	Cz[kN/cm]	tv[cm]
1	-1	-1	3.0
2	0	-1	3.0
3	0	-1	3.0

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

LOADS

RAFTER
collar beam

roofing	g1	=	0.55	kN/m2 r.ac.	Act	99	
construction	g2	=	0.15	kN/m2 r.ac.			
roof space convers. g3		=	0.20	kN/m2 r.ac.			
roof space convers. gu		=	0.40	kN/m2 r.ac.			
man load rafter	P	=	1.00	kN	ÖNorm B 1991-1-1:2017-02	Act	8
snow loads acc ÖNorm B 1991-1-3:2018-12-01							
wind load acc.to ÖNorm B 1991-1-4:2019							
height of building	h	=	7.00	m			
width of wind attack b		=	16.00	m			
ordin.snow load	sk	=	3.97	kN/m2 b.ac.	Act	10	
with snowcatch on leftside, with snowcatch on rigthside							
snow load left	si	=	3.18	kN/m2 (μ=0.80)			
snow load eave le.	Se	=	0.00	kN/m *			
snow load right	si	=	3.18	kN/m2 (μ=0.80)			
snow load eave ri.	Se	=	0.00	kN/m *			
wind imp.press.	q	=	0.39	kN/m2	Act	9	
aerodyn. zones according to ÖNorm B 1991-1-4:2019							
aerodyn. zones referring eaves							
			wind from left				
undergrate blast	wD	=	-0.27	kN/m2			
wind load	wG	=	0.27	kN/m2			
wind load	wH	=	0.23	kN/m2			
wind load	wJ	=	-0.12	kN/m2			
wind load	wI	=	-0.08	kN/m2			
undergrate blast	wE	=	0.12	kN/m2			
			wind from right				
undergrate blast	wD	=	-0.27	kN/m2			
wind load	wG	=	0.27	kN/m2			
wind load	wH	=	0.23	kN/m2			
wind load	wJ	=	-0.12	kN/m2			
wind load	wI	=	-0.08	kN/m2			
undergrate blast	wE	=	0.12	kN/m2			
	e/10	=	1.40	m			
	e(90)/4	=	1.93	m			
- under grate blast only in area of excess							
- roof space conversion from house-edge to ridge.							
* = user setting, otherwise acc code							
dead loads	gk	=	0.15	kN/m2	Act	99	
live loads	pk	=	0.15	kN/m2	Act	1	

CLASSIFICATION OF USED ACTIONS

acc ÖNorm B 1990-1:2004-05
consequence class CC2, k_{FI}= 1.0

no.	load cases	γ_{sup}	γ_{inf}	ψ_0	ψ_1	ψ_2	ldc
99: g	Ständige Lasten	1.35	1.00				ständig
10: S0A	Schnee bis NN +1..	1.50	0.00	0.50	0.20	0.00	kurz
9: WIL	Windlasten	1.50	0.00	0.60	0.20	0.00	kurz
110: WIR	Wind v.re.	1.50	0.00	0.60	0.20	0.00	kurz
1: NLA	Wohnräume	1.50	0.00	0.70	0.50	0.30	mittel

decisive COMBINATIONS

for ultimate limit states
persistent, transient situation

K1	1,35*g+1,35*gk	(kmod = 0.60)
K5	1,35*g+1,35*gk+1,5*wli	(kmod = 0.90)
K6	1,35*g+1,35*gk+1,5*wre	(kmod = 0.90)
K9	1,35*g+1,35*gk+1,5*pk,li+1,5*pk,re	(kmod = 0.80)
K10	1,35*g+1,35*gk+1,5*sli+1,5*sre+1,5*0,6*wli	(kmod = 0.90)
K12	1,35*g+1,35*gk+1,5*sli+1,5*sre+1,5*0,6*wre	(kmod = 0.90)
K31	1,35*g+1,35*gk+1,5*sli+1,5*sre+1,5*0,6*wli+1,5*0,7*pk,re (kmod = 0.90)	
K34	1,35*g+1,35*gk+1,5*sli+1,5*sre+1,5*0,6*wli+1,5*0,7*pk,li+ 1,5*0,7*pk,re	(kmod = 0.90)

decisive COMBINATIONS

K37 1,35*g+1,35*gk+1,5*sli+1,5*sre+1,5*0,6*wre+1,5*0,7*pk,li
(kmod = 0.90)
K43 1,35*g+1,35*gk+1,5*sli+1,5*sre+1,5*0,6*wre+1,5*0,7*pk,li+
1,5*0,7*pk,re (kmod = 0.90)
K46 1,35*g+1,35*gk+1,5*sli+0,5*1,5*sre+1,5*0,6*wli+1,5*0,7*pk,li
(kmod = 0.90)
K49 1,35*g+1,35*gk+0,5*1,5*sli+1,5*sre+1,5*0,6*wre+1,5*0,7*pk,re
(kmod = 0.90)
for serviceability limit states
characteristic (rare) situation
K79 1*g+1*gk+1*sli+1*sre+1*0,6*wli+1*0,7*pk,li (kmod = 0.90)
K91 1*g+1*gk+1*sli+1*sre+1*0,6*wre+1*0,7*pk,re (kmod = 0.90)
K97 1*g+1*gk+1*sli+0,5*sre+1*0,6*wli+1*0,7*pk,li (kmod = 0.90)
K100 1*g+1*gk+0,5*sli+1*sre+1*0,6*wre+1*0,7*pk,re (kmod = 0.90)
quasi-permanent situation
K109 1*g+1*gk+1*0,3*pk,li (kmod = 0.80)
K110 1*g+1*gk+1*0,3*pk,re (kmod = 0.80)
K111 1*g+1*gk+1*0,3*pk,li+1*0,3*pk,re (kmod = 0.80)

legend:
g = dead load, s = snow load, sA = snow bag, Se = snow at eave,
w = wind load, gk = dead load at collar beam, pk = live load CB
~le = left, ~ri = right, ~gb = gable sided, ~(A) = accidental
Fm[Nr] = man's load at beam [Nr]

BUCKLING LENGTH
rafter left
buckling in plane: aus Eigenwert aber max. 0.90*length of beam
buckling out of plane: kontin. gehalten
lateral torsional buck..kontin. gehalten
rafter right
buckling in plane: aus Eigenwert aber max. 0.90*length of beam
buckling out of plane: kontin. gehalten
lateral torsional buck..kontin. gehalten
collar beam
buckling in plane: Stablänge
buckling out of plane: kontin. gehalten
lateral torsional buck..kontin. gehalten

				in case of fire		
bar	sky[m]	skz[m]	sB[m]	sky[m]	skz[m]	sB[m]
1		0.00	0.00	4.03	1.41	1.41
2		0.00	0.00	4.03	2.83	2.83
3		0.00	0.00	4.03	1.20	1.20
4		0.00	0.00	4.03	1.20	1.20
5		0.00	0.00	4.03	2.83	2.83
6		0.00	0.00	4.03	1.41	1.41
7	1.70	0.00	0.00	1.70	1.70	1.70

calculated with BemHo (9.0.4.18)
RAFTER (le) 10 / 20 e = 80 cm
C24 , service class 1 , $\gamma_{M,PT}$ = 1.3 , values in [N/mm2]
$E_{0,mean}$= 11000 $E_{0,05}$ = 7400 G_{mean} = 690 G_{05} = 460
$f_{m,y,k}$ = 24.00 $f_{v,k}$ = 2.30 $f_{c,0,k}$ = 21.00 $f_{t,0,k}$ = 14.50
k_{cr} = 1.00

ultimate limit states acc ÖNORM B 1995-1-1:2019-06, design-values [N/mm2]
based on EN 1995-1-1/A1:2008
Proofs for persistent and transient situation

		$\sigma_{myd,bez}$		f_{myd}	η
K37	PT stress (span)	3.08	<	16.62	0.19
	Nachweis für Querschnitt 10/20				
K37	PT stress (column)	5.46	<	16.62	0.33
	Nachweis für Querschnitt 10/20				
K43	PT stability	4.07	<	16.62	0.24

		τ_d		f_{vd}	η
K46	PT shear stress	0.45	<	1.59	0.29

serviceability limit state acc.to ÖNORM B 1995-1-1:2019-06, deflection [cm]
based on EN 1995-1-1/A1:2008

			w_{exists}		$w_{perm.}$	L/..	η
K109	w_{net}	local	0.05	<	1.13	250	0.04
		total	0.06	<	1.61	250	0.03
K97	w_{inst}	local	0.11	<	0.94	300	0.12
		total	0.20	<	1.34	300	0.15
K97	w_{max}	local	0.13				
		total	0.22				
deflection at cantilever							
K110	w_{net}	total	0.00	<	1.13	125	0.00
K91	w_{inst}	total	0.23	<	0.94	150	0.25
K91	w_{max}	total	0.23				

partially deflections in [cm]

combination		dead load $w_{G,inst}$	dead load $w_{G,fin}$	charact. situation $w_{Q,inst}$	charact. situation $w_{Q,fin}$	permanent situation $w_{Q,inst}$	permanent situation $w_{Q,fin}$
K109	loc	0.03	0.05	0.00	0.00	0.00	0.00
	tot	0.03	0.05	0.00	0.00	0.00	0.00
K97	loc	0.03	0.05	0.09	0.09	0.00	0.00
	tot	0.03	0.05	0.17	0.17	0.00	0.00

RAFTER (ri) **10 / 20** **e = 80 cm**
C24 , service class 1 , $\gamma_{M,PT}$ = 1.3 , values in [N/mm2]
$E_{0,mean}$= 11000 $E_{0,05}$ = 7400 G_{mean} = 690 G_{05} = 460
$f_{m,y,k}$ = 24.00 $f_{v,k}$ = 2.30 $f_{c,0,k}$ = 21.00 $f_{t,0,k}$ = 14.50
k_{cr} = 1.00

ultimate limit states acc ÖNORM B 1995-1-1:2019-06, design-values [N/mm2]
based on EN 1995-1-1/A1:2008
Proofs for persistent and transient situation

		$\sigma_{myd,bez}$		f_{myd}	η
K46	PT stress (span)	3.12	<	16.62	0.19
K31	PT stress (column)	5.46	<	16.62	0.33
	Nachweis für Querschnitt 10/20				
K34	PT stability	4.02	<	16.62	0.24

		τ_d		f_{vd}	η
K49	PT shear stress	0.45	<	1.59	0.29

serviceability limit state acc.to ÖNORM B 1995-1-1:2019-06, deflection [cm]
based on EN 1995-1-1/A1:2008

			w_{exists}		$w_{perm.}$	L/..	η
K110	w_{net}	local	0.05	<	1.13	250	0.04
		total	0.05	<	1.61	250	0.03
K100	w_{inst}	local	0.11	<	0.94	300	0.12
		total	0.19	<	1.34	300	0.14
K100	w_{max}	local	0.13				
		total	0.21				
deflection at cantilever							
K109	w_{net}	total	0.00	<	1.13	125	0.00
K79	w_{inst}	total	0.23	<	0.94	150	0.24
K79	w_{max}	total	0.23				

partially deflections in [cm]

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

combination		dead load $w_{G,inst}$	$w_{G,fin}$	charact. situation $w_{Q,inst}$	$w_{Q,fin}$	permanent situation $w_{Q,inst}$	$w_{Q,fin}$
K110	loc	0.03	0.05	0.00	0.00	0.00	0.00
	tot	0.03	0.05	0.00	0.00	0.00	0.00
K100	loc	0.03	0.05	0.09	0.09	0.00	0.00
	tot	0.03	0.05	0.16	0.16	0.00	0.00

COLLAR BEAM 6 / 20 e = 80 cm
C24 , service class 1 , $\gamma_{M,PT}$ = 1.3 , values in [N/mm2]
$E_{0,mean}$= 11000 $E_{0,05}$ = 7400 G_{mean} = 690 G_{05} = 460
$f_{m,y,k}$ = 24.00 $f_{v,k}$ = 2.30 $f_{c,0,k}$ = 21.00 $f_{t,0,k}$ = 14.50
k_{cr} = 1.00

ultimate limit states acc ÖNORM B 1995-1-1:2019-06, design-values [N/mm2]
based on EN 1995-1-1/A1:2008
Proofs for persistent and transient situation

		$\sigma_{myd,bez}$		f_{myd}	η
K43	PT stress (span)	0.88	<	16.62	0.05
K43	PT stress (column)	0.88	<	16.62	0.05
K43	PT stability	1.19	<	16.62	0.07

		τ_d		f_{vd}	η
K9	PT shear stress	0.04	<	1.42	0.03

serviceability limit state acc.to ÖNORM B 1995-1-1:2019-06, deflection [cm]
based on EN 1995-1-1/A1:2008

			w_{exists}		$w_{perm.}$	L/..	η
K111	w_{net}	local	0.01	<	0.68	250	0.01
		total	0.01	<	0.68	250	0.02
K97	w_{inst}	local	0.00	<	0.57	300	0.00
		total	0.10	<	0.57	300	0.17
K97	w_{max}	local	0.00				
		total	0.10				

partially deflections in [cm]

combination		dead load $w_{G,inst}$	$w_{G,fin}$	charact. situation $w_{Q,inst}$	$w_{Q,fin}$	permanent situation $w_{Q,inst}$	$w_{Q,fin}$
K111	loc	0.00	0.00	0.00	0.00	0.00	0.00
	tot	0.01	0.01	0.00	0.00	0.00	0.00
K97	loc	0.00	0.00	0.00	0.00	0.00	0.00
	tot	0.00	0.01	0.09	0.09	0.00	0.00

support reactions [kN/m], charakteristic values

act		COLUMN 1 max	min	COLUMN 2 max	min	COLUMN 3 max	min
g	V	2.84	2.84	6.07	6.07	2.84	2.84
	H	0.00	0.00	0.00	0.00	0.00	0.00
S0A	V	7.56	-0.70	10.75	5.38	7.56	-0.70
	H	0.00	0.00	0.00	0.00	0.00	0.00
WIL	V	-0.30	-0.30	0.98	0.98	-0.23	-0.23
	H	0.92	0.92	0.00	0.00	0.00	0.00
WIR	V	0.68	0.68	-0.86	-0.86	0.62	0.62
	H	-0.92	-0.92	0.00	0.00	0.00	0.00
NLA	V	0.01	-0.01	0.25	0.13	0.01	-0.01
	H	0.00	0.00	0.00	0.00	0.00	0.00

MAX/MIN SUPPORT REACTIONS design values [kN/m]

INŽENIRING BIRO
ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

in persistent and transient situation					
support	V_{max}	H_{rel}	combi	V_{rel}	H_{max} combi
1	14.74	-0.83	K37	3.39	1.38 K5
2	25.47	0.00	K34	8.20	0.00 K1
3	15.21	0.00	K49	3.84	0.00 K1

min. support reactions are not valid for the proof against upload

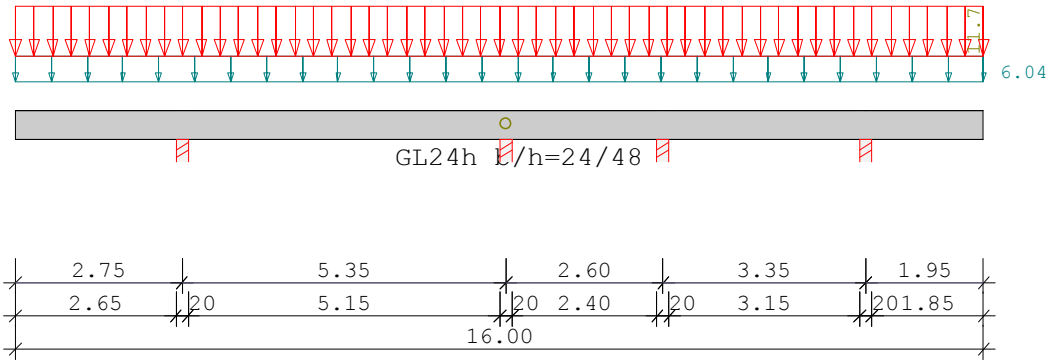
persistent and transient situation					
support	V_{min}	H_{rel}	combi	V_{rel}	H_{min} combi
1	3.39	1.38	K5	4.86	-1.38 K6
2	6.91	0.00	K6	8.20	0.00 K1
3	3.49	0.00	K5	3.84	0.00 K1

INŽENIRING BIRO
ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

2. Item: 101_slemenska REI_30 GL24_24/48cm glej clenek v osi 2

Continuous Beam DLT10 02/2022/D (FRILO R-2025-2/P06)

Scale 1 : 125



Timber girder over 3 Spans GL24h

System	length		cross-section values		
Span	L (m)		b (cm)	h (cm)	Iy (cm4)
1	5.35	constant	24.0	48.0	221184.0
2	2.60	constant	24.0	48.0	221184.0
3	3.35	constant	24.0	48.0	221184.0
Cantilever					
left	2.75	constant	24.0	48.0	221184.0
right	1.95	constant	24.0	48.0	221184.0

Hinges :in span 2 at x = 0.00 m

Beam-related loads (kN,m)

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

Type	EG	Gr	VK	g_l/r	q_l/r	fac.	dist. Lb/Lc	fromItem	Phi
1	J			6.04	11.69	1.00		100a	

Dead load of girder is considered with Gamma = 5.0 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ	KLED
J	3	Snow loads <1000m	0.50	0.20	0.00	1.50	short

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally a reference to the main action.

Results for 1-times loads

Span moments maximum (kNm , kN)

Span				Mf	M le	M ri	V le	V ri	comb
1	x0	=	2.93	53.58	-25.02	0.00	53.64	-44.29	4
2	x0	=	1.25	14.39	0.00	-2.19	22.96	-24.64	5
3	x0	=	1.69	12.71	-13.36	-12.58	30.90	-30.43	6

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Support moments maximum							(kNm , kN)
Column	M le	M ri	V le	V ri	max F	min F	comb
1	-69.22	-69.22	-50.34	61.91	112.25	40.57	2
2	0.00	0.00	-44.29	22.96	67.25	8.22	5
3	-17.68	-17.68	-30.60	32.18	62.78	7.84	8
4	-34.80	-34.80	-38.93	35.70	74.63	25.22	7

Support reactions						(kN)
Column	by g	max q	min q	Fulload	max	min
1	40.57	71.68	0.00	112.25	112.25	40.57
2	20.04	47.21	-11.81	55.44	67.25	8.22
3	18.75	44.03	-10.91	51.87	62.78	7.84
4	26.51	48.12	-1.29	73.34	74.63	25.22
Total:	105.86	211.05	-24.01	292.90	316.91	81.85

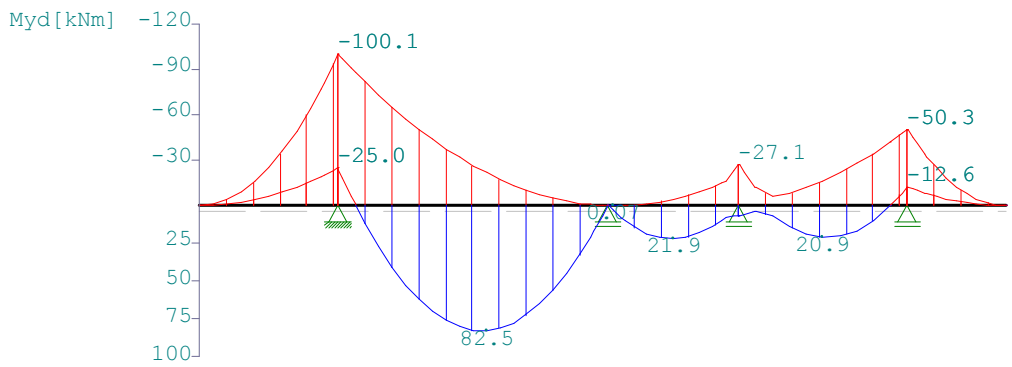
Support reactions								(kN)
	Column 1		Column 2		Column 3		Column 4	
CA	max	min	max	min	max	min	max	min
g	40.6	40.6	20.0	20.0	18.7	18.7	26.5	26.5
J	71.7	0.0	47.2	-11.8	44.0	-10.9	48.1	-1.3
tot	112.2	40.6	67.2	8.2	62.8	7.8	74.6	25.2

Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 spanwise constant

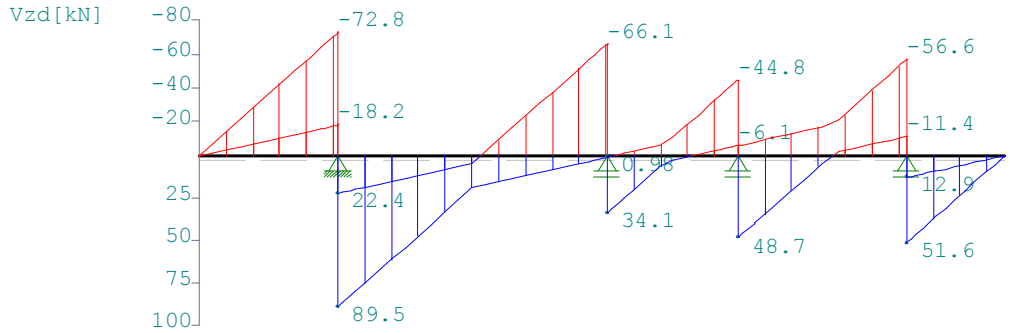
Span moments maximum								(kNm , kN)
Span			Mfd	Mdle	Mdri	V le	V ri	comb
1	x0 =	2.85	82.60	-25.02	0.00	75.47	-66.12	J 4
2	x0 =	1.29	21.95	0.00	-0.83	34.09	-34.73	J 5
3	x0 =	1.76	21.02	-19.81	-12.58	46.49	-42.17	J 6

Support moments maximum							(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb
1	-100.08	-100.08	-72.78	89.50	162.29	40.57	J 2
2	0.00	0.00	-66.12	34.09	100.21	-0.02	J 5
3	-27.14	-27.14	-44.84	48.68	93.52	0.22	J 8
4	-50.32	-50.32	-56.61	51.61	108.22	24.32	J 7

Scale 1 : 150

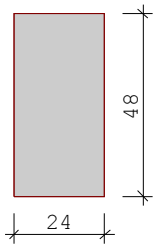


INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR



Design : ÖNORM B 1995-1-1:2019-06 GL24h
based on EN 1995-1-1/A2:2014

service class 1 kdef = 0.60 γM = 1.25 γM(A) = 1.00



Emean = 1150 kN/cm2 Gmean = 65 kN/cm2
fm,k,My = 24.0 N/mm2 fm,k,Mz = 24.0 N/mm2
fv,k,Vz = 2.5 N/mm2 fv,k,Vy = 2.5 N/mm2

Tensions are calculated with FLBemHo901. (version9.0.4.18)
Axial stresses b/h = 24/48

Compression flange is supported continuously.

Span No.	x (m)	My,d (kNm)	σd,t (N/mm2)	σd,b	kcrit	kmod	σd/fm,d	comb
CaLe	0.00	0.00	0.00	0.00	1.00	0.00	0.00	1
	2.75	-100.08	10.86	-10.86	1.00	0.90	0.61	J 2
1	0.00	-100.08	10.86	-10.86	1.00	0.90	0.61	J 2
	2.85	82.60	-8.96	8.96	1.00	0.90	0.51	J 4
	5.35	0.00	0.00	0.00	1.00	0.90	0.00	J 3
2	0.00	0.00	0.00	0.00	1.00	0.00	0.00	1
	1.29	21.95	-2.38	2.38	1.00	0.90	0.13	J 5
	2.60	-27.14	2.94	-2.94	1.00	0.90	0.17	J 8
3	0.00	-27.14	2.94	-2.94	1.00	0.90	0.17	J 8
	1.76	21.02	-2.28	2.28	1.00	0.90	0.13	J 6
	1.76	21.02	-2.28	2.28	1.00	0.90	0.13	J 6
	3.35	-50.32	5.46	-5.46	1.00	0.90	0.31	J 5
CaRi	0.00	-50.32	5.46	-5.46	1.00	0.90	0.31	J 5
	1.95	0.00	0.00	0.00	1.00	0.90	0.00	J 8

the factor kh =1.02 according EN 1995 3.3 (3) is considered.

Shear stresses b/h = 24/48

Column No.	x (m)	Vz,d (kN)	τD (N/mm2)	kmod	τd/fv,d	comb
1 le	0.580	-57.43	0.75	0.90	0.41	J 2
ri	0.580	74.15	0.97	0.90	0.52	J 3
2 le	0.580	-50.77	0.66	0.90	0.36	J 4
ri	0.580	18.74	0.24	0.90	0.13	J 5
3 le	0.580	-29.49	0.38	0.90	0.21	J 8
ri	0.580	33.33	0.43	0.90	0.24	J 8
4 le	0.580	-41.26	0.54	0.90	0.29	J 10
ri	0.580	36.26	0.47	0.90	0.26	J 10

EN 1995 6.1.7 : kcr = 1.00

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

support fc,90,k = 2.50 N/mm2

Column No.	b (cm)	d (cm)	max F (kN)	kmod	kc90	σc,90,d (N/mm2)	fc,90,d	η	comb
1	20.0	24.0	162.3	0.90	2.00	3.38	1.80	0.94	J 3
2	20.0	24.0	100.2	0.90	2.00	2.09	1.80	0.58	J 11
3	20.0	24.0	93.5	0.90	2.00	1.95	1.80	0.54	J 8
4	20.0	24.0	108.2	0.90	2.00	2.25	1.80	0.63	J 10

Analysis serviceability according to ÖNORM B 1995-1-1:2019-06 (2.2.3 , 7.2)
perm. w_{inst} < L/300 perm. w_{net} < L/250

Span	x1 (mm)		wgB (mm	wqB	w	all w)	η	
CaLe	0	inst:	2.1	11.8	13.9	18.3	0.76	2
		fin:	3.4	11.8	15.2			2
		net:	3.4	0.0	3.4	22.0	0.15	2
1	2675	inst:	1.1	4.9	6.0	17.8	0.33	4
		fin:	1.7	4.9	6.6			4
		net:	1.7	0.0	1.7	21.4	0.08	4
2	1300	inst:	0.1	0.3	0.4	8.7	0.05	5
		fin:	0.1	0.3	0.4			5
		net:	0.1	0.0	0.1	10.4	0.01	5
3	2010	inst:	-0.1	-0.6	-0.7	11.2	0.06	5
		fin:	-0.1	-0.6	-0.7			5
		net:	-0.1	0.0	-0.1	13.4	0.01	5
CaRi	1950	inst:	0.9	2.7	3.6	13.0	0.28	5
		fin:	1.5	2.7	4.1			5
		net:	1.5	0.0	1.5	15.6	0.10	5

proof of oscillations span 1 ÖNorm B 1995-1-1:2015-06 / Hamm

The vibration proof is performed only for the permanent load portions.
(National annex to ÖNorm EN 1995-1-1 7.3.3 (2) - (5)).

selected floor class = I
proof of natural freque ncy m = 662 kg/m2
beam: 24/48 EI =25.436 MNm2
Beam spacing e = 100.0 cm
fact.f.effect of continuity kf = 1.29
f0 = 13.85 Hz
f1 = 13.85 Hz

speed of vibration due unit impuls v <= b^(f1*Ksi-1)
modal coeff. of damping Ksi = 0.010
coeffizient b =150.00
p erm. v =0.0133 m/(Ns^2)
γ = 0.95
v =0.0006 m/(Ns^2) Eta = 0.04

Frequency criterion:
Concerning floor class I : f_{gr} = 8.00 Hz
f1 = 13.85 Hz Eta = 0.58

Stiffness criterion:
deflection due concentr. load F
Concerning floor class I : w_{gr} = 0.25 mm/kN
w_{stat} = 0.13 mm/kN Eta = 0.50

The vibration proof is fulfilled.

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

No.	span	Type	Grp		g1	q1	g2	q2	factor	distance	length
2	1	4	J	2	6.04	11.69	6.04	11.69	1.00	0.00	5.35
3	2	4	J	3	6.04	11.69	6.04	11.69	1.00	0.00	2.60
4	3	4	J	4	6.04	11.69	6.04	11.69	1.00	0.00	3.35
Cantilever											
1	CaLe	4	J	1	6.04	11.69	6.04	11.69	1.00	0.00	2.75
5	CaRi	4	J	5	6.04	11.69	6.04	11.69	1.00	0.00	1.95

Calculated combinations from 5 Loads

lc	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12
	g	g	g	g	g	g	g	g	g	g	g	g
1	.	x	x	.	.	.	.	.	.	.	.	x
2	.	.	x	x	.	.	.	.	.	.	x	.
3	.	.	.	.	x	.	.	x	x	.	x	.
4	.	.	.	.	.	x	.	x	.	x	.	x
5	.	.	.	.	x	.	x	.	.	x	x	.

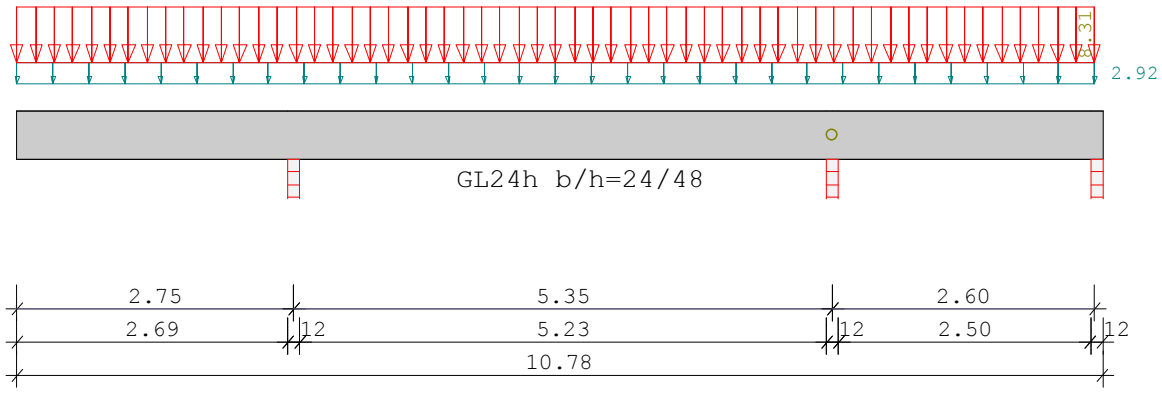
The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
one by one alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

3. Item: 102 kapne lege kot slemenska GL24_24/48cm glej členek os 2

Continuous Beam DLT10 02/2022/D (FRILO R-2025-2/P06)

Scale 1 : 75



Timber girder over 2 Spans GL24h

System	length		cross-section values		
Span	L (m)		b (cm)	h (cm)	Iy (cm4)
1	5.35	constant	24.0	48.0	221184.0
2	2.60	constant	24.0	48.0	221184.0
Cantilever left	2.75	constant	24.0	48.0	221184.0

Hinges :in span 2 at x = 0.00 m

Beam-related loads (kN,m)

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

Type	EG	Gr	VK	g_l/r	q_l/r	fac.	dist. Lb/Lc	fromItem	Phi
1	H			2.92	8.31	1.00		100a	

Dead load of girder is considered with Gamma = 5.0 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ	KLED
H	2	Roof (f.Ex. man load)	0.00	0.00	0.00	1.50	short

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally a reference to the main action.

Results for 1-times loads

Span moments maximum (kNm , kN)

Span			Mf	M le	M ri	V le	V ri	comb
1	x0 =	2.88	35.89	-13.22	0.00	34.05	-29.11	4
2	x0 =	1.30	9.98	0.00	0.00	15.35	-15.35	5

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Support moments maximum							(kNm , kN)
Column	M le	M ri	V le	V ri	max F	min F	comb
1	-44.64	-44.64	-32.47	39.93	72.39	21.44	2
2	0.00	0.00	-29.11	15.35	44.46	5.55	5
3	0.00	0.00	-15.35	0.00	15.35	4.54	5

Support reactions						(kN)
Column	by g	max q	min q	Fulload	max	min
1	21.44	50.96	0.00	72.39	72.39	21.44
2	11.43	33.03	-5.87	38.58	44.46	5.55
3	4.54	10.80	0.00	15.35	15.35	4.54
Total:	37.41	94.79	-5.87	126.32	132.20	31.53

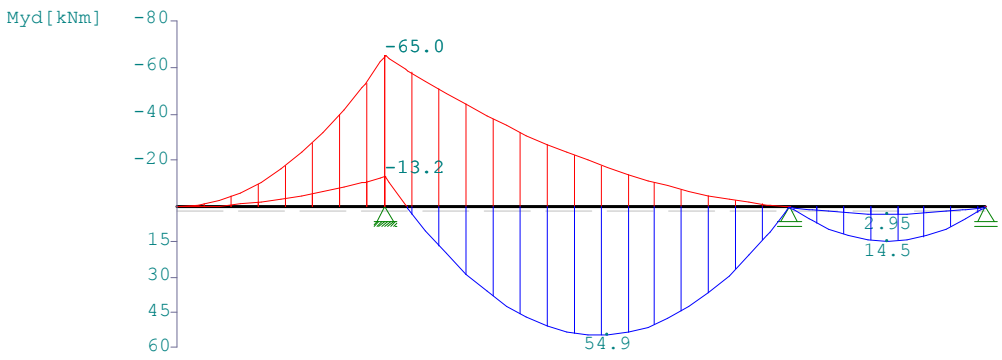
Support reactions							(kN)
	Column 1		Column 2		Column 3		
CA	max	min	max	min	max	min	
g	21.4	21.4	11.4	11.4	4.5	4.5	
H	51.0	0.0	33.0	-5.9	10.8	0.0	
tot	72.4	21.4	44.5	5.6	15.3	4.5	

Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 spanwise constant

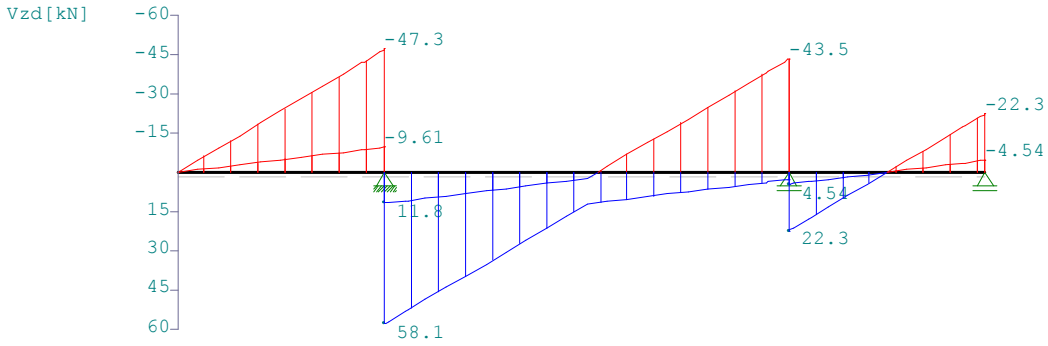
Span moments maximum								(kNm , kN)
Span			Mfd	Mdle	Mdri	V le	V ri	comb
1	x0 =	2.82	55.05	-13.22	0.00	48.44	-43.50	H 4
2	x0 =	1.30	14.52	0.00	0.00	22.34	-22.34	H 5

Support moments maximum							(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb
1	-64.98	-64.98	-47.26	58.11	105.37	21.44	H 2
2	0.00	0.00	-43.50	22.34	65.84	1.75	H 5
3	0.00	0.00	-22.34	0.00	22.34	4.54	H 5

Scale 1 : 100

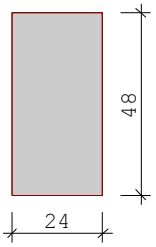


INŽENIRING BIRO
ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR



Design : ÖNORM B 1995-1-1:2019-06 GL24h
based on EN 1995-1-1/A2:2014

service class 2 kdef = 0.80 γM = 1.25 γM(A) = 1.00



Emean = 1150 kN/cm2 Gmean = 65 kN/cm2
fm,k,My = 24.0 N/mm2 fm,k,Mz = 24.0 N/mm2
fv,k,Vz = 2.5 N/mm2 fv,k,Vy = 2.5 N/mm2

Tensions are calculated with FLBemHo901. (version9.0.4.18)
Axial stresses b/h = 24/48

Compression flange is supported continuously.

Span No.	x (m)	My,d (kNm)	σd,t (N/mm2)	σd,b	kcrit	kmod	σd/fm,d		comb
CaLe	0.00	0.00	0.00	0.00	1.00	0.00	0.00		1
	2.75	-64.98	7.05	-7.05	1.00	0.90	0.40	H	2
1	0.00	-64.98	7.05	-7.05	1.00	0.90	0.40	H	2
	2.82	55.05	-5.97	5.97	1.00	0.90	0.34	H	4
	2.82	55.05	-5.97	5.97	1.00	0.90	0.34	H	4
	5.35	0.00	0.00	0.00	1.00	0.90	0.00	H	3
2	0.00	0.00	0.00	0.00	1.00	0.00	0.00		1
	1.30	14.52	-1.58	1.58	1.00	0.90	0.09	H	5
	2.60	0.00	0.00	0.00	1.00	0.90	0.00	H	5

the factor kh =1.02 according EN 1995 3.3 (3) is considered.

Shear stresses b/h = 24/48

Column No.		x (m)	Vz,d (kN)	τD (N/mm2)	kmod	τd/fv,d		comb
1	le	0.540	-37.98	0.49	0.90	0.27	H	2
	ri	0.540	48.83	0.64	0.90	0.35	H	3
2	le	0.540	-34.22	0.45	0.90	0.24	H	4
	ri	0.540	13.06	0.17	0.90	0.09	H	5
3	le	0.520	-13.40	0.17	0.90	0.09	H	5

EN 1995 6.1.7 : kcr = 1.00

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

support fc,90,k = 2.50 N/mm2

Column No.	b (cm)	d (cm)	max F (kN)	kmod	kc90	σc,90,d (N/mm2)	fc,90,d	η	comb
1	12.0	20.0	105.4	0.90	2.50	4.39	1.80	0.98	H 3
2	12.0	20.0	65.8	0.90	2.50	2.74	1.80	0.61	H 6
3	12.0	20.0	22.3	0.90	2.50	0.93	1.80	0.21	H 5

Analysis serviceability according to ÖNORM B 1995-1-1:2019-06 (2.2.3 , 7.2)
perm. w_{inst} < L/300 perm. w_{net} < L/250

Span	x1 (mm)		wgB (mm	wqB	w	all w)	η	
CaLe	0	inst:	1.1	8.4	9.5	18.3	0.52	2
		fin:	2.0	8.4	10.4			2
		net:	2.0	0.0	2.0	22.0	0.09	2
1	2675	inst:	0.6	3.5	4.0	17.8	0.23	4
		fin:	1.0	3.5	4.5			4
		net:	1.0	0.0	1.0	21.4	0.05	4
2	1300	inst:	0.1	0.2	0.3	8.7	0.03	5
		fin:	0.1	0.2	0.3			5
		net:	0.1	0.0	0.1	10.4	0.01	5

proof of oscillations span 1 ÖNorm B 1995-1-1:2015-06 / Hamm

The vibration proof is performed only for the permanent load portions.
(National annex to ÖNorm EN 1995-1-1 7.3.3 (2) - (5)).

selected floor class = I
proof of natural freque ncy m = 350 kg/m2
beam: 24/48 EI =25.436 MNm2
Beam spacing e = 100.0 cm
fact.f.effect of continuity kf = 1.29
f0 = 19.05 Hz
f1 = 19.05 Hz

speed of vibration due unit impuls v <= b^(f1*Ksi-1)
modal coeff. of damping Ksi = 0.010
coeffizient b =150.00
p erm. v =0.0173 m/(Ns^2)
γ = 0.95
v =0.0011 m/(Ns^2) Eta = 0.06

Frequency criterion:
Concerning floor class I : f_{gr} = 8.00 Hz
f1 = 19.05 Hz Eta = 0.42

Stiffness criterion:
deflection due concentr. load F
Concerning floor class I : w_{gr} = 0.25 mm/kN
w_{stat} = 0.13 mm/kN Eta = 0.50

The vibration proof is fulfilled.

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

No.	span	Type	Grp	g1	q1	g2	q2	factor	distance	length
2	1	4	H 2	2.92	8.31	2.92	8.31	1.00	0.00	5.35
3	2	4	H 3	2.92	8.31	2.92	8.31	1.00	0.00	2.60
Cantilever										
1	CaLe	4	H 1	2.92	8.31	2.92	8.31	1.00	0.00	2.75

Calculated combinations from 3 Loads

lc	K1	K2	K3	K4	K5	K6
	g	g	g	g	g	g
1	.	x	x	.	.	.
2	.	.	x	x	.	x
3	.	.	.	.	x	x

The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
one by one alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

4. Item: 101A_steber (pod 101) v steni ali kot soha t. A GL24_20/20cm

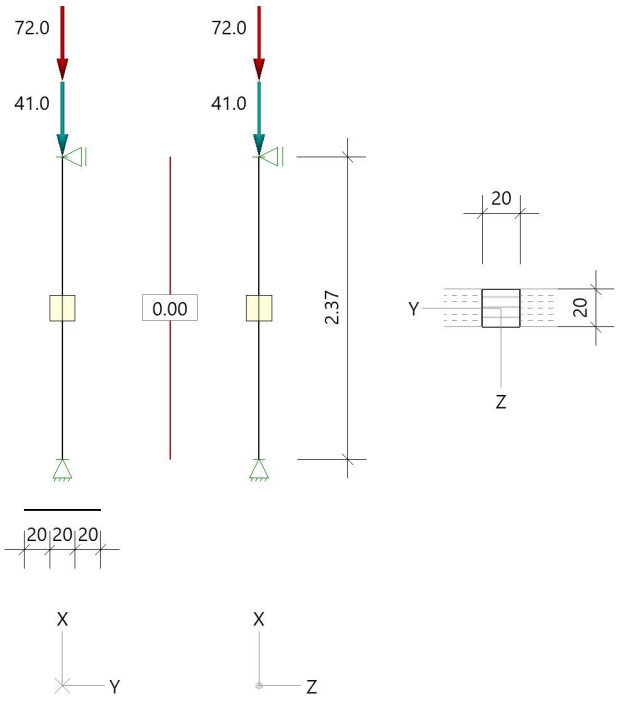
Timber Column (x64) HO1+ 02/2025 (FRILO R-2025-2/P06)

System

Design code

Design	ÖNORM B 1995-1-1:2015-06
Based on	EN 1995-1-1/A2:2014
Combinatorial Analysis	ÖNORM B 1990-1:2013-01-01
Consequence Class	CC 2

System image 2D



System

Simply supported, H=2.37m, b/h=20.0/20.0cm
GL24c, SC 2, γ=5.00 kN/m³, EN 14080:2013 Lamellae/layers parallel b(y)

Foot Binder

No.	b [cm]	h [cm]	a le [cm]	a ri [cm]	Material Name	Ser.Cl. SC
1	20.0	80.0	20.0	20.0	2 GL24c	2
Lamellae/layers parallel h						

Loads

List of Loads

No.	Type	ACT	Q1	a1	Q2	L2	fac	grp	About
1	2 X	99	0.5	2.37			1.00		AUTO_G_Mat
2	2 X	99	41.0	2.37			1.00		
3	2 X	10	72.0	2.37			1.00		
4	10 Z	9	0.00				1.00		

Type: 2 = Concentrated load: Q1[kN] at a1[m]; a1 = Distance from base point
10 = Uniformly distributed load: Q1[kN/m]
ACT: 99=Permanent loads; 9=Wind loads; 10=Snow loads H < 1000 m

Combinations

Special rules

Relieving constant loads	Yes

Design / Analysis

Buckling a. tilting lengths

Calculation of Buckling Lengths
Flexural buckling(E) s_k = π2*E*I/(etaNki*Nx)
Lateral torsion buckling(S) s_b = System lengths

(E) Eigenvalue , See check for lengths because load depending.
(S) Calculation using spacing of the rigid bearing

Design Results Decisive Combination (Brief ouput)

Co 4: Load capacity, permanent/transient

Cross-secti.	N,M,V [kN,kNm]	Coefficients		σ,τ [N/mm²]	f...d [N/mm²]	η
Cross-section Check x=2.37m b/h=20.0/20.0cm						
Nx N,M	-164.0	K_{mod}=0.90	γ_M=1.25	-4.10	15.48	0.26 **0.26**
Stability Check x=2.37m b/h=20.0/20.0cm						
Nx N,M	-164.0	K_{mod}=0.90 $k_{c,y}$=0.95	γ_M=1.25 $k_{c,z}$=0.95	-4.10	15.48	0.28 **0.28**

Co4: 1.35*G1+1.50*S2
Lamella direction || b
Buckling length: s_{ky}=2.37m s_{kz}=2.37m Tilting length: s_b=2.37m
Slenderness: λ_y=41.0 λ_z=41.0 $\lambda_{rel,c,y}$=0.63 $\lambda_{rel,c,z}$=0.63 $\lambda_{rel,m,y}$=0.24 $\lambda_{rel,m,z}$=0.23
Share N(g)/N(g+q) = 34%; ψ2(Lc, σ_{max})= 0.00; K_{def}= 0.80
Strengths modified according to EN 1995-1-1 para.3.3 + NA/NCI 3.3

Co 4: Load capacity, permanent/transient

Foot Binder: Comp.p.Grain

Cr	$F_{c,90,d}$ [kN]	l_{ef} [cm]	b_{ef} [cm]	A_{ef} [cm²]	$k_{c,90}$	K_{mod}	γ_M	$\sigma_{c,90,d}$ [N/mm²]	$f_{c,90,d}$ [N/mm²]	η
1	-164.0	26.0	20.0	520.0	2.00	0.90	1.25	-3.15	-1.80	**0.88**

Co4: 1.35*G1+1.50*S2
Pressure without consideration of eccentricities
User-defined value $k_{c,90}$

Decisive Deformations

w=0 => η=0

5. Item: 101C_steber (pod 101) v steni ali kot soha t. C GL24_20/20cm

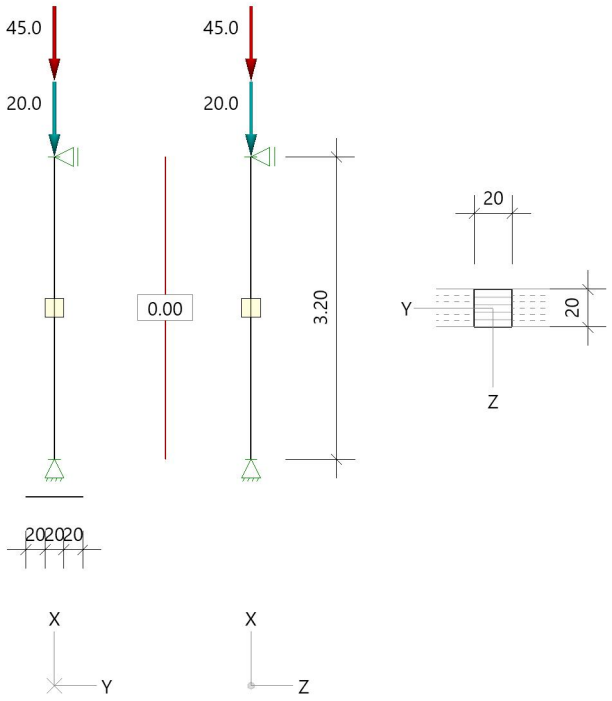
Timber Column (x64) HO1+ 02/2025 (FRILO R-2025-2/P06)

System

Design code

Design	ÖNORM B 1995-1-1:2015-06
Based on	EN 1995-1-1/A2:2014
Combinatorial Analysis	ÖNORM B 1990-1:2013-01-01
Consequence Class	CC 2

System image 2D



System

Simply supported, H=3.20m, b/h=20.0/20.0cm
GL24c, SC 2, γ=5.00 kN/m³, EN 14080:2013 Lamellae/layers parallel b(y)

Foot Binder

No.	b [cm]	h [cm]	a le [cm]	a ri [cm]	Material Name	Ser.Cl. SC
1	20.0	80.0	20.0	20.0	2 GL24c	2
Lamellae/layers parallel h						

Loads

List of Loads

No.	Type	ACT	Q1	a1	Q2	L2	fac	grp	About
1	2 X	99	0.6	3.20			1.00		AUTO_G_Mat
2	2 X	99	20.0	3.20			1.00		
3	2 X	10	45.0	3.20			1.00		
4	10 Z	9	0.00				1.00		

Type: 2 = Concentrated load: Q1[kN] at a1[m]; a1 = Distance from base point
10 = Uniformly distributed load: Q1[kN/m]
ACT: 99=Permanent loads; 9=Wind loads; 10=Snow loads H < 1000 m

Combinations

Special rules

Relieving constant loads	Yes

Design / Analysis

Buckling a. tilting lengths

Calculation of Buckling Lengths
Flexural buckling(E) s_k = π2*E*I/(etaNki*Nx)
Lateral torsion buckling(S) s_b = System lengths

(E) Eigenvalue , See check for lengths because load depending.
(S) Calculation using spacing of the rigid bearing

Design Results Decisive Combination (Brief ouput)

Co 4: Load capacity, permanent/transient

Cross-secti.	N,M,V [kN,kNm]	Coefficients		σ,τ [N/mm²]	f...d [N/mm²]	η
Cross-section Check x=3.20m b/h=20.0/20.0cm						
Nx N,M	-95.4	K_{mod}=0.90	γ_M=1.25	-2.38	15.48	0.15 **0.15**
Stability Check x=3.20m b/h=20.0/20.0cm						
Nx N,M	-95.4	K_{mod}=0.90 $k_{c,y}$=0.87	γ_M=1.25 $k_{c,z}$=0.87	-2.38	15.48	0.18 **0.18**

Co4: 1.35*G1+1.50*S2
Lamella direction || b
Buckling length: s_{ky}=3.20m s_{kz}=3.20m Tilting length: s_b=3.20m
Slenderness: λ_y=55.4 λ_z=55.4 $\lambda_{rel,c,y}$=0.86 $\lambda_{rel,c,z}$=0.86 $\lambda_{rel,m,y}$=0.28 $\lambda_{rel,m,z}$=0.27
Share N(g)/N(g+q) = 29%; ψ2(Lc, σ_{max})= 0.00; K_{def}= 0.80
Strengths modified according to EN 1995-1-1 para.3.3 + NA/NCI 3.3

Co 4: Load capacity, permanent/transient

Foot Binder: Comp.p.Grain

Cr	$F_{c,90,d}$ [kN]	l_{ef} [cm]	b_{ef} [cm]	A_{ef} [cm²]	$k_{c,90}$	K_{mod}	γ_M	$\sigma_{c,90,d}$ [N/mm²]	$f_{c,90,d}$ [N/mm²]	η
1	-95.4	26.0	20.0	520.0	2.00	0.90	1.25	-1.83	-1.80	**0.51**

Co4: 1.35*G1+1.50*S2
Pressure without consideration of eccentricities
User-defined value $k_{c,90}$

Decisive Deformations

w=0 => η=0

6. Item: 101D_steber (pod 101) v zun. steni v t. D GL24_20/20cm

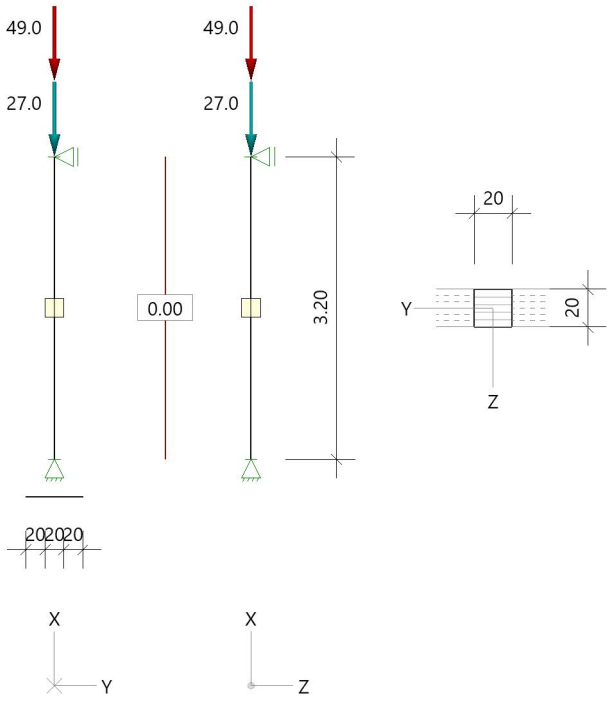
Timber Column (x64) HO1+ 02/2025 (FRILO R-2025-2/P06)

System

Design code

Design	ÖNORM B 1995-1-1:2015-06
Based on	EN 1995-1-1/A2:2014
Combinatorial Analysis	ÖNORM B 1990-1:2013-01-01
Consequence Class	CC 2

System image 2D



System

Simply supported, H=3.20m, b/h=20.0/20.0cm
GL24c, SC 2, γ=5.00 kN/m³, EN 14080:2013 Lamellae/layers parallel b(y)

Foot Binder

No.	b [cm]	h [cm]	a le [cm]	a ri [cm]	Material Name	Ser.Cl. SC
1	20.0	80.0	20.0	20.0	2 GL24c	2
Lamellae/layers parallel h						

Loads

List of Loads

No.	Type	ACT	Q1	a1	Q2	L2	fac	grp	About
1	2 X	99	0.6	3.20			1.00		AUTO_G_Mat
2	2 X	99	27.0	3.20			1.00		
3	2 X	10	49.0	3.20			1.00		
4	10 Z	9	0.00				1.00		

Type: 2 = Concentrated load: Q1[kN] at a1[m]; a1 = Distance from base point
10 = Uniformly distributed load: Q1[kN/m]
ACT: 99=Permanent loads; 9=Wind loads; 10=Snow loads H < 1000 m

Combinations

Special rules

Relieving constant loads	Yes

Design / Analysis

Buckling a. tilting lengths

Calculation of Buckling Lengths
Flexural buckling(E) s_k = π2*E*I/(etaNki*Nx)
Lateral torsion buckling(S) s_b = System lengths

(E) Eigenvalue , See check for lengths because load depending.
(S) Calculation using spacing of the rigid bearing

Design Results Decisive Combination (Brief ouput)

Co 4: Load capacity, permanent/transient

Cross-secti.	N,M,V [kN,kNm]	Coefficients		σ,τ [N/mm²]	f...d [N/mm²]	η
Cross-section Check x=3.20m b/h=20.0/20.0cm						
Nx N,M	-110.8	K_{mod}=0.90	γ_M=1.25	-2.77	15.48	0.18 **0.18**
Stability Check x=3.20m b/h=20.0/20.0cm						
Nx N,M	-110.8	K_{mod}=0.90 $k_{c,y}$=0.87	γ_M=1.25 $k_{c,z}$=0.87	-2.77	15.48	0.21 **0.21**

Co4: 1.35*G1+1.50*S2
Lamella direction || b
Buckling length: s_{ky}=3.20m s_{kz}=3.20m Tilting length: s_b=3.20m
Slenderness: λ_y=55.4 λ_z=55.4 $\lambda_{rel,c,y}$=0.86 $\lambda_{rel,c,z}$=0.86 $\lambda_{rel,m,y}$=0.28 $\lambda_{rel,m,z}$=0.27
Share N(g)/N(g+q) = 34%; ψ2(Lc, σ_{max})= 0.00; K_{def}= 0.80
Strengths modified according to EN 1995-1-1 para.3.3 + NA/NCI 3.3

Co 4: Load capacity, permanent/transient

Foot Binder: Comp.p.Grain

Cr	$F_{c,90,d}$ [kN]	l_{ef} [cm]	b_{ef} [cm]	A_{ef} [cm²]	$k_{c,90}$	K_{mod}	γ_M	$\sigma_{c,90,d}$ [N/mm²]	$f_{c,90,d}$ [N/mm²]	η
1	-110.8	26.0	20.0	520.0	2.00	0.90	1.25	-2.13	-1.80	**0.59**

Co4: 1.35*G1+1.50*S2
Pressure without consideration of eccentricities
User-defined value $k_{c,90}$

Decisive Deformations

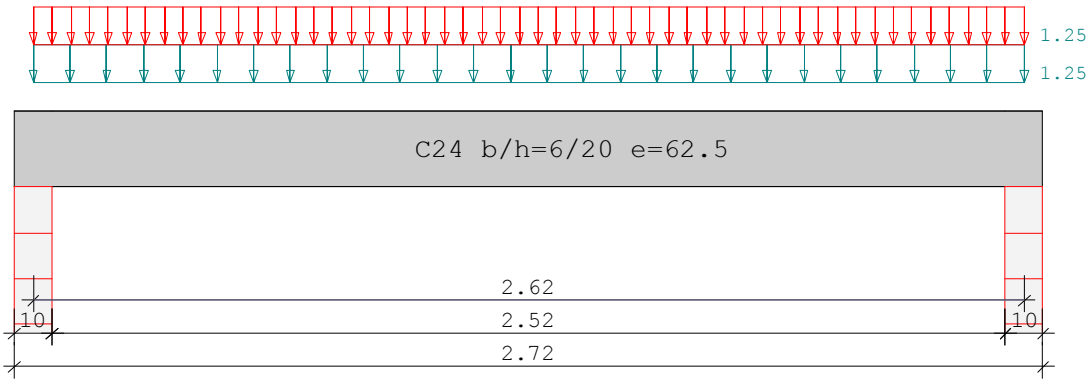
w=0 => η=0

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

7. Item: 200A_ strop P C24_6/20/62,5cm

Timber Beam HO7 02/2019/C (FRILO R-2025-2/P06)

Scale 1 : 20



area load: dead load g = 2.00 kN/m2
live load q = 2.00 kN/m2 Action A

Design : ÖNORM B 1995-1-1:2019-06 6/20 e = 62.5 cm
based on EN 1995-1-1/A2:2014

C24 service class 1 kdef = 0.60 γM = 1.30 γM(A) = 1.00
E_{mean} = 1100 kN/cm2 G_{mean} = 69 kN/cm2
$f_{m,k}$ = 24.0 N/mm2 $f_{v,k}$ = 2.3 N/mm2

proofs: 6.0 / 20.0 e = 62.5 cm kmod = 0.80

max Myd = 3.06 kNm σmd = -7.64 N/mm2 km = 1.00 η = 0.52
Compression flange is supported continuously.
Column 2le x = 0.25 m Vz,d = -3.78 kN τD = 0.47 N/mm2 η = 0.33

support pressure kc,90 = 2.00 kmod = 0.80
Support No. 2: 10.0/6.0
F = 4.7 kN σc,90,d = 0.78 N/mm2 fc,90,d = 1.54 N/mm2 η = 0.25

Analysis serviceability according to ÖNORM B 1995-1-1:2019-06
(2.2.3 , 7.2)
perm. w_{inst} < L/300 perm. w_{net} < L/250

Span	x1 (mm)		wgB (mm	wqB	w	all w)	η
1	1310	inst:	1.7	1.7	3.5	8.7	0.40
		fin:	2.8	2.1	4.8		
		net:	2.8	0.8	3.6	10.5	0.35

Support reactions (kN/m)

CA	Column 1 max	min	Column 2 max	min
g	2.6	2.6	2.6	2.6
A	2.6	0.0	2.6	0.0
tot	5.2	2.6	5.2	2.6

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

proof of oscillations span 1 ÖNorm B 1995-1-1:2015-06 / Hamm

The vibration proof is performed only for the permanent load portions.
(National annex to ÖNorm EN 1995-1-1 7.3.3 (2) - (5)).

selected floor class = I
proof of natural freque ncy m = 200 kg/m2
beam: 6/20 EI = 0.440 MNm2
Beam spacing e = 62.5 cm
additional stiffn. long it EI,l = 0.432 MNm2
total stiffness longit. EI,l = 0.872 MNm2
total stiffness across EI,q = 0.553 MNm2/m
α = 1.01
Breadth of ceiling pane l b1 = 2.10 m
f0 = 19.11 Hz
f1 = 26.76 Hz

speed of vibration due unit impuls v <= b^(f1*Ksi-1)
modal coeff. of damping Ksi = 0.030
coeffizient b =150.00
n40 = 1.065
p erm. v =0.3723 m/(Ns^2)
γ = 1.00
v =0.0032 m/(Ns^2) Eta = 0.01

Frequency criterion:
Concerning floor class I : f_{gr} = 8.00 Hz
f1 = 26.76 Hz Eta = 0.30

Stiffness criterion:
deflection due concentr. load F
Concerning floor class I : w_{gr} = 0.25 mm/kN
w_{stat} = 0.14 mm/kN Eta = 0.57

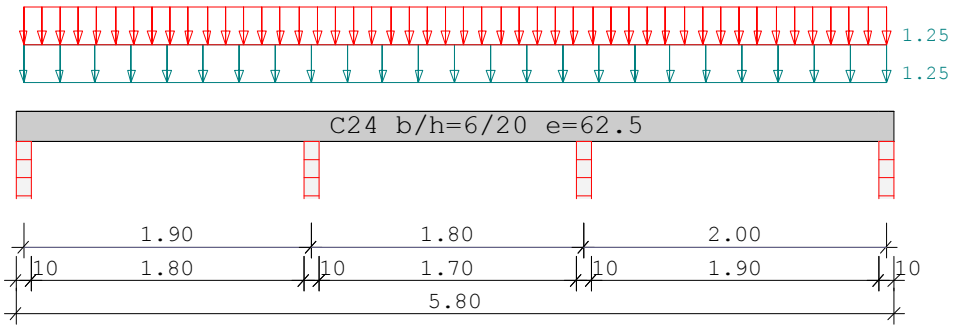
The vibration proof is fulfilled.

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

8. Item: 200B_ strop P C24_6/20/62,5cm

Timber Beam HO7 02/2019/C (FRILO R-2025-2/P06)

Scale 1 : 50



area load: dead load g = 2.00 kN/m2
live load q = 2.00 kN/m2 Action A

Design : ÖNORM B 1995-1-1:2019-06 6/20 e = 62.5 cm
based on EN 1995-1-1/A2:2014

C24 service class 1 kdef = 0.60 γM = 1.30 γM(A) = 1.00
E_{mean} = 1100 kN/cm2 G_{mean} = 69 kN/cm2
$f_{m,k}$ = 24.0 N/mm2 $f_{v,k}$ = 2.3 N/mm2

proofs: 6.0 / 20.0 e = 62.5 cm kmod = 0.80

max Myd = 1.32 kNm σmd = -3.31 N/mm2 km = 1.00 η = 0.22
min Myd = -1.47 kNm σmd = -3.68 N/mm2 km = 1.00 η = 0.25
Compression flange is supported continuously.
Column 3ri x = 0.25 m Vz,d = 3.41 kN τD = 0.43 N/mm2 η = 0.30

support pressure kc,90 = 2.00 kmod = 0.80
Support No. 3: 10.0/6.0
F = 8.0 kN σc,90,d = 1.33 N/mm2 fc,90,d = 1.54 N/mm2 η = 0.43

Analysis serviceability according to ÖNORM B 1995-1-1:2019-06 (2.2.3 , 7.2)
perm. w_{inst} < L/300 perm. w_{net} < L/250

Span	x1 (mm)		wgB (	wqB mm	w	all w)	η
1	950	inst:	0.3	0.4	0.6	6.3	0.10
		fin:	0.4	0.4	0.9		
		net:	0.4	0.2	0.6	7.6	0.08
2	900	inst:	0.0	-0.2	-0.3	6.0	0.04
		fin:	-0.1	-0.3	-0.3		
		net:	-0.1	-0.1	-0.2	7.2	0.02
3	1000	inst:	0.3	0.4	0.8	6.7	0.11
		fin:	0.5	0.5	1.0		
		net:	0.5	0.2	0.7	8.0	0.09

Support reactions (kN/m)

CA	Column 1 max	Column 1 min	Column 2 max	Column 2 min	Column 3 max	Column 3 min	Column 4 max	Column 4 min
g	1.5	1.5	4.0	4.0	4.2	4.2	1.6	1.6
A	1.7	-0.2	4.5	-0.5	4.6	-0.4	1.8	-0.2
tot	3.3	1.4	8.5	3.6	8.8	3.8	3.4	1.5

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

proof of oscillations span 3 ÖNorm B 1995-1-1:2015-06 / Hamm

The vibration proof is performed only for the permanent load portions.
(National annex to ÖNorm EN 1995-1-1 7.3.3 (2) - (5)).

selected floor class = I
proof of natural freque ncy m = 200 kg/m2
beam: 6/20 EI = 0.440 MNm2
Beam spacing e = 62.5 cm
additional stiffn. long it EI,l = 0.432 MNm2
total stiffness longit. EI,l = 0.872 MNm2
total stiffness across EI,q = 0.553 MNm2/m
α = 1.32
Breadth of ceiling pane l b1 = 2.10 m
fact.f.effect of continuity kf = 1.09
f0 = 35.75 Hz
f1 = 41.17 Hz

speed of vibration due unit impuls v <= b^(f1*Ksi-1)
modal coeff. of damping Ksi = 0.030
coeffizient b =150.00
n40 = 0.709
p erm. v =3.2476 m/(Ns^2)
γ = 1.40
v =0.0032 m/(Ns^2) Eta = 0.00

Frequency criterion:
Concerning floor class I : f_{gr} = 8.00 Hz
f1 = 41.17 Hz Eta = 0.19

Stiffness criterion:
deflection due concentr. load F
Concerning floor class I : w_{gr} = 0.25 mm/kN
w_{stat} = 0.08 mm/kN Eta = 0.33

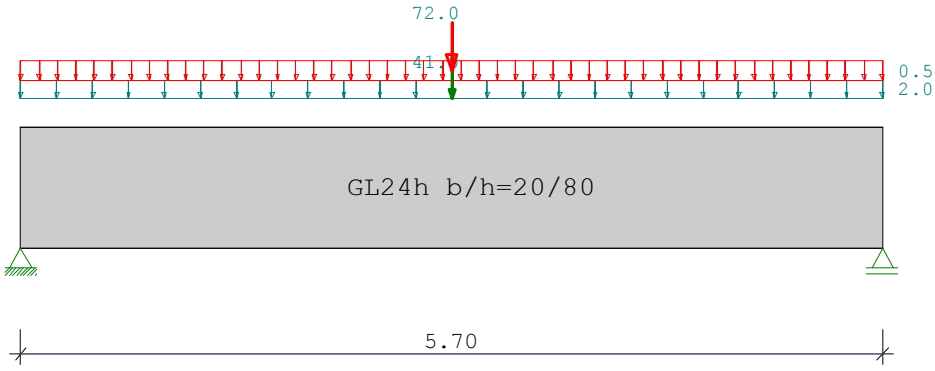
The vibration proof is fulfilled.

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

9. Item: 201 nosilec nad oknom v celni steni t.A GL24h_20/80cm

Continuous Beam DLT10 02/2022/D (FRILO R-2025-2/P06)

Scale 1 : 50



Timber girder GL24h

System	length		cross-section values		
Span	L (m)		b (cm)	h (cm)	Iy (cm4)
1	5.70	constant	20.0	80.0	853333.3

Beam-related loads (kN,m)

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

Type	EG	Gr	VK	g_l/r	q_l/r	fac.	dist. Lb/Lc	fromItem	Phi
1	A			2.00	0.50	1.00		ZS	
2	H		2.85	41.00	72.00	1.00	0.00	101tA	

Dead load of girder is considered with Gamma = 5.0 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ	KLED
A	1	Cat A - domestic	0.70	0.50	0.30	1.50	medium
H	2	Roof (f.Ex. man load)	0.00	0.00	0.00	1.50	short

All actions are considered as independent
Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3

Results for 1-times loads

Span moments maximum (kNm , kN)

Span			Mf	M le	M ri	V le	V ri
1	x0	= 2.85	174.43	0.00	0.00	65.91	-65.90

Support moments maximum (kNm , kN)

Column	M le	M ri	V le	V ri	max F	min F
1	0.00	0.00	0.00	65.90	65.91	28.48
2	0.00	0.00	-65.90	0.00	65.90	28.48

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Support reactions						(kN)
Column	by g	max q	min q	Fulload	max	min
1	28.48	37.43	0.00	65.91	65.91	28.48
2	28.48	37.43	0.00	65.90	65.90	28.48
Total:	56.96	74.85	0.00	131.81	131.81	56.96

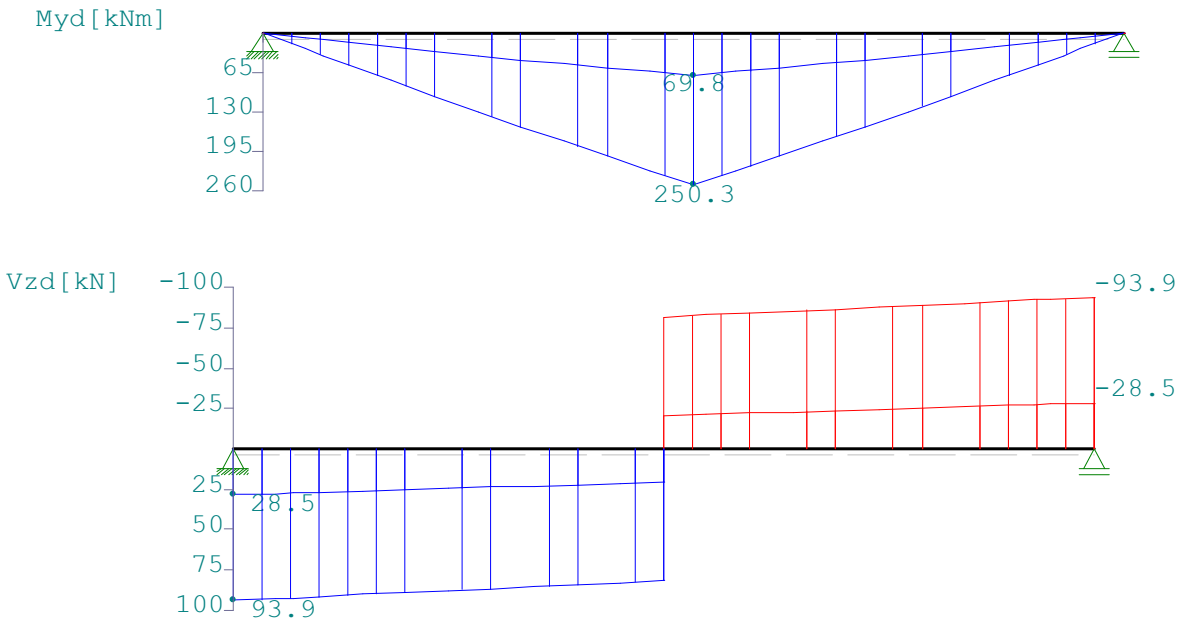
Support reactions					(kN)
	Column 1		Column 2		
CA	max	min	max	min	
g	28.5	28.5	28.5	28.5	
A	1.4	0.0	1.4	0.0	
H	36.0	0.0	36.0	0.0	
tot	65.9	28.5	65.9	28.5	

Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 spanwise constant

Span moments maximum						(kNm , kN)
Span			Mfd	Mdle	Mdri	V le	V ri
1	x0 =	2.85	250.26	0.00	0.00	93.94	-93.94

Support moments maximum						(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F
1	0.00	0.00	0.00	93.94	93.94	28.48
2	0.00	0.00	-93.94	0.00	93.94	28.48

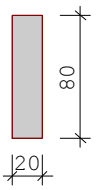
Scale 1 : 50



Design : ÖNORM B 1995-1-1:2019-06 GL24h
based on EN 1995-1-1/A2:2014

service class 1 kdef = 0.60 γM = 1.25 γM(A) = 1.00

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR



Emean = 1150 kN/cm2 Gmean = 65 kN/cm2
fm,k,My = 24.0 N/mm2 fm,k,Mz = 24.0 N/mm2
fv,k,Vz = 2.5 N/mm2 fv,k,Vy = 2.5 N/mm2

Tensions are calculated with FLBemHo901. (version9.0.4.18)
Axial stresses b/h = 20/80

Compression flange is supported continuously.

Span No.	x (m)	My,d (kNm)	σd,t (N/mm2)	σd,b	kcrit	kmod	σd/fm,d
1	0.00	0.00	0.00	0.00	1.00	0.00	0.00
	2.85	250.26	-11.73	11.73	1.00	0.90	0.68
	5.70	0.00	0.00	0.00	1.00	0.90	0.00

the factor kh according EN 1995 3.3 (3) is considered.

Shear stresses b/h = 20/80

Column No.	x (m)	Vz,d (kN)	τD (N/mm2)	kmod	τd/fv,d
1 ri	0.800	90.50	0.85	0.90	0.47
2 le	0.800	-90.50	0.85	0.90	0.47

EN 1995 6.1.7 : kcr = 1.00

Analysis serviceability according to ÖNORM B 1995-1-1:2019-06 (2.2.3 , 7.2)
perm. w_{inst} < L/300 perm. w_{net} < L/250

Span	x1 (mm)		wgB (mm	wqB	w	all w)	η
1	2849	inst:	2.0	2.9	4.9	19.0	0.26
		fin:	3.2	2.9	6.1		
		net:	3.2	0.0	3.2	22.8	0.14

proof of oscillations span 1 ÖNorm B 1995-1-1:2015-06 / Hamm

The vibration proof is performed only for the permanent load portions.
(National annex to ÖNorm EN 1995-1-1 7.3.3 (2) - (5)).

selected floor class = I
proof of natural freque ncy m = 999 kg/m2
beam: 20/80 EI =98.133 MNm2
Beam spacing e = 100.0 cm
additional stiffn. long it EI,l = 0.432 MNm2
total stiffness longit. EI,l =98.565 MNm2
total stiffness across EI,q = 0.553 MNm2/m
α = 3.21
Breadth of ceiling pane l b1 = 5.00 m
f0 = 15.18 Hz
f1 = 15.18 Hz

speed of vibration due unit impuls v <= b^(f1*Ksi-1)
modal coeff. of damping Ksi = 0.030
coeffizient b =150.00
n40 = 5.004
p erm. v =0.0653 m/(Ns^2)
γ = 1.00
v =0.0005 m/(Ns^2) Eta = 0.01

Frequency criterion:

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

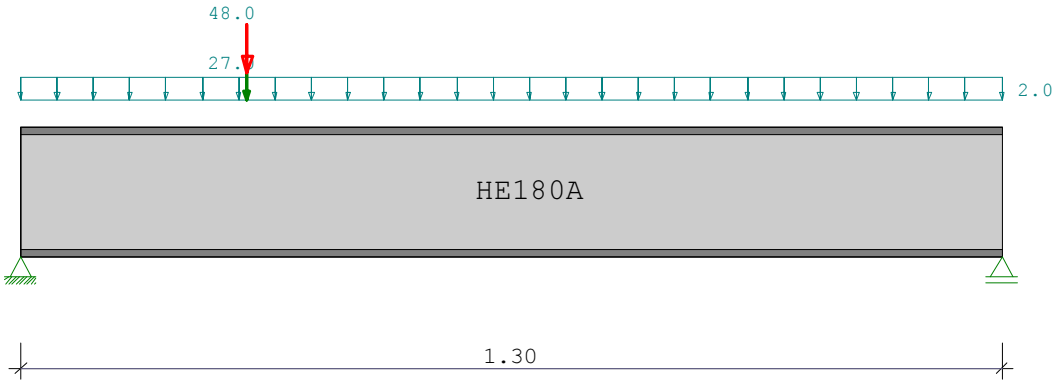
proof of oscillations span 1 ÖNorm B 1995-1-1:2015-06 / Hamm
Concerning floor class I : f_{gr} = 8.00 Hz f1 = 15.18 Hz Eta = 0.53 Stiffness criterion: deflection due concentr. load F Concerning floor class I : w_{gr} = 0.25 mm/kN w_{stat} = 0.03 mm/kN Eta = 0.11 The vibration proof is fulfilled.

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

10. Item: 202 nosilec nad vrati os 4- t.D S235 HEA 200

Continuous Beam DLT10 02/2022/D (FRILO R-2025-2/P06)

Scale 1 : 10



Steel girder S235 ÖNORM B 1993-1-1:2017-11
E-modulus E =210000 N/mm2

System	length		cross-section values				
Span	L (m)		CsNo.	I (cm4)	St (cm3)	Sb (cm3)	
1	1.300	constant	3	2510.0	294.0	294.0	HE180A

Beam-related loads (kN,m)

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

Type	EG	Gr	VK	g_l/r	q_l/r	fac.	dist. Lb/Lc	fromItem	Phi
1	A			2.000	0.000	1.000			ZS
2	H		0.300	27.000	48.000	1.000	0.000		101tD

Dead load of girder is considered with Gamma = 78.5 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
A	1	Cat A - domestic	0.70	0.50	0.30	1.50
H	2	Roof (f.Ex. man load)	0.00	0.00	0.00	1.50

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3

Results for 1-times loads

Span moments maximum (kNm , kN)

Span			Mf	M le	M ri	V le	V ri
1	x0	= 0.301	17.65	0.00	0.00	59.22	-18.84

Support moments maximum (kNm , kN)

Column	M le	M ri	V le	V ri	max F	min F
1	0.00	0.00	0.00	59.22	59.22	22.30
2	0.00	0.00	-18.84	0.00	18.84	7.76

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

Support reactions						(kN)
Column	by g	max q	min q	Fulload	max	min
1	22.30	36.92	0.00	59.22	59.22	22.30
2	7.76	11.08	0.00	18.84	18.84	7.76
Total:	30.06	48.00	0.00	78.06	78.06	30.06

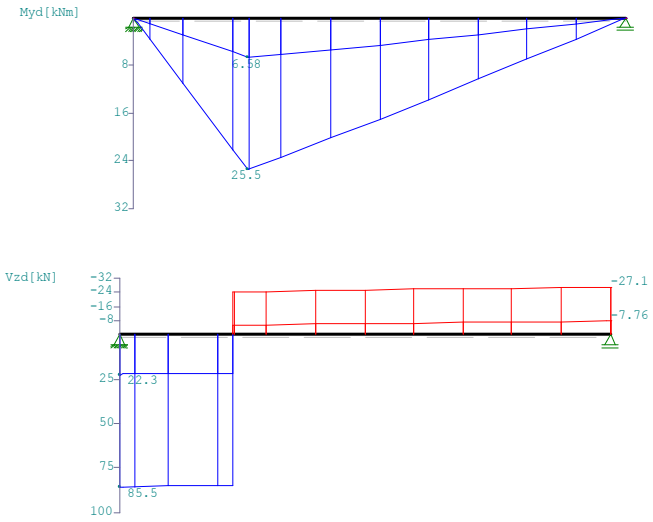
Support reactions					(kN)
	Column 1		Column 2		
CA	max	min	max	min	
g	22.3	22.3	7.8	7.8	
A	0.0	0.0	0.0	0.0	
H	36.9	0.0	11.1	0.0	
tot	59.2	22.3	18.8	7.8	

Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 constant over whole girder length

Span moments maximum						(kNm , kN)
Span		Mfd	Mdle	Mdri	V le	V ri
1	x0 = 0.301	25.49	0.00	0.00	85.49	-27.09

Support moments maximum						(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F
1	0.00	0.00	0.00	85.49	85.49	22.30
2	0.00	0.00	-27.09	0.00	27.09	7.76

Scale 1 : 20



Cross sectionsS235		fyk =	235 N/mm2			
type	name	Npl	Mplyd	Vplzd	Mplzd	Vplyd
21	Doppel-T	2891	265	427	93	1303
21	Doppel-T	3402	263	568	144	1628
3	HE180A	1065	77	197	37	464

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

proof acc. ÖNORM B 1993-1-1:2017-11 6.2.1 (6.1) γM0 = 1.00

Span No.	x (m)	csno.	My,ed (kNm)	Vz,ed (kN)	σv (N/mm2)	τ	CSCL	η
1	0.000	3	0.0	85.5	159	92	1	0.68
	0.299	3	25.5	84.5	159	84	1	0.68
	0.301	3	25.5	-23.9	88	7	1	0.37
	1.300	3	0.0	-27.1	50	29	1	0.21

proof acc. ÖNORM B 1993-1-1:2017-11 6.2.1 (6.2) γM0 = 1.00

Span No.	x (m)	My,ed (kNm)	Vz,ed (kN)	CSCL (-)	ρ (-)	M,Rd (kNm)	η
1	0.000	0.0	85.5	1	0.00	76.5	0.43
	0.299	25.5	84.5	1	0.00	76.5	0.43
	0.301	25.5	-23.9	1	0.00	76.5	0.33
	1.300	0.0	-27.1	1	0.00	76.5	0.14

Compression flange is supported continuously.
Proof of torsional-flexural buckling is not necessary.

Permissible deflection : in span perm.f = L / 300
characteristic combination

Span No.	x (m)	fg (cm)	ftot (cm)	f (cm)	perm.f (cm)	η	
1	0.520	0.02	0.04	0.044	0.433	0.10	2

11. Item: SP1_steber v P-vogalni steber kapne stene GL24_2x 20/20cm (pod 102)

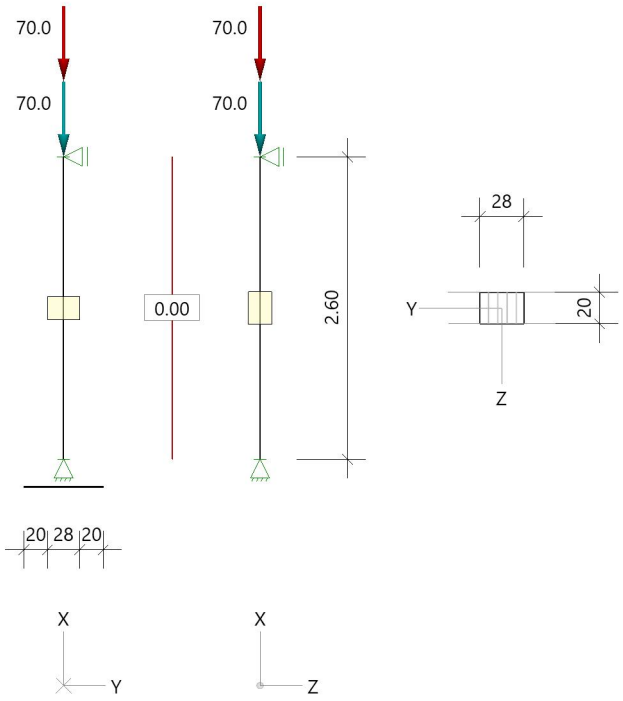
Timber Column (x64) HO1+ 02/2025 (FRILO R-2025-2/P06)

System

Design code

Design	ÖNORM B 1995-1-1:2015-06
Based on	EN 1995-1-1/A2:2014
Combinatorial Analysis	ÖNORM B 1990-1:2013-01-01
Consequence Class	CC 2

System image 2D



System

Simply supported, H=2.60m, b/h=28.0/20.0cm
GL24c, SC 2, γ=5.00 kN/m³, EN 14080:2013 Lamellae/layers parallel h(z)

Foot Binder

No.	b [cm]	h [cm]	a le [cm]	a ri [cm]	Material Name	Ser.Cl. SC
1	20.0	48.0	20.0	20.0	2 C24	2

Material Design code

Material	Design code
GL24c	acc.to ÖNORM B 1995-1-1:2015-06, based on EN 14080:2013
C24	acc.to ÖNORM B 1995-1-1:2015-06, based on EN 338:2016

Loads

List of Loads

No.	Type	ACT	Q1	a1	Q2	L2	fac	grp	About
1	2 X	99	0.7	2.60			1.00		AUTO_G_Mat
2	2 X	99	70.0	2.60			1.00		
3	2 X	10	70.0	2.60			1.00		
4	10 Z	9	0.00				1.00		

Type: 2 = Concentrated load: Q1[kN] at a1[m]; a1 = Distance from base point
10 = Uniformly distributed load: Q1[kN/m]
ACT: 99=Permanent loads; 9=Wind loads; 10=Snow loads H < 1000 m

Combinations

Special rules

Relieving constant loads	Yes

Design / Analysis

Buckling a. tilting lengths

Calculation of Buckling Lengths
Flexural buckling(E) s_k = π2*E*I/(etaNki*Nx)
Lateral torsion buckling(S) s_b = System lengths

(E) Eigenvalue , See check for lengths because load depending.
(S) Calculation using spacing of the rigid bearing

Design Results Decisive Combination (Brief ouput)

Co 4: Load capacity, permanent/transient

Cross-secti.	N,M,V [kN,kNm]	Coefficients		σ,τ [N/mm²]	f...d [N/mm²]	η
Cross-section Check x=2.60m b/h=28.0/20.0cm						
Nx	-200.5	K_{mod}=0.90	γ_M=1.25	-3.58	15.48	0.23
N,M						**0.23**
Stability Check x=2.60m b/h=28.0/20.0cm						
Nx	-200.5	K_{mod}=0.90 $k_{c,y}$=0.93	γ_M=1.25 $k_{c,z}$=0.97	-3.58	15.48	0.25
N,M						**0.25**

Co4: 1.35*G1+1.50*S2
Lamella direction || h
Buckling length: s_{ky}=2.60m s_{kz}=2.60m Tilting length: s_b=2.60m
Slenderness: λ_y=45.0 λ_z=32.2 $\lambda_{rel,c,y}$=0.70 $\lambda_{rel,c,z}$=0.50 $\lambda_{rel,m,y}$=0.19 $\lambda_{rel,m,z}$=0.28
Share N(g)/N(g+q) = 48%; ψ2(Lc, σ_{max})= 0.00; K_{def}= 0.80
Strengths modified according to EN 1995-1-1 para.3.3 + NA/NCI 3.3

Co 4: Load capacity, permanent/transient

Foot Binder: Comp.p.Grain

Cr	$F_{c,90,d}$ [kN]	l_{ef} [cm]	b_{ef} [cm]	A_{ef} [cm²]	$k_{c,90}$	K_{mod}	γ_M	$\sigma_{c,90,d}$ [N/mm²]	$f_{c,90,d}$ [N/mm²]	η
1	-200.5	34.0	20.0	680.0	2.00	0.90	1.30	-2.95	-1.73	**0.85**

Co4: 1.35*G1+1.50*S2
Pressure without consideration of eccentricities
User-defined value $k_{c,90}$

Decisive Deformations

w=0 => η=0

12. Item: SP2_steber v stenah P GL24_20/20cm

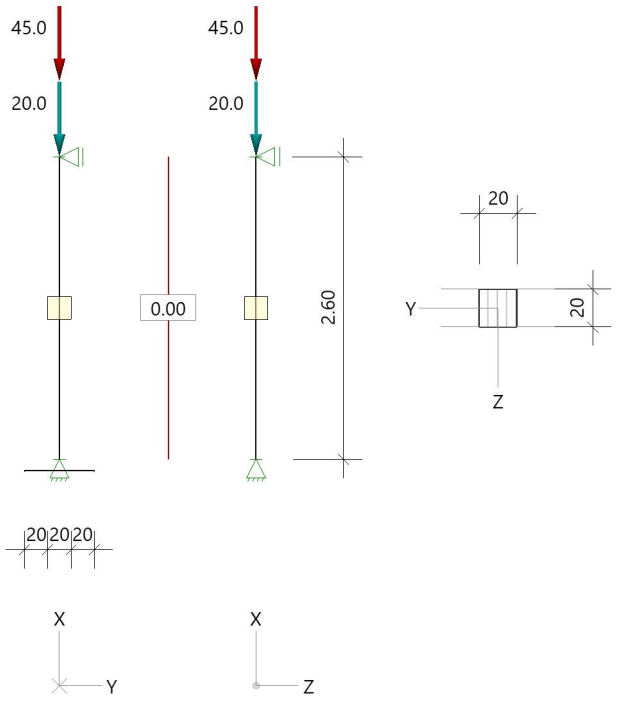
Timber Column (x64) HO1+ 02/2025 (FRILO R-2025-2/P06)

System

Design code

Design	ÖNORM B 1995-1-1:2015-06
Based on	EN 1995-1-1/A2:2014
Combinatorial Analysis	ÖNORM B 1990-1:2013-01-01
Consequence Class	CC 2

System image 2D



System

Simply supported, H=2.60m, b/h=20.0/20.0cm
GL24c, SC 2, γ=5.00 kN/m³, EN 14080:2013 Lamellae/layers parallel h(z)

Foot Binder

No.	b [cm]	h [cm]	a le [cm]	a ri [cm]	Material Name	Ser.Cl. SC
1	20.0	20.0	20.0	20.0	2 C24	2

Material Design code

Material	Design code
GL24c	acc.to ÖNORM B 1995-1-1:2015-06, based on EN 14080:2013
C24	acc.to ÖNORM B 1995-1-1:2015-06, based on EN 338:2016

Loads

List of Loads

No.	Type	ACT	Q1	a1	Q2	L2	fac	grp	About
1	2 X	99	0.5	2.60			1.00		AUTO_G_Mat
2	2 X	99	20.0	2.60			1.00		
3	2 X	10	45.0	2.60			1.00		
4	10 Z	9	0.00				1.00		

Type: 2 = Concentrated load: Q1[kN] at a1[m]; a1 = Distance from base point
10 = Uniformly distributed load: Q1[kN/m]
ACT: 99=Permanent loads; 9=Wind loads; 10=Snow loads H < 1000 m

Combinations

Special rules

Relieving constant loads	Yes

Design / Analysis

Buckling a. tilting lengths

Calculation of Buckling Lengths
Flexural buckling(E) s_k = π2*E*I/(etaNki*Nx)
Lateral torsion buckling(S) s_b = System lengths

(E) Eigenvalue , See check for lengths because load depending.
(S) Calculation using spacing of the rigid bearing

Design Results Decisive Combination (Brief ouput)

Co 4: Load capacity, permanent/transient

Cross-secti.	N,M,V [kN,kNm]	Coefficients		σ,τ [N/mm²]	f...d [N/mm²]	η
Cross-section Check x=2.60m b/h=20.0/20.0cm						
Nx	-95.2	K_{mod}=0.90	γ_M=1.25	-2.38	15.48	0.15
N,M						**0.15**
Stability Check x=2.60m b/h=20.0/20.0cm						
Nx	-95.2	K_{mod}=0.90 $k_{c,y}$=0.93	γ_M=1.25 $k_{c,z}$=0.93	-2.38	15.48	0.16
N,M						**0.16**

Co4: 1.35*G1+1.50*S2
Lamella direction || h
Buckling length: s_{ky}=2.60m s_{kz}=2.60m Tilting length: s_b=2.60m
Slenderness: λ_y=45.0 λ_z=45.0 $\lambda_{rel,c,y}$=0.70 $\lambda_{rel,c,z}$=0.70 $\lambda_{rel,m,y}$=0.24 $\lambda_{rel,m,z}$=0.25
Share N(g)/N(g+q) = 29%; ψ2(Lc, σ_{max})= 0.00; K_{def}= 0.80
Strengths modified according to EN 1995-1-1 para.3.3 + NA/NCI 3.3

Co 4: Load capacity, permanent/transient

Foot Binder: Comp.p.Grain

Cr	$F_{c,90,d}$ [kN]	l_{ef} [cm]	b_{ef} [cm]	A_{ef} [cm²]	$k_{c,90}$	K_{mod}	γ_M	$\sigma_{c,90,d}$ [N/mm²]	$f_{c,90,d}$ [N/mm²]	η
1	-95.2	26.0	20.0	520.0	2.00	0.90	1.30	-1.83	-1.73	**0.53**

Co4: 1.35*G1+1.50*S2
Pressure without consideration of eccentricities
User-defined value $k_{c,90}$

Decisive Deformations

w=0 => η=0

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

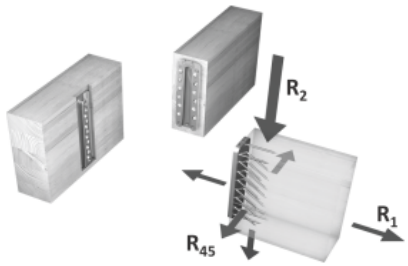
načrt št.: 33/25-K
objekt: P-BGM

Sherpa XS-L-Serie

HBT2-online, Version 4, 09.10.2019
Holzverbinder aus Metall / Einhängeverbinder

Anschluss Nebenträger (NT) an Hauptträger (HT)

SHERPA-Verbinder bestehen aus zwei Aluminiumplatten, die im Prinzip einer klassischen Schwalbenschwanzverbindung kraftschlüssig zusammengefügt werden. Dieses genial-einfache System ermöglicht eine sichere Lastabtragung durch den Einsatz von Sherpa-Spezialschrauben in, entgegen und quer zur Einschubrichtung. Auch Zug- und Druckkräfte werden mühelos aufgenommen und die Berücksichtigung von Torsionsbeanspruchungen ist ebenfalls gewährleistet. Jeder Verbinder verfügt zusätzlich über Bohrungen für Abhebesicherungsschrauben. Detaillierte Angaben siehe SHERPA-Handbuch.
Technischer Support: support@sherpa-connector.com



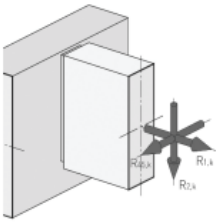
Technische Daten SHERPA-Serien XS, S, M, L

Voraussetzungen zu den Tabellenwerten

Verbinder rechtwinklig montiert
Montage mit Sherpa-Standard-Spezialschrauben
mittige Lasteinleitung
BSH GL24h (Abminderungsfaktor C24 = 0.81)
weitere Holzgüten siehe SHERPA-Handbuch Kapitel 3
bei Laubholz: min. Rohdichte ρ_k= 680 kg/m3
Feuchteklasse1 (NKL1) , Abminderunsgfaktor NKL2= 0.80
Mindestrandabstände
seitlich = 10mm
oben / unten = 15mm*

Erklärungen / Begriffe / Abkürzungen

R_k	charakteristischer Wert Tragfähigkeit
R_d	Bemessungswert Tragfähigkeit: Tabellen-Werte für GL24h mit k_{mod} = 0.8, γ_m = 1.25
SS	Sherpa-Spezial-Vollgewindeschraube
η_2	Abminderungsfaktor bei ausmittiger Beanspruchung
η_s	Faktor Tragfähigkeitswerte variable Schraubenlänge
e	Ausmittung der Lasteinwirkung bezogen auf die Trägerachse
*	Standard-Schraubenlänge



Bezeichnung Lastrichtung

R_{2k}	in Einschubrichtung (vertik.)
R_{45k}	quer zur Einschubri. (horiz.)
R_{1k}	in Richtung Nebenträgera. (Zug)

Positionierung

Federplatte (Schwalbenschwanz) an HT montieren
Nutplatte an NT montieren
Querzug beachten: bei a/h < 0.7 ist ein Querzugnachweis erf.
h = Trägerhöhe
a = für NT: Distanz ok NT zur untersten Schraube
für HT: Distanz uk HT zur obersten Schraube
siehe auch SHERPA- Handbuch

Tabelle 1: Technische Werte Sherpa XS, S, M, L — **Zulassung: ETA-12/0067**

XS, S	Abmessung B/H/D	Befestigung (Stk) SS 4.5x50*		Min. Querschnitt HT	Min. Querschnitt NT	charakt. Werte (KN) vertik.	charakt. Werte (KN) horiz.	charakt. Werte (KN) Zug	Bemessungswerte (KN) vertik.	Bemessungswerte (KN) horiz.	Bemessungswerte (KN) Zug
	mm	HT	NT	mm	mm	R_{2k}	R_{45k}	R_{1k}	R_{2d}	R_{45d}	R_{1d}
XS5, S5	30-40/50/12	6	6	50-60/80	50/80	6.3	3.3	4.4	4.0	2.1	2.8
XS10, S10	30-40/70/12	8	10	50-60/100	50/100	11.8	5.2	8.3	7.6	3.3	5.3
XS15, S15	30-40/90/12	9	12	50-60/120	50/120	14.4	6.2	10.1	9.2	4.0	6.5
XS20, S20	30-40/110/12	11	14	50-60/140	50/140	19.5	7.1	13.7	12.5	4.5	8.8
M	Abmessung B/H/D	Befestigung (Stk) SS 6.5x65*		Min. Querschnitt HT	Min. Querschnitt NT	charakt. Werte (KN) vertik.	charakt. Werte (KN) horiz.	charakt. Werte (KN) Zug	Bemessungswerte (KN) vertik.	Bemessungswerte (KN) horiz.	Bemessungswerte (KN) Zug
	mm	HT	NT	mm	mm	R_{2k}	R_{45k}	R_{1k}	R_{2d}	R_{45d}	R_{1d}
M15	60/90/14	7	9	65/120	80/120	14.9	8.4	10.5	9.5	5.4	6.7
M20	60/110/14	9	11	65/140	80/140	23.6	10.0	16.6	15.1	6.4	10.6
M25	60/130/14	10	13	65/160	80/160	27.8	11.7	19.5	17.8	7.5	12.5
M30	60/150/14	11	15	65/180	80/180	32.0	13.3	22.4	20.5	8.5	14.3
M40	60/170/14	13	17	65/200	80/200	40.1	14.9	28.1	25.7	9.5	18.0
L	Abmessung B/H/D	Befestigung (Stk) SS 8x100*		Min. Querschnitt HT	Min. Querschnitt NT	charakt. Werte (KN) vertik.	charakt. Werte (KN) horiz.	charakt. Werte (KN) Zug	Bemessungswerte (KN) vertik.	Bemessungswerte (KN) horiz.	Bemessungswerte (KN) Zug
	mm	HT	NT	mm	mm	R_{2k}	R_{45k}	R_{1k}	R_{2d}	R_{45d}	R_{1d}
L30	80/150/18	6	9	100/180	100/180	36.1	15.3	25.4	23.1	9.8	16.3
L40	80/170/18	7	11	100/200	100/200	44.2	18.2	31.0	28.3	11.6	19.8
L50	80/210/18	8	13	100/240	100/240	52.0	21.2	36.5	33.3	13.6	23.4
L60	80/250/18	10	15	100/280	100/280	67.4	24.2	47.3	43.1	15.5	30.3
L80	80/290/18	12	17	100/320	100/320	82.4	27.1	57.9	52.7	17.3	37.1
L100	80/330/18	14	19	100/360	100/360	97.1	30.1	68.2	62.1	19.3	43.6
L120	80/370/18	16	21	100/400	100/400	111.6	33.1	78.3	71.4	21.2	50.1

weitere Werte siehe SHERPA-Handbuch Kapitel 3

einseitige Lasteinleitung siehe Folgeseite

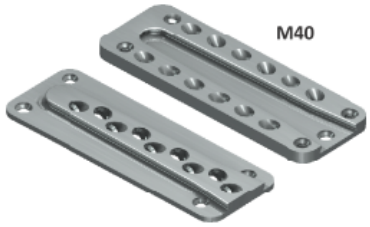
Sherpa Connection Systems GmbH
Badl 31, 8130 Frohnleiten, Österreich
swiss@sherpa-connector.com, http://de.sherpa-connector.com/swiss

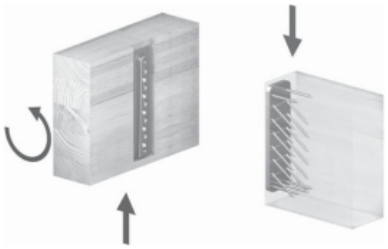
INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

načrt št.: 33/25-K
objekt: P-BGM

Sherpa XS-L-Serie

HBT2-online, Version 4, 09.10.2019
Holzverbinder aus Metall / Einhängeverbinder





einseitige Lasteinleitung zu SHERPA-Serien XS, S, M, L

Abminderungsfaktor η_2 für Tragfähigkeit R_2 (in Einschubrichtung)
für einseitige Anschlüsse
Ausmittung e = 1/2 Breite HT + halbe Verbinderdicke

$R_{2d\eta 2} = R_{2d} * \eta_2$

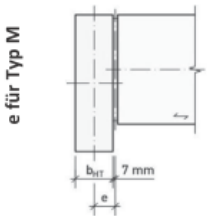


Tabelle 2: Abminderungsfaktor Ausmittung e: η_2

XS, S	e mm]	36	46	56	66	76	86	96	106	116	126
	Breite HT [mm]	60	80	100	120	140	160	180	200	220	240
	XS5, S5	0.795	0.688	0.596	0.520	0.459	0.410	0.370	0.336	0.308	0.284
	XS10, S10	0.622	0.482	0.388	0.324	0.277	0.242	0.215	0.193	0.175	0.160
	XS15, S15	0.711	0.546	0.434	0.357	0.303	0.262	0.231	0.207	0.187	0.171
	XS20, S20	0.792	0.607	0.475	0.387	0.325	0.279	0.245	0.218	0.196	0.178
M	e m	47	57	67	77	87	97	107	117	127	137
	Breite HT mm	80	100	120	140	160	180	200	220	240	260
	M15	0.736	0.626	0.536	0.465	0.410	0.365	0.329	0.299	0.274	0.253
	M20	0.721	0.599	0.505	0.433	0.378	0.335	0.300	0.272	0.248	0.229
	M25	0.737	0.605	0.503	0.428	0.370	0.326	0.291	0.263	0.239	0.220
	M30	0.767	0.623	0.513	0.432	0.371	0.325	0.288	0.259	0.235	0.216
	M40	0.807	0.652	0.531	0.443	0.378	0.328	0.290	0.260	0.235	0.215
L	e mm]	59	69	79	89	99	109	119	129	139	149
	Breite HT [mm]	100	120	140	160	180	200	220	240	260	280
	L30	0.667	0.567	0.488	0.427	0.378	0.339	0.307	0.280	0.258	0.239
	L40	0.685	0.578	0.494	0.429	0.378	0.337	0.304	0.277	0.254	0.235
	L50	0.824	0.708	0.606	0.523	0.458	0.405	0.363	0.329	0.300	0.276
	L60	0.876	0.752	0.636	0.542	0.468	0.410	0.365	0.328	0.297	0.272
	L80	0.934	0.816	0.687	0.578	0.493	0.427	0.376	0.335	0.302	0.275
	L100	0.979	0.892	0.760	0.635	0.534	0.458	0.398	0.352	0.315	0.285
	L120	0.998	0.954	0.839	0.703	0.585	0.495	0.426	0.372	0.330	0.297

weitere Werte siehe SHERPA-Handbuch Kapitel 3

variable Schraubenlängen SHERPA-M- und L-Serie

Die variablen Schraubenlängen ermöglichen eine Erhöhung
der SHERPA-Tragfähigkeit um bis zu 80% (SS 6.5x105mm, Nadelholz)

M-Serie Tabelle 3.1: η_s

SS	Breite HT	Nadelholz η_s	Laubholz η_{sL}
6.5x65*	65	1.00	2.3
6.5x85	85	1.40	2.3
6.5x105	105	1.80	nz

L-Serie Tabelle 3.2: η_s

SS	Breite HT	Nadelholz η_s	Laubholz η_{sL}
8 x 100*	100mm	1.00	2.3
8 x 120	120mm	1.25	2.3
8 x 140	140mm	1.49	nz

$R_{2d,\eta s} = R_{2d} * \eta_s * \eta sL$



Abhebesicherung



$R_{3d} = R_{3k} / 1.25 \times 0.90$
R_{3d} mit k_{mod} 0.9

Abhebesicherung XS-L-Serie Tabelle 4: R_3

Serie	Sperr-S	R_{3k} (KN)	R_{3d} (KN)
XS	1 St 3x12	3.76	2.71
S	1 St 3x20	5.67	4.08
M	1 St 4x20	8.95	6.44
L	2 St 5x47.8	17.50	12.60

Sherpa Connection Systems GmbH
Badl 31, 8130 Frohnleiten, Österreich
swiss@sherpa-connector.com, http://de.sherpa-connector.com/swiss

Potresna analiza

Mesto : GOZD MARTULJEK α **=0,250**

Potresna sila:

$$F_b = S_d(T_1)\cdot W \qquad (3.3) \qquad \text{(EC 8 del 1-2)}$$

a) teža W

$$W_i = \Sigma G_{kj} + \Sigma \Psi_{Ei} \times Q_{ki} \qquad (4.12) \qquad \text{EC 8 del 1-1}$$

$$\Psi_{Ei} = \varphi \times Q_{2i} \qquad (3.15) \qquad \text{EC 8 del 1-2}$$

φ … tabela 3.2, EC 8 del 1-2

(1,0 ... ktg. A, zadnje nadstropje 0,5 ... ktg. A, ostale etaže)

Ψ_{2i} … tabela 9.3, EC 1 del 9.4.4

(0,3 ... ktg. A 0,0 ... sneg, veter, ktg.H)

pritličje

$$W_1 = \Sigma G_{k1} + \Sigma \Psi_{E1} \times Q_{k1} = G_{k1} + \varphi \times \Psi_{21} \times Q_1 + \varphi \times \Psi_{21} \times Q_{snow,1}$$

$W_1 = G_1 + 0{,}30 * Q_1$

Po EC 8:
1,0×0,3=0,30

nadstropje

$$W_2 = \Sigma G_{k2} + \Sigma \Psi_{E2} \times Q_{k2} = G_{k2} + \varphi \times \Psi_{22} \times Q_{utile2} + \varphi \times \Psi_{22} \times Q_{neve,2}$$

$W_2 = G_2$

=1,0 × 0(ktg.H)

=1,0 × 0(sneg)

b) S_d (T_1)

$T_1=C_t \times H^{0.75}$ (C.1) (EC 8)

C_t = 0,050 (po EC 8) H…..višina objekta

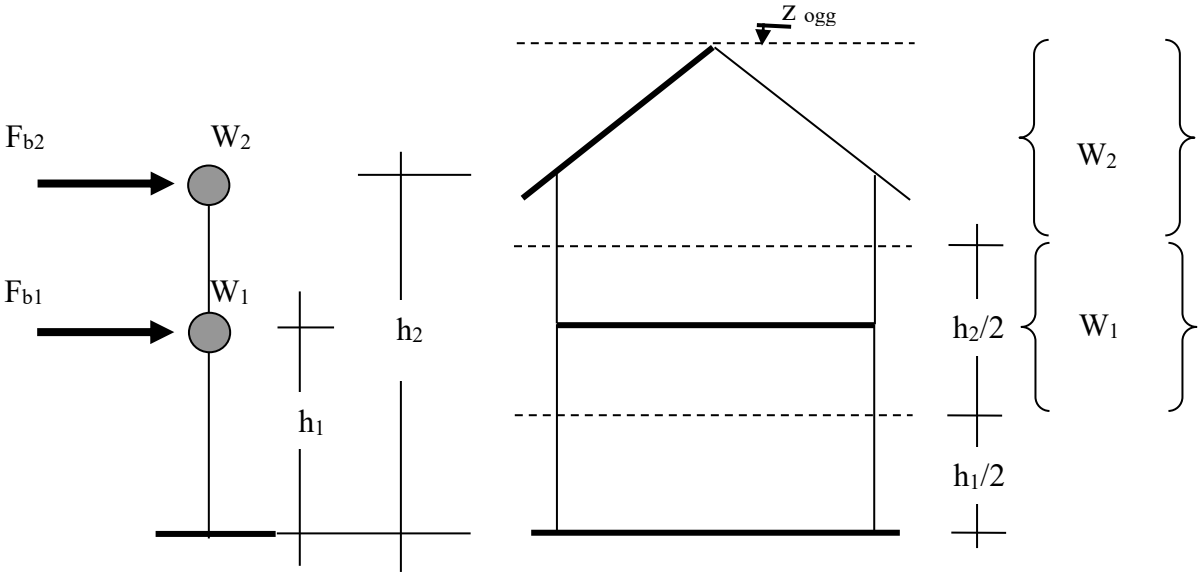
$$T_B \le T_1 \le T_C \rightarrow S_d(T) = \alpha \cdot S \cdot \frac{\beta_0}{q} = 0{,}15/g \times 1{,}15 \times (2{,}5/5)$$

q … faktor obnašanja konstrukcije 5,0 (velikostenski panelni sistemi)

c) Potresna sila:

F_b = S_d x W

d) potresna sila po etažah:

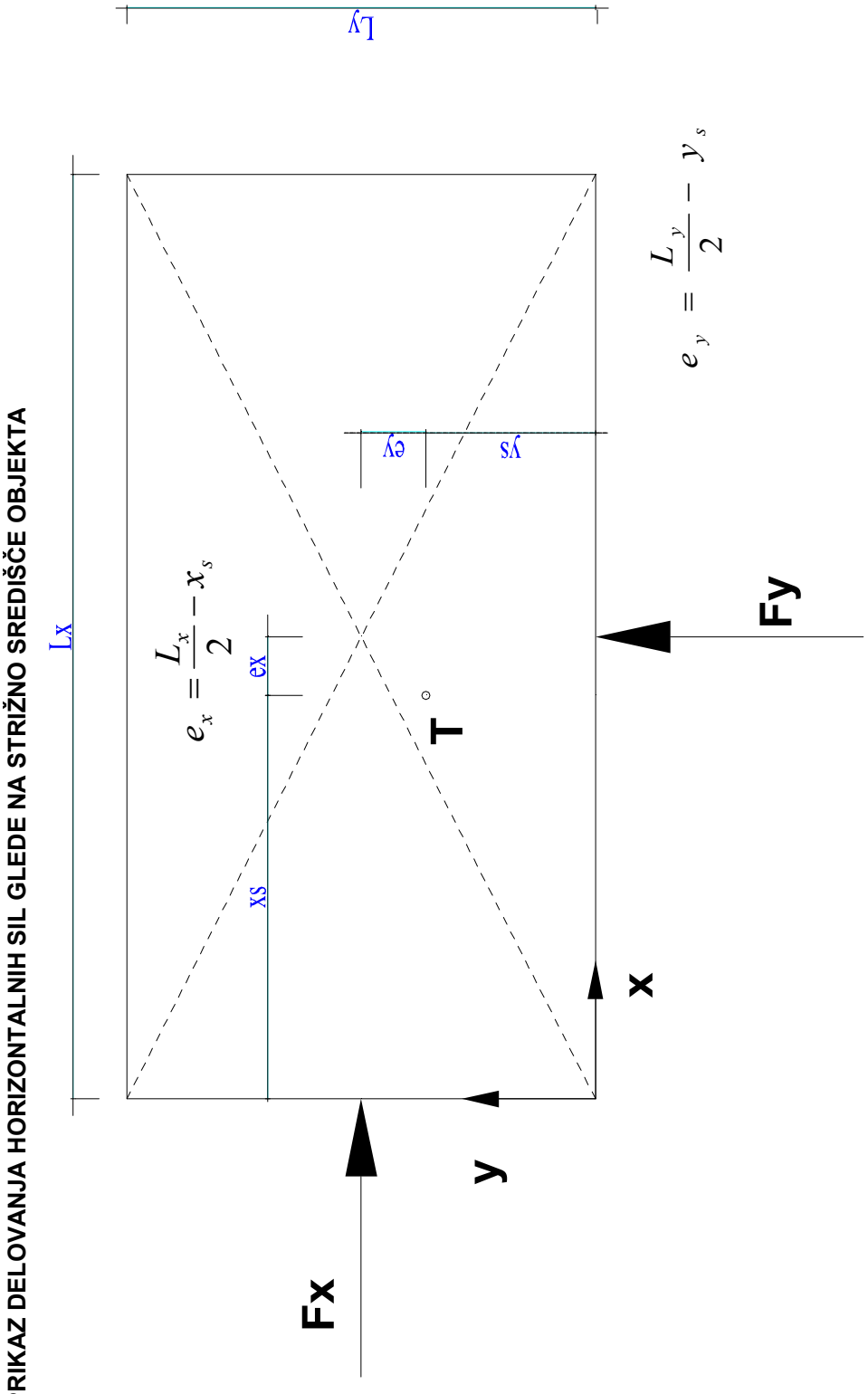


$F_{b1} = F_b \times (h_1 \times W_1 / (h_1 \times W_1 + h_2 \times W_2))$

$F_{b2} = F_b \times (h_2 \times W_2 / (h_1 \times W_1 + h_2 \times W_2)) = F_b - F_{b1}$

INŽENIRING BIRO
ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

PRIKAZ DELOVANJA HORIZONTALNIH SIL GLEDE NA STRIŽNO SREDIŠČE OBJEKTA



INŽENIRING BIRO
ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

DOLOČITEV HORIZONTALNIH SIL V NIVOJU STROPOV

			teza	redukcijski faktorji po EC8			
W2					upoštevan naklon strehe!!!		
streha	5,37	12,02	1	1	64,55		g
	5,37	12,02	1	1	64,55		g
stene M	m	višina stene					
H STENE	24,04	0	0,5	1	0,00		
V STENE	12,80	0	0,5	1	0,00		
					129,09		
W1							
stene P	m						
H STENE	24,04	3,88	0,5	1	46,64		
V STENE	12,80	3,88	0,5	1	24,83		
strop	6,4	3,37	2	1	43,14		g
	0	0	2	1	0,00		p
	6,4	3,37	2	0,3	12,94		p
	0	0	2	0,3	0,00		p
					127,55		
						potres Sd	**Fd skupaj**
					256,64	**0,14**	**36,89** kN

h1	3,88	m	ag= 0,25
h2	7,00	m	H= 7 m
W1	127,55	kN	
W2	129,09	kN	
Fb1	**13,05**		**na pritlicje**
Fb2	**23,84**		**na etazo-streha**

smer delovanja potresa X

NADSTROPJE		Fb_x	$e_y=Ly/2-y_s$	$M_y=Fb_x*e_y$						L=1.25
		23,84	0,00	0,00						
	OZNAKA STENE	bxi	(yi)	bxi*yi	y_s	syi=yi-ys	syi^2	bxi*syi^2	Hxi	Fb
hor. sile na stene v smeri "x"	H11	12,02	0,00	0,00		-3,20	10,24	123,08	11,92	1,2395
	H21	12,02	6,40	76,93		3,20	10,24	123,08	11,92	1,2395
		24,04		76,93	3,20			246,17		1,2395

D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR
INŽENIRING BIRO ARMATURA

načrt št.: 33/25-K
objekt: P-BGM

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

	OZNAKA STENE	byi	(xi)	byi*xi	x_s	sxi=xi-xs	sxi^2	byi*sxi^2	Hyi	Fb
hor. sile na stene v smeri "y"	V11	6,40	0,00	0,00		-6,01	36,12	231,17	11,92	2,3279
	V21	6,40	12,02	76,93		6,01	36,12	231,17	11,92	2,3279
		12,80		**76,93**	**6,01**			**462,34**		**2,3279**

smer delovanja potresa Y

NADSTROPJE		Fb_y	e_x=Lx/2-x_s	M_x=Fb_y*e_x						L=1.25
		23,84	0,00	0,00						
	OZNAKA STENE	bxi	(yi)	bxi*yi	y_s	syi=yi-ys	syi^2	bxi*syi^2	Hxi	Fb
hor. sile na stene v smeri "Y"	V11	6,40	0,00	0,00		-6,01	36,12	231,17	11,92	2,3279
	V21	6,40	12,02	76,93		6,01	36,12	231,17	11,92	2,3279
		12,80		76,93	6,01			462,34		2,3279

	OZNAKA STENE	byi	(xi)	byi*xi	x_s	sxi=xi-xs	sxi^2	byi*sxi^2	Hyi	Fb
hor. sile na stene v smeri "X"	H11	12,02	0,00	0,00		-3,20	10,24	123,08	11,92	1,2395
	H21	12,02	6,40	76,93		3,20	10,24	123,08	11,92	1,2395
		24,04		**76,93**	**3,20**			**246,17**		**1,2395**

D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

ARMATURA INŽENIRING BIRO

načrt št.: 33/25-K
objekt: P-BGM

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

smer delovanja potresa X

PRITLIČJE		Fb_x	$e_y=Ly/2-y_s$	$M_y=Fb_x*e_y$						L=1.25
		13,05	**0,00**	**0,00**						
	OZNAKA STENE	**bxi**	**(yi)**	**bxi*yi**	y_s	**syi=yi-ys**	**syi^2**	**bxi*syi^2**	**Hxi**	**Fb**
hor. sile na stene v smeri "x"	H11	12,02	0,00	0,00		-3,20	10,24	123,08	6,53	0,6788
	H21	12,02	6,40	76,93		3,20	10,24	123,08	6,53	0,6788
		24,04		**76,93**	**3,20**			**246,17**		**0,6788**

	OZNAKA STENE	byi	(xi)	byi*xi	x_s	sxi=xi-xs	sxi^2	byi*sxi^2	Hyi	Fb
hor. sile na stene v smeri "y"	V11	6,40	0,00	0,00		-6,01	36,12	231,17	6,53	1,2748
	V21	6,40	12,02	76,93		6,01	36,12	231,17	6,53	1,2748
		12,80		**76,93**	**6,01**			**462,34**		**1,2748**

D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

smer delovanja potresa Y

PRITLIČJE		Fb_y	$e_x=Lx/2-x_s$	$M_x=Fb_y*e_x$						L=1.25
		13,05	0,00	0,00						
	OZNAKA STENE	bxi	(yi)	bxi*yi	y_s	syi=yi-ys	syi^2	bxi*syi^2	Hxi	Fb
hor. sile na stene v smeri "Y"	V11	6,40	0,00	0,00		-6,01	36,12	231,17	6,53	1,2748
	V21	6,40	12,02	76,93		6,01	36,12	231,17	6,53	1,2748
		12,80		76,93	6,01			462,34		1,2748

D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR
ARMATURA INŽENIRING BIRO

načrt št.: 33/25-K
objekt: P-BGM

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

	OZNAKA STENE	byi	(xi)	byi*xi	x_s	sxi=xi-xs	sxi^2	byi*sxi^2	Hyi	Fb
hor. sile na stene v smeri "X"	H11	12,02	0,00	0,00		-3,20	10,24	123,08	6,53	0,6788
	H21	12,02	6,40	76,93		3,20	10,24	123,08	6,53	0,6788
		24,04		**76,93**	**3,20**			**246,17**		**0,6788**

2-hor sile _ potres MARTULJEK ag0.25

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

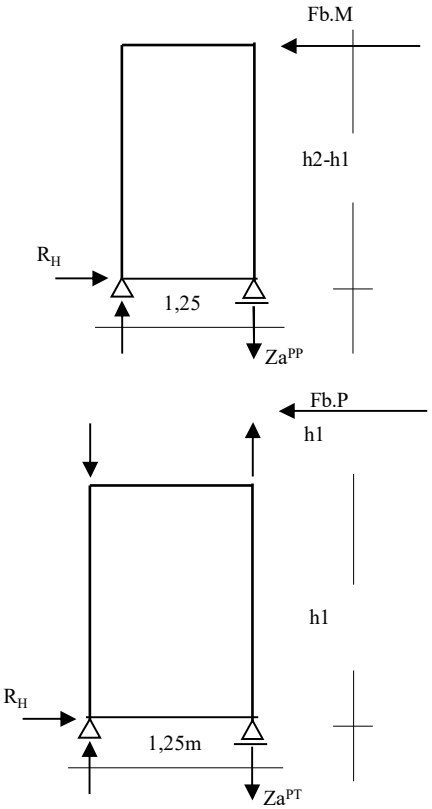
SMER DELOVANJA POTRESNE SILE X

Vrednost hor. sil iz potresne analize v M — Fb.nad v kN= **1,24**

h2 7,00 m

Vrednost hor. sil iz potresne analize v P — Fb.prit v kN= **0,68**

h1 3,88 m



Sidrna sila max. V M - lastna teža sten/stropa/strehe:
upoštevana razbremenitev vsled lastne teže v M=5kN/m

Z.a M= -0,03 kN Raster sider e=1,25m

Sidrna sila max. V P - lastna teža sten/stropa/strehe:
upoštevana razbremenitev vsled lastne teže v P=12kN/m

Z.a P= 2,80 kN Raster sider e=1,25m

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

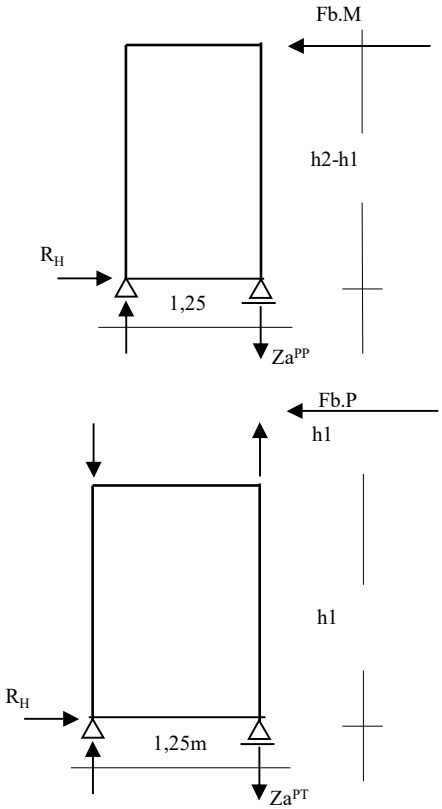
SMER DELOVANJA POTRESNE SILE Y

Vrednost hor. sil iz potresne analize v M — Fb.nad v kN= **2,33**

h2 7,00 m

Vrednost hor. sil iz potresne analize v P — Fb.prit v kN= **1,27**

h1 3,88 m



Sidrna sila max. V M - lastna teža sten/stropa/strehe:
upoštevana razbremenitev vsled lastne teže v M=5kN/m

Z.a M= 2,69 kN Raster sider e=1,25m

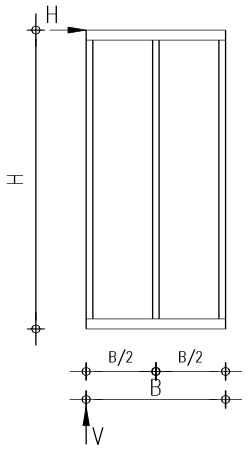
Sidrna sila max. V P - lastna teža sten/stropa/strehe:
upoštevana razbremenitev vsled lastne teže v P=10kN/m

Z.a P= 10,74 kN Raster sider e=1,25m

INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

DOLOČITEV SIDER VSLED HORIZONTALNIH SIL

VELIKOSTENSKI PANELNI SISTEM,
OBOJESTRANSKA OBLOGA STEN - ZAVETROVANJE min. deb 15mm



B = 1,25 m
H = 3,00 m

Sposobnost prevzema hor. sile z MVP ploščo
H_d = **5,0** kN

IZBOR SIDER NA OSNOVI IZRAČUNANIH SIDRNIH SIL

Raster sider e=1,25m oz. po načrtu!!!
PREDZNAK "-" POMENI, DA ZADOŠČA KONSTRUKTIVNO SIDRANJE!

Nadstropje: Za.max= 2,69kN

Natezna sidra:
1x Wurth Typ A-100 ; ŽIČNIKI Ankernägel Wührt 24 x 4/50
+ Pločevinast trak zunaj BP 60-340_
LOCHPLATTE 200/120/2mm Kammnägel 2x12x 4/50

Pritličje: Za.max= 10,74N

Natezna sidra: (F1.Rd=19,0 kN za kratkotrajno obtežbo)
1x WURTH Winkelverbinder V 285 + Ankernägel Wührt 4/40
+ Wurth W-FA/S M12

ALTERNATIVA HTA-2PIL in trak BP 60-340_dvodelno

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

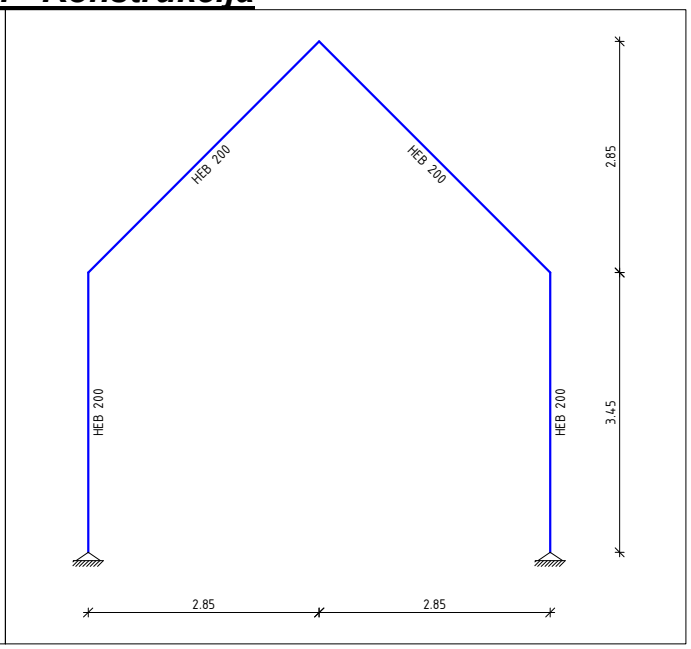
načrt št.: 33/25-K
objekt: P-BGM

JEKLEN OKVIR

Vhodni podatki - Konstrukcija



Izometrija



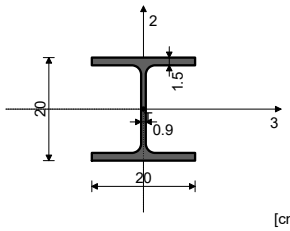
Tabele materialov

No	Naziv materiala	E[kN/m2]	μ	γ[kN/m3]	αt[1/C]	Em[kN/m2]	μm
1	Jeklo S235	2.100e+8	0.30	78.50	1.000e-5	2.100e+8	0.30

Seti gred

Set: 1 Prerez: HEB 200, Fiktivna ekscentričnost

Mat.	A1	A2	A3	I1	I2	I3
1 - Jeklo S235	7.810e-3	2.485e-3	5.325e-3	5.950e-7	2.000e-5	5.700e-5



[cm]

Seti točkovnih podpor

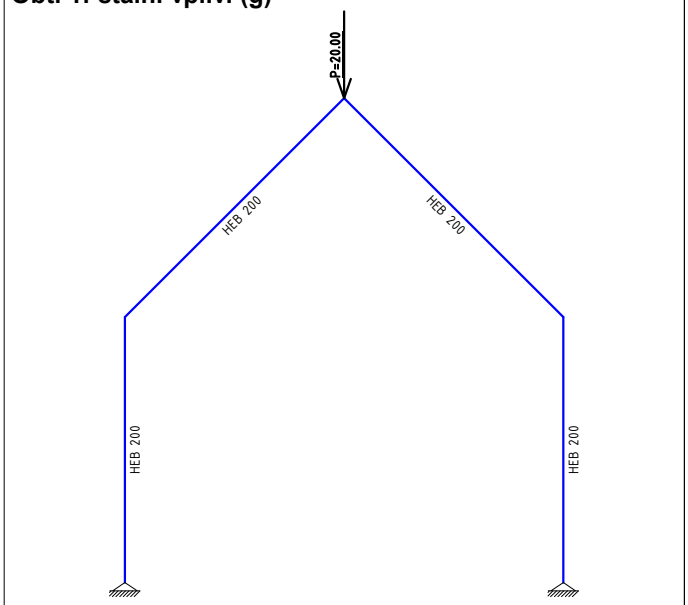
Set	K,R1	K,R2	K,R3	K,M1	K,M2	K,M3
1	1.000e+10	1.000e+10	1.000e+10			

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

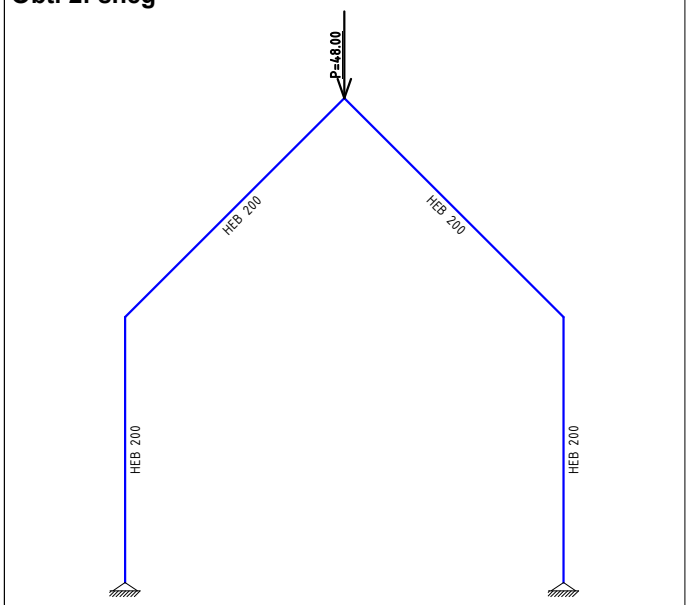
Vhodni podatki - Obtežba

Vplivi na okvir so povzeti izs tatičnega računa lesene konstrukcije. Setavljajo jih reakciji slemenske lege in vplivi vetra. Kapne lege slonijo na stenah in so z okvirjem samo konstruktivno povezane.

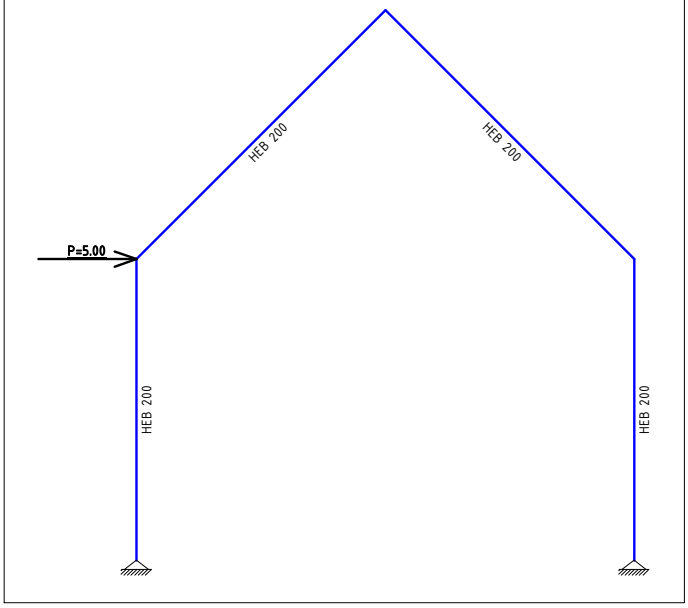
Obt. 1: stalni vplivi (g)



Obt. 2: sneg



Obt. 3: veter



Lista obtežnih primerov

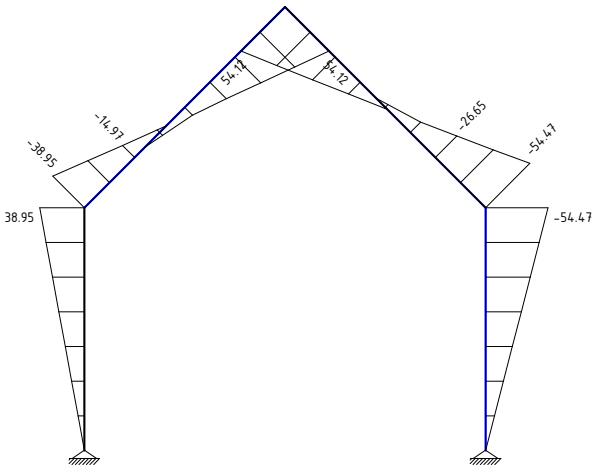
LC	Naziv
1	stalni vplivi (g)
2	sneg
3	veter
4	Komb.: MSN1 (1.35xI+1.5xII+0.9xIII)

LC	Naziv
5	Komb.: MSN2 (1.35xI+0.75xII+1.5xIII)
6	Komb.: MSU1 (I+II+0.6xIII)
7	Komb.: MSU2 (I+0.5xII+III)

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

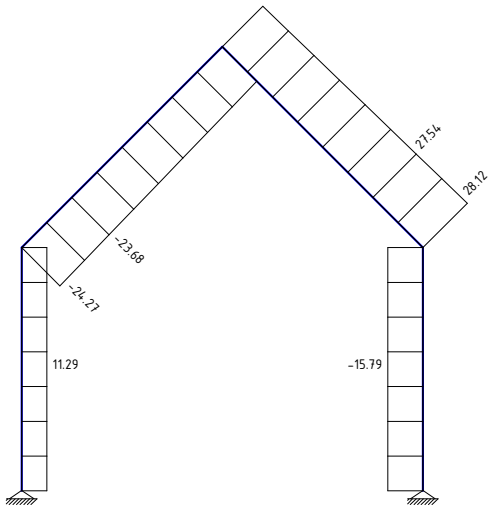
Statični preračun

Obt. 8: [MSN] 4,5



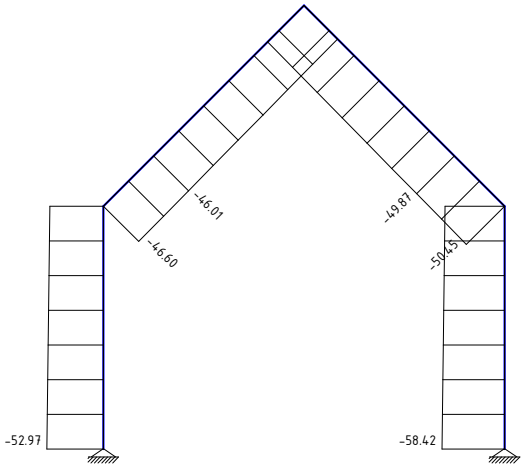
Vplivi v gredi: max M3= 54.12 / min M3= -54.47 kNm

Obt. 8: [MSN] 4,5



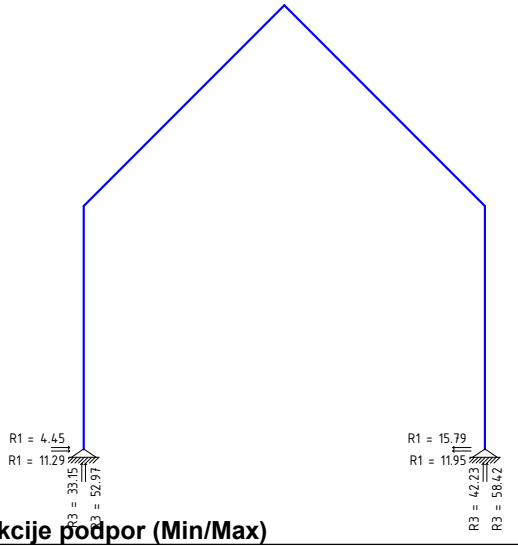
Vplivi v gredi: max V2= 28.12 / min V2= -24.27 kN

Obt. 8: [MSN] 4,5



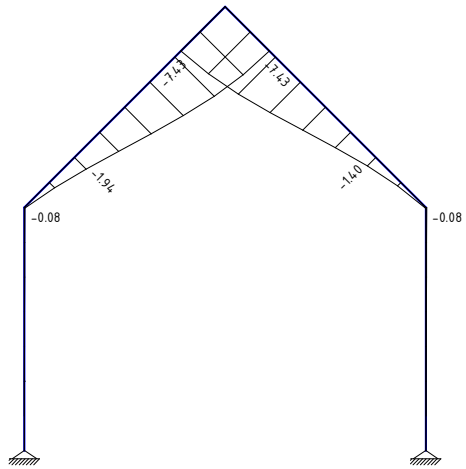
Vplivi v gredi: max N1= -27.51 / min N1= -58.42 kN

Obt. 8: [MSN] 4,5



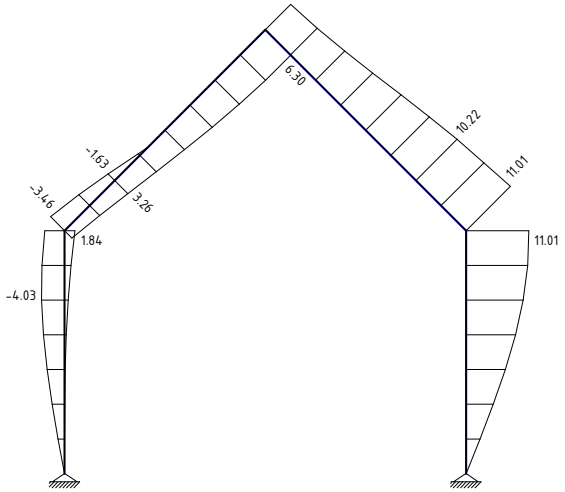
Reakcije podpor (Min/Max)

Obt. 9: [MSU] 6,7



Vplivi v gredi: max Zp= -0.00 / min Zp= -7.43 m / 1000

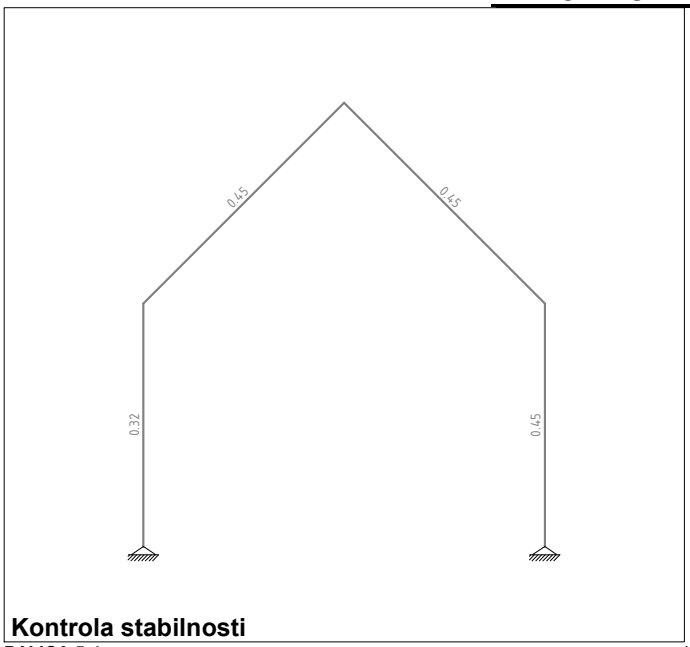
Obt. 9: [MSU] 6,7



Vplivi v gredi: max Xp= 11.01 / min Xp= -4.03 m / 1000

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

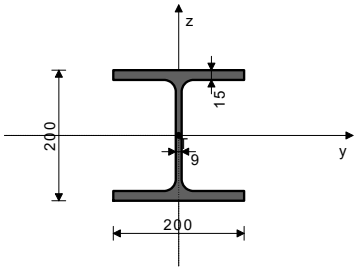
Dimenzioniranje (jeklo)



Kontrola stabilnosti

PALICA 5-4
PREČNI PREREZ: HEB 200 [S 235] [Set: 1]
EUROCODE 3 (EN 1993-1-1:2005)

GEOMETRIJSKE KARAKTERISTIKE prereza



Ax =	78.100 cm2
Ay =	53.250 cm2
Az =	24.850 cm2
Ix =	59.500 cm4
Iy =	5700.0 cm4
Iz =	2000.0 cm4
Wy =	570.00 cm3
Wz =	200.00 cm3
Wy,pl =	635.93 cm3
Wz,pl =	300.00 cm3
γM0 =	1.000
γM1 =	1.000
γM2 =	1.250
Anet/A =	0.900

(fy = 23.5 kN/cm2, fu = 36.0 kN/cm2)

FAKTORJI IZKORIŠČENOSTI PO KOMBINACIJAH OBTEŽB
4. γ=0.45 5. γ=0.34

PALICA IZPOSTAVLJENA PRITISKU IN UPOGIBU
(obtežni primer 4, konec palice)

Računska osna sila	N_{Ed} =	-50.451 kN
Prečna sila v z smeri	$V_{Ed,z}$ =	28.122 kN
Upogibni moment okoli y osi	$M_{Ed,y}$ =	-54.474 kNm
Sistemska dolžina palice	L =	403.05 cm

5.5 KLASIFIKACIJA PREČNIH PREREZOV
Razred prereza 1

6.2 NOSILNOST PREČNIH PREREZOV
6.2.4 Tlak
Računska nosilnost na tlak $N_{c,Rd}$ = 1835.4 kN
Pogoj 6.9: N_{Ed} <= $N_{c,Rd}$ (50.45 <= 1835.35)

6.2.5 Upogib y-y
Upoštevajo se tudi luknje za vezna sredstva.
Efektivni odpornostni moment $W_{y,eff}$ = 512.71 cm3
Računska nosilnost na upogib $M_{c,Rd}$ = 120.49 kNm
Pogoj 6.12: $M_{Ed,y}$ <= $M_{c,Rd,y}$ (54.47 <= 120.49)

6.2.6 Strig
Računska strižna nosilnost $V_{pl,Rd,z}$ = 337.16 kN
Računska strižna nosilnost $V_{c,Rd,z}$ = 337.16 kN
Pogoj 6.17: $V_{Ed,z}$ <= $V_{c,Rd,z}$ (28.12 <= 337.16)

6.2.10 Upogib z osno in prečno silo
Ni potrebno zmanjšanje upogibne nosilnosti
Pogoj: $V_{Ed,z}$ <= 50%$V_{pl,Rd,z}$

6.2.9 Upogib in osna sila
Razmerje N_{Ed} / $N_{pl,Rd}$ 0.027
Zmanjšana plast.upogibna nosilnost $M_{N,y,Rd}$ = 149.44 kNm

Koeficient α = 1.000
Razmerje ($M_{y,Ed}$ / $M_{N,y,Rd}$)^α 0.365
Pogoj 6.41: (0.36 <= 1)

6.3 NOSILNOST ELEMENTA NA UKLON
6.3.1.1 Nosilnost na uklon

Uklonska dolžina y-y	l,y =	403.05 cm
Relativna vitkost y-y	λ_y =	0.502
Uklonska krivulja za os y-y: B	α =	0.340
Elastična kritična sila	$N_{cr,y}$ =	7272.3 kN
Koeficient nepopolnosti	χ,y =	0.883
Računska uklonska nosilnost	$N_{b,Rd,y}$ =	1620.9 kN

Pogoj 6.46: N_{Ed} <= $N_{b,Rd,y}$ (50.45 <= 1620.92)

Uklonska dolžina z-z	l,z =	403.05 cm
Relativna vitkost z-z	λ_z =	0.848
Uklonska krivulja za os z-z: C	α =	0.490
Koeficient nepopolnosti	χ,z =	0.632
Računska uklonska nosilnost	$N_{b,Rd,z}$ =	1160.0 kN

Pogoj 6.46: N_{Ed} <= $N_{b,Rd,z}$ (50.45 <= 1159.96)

6.3.2.1 Nosilnost na bočno-torzijski uklon

Koeficient	C1 =	2.757
Koeficient	C2 =	0.000
Koeficient	C3 =	0.010
Koef.ukl.dolžine za uklon	k =	1.000
Koef.ukl.dolžine za vbočenje	kw =	1.000
Koordinata	zg =	0.000 cm
Koordinata	zj =	0.000 cm
Razmak med bočnimi podporami	L =	403.05 cm
Sektorski vztrajnostni moment	Iw =	1.71e+5 cm6
Krit.moment bočne zvrnitve	Mcr =	1164.1 kNm
Ustrezni odpornostni moment	W_y =	635.93 cm3
Koeficient imperf.	αLT =	0.340
Brezdimenz.vitkost	λLT_ =	0.358
Koeficient zmanjšanja (6.3.2.3.)	χLT =	1.000
Računska uklonska nosilnost	$M_{b,Rd}$ =	149.44 kNm

Pogoj 6.54: $M_{Ed,y}$ <= $M_{b,Rd}$ (47.47 <= 149.44)

6.3.3. Elementi konstantnega prečnega prereza obremenjeni z upogibom in osnim tlakom
Preračun koeficienta interakcije je izvršen z alternativno metodo št.2 (Aneks B)

Koeficient oblike momenta	C_{my} =	0.400
Koeficient oblike momenta	C_{mz} =	1.000
Koeficient oblike momenta	C_{mLT} =	0.400
Koeficient interakcije	k_{yy} =	0.404
Koeficient interakcije	k_{yz} =	0.629
Koeficient interakcije	k_{zy} =	0.975
Koeficient interakcije	k_{zz} =	1.048

Koeficient nepopolnosti χy = 0.883
N_{Ed} / (χy N_{Rk} / γM1) 0.031
kyy * (M_{yEd} + ΔM_{yEd}) / ... 0.147
Pogoj 6.61: (0.18 <= 1)

Koeficient nepopolnosti χz = 0.632
N_{Ed} / (χz N_{Rk} / γM1) 0.043
kzy * (M_{yEd} + ΔM_{yEd}) / ... 0.356
Pogoj 6.62: (0.40 <= 1)

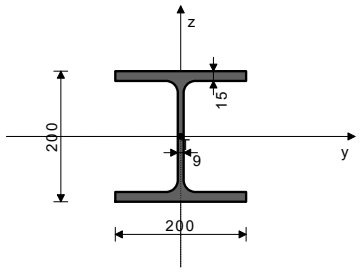
INŽENIRING BIRO ARMATURA

D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

PALICA 3-5

PREČNI PREREZ: HEB 200 [S 235] [Set: 1]
EUROCODE 3 (EN 1993-1-1:2005)

GEOMETRIJSKE KARAKTERISTIKE prereza



[mm]

Ax =	78.100 cm2
Ay =	53.250 cm2
Az =	24.850 cm2
Ix =	59.500 cm4
Iy =	5700.0 cm4
Iz =	2000.0 cm4
Wy =	570.00 cm3
Wz =	200.00 cm3
Wy,pl =	635.93 cm3
Wz,pl =	300.00 cm3
γM0 =	1.000
γM1 =	1.000
γM2 =	1.250
Anet/A =	0.900

(fy = 23.5 kN/cm2, fu = 36.0 kN/cm2)

FAKTORJI IZKORIŠČENOSTI PO KOMBINACIJAH OBTEŽB
4. γ=0.45 5. γ=0.34

PALICA IZPOSTAVLJENA PRITISKU IN UPOGIBU
(obtežni primer 4, začetek palice)

Računska osna sila	N_{Ed} =	-55.560 kN
Prečna sila v z smeri	$V_{Ed,z}$ =	-15.789 kN
Upogibni moment okoli y osi	$M_{Ed,y}$ =	-54.474 kNm
Sistemska dolžina palice	L =	345.00 cm

5.5 KLASIFIKACIJA PREČNIH PREREZOV
Razred prereza 1

6.2 NOSILNOST PREČNIH PREREZOV
6.2.4 Tlak

Računska nosilnost na tlak	$N_{c,Rd}$ =	1835.4 kN

Pogoj 6.9: N_{Ed} <= $N_{c,Rd}$ (55.56 <= 1835.35)

6.2.5 Upogib y-y
Upoštevajo se tudi luknje za vezna sredstva.

Efektivni odpornostni moment	$W_{y,eff}$ =	512.71 cm3
Računska nosilnost na upogib	$M_{c,Rd}$ =	120.49 kNm

Pogoj 6.12: $M_{Ed,y}$ <= $M_{c,Rd,y}$ (54.47 <= 120.49)

6.2.6 Strig

Računska strižna nosilnost	$V_{pl,Rd,z}$ =	337.16 kN
Računska strižna nosilnost	$V_{c,Rd,z}$ =	337.16 kN

Pogoj 6.17: $V_{Ed,z}$ <= $V_{c,Rd,z}$ (15.79 <= 337.16)

6.2.10 Upogib z osno in prečno silo
Ni potrebno zmanjšanje upogibne nosilnosti
Pogoj: $V_{Ed,z}$ <= 50%$V_{pl,Rd,z}$

6.2.9 Upogib in osna sila

Razmerje N_{Ed} / $N_{pl,Rd}$		0.030
Zmanjšana plast.upogibna nosilnost	$M_{N,y,Rd}$ =	149.44 kNm
Koeficient	α =	1.000
Razmerje ($M_{y,Ed}$ / $M_{N,y,Rd}$)^α		0.365

Pogoj 6.41: (0.36 <= 1)

6.3 NOSILNOST ELEMENTA NA UKLON
6.3.1.1 Nosilnost na uklon

Uklonska dolžina y-y	l,y =	345.00 cm
Relativna vitkost y-y	λ_y =	0.430
Uklonska krivulja za os y-y: B	α =	0.340
Elastična kritična sila	$N_{cr,y}$ =	9925.6 kN
Koeficient nepopolnosti	χ,y =	0.914
Računska uklonska nosilnost	$N_{b,Rd,y}$ =	1677.5 kN

Pogoj 6.46: N_{Ed} <= $N_{b,Rd,y}$ (55.56 <= 1677.48)

Uklonska dolžina z-z	l,z =	345.00 cm
Relativna vitkost z-z	λ_z =	0.726
Uklonska krivulja za os z-z: C	α =	0.490
Koeficient nepopolnosti	χ,z =	0.709
Računska uklonska nosilnost	$N_{b.Rd,z}$ =	1300.5 kN

Pogoj 6.46: N_{Ed} <= $N_{b,Rd,z}$ (55.56 <= 1300.47)

6.3.2.1 Nosilnost na bočno-torzijski uklon

Koeficient	C1 =	1.879
Koeficient	C2 =	0.000
Koeficient	C3 =	0.939
Koef.ukl.dolžine za uklon	k =	1.000
Koef.ukl.dolžine za vbočenje	kw =	1.000
Koordinata	zg =	0.000 cm
Koordinata	zj =	0.000 cm
Razmak med bočnimi podporami	L =	345.00 cm
Sektorski vztrajnostni moment	Iw =	1.71e+5 cm6
Krit.moment bočne zvrnitve	Mcr =	978.43 kNm
Ustrezni odpornostni moment	W_y =	635.93 cm3
Koeficient imperf.	αLT =	0.340
Brezdimenz.vitkost	λLT_ =	0.391
Koeficient zmanjšanja (6.3.2.3.)	χLT =	1.000
Računska uklonska nosilnost	$M_{b,Rd}$ =	149.44 kNm

Pogoj 6.54: $M_{Ed,y}$ <= $M_{b,Rd}$ (54.47 <= 149.44)

6.3.3. Elementi konstantnega prečnega prereza obremenjeni z upogibom in osnim tlakom
Preračun koeficienta interakcije je izvršen z alternativno metodo št.2 (Aneks B)

Koeficient oblike momenta	C_{my} =	0.600
Koeficient oblike momenta	C_{mz} =	1.000
Koeficient oblike momenta	C_{mLT} =	0.600
Koeficient interakcije	k_{yy} =	0.605
Koeficient interakcije	k_{yz} =	0.622
Koeficient interakcije	k_{zy} =	0.991
Koeficient interakcije	k_{zz} =	1.036
Koeficient nepopolnosti	χy =	0.914
N_{Ed} / (χy N_{Rk} / γM1)		0.033
kyy * (M_{yEd} + ΔM_{yEd}) / ...		0.220

Pogoj 6.61: (0.25 <= 1)

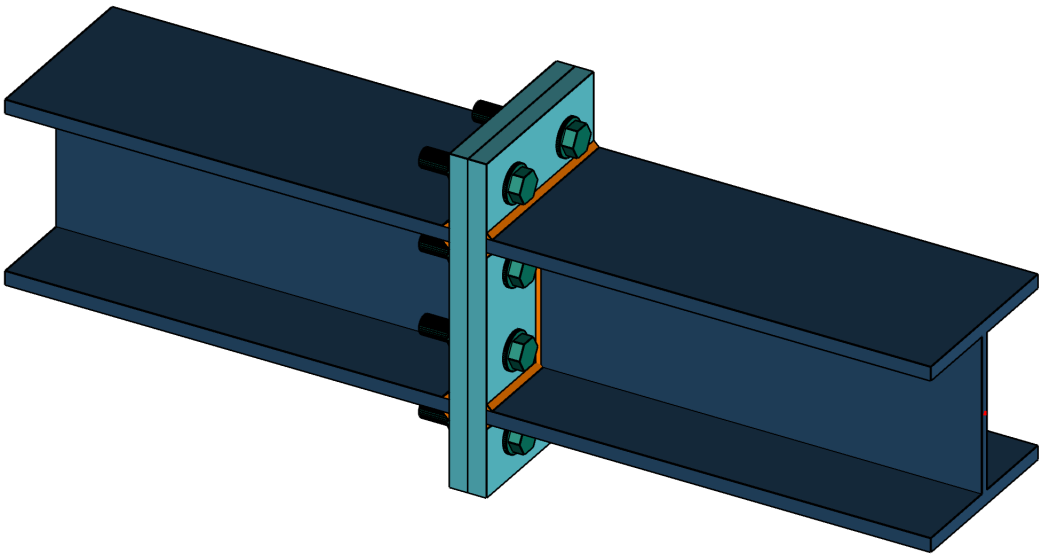
Koeficient nepopolnosti	χz =	0.709
N_{Ed} / (χz N_{Rk} / γM1)		0.043
kzy * (M_{yEd} + ΔM_{yEd}) / ...		0.361

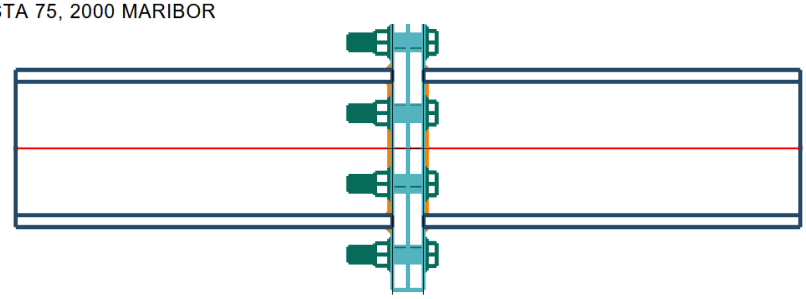
Pogoj 6.62: (0.40 <= 1)

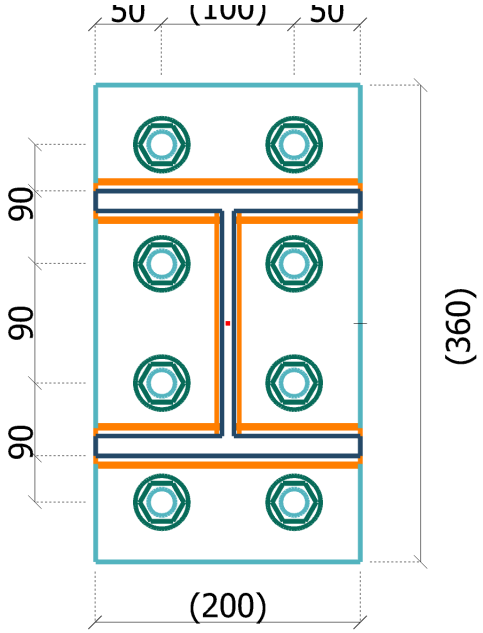
SPOJ JEKLENEGA OKVIRJA

Apex Report

Maximum Work Ratio	71.98 %	Passed







1 Joint description

Beam HEB200 (Section Class 1)

Material: S235 (EN 10025-2)

Bolts dimensions and properties

Diameter $d = 20$ mm Class 8.8 Bolt shear area $A_s = 245$ mm²

End Plate

360 mm×200 mm×20 mm Material: S235 (EN 10025-2)

---

2 Design Assumptions

Design standards

EN 1993-1-8 Design of Steel Structures. Design of Joints

EN 1993-1 National Annex: General Eurocode.

EN 1993-1-1 Design of Steel Structures. General Rules and Rules for Buildings

Bolts

The shear plane passes through the threaded part of the bolt.

3 Holes distances conditions

Next, "Right" holes and "Left" holes refer to the joint part to which the holes belong (e.g. right plate).

3.1 Minimum distance conditions for round holes §3.5, Tab. 3.3

Minimum edge distance on the load direction

Left holes **Passed**

Right holes **Passed**

Minimum edge distance perpendicular on the load direction

Left holes **Passed**

Right holes **Passed**

Minimum spacing between the centers of 2 holes, measured on the direction of the load

Left holes **Passed**

Right holes **Passed**

Minimum spacing between the centers of 2 holes, measured perpendicular on the direction of the load

Left holes **Passed**

Right holes **Passed**

3.2 Maximum distance for steel without improved atmospheric corrosion resistance (acc. EN 10025-5*)

Maximum edge distance on the load direction

Left holes **Passed**

Right holes **Passed**

Maximum edge distance perpendicular on the load direction

Left holes **Passed**

Right holes **Passed**

Maximum spacing between the centers of 2 holes on the load direction

Left holes **Passed**

Right holes **Passed**

Maximum spacing between the centers of 2 holes perpendicular on the load direction

Left holes **Passed**

Right holes **Passed**

4 Compression verifications

Compression verifications are done for each beam of the connection (left and right). In the following are presented only those verifications which are giving the smallest resistance (or work ratio).

4.1 Compression resistance of the beam - Left side of the connection

The Design Combined Compression Resistance of the Beam Flange and the Adjacent Compression Zone of the Beam Web

$F_{c,fb,Rd} = 816.15$ kN EN 1993-1-8, 6.2.6.7

5 Bolt verifications with loads from right beam

For each side of the connection, bolts connecting the member with the end plate are verified with torsor (group of efforts) from right side of the connection.
Only verifications giving the maximum work ratio are displayed.

5.1 Bolt tension verification

Combination: [1]: max(M)

$F_{t,Ed} = 81.42$ kN *Design Tension Force for one Bolt*

$F_{t,Rd} = 141.12$ kN *Design Tension Resistance for one Bolt* §3.6.1, tab. 3.4

Check relation: $F_{t,Ed} \le F_{t,Rd}$ $81.42 \text{ kN} \le 141.12 \text{ kN}$ **Work Ratio: 57.7 %** **Passed**

5.2 Bolts shear verification

Combination: [1]: max(M)

$F_{v,Ed} = \frac{V_{Ed}}{n_b \times n} = 3.75$ kN *Effective Shear Force for one bolt*

$F_{v,Rd} = 94.08$ kN *Design Shear Resistance for one bolt* §3.6.1, tab. 3.4

Check relation: $F_{v,Ed} \le F_{v,Rd}$ $|3.75 \text{ kN}| \le 94.08 \text{ kN}$ **Work Ratio: 3.99 %** **Passed**

5.3 Diametral bearing verification of the bolt on the Left End Plate

Combination: [1]: max(M)

$F_{v,Ed} = \frac{V_{Ed}}{n_b \times n} = 3.75$ kN *Effective Shear Force for one bolt*

$F_{b,Rd} = 216$ kN *Design Bearing Resistance for one bolt* §3.6.1, tab. 3.4

Check relation: $F_{v,Ed} \le F_{b,Rd}$ $3.75 \text{ kN} \le 216 \text{ kN}$ **Work Ratio: 1.74 %** **Passed**

5.4 Diametral bearing verification of the bolt on the Right End Plate

Combination: [1]: max(M)

$F_{v,Ed} = \frac{V_{Ed}}{n_b \times n} = 3.75$ kN *Effective Shear Force for one bolt*

$F_{b,Rd} = 216$ kN *Design Bearing Resistance for one bolt* §3.6.1, tab. 3.4

Check relation: $F_{v,Ed} \le F_{b,Rd}$ $3.75 \text{ kN} \le 216 \text{ kN}$ **Work Ratio: 1.74 %** **Passed**

5.5 Bolts punching shear verification

Combination: [1]: max(M)

$F_{t,Ed} = 81.42$ kN

$B_{p,Rd} = 360.9$ kN *Design Punching Shear Resistance for one Bolt* §3.6.1, tab. 3.4

Check relation: $F_{t,Ed} \le B_{p,Rd}$ $81.42 \text{ kN} \le 360.9 \text{ kN}$ **Work Ratio: 22.56 %** **Passed**

5.6 Bolt Slip Resistance Verification

Verification is not required.

5.7 Bolt combined shear-tension verification

Combination: [1]: max(M)

$$\left|\frac{F_{v,Ed}}{F_{v,Rd}}\right| + \left|\frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}}\right| = \left|\frac{3.75 \text{ kN}}{94.08 \text{ kN}}\right| + \left|\frac{81.42 \text{ kN}}{1.4 \times 141.12 \text{ kN}}\right| = 0.45$$ §3.6.1, tab. 3.4

Check relation: $\left|\frac{F_{v,Ed}}{F_{v,Rd}}\right| + \left|\frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}}\right| \le 1$ $0.45 \le 1$ **Work Ratio: 45.2 %** **Passed**

6 Bolt verifications with loads from left beam

For each side of the connection, bolts connecting the member with the end plate are verified with torsor (group of efforts) from left side of the connection.
Only verifications giving the maximum work ratio are displayed.

6.1 Bolt tension verification

Combination: [1]: max(M)

$F_{t,Ed} = 81.42$ kN *Design Tension Force for one Bolt*

$F_{t,Rd} = 141.12$ kN *Design Tension Resistance for one Bolt* §3.6.1, tab. 3.4

Check relation: $F_{t,Ed} \le F_{t,Rd}$ $81.42 \text{ kN} \le 141.12 \text{ kN}$ **Work Ratio: 57.7 %** **Passed**

6.2 Bolts shear verification

Combination: [1]: max(M)

$F_{v,Ed} = \frac{V_{Ed}}{n_b \times n} = 3.75$ kN *Effective Shear Force for one bolt*

$F_{v,Rd} = 94.08$ kN *Design Shear Resistance for one bolt* §3.6.1, tab. 3.4

Check relation: $F_{v,Ed} \le F_{v,Rd}$ $|3.75 \text{ kN}| \le 94.08 \text{ kN}$ **Work Ratio: 3.99 %** **Passed**

6.3 Diametral bearing verification of the bolt on the Right End Plate

Combination: [1]: max(M)

$F_{v,Ed} = \frac{V_{Ed}}{n_b \times n} = 3.75 \text{ kN}$ *Effective Shear Force for one bolt*

$F_{b,Rd} = 216 \text{ kN}$ *Design Bearing Resistance for one bolt* §3.6.1, tab. 3.4

Check relation: $F_{v,Ed} \le F_{b,Rd}$ $3.75 \text{ kN} \le 216 \text{ kN}$ **Work Ratio: 1.74 %** **Passed**

6.4 Diametral bearing verification of the bolt on the Left End Plate

Combination: [1]: max(M)

$F_{v,Ed} = \frac{V_{Ed}}{n_b \times n} = 3.75 \text{ kN}$ *Effective Shear Force for one bolt*

$F_{b,Rd} = 216 \text{ kN}$ *Design Bearing Resistance for one bolt* §3.6.1, tab. 3.4

Check relation: $F_{v,Ed} \le F_{b,Rd}$ $3.75 \text{ kN} \le 216 \text{ kN}$ **Work Ratio: 1.74 %** **Passed**

6.5 Bolts punching shear verification

Combination: [1]: max(M)

$F_{t,Ed} = 81.42 \text{ kN}$

$B_{p,Rd} = 360.9 \text{ kN}$ *Design Punching Shear Resistance for one Bolt* §3.6.1, tab. 3.4

Check relation: $F_{t,Ed} \le B_{p,Rd}$ $81.42 \text{ kN} \le 360.9 \text{ kN}$ **Work Ratio: 22.56 %** **Passed**

6.6 Bolt Slip Resistance Verification

Verification is not required.

6.7 Bolt combined shear-tension verification

Combination: [1]: max(M)

$\left|\frac{F_{v,Ed}}{F_{v,Rd}}\right| + \left|\frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}}\right| = \left|\frac{3.75 \text{ kN}}{94.08 \text{ kN}}\right| + \left|\frac{81.42 \text{ kN}}{1.4 \times 141.12 \text{ kN}}\right| = 0.45$ §3.6.1, tab. 3.4

Check relation: $\left|\frac{F_{v,Ed}}{F_{v,Rd}}\right| + \left|\frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}}\right| \le 1$ $0.45 \le 1$ **Work Ratio: 45.2 %** **Passed**

SIDRANJE JEKLENEGA OKVIRJA

1 Input data

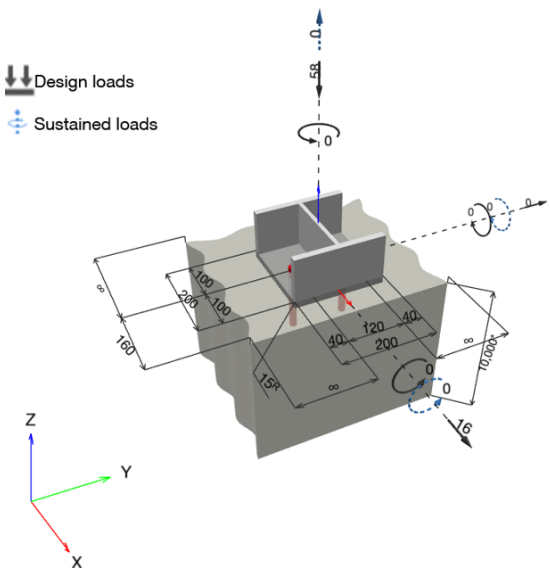


SAFE-SET

Anchor type and diameter:	**HUS4-H Bonded Screw 16**
Return period (service life in years):	50
Item number:	2333577 HUS4-H 16x165 80/35 (element) / 2344440 HUS4-MAX 16 (capsule)
Specification text:	Hilti HUS Carbon Steel screw anchor with MAX capsule mortar with 130 mm embedment hef, 16, Steel galvanized, Hammer drill bit installation per ETA-18/1160
Effective embedment depth:	h_{ef} = 104.9 mm ($h_{ef,ETA}$ = 130.0 mm), h_{nom} = 150.0 mm
Material:	Carbon Steel
Evaluation Service Report:	ETA-18/1160
Issued I Valid:	16. 01. 2025 | -
Proof:	Design Method EN 1992-4, EOTA TR 075
Stand-off installation:	e_b = 0.0 mm (no stand-off); t = 15.0 mm
Anchor plate[R]:	l_x x l_y x t = 200.0 mm x 200.0 mm x 15.0 mm; (Recommended plate thickness: not calculated)
Profile:	IPB/HEB, IPB 200 / HE 200 B; (L x W x T x FT) = 200.0 mm x 200.0 mm x 9.0 mm x 15.0 mm
Base material:	cracked concrete, C25/30, $f_{c,cyl}$ = 25.00 N/mm^2; h =10,000.0 mm, Temp. short/long: 0/0 °C, partial material safety factor γ_c = 1.500; $\gamma_{c,seismic}$ = 1.500
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm) no longitudinal edge reinforcement

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



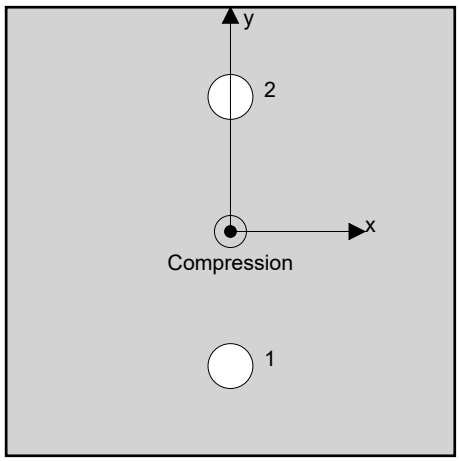
INŽENIRING BIRO ARMATURA
D.O.O. ZRKOVSKA CESTA 75, 2000 MARIBOR

1.1 Load combination

Case	Description	Forces [kN] / Moments [kNm]	Seismic	Fire	Max. Util. Anchor [%]
1	MSN 1	$N = -58.000$; $V_x = 16.000$; $V_y = 0.000$; $M_x = 0.000$; $M_y = 0.000$; $M_z = 0.000$; $N_{sus} = 0.000$; $M_{x,sus} = 0.000$; $M_{y,sus} = 0.000$;	no	no	64
2	MSN 2	$N = -42.000$; $V_x = 12.000$; $V_y = 0.000$; $M_x = 0.000$; $M_y = 0.000$; $M_z = 0.000$; $N_{sus} = 0.000$; $M_{x,sus} = 0.000$; $M_{y,sus} = 0.000$;	no	no	48

2 Load case/Resulting anchor forces

Controlling load case: 1 MSN 1



Anchor reactions [kN]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	8.000	8.000	0.000
2	0.000	8.000	8.000	0.000

Max. concrete compressive strain: 0.05 [‰]
Max. concrete compressive stress: 1.45 [N/mm^2]
Resulting tension force in (x/y)=(-/-): 0.000 [kN]
Resulting compression force in (x/y)=(0.0/0.0): 58.000 [kN]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
PROFIS Engineering (c) 2003-2025 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

3 Tension load (EN 1992-4, Section 7.2.1)

	Load [kN]	Capacity [kN]	Utilization β_N [%]	Status
Steel Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Splitting failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
PROFIS Engineering (c) 2003-2025 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

4 Shear load (EN 1992-4, Section 7.2.2)

	Load [kN]	Capacity [kN]	Utilization β_V [%]	Status
Steel Strength (without lever arm)*	8.000	46.784	18	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	16.000	85.693	19	OK
Concrete edge failure in direction x+**	16.000	25.023	64	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength (without lever arm)

$$V_{Ed} \le V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}}$$ EN 1992-4, Table 7.2

$$V_{Rk,s} = k_7 \cdot V^0_{Rk,s}$$ EN 1992-4, Eq. (7.35)

$V^0_{Rk,s}$ [kN]	k_7	$V_{Rk,s}$ [kN]	γ_{Ms}	$V_{Rd,s}$ [kN]	V_{Ed} [kN]
73.100	0.800	58.480	1.250	46.784	8.000

4.2 Pryout Strength

$$V_{Ed} \le V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc,p}}$$ EN 1992-4, Table 7.2

$$V_{Rk,cp} = k_8 \cdot N_{Rk,c}$$ EN 1992-4, Eq. (7.39a)

$$N_{Rk,c} = N^0_{Rk,c} \cdot \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec1,N} \cdot \psi_{ec2,N} \cdot \psi_{M,N}$$ EN 1992-4, Eq. (7.1)

$$N^0_{Rk,c} = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1,5}$$ EN 1992-4, Eq. (7.2)

$$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N}$$ EN 1992-4, Eq. (7.3)

$$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \le 1.00$$ EN 1992-4, Eq. (7.4)

$$\psi_{ec1,N} = \frac{1}{1 + \left(\frac{2 \cdot e_{V,1}}{s_{cr,N}}\right)} \le 1.00$$ EN 1992-4, Eq. (7.6)

$$\psi_{ec2,N} = \frac{1}{1 + \left(\frac{2 \cdot e_{V,2}}{s_{cr,N}}\right)} \le 1.00$$ EN 1992-4, Eq. (7.6)

$$\psi_{M,N} = 1$$ EN 1992-4, Eq. (7.7)

$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k_8	$f_{c,cyl}$ [N/mm²]	
181,050	152,100	195.0	390.0	2.000	25.00	
$e_{c1,V}$ [mm]	$\psi_{ec1,N}$	$e_{c2,V}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$	$\psi_{M,N}$
0.0	1.000	0.0	1.000	0.946	1.000	1.000
k_1	$N^0_{Rk,c}$ [kN]	$\gamma_{Mc,p}$	$V_{Rd,cp}$ [kN]	V_{Ed} [kN]		
7.700	57.066	1.500	85.693	16.000		

Group anchor ID
1, 2

4.3 Concrete edge failure in direction x+

$V_{Ed} \le V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$ EN 1992-4, Table 7.2

$V_{Rk,c} = V^0_{Rk,c} \cdot \frac{A_{c,V}}{A^0_{c,V}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ EN 1992-4, Eq. (7.40)

$V^0_{Rk,c} = k_9 \cdot d_{nom}^{\alpha} \cdot l_f^{\beta} \cdot \sqrt{f_{ck}} \cdot c_1^{1,5}$ EN 1992-4, Eq. (7.41)

$\alpha = 0.1 \cdot \left(\frac{l_f}{c_1}\right)^{0,5}$ EN 1992-4, Eq. (7.42)

$\beta = 0.1 \cdot \left(\frac{d_{nom}}{c_1}\right)^{0,2}$ EN 1992-4, Eq. (7.43)

$A^0_{c,V} = 4.5 \cdot c_1^2$ EN 1992-4, Eq. (7.44)

$\psi_{s,V} = 0.7 + 0.3 \cdot \frac{c_2}{1.5 \cdot c_1} \le 1.00$ EN 1992-4, Eq. (7.45)

$\psi_{h,V} = \left(\frac{1.5 \cdot c_1}{h}\right)^{0,5} \ge 1.00$ EN 1992-4, Eq. (7.46)

$\psi_{ec,V} = \frac{1}{1 + \left(\frac{2 \cdot e_V}{3 \cdot c_1}\right)} \le 1.00$ EN 1992-4, Eq. (7.47)

$\psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.5 \cdot \sin \alpha_V)^2}} \ge 1.00$ EN 1992-4, Eq. (7.48)

l_f [mm]	d_{nom} [mm]	k_9	α	β	$f_{c,cyl}$ [N/mm^2]	c_1 [mm]
130.0	16.00	1.700	0.090	0.063	25.00	160.0

$A_{c,V}$ [mm^2]	$A^0_{c,V}$ [mm^2]	$\psi_{s,V}$	$\psi_{h,V}$	$e_{c,V}$ [mm]	$\psi_{ec,V}$
144,000	115,200	1.000	1.000	0.0	1.000

α_V [°]	$\psi_{\alpha,V}$	$\psi_{re,V}$
0.00	1.000	1.000

$V^0_{Rk,c}$ [kN]	γ_{Mc}	$V_{Rd,c}$ [kN]	V_{Ed} [kN]
30.027	1.500	25.023	16.000

Group anchor ID
1, 2

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

PROFIS Engineering (c) 2003-2025 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

5 Displacements (highest loaded anchor)

Short term loading:

N_{Sk}	=	0.000 [kN]	δ_N	=	0.0000 [mm]
V_{Sk}	=	5.926 [kN]	δ_V	=	0.2552 [mm]
			δ_{NV}	=	0.2552 [mm]

Long term loading:

N_{Sk}	=	0.000 [kN]	δ_N	=	0.0000 [mm]
V_{Sk}	=	5.926 [kN]	δ_V	=	0.3828 [mm]
			δ_{NV}	=	0.3828 [mm]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the anchor plate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on metric units. When inputs are displayed in imperial units, the user should be aware that the equations remain in their metric format.
- In general, the conditions given in ETAG 001, Annex C, section 4.2.2.1 and 4.2.2.3 b) are not fulfilled because the diameter of the clearance hole in the fixture acc. to Annex 3, Table 3 is greater than the values given in Annex C, Table 4.1 and AS5126 for the corresponding diameter of the anchor. Therefore the design resistance for anchor groups is limited to twice the steel resistance (of a single anchor) in accordance with the approval.
- Checking the transfer of loads into the base material is required in accordance with EN 1992-4, Annex A!
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 6.1 of EN 1992-4! For larger diameters of the clearance hole see section 6.2.2 of EN 1992-4!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- For the determination of the $\psi_{re,v}$ (concrete edge failure) the minimum concrete cover defined in the design settings is used as the concrete cover of the edge reinforcement.
- The characteristic bond resistances depend on the return period (service life in years): 50

Fastening meets the design criteria!

PROFIS Engineering (c) 2003-2025 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

7 Installation data

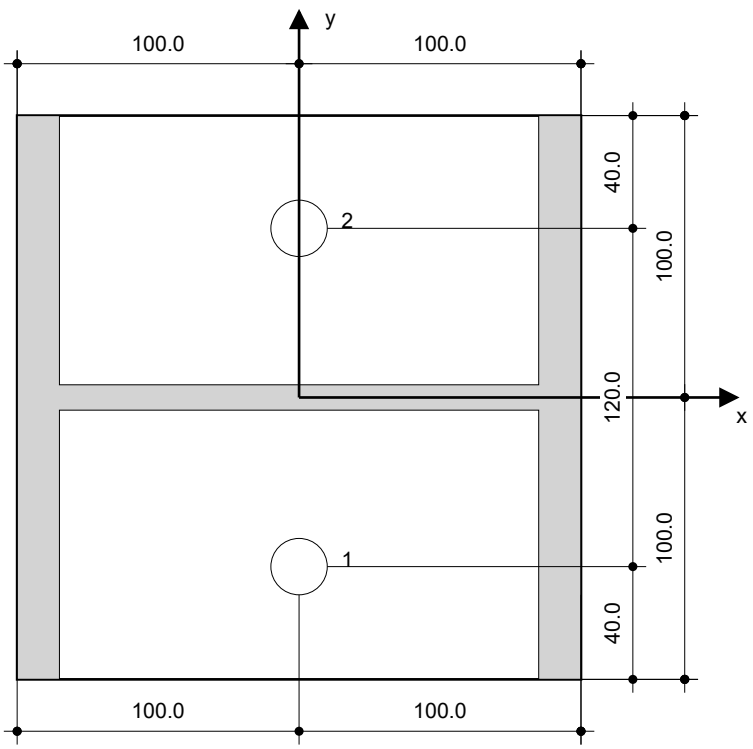
Anchor plate, steel: S 235; E = 210,000.00 N/mm^2; f_{yk} = 235.00 N/mm^2
Profile: IPB/HEB, IPB 200 / HE 200 B; (L x W x T x FT) = 200.0 mm x 200.0 mm x 9.0 mm x 15.0 mm
Hole diameter in the fixture: d_f = 20.0 mm
Plate thickness (input): 15.0 mm
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Clean the drill hole. Under the conditions - according to fastener size and drilling direction - given in the Assessment document and MPII (IFU), the cleaning of the drill hole may be omitted.

Anchor type and diameter: HUS4-H Bonded Screw 16
Item number: 2333577 HUS4-H 16x165 80/35 (element) / 2344440 HUS4-MAX 16 (capsule)
Maximum installation torque: Hilti SIW 22T-A
Hole diameter in the base material: 16.0 mm
Hole depth in the base material: 160.0 mm
Minimum thickness of the base material: 200.0 mm

Hilti HUS Carbon Steel screw anchor with MAX capsule mortar with 130 mm embedment hef, 16, Steel galvanized, Hammer drill bit installation per ETA-18/1160

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Hilti SIW 22T-A impact screw driver



Coordinates Anchor [mm]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.0	-60.0	-	160.0	-	-
2	0.0	60.0	-	160.0	-	-

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

TEMELJNA PLOŠČA

Osnovni podatki o modelu, Vhodni podatki - Konstrukcija

Datoteka: Temeljna_plošča.twp
Datum preračuna: 20.10.2025

Način preračuna: 3D model

☒ Teorija I-ga reda ☐ Modalna analiza ☐ Stabilnost
☐ Teorija II-ga reda ☐ Seizmični preračun ☐ Faze gradnje
☐ Nelinearen preračun

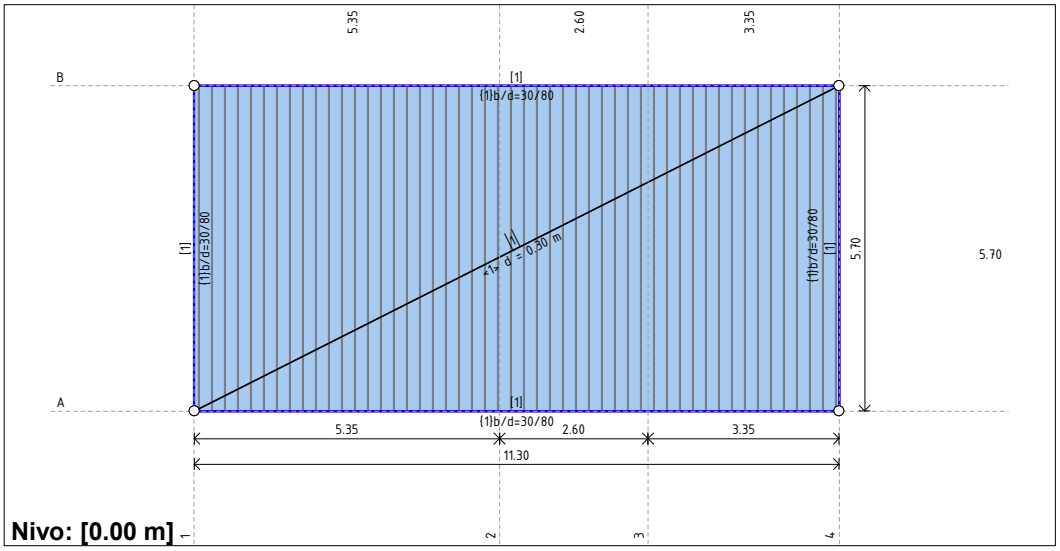
Velikost modela

Število vozlišč:	780
Število ploskovnih elementov:	722
Število grednih elementov	114
Število robnih elementov	9348
Število osnovnih obtežnih primerov:	5
Število kombinacij obtežb:	6

Enote mer

Dolžina:	m [cm,mm]
Sila:	kN
Temperatura:	Celsius

TLORISNA GEOMETRIJA KONSTRUKCIJE



Tabele materialov

No	Naziv materiala	E[kN/m2]	μ	γ[kN/m3]	αt[1/C]	Em[kN/m2]	μm
1	C 25/30	3.100e+7	0.20	25.00	1.000e-5	3.100e+7	0.20

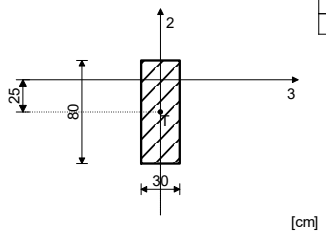
Seti plošč

No	d[m]	e[m]	Material	Tip preračuna	Ortotropija	E2[kN/m2]	G[kN/m2]	α
<1>	0.300	0.150	1	Tanka plošča	Izotropna			

Seti gred

Set: 1 Prerez: b/d=30/80, Fiktivna ekscentričnost

Mat.	A1	A2	A3	I1	I2	I3
1 - C 25/30	2.400e-1	2.000e-1	2.000e-1	5.502e-3	1.800e-3	1.280e-2



Seti površinskih podpor

Set	K,R1	K,R2	K,R3
1	2.000e+4	2.000e+4	3.000e+4

Seti linijskih podpor

Set	K,R1	K,R2	K,R3	K,M1	Tla [m]
1	2.000e+4	3.000e+4	2.000e+4		0.300

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

Vhodni podatki - Obtežba

DOLOČITEV OBTEŽB

VPLIVI NA KONSTRUKCIJO

STALNI VPLIVI:

Lesene stene

- fasadna obloga 4cm.....0,10 kN/m2
- zračni sloj.....0,15 kN/m2
- letve 5/8 cm+ kontra letve.....0,10 kN/m2
- obloga (deske) 2,2cm.....0,18 kN/m2
- vetrna zapora.....0,05 kN/m2
- izolacija.....0,10 kN/m2
- obloga (deske) 2,2cm.....0,18 kN/m2
- steber 8/15.....0,06 kN/m2
- izolacija.....0,10 kN/m2
- notranja obloga 2,2cm.....0,18 kN/m2
- SKUPAJ.....1,20 kN/m2

Temelnja plošča

- finalni tlak.....0,20 kN/m2
- estrih 6cm.....1,44 kN/m2
- izolacija 0,26 kN/m2
- hidroizolacija 1cm.....0,10 kN/m2
- AB plošča.....avtomatsko
- SKUPAJ.....2,00 kN/m2

SPREMENLJIVI VPLIVI:

- koristna obtežba (kat. A).....2,00 kN/m2

Linjska obtežba sten:

3,35m * 1,20 kN/m2 = 4,02 kN/m

(3,35m+(1/3*2.84)) * 1,20 kN/m2 = 5,16 kN/m

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

KOMBINACIJE OBTEŽB

Pri obdelavi rezultatov so zraven spodaj navedenih kombinacij obtežb uporabljene tudi različne ovojnice kombinacij. Ovojnice kombinacij obtežb so posebej označene s številkami obtežnih primerov, ki sestavljajo ovojnico.

Lista obtežnih primerov

LC	Naziv
1	Stalna (g)
2	Koristna A
3	Koristna H
4	Sneg
5	Veter
6	Komb.: MSN 1 (1.35xI+1.5xII+1.5xIII+0.75xIV+0.9xV)

LC	Naziv
7	Komb.: MSN 2 (1.35xI+1.05xII+1.5xIV+0.9xV)
8	Komb.: MSN 3 (1.35xI+1.5xII+0.75xIV+1.5xV)
9	Komb.: MSU 1 (I+II+III+0.5xIV+0.6xV)
10	Komb.: MSU 2 (I+0.7xII+IV+0.6xV)
11	Komb.: MSU 3 (I+0.7xII+0.5xIV+V)

Seznam obtežnih primerov ovojnic

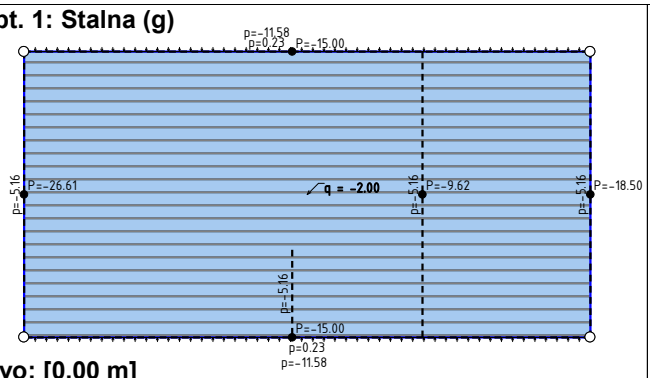
LC	Naziv
12	[MSN mirni] 6-8

LC	Naziv
13	[MSU mirni] 9-11

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

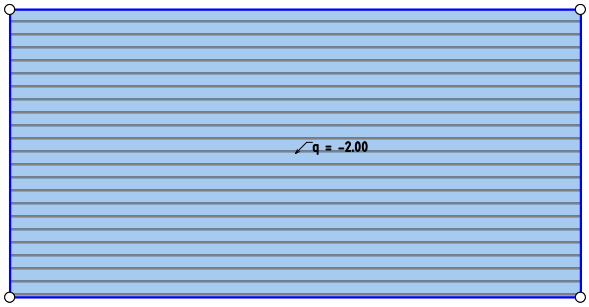
GRAFIČNI PRIKAZ STALNIH in SPREMENLJIVIH VPLIVOV

Obt. 1: Stalna (g)



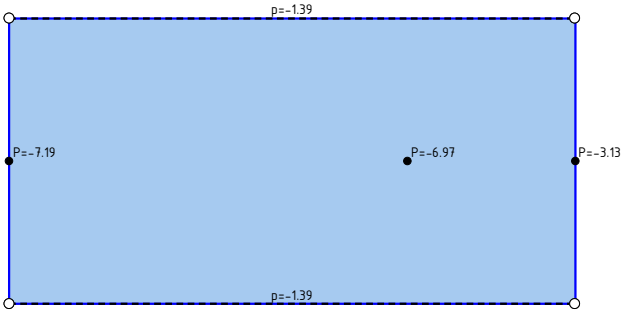
Nivo: [0.00 m]

Obt. 2: Koristna A



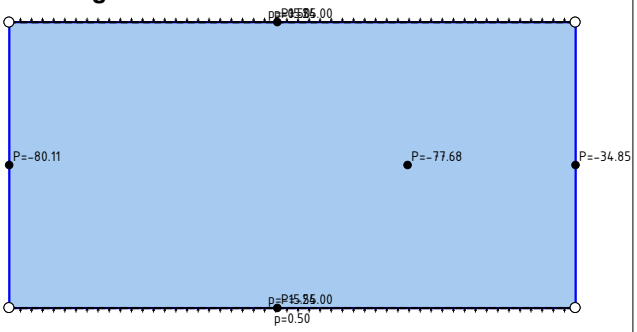
Nivo: [0.00 m]

Obt. 3: Koristna H



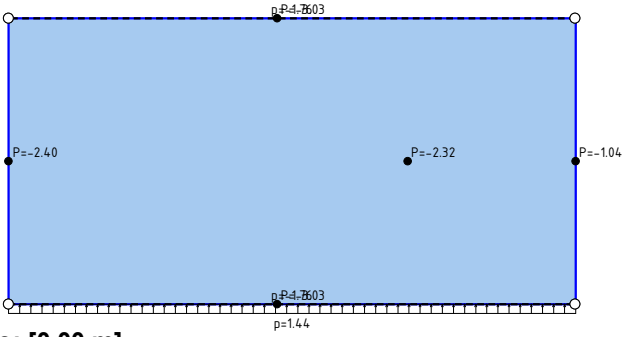
Nivo: [0.00 m]

Obt. 4: Sneg



Nivo: [0.00 m]

Obt. 5: Veter



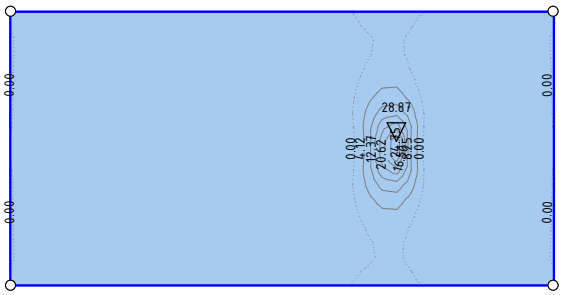
Nivo: [0.00 m]

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

Statični preračun

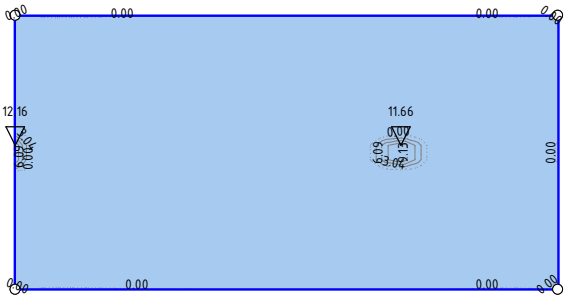
NSK V TEMELJNI PLOŠČI

Obt. 12: [MSN mirni] 6-8



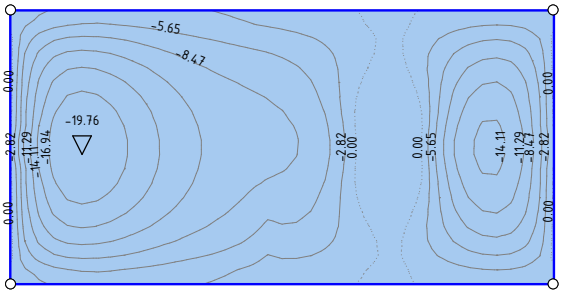
Nivo: [0.00 m]
Vplivi v plošči: max Mx= 28.87 / min Mx= 0.00 kNm/m

Obt. 12: [MSN mirni] 6-8



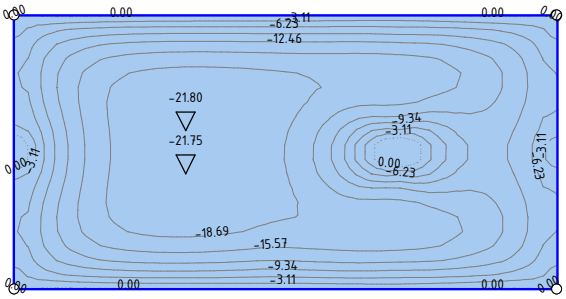
Nivo: [0.00 m]
Vplivi v plošči: max My= 12.16 / min My= 0.00 kNm/m

Obt. 12: [MSN mirni] 6-8



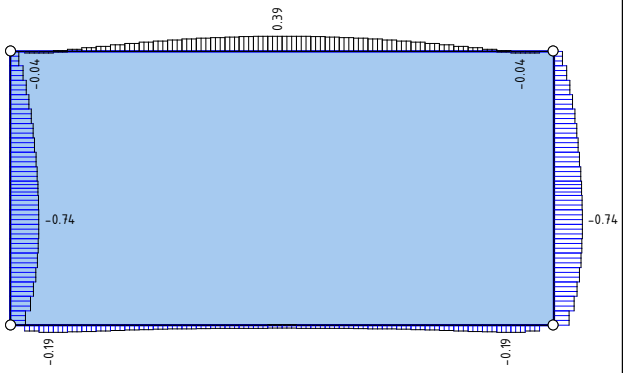
Nivo: [0.00 m]
Vplivi v plošči: max Mx= 0.00 / min Mx= -19.76 kNm/m

Obt. 12: [MSN mirni] 6-8



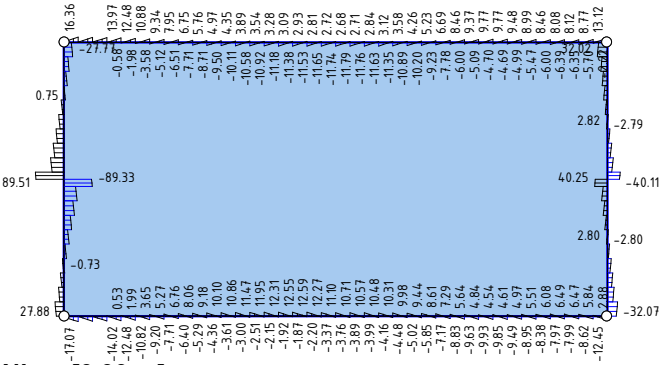
Nivo: [0.00 m]
Vplivi v plošči: max My= 0.00 / min My= -21.80 kNm/m

Obt. 12: [MSN mirni] 6-8



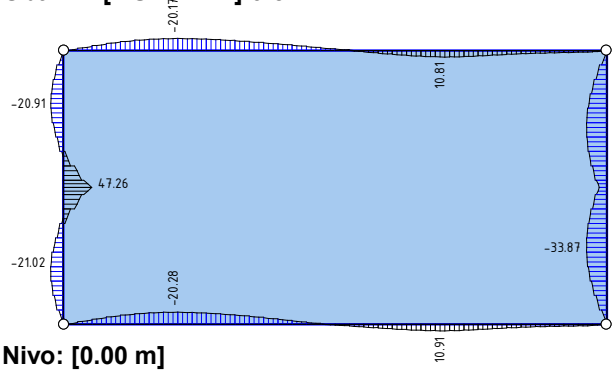
Nivo: [0.00 m]
Vplivi v gredi: max N1= 0.40 / min N1= -0.74 kN

Obt. 12: [MSN mirni] 6-8



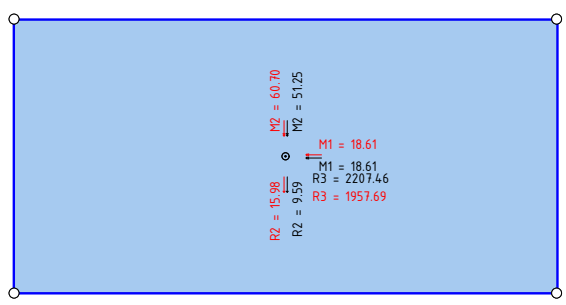
Nivo: [0.00 m]
Vplivi v gredi: max T2= 89.51 / min T2= -89.33 kN

Obt. 12: [MSN mirni] 6-8



Nivo: [0.00 m]
Vplivi v gredi: max M3= 47.26 / min M3= -33.87 kNm

Obt. 12: [MSN mirni] 6-8



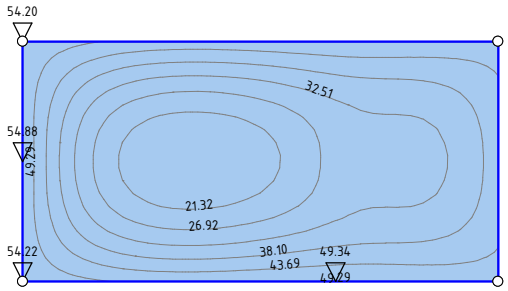
Nivo: [0.00 m]
Rezultanta površinske podpore

INŽENIRING BIRO ARMATURA

D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

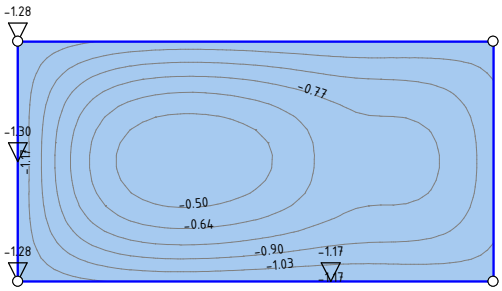
Napetosti in pomiki pod temeljno ploščo

Obt. 12: [MSN mirni] 6-8



Nivo: [0.00 m]
Vplivi v pov.podpori: max σ,tal= 54.88 / min σ,tal= 15....

Obt. 13: [MSU mirni] 9-11



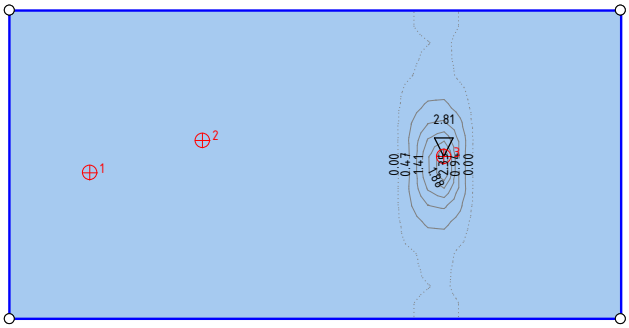
Nivo: [0.00 m]
Vplivi v pov.podpori: max s,tal= -0.38 / min s,tal= -1.3...

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

Dimenzioniranje (beton)

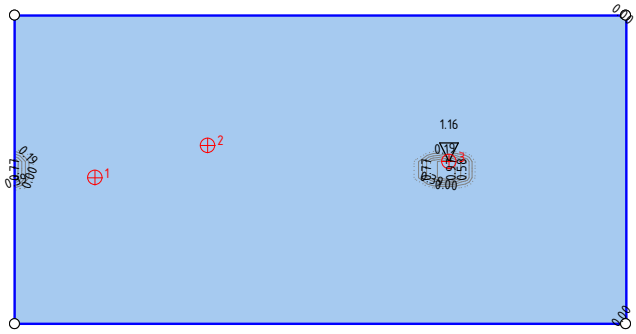
DIMENZIONIRANJE TEMLJNE PLOŠČE

Merodajna obtežba: Kompletna shema
EC2 (EN 1992-1-1:2004), C 25/30, S500B, a=6.00 cm



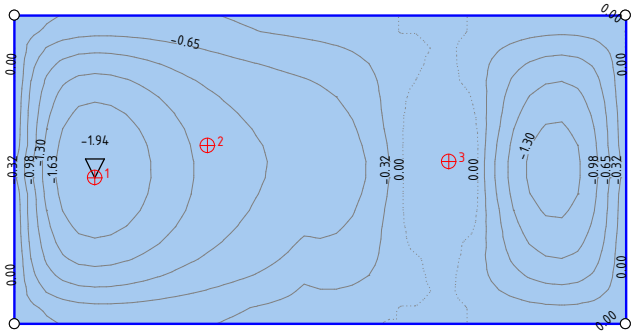
Nivo: [0.00 m]
Aa - sp.cona - Smer 1 - max Aa1,s= 2.81 cm²/m

Merodajna obtežba: Kompletna shema
EC2 (EN 1992-1-1:2004), C 25/30, S500B, a=6.00 cm



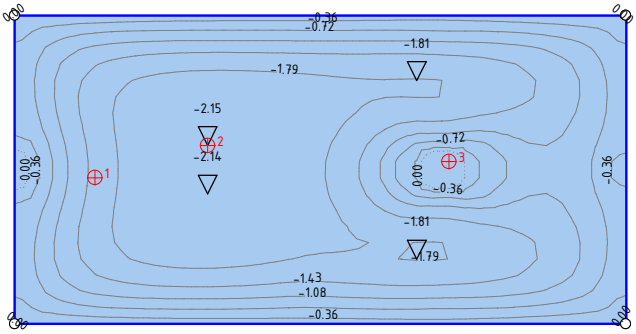
Nivo: [0.00 m]
Aa - sp.cona - Smer 2 - max Aa2,s= 1.16 cm²/m

Merodajna obtežba: Kompletna shema
EC2 (EN 1992-1-1:2004), C 25/30, S500B, a=6.00 cm



Nivo: [0.00 m]
Aa - zg.cona - Smer 1 - max Aa1,z= -1.94 cm²/m

Merodajna obtežba: Kompletna shema
EC2 (EN 1992-1-1:2004), C 25/30, S500B, a=6.00 cm



Nivo: [0.00 m]
Aa - zg.cona - Smer 2 - max Aa2,z= -2.15 cm²/m

Nivo: [0.00 m]
EC2 (EN 1992-1-1:2004)
d,pl=30.0 cm
C 25/30 (γC = 1.50, γS = 1.15) [SP]
Zgornja cona: S500B (a=6.0 cm)
Spodnja cona: S500B (a=6.0 cm)
Kompletna obtežna shema

Točka 1
X=1.64 m; Y=2.85 m; Z=0.00 m
Smer 1: (α=0°)
Merodajna kombinacija:
1.35xI+1.50xIV+0.90xV
Med = -20.03 kNm
Ned = 0.00 kN
εb/εa = -1.653/45.000 ‰
Az1 = 1.94 cm²/m
As1 = 0.00 cm²/m

Smer 2: (α=90°)
Merodajna kombinacija:
1.35xI+1.50xIV+0.90xV
Med = -15.93 kNm
Ned = 0.00 kN
εb/εa = -1.434/45.000 ‰
Az2 = 1.54 cm²/m
As2 = 0.00 cm²/m

Točka 2
X=3.72 m; Y=3.45 m; Z=0.00 m
Smer 1: (α=0°)
Merodajna kombinacija:
1.35xI+1.50xIV+0.90xV
Med = -12.35 kNm
Ned = 0.00 kN
εb/εa = -1.232/45.000 ‰
Az1 = 1.19 cm²/m
As1 = 0.00 cm²/m

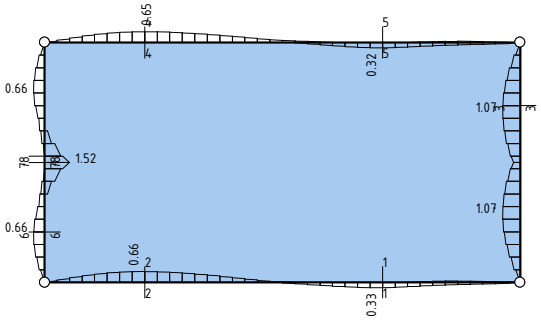
Smer 2: (α=90°)
Merodajna kombinacija:
1.35xI+1.50xIV+0.90xV
Med = -22.11 kNm
Ned = 0.00 kN
εb/εa = -1.762/45.000 ‰
Az2 = 2.15 cm²/m
As2 = 0.00 cm²/m

Točka 3
X=8.18 m; Y=3.15 m; Z=0.00 m
Smer 1: (α=0°)
Merodajna kombinacija:
1.35xI+1.05xII+1.50xIV+0.90xV
Med = 28.87 kNm
Ned = 0.00 kN
εb/εa = -2.113/45.000 ‰
Az1 = 0.00 cm²/m
As1 = 2.81 cm²/m

Smer 2: (α=90°)
Merodajna kombinacija:
1.35xI+1.05xII+1.50xIV
Med = 11.98 kNm
Ned = 0.00 kN
εb/εa = -1.210/45.000 ‰
Az2 = 0.00 cm²/m
As2 = 1.16 cm²/m

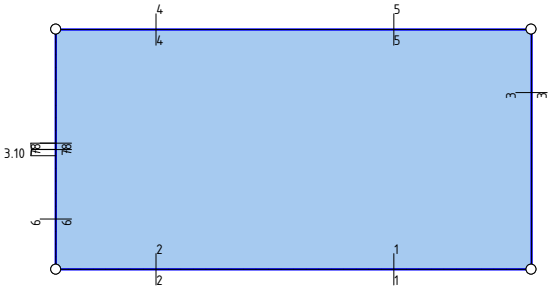
INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

Merodajna obtežba: Kompletna shema
EC2 (EN 1992-1-1:2004), C 25/30, S500B



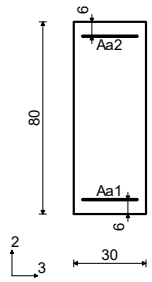
Nivo: [0.00 m]
Armatura v gredah: max Aa2/Aa1= 1.07 / 1.52 cm²

Merodajna obtežba: Kompletna shema
EC2 (EN 1992-1-1:2004), C 25/30, S500B



Nivo: [0.00 m]
Armatura v gredah: max Aa,st= 3.10 cm²

Greda 722-2
EC2 (EN 1992-1-1:2004)
C 25/30 (γC = 1.50, γS = 1.15) [SP]
S500B
Kompletna obtežna shema



[cm]

Prerez 1-1 x = 3.27m
Merodajna kombinacija za upogib:
1.35xI+1.05xII+1.50xIV+0.90xV
N1ed = -0.02 kN
M2ed = -0.02 kNm
M3ed = 10.44 kNm

Merodajna kombinacija za strig:
1.35xI+1.50xIV+0.90xV
V2ed = -7.22 kN
V3ed = 0.35 kN
M1ed = 0.00 kNm

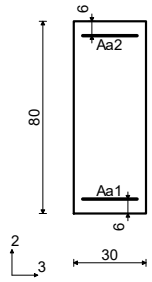
Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -0.633/45.000 ‰
Aa1 = 0.33 cm²
Aa2 = 0.00 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 0.00 cm²/m (m=1)

Prerez 2-2 x = 8.92m
Merodajna kombinacija za upogib:
1.35xI+1.50xIV
N1ed = 0.01 kN
M2ed = -0.01 kNm
M3ed = -21.05 kNm

Merodajna kombinacija za strig:
1.35xI+1.50xIV+0.90xV
V2ed = -7.71 kN
V3ed = 0.36 kN
M1ed = 0.00 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -0.920/45.000 ‰
Aa1 = 0.00 cm²
Aa2 = 0.66 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 0.00 cm²/m (m=1)

Greda 1007-722
EC2 (EN 1992-1-1:2004)
C 25/30 (γC = 1.50, γS = 1.15) [SP]
S500B
Kompletna obtežna shema



[cm]

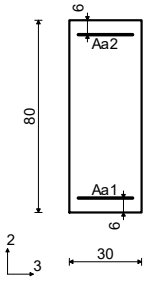
Prerez 3-3 x = 1.50m
Merodajna kombinacija za upogib:
1.35xI+1.50xIV+0.90xV
N1ed = -0.59 kN
M2ed = -0.00 kNm
M3ed = -34.35 kNm

Merodajna kombinacija za torzijo:
1.35xI+1.05xII+1.50xIII+0.75xIV
+0.90xV
M1ed = 0.06 kNm

Merodajna kombinacija za strig:
1.35xI+1.05xII+1.50xIV
V2ed = -3.08 kN
V3ed = 0.01 kN
M1ed = 0.06 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -1.218/45.000 ‰
Aa1 = 0.00 cm²
Aa2 = 1.07 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 0.00 cm²/m (m=1)

Greda 287-1007
EC2 (EN 1992-1-1:2004)
C 25/30 (γC = 1.50, γS = 1.15) [SP]
S500B
Kompletna obtežna shema



[cm]

Prerez 4-4 x = 2.38m
Merodajna kombinacija za upogib:
1.35xI+1.50xIV
N1ed = 0.02 kN
M2ed = -0.01 kNm
M3ed = -20.94 kNm

Merodajna kombinacija za strig:
1.35xI+1.50xIV+0.90xV
V2ed = -7.73 kN
V3ed = 0.15 kN
M1ed = 0.00 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -0.917/45.000 ‰

INŽENIRING BIRO ARMATURA
D.O.O., ZRKOVSKA CESTA 75, 2000 MARIBOR

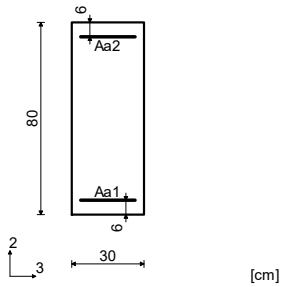
Aa1 = 0.00 cm²
Aa2 = 0.66 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 0.00 cm²/m (m=1)

Prerez 5-5 x = 8.03m
Merodajna kombinacija za upogib:
1.35xI+1.05xII+1.50xIV+0.90xV
N1ed = 0.22 kN
M2ed = -0.01 kNm
M3ed = 10.38 kNm

Merodajna kombinacija za strig:
1.35xI+1.50xIV+0.90xV
V2ed = 6.73 kN
V3ed = -0.15 kN
M1ed = 0.00 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -0.625/45.000 ‰
Aa1 = 0.33 cm²
Aa2 = 0.00 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 0.00 cm²/m (m=1)

Greda 2-287
EC2 (EN 1992-1-1:2004)
C 25/30 (γC = 1.50, γS = 1.15) [SP]
S500B
Kompletna obtežna shema



[cm]

Prerez 6-6 x = 1.20m
Merodajna kombinacija za upogib:
1.35xI+1.50xIV+0.90xV
N1ed = -0.61 kN
M2ed = 0.00 kNm
M3ed = -21.45 kNm

Merodajna kombinacija za torzijo:
1.35xI+1.05xII+1.50xIV+0.90xV
M1ed = -0.07 kNm

Merodajna kombinacija za strig:
1.35xI+1.50xV
V2ed = 5.42 kN
V3ed = 0.04 kN
M1ed = -0.07 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -0.933/45.000 ‰
Aa1 = 0.00 cm²
Aa2 = 0.66 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 0.00 cm²/m (m=1)

Prerez 7-7 x = 2.85m
Merodajna kombinacija za upogib:
1.35xI+1.05xII+1.50xIV
N1ed = -0.43 kN
M2ed = 0.00 kNm
M3ed = 48.43 kNm

Merodajna kombinacija za torzijo:
1.35xI+1.05xII+1.50xIII+0.75xIV
+0.90xV
M1ed = -0.07 kNm

Merodajna kombinacija za strig:
1.35xI+1.50xIV+0.90xV
V2ed = -87.07 kN
V3ed = 0.01 kN
M1ed = -0.07 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -1.495/45.000 ‰
Aa1 = 1.52 cm²
Aa2 = 0.00 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 3.02 cm²/m (m=1)

Prerez 8-8 x = 3.00m
Merodajna kombinacija za upogib:
1.35xI+1.05xII+1.50xIV
N1ed = -0.43 kN
M2ed = 0.00 kNm
M3ed = 35.34 kNm

Merodajna kombinacija za torzijo:
1.35xI+1.05xII+1.50xIII+0.75xIV
+0.90xV
M1ed = -0.07 kNm

Merodajna kombinacija za strig:
1.35xI+1.05xII+1.50xIV+0.90xV
V2ed = 89.51 kN
V3ed = 0.01 kN
M1ed = -0.07 kNm

Vrd,max,2 = 899.10 kN
Vrd,max,3 = 777.60 kN
εb/εa = -1.237/45.000 ‰
Aa1 = 1.10 cm²
Aa2 = 0.00 cm²
Aa3 = 0.00 cm²
Aa4 = 0.00 cm²
Aa,st = 3.10 cm²/m (m=1)

---

računali:

lesena konstrukcija Benedikt Boršič, univ. dipl. inž. grad.

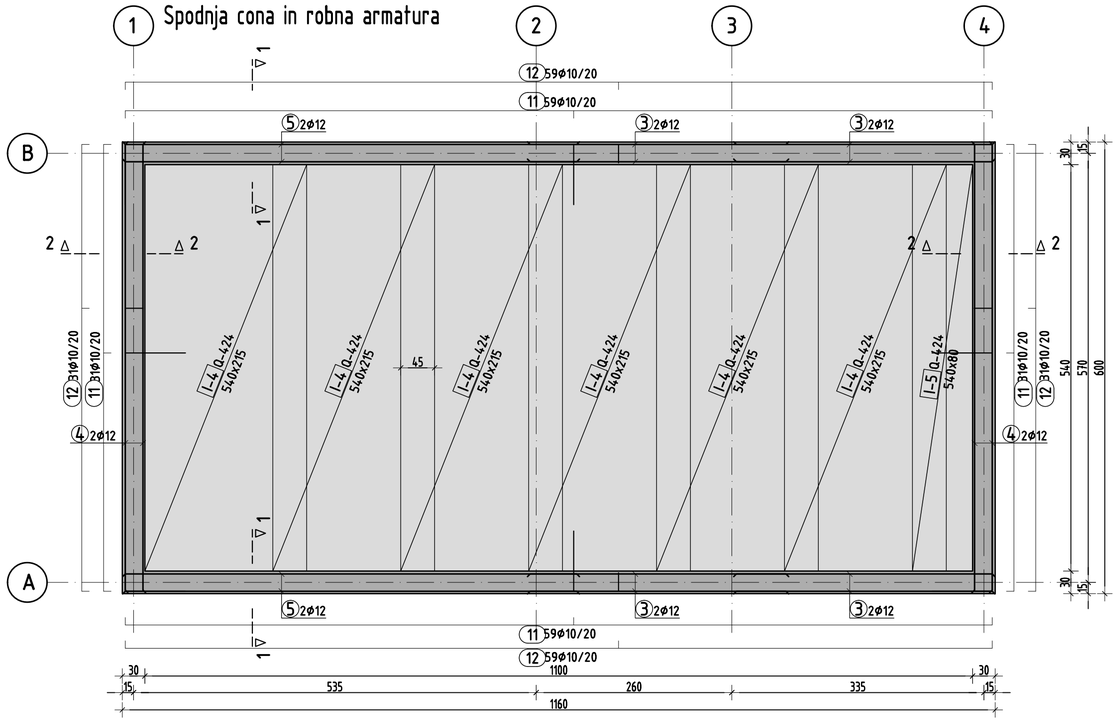
jeklena konstrukcija Uroš Žvan, univ. dipl. inža. gard.

temeljna plošča Lea Calliku, mag. inž. civ. (ITA)

TEMELJNA PLOŠČA IN PROTIZMRZLINSKI ZOB

M 1:50

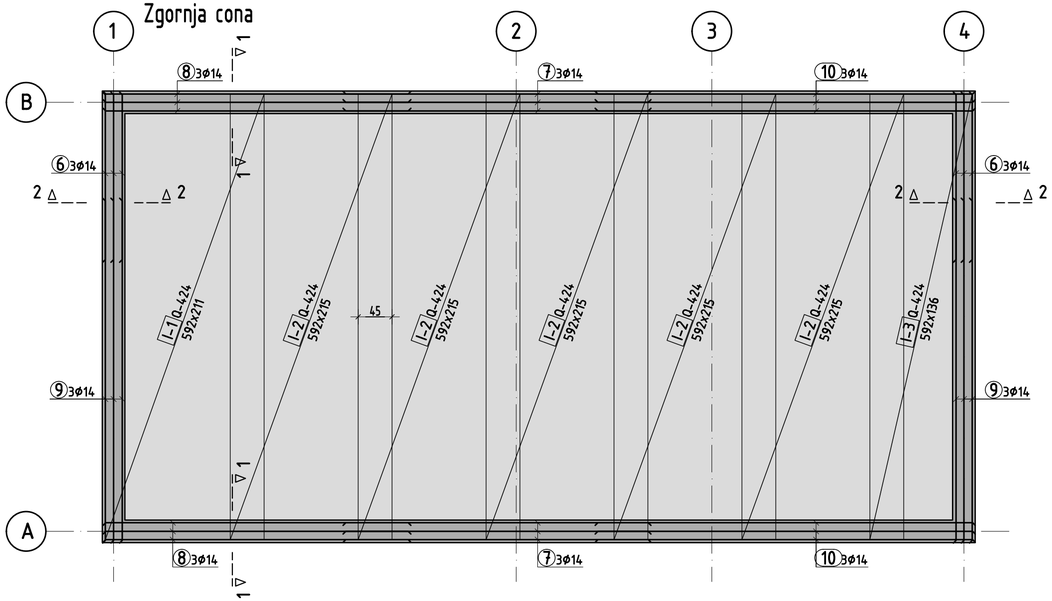
Spodnja cona in robna armatura



TEMELJNA PLOŠČA IN PROTIZMRZLINSKI ZOB

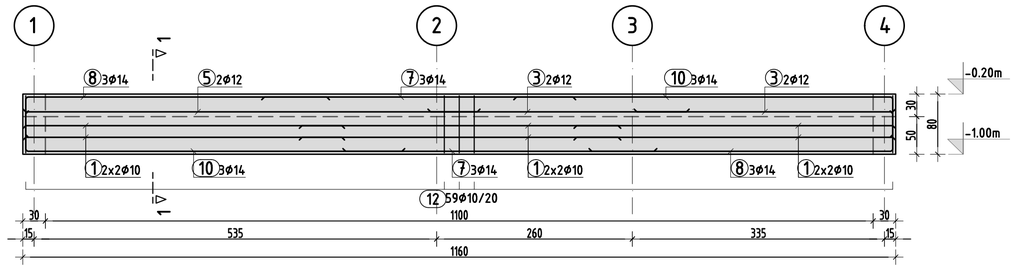
M 1:50

Zgornja cona



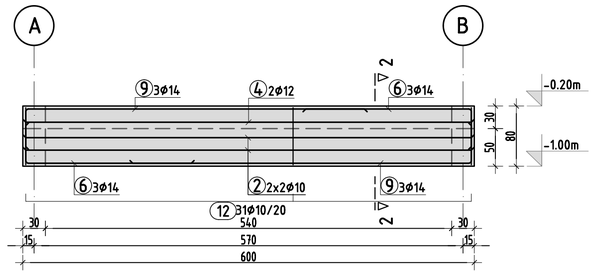
POGLED NA PROTIZMRZLINSKI ZOB V OSI A IN B

M 1:50



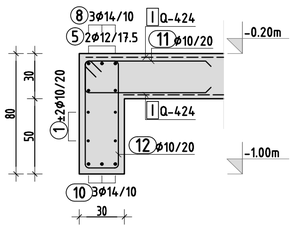
POGLED NA PROTIZMRZLINSKI ZOB V OSI 1 IN 4

M 1:50



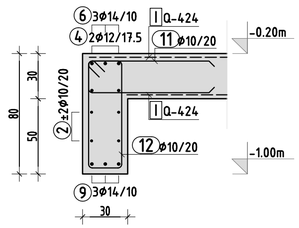
Prerez 1-1

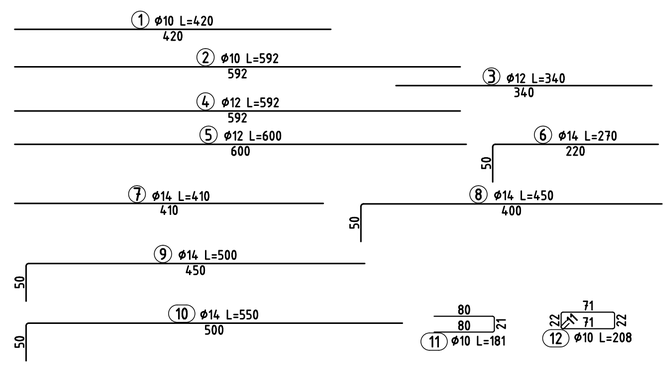
M 1:25



Prerez 2-2

M 1:25





UPORABLJENI MATERIALI:

-BETONI	C25/30 XC2 Cnom = 4,0 cm
-ARMATURA	S500B

OPOMBE:

Če preklopi niso kotirani upoštevati naslednje:

-Preklopi armaturnih palic min. 50xØ palice

- Distančniki in nosilci zgornje armature (npr. jahači, kače itd.) po izbiri izvajalca. Višina distančnikov 19,0cm.
- Morebitna naskladja med posamenimi načrti je potrebno nemudoma sporočiti odg. vodji projekta oziroma nadzorniku.
- Samovoljno prilagajanje projekta ni dovoljeno!

datum: spremememba:

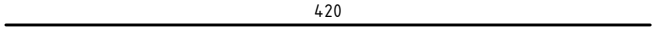
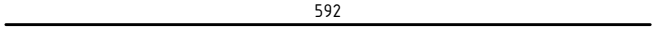
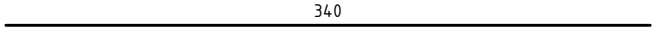
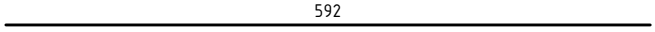
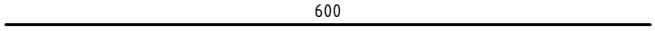
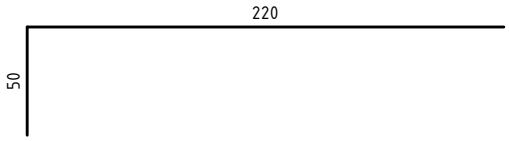
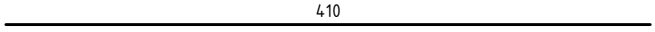
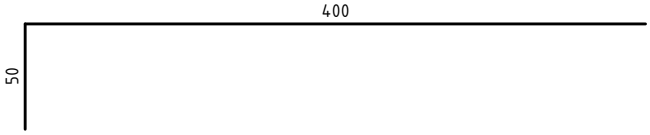
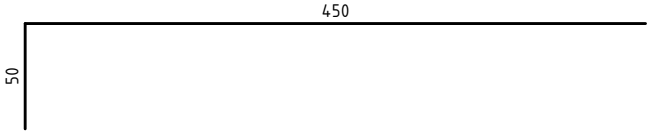
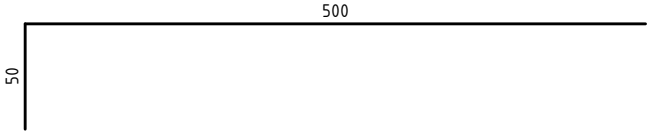
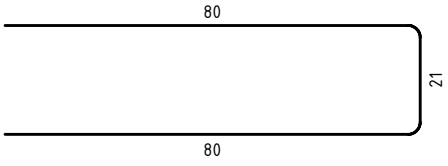
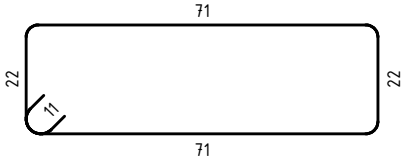
projektant:

INŽENIRING BIRO ARMATURA

INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (0)41 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN .UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	LEA CALLIKU, mag. ing. civ. (Ita)	
načrt, st.načrta:	GR. KONSTRUKCIJA	04/24-K
risba, merilo:	OPAŽNO-ARMATURNA RISBA TEMELJNA PLOŠČA	1:50 1:25
datum, št. lista:	OKTOBER 2025	1

Palice - specifikacija

ozn	oblika in mere [cm]	Ø	lg [m]	n [kos]	lgn [m]
temeljna plošča (1 pcs.)					
1		10	4.20	24	100.80
2		10	5.92	8	47.36
3		12	3.40	8	27.20
4		12	5.92	4	23.68
5		12	6.00	4	24.00
6		14	2.70	12	32.40
7		14	4.10	12	49.20
8		14	4.50	12	54.00
9		14	5.00	12	60.00
10		14	5.50	12	66.00
11		10	1.81	180	325.80
12		10	2.08	180	374.40

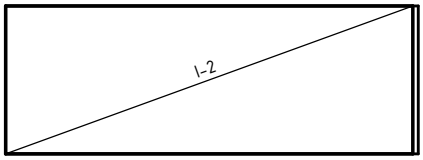
Palice - izvleček			
Ø [mm]	lgn [m]	Teža enote [kg/m']	Teža [kg]
S500B, Ø <= 12 mm			
10	848.36	0.62	523.44
12	74.88	0.89	66.49
Skupaj (S500B, Ø <= 12 mm)			589.93
S500B, Ø > 12 mm			
14	261.60	1.21	316.54
Skupaj (S500B, Ø > 12 mm)			316.54
Skupaj			906.47

Mreže - specifikacija						
Pozicija	Oznaka mreže	B [cm]	L [cm]	n	Teža enote [kg/m2]	Skupna teža [kg]
temeljna plošča (1 pcs.)						
I-1	Q-424	211	592	1	6.81	85.03
I-2	Q-424	215	592	5	6.81	433.20
I-3	Q-424	136	592	1	6.81	54.80
I-4	Q-424	215	540	6	6.81	474.18
I-5	Q-424	80	540	1	6.81	29.41
Skupaj						1076.61

Mreže - izvleček						
Oznaka mreže	B [cm]	L [cm]	n	Teža enote [kg/m2]	Skupna teža [BRUTO] [kg]	Skupna teža [NETO] [kg]
Q-424	215	600	14	6.81	1229.34	1076.61
Skupaj					1229.34	1076.61

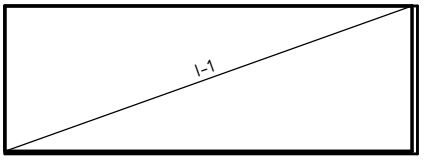
Mreže - načrt razreza
temeljna plošča
Q-424 (600 cm x 215 cm)

5x



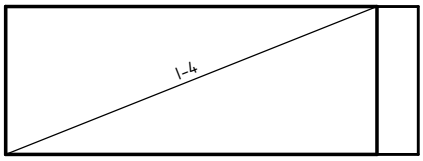
I-2 592 x 215

1x



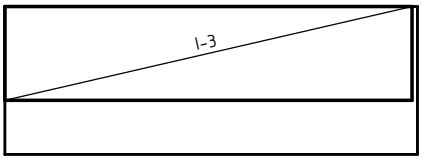
I-1 592 x 211

6x



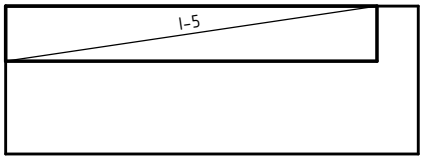
I-4 540 x 215

1x

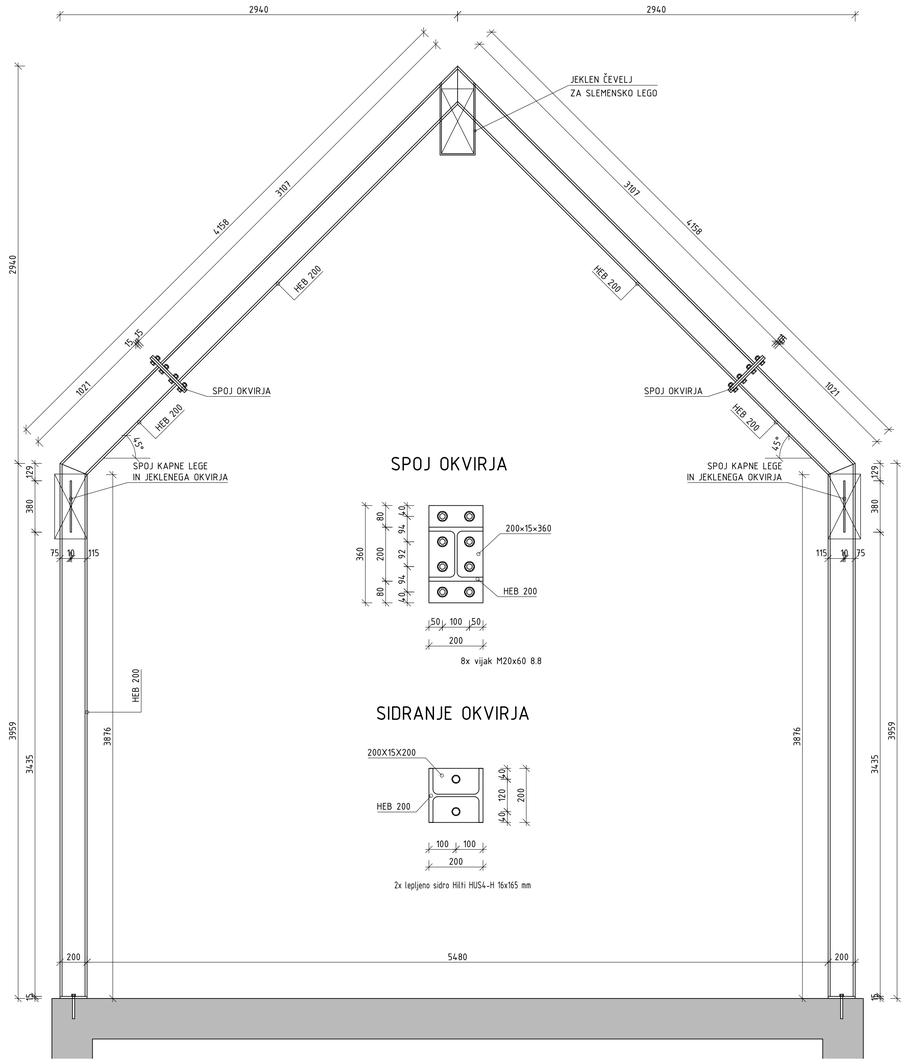


I-3 592 x 136

1x



I-5 540 x 80



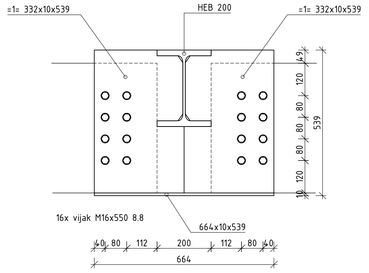
SPOJ OKVIRJA

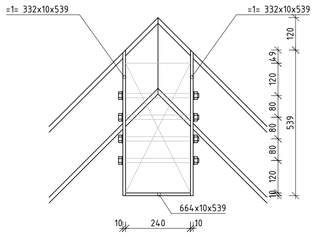
8x vijak M20x60 8.8

SIDRANJE OKVIRJA

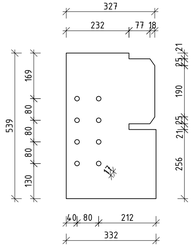
2x lepljeno sidro Hilti HUS4-H 16x165 mm

JEKLEN ČEVELJ
ZA SLEMENSKO LEGO

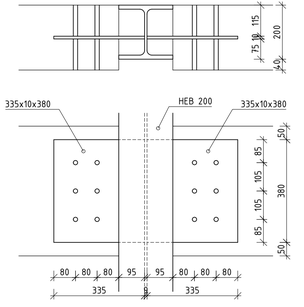




=1= 332x10x539 mm
4 kom.



SPOJ KAPNE LEGE
IN JEKLENEGA OKVIRJA



12x trn Φ16x240 S355

S 235 JR
EXC 2
protikorozijska zaščita trajnost H, korozivnost okolja C1 (ISO 12944)

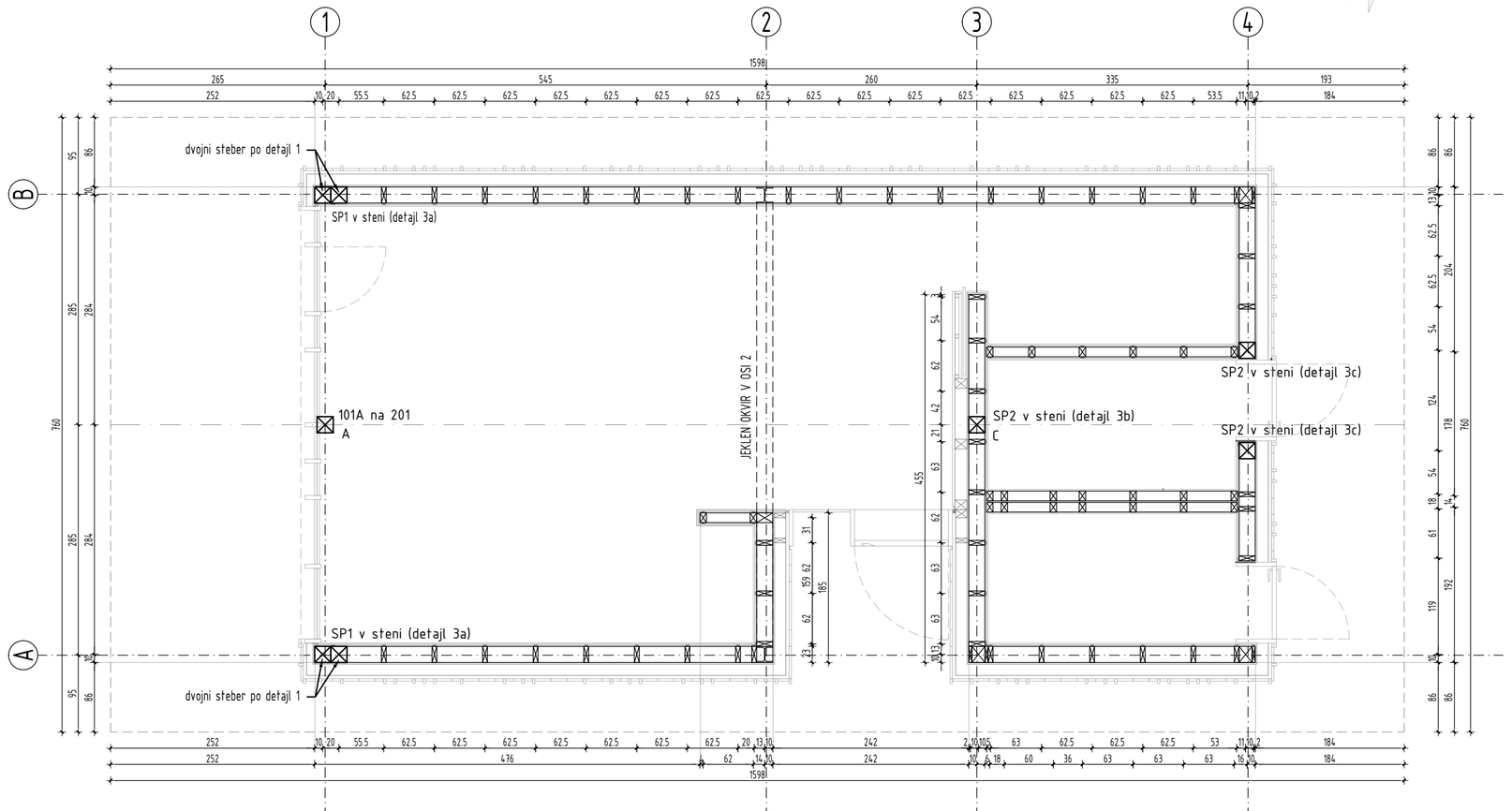
datum:	spremememba:

projektant:
INŽENIRING BIRO
ARMATURA
INŽENIRING BIRO ARMATURA d.o.o., ZIKOVSKA CESTA 35, MARIBOR
www.armatura-uzi.si info@armatura-uzi.si GSM: 041/ 435 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA KOLODVORSKA 18, 4280 KRANJSKA GORA	
objekt:	P-BGM BRUNARICA S PARKIRNIMI POVRŠINAMI - GM	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIC, univ. dipl. inž. arh.	ZAPS A-1430
pooblaščen inženir:	U. ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:		
načrt, št.načrta:	GRADBENE KONSTRUKCIJE	33/25-K
risba, merilo:	JEKLENI OKVIR	1:20 1:10
datum, št. lista:	OKTOBER 2025	2

TLORIS NOSILCEV IN STEBROV V STENAH

M 1:50



A - podpora 101 glej POZ 101A
C - podpora 101 glej POZ 101C
D - podpora 101 glej POZ 101D

UPORABLJENI MATERIALI:

-LES GL24

datum: sprememba:

projektant:

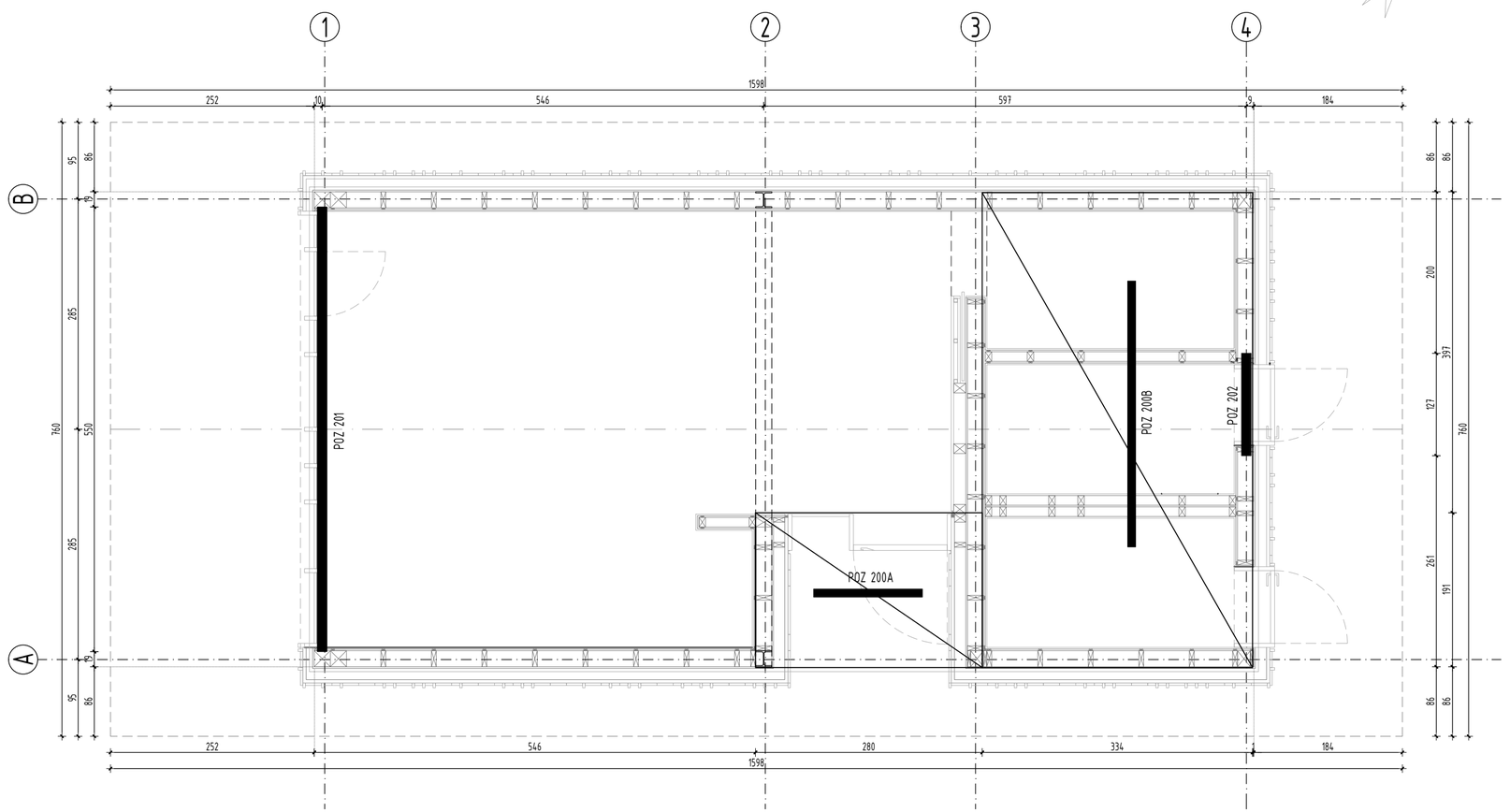
INŽENIRING BIRO ARMATURA

INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (041) 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad.	IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA	33/25-K
risba, merilo:	POZICIJSKI NAČRT NOSILCEV IN STEBROV V STENAH	1:50
datum, št. lista:	OKTOBER 2025	3

TLORIS STROPA

M 1:50



UPORABLJENI MATERIALI:

-LES GL24

datum: sprememba:

projektant:

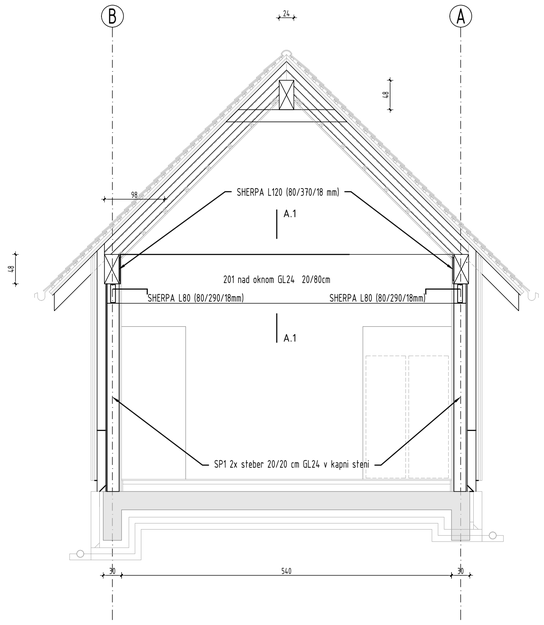
INŽENIRING BIRO ARMATURA

INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (041) 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad.	IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA	33/25-K
risba, merilo:	POZICIJSKI NAČRT STROPA	1:50
datum, št. lista:	OKTOBER 2025	4

POGLED NA NOSILNI SISTEM

M 1:50

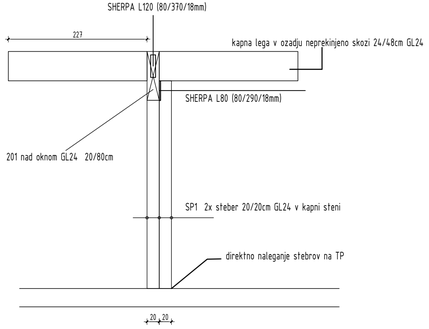


DETAJL 1

M 1:50

NALEGANJE IN STIKOVANJE OS 1 POZ 201

Prerez A.1

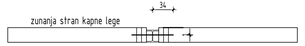


DETAJL 2

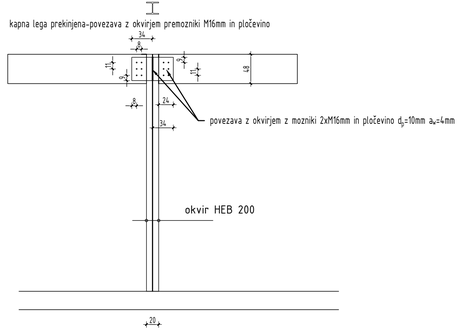
M 1:50

STIK os 2 kapna na okvir HEB 200 preko plošče v sredini z mozniki

Tloris:



Prerez A.1 - A.1 :



datum: sprememba:

projektant:

INŽENIRING BIRO ARMATURA

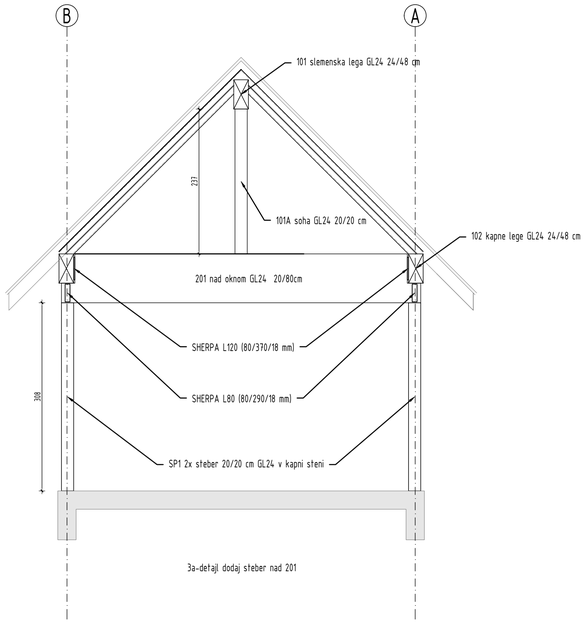
INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (0)41 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad.	IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA	33/25-K
risba, merilo:	POZICIJSKI NAČRT DETAJLA 1 IN 2	1:50
datum, št. lista:	OKTOBER 2025	5

POGLED NA NOSILNI SISTEM V OSI 1

DETAJL 3a

M 1:50

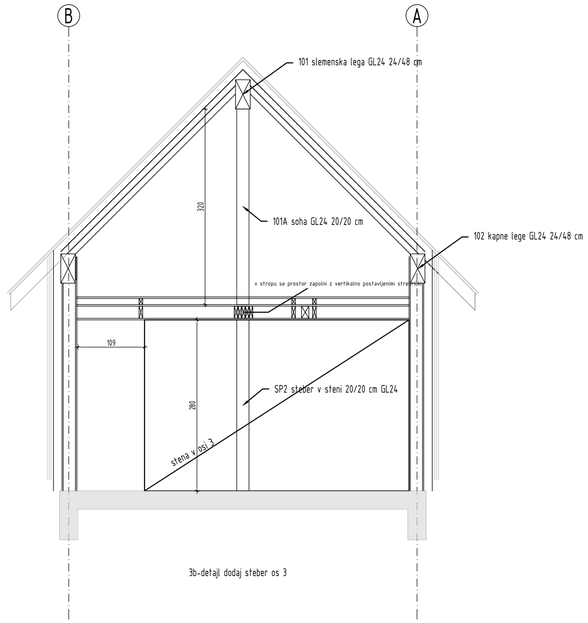


3a-detajl dodaj steber nad 201

POGLED NA NOSILNI SISTEM V OSI 3

DETAJL 3b

M 1:50

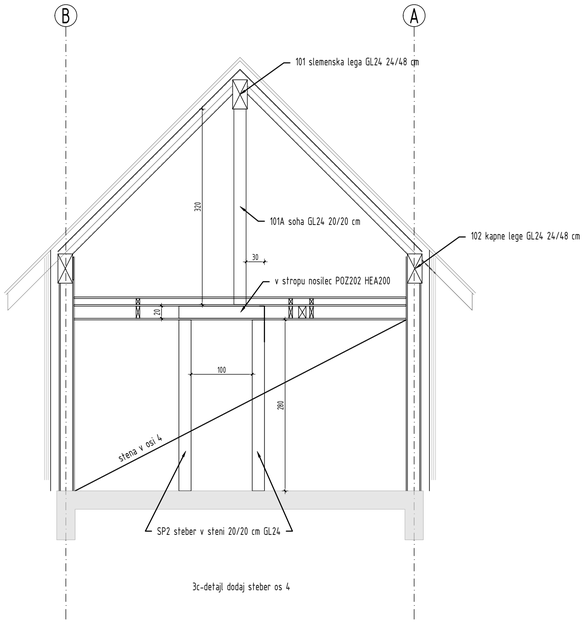


3b-detajl dodaj steber os 3

POGLED NA NOSILNI SISTEM V OSI 4

DETAJL 3c

M 1:50

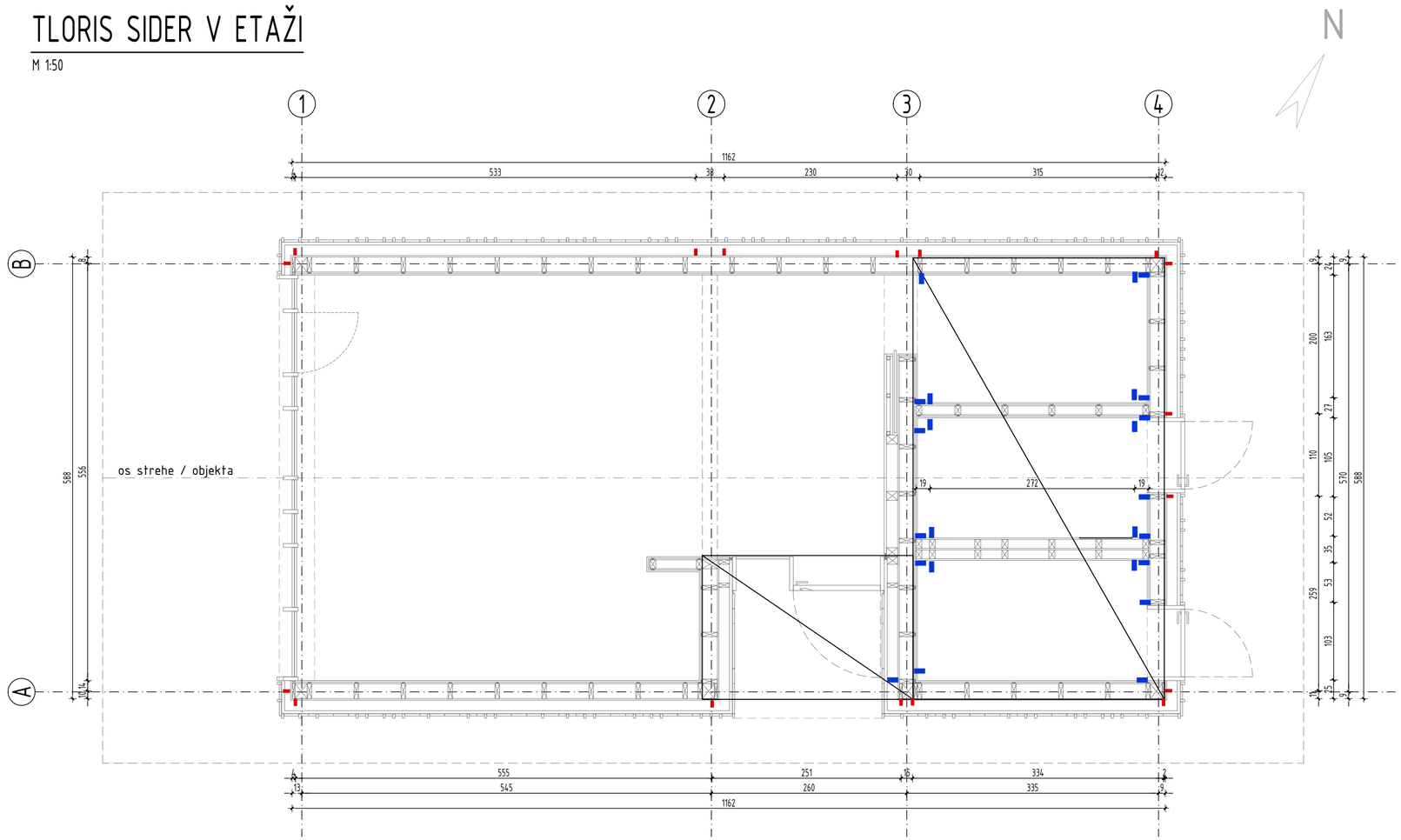


3c-detajl dodaj steber os 4

datum:	sprememba:
projektant:	INŽENIRING BIRO ARMATURA INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 35, MARIBOR www.armatura-uz.si info@armatura-uz.si +386 (0)41 425 341
št. projekta:	190-23 PZI
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO
vrsta projekta:	PZI
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh. ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad. IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad. IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA 33/25-K
risba, merilo:	POZICIJSKI NAČRT DETAJLA 3 1:50
datum, št. lista:	OKTOBER 2025 6

TLORIS SIDER V ETAŽI

M 1:50



IZBRANA SIDRA:

- Wurth Typ A-100 ; CNA 24 x 4/50
- \+ Pločevinast trak zunaj LOCHPLATTE 200/120/2mm ; Kammnägel 2x 12 x 4/50

VMESNA SIDRA NA e< 1,25m:
2x Wurth ASSZ plus VG 6x 160 e=1.25 m

datum: sprememba:

projektant:

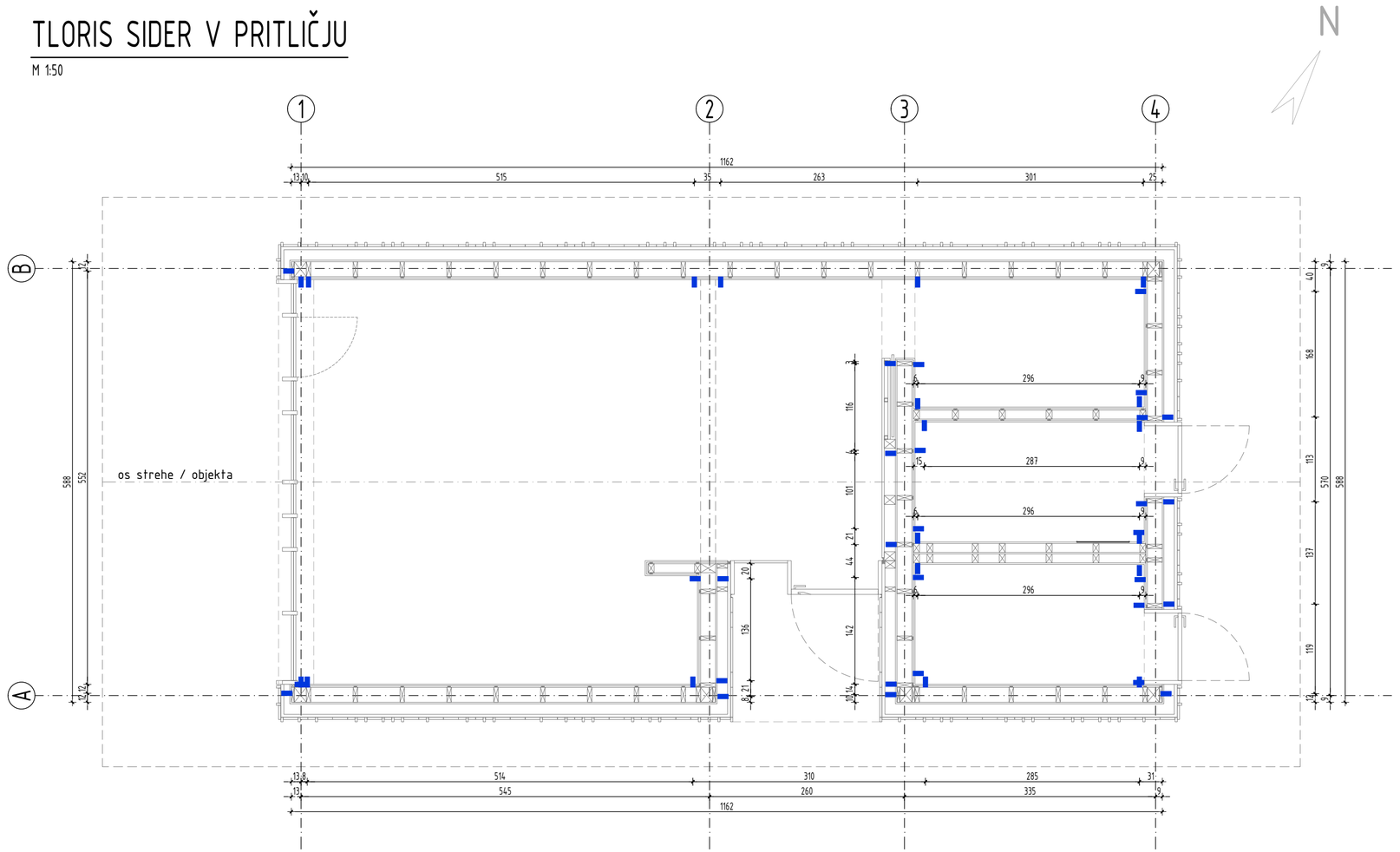
INŽENIRING BIRO ARMATURA

INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (041) 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad.	IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA	33/25-K
risba, merilo:	POZICIJSKI NAČRT SIDER V ETAŽI	1:50
datum, št. lista:	OKTOBER 2025	7a

TLORIS SIDER V PRITLIČJU

M 1:50



IZBRANA SIDRA:

1x WURTH Winkelverbinder Typ V 285 + sidrni žičniki (Ankernägel) Würht 4/40 + sidrni vijak Wurth W-FA/S M12

VMESNA SIDRA: konstruktivno vse stene e < 1,25m Wurth Typ A-100; žičniki CNA 24 x 50 in sidrni vijak Wurth W-FA/S M10

datum: sprememba:

projektant:

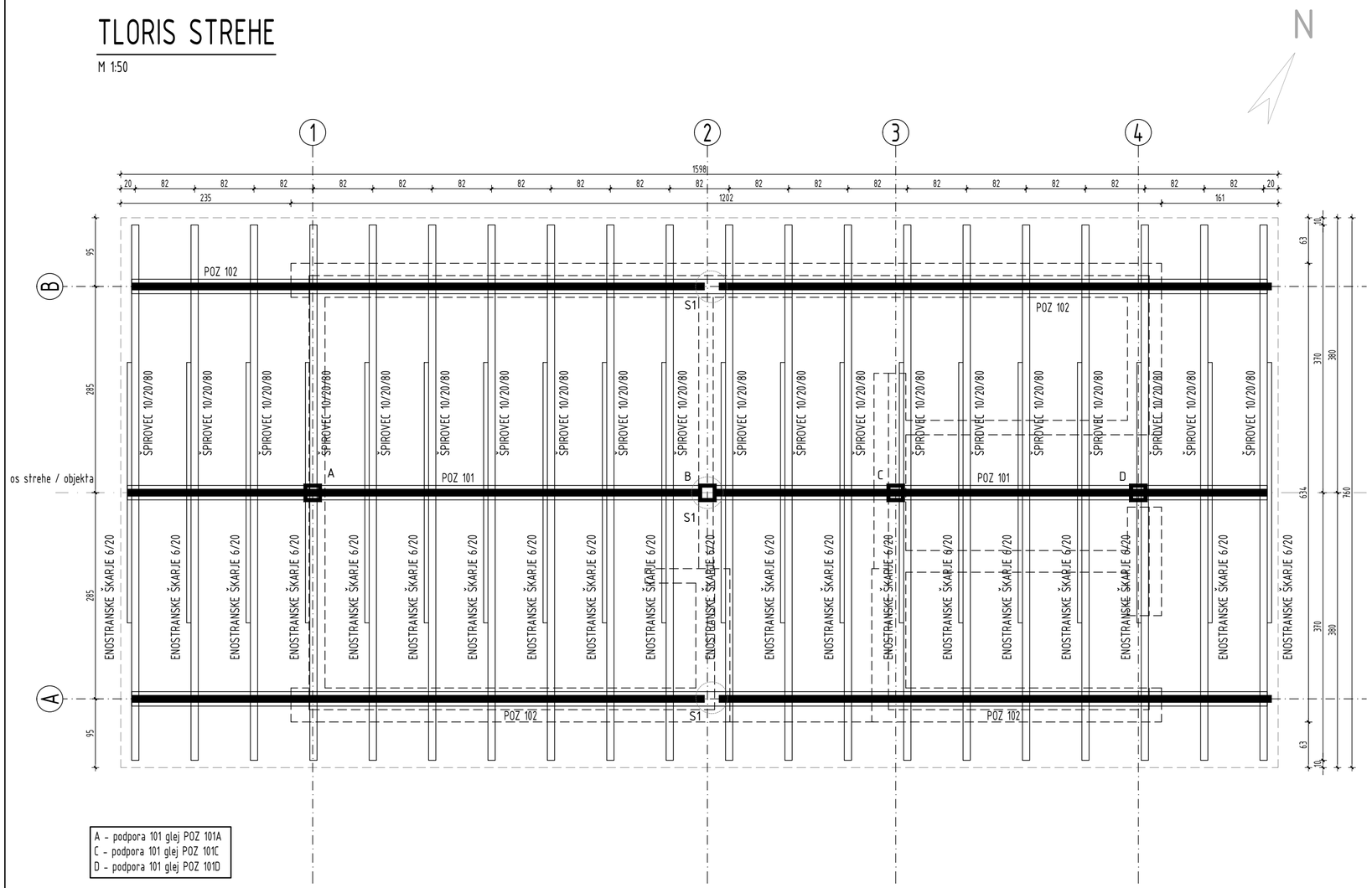
INŽENIRING BIRO ARMATURA

INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (041) 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad.	IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA	33/25-K
risba, merilo:	POZICIJSKI NAČRT SIDER V PRITLIČJU	1:50
datum, št. lista:	OKTOBER 2025	7b

TLORIS STREHE

M 1:50



A - podpora 101 glej POZ 101A
C - podpora 101 glej POZ 101C
D - podpora 101 glej POZ 101D

OPOMBA: STREŠNA KONSTRUKCIJA OZ. TUDI OSTALE KONSTRUKCIJE SE LAHKO OPCIJSKO IZVEDE IZ KRIŽNO LEPLJENIH PLOŠČ.

datum:	spremememba:

projektant:
INŽENIRING BIRO ARMATURA
INŽENIRING BIRO ARMATURA d.o.o., ZRKOVSKA CESTA 75, MARIBOR
www.armatura-uz.si info@armatura-uz.si +386 (041) 425 341

št. projekta:	190-23 PZI	
investitor:	OBČINA KRANJSKA GORA Kolodvorska 1b, 4280 Kranjska Gora	
objekt:	P-BGM-BRUNARICA S PARKIRNIMI POVRŠINAMI-GM NOVOGRADNJA BRUNARICE S PARKIRIŠČI IN PRIPADAJOČ. ZUN. UREDITVIJO	
vrsta projekta:	PZI	
vodja projekta:	JERNEJ ŽIVIČ, univ. dipl. inž. arh.	ZAPS 2071
pooblaščen inženir:	UROŠ ŽVAN, univ. dipl. inž. grad.	IZS G-0028
sodelavec:	BENEDIKT BORŠIČ, univ. dipl. inž. grad.	IZS G-0284
načrt, st.načrta:	GR. KONSTRUKCIJA	33/25-K
risba, merilo:	POZICIJSKI NAČRT OSTREŠJA	1:50
datum, št. lista:	OKTOBER 2025	8