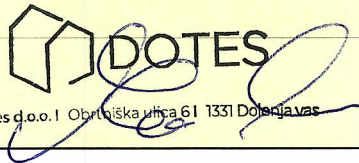


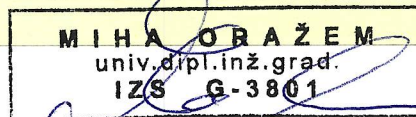
NASLOVNA STRAN

NAČRTA



2. Načrt s področja gradbeništva

PODATKI O GRADNJI	
naziv gradnje	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA
kratek opis gradnje	Gradnja prizidka igralnic in kuhinje k obstoječemu kompleksu vrtca ter rekonstrukcija in obnova notranjih prostorov v obstoječih igralnicah.
VRSTE GRADNJE	☐ NOVOGRADNJA - NOVOZGRAJEN OBJEKT
označiti vse ustrezne vrste gradnje	☒ NOVOGRADNJA - PRIZIDAVA
	☒ REKONSTRUKCIJA
	☐ SPREMEMBA NAMEMBNOСТИ
	☐ ODSTRANITEV CELOTNEGA OBJEKTA
	☐ LEGALIZACIJA
	☐ MANJŠA REKONSTRUKCIJA
PODATKI O PROJEKTNIM DOKUMENTACIJI	
vrsta dokumentacije	PZI - projekt za izvedbo
številka projekta	01 / 2025
PODATKI O NAČRTU	
strokovno področje načrta	2 - gradbeništvo
naziv načrta	Načrt s področja gradbeništva
številka načrta	18/2026
datum izdelave	maj 2026
datum spremembe	
PODATKI O PROJEKTANTU NAČRTA	
projektant načrta (naziv družbe)	DOTES d.o.o.
naslov	Obrtniška ulica 6, 1331 Dolenja vas
odgovorna oseba projektanta načrta	Miha Oražem
podpis odgovorne osebe projektanta načrta	 Dotes d.o.o. Obrtniška ulica 6 1331 Dolenja vas
PODATKI O IZDELOVALCU NAČRTA	
ime in priimek pooblaščenega arhitekta, pooblaščenega inženirja	Miha Oražem, univ. dipl. inž. grad.
identifikacijska številka	IZS G-3801



PRILOGA 2C

IZJAVA PROJEKTANTA NAČRTA IN POOBLAŠČENEGA STOKOVNJAKA, KI JE IZDELAL NAČRT V PZI IN PID

PROJEKTANT NAČRTA

projektant načrta (naziv družbe)	DOTES d.o.o.
naslov	Obrtniška ulica 6, 1331 Dolenja vas
odgovorna oseba projektanta načrta	Miha Oražem

IN POOBLAŠČENI STROKOVNJAK, KI JE IZDELAL NAČRT

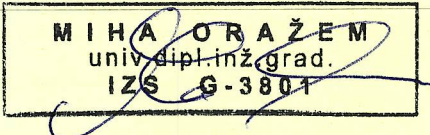
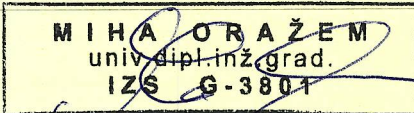
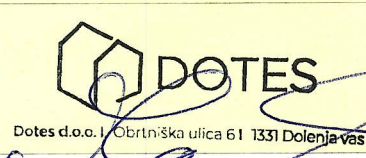
pooblaščen strokovnjak	Miha Oražem, univ. dipl. inž. grad.
------------------------	-------------------------------------

IZJAVLJAVA:

da načrt

vrsta dokumentacije	PZI - projekt za izvedbo
strokovno področje načrta	2 - gradbeništvo
naziv načrta	Načrt s področja gradbeništva
številka načrta	18/2026
datum izdelave	maj 2026

upošteva relevantne predpise in druge normativne dokumente ter da so upoštewane ustrezne bistvene in druge zahteve.

pooblaščen strokovnjak	Miha Oražem, univ. dipl. inž. grad.
identifikacijska številka	IZS G-3801
podpis pooblaščenega strokovnjaka	 
odgovorna oseba projektanta načrta	Miha Oražem
podpis odgovorne osebe projektanta načrta	 

2.2	KAZALO VSEBINE NAČRTA S PODROČJA GRADBENIŠTVA
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2.2	Kazalo vsebine načrta	
2.3	Tehnično poročilo	
2.4	Statični izračun – TEMELJNE KONSTRUKCIJE	
2.5	Statični izračun – LESENE IN JEKLENE KONSTRUKCIJE	
2.6	Tehnični prikazi	
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	15. KUHINJA - TLORIS - NOSILNA KONSTRUKCIJA	M 1:50
	16. KUHINJA - TLORIS PRITLIČJA - SIDRANJE	M 1:50
	17. KUHINJA - PREREZI	M 1:50
	18. KUHINJA - 3D POGLEDI - NOSILNA KONSTRUKCIJA	M 1:50

Projektna naloga

Priložena tehnična dokumentacija obravnava načrt s področja gradbeništva novega prizidka vrtca in kuhinje k obstoječemu vrtcu v Mežici. Prizidka sta enoetažna. Prizidek vrtca ima dvokapno streho z neizkoriščen podstrešjem, kjer se vodijo razne inštalacije. Na južni strani prizidka vrtca je zasnovana tudi terasa z jekleno pergolo. Prizidek kuhinje ima ravno streho z minimalnim naklonom na kateri stoji prezračevalne naprave in druge inštalacije. Sistem vertikalne lesene konstrukcije je zasnovan v leseni skeletni izvedbi različnih debelin sten, ki so obojestransko opaščene z uporabo OSB in Farmacell plošč. Medetažna konstrukcija prizidka vrtca je zasnovana iz križno lepljenih plošč, ki je mestoma ojačena z uporabo lesenih in jeklenih nosilcev. Ravna streha prizidka kuhinje je zasnovana iz križno lepljenih plošč. Temeljenje obeh prizidkov je izvedeno na temeljni plošči z dvignjenimi parapeti pod stenami zaradi zagotovitve trajnosti konstrukcije (hidroizolacija). Zunanje tlorisne dimenzije prizidka vrtca znašajo 28,02 x 11,06 m, prizidka kuhinje pa 10,90 x 6,62 m.

Lokacija in projektna izhodišča

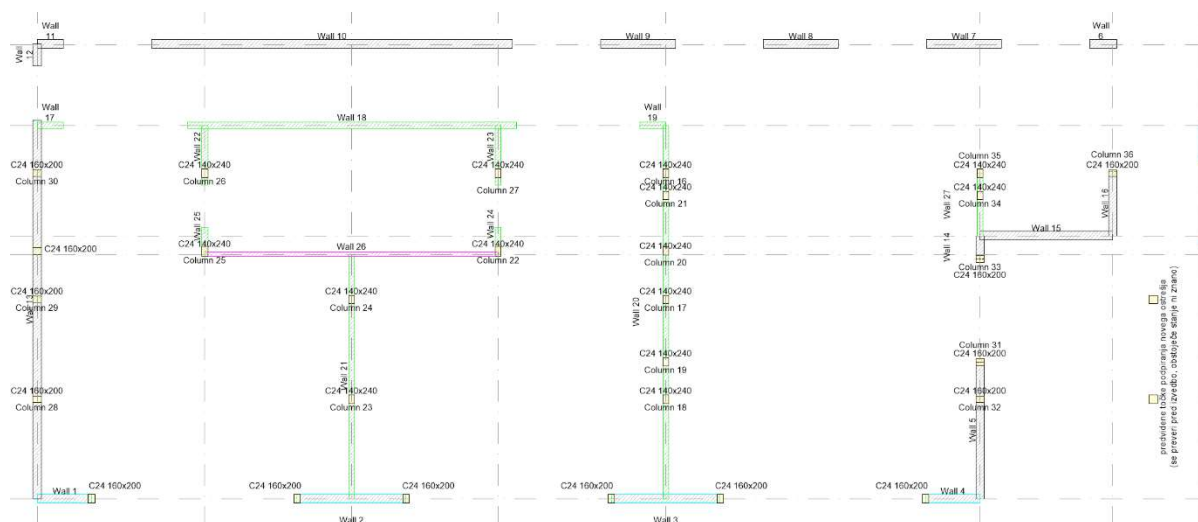
Iz naslova lokacije objekta je pri izdelavi načrta gradbenih konstrukcij upoštevano naslednje:

- objekt se nahaja v I coni delovanja vetra po SIST EN 1991-1-4, kategorija terena II, nadmorska višina cca 485 m,
- vpliv obtežbe snega, cona A3, nadmorska višina cca 485 m, po SIST EN 1991-1-3,
- delovanja potresa $a_g = 0,150$ g, tip temeljnih tal B, po SIST EN 1998-1.

Konstrukcijska zasnova – prizidek vrtca

Vertikalna nosilna konstrukcija prizidka vrtca je zasnovna v skeletni leseni izvedbi. Načelno govorimo okvirjih konstrukcijske širine 100 mm, 140 mm in 200 mm. Spodnja slika prikazuje konstrukcijske sisteme vertikalnih nosilnih elementov prizidka vrtca:

- **črna** barva ... okvir 200 ... pokončniki 80/200 na razmaku 625 mm
- **turkizna** barva ... okvir 200 ... pokončniki robni 160/200, pokončniki vmesi 80/200 na razmaku 625 mm
- **zelena** barva ... okvir 140 ... pokončniki 80/140 na razmaku 625 mm
- **vijola** barva ... okvir 100 ... pokončniki 80/100 na razmaku 417 mm



Slika 1: Konstrukcijski sistem vertikalnih nosilnih elementov prizidka vrtca

2.3	TEHNIČNO POROČILO	2/6
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Sidranje lesenih vertikalnih stenskih elementov je zasnovano preko dvižnih in strižnih povezav:

- dvižni kotniki WKR 53035 ... povezava s steno 16 x žebelj LBA 4 * 60 mm, povezava s temelji 1 x navojna palica M 12 (5.8) x 195 + kemična masa HYB-FIX
- strižna sidra... sidrni vijaki M 16 * 240 mm + kemična masa HYB-FIX skozi zgornjo pasnico na maksimalni razdalji 1000 mm

Medetažna nosilna konstrukcija je zasnovana iz 5 slojnih križno lepljenih plošč debeline 140 mm (BBS XL 140 5s, 40-20-20-20-40), ki so orientirane v prečni smeri objekta. Zaradi vmesnega podpiranja so medetažne plošče vmesno tudi podprte preko navzgor obrnjenih lesenih nosilcev 26/28 cm (GL 28h) in jeklenih nosilcev HEB 240 (S235). Plošče se navzgor obešajo z uporabo navojnih palic M 16(8.8) na razmiku 300 mm.

Sidranje sten v medetažno nosilno konstrukcijo se izvede preko dvižnih in strižnih povezav:

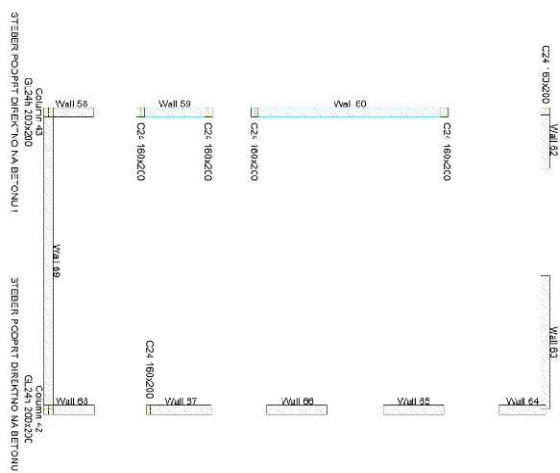
- natezni trakovi 60x1,5 mm (S350) ... povezava s steno in ploščo 18 x žebelj LBA 4 * 60 mm
- strižna sidra ... lesni vijaki HBS 8 * 200 mm na maksimalni razdalji 300 mm
- dvojni dvižni kotnik WKR 285 ... povezava s steno spodaj 16 x žebelj LBA 4 * 60 mm, povezava s steno zgoraj 16 x žebelj LBA 4 * 60 mm, povezava preko medetažne plošče svorniki M 12 (5.8)

Strešna konstrukcija lesenega dvokapnega ostrešja je sestavljena iz vmesnih leg dimenzij 20/40 cm (GL 28h) oziroma 20/24 cm (GL 24h) in špirovcev 10/18 cm (C 24) na medsebojnem razmiku cca 92 cm. **Strešne lege dimenzij 20/24 cm (GL 24h) je potrebno v delu obstoječega objekta podpirati preko dveh stebričev, kar pa je bilo v fazi projektiranja zaradi nezmožnosti ugotovitve obstoječega stanja onemogočeno. To je nujno potrebo izvesti med izvedbo in o tem obvestiti pooblaščenega inženirja s področja gradbeništva.** Vmesne lege podpirajo sohe dimenzij 20/20 cm (C 24). Kapne lege v tem delu objekta so zasnovane v dimenzijah 16/16 cm (C 24) in preko skeletne lesene stene debeline 20 cm, ki je na vrhu zaključena z lego 20/20 cm (C 24). Vzdržno zavetorovanje strehe je zasnovano z uporabo ročic 16/16 cm (C 24).

Konstrukcijska zasnova – prizidek kuhinje

Vertikalna nosilna konstrukcija prizidka kuhinje je zasnovna v skeletni leseni izvedbi debeline 200 mm. Spodnja slika prikazuje konstrukcijske sisteme vertikalnih nosilnih elementov prizidka kuhinje:

- **črna** barva ... okvir 200 ... pokončniki 80/200 na razmaku 625 mm
- **turkizna** barva ... okvir 200 ... pokončniki robni 160/200, pokončniki vmesi 80/200 na razmaku 625 mm

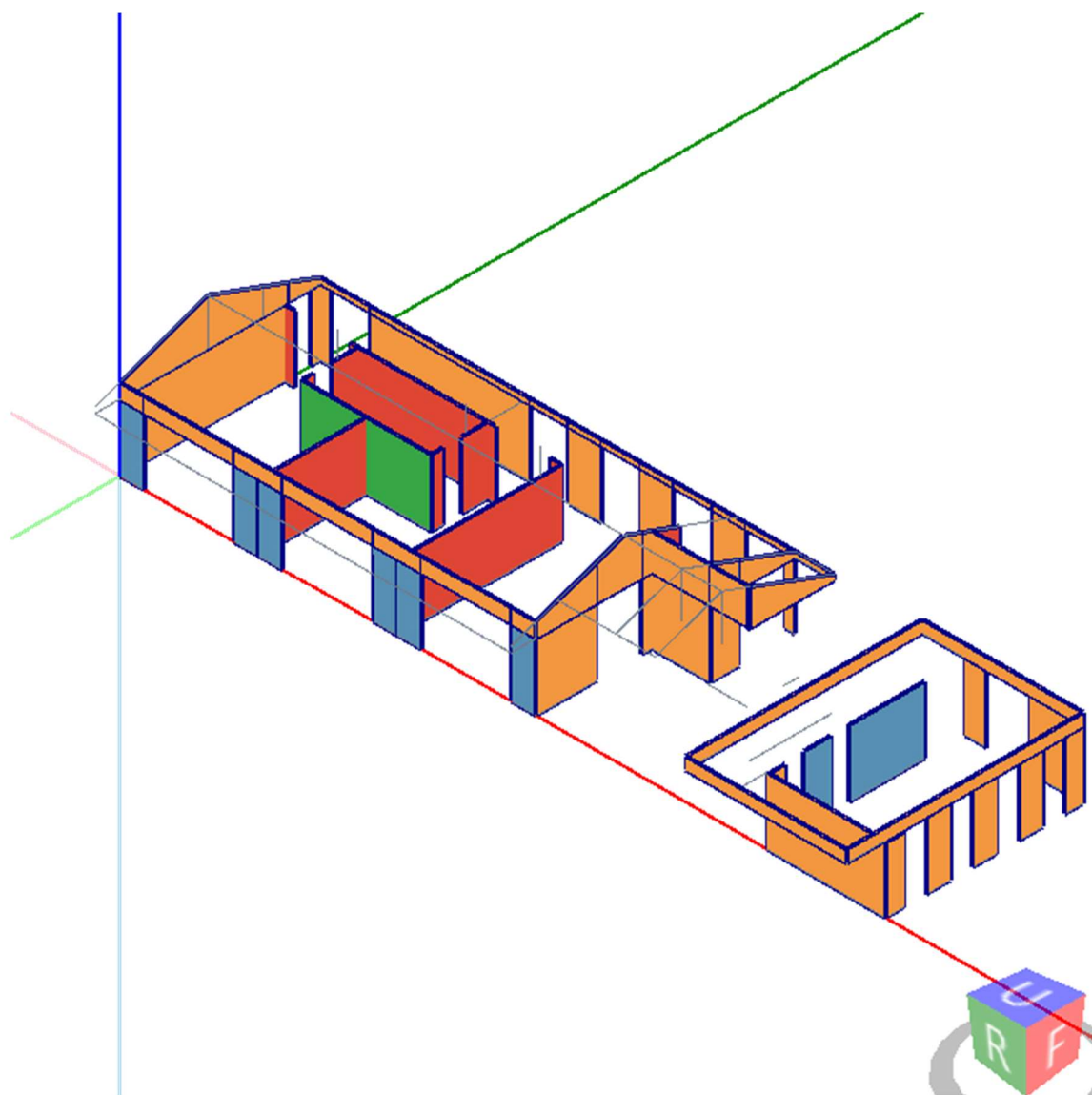


Slika 2: Konstrukcijski sistem vertikalnih nosilnih elementov prizidka kuhinje

Strešna nosilna konstrukcija je zasnovana iz 5 slojnih križno lepljenih plošč debeline 220 mm (BBS XL 220 5s, 60-30-40-30-60), ki so orientirane v prečni smeri objekta. Zaradi podpiranja teh plošč, ki segajo v konzole sta zasnovana dva lepljena lamelirana nosilca (os 1 in 2) dimenzij 20/52 cm (GL 24h), le ta pa sta postavljena na vrhu zunanjih sten v istih oseh.

Sidranje sten v strešno nosilno konstrukcijo se izvede preko strižnih povezav:

- strižna sidra ... lesni vijaki HBS 10 * 280 mm na maksimalni razdalji 300 mm



Slika 3: Konstrukcijski model – vertikalne nosilne konstrukcije – skeletne stene

2.3	TEHNIČNO POROČILO	4/6
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Splošna navodila za vijačenje lesene konstrukcije in rezkanje konstrukcijskih elementov

Vogalni in T-spoji sten se vijačijo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji $e=300\text{mm}$ ali z navojnimi palicami M12 (5.8) na največji razdalji $e=1000\text{mm}$. Vsak špirovec se vijači v steno ali lego s samovreznim lesnim vijakom premera Ø8 in s povečano glavo ali podložko. Vsaka strešna lega, nosilec ali preklada, ki je vzporedna steni, se vijači v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji $e=300\text{mm}$. Medetažne križno lepljene plošče se v stene vijačijo z vijaki premera Ø8 na največji razdalji $e=300\text{mm}$. Medetažne križno lepljene plošče se na L-preklopih medsebojno vijačijo z vijaki premera Ø8 na največji razdalji $e=250\text{mm}$. Stebri in ojačitve ter vsi konstrukcijski elementi se vijačijo z vijaki premera Ø8 na največji razdalji $e=300\text{mm}$,

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Križno lepljene (CLT) plošče se ne sme dolžinsko rezkati brez pisne odobritve pooblaščenega inženirja s področja gradbeništva. Točkovne preboje, večje od premera 100 mm, mora ravno tako pisno odobriti projektant. Pri statičnem izračunu lesene konstrukcije oslabitve in preboji, ki niso vrisani, niso bili upoštevani. Kjer je dovoljeno rezkanje kanalov v križno lepljene stene in plošče, se te izvaja vzporedno z usmeritvijo desk. v nobenem primeru se ne reže sten in plošč prečno na usmeritev desk.

Požarna odpornost lesene in jeklene konstrukcije

Celotna konstrukcija ima požarno odpornost najmanj R30. Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo. Notranja stena debeline 100 mm je obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm. Zunanje stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60. Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60. Jekleni deli konstrukcije morajo biti zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odpornosti najmanj R30.

Obtežbe in izračun:

Obtežbe so določene po SIST EN 1991 in tehnoloških zahtevah.

KORISTNA OBTEŽBA:

namembnost	kategorija	[kN/m ²]
Servis strehe	H	0,40
Neizkoriščena mansarda prizidka vrtca	A	1,00
Hodniki, dostopi, kuhinja	C3	5,00
Igralnice	C1	3,00

OBTEŽBA SNEGA:

Lokacija	Ljubljana	
Cona	A3	
n.m.v.	485	[m]
Ce	1	(običajen)
Ct	1	
sk	2,79	[kN/m ²]

2.3	TEHNIČNO POROČILO	5/6
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Na mestih kjer je morebitno lahko preprečen iztok vode iz streh (povišane atike) je obtežba snega povečana z faktorjem 1,50, kot to predvideva SIST EN 1991-1-3.

POTRESNA OBTEŽBA:

projektni pospešek	ag	0,150	g
tip tal	S	B	
faktor obnašanja	q	2,50	
kat. pomembnosti		II	
fak. pomembnosti	γ_{II}	1,00	

Kombinacijo največjih vplivov posameznih nihajnih oblik za posamezno smer izvedemo s kvadratno SRSS kombinacijo. Posamezne kvadratne kombinacije SRSS v posamezni smeri se kombinirajo na naslednji način:

$$\begin{aligned} \text{PPS V X:} & \quad 1,00 \cdot E_x + 0,30 \cdot E_y \\ \text{PPS V Y:} & \quad 0,30 \cdot E_x + 1,00 \cdot E_y \end{aligned}$$

Izračun je bil narejen za naslednja projektna stanja:

- mejno stanje nosilnosti MSN,
- mejno stanje uporabnosti MSU,
- potresno projektno stanje PPS.

Montažni elementi:

Za izvedbo montažnih konstrukcij objekta je potrebno izdelati delavniške načrte jeklenih in lesenih konstrukcijskih elementov, ki jih mora potrditi pooblaščen inženir s področja gradbeništva. Krojenje lesene konstrukcije je predvideno strojno, na podlagi izdelanih delavniških načrtov in v kontroliranem okolju v delavnici. Med in po montaži lesene konstrukcije se mora ta zaščititi pred zunanjimi vplivi. Dela pri izdelavi in montaži lesenih konstrukcij mora opravljati ustrezno usposobljeni, ki imajo ustrezne ateste in so pri montaži sposobni delati na višini.

Vsi jekleni elementi konstrukcije morajo biti pred antikorozijsko zaščito in transportom na gradbišče predhodno poizkusno sestavljeni z upoštevanjem nadvišanj in dodatnih deformacij pri montaži. Pri transportu in montaži je potrebno paziti, da se konstrukcija ne deformira in da se ne poškoduje antikorozijska zaščita. Sidrne plošče in profili morajo nalegati na beton. Na mestih naleganja je potrebno preveriti kvaliteto naležne površine, slabe dele naležne površine je potrebo odstraniti in o tem obvestiti pooblaščenega inženirja s področja gradbeništva. Vbetonirane sidrne plošče je potrebno postaviti na mesta, ki so določena v tehničnih prikazih pred pričetkom betoniranja. Sidrne plošče morajo biti pritrjene na armaturo oz. na kateri drug element tako, da med vgrajevanjem betona ne pride do premika sidrnih plošč. Vsa ključavničarska in varilska dela se morajo izdelati strokovno in solidno z delavci, ki so za ta dela usposobljeni, imajo ustrezne ateste in so pri montaži sposobni delati na višini. Upoštevati se morajo vsi veljavni standardi in predpisi za jeklo, zvare, vijake in jeklene konstrukcije.

2.3	TEHNIČNO POROČILO	6/6
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Kvaliteta vgrajenih materialov:

Ves vgrajeni material pri gradnji mora biti opremljen s potrdili o kvaliteti v skladu z zakonom o standardizaciji.

Predvideva se uporaba lesa trdnostnega razreda C24, dolžinsko spojenega lesa KVH trdnostnega razreda C 24 in lepljenega lameliranega lesa GL 24h ter GL 28h. Linijski jekleni elementi in ojačitve so zasnovani v jeklu kvalitete S235 JR.

Zahteve pri izvedbi in zaključek:

Celotna gradbena konstrukcija je preverjena na predvidljive vplive obtežb in drugih obremenilnih slučajev. Obtežbe in vplivi so določene po SIST EN 1991 in zahtev investitorja oz. naročnika.

Obremenitve v fazi gradnje niso posebej upoštevane, zato mora izvajalec pri gradnji skladiščenju in deponiranju materiala, postopkih vgradnje in montaže to upoštevati in paziti, da ne pride do preobremenitve in škodljivih vplivov na konstrukcije, ter s tem do poškodb (trenutnih, dolgoročnih, vidnih in nevidnih). Opozorilo velja tudi za fazo uporabe objekta saj, je potrebno paziti na prekomerne obtežbe in obremenitve ter način uporabe konstrukcije pri vseh vrstah obratovanja in vzdrževanja, tudi v fazi zmanjševanja teh obtežb.

Za montažne konstrukcije je potrebno pripraviti delavniške in montažne načrte, ki jih mora potrditi pooblaščen inženir s področja gradbeništva tega načrta.

Vse dela pri gradnji mora izvajati in nadzirati ustrezno strokovno usposobljeno podjetje ali oseba v skladu z Gradbenim zakonom (GZ-1)! Nivo nadzora je določi investitor po priporočilu SIST EN 1990.

Upravljalca objekta je dolžan poskrbeti za vizualne kontrolne preglede nosilne konstrukcije v časovnih razdobjih po pet let. Na konstrukciji je potrebno pregledati in evidentirati pojav eventualnih deformacij, razpok in sledi korozije armature. V primeru, da se pri vizualnem pregledu ugotovi večje poškodbe, je potrebno opraviti točnejše meritve in ugotovitve dejanske vrednosti poškodb ter označiti njihovo lokacijo. V kolikor dejanska vrednost poškodb, razpok, povesov in diferenčnih posredkov presega s predpisom določene vrednosti, je potrebno o tem obvestiti pooblaščenega inženirja s področja gradbeništva tega načrta, ki bo s pomočjo pooblaščenega institucije izvedel analizo vzrokov in vplivov poškodb na varnost in uporabnost objekta ter po potrebi izdelal predlog sanacijskih ukrepov.

Vzdrževanje objekta mora biti tako, da bo objekt varen in funkcionalen ter s tem odgovarjal namenu, kakor je predvideno v projektu.

V primeru novih posegov v nosilno konstrukcijo je upravljavec o tem dolžan obvestiti pooblaščenega inženirja s področja gradbeništva tega načrta.

Dolenja vas, maj 2026

Sestavil:
Miha Oražem; univ.dipl.inž.grad.



2.4	STATIČNI IZRAČUN – TEMELJNE KONSTRUKCIJE	
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Izračuni in dimenzioniranje je izvedeno s pomočjo računalniškega programa za statično in dinamično analizo prostorskih konstrukcij RFEM 5 in Timbertech.

Izračun je bil narejen za naslednja projektna stanja:

- mejno stanje nosilnosti MSN,
- mejno stanje uporabnosti MSU,
- potresno projektno stanje PPS.

Dolenja vas, maj 2026

Sestavil:
Miha Oražem; univ.dipl.inž.grad.



Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

MODEL - GENERAL DATA

	General	Model name	: TEMELJI_VRTEC PRIZIDEK
		Project name	: VRTEC MEZICA
		Type of model	: 3D
		Positive direction of global axis Z	: Downward
		Classification of load cases and combinations	: According to Standard: EN 1990 National Annex: SIST - Slovenia
		☒ Automatically create combinations	: ☒ Load Combinations
	Options	☐ RF-FORM-FINDING - Find initial equilibrium shapes of membrane and cable structures	
		☐ RF-CUTTING-PATTERN	
		☐ Piping analysis	
		☐ Use CQC Rule	
		☐ Enable CAD/BIM model	
		Standard Gravity g	: 10.00 m/s ²

FE MESH SETTINGS

	General	Target length of finite elements	l_{FE}	: 0.500 m
		Maximum distance between a node and a line to integrate it into the line	ε	: 0.001 m
		Maximum number of mesh nodes (in thousands)		: 500
	Members	Number of divisions of members with cable, elastic foundation, taper, or plastic characteristic		: 10
		☒ Activate member divisions for large deformation or post-critical analysis		
		☒ Use division for members with node lying on them		
	Surfaces	Maximum ratio of FE rectangle diagonals	Δ_D	: 1.800
		Maximum out-of-plane inclination of two finite elements	α	: 0.50 °
		Shape direction of finite elements		: Triangles and quadrangles ☒ Same squares where possible

1.3 MATERIALS

Matl. No.	Modulus E [kN/cm ²]	Modulus G [kN/cm ²]	Poisson's Ratio ν [-]	Spec. Weight γ [kN/m ³]	Coeff. of Th. Ex α [1/°C]	Partial Factor γ_M [-]	Material Model
1	Concrete C30/37 3300.00	EN 1992-1-1:2004/A1:2014 1375.00	0.200	25.00	1.00E-05	1.00	Isotropic Linear Elastic

1.9 SURFACE SUPPORTS



Found. No.	On Surfaces No.	Spring Constants RF-SOILIN	Translation Support or Spring [kN/m ³]			Shear Spring [kN/m]	
			u_x	u_y	u_z	v_{xz}	v_{yz}
1	1-6	-	11250.000	11250.000	15000.000	☐	☐

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 SIST Action Category	Self-Weight - Factor in Direction			
			Active	X	Y	Z
LC1	lastna in stalna	Permanent	☒	0.000	0.000	1.000
LC2	koristna C	Imposed - Category C: congregation areas	☐			
LC3	koristna A	Imposed - Category A: domestic, residential areas	☐			
LC4	sneg	Snow ($H \leq 1000$ m a.s.l.)	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters	
LC1	lastna in stalna	Method of analysis	: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations	: ☒ Newton-Raphson
		Activate stiffness factors of:	: ☒ Cross-sections (factor for J, I _y , I _z , A, A _y , A _z) : ☒ Members (factor for GJ, EI _y , EI _z , EA, GA _y , GA _z)
LC2	koristna C	Method of analysis	: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations	: ☒ Newton-Raphson
		Activate stiffness factors of:	: ☒ Cross-sections (factor for J, I _y , I _z , A, A _y , A _z) : ☒ Members (factor for GJ, EI _y , EI _z , EA, GA _y , GA _z)
LC3	koristna A	Method of analysis	: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations	: ☒ Newton-Raphson

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters
LC4	sneg	Activate stiffness factors of:
		: ☒ Cross-sections (factor for J, I _y , I _z , A, A _y , A _z)
		: ☒ Members (factor for GJ, EI _y , EI _z , EA, GA _y , GA _z)
		Method of analysis: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations: ☒ Newton-Raphson
		Activate stiffness factors of:
		: ☒ Cross-sections (factor for J, I _y , I _z , A, A _y , A _z)
		: ☒ Members (factor for GJ, EI _y , EI _z , EA, GA _y , GA _z)

2.7 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	CO1/p or to CO13
RC2	SLS - Quasi-permanent	CO14/p or to CO17

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC1: lastna in stalna

No.	On Nodes No.	Coordinate System	Force [kN]			Moment [kNm]		
			P _X / P _U	P _Y / P _V	P _Z / P _W	M _X / M _U	M _Y / M _V	M _Z / M _W
1	87	0 Global XYZ	0.000	0.000	27.000	0.000	0.000	0.000
2	88	0 Global XYZ	0.000	0.000	30.000	0.000	0.000	0.000
3	89	0 Global XYZ	0.000	0.000	24.000	0.000	0.000	0.000
4	90	0 Global XYZ	0.000	0.000	21.000	0.000	0.000	0.000
5	91	0 Global XYZ	0.000	0.000	27.000	0.000	0.000	0.000
6	92,95	0 Global XYZ	0.000	0.000	16.000	0.000	0.000	0.000
7	13,20	0 Global XYZ	0.000	0.000	23.000	0.000	0.000	0.000
8	94	0 Global XYZ	0.000	0.000	21.000	0.000	0.000	0.000
9	96,97	0 Global XYZ	0.000	0.000	27.000	0.000	0.000	0.000
10	99	0 Global XYZ	0.000	0.000	8.000	0.000	0.000	0.000
11	98	0 Global XYZ	0.000	0.000	8.000	0.000	0.000	0.000
12	100	0 Global XYZ	0.000	0.000	8.000	0.000	0.000	0.000
13	72	0 Global XYZ	0.000	0.000	38.000	0.000	0.000	0.000
14	101	0 Global XYZ	0.000	0.000	12.000	0.000	0.000	0.000
15	75	0 Global XYZ	0.000	0.000	12.000	0.000	0.000	0.000
16	93	0 Global XYZ	0.000	0.000	32.000	0.000	0.000	0.000
17	56,62	0 Global XYZ	0.000	0.000	17.000	0.000	0.000	0.000
18	102-104	0 Global XYZ	0.000	0.000	15.000	0.000	0.000	0.000
19	105	0 Global XYZ	0.000	0.000	11.000	0.000	0.000	0.000

3.3 LINE LOADS

LC1: lastna in stalna

No.	Reference to	On Lines No.	Load Type	Load Distribution	Load Direction	Load Parameters		
						Symbol	Value	Unit
1	Lines	12,19,94,96,103,105,117	Force	Uniform	ZL	p	35.000	kN/m
2	Lines	1,5,6,10,14,16,21,84,104	Force	Uniform	ZL	p	7.500	kN/m
3	Lines	83	Force	Uniform	ZL	p	13.000	kN/m
4	Lines	4,9,71	Force	Uniform	ZL	p	14.500	kN/m
5	Lines	11,111,113,115	Force	Uniform	ZL	p	17.000	kN/m
6	Lines	95,109	Force	Uniform	ZL	p	9.500	kN/m
7	Lines	2,3,7,8,13,17	Force	Uniform	ZL	p	5.000	kN/m

3.4 SURFACE LOADS

LC1: lastna in stalna

No.	On Surfaces No.	Load Type	Load Distribution	Load Direction	Load Parameters		
					Symbol	Value	Unit
1	1-6	Force	Uniform	ZL	p	2.20	kN/m ²

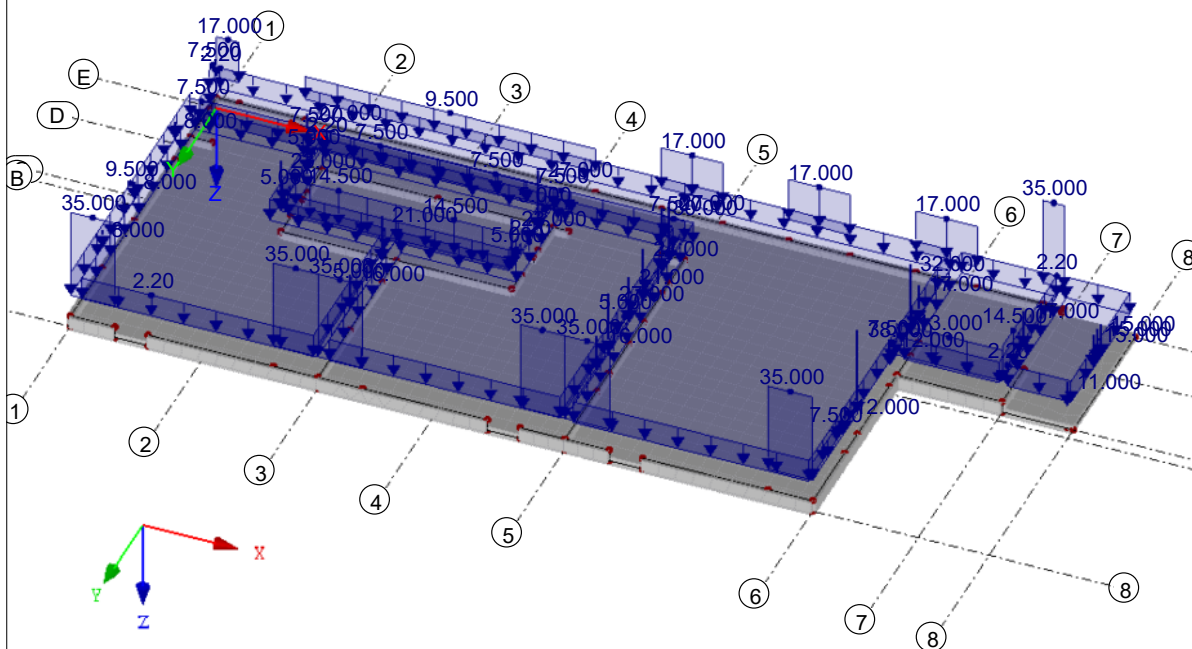
Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

LC1: LASTNA IN STALNA

LC1 : lastna in stalna
Loads [kN/m], [kN/m²], [kN]

Isometric



LC2
koristna C

3.4 SURFACE LOADS

LC2: koristna C

No.	On Surfaces No.	Load Type	Load Distribution	Load Direction	Load Parameters		
					Symbol	Value	Unit
1	1-3	Force	Uniform	ZL	p	3.00	kN/m ²
2	4-6	Force	Uniform	ZL	p	5.00	kN/m ²

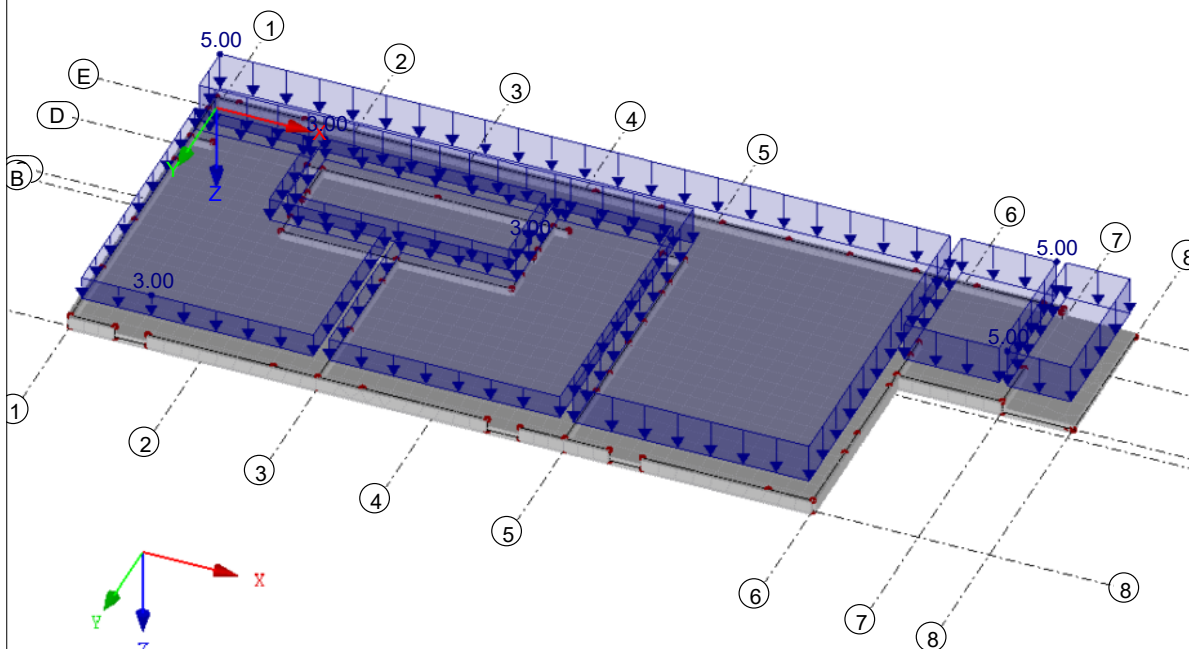
Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

LC2: KORISTNA C

LC2 : koristna C
Loads [kN/m²]

Isometric



3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC3: koristna A

No.	On Nodes No.	Coordinate System	Force [kN]			Moment [kNm]		
			P_x / P_U	P_y / P_V	P_z / P_W	M_x / M_U	M_y / M_V	M_z / M_W
1	88	0 Global XYZ	0.000	0.000	16.000	0.000	0.000	0.000
2	89	0 Global XYZ	0.000	0.000	14.000	0.000	0.000	0.000
3	91	0 Global XYZ	0.000	0.000	15.000	0.000	0.000	0.000
4	13,20	0 Global XYZ	0.000	0.000	13.000	0.000	0.000	0.000
5	72	0 Global XYZ	0.000	0.000	15.000	0.000	0.000	0.000
6	93	0 Global XYZ	0.000	0.000	17.000	0.000	0.000	0.000
7	102-104	0 Global XYZ	0.000	0.000	3.000	0.000	0.000	0.000

3.3 LINE LOADS

LC3: koristna A

No.	Reference to	On Lines No.	Load Type	Load Distribution	Load Direction	Load Parameters		
						Symbol	Value	Unit
1	Lines	4,9,12,19,94,96,103,105,117	Force	Uniform	ZL	p	7.000	kN/m
2	Lines	71,83	Force	Uniform	ZL	p	2.500	kN/m
3	Lines	11,111,113,115	Force	Uniform	ZL	p	4.000	kN/m
4	Lines	1,5,6,10,14,16,95,109	Force	Uniform	ZL	p	2.000	kN/m

LC3
koristna A

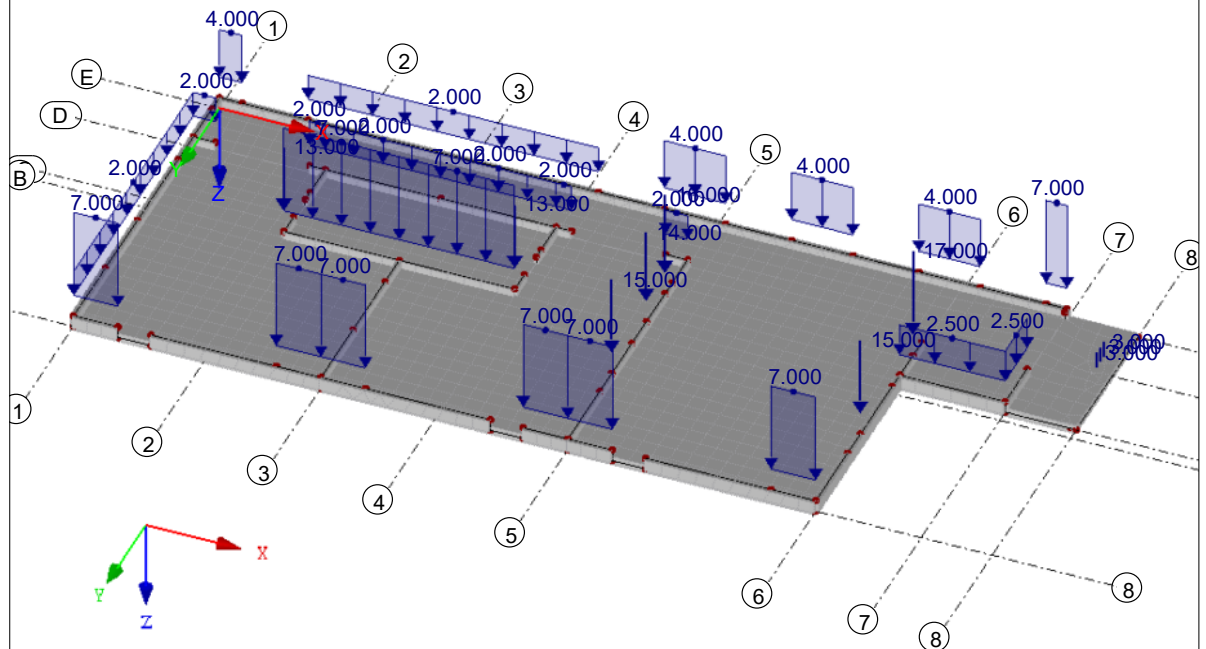
Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

LC3: KORISTNA A

LC3 : koristna A
Loads [kN/m], [kN]

Isometric



3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC4: sneg

No.	On Nodes No.	Coordinate System	Force [kN]			Moment [kNm]		
			P_X / P_U	P_Y / P_V	P_Z / P_W	M_X / M_U	M_Y / M_V	M_Z / M_W
1	87	0 Global XYZ	0.000	0.000	60.000	0.000	0.000	0.000
2	90	0 Global XYZ	0.000	0.000	8.000	0.000	0.000	0.000
3	92,95	0 Global XYZ	0.000	0.000	44.000	0.000	0.000	0.000
4	94	0 Global XYZ	0.000	0.000	40.000	0.000	0.000	0.000
5	96,97	0 Global XYZ	0.000	0.000	57.000	0.000	0.000	0.000
6	99	0 Global XYZ	0.000	0.000	16.000	0.000	0.000	0.000
7	98	0 Global XYZ	0.000	0.000	16.000	0.000	0.000	0.000
8	100	0 Global XYZ	0.000	0.000	16.000	0.000	0.000	0.000
9	72	0 Global XYZ	0.000	0.000	9.000	0.000	0.000	0.000
10	101	0 Global XYZ	0.000	0.000	28.000	0.000	0.000	0.000
11	75	0 Global XYZ	0.000	0.000	13.000	0.000	0.000	0.000
12	62	0 Global XYZ	0.000	0.000	36.000	0.000	0.000	0.000
13	56	0 Global XYZ	0.000	0.000	25.000	0.000	0.000	0.000
14	102-104	0 Global XYZ	0.000	0.000	6.000	0.000	0.000	0.000
15	105	0 Global XYZ	0.000	0.000	6.000	0.000	0.000	0.000

3.3 LINE LOADS

LC4: sneg

No.	Reference to	On Lines No.	Load Type	Load Distribution	Load Direction	Load Parameters		
						Symbol	Value	Unit
1	Lines	12,19,94,96,103,105,117	Force	Uniform	ZL	p	17.000	kN/m
2	Lines	71,83	Force	Uniform	ZL	p	2.500	kN/m
3	Lines	11,111,113,115	Force	Uniform	ZL	p	8.500	kN/m
4	Lines	109	Force	Uniform	ZL	p	3.500	kN/m

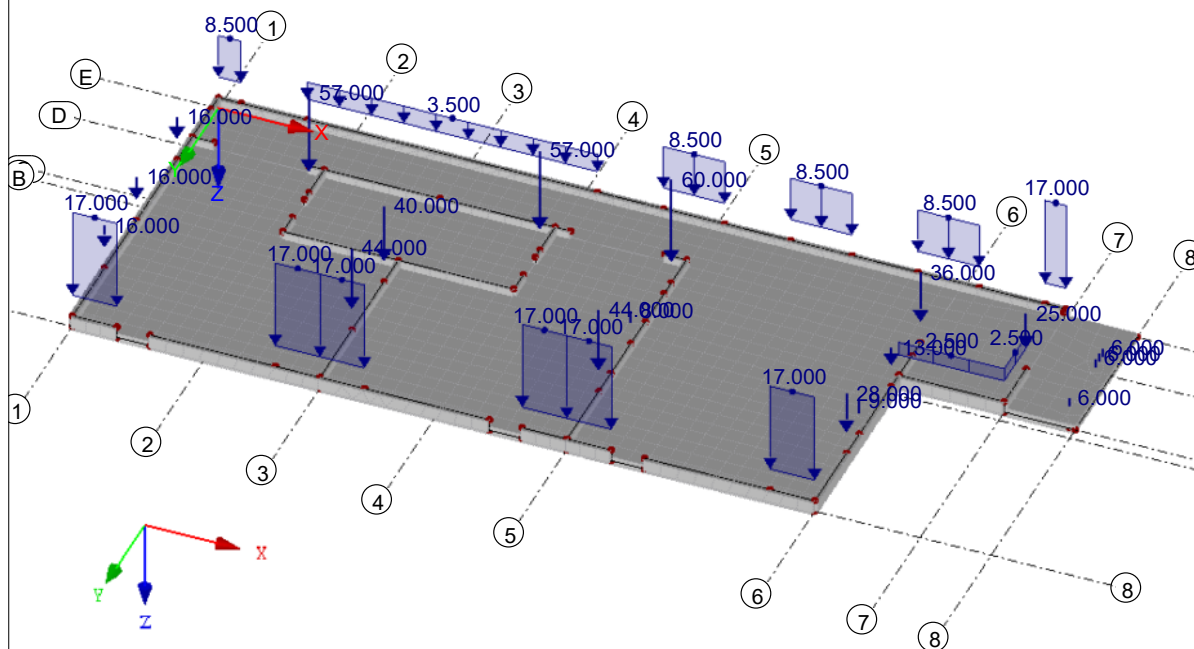
Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

LC4: SNEG

LC4 : sneg
Loads [kN/m], [kN]

Isometric



Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

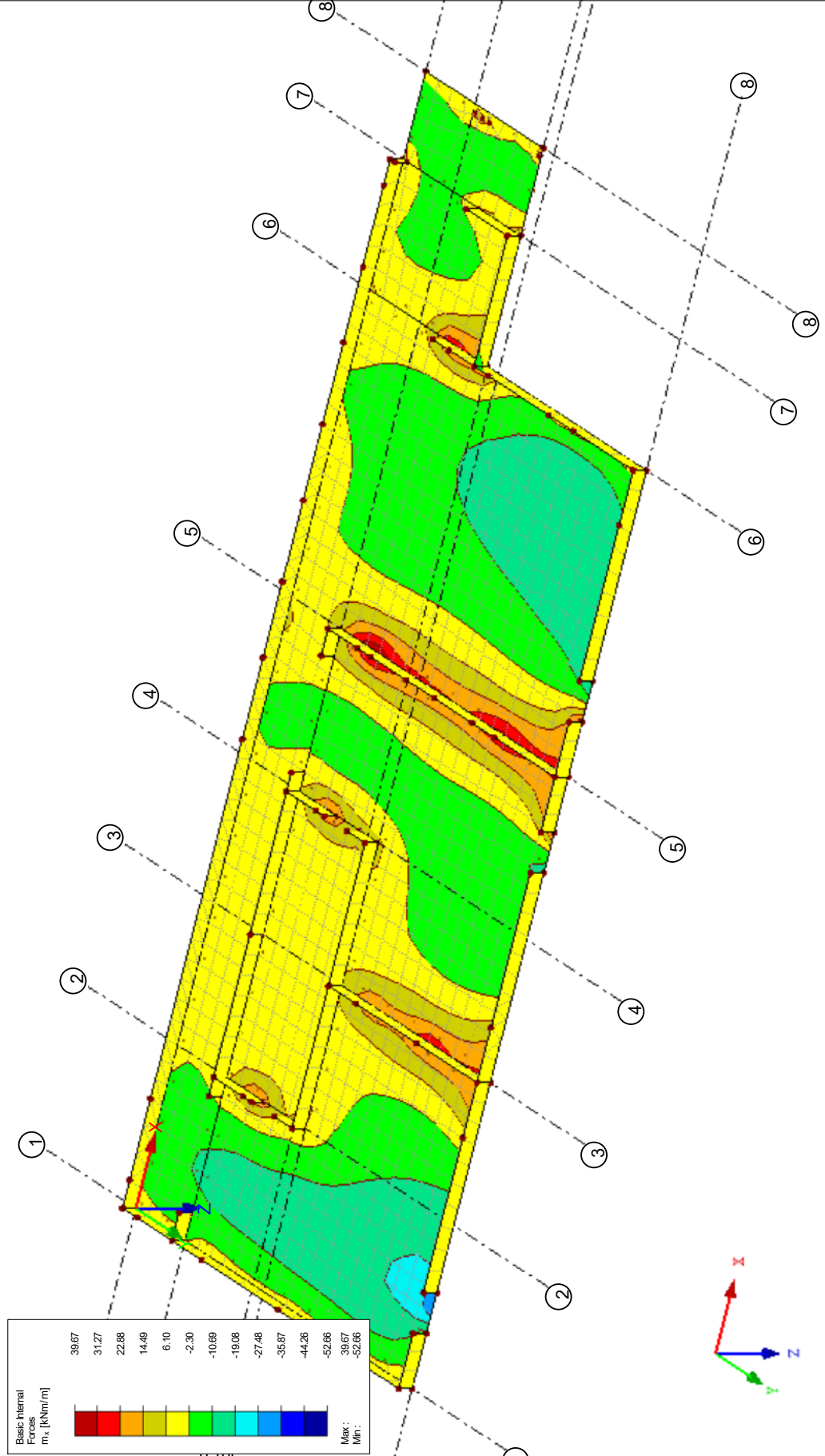
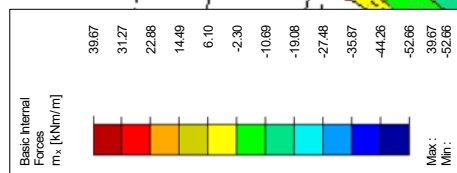
■ BASIC INTERNAL FORCES m_x

Isometric

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10

Basic Internal Forces m_x [kNm/m]

Result Combinations: Max and Min Values

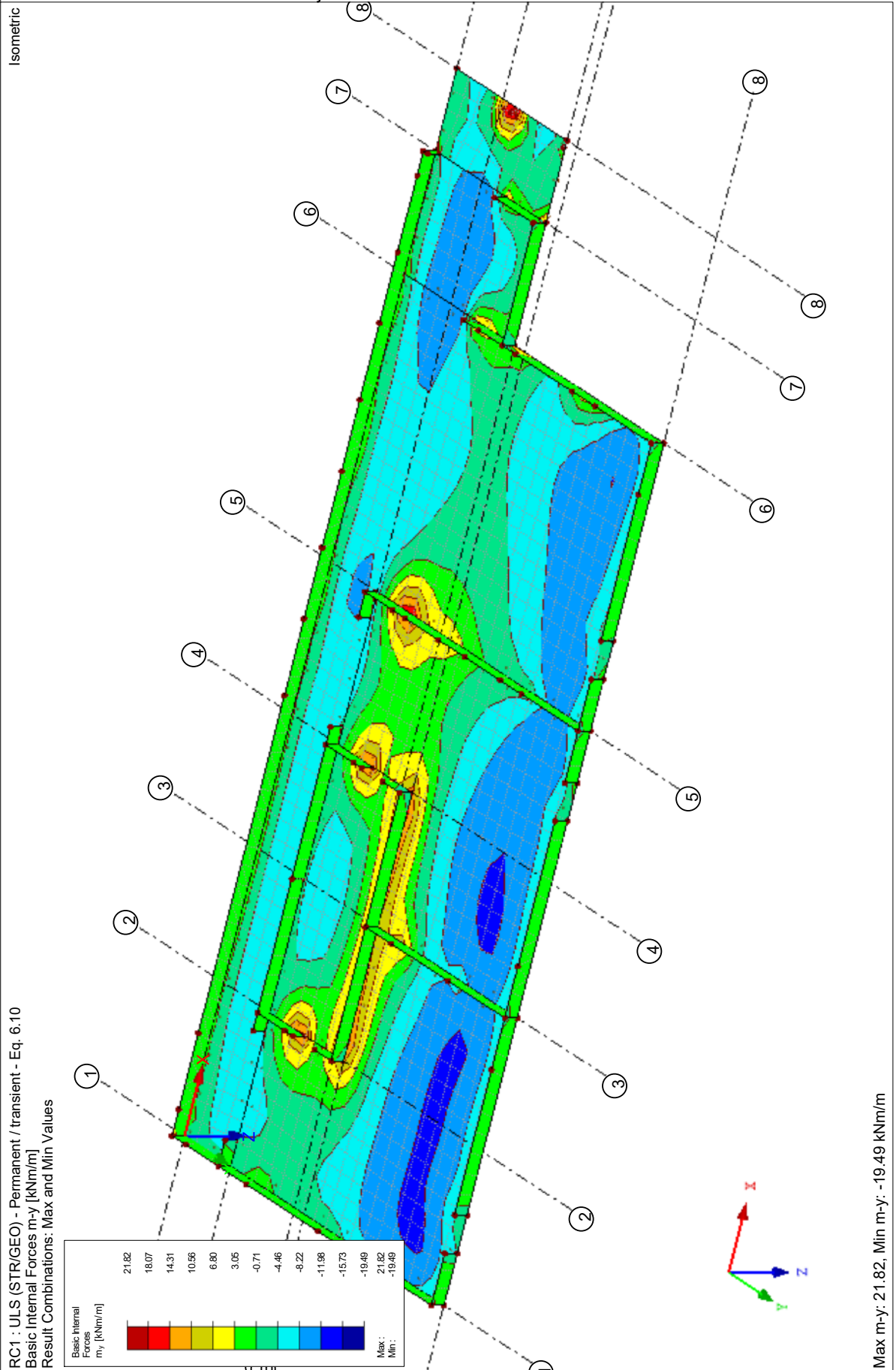


Max m_x : 39.67, Min m_x : -52.66 kNm/m

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

■ BASIC INTERNAL FORCES m_y



Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

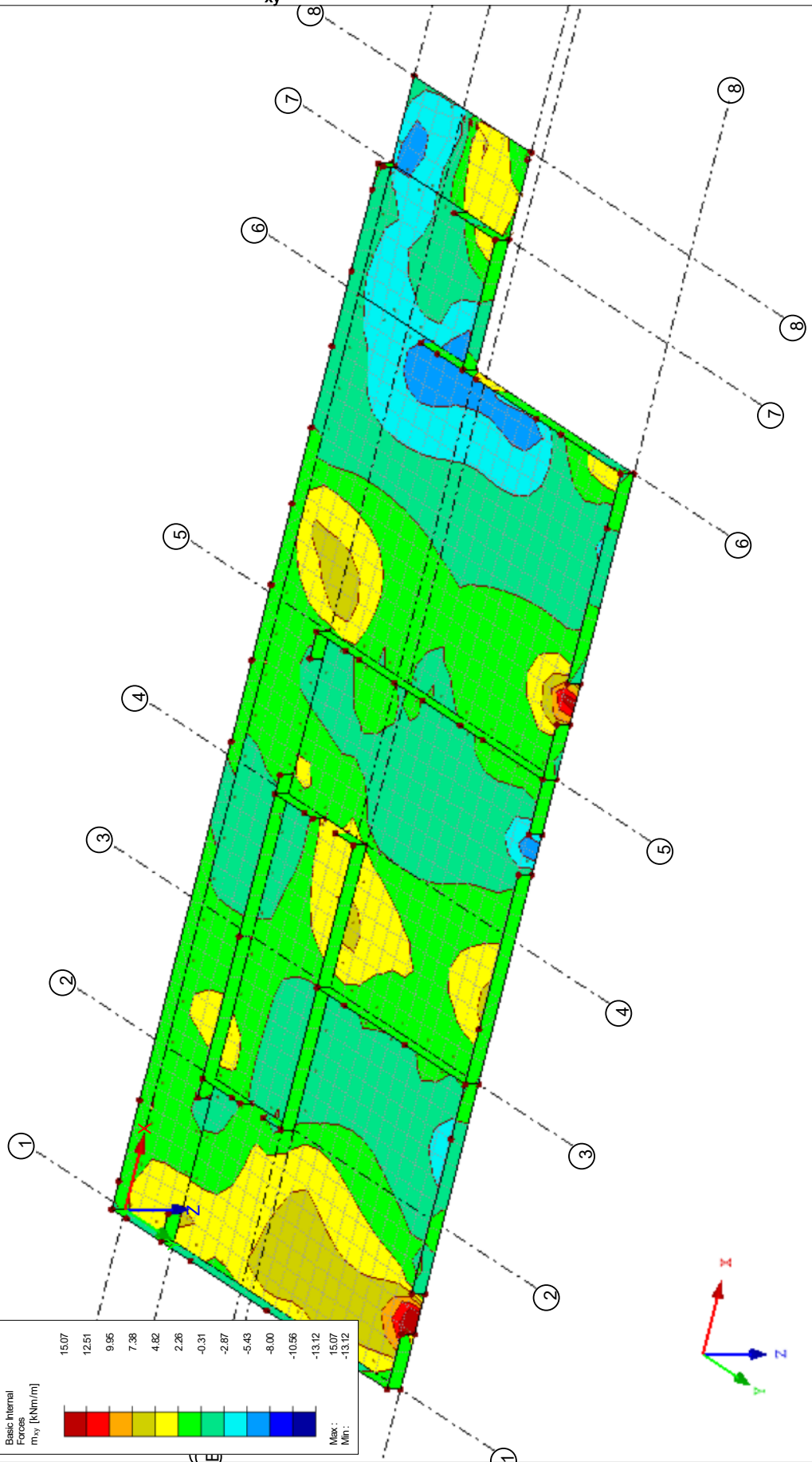
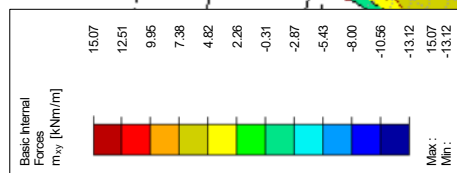
■ BASIC INTERNAL FORCES m_{xy}

Isometric

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10

Basic Internal Forces m_{xy} [kNm/m]

Result Combinations: Max and Min Values

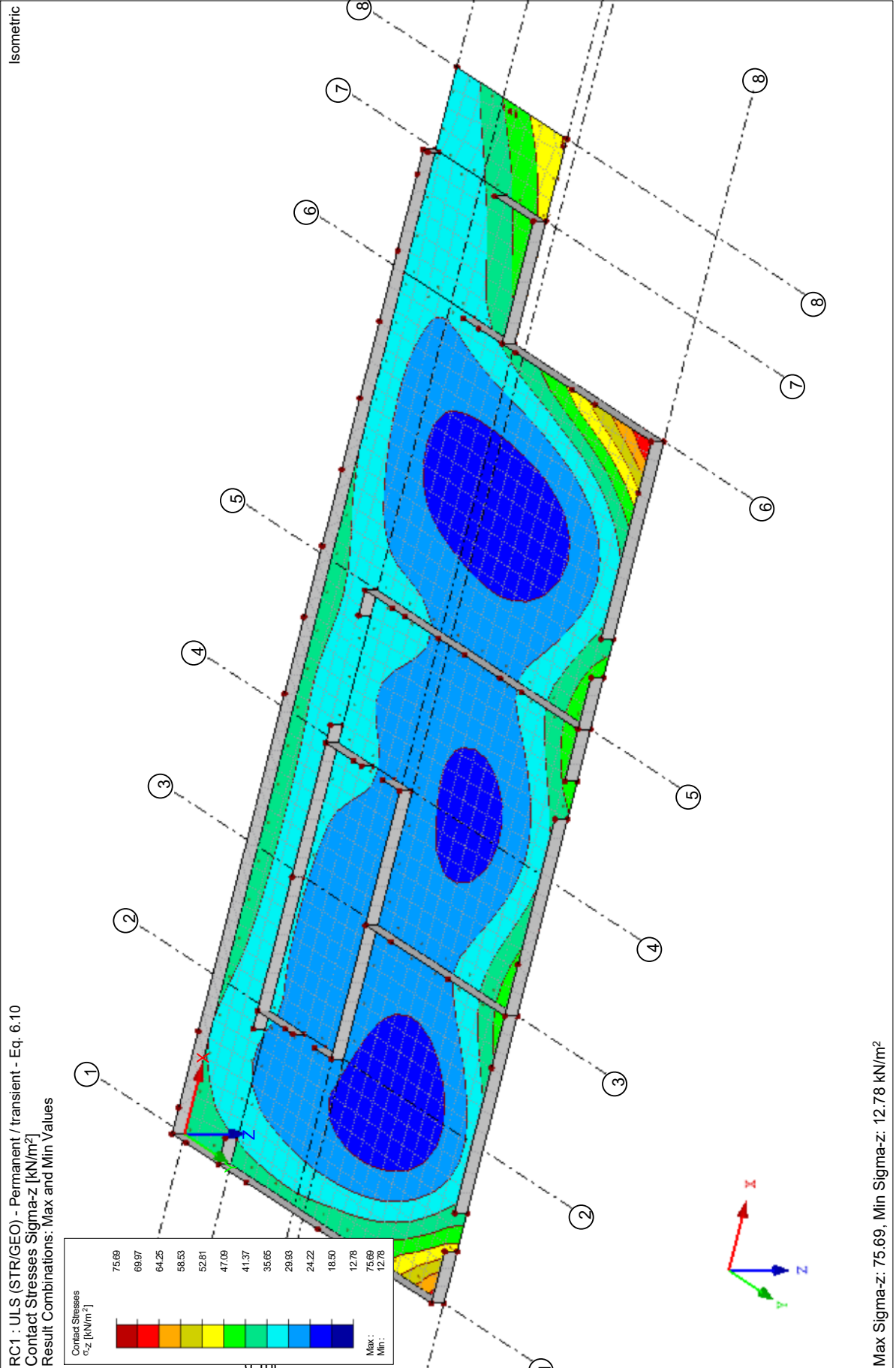


Max m_{xy} : 15.07, Min m_{xy} : -13.12 kNm/m

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

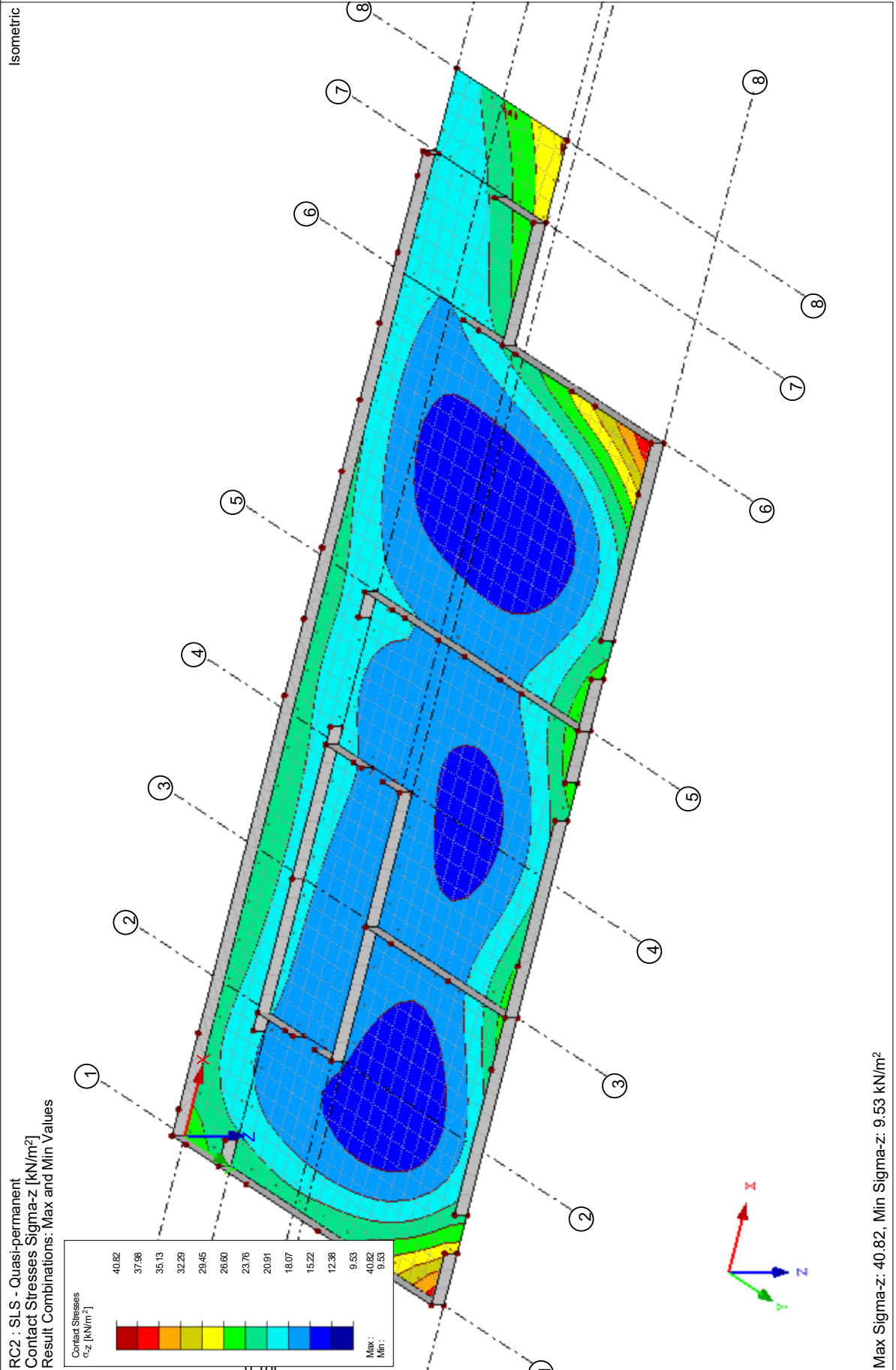
■ **CONTACT STRESSES σ_z**



Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

■ **CONTACT STRESSES σ_z**



RF-CONCRETE Surfaces

CA1

Reinforced concrete design

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

1.1 GENERAL DATA

Design according to Standard:		SIST EN 1992-1-1:2005/A101:2006	
ULTIMATE LIMIT STATE			
Result combination for design:	RC1	ULS (STR/GEO) - Permanent / transient - Eq. 6.10 Persistent and Transient	
SERVICEABILITY LIMIT STATE			
Load combinations for design:	CO16	G + 0.6Q _{iC} + 0.3Q _{iA} Quasi-permanent, k _i 0.600	
Definition of Provided Additional Reinforcement	Automatic arrangement according to the specifications in Table 1.4		
Type of SLS method:	Nonlinear Method According to EN 1992-1-1, 5.7 (4): 'Nonlinear Analysis'		
Consider creeping	☒		
Consider shrinkage	☐		
Design of			
Deformation Analysis	☒		
Crack width control	☒		
Concrete Stress Analysis	☐		
Steel Stress Analysis	☒		
Stress-strain diagram for concrete in compression:	Parabolic		
Stress-strain diagram for concrete in tension:	Tension stiffening with concrete tensile strength (Quast method)		
Adjustment factor of tensile strength f _{ct,R} :	0.60		
Material Concrete - Calculation Parameters:			
Concrete C30/37	Factor	21.84	
	v = f _c / f _{ct} , R:		
	Exponent	2.01	
	n-PR:		
	Exponent	1.00	
	n-VMB:		
Use strength of steel up to ultimate tensile strength	☒		
Settings for the iteration process:			
Maximum number of iterations:	200		
Number of load increments:	1		
Number of layers in net element:	10		
DETAILS			
Analysis Method for Reinforcement Envelope	Mixed		
Apply the internal forces without the rib components	☐		
Design Situation Settings for Serviceability Limit State Checks			
Load combination:			
Characteristic with direct load	Checks: k ₁ *f _{ck} , k ₃ *f _{yk}		
Characteristic with imposed deformation	Checks: k ₁ *f _{ck} , k ₄ *f _{yk}		
Frequent	Checks: w _k		
Quasi-permanent	Checks: k ₂ *f _{ck} , w _k , u _i		

1.2 MATERIALS

Material No.	Material Description		Comment
	Concrete Strength Class	Steel Description	
1	Concrete C30/37	B 500 S (A)	

1.3 SURFACES

Surface No.	Matl. No.	Creep coeff. φ [-]	U _{z,max} [mm]	W _{k,+z} (top) [mm]	W _{k,-z} (bottom) [mm]	Notes
1	Thickness Type: Constant, Thickness: 300.00 mm 1 2.21496		1.880	0.300	0.300	Deformation referred to undeformed system
2	Thickness Type: Constant, Thickness: 300.00 mm 1 2.21496		2.780	0.300	0.300	Deformation referred to undeformed system
3	Thickness Type: Constant, Thickness: 300.00 mm 1 2.21496		1.880	0.300	0.300	Deformation referred to undeformed system
4	Thickness Type: Constant, Thickness: 300.00 mm 1 2.21496		1.880	0.300	0.300	Deformation referred to undeformed system
5	Thickness Type: Constant, Thickness: 300.00 mm 1 2.21496		0.720	0.300	0.300	Deformation referred to undeformed system
6	Thickness Type: Constant, Thickness: 300.00 mm 1 2.21496		0.720	0.300	0.300	Deformation referred to undeformed system
7	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	Deformation referred to undeformed system
8	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	Deformation referred to undeformed system
9	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	Deformation referred to undeformed system
10	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	Deformation referred to undeformed system

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

■ 1.3 SURFACES

Surface No.	Matl. No.	Creep coeff. φ [-]	$u_{z,max}$ [mm]	$W_{k,+z}$ (top) [mm]	$W_{k,-z}$ (bottom) [mm]	Notes
Deformation referred to undeformed system						
11	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	
Deformation referred to undeformed system						
12	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	
Deformation referred to undeformed system						
13	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	
Deformation referred to undeformed system						
14	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	
Deformation referred to undeformed system						
15	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	
Deformation referred to undeformed system						
16	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		0.720	0.300	0.300	
Deformation referred to undeformed system						
17	Thickness Type: Constant, Thickness: 215.00 mm 1 2.32595		1.520	0.300	0.300	
Deformation referred to undeformed system						
18	Thickness Type: Constant, Thickness: 130.00 mm 1 2.51681		1.520	0.300	0.300	
Deformation referred to undeformed system						
19	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
20	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
21	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
22	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
23	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
24	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
25	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
26	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
27	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						
28	Thickness Type: Constant, Thickness: 170.00 mm 1 2.41129		1.520	0.300	0.300	
Deformation referred to undeformed system						

■ 1.4 REINFORCEMENT GROUP NO. 1 - AB TEMELJNA PLOSCA IN PARAPETI

Applied to surfaces:	All
REINFORCEMENT RATIO	
Minimum secondary reinforcement	20.0 %
Basic minimum reinforcement	0.0 %
Minimum compression reinforcement	0.0 %
Minimum tension reinforcement	0.0 %
Maximum reinforcement percentage	4.0 %
Minimum shear reinforcement percentage	0.0 %
REINFORCEMENT AREA FOR DESIGN OF SLS	
Use provided basic reinforcement and required additional reinforcement acc. to Tables 2.1, 2.2, 2.3	
Concrete cover acc. to Standard	☐
BASIC REINFORCEMENT LAYOUT - TOP (-z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area	As-1,-z (top): 3.85, As-2,-z (top): 3.85 cm²/m
BASIC REINFORCEMENT LAYOUT - BOTTOM (+z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area	As-1,+z (bottom): 5.03, As-2,+z (bottom): 5.03 cm²/m
ADDITIONAL REINFORCEMENT LAYOUT - TOP (-z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

1.4 REINFORCEMENT GROUP NO. 1 - AB TEMELJNA PLOSCA IN PARAPETI

Reinforcement area Use required additional reinforcement acc. to Tables 2.1, 2.2, 2.3

ADDITIONAL REINFORCEMENT LAYOUT - BOTTOM (+z)

Number of directions 2
Cover to rebar centroid d-1: 30.00, d-2: 40.00 mm
Bar diameter ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area Use required additional reinforcement acc. to Tables 2.1, 2.2, 2.3

LONGITUDINAL REINFORCEMENT FOR SHEAR FORCE DESIGN

Apply the greater value resulting from either the required or provided reinforcement (basic and add. reinforcement) per reinforcement direction

OPTIONS FOR SIST EN 1992-1-1:2005/A101:2006

Minimum longitudinal reinforcement for plates acc. to 9.3.1 ☐
Minimum longitudinal reinforcement for walls acc. to 9.6 ☒
Minimum shear reinforcement ☒
Neutral axis depth limitation ☒
Variable strut inclination - min 21.801°
Variable concrete strut inclination - max 45.000°
Partial safety factor γ_s PT 1.15, AC 1.00, SLS 1.00
Partial safety factor γ_c PT 1.50, AC 1.20, SLS 1.00
Consideration of long-term effects Alpha-cc PT 1.00, AC 1.00, SLS 1.00
Consideration of long-term effects Alpha-ct SLS 1.00

2.1 REQUIRED REINFORCEMENT TOTAL

Surface No.	Point No.	Point Coordinates [m]			Symbol	Required Reinf. ULS	Basic Reinf.	Additional Reinforcement		Unit	Notes
		X	Y	Z				Required	Provided		
12	M1221	19.515	10.833	-0.380	$a_{s,1,-z}$ (top)	10.71	3.85	6.86	6.86	cm ² /m	
28	M93 - E1332	22.500	3.617	-0.380	$a_{s,2,-z}$ (top)	6.76	3.85	2.91	2.91	cm ² /m	
12	M1221	19.515	10.833	-0.380	$a_{s,1,+z}$ (bottom)	10.43	5.03	5.40	5.40	cm ² /m	
28	M93 - E1332	22.500	3.617	-0.380	$a_{s,2,+z}$ (bottom)	7.61	5.03	2.58	2.58	cm ² /m	
13	M60	22.500	4.600	-0.380	a_{sw}	11.10	-	-	-	cm ² /m ²	

3.1 NONLINEAR CALCULATION TOTAL

Surface No.	Point No.	Point Coordinates [m]			Load Case	Type	Exist. Value	Design Limit Value	Unit	Ratio	Notes
		X	Y	Z							
6	M70	27.850	4.600	0.000	CO16	$U_{z,local}$	2.090	0.720	mm	3.0	100)
1	M1	0.000	9.844	0.000	CO16	w_k	0.000	0.300	mm	0.0	
1	M1	0.000	9.844	0.000		σ_s	undefined	400.00	N/mm ²		108)

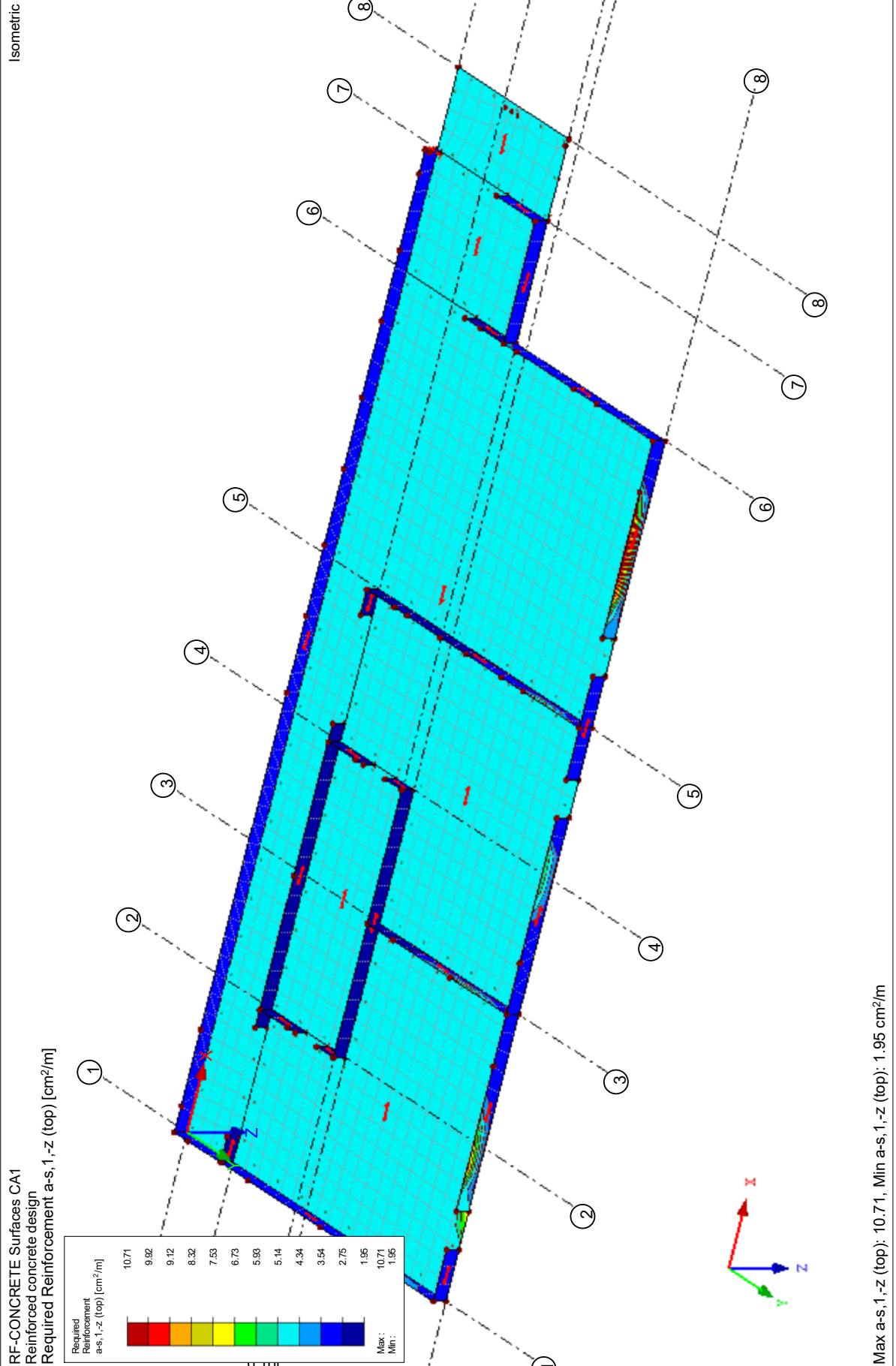
NONLINEAR - NOTES

No.	Description
100)	The border limit of deformation was exceeded.
108)	The design is not possible! The specified loads (LC/CO/RC) with the selected types of load combinations (characteristic/frequent/quasi-permanent) do not contain the required checks (stress/crack width/deflection).

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

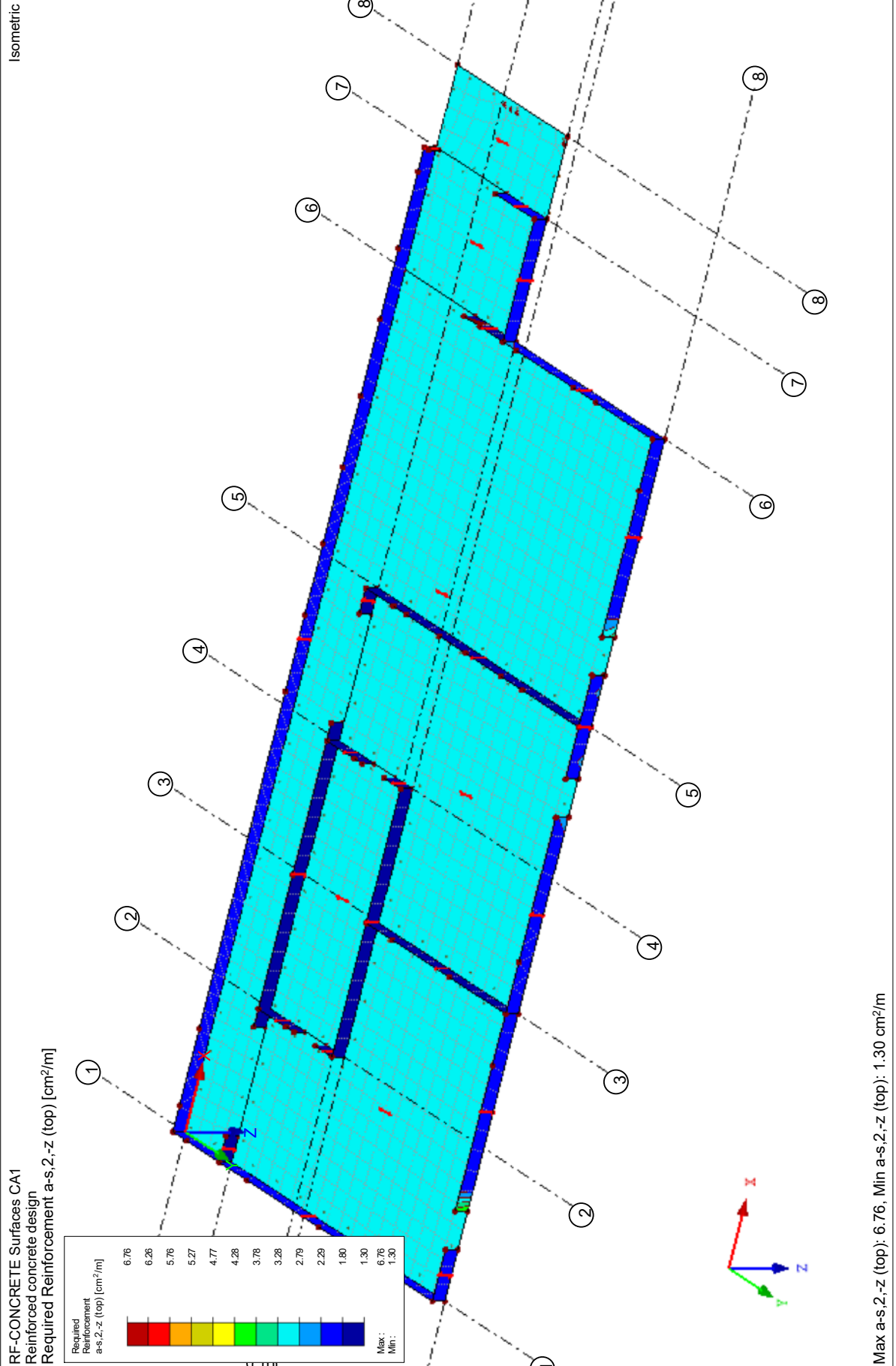
■ REQUIRED REINFORCEMENT $a_{s,1,-z} \text{ (top)}$



Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

■ REQUIRED REINFORCEMENT $a_{s,2-z}$ (top)



Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

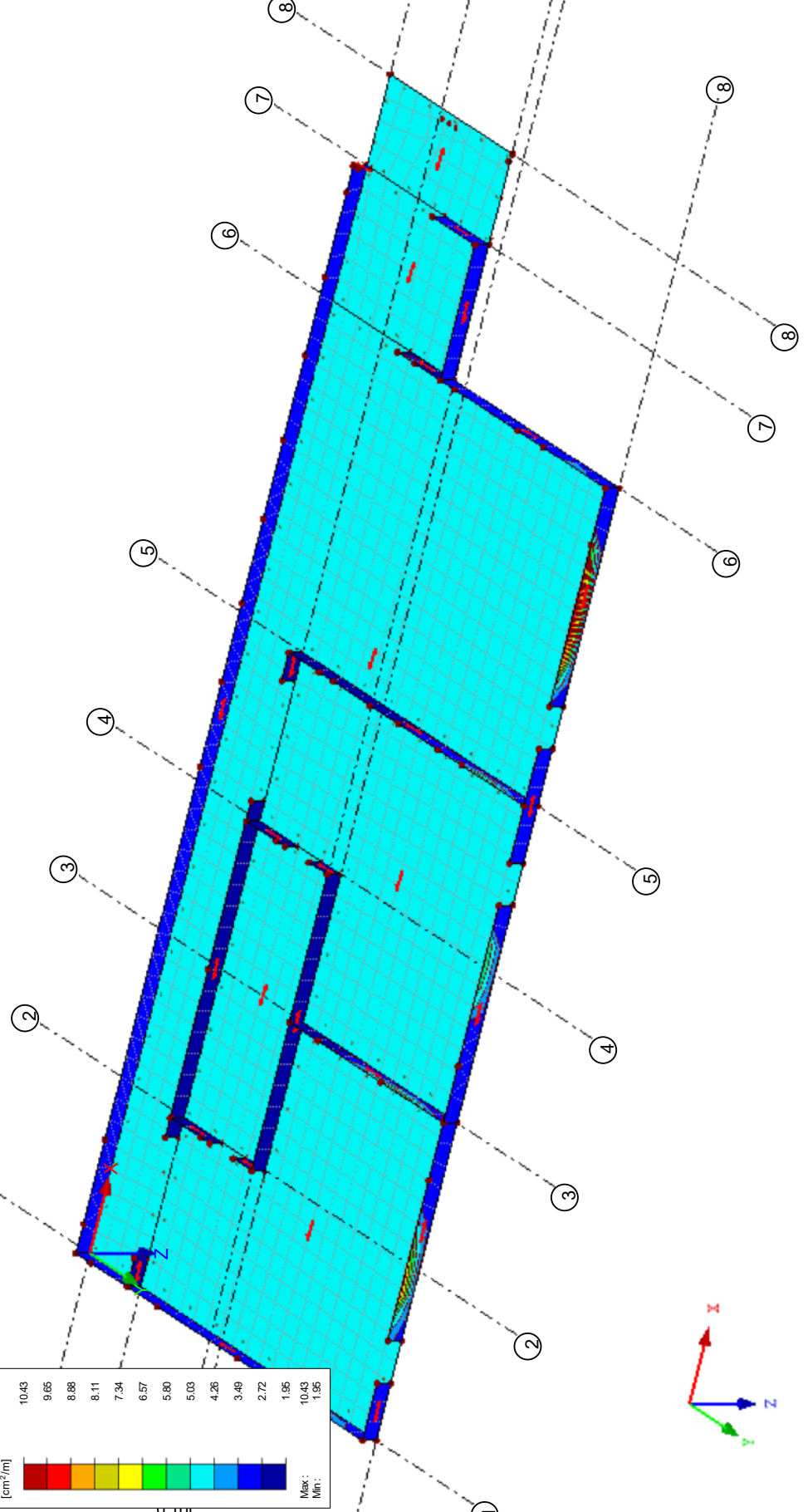
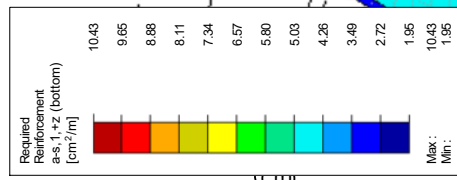
■ REQUIRED REINFORCEMENT $a_{s,1,+z}$ (bottom)

Isometric

RF-CONCRETE Surfaces CA1

Reinforced concrete design

Required Reinforcement $a_{s,1,+z}$ (bottom) [cm²/m]

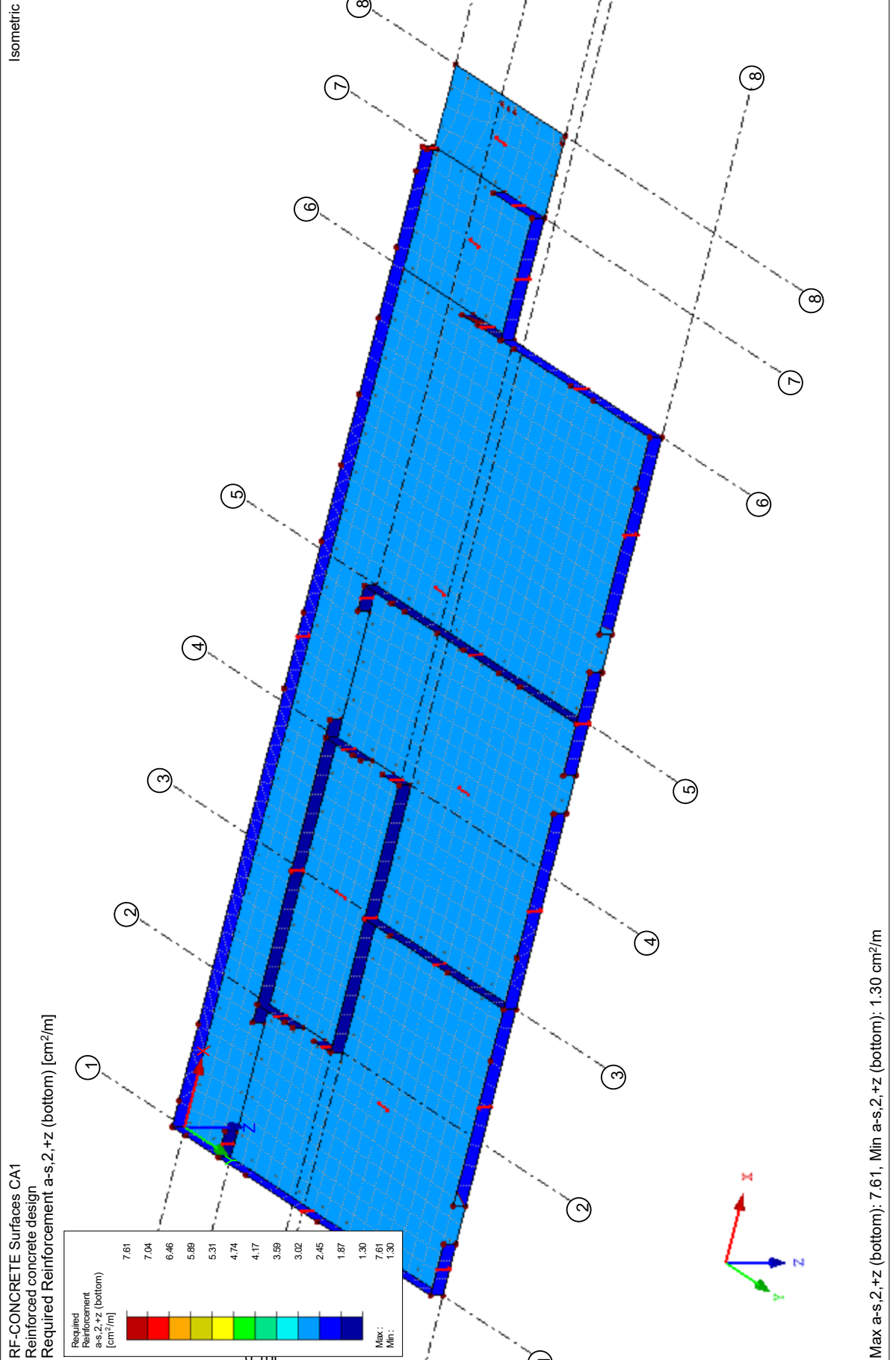


Max $a_{s,1,+z}$ (bottom): 10.43, Min $a_{s,1,+z}$ (bottom): 1.95 cm²/m

Project: VRTEC MEZICA

Model: TEMELJI_VRTEC PRIZIDEK

■ REQUIRED REINFORCEMENT $a_{s,2,+z}$ (bottom)



Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA PRIZIDEK

MODEL - GENERAL DATA

	General	Model name	: TEMELJI_KUHINJA PRIZIDEK
		Project name	: VRTEC MEZICA
		Type of model	: 3D
		Positive direction of global axis Z	: Downward
		Classification of load cases and combinations	: According to Standard: EN 1990 National Annex: SIST - Slovenia
		☒ Automatically create combinations	: ☒ Load Combinations
	Options	☐ RF-FORM-FINDING - Find initial equilibrium shapes of membrane and cable structures	
		☐ RF-CUTTING-PATTERN	
		☐ Piping analysis	
		☐ Use CQC Rule	
		☐ Enable CAD/BIM model	
		Standard Gravity g	: 10.00 m/s ²

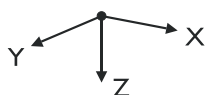
FE MESH SETTINGS

	General	Target length of finite elements	l_{FE}	: 0.500 m
		Maximum distance between a node and a line to integrate it into the line	ϵ	: 0.001 m
		Maximum number of mesh nodes (in thousands)		: 500
	Members	Number of divisions of members with cable, elastic foundation, taper, or plastic characteristic		: 10
		☒ Activate member divisions for large deformation or post-critical analysis		
		☒ Use division for members with node lying on them		
	Surfaces	Maximum ratio of FE rectangle diagonals	Δ_D	: 1.800
		Maximum out-of-plane inclination of two finite elements	α	: 0.50 °
		Shape direction of finite elements		: Triangles and quadrangles ☒ Same squares where possible

1.3 MATERIALS

Matl. No.	Modulus E [kN/cm ²]	Modulus G [kN/cm ²]	Poisson's Ratio ν [-]	Spec. Weight γ [kN/m ³]	Coeff. of Th. Ex α [1/°C]	Partial Factor γ_M [-]	Material Model
1	Concrete C30/37 3300.00	EN 1992-1-1:2004/A1:2014 1375.00	0.200	25.00	1.00E-05	1.00	Isotropic Linear Elastic

1.9 SURFACE SUPPORTS



Found. No.	On Surfaces No.	Spring Constants RF-SOILIN	Translation Support or Spring [kN/m ³]			Shear Spring [kN/m]	
			u_x	u_y	u_z	v_{xz}	v_{yz}
1	29	-	11250.000	11250.000	15000.000	☐	☐

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 SIST Action Category	Self-Weight - Factor in Direction			
			Active	X	Y	Z
LC1	lastna in stalna	Permanent	☒	0.000	0.000	1.000
LC2	koristna C	Imposed - Category C: congregation areas	☐			
LC3	koristna A	Imposed - Category A: domestic, residential areas	☐			
LC4	sneg	Snow ($H \leq 1000$ m a.s.l.)	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters	
LC1	lastna in stalna	Method of analysis	: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations	: ☒ Newton-Raphson
		Activate stiffness factors of:	: ☒ Cross-sections (factor for J, I _y , I _z , A, A _y , A _z) : ☒ Members (factor for GJ, EI _y , EI _z , EA, GA _y , GA _z)
LC2	koristna C	Method of analysis	: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations	: ☒ Newton-Raphson
		Activate stiffness factors of:	: ☒ Cross-sections (factor for J, I _y , I _z , A, A _y , A _z) : ☒ Members (factor for GJ, EI _y , EI _z , EA, GA _y , GA _z)
LC3	koristna A	Method of analysis	: ☒ Geometrically linear analysis
		Method for solving system of nonlinear algebraic equations	: ☒ Newton-Raphson

Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA PRIZIDEK

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters
		Activate stiffness factors of: ☒ Cross-sections (factor for J, I_y, I_z, A, A_y, A_z) ☒ Members (factor for $GJ, EI_y, EI_z, EA, GA_y, GA_z$)
LC4	sneg	Method of analysis: ☒ Geometrically linear analysis Method for solving system of nonlinear algebraic equations: ☒ Newton-Raphson Activate stiffness factors of: ☒ Cross-sections (factor for J, I_y, I_z, A, A_y, A_z) ☒ Members (factor for $GJ, EI_y, EI_z, EA, GA_y, GA_z$)

2.7 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	CO1/p or to CO13
RC2	SLS - Quasi-permanent	CO14/p or to CO17

LC1
lastna in stalna

3.3 LINE LOADS

LC1: lastna in stalna

No.	Reference to	On Lines No.	Load Type	Load Distribution	Load Direction	Symbol	Value	Unit
2	Lines	139,154	Force	Uniform	ZL	p	7.500	kN/m
8	Lines	138	Force	Uniform	ZL	p	57.000	kN/m
9	Lines	140-143,156,158,160,162	Force	Uniform	ZL	p	28.000	kN/m

3.4 SURFACE LOADS

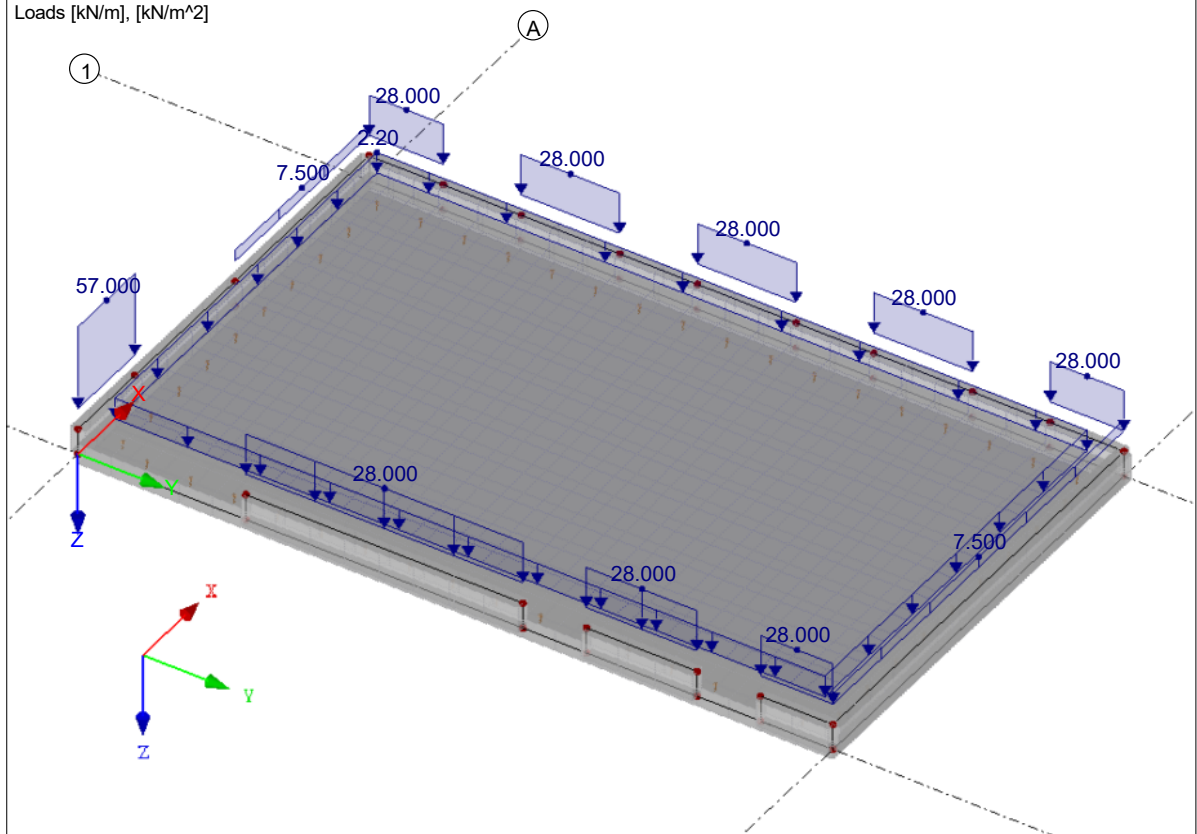
LC1: lastna in stalna

No.	On Surfaces No.	Load Type	Load Distribution	Load Direction	Symbol	Value	Unit
1	29	Force	Uniform	ZL	p	2.20	kN/m ²

LC1: LASTNA IN STALNA

LC1 : lastna in stalna
Loads [kN/m], [kN/m²]

Isometric



Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA PRIZIDEK

LC2
koristna C

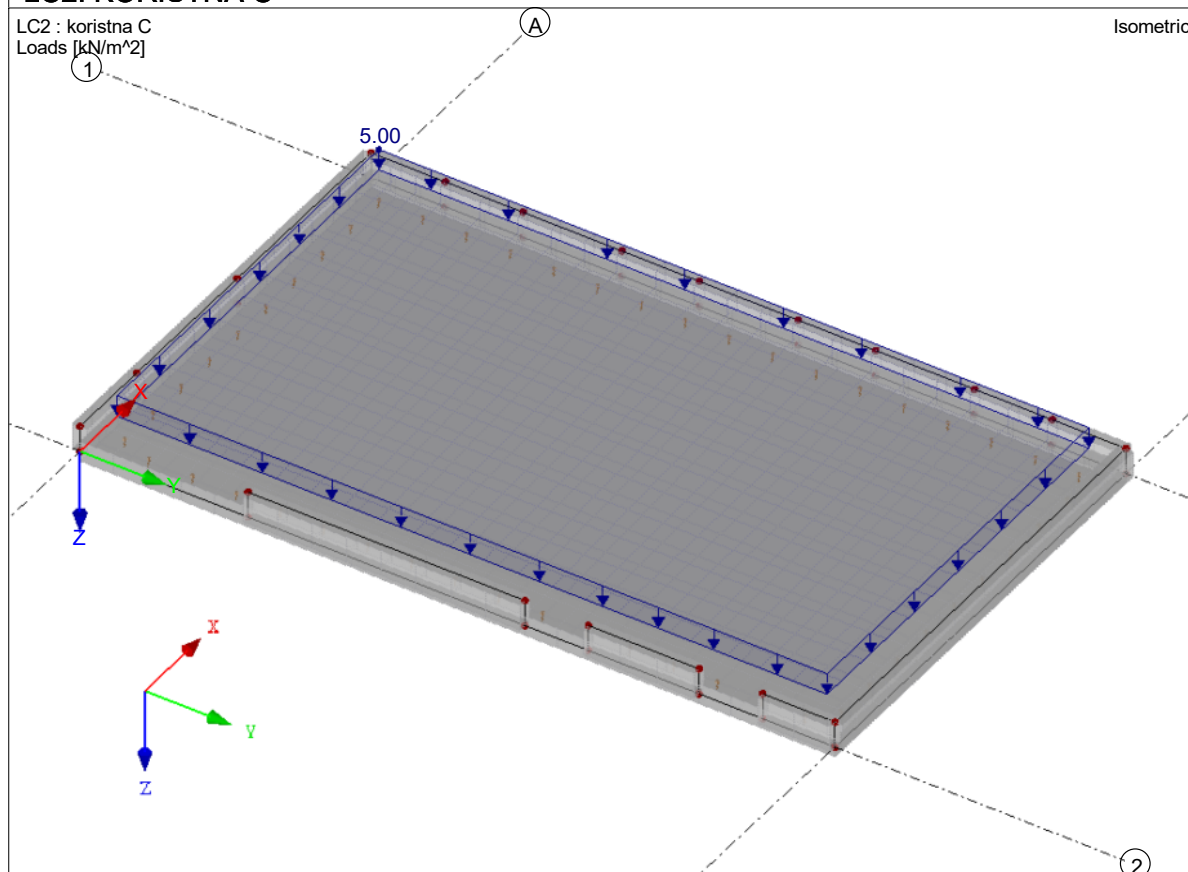
3.4 SURFACE LOADS

LC2: koristna C

No.	On Surfaces No.	Load Type	Load Distribution	Load Direction	Symbol	Value	Unit
2	29	Force	Uniform	ZL	p	5.00	kN/m ²

LC2: KORISTNA C

LC2 : koristna C
Loads [kN/m²]



LC4
sneg

3.3 LINE LOADS

LC4: sneg

No.	Reference to	On Lines No.	Load Type	Load Distribution	Load Direction	Symbol	Value	Unit
5	Lines	138	Force	Uniform	ZL	p	35.000	kN/m
6	Lines	141-143	Force	Uniform	ZL	p	25.000	kN/m
7	Lines	140,156,158,160,162	Force	Uniform	ZL	p	20.000	kN/m

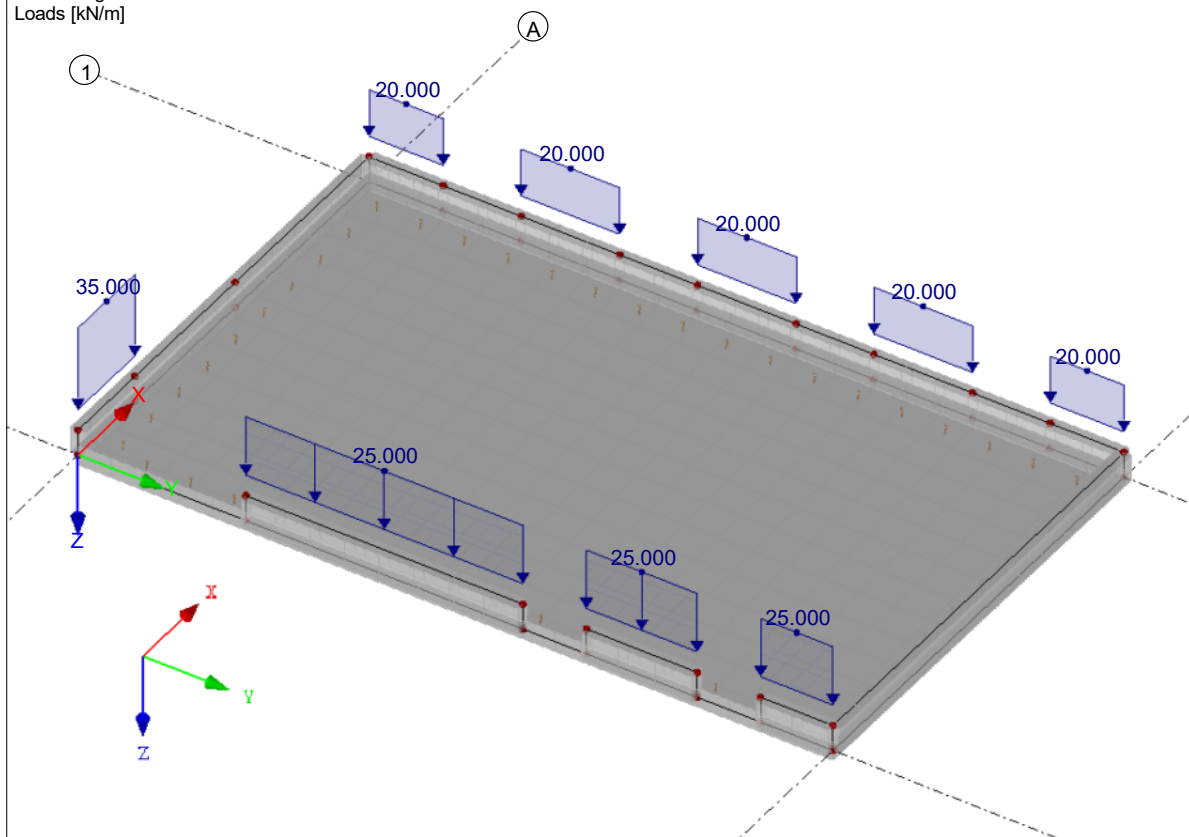
Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

LC4: SNEG

LC4 : sneg
Loads [kN/m]

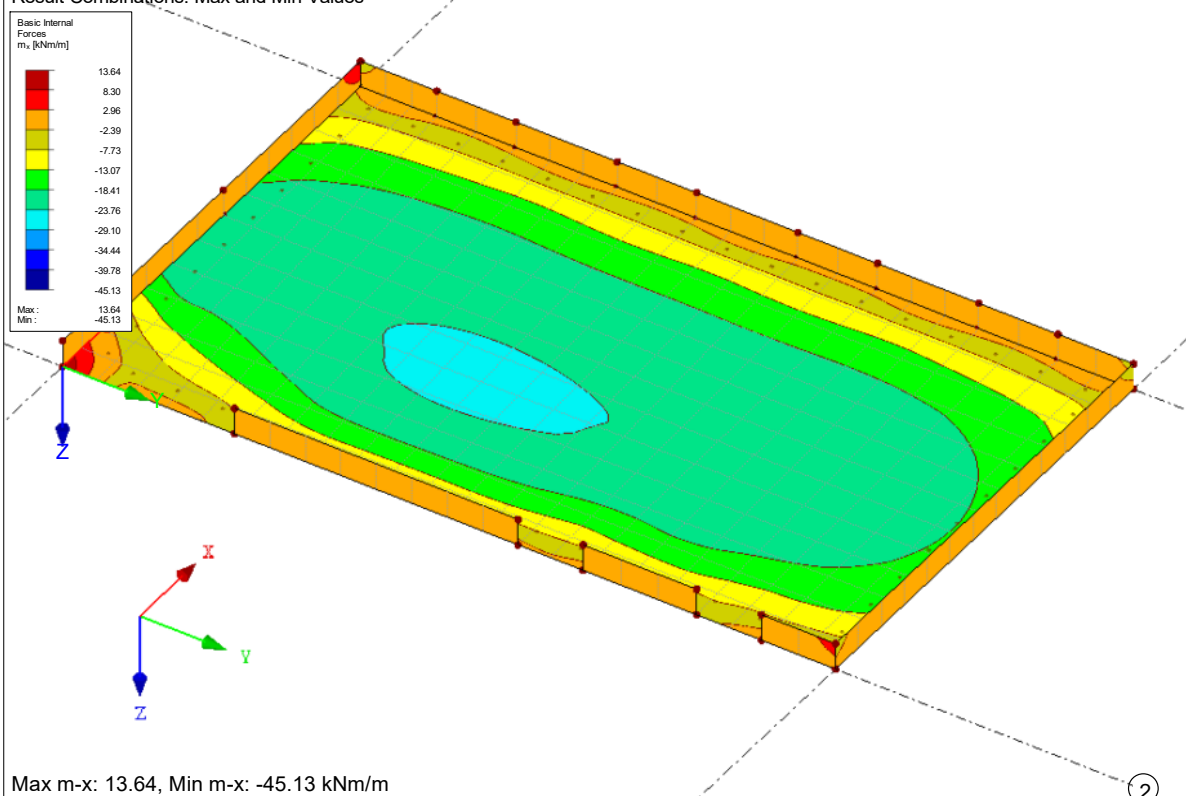
Isometric



BASIC INTERNAL FORCES m_x

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10
Basic Internal Forces m-x [kNm/m]
Result Combinations: Max and Min Values

Isometric



Max m-x: 13.64, Min m-x: -45.13 kNm/m

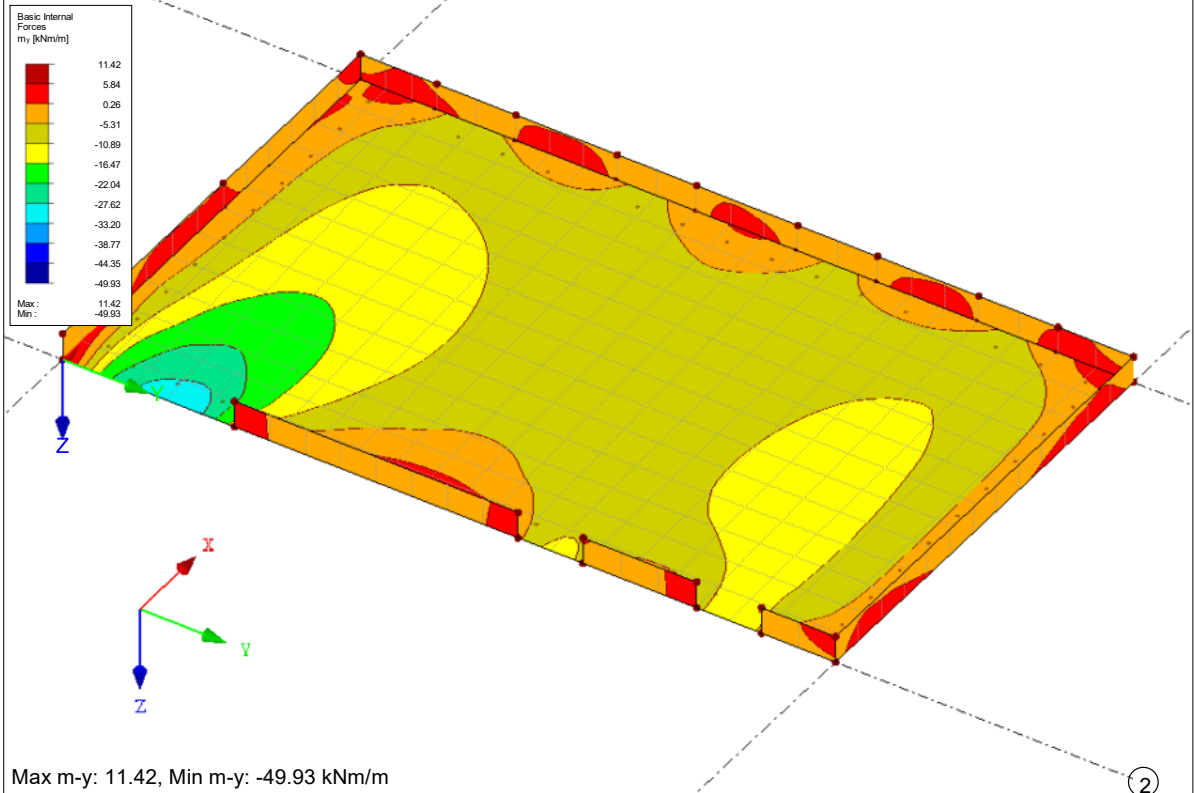
Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

■ BASIC INTERNAL FORCES m_y

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10
Basic Internal Forces m_y [kNm/m]
Result Combinations: Max and Min Values

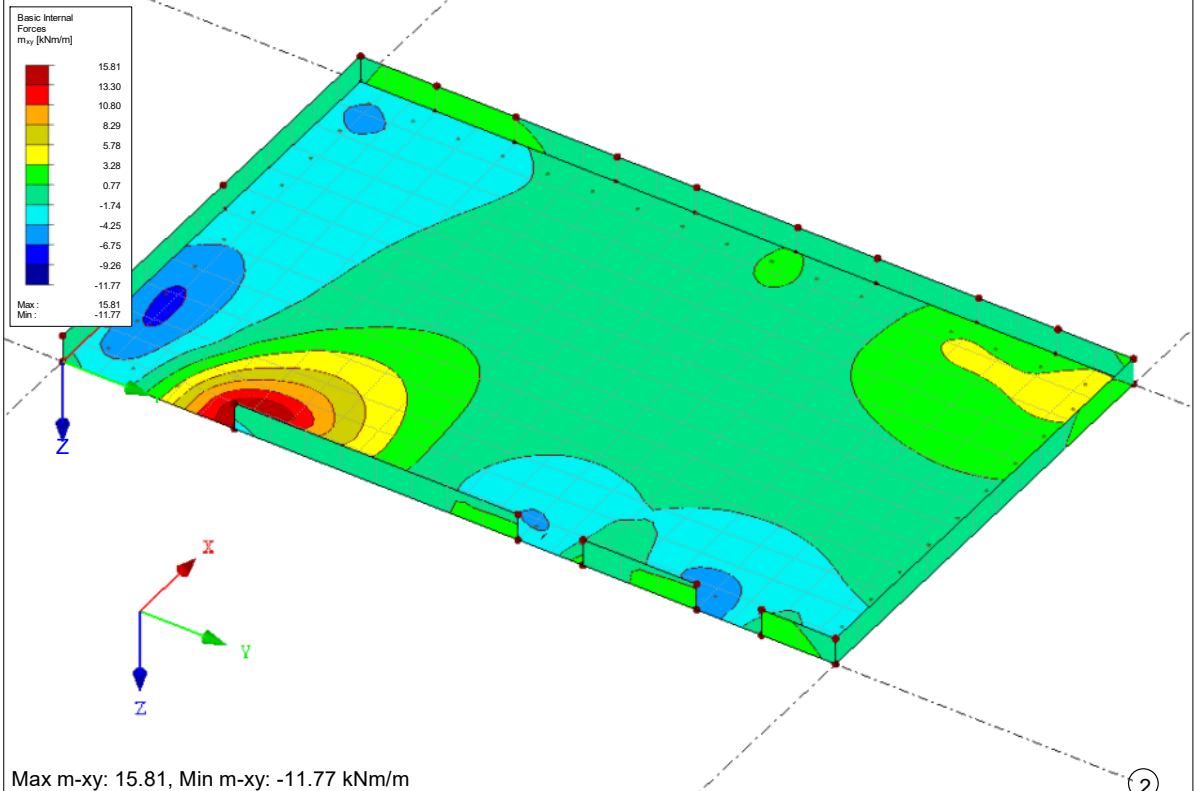
Isometric



■ BASIC INTERNAL FORCES m_{xy}

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10
Basic Internal Forces m_{xy} [kNm/m]
Result Combinations: Max and Min Values

Isometric



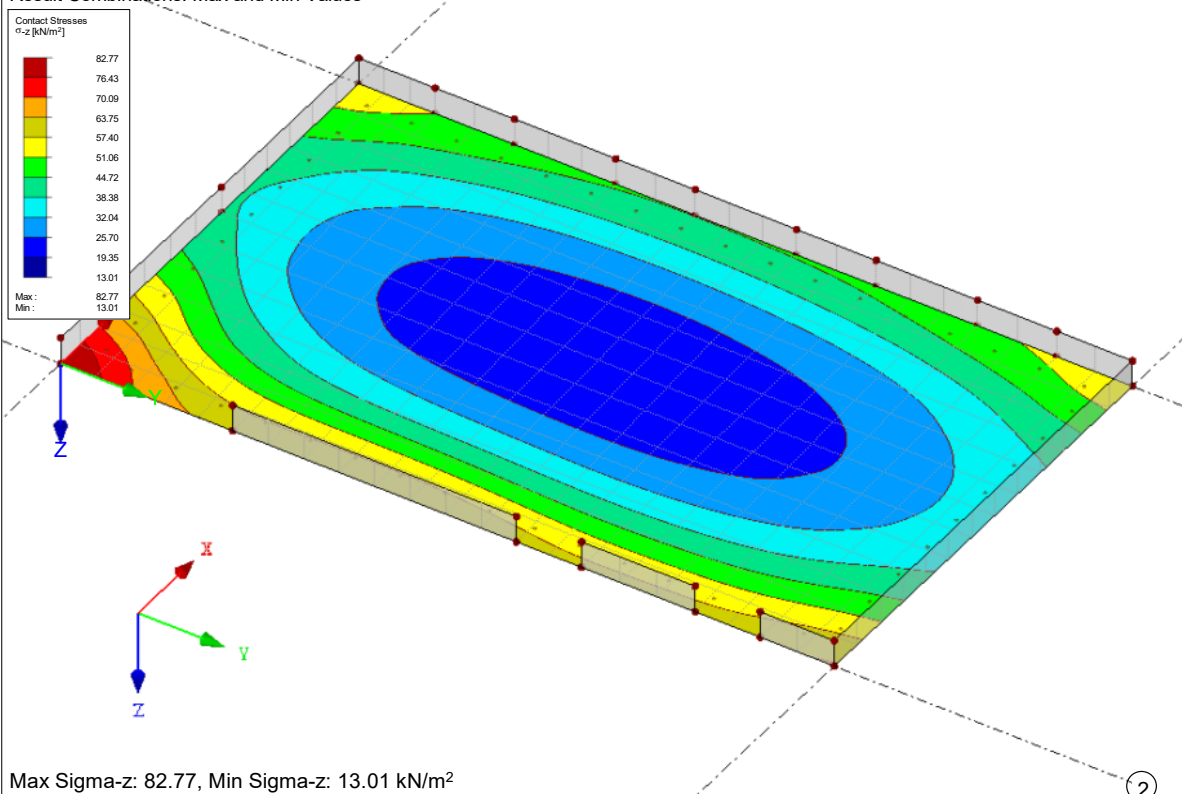
Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

CONTACT STRESSES σ_z

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10
Contact Stresses Sigma-z [kN/m²]
Result Combinations: Max and Min Values

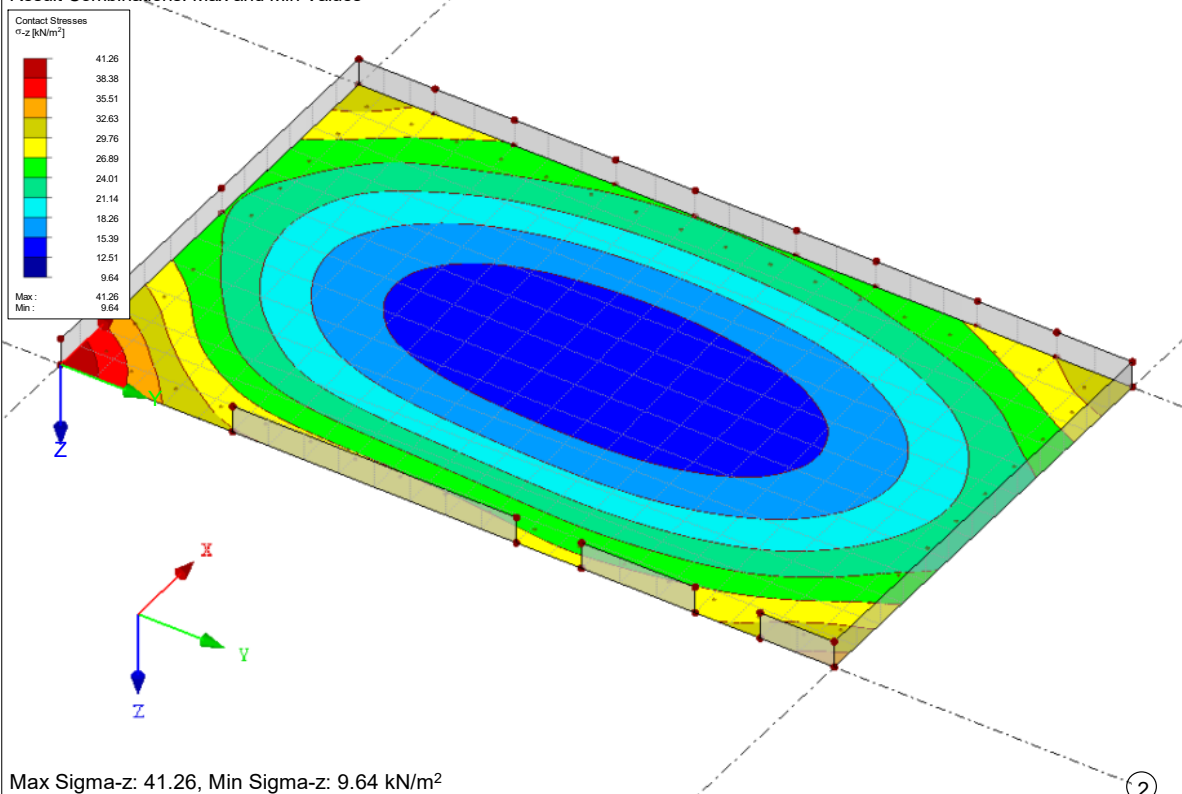
Isometric



CONTACT STRESSES σ_z

RC2 : SLS - Quasi-permanent
Contact Stresses Sigma-z [kN/m²]
Result Combinations: Max and Min Values

Isometric



RF-CONCRETE Surfaces

CA1

Reinforced concrete design

Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

1.1 GENERAL DATA

Design according to Standard:	SIST EN 1992-1-1:2005/A101:2006
ULTIMATE LIMIT STATE	
Result combination for design:	RC1 ULS (STR/GEO) - Permanent / transient - Eq. 6.10 Persistent and Transient
SERVICEABILITY LIMIT STATE	
Load combinations for design:	CO16 G + 0.6Q _{iC} + 0.3Q _{iA} Quasi-permanent, k _i 0.600
Definition of Provided Additional Reinforcement	Automatic arrangement according to the specifications in Table 1.4
Type of SLS method:	Nonlinear Method According to EN 1992-1-1, 5.7 (4): 'Nonlinear Analysis'
Consider creeping	☒
Consider shrinkage	☐
Design of	
Deformation Analysis	☒
Crack width control	☒
Concrete Stress Analysis	☐
Steel Stress Analysis	☒
Stress-strain diagram for concrete in compression:	Parabolic
Stress-strain diagram for concrete in tension:	Tension stiffening with concrete tensile strength (Quast method)
Adjustment factor of tensile strength f _{ct,R} :	0.60
Material Concrete - Calculation Parameters:	
Concrete C30/37	Factor 21.84 v = f _c / f _{ct} , R: Exponent 2.01 nt n-PR: Exponent 1.00 nt n-VMB:
Use strength of steel up to ultimate tensile strength	☒
Settings for the iteration process:	
Maximum number of iterations:	200
Number of load increments:	1
Number of layers in net element:	10
DETAILS	
Analysis Method for Reinforcement Envelope	Mixed
Apply the internal forces without the rib components	☐
Design Situation Settings for Serviceability Limit State Checks	
Load combination:	
Characteristic with direct load	Checks: k ₁ *f _{ck} , k ₃ *f _{yk}
Characteristic with imposed deformation	Checks: k ₁ *f _{ck} , k ₄ *f _{yk}
Frequent	Checks: w _k
Quasi-permanent	Checks: k ₂ *f _{ck} , w _k , u _i

1.2 MATERIALS

Material No.	Material Description	Comment
1	Concrete C30/37 B 500 S (A)	

1.3 SURFACES

Surface No.	Matl. No.	Creep coeff. φ [-]	u _{z,max} [mm]	w _{k,+z} (top) [mm]	w _{k,-z} (bottom) [mm]	Notes
29	1	Thickness Type: Constant, Thickness: 300.00 mm 2.21496	3.600	0.300	0.300	Deformation referred to undeformed system
30	1	Thickness Type: Constant, Thickness: 215.00 mm 2.32595	1.520	0.300	0.300	Deformation referred to undeformed system
31	1	Thickness Type: Constant, Thickness: 215.00 mm 2.32595	1.520	0.300	0.300	Deformation referred to undeformed system
32	1	Thickness Type: Constant, Thickness: 215.00 mm 2.32595	1.520	0.300	0.300	Deformation referred to undeformed system
33	1	Thickness Type: Constant, Thickness: 215.00 mm 2.32595	1.520	0.300	0.300	Deformation referred to undeformed system
34	1	Thickness Type: Constant, Thickness: 215.00 mm 2.32595	1.520	0.300	0.300	Deformation referred to undeformed system
35	1	Thickness Type: Constant, Thickness: 215.00 mm 2.32595	1.520	0.300	0.300	Deformation referred to undeformed system

1.4 REINFORCEMENT GROUP NO. 1 - AB TEMELJNA PLOSCA IN PARAPETI

Applied to surfaces:	29-35
REINFORCEMENT RATIO	
Minimum secondary reinforcement	20.0 %

Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

1.4 REINFORCEMENT GROUP NO. 1 - AB TEMELJNA PLOSCA IN PARAPETI

Basic minimum reinforcement	0.0 %
Minimum compression reinforcement	0.0 %
Minimum tension reinforcement	0.0 %
Maximum reinforcement percentage	4.0 %
Minimum shear reinforcement percentage	0.0 %
REINFORCEMENT AREA FOR DESIGN OF SLS	
Use provided basic reinforcement and required additional reinforcement acc. to Tables 2.1, 2.2, 2.3	
Concrete cover acc. to Standard	☐
BASIC REINFORCEMENT LAYOUT - TOP (-z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area	As-1,-z (top): 3.85, As-2,-z (top): 3.85 cm²/m
BASIC REINFORCEMENT LAYOUT - BOTTOM (+z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area	As-1,+z (bottom): 5.03, As-2,+z (bottom): 5.03 cm²/m
ADDITIONAL REINFORCEMENT LAYOUT - TOP (-z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area	Use required additional reinforcement acc. to Tables 2.1, 2.2, 2.3
ADDITIONAL REINFORCEMENT LAYOUT - BOTTOM (+z)	
Number of directions	2
Cover to rebar centroid	d-1: 30.00, d-2: 40.00 mm
Bar diameter	ds-1: 10.00, ds-2: 10.00 mm
Directions of reinforcement	Phi-1: 0.000°, Phi-2: 90.000°
Reinforcement area	Use required additional reinforcement acc. to Tables 2.1, 2.2, 2.3
LONGITUDINAL REINFORCEMENT FOR SHEAR FORCE DESIGN	
Apply the greater value resulting from either the required or provided reinforcement (basic and add. reinforcement) per reinforcement direction	
OPTIONS FOR SIST EN 1992-1-1:2005/A101:2006	
Minimum longitudinal reinforcement for plates acc. to 9.3.1	☐
Minimum longitudinal reinforcement for walls acc. to 9.6	☒
Minimum shear reinforcement	☒
Neutral axis depth limitation	☒
Variable strut inclination - min	21.801 °
Variable concrete strut inclination - max	45.000 °
Partial safety factor γ_s	PT 1.15, AC 1.00, SLS 1.00
Partial safety factor γ_c	PT 1.50, AC 1.20, SLS 1.00
Consideration of long-term effects Alpha-cc	PT 1.00, AC 1.00, SLS 1.00
Consideration of long-term effects Alpha-ct	SLS 1.00

2.1 REQUIRED REINFORCEMENT TOTAL

Surface No.	Point No.	Point Coordinates [m]			Symbol	Required Reinf. ULS	Basic Reinf.	Additional Reinforcement Required	Additional Reinforcement Provided	Unit	Notes
30	M329	2.898	0.000	-0.380	a _{s,1,-z} (top)	13.53	3.85	9.68	9.68	cm²/m	
29	M58	0.000	1.904	0.000	a _{s,2,-z} (top)	5.76	3.85	1.91	1.91	cm²/m	
30	M329	2.898	0.000	-0.380	a _{s,1,+z} (bottom)	13.74	5.03	8.71	8.71	cm²/m	
30	M133	1.248	0.000	-0.380	a _{s,2,+z} (bottom)	5.03	5.03	0.00	0.00	cm²/m	
29	M86	0.000	2.871	0.000	a _{sw}	8.76	-	-	-	cm²/m²	

3.1 NONLINEAR CALCULATION TOTAL

Surface No.	Point No.	Point Coordinates [m]			Load Case	Type	Exist. Value	Design Limit Value	Unit	Ratio	Notes
29	M106	0.000	0.000	0.000	CO16	U _{z,local}	3.437	3.600	mm	1.0	
29	M1	3.445	3.214	0.000	CO16	W _k	0.000	0.300	mm	0.0	
29	M1	3.445	3.214	0.000		σ _s	undefined	400.00	N/mm²		108)

NONLINEAR - NOTES

No.	Description
108)	The design is not possible! The specified loads (LC/CO/RC) with the selected types of load combinations (characteristic/frequent/quasi-permanent) do not contain the required checks (stress/crack width/deflection).

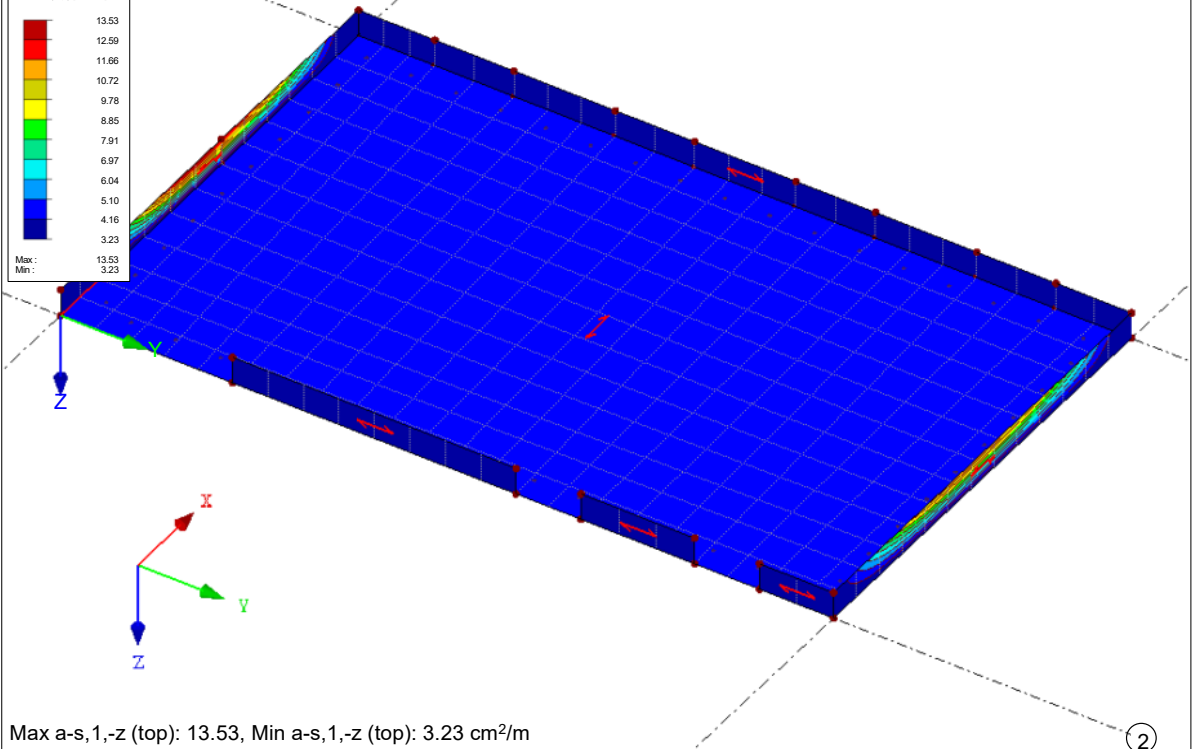
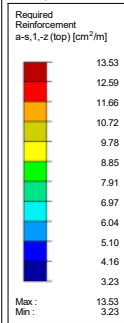
Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

■ REQUIRED REINFORCEMENT $a_{s,1,-z}$ (top)

RF-CONCRETE Surfaces CA1
Reinforced concrete design

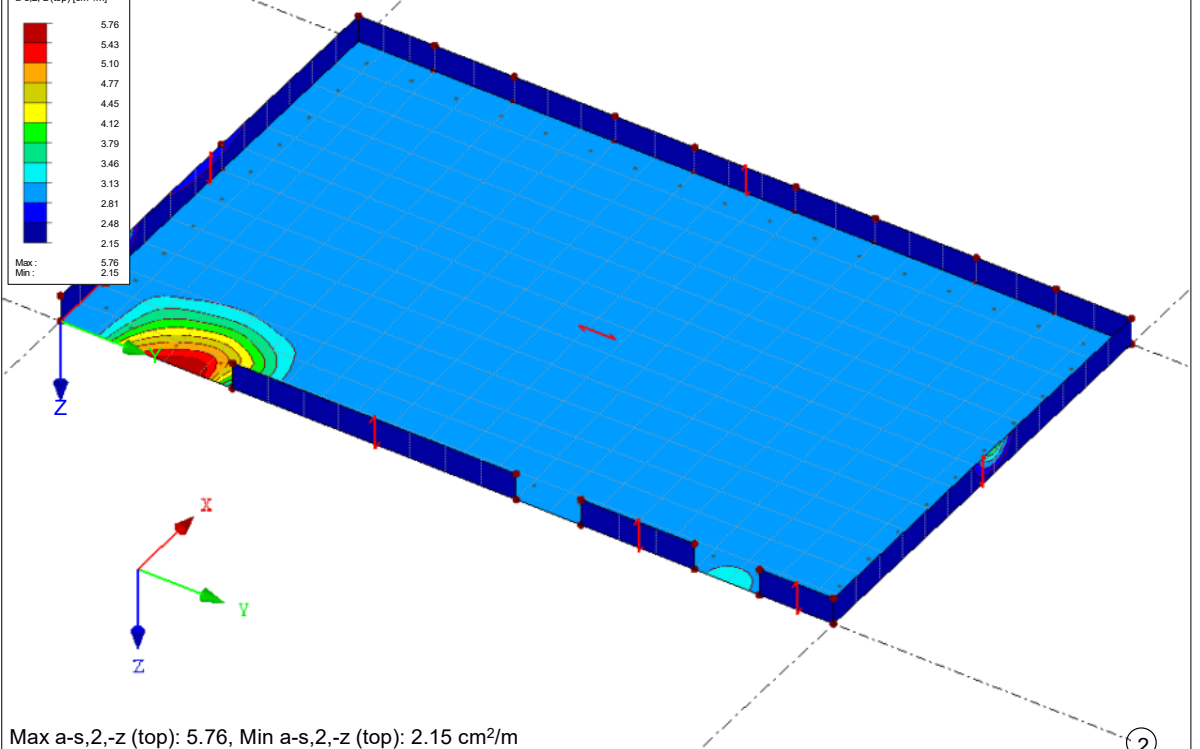
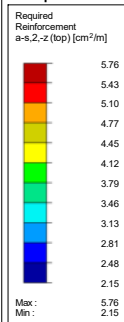
Isometric

Required Reinforcement $a_{s,1,-z}$ (top) [cm²/m]


■ REQUIRED REINFORCEMENT $a_{s,2,-z}$ (top)

RF-CONCRETE Surfaces CA1
Reinforced concrete design

Isometric

Required Reinforcement $a_{s,2,-z}$ (top) [cm²/m]


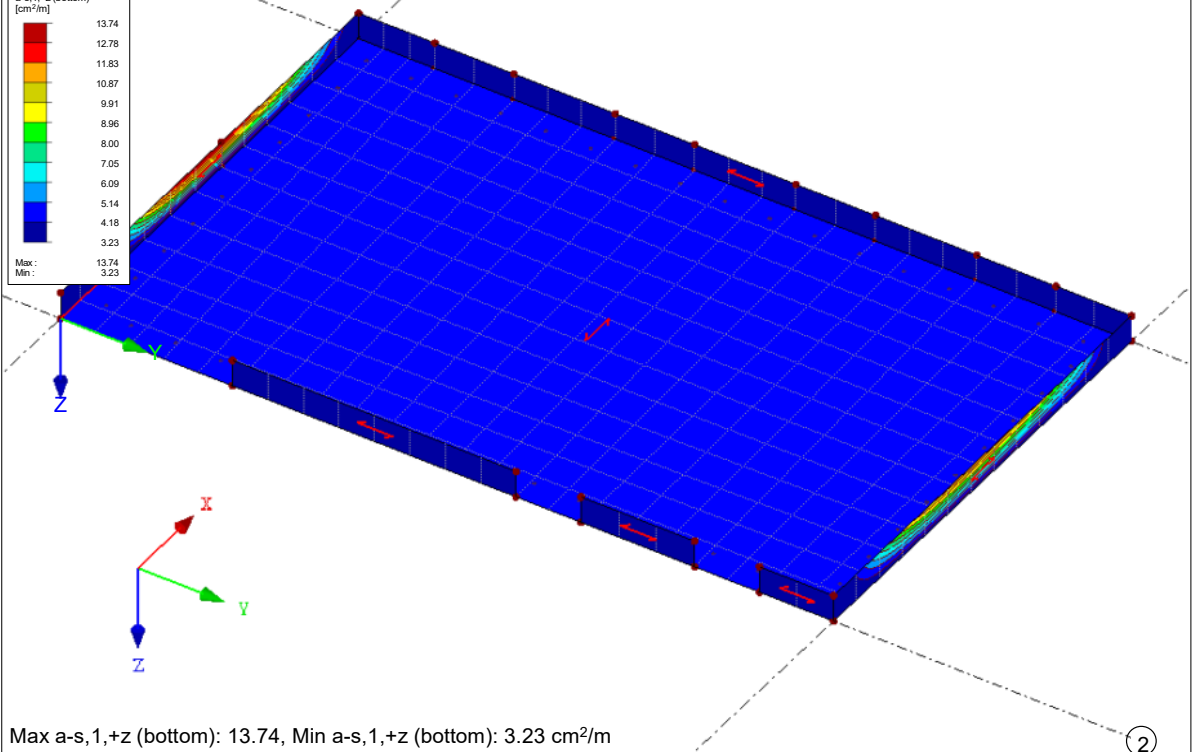
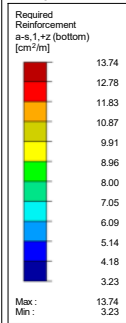
Project: VRTEC MEZICA

Model: TEMELJI_KUHINJA_PRIZIDEK

REQUIRED REINFORCEMENT $a_{s,1,+z}$ (bottom)

RF-CONCRETE Surfaces CA1
Reinforced concrete design

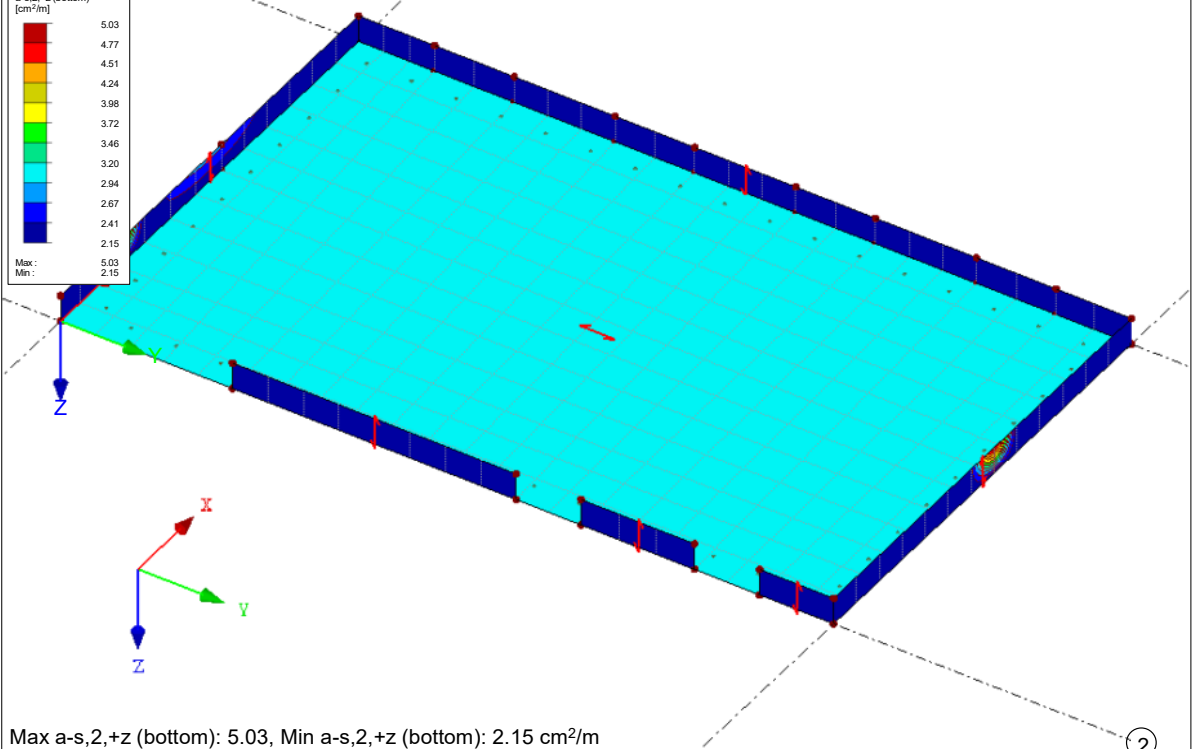
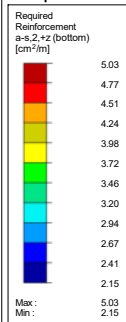
Required Reinforcement $a_{s,1,+z}$ (bottom) [cm²/m]



REQUIRED REINFORCEMENT $a_{s,2,+z}$ (bottom)

RF-CONCRETE Surfaces CA1
Reinforced concrete design

Required Reinforcement $a_{s,2,+z}$ (bottom) [cm²/m]



2.5	STATIČNI IZRAČUN – LESENE IN JEKLENE KONSTRUKCIJE	
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Izračuni in dimenzioniranje je izvedeno s pomočjo računalniškega programa za statično in dinamično analizo prostorskih konstrukcij RFEM 5 in Timbertech.

Izračun je bil narejen za naslednja projektna stanja:

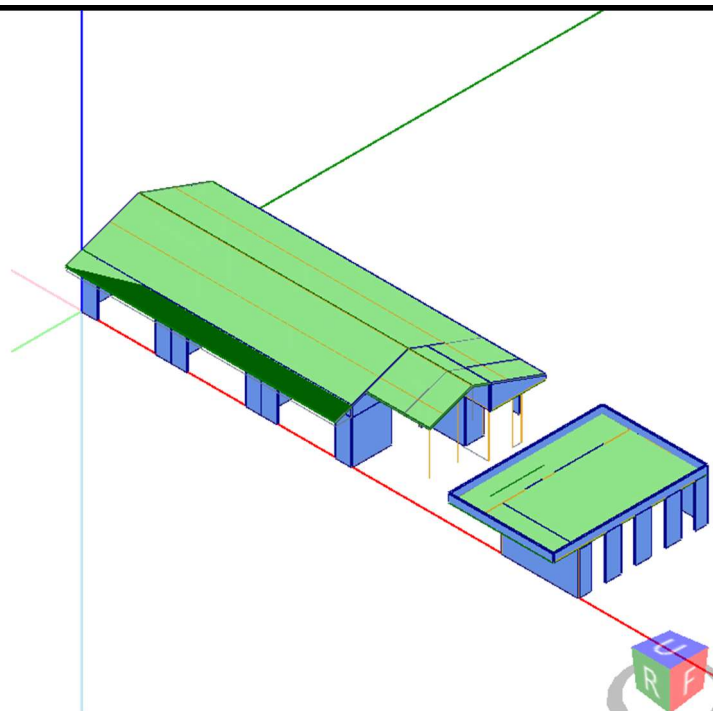
- mejno stanje nosilnosti MSN,
- mejno stanje uporabnosti MSU,
- potresno projektno stanje PPS.

Dolenja vas, maj 2026

Sestavil:
Miha Oražem; univ.dipl.inž.grad.



STATIČNI IZRAČUN LESENIH IN JEKLENIH KONSTRUKCIJ OBJEKTA – nove konstrukcije



Project: OŠ MEŽICA VRTEC - PRIZIDEK in REKONSTRUKCIJA

Date: maj 2026

Design codes and standards

The analysis are done according to: Eurocodes.

1. EN 1990 – Eurocode 0

Basis of structural design

2. EN 1993-1-1 – Eurocode 3

Design of steel structures - Part 1-1: General rules and rules for buildings

3. EN 1993-1-5 – Eurocode 3

Design of steel structures - Part 1-5: Plated structural elements

4. EN 1993-1-8 – Eurocode 3

Design of steel structures - Part 1-8: Design of joints

5. EN 1995-1-1 – Eurocode 5

Design of timber structures - Part 1-1: General - Common rules and rules for buildings

6. EN 1995-1-2 – Eurocode 5

Design of timber structures - Part 1-2: General – Structural fire design

7. EN 1998-1-1 – Eurocode 8

Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings

8. EN 338

Structural timber - Strength classes

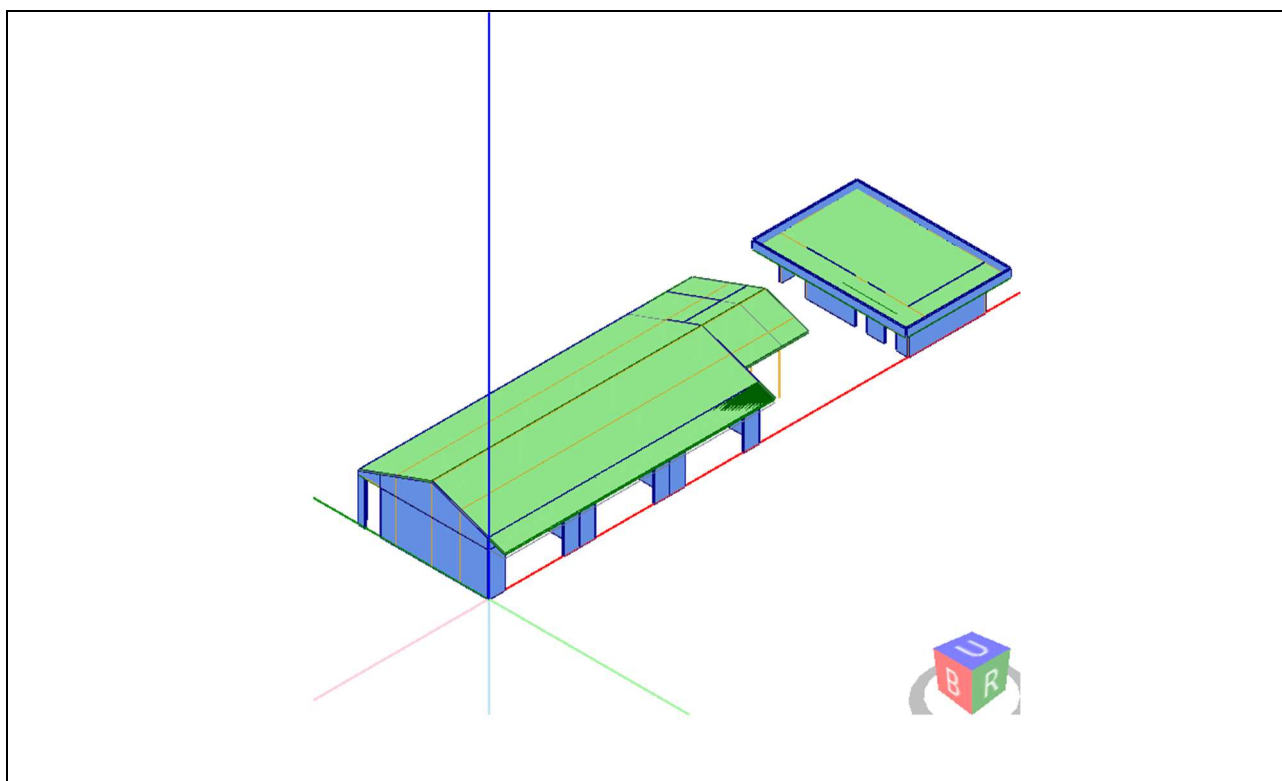
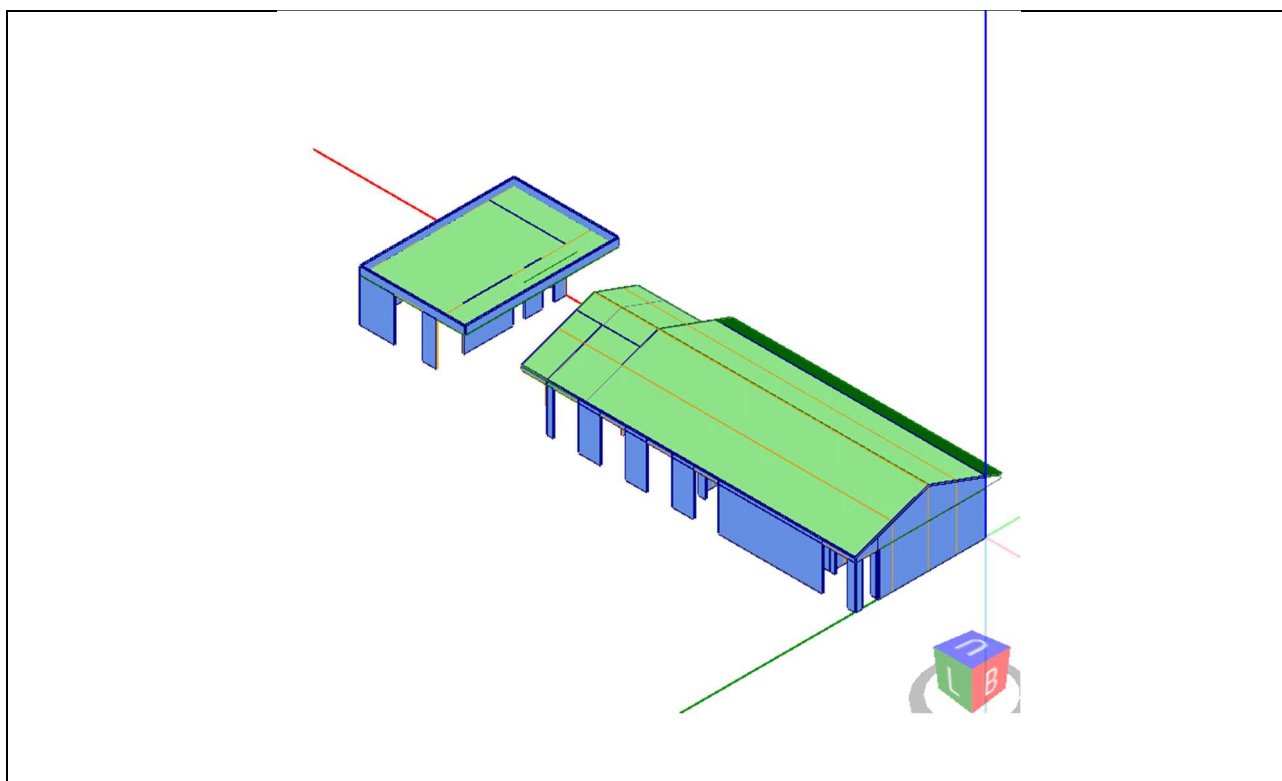
9. EN 14080

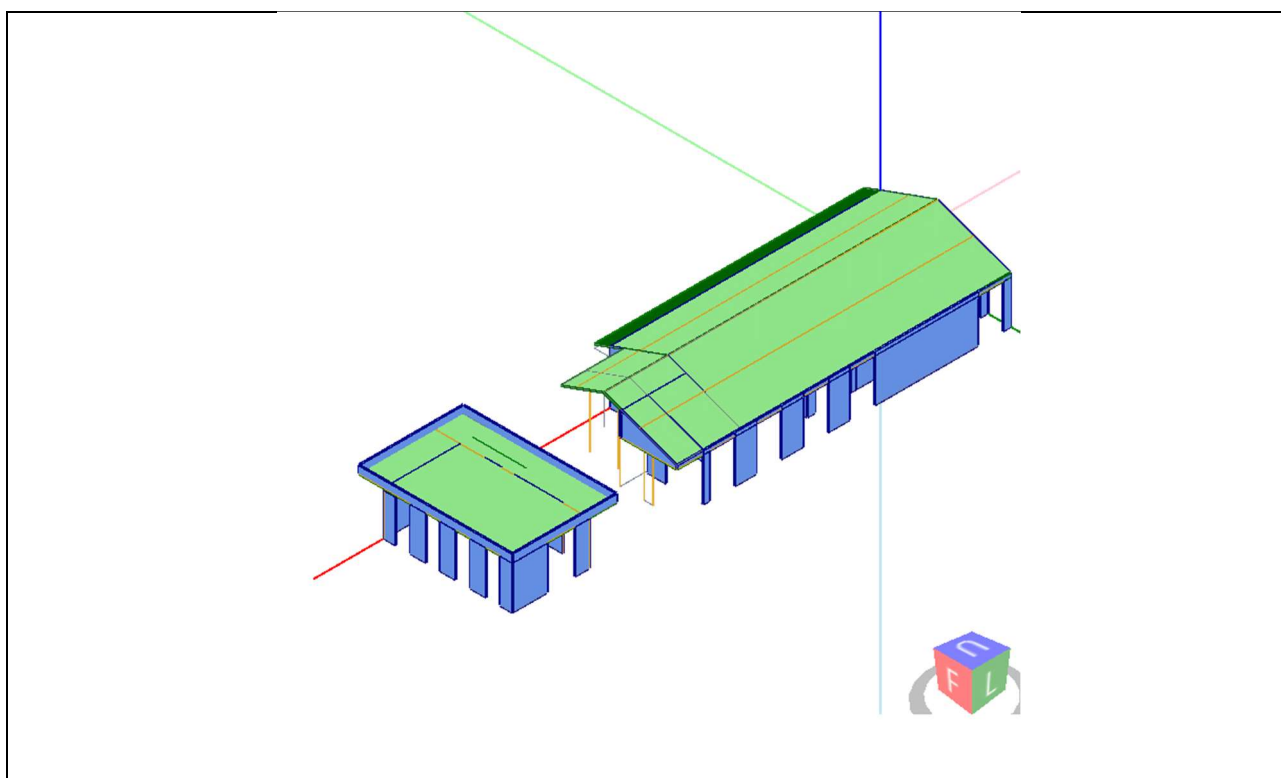
Timber structures - Glued laminated timber and glued solid timber - Requirements

10. EN 10025

Hot rolled products of structural steels

General description of the building





Calculation software used

Calculation software features

The software used is *Timber Tech Buildings*, developed by Timber Tech srl, start up of the University of Trento (Italy).

Technical specifications

Name:	TimberTech Buildings
Software Producer:	TimberTech srl
	Via della Villa, 22/A
	I-38123 – Trento (TN) – Italy
Version:	111

Materials

Wooden materials

The materials used in the project are listed in the following tables.

Descr. Description

$f_{m,k}$ Characteristic bending strength

$f_{t,0,k}$ Characteristic tensile strength along the grain

$f_{t,90,k}$ Characteristic tensile strength perpendicular to the grain

$f_{c,0,k}$ Characteristic compressive strength along the grain

$f_{c,90,k}$ Characteristic compressive strength perpendicular to the grain

$f_{v,k}$ Characteristic shear strength

$E_{0,mean}$ Mean value of modulus of elasticity along the grain

$E_{0,05}$ Fifth percentile value of modulus of elasticity along the grain

$E_{90,mean}$ Mean value of modulus of elasticity perpendicular to the grain

G_{mean} Mean value of shear modulus

ρ_k Characteristic density

$f_{v,k,inplane}$ Characteristic in-plane shear strength of CLT panel

$f_{R,k}$ Characteristic rolling shear strength

$f_{T,k}$ Torsional resistance of crossing surfaces

$G_{R,mean}$ Mean value of rolling shear modulus

Homogeneous glued-laminated timber

Descr.	$f_{m,k}$ [MPa]	$f_{t,0,k}$ [MPa]	$f_{t,90,k}$ [MPa]	$f_{c,0,k}$ [MPa]	$f_{c,90,k}$ [MPa]	$f_{v,k}$ [MPa]	$E_{0,mean}$ [MPa]	$E_{0,05}$ [MPa]	$E_{90,mean}$ [MPa]	G_{mean} [MPa]	ρ_k [kg/m ³]
GL 24h - EN 14080	24	19.2	0.5	24	2.5	3.5	11500	9600	300	650	385
GL 28h - EN 14080	28	22.3	0.5	28	2.5	3.5	12600	10500	300	650	425

Softwood

Descr.	$f_{m,k}$ [MPa]	$f_{t,0,k}$ [MPa]	$f_{t,90,k}$ [MPa]	$f_{c,0,k}$ [MPa]	$f_{c,90,k}$ [MPa]	$f_{v,k}$ [MPa]	$E_{0,mean}$ [MPa]	$E_{0,05}$ [MPa]	$E_{90,mean}$ [MPa]	G_{mean} [MPa]	ρ_k [kg/m ³]
C 24	24	14.5	0.4	21	2.5	4	11000	7400	370	690	350

CLT

Descr	$f_{m,k}$ [MPa]	$f_{t,0,k}$ [MPa]	$f_{t,90,k}$ [MPa]	$f_{c,0,k}$ [MPa]	$f_{c,90,k}$ [MPa]	$f_{v,k,plast}$ [MPa]	$f_{R,k}$ [MPa]	$f_{v,k,last}$ [MPa]	$f_{T,k}$ [MPa]	$E_{0,mean}$ [MPa]	$E_{0,05}$ [MPa]	$E_{90,mean}$ [MPa]	G_{mean} [MPa]	$G_{R,mean}$ [MPa]	ρ_k [kg/m ³]
C 24 BBS XL	24	14	0.4	21	2.5	4	1	8	2.5	12000	7400	370	690	50	350

OSB

Descr.	Standard	Utilization	$f_{v,k}$ [MPa]	G_{mean} [MPa]	ρ_k [kg/m ³]
OSB/3	EN 300 Type OSB/3	Humid conditions	6.8	1080	550

Steel material

The materials used in the project are listed in the following tables.

Descr. Description

t Nominal thickness of the element

$f_{y,k}$ Characteristic values of yield strength

$f_{u,k}$ Characteristic values of ultimate strength

E Modulus of elasticity

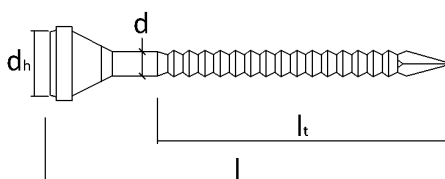
G Shear modulus

ν Poisson's ratio in elastic stage

Descr.	t	$f_{y,k}$ [MPa]	$f_{u,k}$ [MPa]	E [MPa]	G [MPa]	ν [MPa]
S235 - EN 10025 -2	$t \leq 40$ mm	235	360	210000	80770	0.3

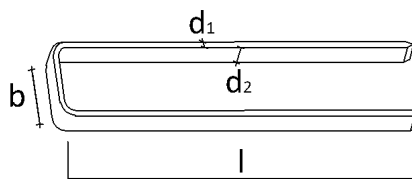
Metal fasteners

Anker nails



Manufacturer	Code	Descr.	l [mm]	l_t [mm]	d [mm]	d_h [mm]
Rotho Blaas	LBA460	Anker nail - LBA 4,0 X 60	60	50	4	8

Staples



Manufacturer	Code	Descr.	l [mm]	Cross-section type	Cross-section dimensions d1xd2 [mm]	Width b [mm]
Würth	0480 112 45	Staples type Q 45 galv.	45	Rectangular	1.56 x 1.87	11.3

Concrete anchors

Manufacturer	Threaded-rod / Mechanical anchor code	Threaded-rod / Mechanical anchor descr.	Chemical anchor code	Chemical anchor descr.
Rotho Blaas	INA5812195	Threaded rod INA - 5.8 - M12 x 195	HYB420	Hybrid chemical anchor ETA-20/1285
User defined	BR ATC1	Threaded rod - angle bracket ATC1	CH ATC1	Chemical anchor - angle bracket ATC1

Identification of structural elements

Vertical elements

The following table shows the positions of the individual walls. The last four columns show the coordinates of the ends of each wall.

X1 and Y1 indicate the coordinates of the starting point of the wall

X2 and Y2 indicate the coordinates of the end point of the wall

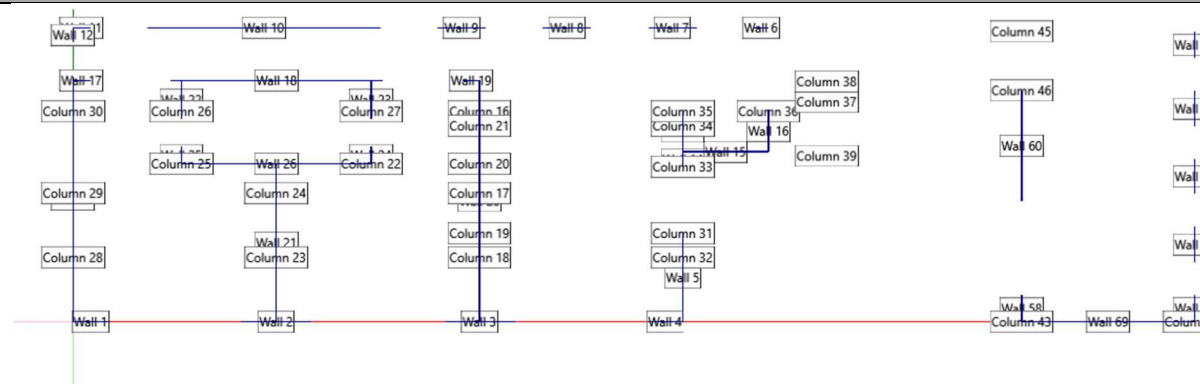
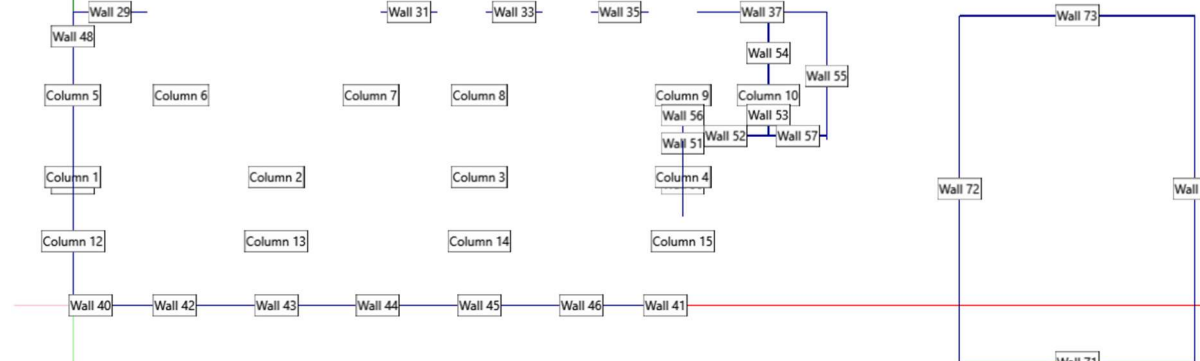
Wall name	Type of wall	Element resistant to horizontal loads	Height [m]	Length [m]	Altitude [m]	X1 [m]	Y1 [m]	X2 [m]	Y2 [m]
Wall 1	Frame	Yes	3.6	1.3	0	0	0	1.3	0
Wall 10	Frame	Yes	3.6	8.61	0	11.34	10.83	2.73	10.83
Wall 11	Frame	No	3.6	0.63	0	0.63	10.83	0	10.83
Wall 12	Frame	No	3.6	0.53	0	0	10.83	0	10.3
Wall 13	Frame	Yes	3.6	9.01	0	0	9.01	0	0
Wall 14	Frame	No	3.6	0.55	0	22.5	5.71	22.5	6.26
Wall 15	Frame	Yes	3.6	3.16	0	22.5	6.26	25.66	6.26
Wall 16	Frame	Yes	3.6	1.54	0	25.66	6.26	25.66	7.8
Wall 17	Frame	No	3.6	0.63	0	0	8.89	0.63	8.89
Wall 18	Frame	Yes	3.6	7.86	0	3.58	8.89	11.44	8.89
Wall 19	Frame	No	3.6	0.62	0	14.38	8.89	15	8.89
Wall 2	Frame	Yes	3.6	2.6	0	6.2	0	8.8	0
Wall 20	Frame	Yes	3.6	8.89	0	15	8.89	15	0
Wall 21	Frame	Yes	3.6	5.82	0	7.5	0	7.5	5.82
Wall 22	Frame	Yes	3.6	1.43	0	4	7.46	4	8.89
Wall 23	Frame	Yes	3.6	1.43	0	11	7.46	11	8.89
Wall 24	Frame	No	3.6	0.64	0	11	5.82	11	6.46
Wall 25	Frame	No	3.6	0.64	0	4	5.82	4	6.46
Wall 26	Frame	Yes	3.6	7	0	4	5.82	11	5.82
Wall 27	Frame	Yes	3.6	1.49	0	22.5	7.75	22.5	6.26
Wall 29	Frame	No	0.3	2.73	3.6	0	10.83	2.73	10.83
Wall 3	Frame	Yes	3.6	2.6	0	13.7	0	16.3	0
Wall 31	Frame	No	0.3	2.1	3.6	11.34	10.83	13.44	10.83
Wall 33	Frame	No	0.3	2.1	3.6	15.23	10.83	17.33	10.83
Wall 35	Frame	No	0.3	2.1	3.6	19.12	10.83	21.22	10.83
Wall 37	Frame	No	0.3	4.8	3.6	23.01	10.83	27.81	10.83
Wall 4	Frame	Yes	3.6	1.3	0	21.2	0	22.5	0
Wall 40	Frame	Yes	0.79	1.3	3.6	0	0	1.3	0
Wall 41	Frame	Yes	0.79	1.3	3.6	21.2	0	22.5	0
Wall 42	Frame	No	0.79	4.9	3.6	1.3	0	6.2	0
Wall 43	Frame	Yes	0.79	2.6	3.6	6.2	0	8.8	0

Wall 44	Frame	No	0.79	4.9	3.6	8.8	0	13.7	0
Wall 45	Frame	Yes	0.79	2.6	3.6	13.7	0	16.3	0
Wall 46	Frame	No	0.79	4.9	3.6	16.3	0	21.2	0
Wall 47	Frame	Yes	1.68	9.01	3.6	0	9.01	0	0
Wall 48	Frame	No	0.63	1.82	3.6	0	10.83	0	9.01
Wall 5	Frame	Yes	3.6	3.26	0	22.5	0	22.5	3.26
Wall 50	Frame	No	2.27	2.45	3.6	22.5	3.26	22.5	5.71
Wall 51	Frame	No	2.05	0.55	3.6	22.5	5.71	22.5	6.26
Wall 52	Frame	Yes	1.95	3.16	3.6	22.5	6.26	25.66	6.26
Wall 53	Frame	Yes	1.67	1.54	3.6	25.66	6.26	25.66	7.8
Wall 54	Frame	No	0.85	3.03	3.6	25.66	7.8	25.66	10.83
Wall 55	Frame	No	1.15	4.72	3.6	27.81	6.11	27.81	10.83
Wall 56	Frame	Yes	1.68	1.49	3.6	22.5	6.26	22.5	7.75
Wall 57	Frame	No	1.95	2.15	3.6	25.66	6.26	27.81	6.26
Wall 58	Frame	No	3.5	0.98	0	35	0	35	0.98
Wall 6	Frame	No	3.6	0.55	0	25.66	10.83	25.11	10.83
Wall 60	Frame	Yes	3.6	4.08	0	35	4.44	35	8.52
Wall 64	Frame	Yes	3.6	0.99	0	41.39	10.69	41.39	9.7
Wall 65	Frame	Yes	3.6	1.3	0	41.39	8.5	41.39	7.2
Wall 66	Frame	Yes	3.6	1.3	0	41.39	6	41.39	4.7
Wall 67	Frame	Yes	3.6	1.31	0	41.39	3.5	41.39	2.19
Wall 68	Frame	No	3.5	1	0	41.39	1	41.39	0
Wall 69	Frame	Yes	3.6	6.39	0	41.39	0	35	0
Wall 7	Frame	Yes	3.6	1.79	0	23.01	10.83	21.22	10.83
Wall 70	Frame	No	0.65	12.79	3.6	41.39	-2.1	41.39	10.69
Wall 71	Frame	No	0.65	8.69	3.6	41.39	-2.1	32.7	-2.1
Wall 72	Frame	No	0.65	12.79	3.6	32.7	-2.1	32.7	10.69
Wall 73	Frame	No	0.65	8.69	3.6	32.7	10.69	41.39	10.69
Wall 8	Frame	Yes	3.6	1.79	0	19.12	10.83	17.33	10.83
Wall 9	Frame	Yes	3.6	1.79	0	15.23	10.83	13.44	10.83

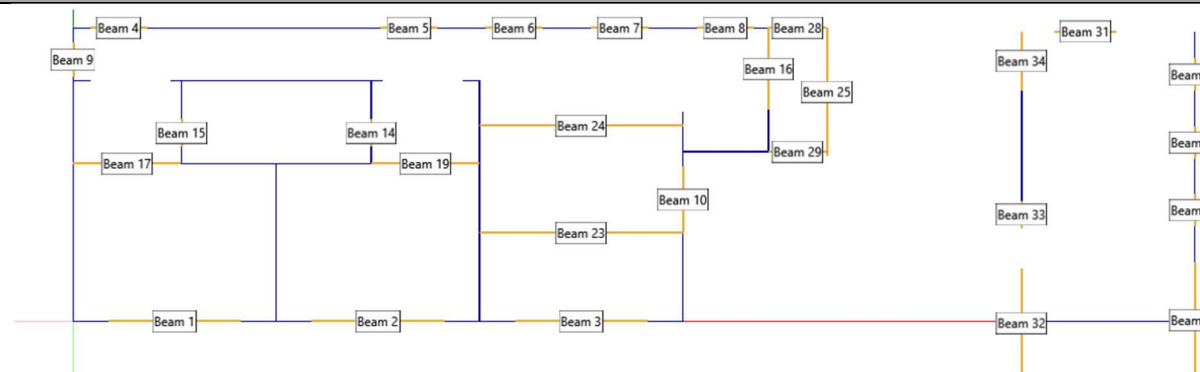
The following table shows the positions of the individual columns. X and Y are the coordinates of the point where the column is located.

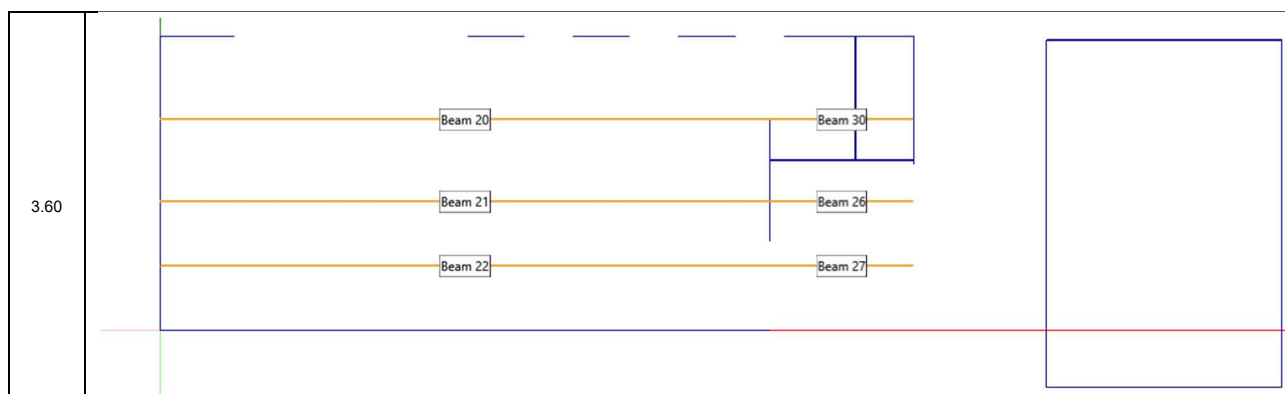
Column name	Height [m]	Altitude [m]	X [m]	Y [m]
Column 1	2.5	3.6	0	4.74
Column 10	1.413	3.6	25.66	7.75
Column 12	1.644	3.6	0	2.37
Column 13	1.644	3.6	7.5	2.37
Column 14	1.644	3.6	15	2.37
Column 15	1.644	3.6	22.5	2.37
Column 16	3.6	0	15	7.75
Column 17	3.6	0	15	4.74
Column 18	3.6	0	15	2.37
Column 19	3.6	0	15	3.26
Column 2	2.5	3.6	7.5	4.74
Column 20	3.6	0	15	5.82
Column 21	3.6	0	15	7.22
Column 22	3.6	0	11	5.82
Column 23	3.6	0	7.5	2.37
Column 24	3.6	0	7.5	4.74
Column 25	3.6	0	4	5.82
Column 26	3.6	0	4	7.75
Column 27	3.6	0	11	7.75
Column 28	3.6	0	0	2.37
Column 29	3.6	0	0	4.74
Column 3	2.5	3.6	15	4.74
Column 30	3.6	0	0	7.75
Column 31	3.6	0	22.5	3.26
Column 32	3.6	0	22.5	2.37
Column 33	3.6	0	22.5	5.71
Column 34	3.6	0	22.5	7.22
Column 35	3.6	0	22.5	7.75
Column 36	3.6	0	25.66	7.75
Column 37	3.6	0	27.81	8.1
Column 38	3.6	0	27.81	8.85
Column 39	3.6	0	27.81	6.11
Column 4	2.5	3.6	22.5	4.74
Column 42	3.6	0	41.39	0
Column 43	3.6	0	35	0
Column 45	3.6	0	35	10.69
Column 46	3.6	0	35	8.52
Column 5	1.413	3.6	0	7.75
Column 6	1.413	3.6	4	7.75
Column 7	1.413	3.6	11	7.75
Column 8	1.413	3.6	15	7.75
Column 9	1.413	3.6	22.5	7.75

Identification of vertical elements

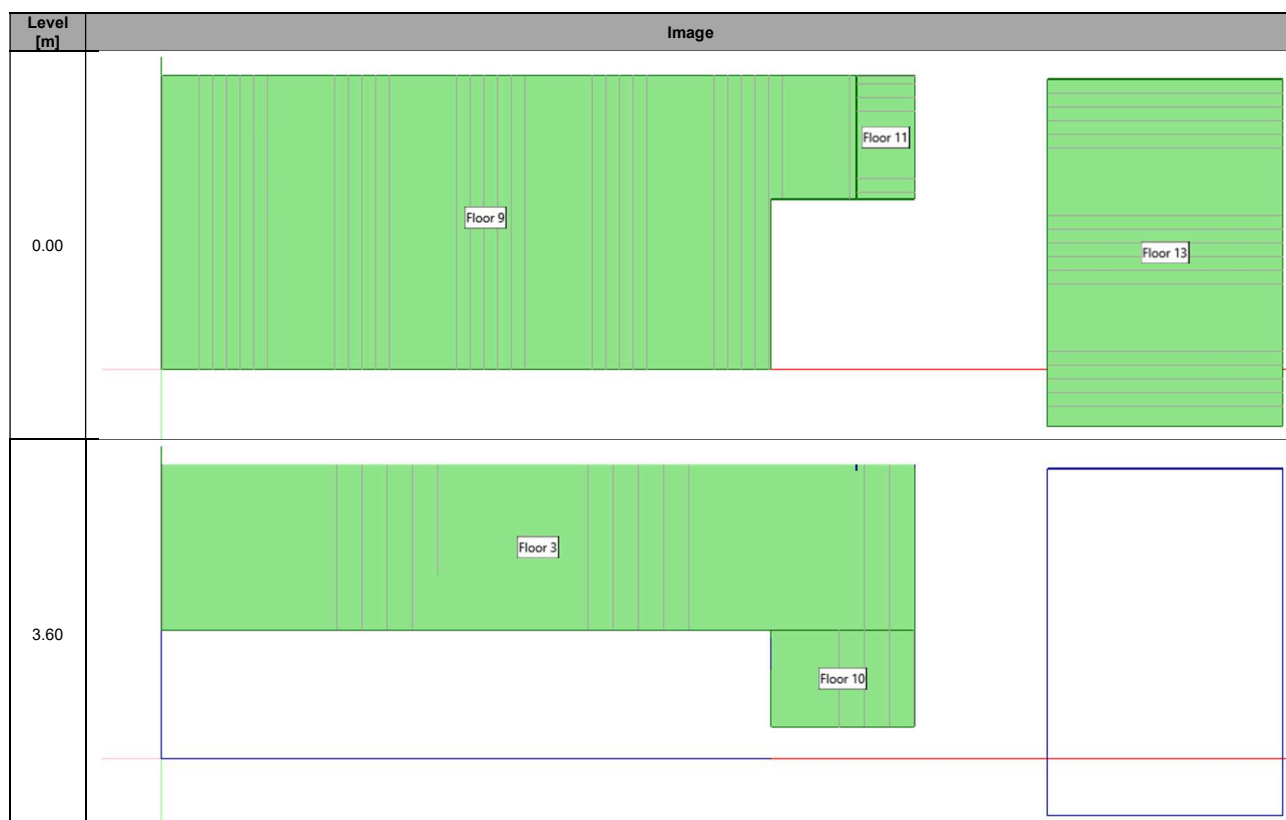
Level [m]	Image
0.00	
3.60	

Identification of beams

Level [m]	Image
0.00	



Identification of floors



Calculation method and numerical model

Model Description

Hypothesis adopted for the elements

The timber walls are constrained at the base by means of connection systems capable of transmitting both in-plane and out-of-plane actions action on the wall.

In the analysis, in presence of horizontal loads, some elements may be defined as “secondary”: this mean that their strength and stiffness are neglected in the calculation of the response of the building. In the model these elements are represented in terms of mass and they are designed only for vertical loads.

Rigid body rocking – Forces on hold-down / tie-down

The hold-down or tie-down systems are used to prevent the rotation of the wall caused by the overturning moment of the horizontal force. The hold-down, placed on the in-tensile edge of the wall, is loaded by a force equal to

$$T = \begin{cases} \left(\frac{M_{3-3}}{b} - \frac{N}{2} \right) \cdot \frac{1}{n_{anc}} & \text{for active hold – down} \\ 0 & \text{for inactive hold – down} \end{cases}$$

where:

b is the lever arm for the internal couple;

N is the axial vertical load acting on the wall;

M_{3-3} is the moment acting in the plane of the wall;

n_{anc} is the number of connections present at each end of the wall.

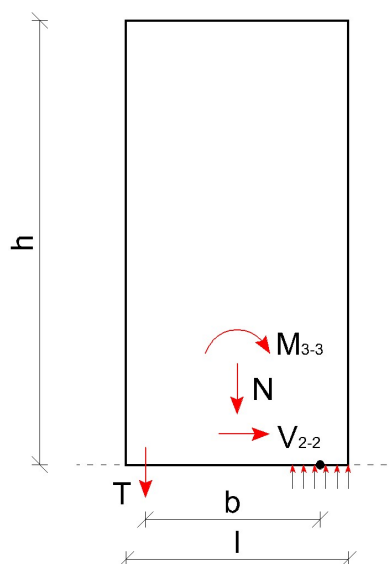


Figure: Calculation model of tensile force acting on the hold-down

Wall horizontal stiffness

The wall stiffness can be estimated considering the contributions of all the components, as shown below.

Timber framed shear walls

In the case of framed walls the overall stiffness is calculated taking into account the contribution of the following components:

- sheathing boards (k_s)
- sheet-fasteners slip (k_c)
- shear connections – angle brackets (k_a)
- hold-down or tie-down (k_h)

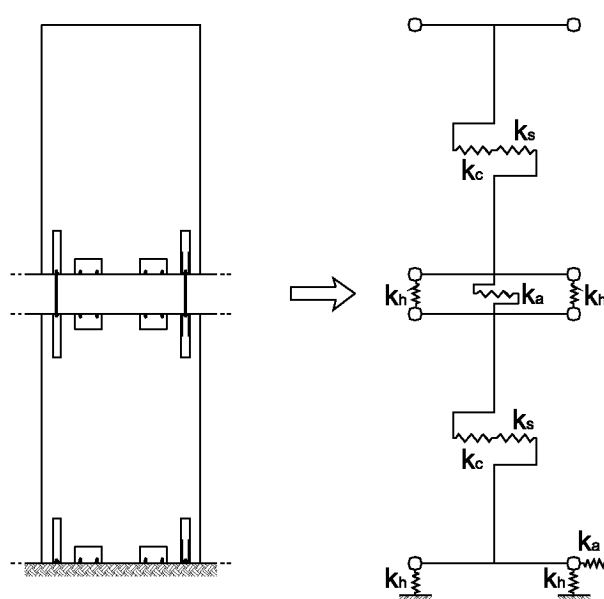


Figure: Mechanical model for framed walls overall stiffness

The following table indicates the positions of the walls and their equivalent shear stiffness.

Wall name	Type of wall	Element resistant to horizontal loads	Height [m]	Length [m]	Equivalent shear stiffness [kN/m]
Wall 1	Frame	Yes	3.6	1.3	619
Wall 10	Frame	Yes	3.6	8.61	5229
Wall 11	Frame	No	3.6	0.63	0
Wall 12	Frame	No	3.6	0.53	0
Wall 13	Frame	Yes	3.6	9.01	5629
Wall 14	Frame	No	3.6	0.55	0
Wall 15	Frame	Yes	3.6	3.16	1719
Wall 16	Frame	Yes	3.6	1.54	741
Wall 17	Frame	No	3.6	0.63	0
Wall 18	Frame	Yes	3.6	7.86	7524
Wall 19	Frame	No	3.6	0.62	0
Wall 2	Frame	Yes	3.6	2.6	1436
Wall 20	Frame	Yes	3.6	8.89	8698
Wall 21	Frame	Yes	3.6	5.82	5198
Wall 22	Frame	Yes	3.6	1.43	947
Wall 23	Frame	Yes	3.6	1.43	947
Wall 24	Frame	No	3.6	0.64	0
Wall 25	Frame	No	3.6	0.64	0
Wall 26	Frame	Yes	3.6	7	5457
Wall 27	Frame	Yes	3.6	1.49	1189
Wall 29	Frame	No	0.3	2.73	0
Wall 3	Frame	Yes	3.6	2.6	1436
Wall 31	Frame	No	0.3	2.1	0

Wall 33	Frame	No	0.3	2.1	0
Wall 35	Frame	No	0.3	2.1	0
Wall 37	Frame	No	0.3	4.8	0
Wall 4	Frame	Yes	3.6	1.3	619
Wall 40	Frame	Yes	0.79	1.3	2161
Wall 41	Frame	Yes	0.79	1.3	2161
Wall 42	Frame	No	0.79	4.9	0
Wall 43	Frame	Yes	0.79	2.6	4463
Wall 44	Frame	No	0.79	4.9	0
Wall 45	Frame	Yes	0.79	2.6	4463
Wall 46	Frame	No	0.79	4.9	0
Wall 47	Frame	Yes	1.68	9.01	10886
Wall 48	Frame	No	0.63	1.82	0
Wall 5	Frame	Yes	3.6	3.26	1766
Wall 50	Frame	No	2.27	2.45	0
Wall 51	Frame	No	2.05	0.55	0
Wall 52	Frame	Yes	1.95	3.16	3117
Wall 53	Frame	Yes	1.67	1.54	1624
Wall 54	Frame	No	0.85	3.03	0
Wall 55	Frame	No	1.15	4.72	0
Wall 56	Frame	Yes	1.68	1.49	1997
Wall 57	Frame	No	1.95	2.15	0
Wall 58	Frame	No	3.5	0.98	0
Wall 6	Frame	No	3.6	0.55	0
Wall 60	Frame	Yes	3.6	4.08	2369
Wall 64	Frame	Yes	3.6	0.99	452
Wall 65	Frame	Yes	3.6	1.3	619
Wall 66	Frame	Yes	3.6	1.3	619
Wall 67	Frame	Yes	3.6	1.31	624
Wall 68	Frame	No	3.5	1	0
Wall 69	Frame	Yes	3.6	6.39	3915
Wall 7	Frame	Yes	3.6	1.79	860
Wall 70	Frame	No	0.65	12.79	0
Wall 71	Frame	No	0.65	8.69	0
Wall 72	Frame	No	0.65	12.79	0
Wall 73	Frame	No	0.65	8.69	0
Wall 8	Frame	Yes	3.6	1.79	860
Wall 9	Frame	Yes	3.6	1.79	860

Types of structural elements and sign conventions

Linear elements

The linear elements are used to model beams and columns. They have a local reference system with respect to which stress/force components are shown. The sign convention adopted is shown in the figure below.

Force	Description	Unit of measure
N	Axial force	kN
M_{3-3}	Bending moment about local axis 3	kN m
V_2	Shear along local axis 2	kN
M_{2-2}	Bending moment about local axis 2	kN m
V_3	Shear along local axis 3	kN

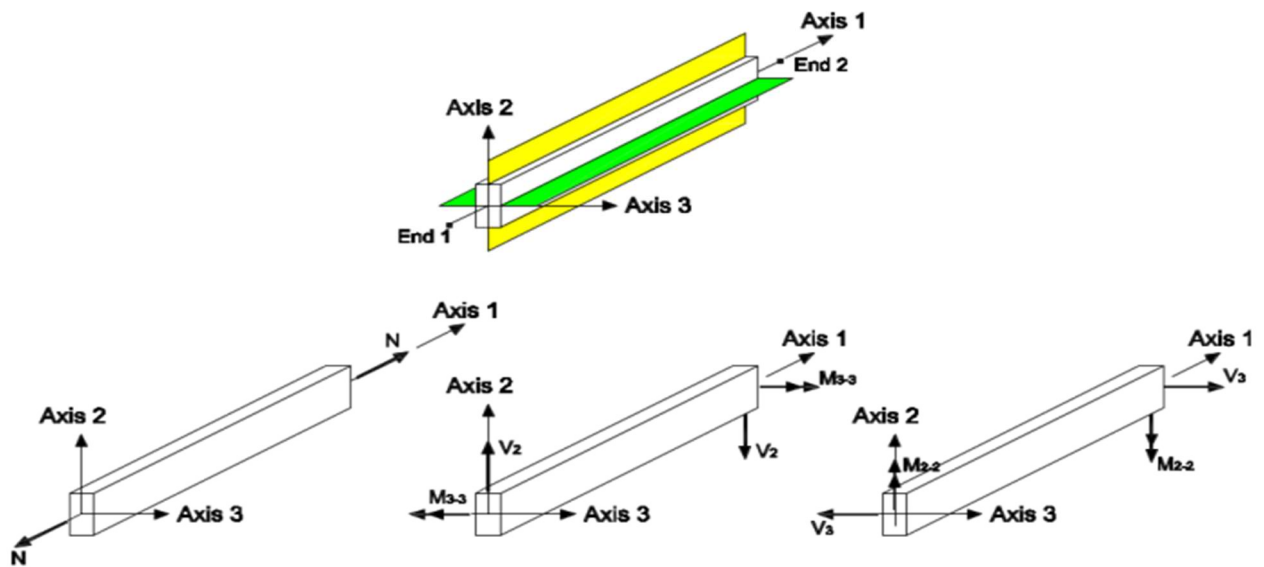


Figure: sign conventions for beams

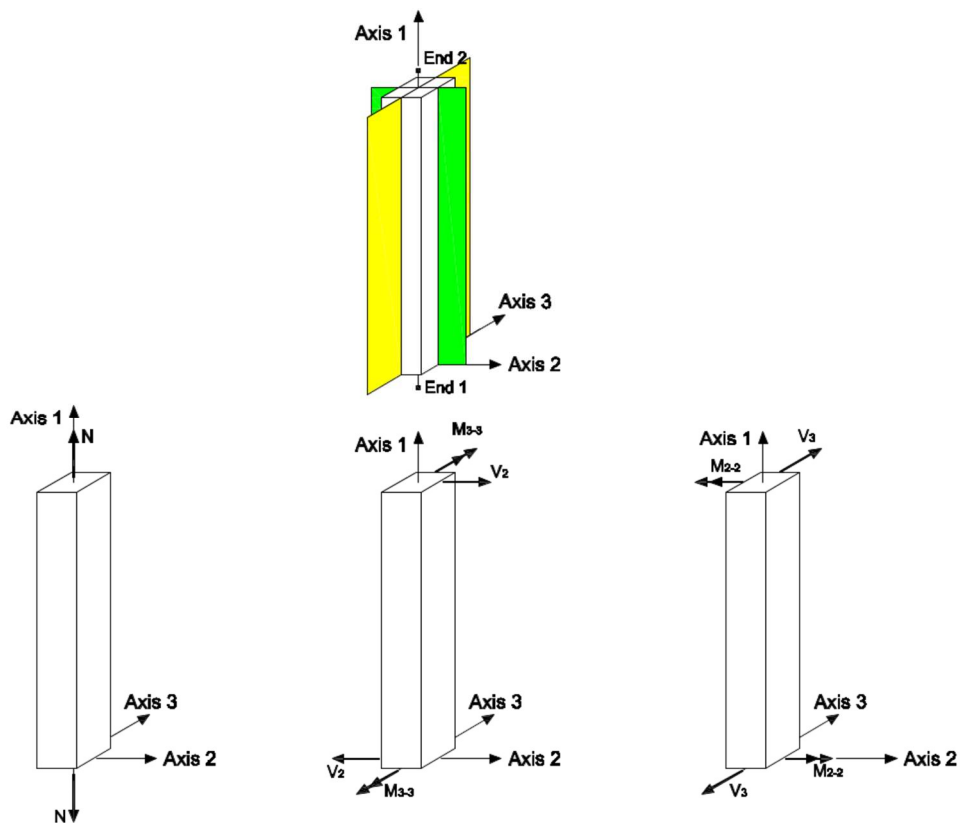


Figure: sign conventions for columns

Wall elements

The walls, regardless of type, have the following sign conventions.

	Actions per unit length	Description	Unit of measure
In-plane actions	n	Axial stress (per unit length)	kN/m
	m_{3-3}	Bending moment about local axis 3 (per unit length)	kNm/m
	v_2	Shear along local axis 2 (per unit length)	kN/m
Out-of-plane actions (plate)	m_{2-2}	Bending moment about local axis 2 (per unit length)	kNm/m
	v_3	Shear along local axis 3 (per unit length)	kN/m

	Actions	Description	Unit of measure
In-plane actions	N	Total axial force	kN
	M_{3-3}	Bending moment about local axis 3	kNm
	V_2	Shear along local axis 2	kN
Out-of-plane actions (plate)	M_{2-2}	Bending moment about local axis 2	kNm
	V_3	Shear along local axis 3	kN

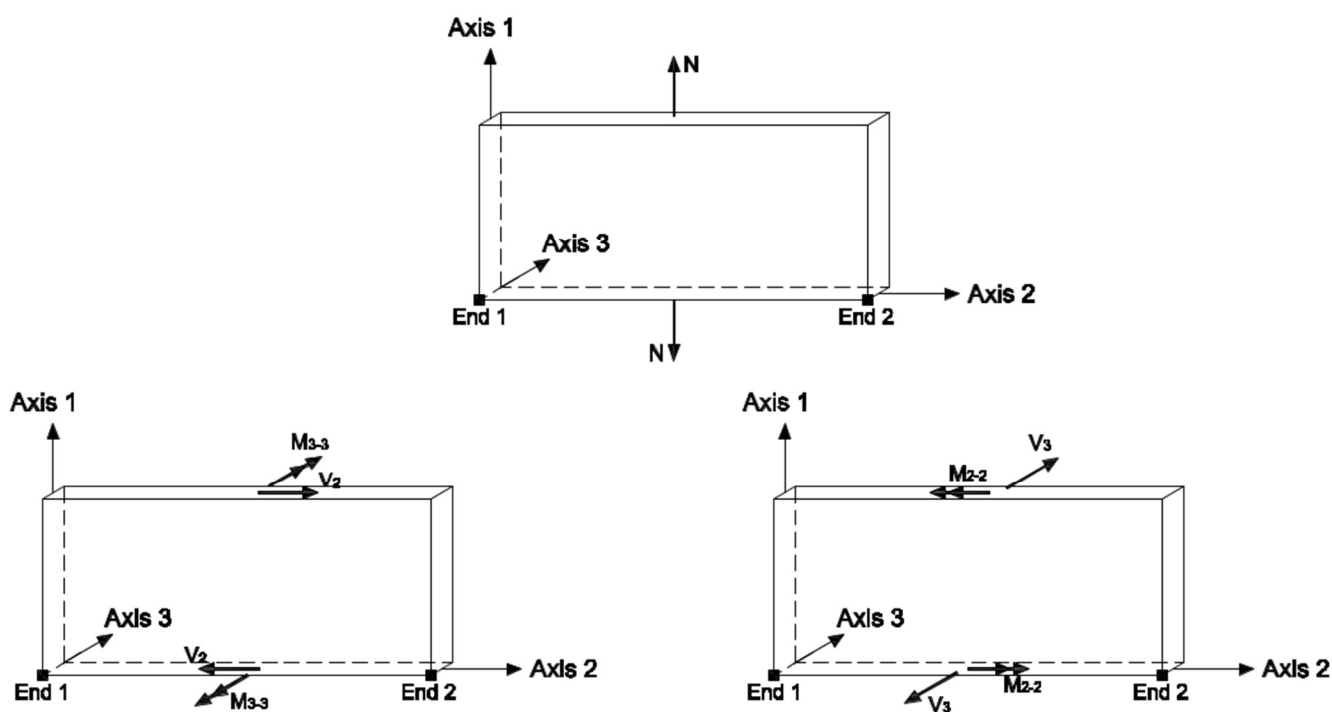


Figure: sign conventions for walls

Actions and design loads

Self-weight of structural materials

The weights of the structural materials are shown in the table below in kN/m^3

Description	Specific weight γ [kN/m^3]
GL 24h - EN 14080	5.00
C 24	5.00
GL 28h - EN 14080	5.00
OSB/3	8.00
C 24 BBS XL	5.00

Wind action

The wind action is evaluated in accordance with the European standard EN 1-1-4. The wind action is represented by a simplified set of forces whose effects is equivalent to the extreme effects of the turbulent wind.

Project data

Terrain category: Terrain category III

Basic wind velocity: 20 m/s

Mean wind

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity according to the following expression:

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b$$

where:

$c_r(z)$ is the roughness factor that depends on the terrain category and on the height z above the terrain of the relevant point;

$c_o(z)$ is the orography factor taken as 1;

v_b is the basic wind velocity.

According to section 4.3.2 of EN 1991-1-4, the roughness factor can be determined as follows:

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \quad \text{for } z_{min} \leq z \leq z_{max}$$

$$c_r(z) = c_r(z_{min}) \quad \text{for } z \leq z_{min}$$

where:

z_0 and z_{min} are defined according to the terrain category:

Terrain category		z_0	z_{min}
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m	1,0	10

k_r is the terrain factor calculated using:

$$k_r = 0,19 \cdot \left(\frac{z_0}{z_{0,II}} \right)^{0,07}$$

with: $z_{0,II} = 0,05 \text{ m}$

Wind turbulence

The turbulence intensity $I_v(z)$ at a height z is defined as the standard deviation of the turbulence divided by the mean wind velocity and it can be determined as:

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{1}{c_o(z) \cdot \ln\left(\frac{z}{z_0}\right)} \quad \text{for } z_{min} \leq z \leq z_{max}$$

$$I_v(z) = I_v(z_{min}) \quad \text{for } z \leq z_{min}$$

Peak velocity pressure

The peak velocity pressure $q_p(z)$ at a height z can be determined as:

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b$$

where:

$I_v(z)$ is the turbulence intensity;

ρ is the air density taken as 1.25 kg/m^3 ;

$v_m(z)$ is the mean wind velocity;

$c_e(z)$ is the exposure factor defined as $c_e(z) = \frac{q_p(z)}{q_b}$;

q_b is the basic wind pressure defined as $q_b = \frac{1}{2} \cdot \rho \cdot v_b^2$.

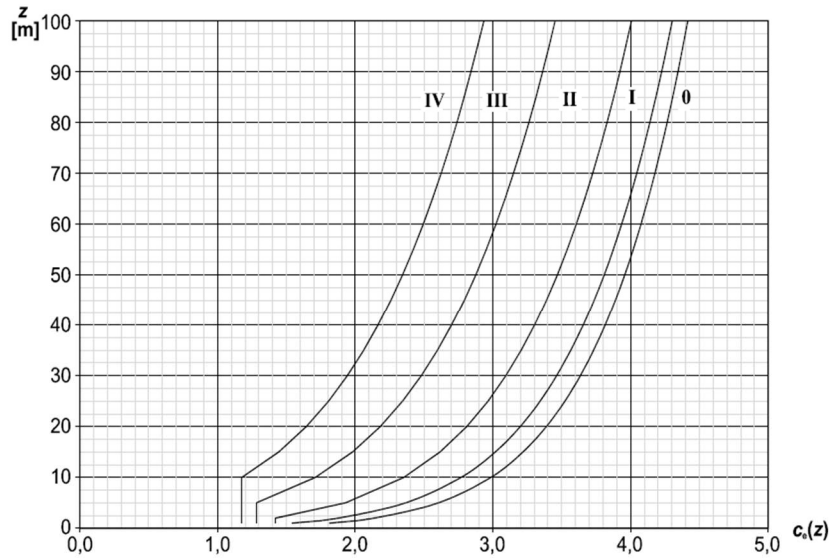


Figure 4.2 — Illustrations of the exposure factor $c_e(z)$ for $c_0=1,0$, $k_f=1,0$

The basic wind pressure results to be equal to:

$$q_b = 250 \text{ N/m}^2$$

Wind pressure on surfaces

The wind pressure acting on the external surfaces can be determined according to:

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

The wind pressure acting on the internal surfaces can be determined according to:

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

where:

c_{pe} is the pressure coefficient for the external pressure that can be calculated according to section 7.2 of EN 1991-1-4 or according to data supported by appropriate documentation or by experimental campaigns in wind tunnel;

c_{pi} is the pressure coefficient for the internal pressure.

The reference heights z_e to be considered in the calculation of the peak reference wind pressure are defined as follows:

- Windward surfaces of walls: the reference height changes along the structure height in accordance with point 7.2.2 on EN 1991-1-4;

- Leeward surfaces of walls: the reference height is equal to the maximum height of the building;
- Internal pressures: the reference height is equal to the maximum height of the building.

Wind forces

The wind forces for the whole structure or a structural component can be determined by calculating forces from surface pressures according to the following equations.

External forces:

$$F_{w,e} = c_s \cdot c_d \cdot \sum_{surfaces} w_e(z_e) \cdot A_{ref}$$

Internal forces:

$$F_{w,i} = \sum_{surfaces} w_i(z_i) \cdot A_{ref}$$

where:

- c_s is the size factor that takes into account the effect on wind actions from the non-simultaneous occurrence of peak wind pressures on the surfaces. It is taken equal to 1;
- c_d is the dynamic factor that takes into the effect of the vibrations of the structure due to turbulence. It is taken equal to 1.

Loads acting on the walls

The following table shows the loads acting on the walls.

Load name: Load ID

Position: Position of the wall: internal or external

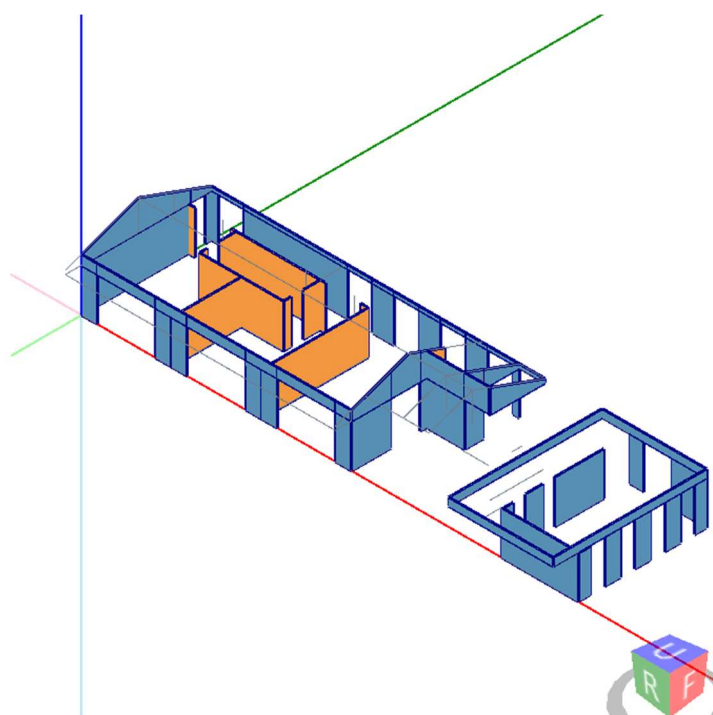
$g_{1,k}$: Permanent action: self weight

$g_{2,k}$: Permanent action

$q_{wind,k}$: Variable actions: wind load on windward, leeward and lateral surfaces

Wall name	Position	Load name	$g_{1,k}$ [kN/m ²]	$g_{2,k}$ [kN/m ²]	$q_{wind,k}$ downwind [kN/m ²]	$q_{wind,k}$ windward [kN/m ²]	$q_{wind,k}$ lateral [kN/m ²]
Wall 1	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 2	External	Obtezba zunanja stena	0.37	1	-0.2	0.36	-0.49
Wall 3	External	Obtezba zunanja stena	0.37	1	-0.2	0.36	-0.49
Wall 4	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 5	External	Obtezba zunanja stena	0.33	1	-0.17	0.35	-0.49
Wall 6	External	Obtezba zunanja stena	0.44	1	-0.2	0.36	-0.49
Wall 7	External	Obtezba zunanja stena	0.34	1	-0.2	0.36	-0.49
Wall 8	External	Obtezba zunanja stena	0.34	1	-0.2	0.36	-0.49
Wall 9	External	Obtezba zunanja stena	0.34	1	-0.2	0.36	-0.49
Wall 10	External	Obtezba zunanja stena	0.3	1	-0.2	0.36	-0.49
Wall 11	External	Obtezba zunanja stena	0.41	1	-0.2	0.36	-0.49
Wall 12	External	Obtezba zunanja stena	0.45	1	-0.17	0.35	-0.49
Wall 13	External	Obtezba zunanja stena	0.31	1	-0.17	0.35	-0.49
Wall 14	External	Obtezba zunanja stena	0.44	1	-0.17	0.35	-0.49
Wall 15	External	Obtezba zunanja stena	0.31	1	-0.2	0.36	-0.49
Wall 16	External	Obtezba zunanja stena	0.37	1	-0.17	0.35	-0.49
Wall 17	Internal	Obtezba notranja stena	0.44	0.8	0	0	0
Wall 18	Internal	Obtezba notranja stena	0.37	0.8	0	0	0
Wall 19	Internal	Obtezba notranja stena	0.44	0.8	0	0	0
Wall 20	Internal	Obtezba notranja stena	0.37	0.8	0	0	0
Wall 21	Internal	Obtezba notranja stena	0.38	0.8	0	0	0
Wall 22	Internal	Obtezba notranja stena	0.42	0.8	0	0	0
Wall 23	Internal	Obtezba notranja stena	0.42	0.8	0	0	0
Wall 24	Internal	Obtezba notranja stena	0.44	0.8	0	0	0
Wall 25	Internal	Obtezba notranja stena	0.44	0.8	0	0	0
Wall 26	Internal	Obtezba notranja stena	0.36	0.8	0	0	0
Wall 27	Internal	Obtezba notranja stena	0.42	0.8	0	0	0
Wall 29	External	Obtezba zunanja stena	0.8	1	-0.2	0.36	-0.49
Wall 31	External	Obtezba zunanja stena	0.8	1	-0.2	0.36	-0.49
Wall 33	External	Obtezba zunanja stena	0.8	1	-0.2	0.36	-0.49
Wall 35	External	Obtezba zunanja stena	0.8	1	-0.2	0.36	-0.49
Wall 37	External	Obtezba zunanja stena	0.79	1	-0.2	0.36	-0.49

Wall 40	External	Obtezba zunanja stena	0.48	1	-0.2	0.36	-0.49
Wall 41	External	Obtezba zunanja stena	0.48	1	-0.2	0.36	-0.49
Wall 42	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 43	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 44	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 45	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 46	External	Obtezba zunanja stena	0.46	1	-0.2	0.36	-0.49
Wall 47	External	Obtezba zunanja stena	0.35	1	-0.17	0.35	-0.49
Wall 48	External	Obtezba zunanja stena	0.53	1	-0.17	0.35	-0.49
Wall 50	External	Obtezba zunanja stena	0.35	1	-0.17	0.35	-0.49
Wall 51	External	Obtezba zunanja stena	0.47	1	-0.17	0.35	-0.49
Wall 52	External	Obtezba zunanja stena	0.35	1	-0.2	0.36	-0.49
Wall 53	External	Obtezba zunanja stena	0.41	1	-0.17	0.35	-0.49
Wall 54	External	Obtezba zunanja stena	0.46	1	-0.17	0.35	-0.49
Wall 55	External	Obtezba zunanja stena	0.4	1	-0.17	0.35	-0.49
Wall 56	Internal	Obtezba notranja stena	0.45	0.8	0	0	0
Wall 57	External	Obtezba zunanja stena	0.38	1	-0.2	0.36	-0.49
Wall 58	External	Obtezba zunanja stena	0.4	1	-0.17	0.35	-0.49
Wall 60	External	Obtezba zunanja stena	0.36	1	-0.17	0.35	-0.49
Wall 64	External	Obtezba zunanja stena	0.4	1	-0.17	0.35	-0.49
Wall 65	External	Obtezba zunanja stena	0.34	1	-0.17	0.35	-0.49
Wall 66	External	Obtezba zunanja stena	0.34	1	-0.17	0.35	-0.49
Wall 67	External	Obtezba zunanja stena	0.34	1	-0.17	0.35	-0.49
Wall 68	External	Obtezba zunanja stena	0.4	1	-0.17	0.35	-0.49
Wall 69	External	Obtezba zunanja stena	0.31	1	-0.2	0.36	-0.49
Wall 70	External	Obtezba zunanja stena	0.5	1	-0.17	0.35	-0.49
Wall 71	External	Obtezba zunanja stena	0.5	1	-0.2	0.36	-0.49
Wall 72	External	Obtezba zunanja stena	0.5	1	-0.17	0.35	-0.49
Wall 73	External	Obtezba zunanja stena	0.5	1	-0.2	0.36	-0.49



Legend	
■	Obtezba zunanja stena
■	Obtezba notranja stena

Loads acting on the floors

The following table shows the characteristic values of the loads acting on the decks.

Load name: Load ID

Position: Position of the floor: internal or external

Environment: Load category

α : Roof pitch angle

$g_{1,k}$: Permanent action: self weight

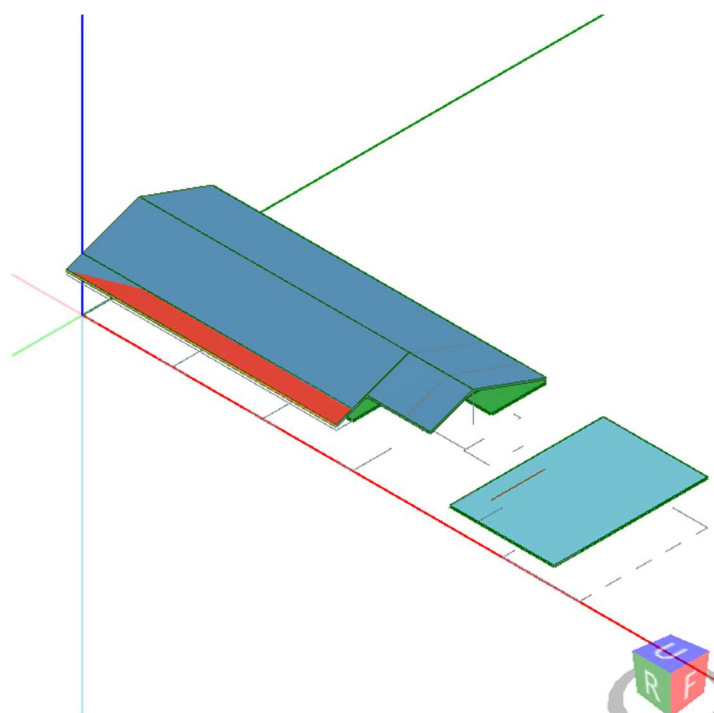
$g_{2,k}$: Permanent action

q_{k} : Variable actions

$q_{\text{snow},k}$: Variable actions: snow load

$q_{\text{wind},k}$: Variable actions: wind load

Floor name	Position	α [°]	Load name	Environment	$g_{1,k}$ [kN/m ²]	$g_{2,k}$ [kN/m ²]	q_{k} [kN/m ²]	$q_{\text{snow},k}$ [kN/m ²]	$q_{\text{wind},k}$ in depressure [kN/m ²]	$q_{\text{wind},k}$ in pressure [kN/m ²]
Floor 3	Roof	20	Obtezba streha	Imposed loads category H	0.1	0.75	0.4	2.22	-0.45	0.17
Floor 9	Internal floor	0	Obtezba strop	Imposed loads category A: floors	0.7	1	1	0	0	0
Floor 10	Roof	20	Obtezba streha	Imposed loads category H	0.1	0.75	0.4	2.22	-0.45	0.17
Floor 11	Internal floor	0	Obtezba strop	Imposed loads category A: floors	0.7	1	1	0	0	0
Floor 13	Roof	0	Obtezba ravna streha	Imposed loads category H	1.1	1.65	0.4	3.33	-0.45	0.17



Line loads

The following table shows the characteristic values of the line loads acting on the beams and the decks.

Load name: Load ID

Position: Position of the element: internal or external

Environment: Load category

$G_{1,k}$: Permanent action: self weight

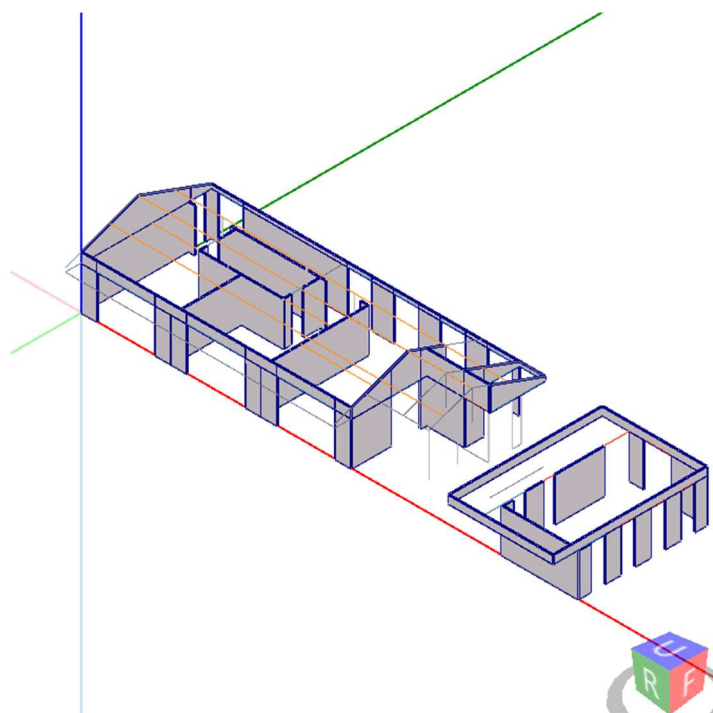
$G_{2,k}$: Permanent action

Q_{k} : Variable actions

$Q_{snow,k}$: Variable actions: snow load

$Q_{wind,k}$: Variable actions: wind load

Element name	Position	Load name	Environment	G _{1,k} [kN/m]	G _{2,k} [kN/m]	Q _k [kN/m]	Q _{snow,k} [kN/m]	Q _{wind,k} In depression [kN/m]	Q _{wind,k} In pressure [kN/m]
Beam 1	Internal load	g=1.05	-	0.44	1.05	-	0	0	0
Beam 2	Internal load	g=1.05	-	0.44	1.05	-	0	0	0
Beam 3	Internal load	g=1.05	-	0.44	1.05	-	0	0	0
Beam 4	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 5	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 6	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 7	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 8	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 9	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 10	Internal load	g=1.05	-	0.24	1.05	-	0	0	0
Beam 14	Internal load	g=1.05	-	0.06	1.05	-	0	0	0
Beam 15	Internal load	g=1.05	-	0.06	1.05	-	0	0	0
Beam 16	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 17	Internal load	Lastna	-	0.36	0	-	0	0	0
Beam 19	Internal load	Lastna	-	0.36	0	-	0	0	0
Beam 20	Internal load	Lastna	-	0.4	0	-	0	0	0
Beam 21	Internal load	Lastna	-	0.4	0	-	0	0	0
Beam 22	Internal load	Lastna	-	0.4	0	-	0	0	0
Beam 23	Internal load	Lastna	-	0.83	0	-	0	0	0
Beam 24	Internal load	Lastna	-	0.83	0	-	0	0	0
Beam 25	Internal load	g=1.05	-	0.4	1.05	-	0	0	0
Beam 26	Internal load	Lastna	-	0.24	0	-	0	0	0
Beam 27	Internal load	Lastna	-	0.24	0	-	0	0	0
Beam 28	Internal load	g=1.05	-	0.2	1.05	-	0	0	0
Beam 29	Internal load	Lastna	-	0.2	0	-	0	0	0
Beam 30	Internal load	Lastna	-	0.24	0	-	0	0	0
Beam 31	Internal load	g=1.3	-	0.1	1.3	-	0	0	0
Beam 32	Internal load	g=1.05	-	0.52	1.05	-	0	0	0
Beam 33	Internal load	g=1.3	-	0.2	1.3	-	0	0	0
Beam 34	Internal load	g=1.3	-	0.28	1.3	-	0	0	0
Beam 35	Internal load	g=1.05	-	0.52	1.05	-	0	0	0
Beam 36	Internal load	g=1.3	-	0.1	1.3	-	0	0	0
Beam 37	Internal load	g=1.3	-	0.1	1.3	-	0	0	0
Beam 38	Internal load	g=1.3	-	0.1	1.3	-	0	0	0
Floor 13 - Linear load 3	Internal load	Klimat	-	1.1	2.15	-	0	0	0



Legend	
■	g=1.05
■	Lastna
■	g=1.3

Seismic actions

The seismic actions are evaluated according to the Eurocode 8 and the National annex. The earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum.

The response spectra are calculated using the design ground acceleration a_g on type A ground: the acceleration is equal to a_{gR} , the value of the reference peak ground acceleration on type A ground, times the importance factor γ_I .

The response spectrum for the damage limit requirement is obtained multiplying the reduction of the design ground acceleration a_g by the reduction factor ν .

The parameters defining the design ground acceleration a_g on type A ground and the acceleration values for the ULS and DLS are reported below:

The reduction factor of the Damage limitation Limit State spectrum: 0.4

Importance factor: 1.2

Limit States	a_g [g]
ULS – Ultimate Limit State	0.18
DLS – Damage Limitation limit State	0.072

The value of the periods T_B , T_C , T_D and of the soil factor S defining the shape of the elastic response spectrum depend on the ground type and on the spectrum type. The parameters used are reported below:

Spectrum type: Type 1

Ground type: B

S soil factor: 1.2

T_B lower limit of the period of the constant spectral acceleration branch: 0.15 s

T_C upper limit of the period of the constant spectral acceleration branch: 0.5 s

T_D value defining the beginning of the constant displacement response range of the spectrum: 2 s

Ground elastic response spectrum

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions:

$$0 \leq T \leq T_B \quad S_e(T) = a_g \times S \times \left[1 + \frac{T}{T_B} \times (\eta \times 2,5 - 1) \right]$$

$$T_B \leq T \leq T_C \quad S_e(T) = a_g \times S \times \eta \times 2,5$$

$$T_C \leq T \leq T_D \quad S_e(T) = a_g \times S \times \eta \times 2,5 \left[\frac{T_C}{T} \right]$$

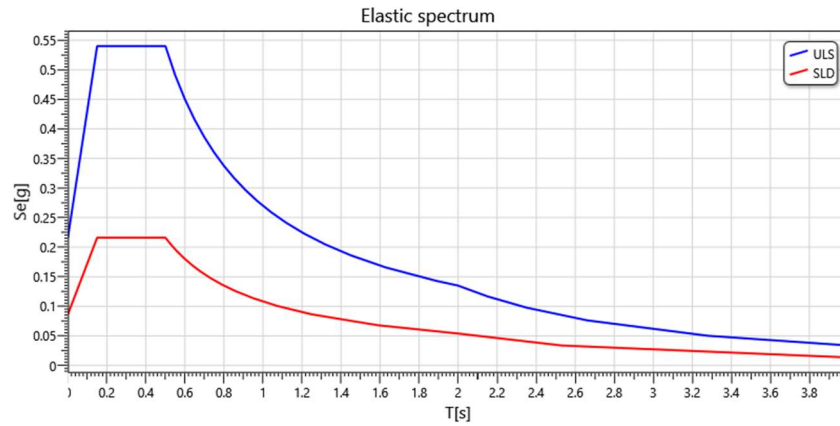
$$T \leq T_D \quad S_e(T) = a_g \times S \times \eta \times 2,5 \left[\frac{T_C T_D}{T^2} \right]$$

Horizontal elastic response spectra are reported below; they are calculated using the following values of the parameters η and ξ

$$\eta \quad 1$$

$$\xi \quad 5\%$$

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping.



Design spectrum for elastic analysis (NO-COLLAPSE)

To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behavior factor q . The design spectrum is defined by the following expressions:

$$0 \leq T \leq T_B \quad S_d(T) = a_g \times S \times \left[\frac{2}{3} + \frac{T}{T_B} \times \left(\frac{2,5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C \quad S_d(T) = a_g \times S \times \frac{2,5}{q}$$

$$T_C \leq T \leq T_D \quad S_d(T) = \begin{cases} a_g \times S \times \frac{2,5}{q} \times \left[\frac{T_C}{T} \right] \\ \geq \beta \times a_g \end{cases}$$

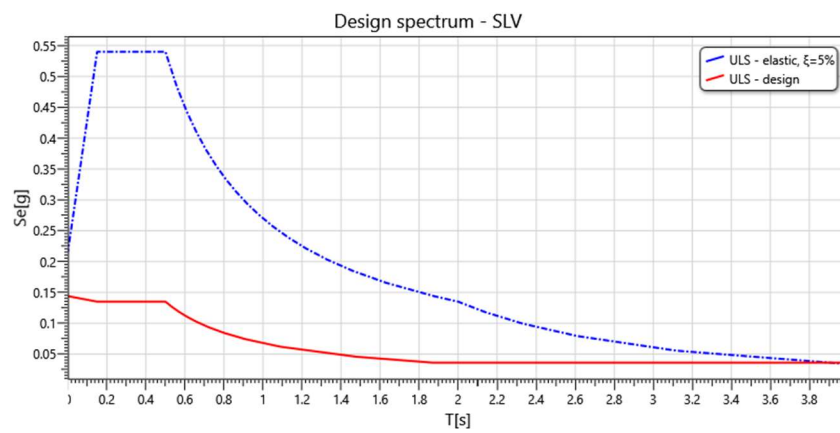
$$T \leq T_D \quad S_d(T) = \begin{cases} a_g \times S \times \frac{2,5}{q} \times \left[\frac{T_c T_D}{T^2} \right] \\ \geq \beta \times a_g \end{cases}$$

where:

β is the lower bound factor for the horizontal design spectrum equal to 0.2;

q is the behaviour factor: 4.

The horizontal elastic response spectra and the horizontal design spectrum (Ultimate Limit State) are shown below:



Sections of the structural elements

Framed walls

Frame geometric characteristics

t :	thickness of the frame
h_b :	thickness of top and sole plates
$b_{s,int}$:	width of the internal studs
$b_{s,ext}$:	width of the external studs
i_m :	average studs spacing

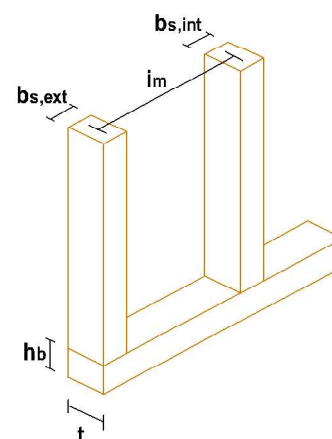


Figure: dimensions of the frame elements

Sheathing boards geometric characteristics

b_s :	sheathing boards width
$s_{c,b}$:	spacing of fasteners along the perimeter of every sheet
$s_{c,i}$:	spacing of internal fasteners

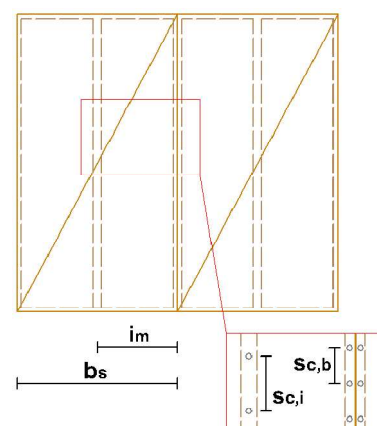


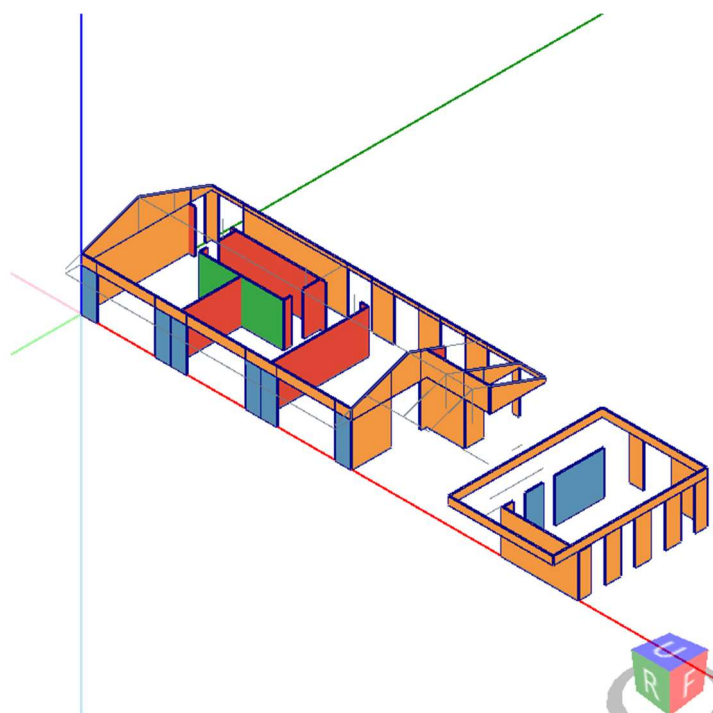
Figure: dimensions of the sheathing boards and fasteners spacing

In the first of the following tables are reported the characteristics of the frame of each wall, while in the second are reported the characteristics of the sheathing board.

Section name	# sides with sheathing board	Material	Frame thickness t [mm]	Thickness of top and sole plates h _b [mm]	Width of the internal studs b _{s,int} [mm]	Width of the external studs b _{s,ext} [mm]	Average studs spacing i _m [mm]
Okvir 200_ext=160	1	C 24	200	80	80	160	625
Okvir 200	1	C 24	200	80	80	80	625
Okvir 140	2	C 24	140	80	80	80	625
Okvir 100 e=417mm	2	C 24	100	80	80	80	417

Section name	Side	Material	Sheathing board thickness t _s [mm]	Sheathing boards width b _s [mm]	Frame-sheathing board fastener	Perimeter fasteners spacing S _{c,b} [mm]	Internal fasteners spacing i S _{c,i} [mm]
Okvir 200_ext=160	1	OSB/3	15	1250	Staples type Q 45 galv.	70	140
Okvir 200	1	OSB/3	15	1250	Staples type Q 45 galv.	70	140
Okvir 140	1	OSB/3	15	1250	Staples type Q 45 galv.	70	140
Okvir 140	2	OSB/3	15	1250	Staples type Q 45 galv.	70	140
Okvir 100 e=417mm	1	OSB/3	15	834	Staples type Q 45 galv.	70	140
Okvir 100 e=417mm	2	OSB/3	15	834	Staples type Q 45 galv.	70	140

Walls definition



Legend	
	Okvir 200_ext=160
	Okvir 200
	Okvir 140
	Okvir 100 e=417mm

Floors with timber joists

Elements geometric characteristics

h_b : Cross section height

b_b : Cross section width

i_b : Joists spacing

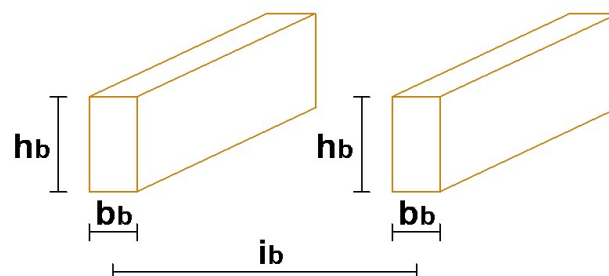


Figure: geometric characteristics of the floor

The following table sets out the details concerning the floor with joists.

Section name	Material	Cross section height h_b [mm]	Cross section width b_b [mm]	Joists spacing i_b [mm]
Spirovci 100x180 e=920	C 24	180	100	927

CLT floors

Floor geometric characteristic

h_b : CLT panel thickness

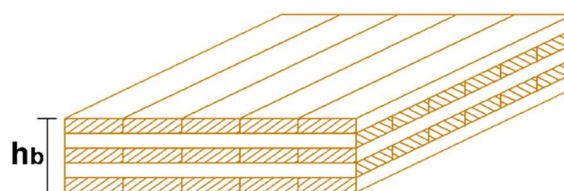
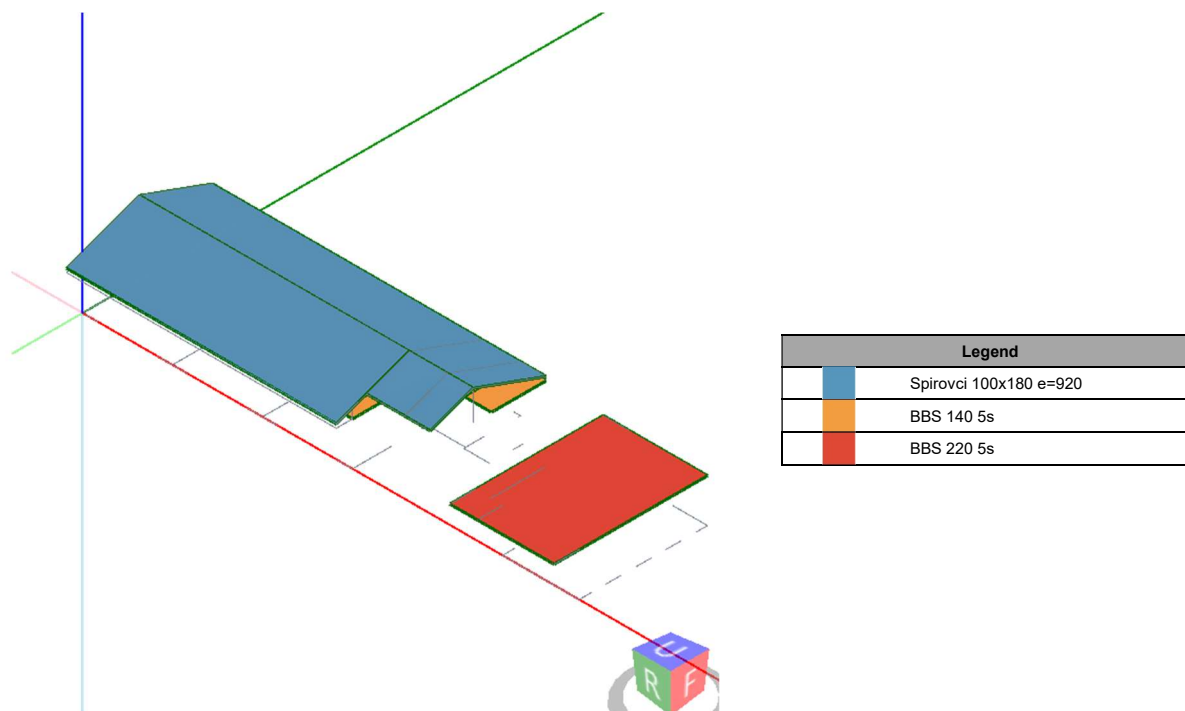


Figure: CLT floor geometric characteristics

The following table sets out the details concerning the CLT floors.

Section name	Manufacturer	CLT panel name	Material	Number of layers	Thickness h_b [mm]	Layers	External layers orientation
BBS 140 5s	Binderholz	BBS XL 140 5s	C 24 BBS XL	5	140	40 - 20 - 20 - 20 - 40	Parallel to the calculation direction
BBS 220 5s	Binderholz	BBS XL 220 5s	C 24 BBS XL	5	220	60 - 30 - 40 - 30 - 60	Parallel to the calculation direction

Floors definition



Cross section of timber linear elements

Rectangular sections

The following table sets out the details concerning every linear element with a rectangular cross section.

Section name	Material	Width b [mm]	Height h [mm]	b ₁ [mm]	h ₁ [mm]	b ₂ [mm]	h ₂ [mm]	Area A [mm ²]	J _{y-y} [mm ⁴]	J _{z-z} [mm ⁴]
200x200 GL24h	GL 24h - EN 14080	200	200	-	-	-	-	40000	1.33E8	1.33E8
140x240	C 24	140	240	-	-	-	-	33600	1.61E8	5.49E7
160x200	C 24	160	200	-	-	-	-	32000	1.07E8	6.83E7
200x440 GL24h	GL 24h - EN 14080	200	440	-	-	-	-	88000	1.42E9	2.93E8
200x240 GL24h	GL 24h - EN 14080	200	240	-	-	-	-	48000	2.30E8	1.60E8
140x80	C 24	140	80	-	-	-	-	11200	5.97E6	1.83E7
260x280 GL28h	GL 28h - EN 14080	260	280	-	-	-	-	72800	4.76E8	4.10E8
200x400 GL28h	GL 28h - EN 14080	200	400	-	-	-	-	80000	1.07E9	2.67E8
200x100	C 24	200	100	-	-	-	-	20000	1.67E7	6.67E7
200x520 GL24h	GL 24h - EN 14080	200	520	-	-	-	-	104000	2.34E9	3.47E8
200x280 GL24h	GL 24h - EN 14080	200	280	-	-	-	-	56000	3.66E8	1.87E8

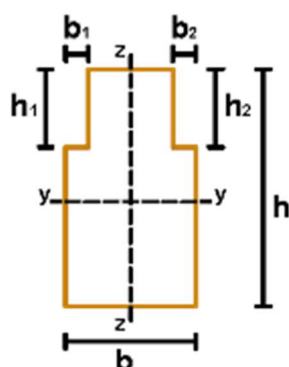


Figure: Geometric size of every timber rectangular cross section

Steel cross-sections

The following table provides the main proprieties of the steel cross-sections used in the project:

Section name	Material	Profile	Height h [mm]	Width b [mm]	t_f [mm]	t_w [mm]	A $\times 10^2$ [mm ²]	A_v $\times 10^2$ [mm ²]	$W_{pl,y}$ $\times 10^3$ [mm ³]	$W_{el,y}$ $\times 10^3$ [mm ³]	J_{y-y} $\times 10^4$ [mm ⁴]	J_{z-z} $\times 10^4$ [mm ⁴]
150x5	S235 - EN 10025 -2	SHS CF 150 5.0	150	150	5	5	28.4	14.2	153	131	982	982
150x100x5	S235 - EN 10025 -2	RHS CF 150x100 5.0	150	100	5	5	23.4	14.04	117	95.9	719	384
HEB 240	S235 - EN 10025 -2	HE 240 B	240	240	17	10	106	33.23	1053	938.3	11260	3923

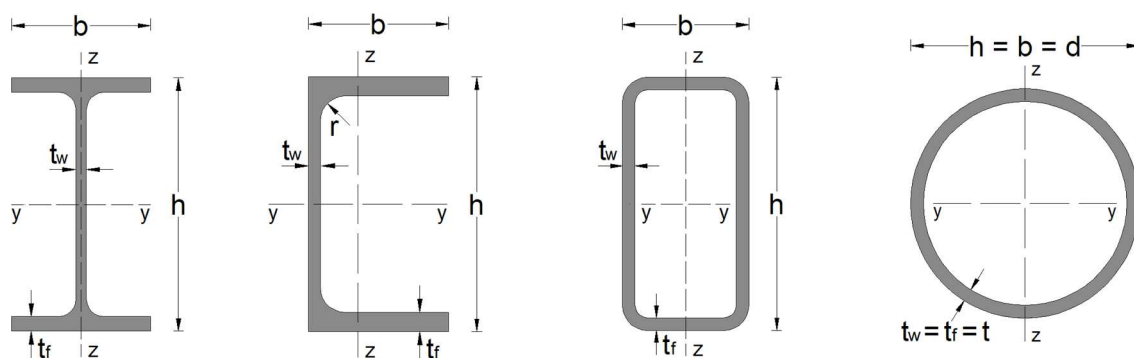
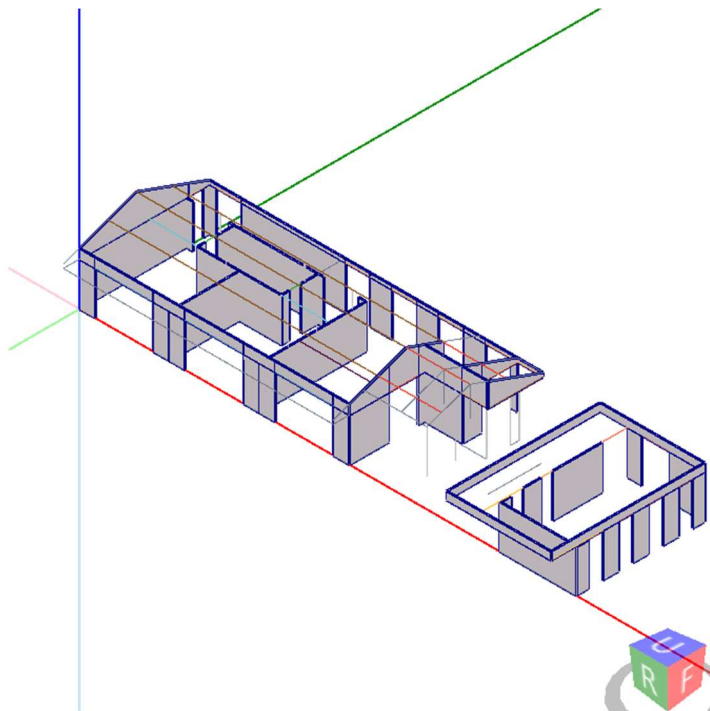
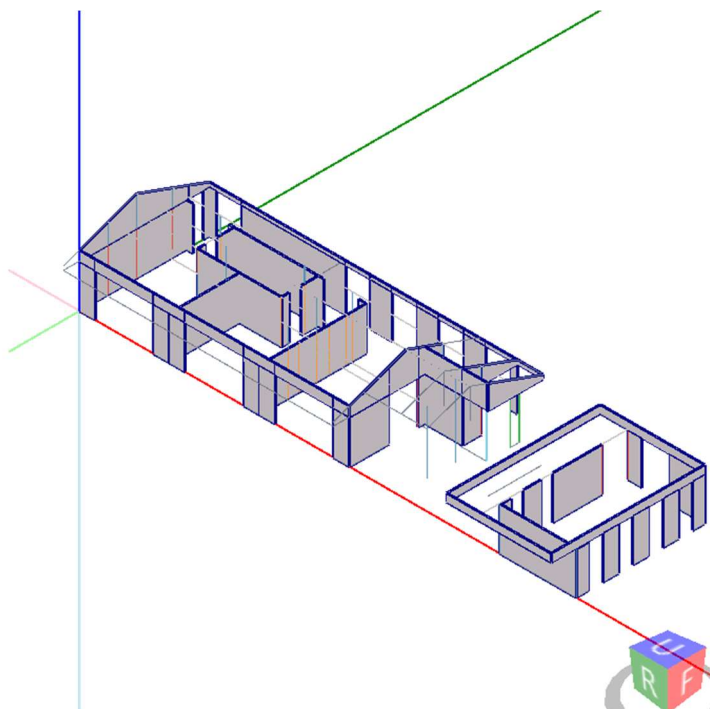


Figure: Geometric size of the steel cross-sections

Beams and columns definition



Legend	
	200x440 GL24h
	200x200 GL24h
	200x240 GL24h
	140x80
	260x280 GL28h
	200x400 GL28h
	HEB 240
	200x100
	200x520 GL24h
	200x280 GL24h



Legend	
	200x200 GL24h
	140x240
	160x200
	150x5
	150x100x5

Connections

Hold down

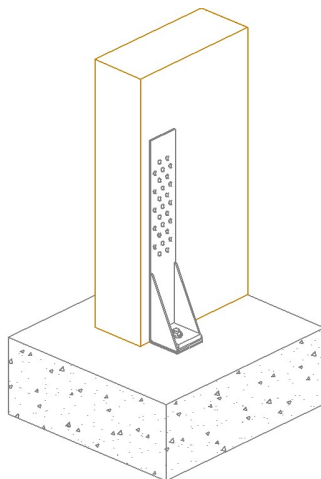


Figure: graphical representation of a hold down in a base connection (timber wall – foundation connection)

Connection name	Connection position	Manufacturer	Description	Fasteners	Threaded rods / Mechanical anchors	Chemical anchor	N° connection elements at each wall end
WKR 530	Ground connection	Rotho Blaas	WKR 53035	16 x Anker nail - LBA 4,0 x 60	1 x Threaded rod INA - 5.8 - M12 x 195	Hybrid chemical anchor ETA-20/1285	1
2x WKR 530	Ground connection	Rotho Blaas	WKR 53035	16 x Anker nail - LBA 4,0 x 60	1 x Threaded rod INA - 5.8 - M12 x 195	Hybrid chemical anchor ETA-20/1285	2

Timber to concrete angle bracket

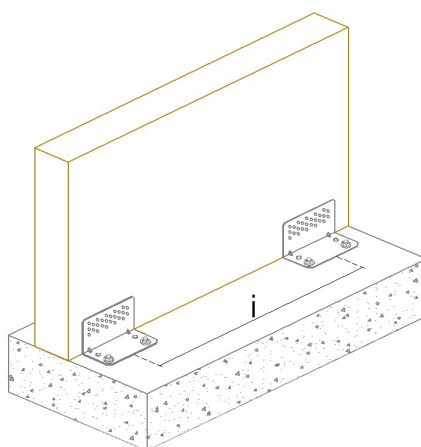


Figure: graphical representation of the shear connection with angle brackets

Connection name	Connection position	Manufacturer	Description	Fasteners	Threaded rods / Mechanical anchors	Chemical anchor	Number of sides	Spacing i [mm]
Sidni vijak M16	Ground connection	User defined	Angle bracket TC1	1 x Anker nail - 4,0 x 40	1 x Threaded rod - angle bracket ATC1	Chemical anchor - angle bracket ATC1	1	1250

Double hold down

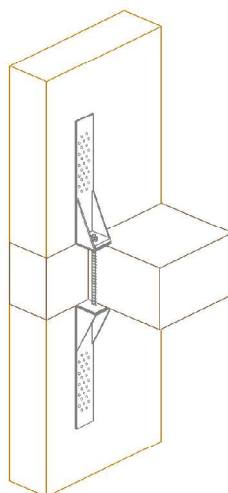


Figure: graphical representation of the hold down connection at the upper floors

Connection name	Connection position	Manufacturer	Description	Fasteners	Bolts / Threaded rods	N° connection elements at each wall end
WKR 285 - WKR 285	Upper level	Rotho Blaas	WKR 28535	16 x Anker nail - LBA 4,0 x 60	1 x Threaded rod MGS - 4.8 - M12	1

Timber to timber tensile plate

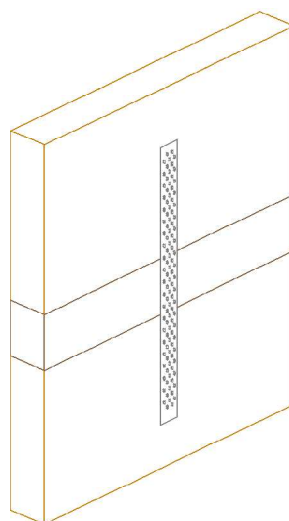


Figure: graphical representation of a punched strap

Connection name	Connection position	Manufacturer	Description	Width [mm]	Length [mm]	Thickness [mm]	Steel grade	Fasteners	N° connection elements at each wall end
Kovinski trak 60mm	Upper level	Rotho Blaas	Perforated strap 60 mm sp. 1,5 mm	60	50000	1.5	S350	18 x Anker nail - LBA 4,0 X 60	1

Timber to timber angle bracket

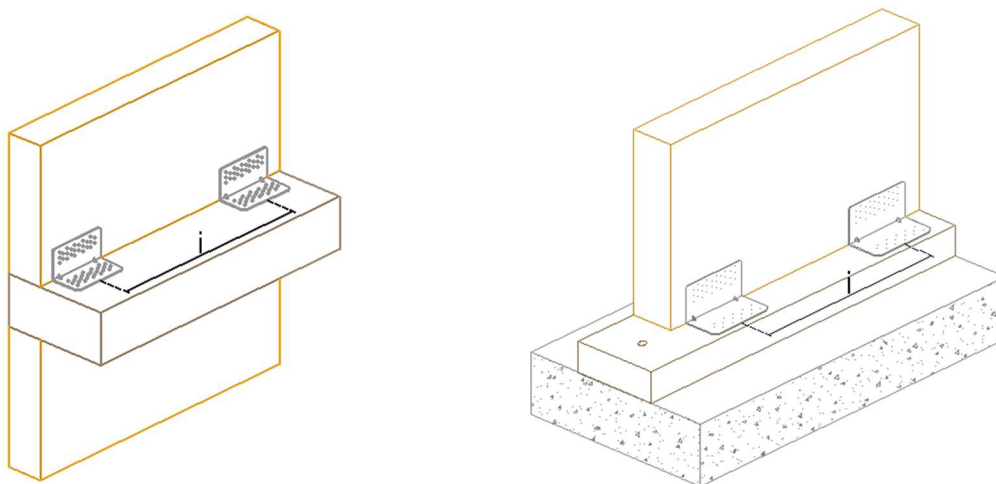
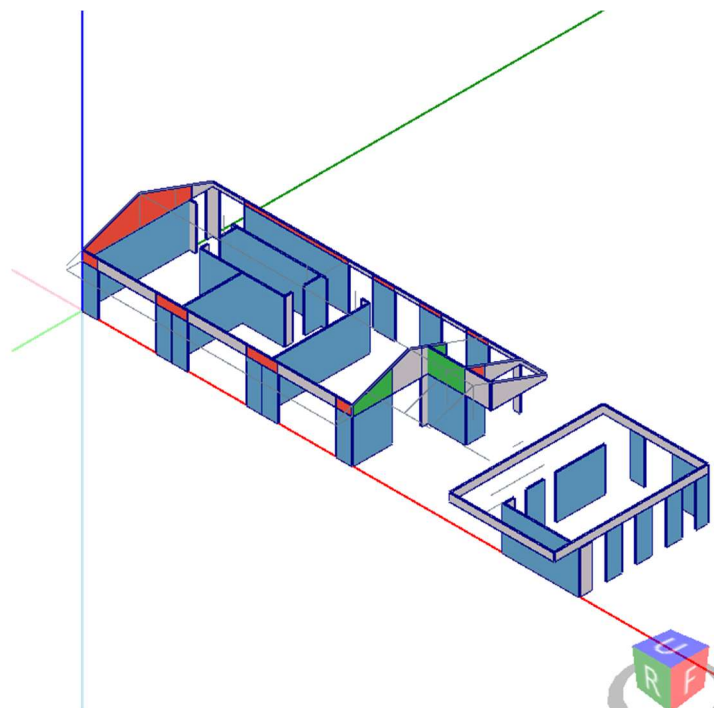


Figure: graphical representations of the timber to timber shear connection with angle brackets

Connection name	Connection position	Manufacturer	Description	Number of fasteners on the vertical plate	Number of fasteners on the horizontal plate	Number of sides	Spacing i [mm]
Vijaki 8mm	Upper level	User defined	Angle bracket TT1	1 x Anker nail - 4,0 x 40	1 x Anker nail - 4,0 x 40	1	300

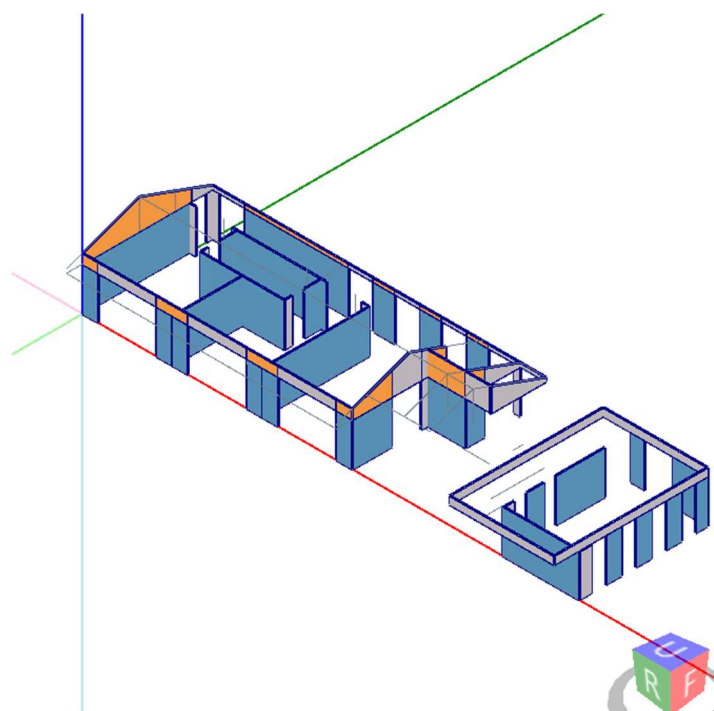
Connections definition

Tensile



Legend	
	WKR 530
	2x WKR 530
	Kovinski trak 60mm
	WKR 285 - WKR 285

Shear



Legend	
	Sidni vijak M16
	Vijaki 8mm

Combinations of actions

For each critical load case, the design values of the effects of actions shall be determined by combining the values of actions that are considered to occur simultaneously.

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

$$= \gamma_{G1} \cdot G_1 + \gamma_{G2} \cdot G_2 + \gamma_Q \cdot Q_{k1} + \gamma_{Q2} \cdot \psi_{02} \cdot Q_{k2} + \gamma_{Q3} \cdot \psi_{03} \cdot Q_{k3} + \dots$$

Combinations of actions for seismic design situations:

$$\sum_{j \geq 1} G_{k,j} + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} =$$

$$= G_1 + G_2 + A_{Ed} + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \dots$$

being:

- G₁ permanent actions: self weight;
- G₂ permanent actions;
- Q₁ characteristic value of the main variable action;
- Q_{ki} characteristic value of the i-th variable action;
- γ_{G1} is the partial factor for the self-weight action;
- γ_{G2} is the partial factor for the permanent actions action.

When permanent actions, as well as a portion of those, are fully defined in design process, it is possible to adopt the same partial factor employed for self-weight actions.

The following are the values of the combination coefficients used.

Action name	Description	Duration	ψ_0	ψ_1	ψ_2
Q cat A	Category A: domestic, residential areas	Medium-term	0.7	0.5	0.3
Q cat B	Category B: office areas	Medium-term	0.7	0.5	0.3
Q cat C	Category C: congregation areas	Medium-term	0.7	0.7	0.6
Q cat D	Category D: shopping areas	Medium-term	0.7	0.7	0.6
Q cat E	Category E: storage areas	Long-term	1	0.9	0.8
Q cat F	Category F: traffic area, vehicle weight ≤ 30 kN	Long-term	0.7	0.7	0.6
Q cat G	Category G: traffic area, vehicle weight ≤ 160 kN	Long-term	0.7	0.5	0.3
Q cat H	Category H: roofs	Medium-term	0	0	0

Q cat I-A	Category I-A: practicable roofs of category A	Medium-term	0.7	0.5	0.3
Q cat I-B	Category I-B: practicable roofs of category B	Medium-term	0.7	0.5	0.3
Q cat I-C	Category I-C: practicable roofs of category C	Medium-term	0.7	0.7	0.6
Q cat I-D	Category I-D: practicable roofs of category D	Medium-term	0.7	0.7	0.6
Q cat I-E	Category I-E: practicable roofs of category E	Long-term	1	0.9	0.8
Ortho wind	Wind load	Instantaneous	0.6	0.2	0
Snow	Snow load (altitude <= 1000 mamsl)	Short-term	0.5	0.2	0
Snow	Snow load (altitude > 1000 mamsl)	Medium-term	0.7	0.5	0.2

Combinations of actions used

Vertical ULS loads combinations

The following table shows the ULS load combinations relevant for verifications in conditions of vertical load. The coefficient values listed correspond to the product of the partial safety factor γ_j and the combination factors ψ_{0j} .

The action of the wind is schematized with a uniform load orthogonal to each external wall.

Name	Duration	G1	G2	Q cat A	Q cat H	Snow	Ortho wind	Wind X	Wind Y	Dynamic SLV X	Dynamic SLV Y	Dynamic SLD X	Dynamic SLD Y
ULS 1	Permanent	1	1	0	0	0	0	0	0	0	0	0	0
ULS 2	Medium-term	1	1	1.5	0	0	0	0	0	0	0	0	0
ULS 3	Short-term	1	1	1.5	0	0.75	0	0	0	0	0	0	0
ULS 4	Instantaneous	1	1	1.5	0	0	0.9	0	0	0	0	0	0
ULS 5	Instantaneous	1	1	1.5	0	0.75	0.9	0	0	0	0	0	0
ULS 6	Medium-term	1	1	0	1.5	0	0	0	0	0	0	0	0
ULS 7	Medium-term	1	1	1.05	1.5	0	0	0	0	0	0	0	0
ULS 8	Short-term	1	1	0	1.5	0.75	0	0	0	0	0	0	0
ULS 9	Short-term	1	1	1.05	1.5	0.75	0	0	0	0	0	0	0
ULS 10	Instantaneous	1	1	0	1.5	0	0.9	0	0	0	0	0	0
ULS 11	Instantaneous	1	1	1.05	1.5	0	0.9	0	0	0	0	0	0
ULS 12	Instantaneous	1	1	0	1.5	0.75	0.9	0	0	0	0	0	0
ULS 13	Instantaneous	1	1	1.05	1.5	0.75	0.9	0	0	0	0	0	0
ULS 14	Short-term	1	1	0	0	1.5	0	0	0	0	0	0	0
ULS 15	Short-term	1	1	1.05	0	1.5	0	0	0	0	0	0	0
ULS 16	Instantaneous	1	1	0	0	1.5	0.9	0	0	0	0	0	0
ULS 17	Instantaneous	1	1	1.05	0	1.5	0.9	0	0	0	0	0	0
ULS 18	Instantaneous	1	1	0	0	0	1.5	0	0	0	0	0	0
ULS 19	Instantaneous	1	1	1.05	0	0	1.5	0	0	0	0	0	0
ULS 20	Instantaneous	1	1	0	0	0.75	1.5	0	0	0	0	0	0
ULS 21	Instantaneous	1	1	1.05	0	0.75	1.5	0	0	0	0	0	0
ULS 22	Permanent	1	1.35	0	0	0	0	0	0	0	0	0	0
ULS 23	Medium-term	1	1.35	1.5	0	0	0	0	0	0	0	0	0
ULS 24	Short-term	1	1.35	1.5	0	0.75	0	0	0	0	0	0	0
ULS 25	Instantaneous	1	1.35	1.5	0	0	0.9	0	0	0	0	0	0
ULS 26	Instantaneous	1	1.35	1.5	0	0.75	0.9	0	0	0	0	0	0
ULS 27	Medium-term	1	1.35	0	1.5	0	0	0	0	0	0	0	0
ULS 28	Medium-term	1	1.35	1.05	1.5	0	0	0	0	0	0	0	0
ULS 29	Short-term	1	1.35	0	1.5	0.75	0	0	0	0	0	0	0
ULS 30	Short-term	1	1.35	1.05	1.5	0.75	0	0	0	0	0	0	0
ULS 31	Instantaneous	1	1.35	0	1.5	0	0.9	0	0	0	0	0	0
ULS 32	Instantaneous	1	1.35	1.05	1.5	0	0.9	0	0	0	0	0	0
ULS 33	Instantaneous	1	1.35	0	1.5	0.75	0.9	0	0	0	0	0	0
ULS 34	Instantaneous	1	1.35	1.05	1.5	0.75	0.9	0	0	0	0	0	0
ULS 35	Short-term	1	1.35	0	0	1.5	0	0	0	0	0	0	0
ULS 36	Short-term	1	1.35	1.05	0	1.5	0	0	0	0	0	0	0
ULS 37	Instantaneous	1	1.35	0	0	1.5	0.9	0	0	0	0	0	0
ULS 38	Instantaneous	1	1.35	1.05	0	1.5	0.9	0	0	0	0	0	0
ULS 39	Instantaneous	1	1.35	0	0	0	1.5	0	0	0	0	0	0
ULS 40	Instantaneous	1	1.35	1.05	0	0	1.5	0	0	0	0	0	0
ULS 41	Instantaneous	1	1.35	0	0	0.75	1.5	0	0	0	0	0	0
ULS 42	Instantaneous	1	1.35	1.05	0	0.75	1.5	0	0	0	0	0	0
ULS 43	Permanent	1.35	1	0	0	0	0	0	0	0	0	0	0
ULS 44	Medium-term	1.35	1	1.5	0	0	0	0	0	0	0	0	0
ULS 45	Short-term	1.35	1	1.5	0	0.75	0	0	0	0	0	0	0
ULS 46	Instantaneous	1.35	1	1.5	0	0	0.9	0	0	0	0	0	0
ULS 47	Instantaneous	1.35	1	1.5	0	0.75	0.9	0	0	0	0	0	0
ULS 48	Medium-term	1.35	1	0	1.5	0	0	0	0	0	0	0	0
ULS 49	Medium-term	1.35	1	1.05	1.5	0	0	0	0	0	0	0	0
ULS 50	Short-term	1.35	1	0	1.5	0.75	0	0	0	0	0	0	0
ULS 51	Short-term	1.35	1	1.05	1.5	0.75	0	0	0	0	0	0	0

ULS 52	Instantaneous	1.35	1	0	1.5	0	0.9	0	0	0	0	0	0
ULS 53	Instantaneous	1.35	1	1.05	1.5	0	0.9	0	0	0	0	0	0
ULS 54	Instantaneous	1.35	1	0	1.5	0.75	0.9	0	0	0	0	0	0
ULS 55	Instantaneous	1.35	1	1.05	1.5	0.75	0.9	0	0	0	0	0	0
ULS 56	Short-term	1.35	1	0	0	1.5	0	0	0	0	0	0	0
ULS 57	Short-term	1.35	1	1.05	0	1.5	0	0	0	0	0	0	0
ULS 58	Instantaneous	1.35	1	0	0	1.5	0.9	0	0	0	0	0	0
ULS 59	Instantaneous	1.35	1	1.05	0	1.5	0.9	0	0	0	0	0	0
ULS 60	Instantaneous	1.35	1	0	0	0	1.5	0	0	0	0	0	0
ULS 61	Instantaneous	1.35	1	1.05	0	0	1.5	0	0	0	0	0	0
ULS 62	Instantaneous	1.35	1	0	0	0.75	1.5	0	0	0	0	0	0
ULS 63	Instantaneous	1.35	1	1.05	0	0.75	1.5	0	0	0	0	0	0
ULS 64	Permanent	1.35	1.35	0	0	0	0	0	0	0	0	0	0
ULS 65	Medium-term	1.35	1.35	1.5	0	0	0	0	0	0	0	0	0
ULS 66	Short-term	1.35	1.35	1.5	0	0.75	0	0	0	0	0	0	0
ULS 67	Instantaneous	1.35	1.35	1.5	0	0	0.9	0	0	0	0	0	0
ULS 68	Instantaneous	1.35	1.35	1.5	0	0.75	0.9	0	0	0	0	0	0
ULS 69	Medium-term	1.35	1.35	0	1.5	0	0	0	0	0	0	0	0
ULS 70	Medium-term	1.35	1.35	1.05	1.5	0	0	0	0	0	0	0	0
ULS 71	Short-term	1.35	1.35	0	1.5	0.75	0	0	0	0	0	0	0
ULS 72	Short-term	1.35	1.35	1.05	1.5	0.75	0	0	0	0	0	0	0
ULS 73	Instantaneous	1.35	1.35	0	1.5	0	0.9	0	0	0	0	0	0
ULS 74	Instantaneous	1.35	1.35	1.05	1.5	0	0.9	0	0	0	0	0	0
ULS 75	Instantaneous	1.35	1.35	0	1.5	0.75	0.9	0	0	0	0	0	0
ULS 76	Instantaneous	1.35	1.35	1.05	1.5	0.75	0.9	0	0	0	0	0	0
ULS 77	Short-term	1.35	1.35	0	0	1.5	0	0	0	0	0	0	0
ULS 78	Short-term	1.35	1.35	1.05	0	1.5	0	0	0	0	0	0	0
ULS 79	Instantaneous	1.35	1.35	0	0	1.5	0.9	0	0	0	0	0	0
ULS 80	Instantaneous	1.35	1.35	1.05	0	1.5	0.9	0	0	0	0	0	0
ULS 81	Instantaneous	1.35	1.35	0	0	0	1.5	0	0	0	0	0	0
ULS 82	Instantaneous	1.35	1.35	1.05	0	0	1.5	0	0	0	0	0	0
ULS 83	Instantaneous	1.35	1.35	0	0	0.75	1.5	0	0	0	0	0	0
ULS 84	Instantaneous	1.35	1.35	1.05	0	0.75	1.5	0	0	0	0	0	0

Horizontal ULS loads combinations

The following table shows the ULS load combinations relevant for verifications in conditions of vertical load. The coefficient values listed correspond to the product of the partial safety factor γ_j and the combination factors ψ_{0j} .

The action of the wind is schematized with a uniform load orthogonal to each external wall and it acts separately in the directions x, -x, y, -y.

Name	Duration	G1	G2	Q cat A	Q cat H	Snow	Ortho wind	Wind X	Wind Y	Dynamic SLV X	Dynamic SLV Y	Dynamic SLD X	Dynamic SLD Y
Horizontal ULS 1	Instantaneous	1	1	0	0	0	0	1.5	0	0	0	0	0
Horizontal ULS 2	Instantaneous	1	1	0	0	0	0	0	1.5	0	0	0	0
Horizontal ULS 3	Instantaneous	1	1	0	0	0	0	-1.5	0	0	0	0	0
Horizontal ULS 4	Instantaneous	1	1	0	0	0	0	0	-1.5	0	0	0	0
Horizontal ULS 5	Instantaneous	1.35	1.35	1.05	0	0.75	0	1.5	0	0	0	0	0
Horizontal ULS 6	Instantaneous	1.35	1.35	1.05	0	0.75	0	0	1.5	0	0	0	0
Horizontal ULS 7	Instantaneous	1.35	1.35	1.05	0	0.75	0	-1.5	0	0	0	0	0
Horizontal ULS 8	Instantaneous	1.35	1.35	1.05	0	0.75	0	0	-1.5	0	0	0	0

Combination of actions for rare SLS

Name	Duration	G1	G2	Q cat A	Q cat H	Snow	Ortho wind	Wind X	Wind Y	Dynamic SLV X	Dynamic SLV Y	Dynamic SLD X	Dynamic SLD Y
SLS characteristic 1	Permanent	1	1	0	0	0	0	0	0	0	0	0	0
SLS characteristic 2	Medium-term	1	1	1	0	0	0	0	0	0	0	0	0
SLS characteristic 3	Short-term	1	1	1	0	0.5	0	0	0	0	0	0	0
SLS characteristic 4	Instantaneous	1	1	1	0	0	0.6	0	0	0	0	0	0
SLS characteristic 5	Instantaneous	1	1	1	0	0.5	0.6	0	0	0	0	0	0

SLS characteristic 6	Medium-term	1	1	0	1	0	0	0	0	0	0	0	0
SLS characteristic 7	Medium-term	1	1	0.7	1	0	0	0	0	0	0	0	0
SLS characteristic 8	Short-term	1	1	0	1	0.5	0	0	0	0	0	0	0
SLS characteristic 9	Short-term	1	1	0.7	1	0.5	0	0	0	0	0	0	0
SLS characteristic 10	Instantaneous	1	1	0	1	0	0.6	0	0	0	0	0	0
SLS characteristic 11	Instantaneous	1	1	0.7	1	0	0.6	0	0	0	0	0	0
SLS characteristic 12	Instantaneous	1	1	0	1	0.5	0.6	0	0	0	0	0	0
SLS characteristic 13	Instantaneous	1	1	0.7	1	0.5	0.6	0	0	0	0	0	0
SLS characteristic 14	Short-term	1	1	0	0	1	0	0	0	0	0	0	0
SLS characteristic 15	Short-term	1	1	0.7	0	1	0	0	0	0	0	0	0
SLS characteristic 16	Instantaneous	1	1	0	0	1	0.6	0	0	0	0	0	0
SLS characteristic 17	Instantaneous	1	1	0.7	0	1	0.6	0	0	0	0	0	0
SLS characteristic 18	Instantaneous	1	1	0	0	0	1	0	0	0	0	0	0
SLS characteristic 19	Instantaneous	1	1	0.7	0	0	1	0	0	0	0	0	0
SLS characteristic 20	Instantaneous	1	1	0	0	0.5	1	0	0	0	0	0	0
SLS characteristic 21	Instantaneous	1	1	0.7	0	0.5	1	0	0	0	0	0	0

Seismic load combinations

The action effects due to the combination of the horizontal components of the seismic action are computed using the following combinations:

$$E_{Edx} + 0,3 \cdot E_{Edy}$$

$$0,3 \cdot E_{Edx} + E_{Edy}$$

Combinations of actions for Damage Limit State (SLD)

Name	Duration	G1	G2	Q cat A	Q cat H	Snow	Ortho wind	Wind X	Wind Y	Dynamic SLV X	Dynamic SLV Y	Dynamic SLD X	Dynamic SLD Y
Dynamic SLD 1 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	0.3
Dynamic SLD 1 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	0.3
Dynamic SLD 1 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	0.3
Dynamic SLD 1 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	0.3
Dynamic SLD 2 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	-0.3
Dynamic SLD 2 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	-0.3
Dynamic SLD 2 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	-0.3

Dynamic SLD 2 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	1	-0.3
Dynamic SLD 3 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	0.3
Dynamic SLD 3 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	0.3
Dynamic SLD 3 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	0.3
Dynamic SLD 3 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	0.3
Dynamic SLD 4 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	-0.3
Dynamic SLD 4 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	-0.3
Dynamic SLD 4 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	-0.3
Dynamic SLD 4 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-1	-0.3
Dynamic SLD 5 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	1
Dynamic SLD 5 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	1
Dynamic SLD 5 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	1
Dynamic SLD 5 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	1
Dynamic SLD 6 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	-1
Dynamic SLD 6 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	-1
Dynamic SLD 6 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	-1
Dynamic SLD 6 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	0.3	-1
Dynamic SLD 7 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	1
Dynamic SLD 7 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	1
Dynamic SLD 7 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	1
Dynamic SLD 7 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	1
Dynamic SLD 8 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	-1
Dynamic SLD 8 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	-1
Dynamic SLD 8 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	-1
Dynamic SLD 8 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0	0	-0.3	-1

Combinations of actions for Life Safety Limit State (SLV)

Name	Duration	G1	G2	Q cat A	Q cat H	Snow	Ortho wind	Wind X	Wind Y	Dynamic SLV X	Dynamic SLV Y	Dynamic SLD X	Dynamic SLD Y
Dynamic SLV 1 ex+ey+	Instantaneous	1	1	0.3	0	0	0	0	0	1	0.3	0	0
Dynamic SLV 1 ex+ey-	Instantaneous	1	1	0.3	0	0	0	0	0	1	0.3	0	0
Dynamic SLV 1 ex-ey+	Instantaneous	1	1	0.3	0	0	0	0	0	1	0.3	0	0
Dynamic SLV 1 ex-ey-	Instantaneous	1	1	0.3	0	0	0	0	0	1	0.3	0	0

Dynamic SLV 2 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	1	-0.3	0	0
Dynamic SLV 2 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	1	-0.3	0	0
Dynamic SLV 2 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	1	-0.3	0	0
Dynamic SLV 2 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	1	-0.3	0	0
Dynamic SLV 3 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-1	0.3	0	0
Dynamic SLV 3 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-1	0.3	0	0
Dynamic SLV 3 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-1	0.3	0	0
Dynamic SLV 3 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-1	0.3	0	0
Dynamic SLV 4 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-1	-0.3	0	0
Dynamic SLV 4 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-1	-0.3	0	0
Dynamic SLV 4 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-1	-0.3	0	0
Dynamic SLV 4 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-1	-0.3	0	0
Dynamic SLV 5 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	1	0	0
Dynamic SLV 5 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	1	0	0
Dynamic SLV 5 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	1	0	0
Dynamic SLV 5 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	1	0	0
Dynamic SLV 6 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	-1	0	0
Dynamic SLV 6 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	-1	0	0
Dynamic SLV 6 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	-1	0	0
Dynamic SLV 6 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	0.3	-1	0	0
Dynamic SLV 7 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	1	0	0
Dynamic SLV 7 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	1	0	0
Dynamic SLV 7 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	1	0	0
Dynamic SLV 7 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	1	0	0
Dynamic SLV 8 ex+ ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	-1	0	0
Dynamic SLV 8 ex+ ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	-1	0	0
Dynamic SLV 8 ex- ey+	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	-1	0	0
Dynamic SLV 8 ex- ey-	Instantaneous	1	1	0.3	0	0	0	0	0	-0.3	-1	0	0

Horizontal actions

Modal analysis

The modal analysis is used to determine the vibration modes of the structure, useful to understand the seismic behaviour of the building and to proceed with the linear dynamic analysis.

The modal analysis involves the solution of the generalized eigenvalue problem:

$$[K - \Omega^2 M] \Phi = 0$$

where K is the stiffness matrix, M the mass matrix, Ω^2 is the diagonal matrix of the eigenvalues and Φ is the corresponding matrix of eigenvectors or modal shapes (normalized with respect to the mass matrix); the seismic masses of the diaphragms are calculated with the following combination of vertical loads:

$$\sum G_{k,j} + \sum \psi_{Ei} \cdot Q_{ki} = G_1 + G_2 + \sum \psi_{Ei} \cdot Q_{ki}$$

where ψ_{Ei} is the combination coefficient for variable action i . The combination coefficients shall be computed from the following expression:

$$\psi_{Ei} = \varphi \cdot \psi_{2i}$$

Type of variable action	Storey	φ
Categories A-C (as defined in EN 1991-1-1)	Roof	1
	Storeys with correlated occupancies	0.8
	Independently occupied storeys	0.5
Categories D-F (as defined in EN 1991-1-1)		1

The base shear forces for DLS and ULS and the respective acceleration values are given below.

The eigenvalue, obtained by the solution of the generalized eigenvalue problem, is the square of the circular frequency ω related to the period, T , and to the frequency, f , by the following equations:

$$T = \frac{1}{f} \text{ and } f = \frac{\omega}{2\pi}$$

The participating mass ratio for the mode i -th, corresponding to an acceleration in the global axis X and Y and to a rotational acceleration around the vertical axis Z, is given by:

$$M_x^i = \frac{m_x^i}{\sum m_{x,j}} [\%]$$

$$M_y^i = \frac{m_y^i}{\sum m_{y,j}} [\%]$$

$$M_z^i = \frac{m_z^i}{\sum I_{z,j}} [\%]$$

where

$$m_x^i = \frac{([\Phi^i]^T M R_x)^2}{[\Phi^i]^T M \Phi^i}$$

$$m_y^i = \frac{([\Phi^i]^T M R_y)^2}{[\Phi^i]^T M \Phi^i}$$

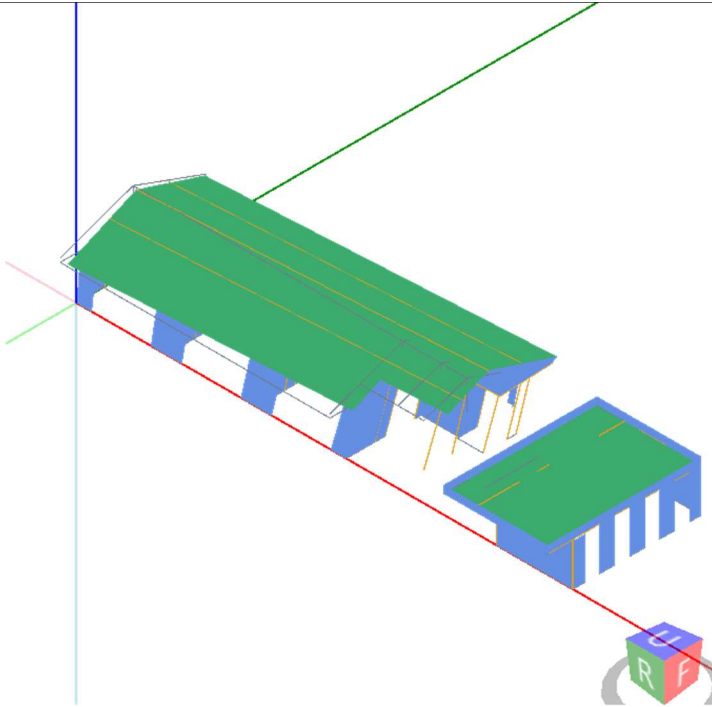
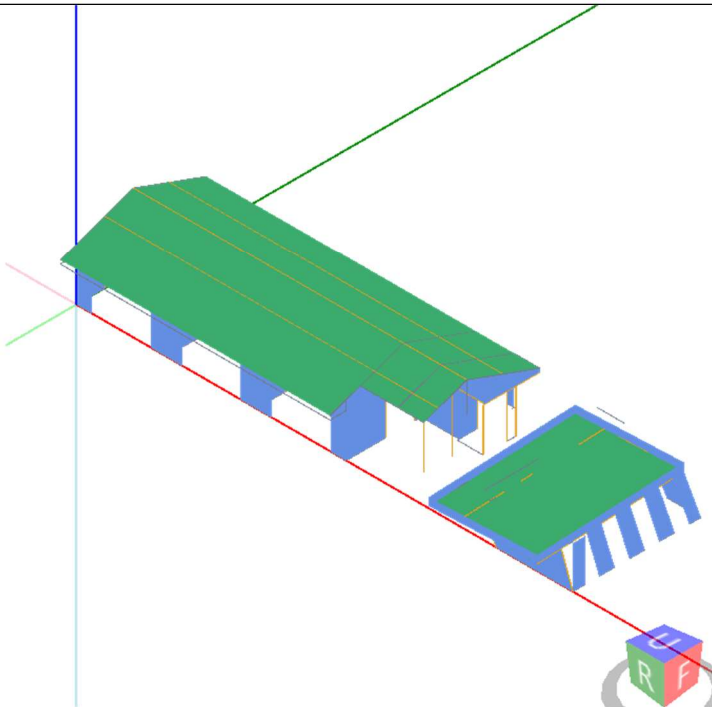
$$m_z^i = \frac{([\Phi^i]^T M R_z)^2}{[\Phi^i]^T M \Phi^i}$$

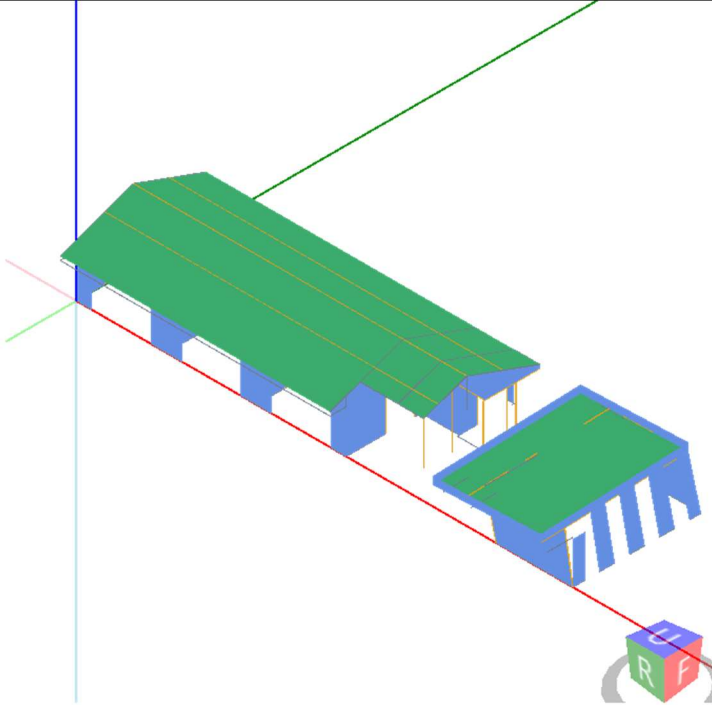
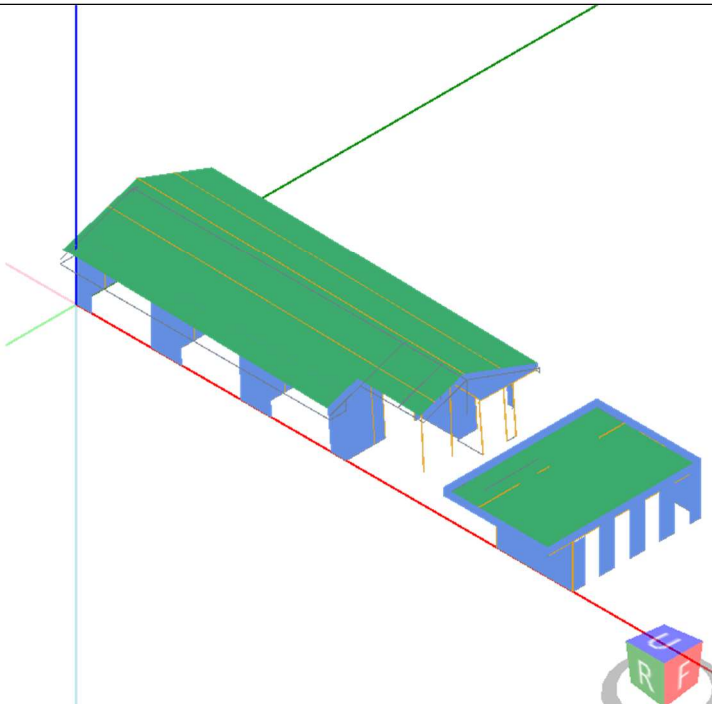
and where $\sum m_{x,j}$, $\sum m_{y,j}$ and $\sum I_{z,j}$ are the total masses acting in the axis X, Y and the total rotational inertia about the axis Z of the unrestrained j -th degrees of freedom.

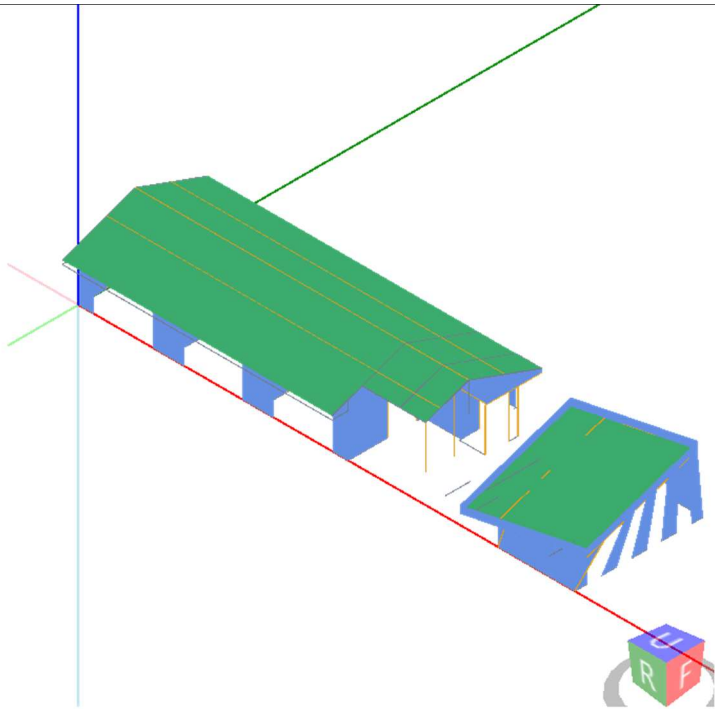
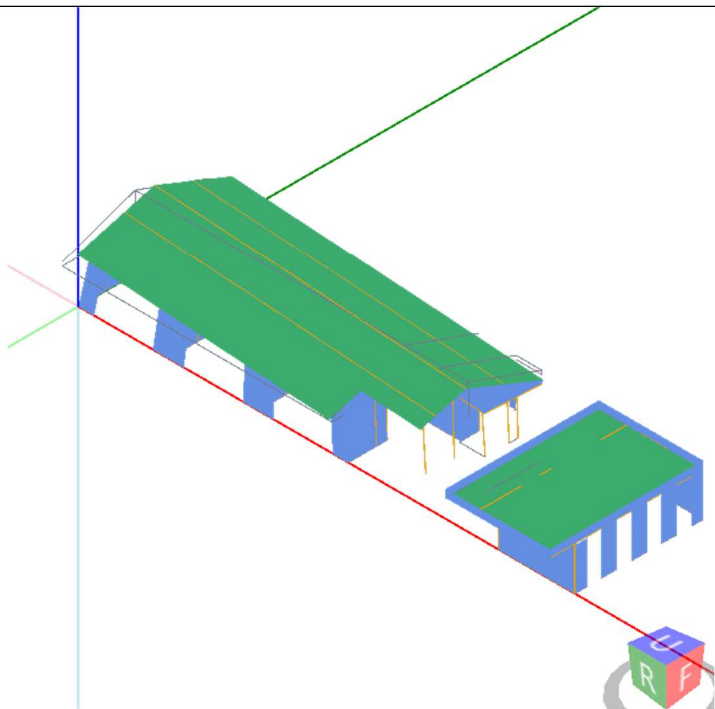
Mode	Period [s]	Frequency [Hz]	M _x [%]	Sum M _x [%]	M _y [%]	Sum M _y [%]	M _z [%]	Sum M _z [%]
Mode 1	0.57	1.76	12.40	12.40	30.26	30.26	34.96	34.96
Mode 2	0.56	1.78	0.49	12.89	24.09	54.36	0.26	35.22
Mode 3	0.53	1.90	24.14	37.03	0.67	55.03	0.17	35.39
Mode 4	0.46	2.15	36.01	73.04	31.84	86.87	5.35	40.74
Mode 5	0.41	2.46	0.68	73.72	0.54	87.40	8.50	49.24
Mode 6	0.39	2.59	25.58	99.30	8.04	95.44	49.32	98.56
Mode 7	0.23	4.29	0.03	99.33	4.33	99.77	0.20	98.76
Mode 8	0.17	6.04	0.47	99.80	0.20	99.97	1.24	100.00
Mode 9	0.12	8.08	0.20	100.00	0.03	100.00	0.00	100.00

The table shows, limited to the first six modes of vibration, the corresponding period and the modal deformed shape.

Mode	Period [s]	Modal deformed shape
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Mode 1	0.57	
Mode 2	0.56	

Mode 3	0.53	
Mode 4	0.46	

Mode 5	0.41	
Mode 6	0.39	

Dynamic linear analysis

The dynamic linear analysis consists of:

- the calculation of the seismic effects (the seismic action is represented by the design response spectrum), of each of the vibration modes calculated in the modal analysis;
- combination of these effects.

The seismic effects of the structural model are obtained by the application of the following external loads:

$$F_x^i = \Gamma_x^i S_d(T_i) M \Phi^i$$

and

$$F_y^i = \Gamma_y^i S_d(T_i) M \Phi^i$$

where:

F_x^i and F_y^i are the external loads of the i -th vibration mode due to seismic action along X and Y;

$S_d(T_i)$ is the spectrum value corresponding to the i -th period;

Φ^i is the i -th modal shape;

Γ_x^i and Γ_y^i are the participating modal factor of the i -th mode given by:

$$\Gamma_x^i = \frac{[\Phi^i]^T M R_x}{[\Phi^i]^T M \Phi^i} \text{ and } \Gamma_y^i = \frac{[\Phi^i]^T M R_y}{[\Phi^i]^T M \Phi^i}$$

The effects for a given direction of acceleration (along X or Y) and for each of the vibration modes are combined with the Complete Quadratic Combination technique defined as:

$$E = \sqrt{\sum_j \sum_i \rho_{ij} \cdot E_i \cdot E_j}$$

where:

E_j is the effect of the j -th vibration mode;

ρ_{ij} is the correlation coefficient of the i -th mode and the j -th mode, given by:

$$\rho_{ij} = \frac{8 \cdot \xi^2 \cdot \beta_{ij}^{3/2}}{(1 + \beta_{ij}) \cdot [(1 - \beta_{ij})^2 + 4 \cdot \xi^2 \cdot \beta_{ij}]}$$

ξ is the damping ratio in the i -th and j -th modes;

$$\beta_{ij} = T_j / T_i.$$

The following table provides the diaphragms properties.

Diaphragm	Height above the base of the timber structure [m]	xG [m]	yG [m]	Accidental eccentricity ex [m]	Accidental eccentricity ey [m]	Mass i [kg]
1	3.60	12.65	5.72	1.39	0.54	80609
2	3.60	37.15	4.30	0.43	0.64	41916
3	4.98	13.40	4.27	1.39	0.61	43120

The following table provides, for each of the vibration mode, the corresponding period and value of the response spectrum, both SLV value and the SLD value.

Mode	Period [s]	SLV spectrum value [g]	SLD spectrum value [g]
Mode 1	0.60	0.11	0.18
Mode 2	0.56	0.12	0.19
Mode 3	0.54	0.12	0.20
Mode 4	0.45	0.14	0.22
Mode 5	0.40	0.14	0.22
Mode 6	0.38	0.14	0.22
Mode 7	0.24	0.14	0.22
Mode 8	0.16	0.14	0.22
Mode 9	0.13	0.14	0.19

Wind

The table below illustrates the horizontal forces acting on the storeys due to the wind action and the coordinates of their respective application points.

Diaphragm	Height above the reference plane [m]	xG,wind [m]	yG,wind [m]	Fx [kN]	Fy [kN]
1	3.60	13.91	5.42	8.61	24.06
2	3.60	37.05	4.30	9.51	7.03
3	4.98	13.91	4.74	2.01	5.00

Design of the structural elements

CLT floors

Calculation model

The calculation model adopted for the design of CLT in bending out-of-plane is that of mechanically jointed beams with deformable connection in accordance with Appendix B of EN 1995-1-1. The shear flexibility of the transverse layers is considered using the γ -method (gamma): namely with Möhler theory for CLT panel having up to 3 layers oriented in the direction of calculation and with Shelling theory for CLT panel having more than 3 layers oriented in the direction of calculation.

The effective bending stiffness is taken as:

$$EJ_{eff} = \sum_{i=1}^n (E_i J_i + \gamma_i E_i A_i a_i^2)$$

$$\gamma_i = \left[1 + \frac{\pi^2 E_i A_i}{G_R \cdot \frac{b}{d} \cdot l_{ref}^2} \right]^{-1}$$

where:

J_i is the moment of inertia of layer i in reference to its neutral axis;

A_i is the cross-sectional area of layer i ;

a_i is the distance between the centre of gravity of layer i and centre of gravity of the CLT element;

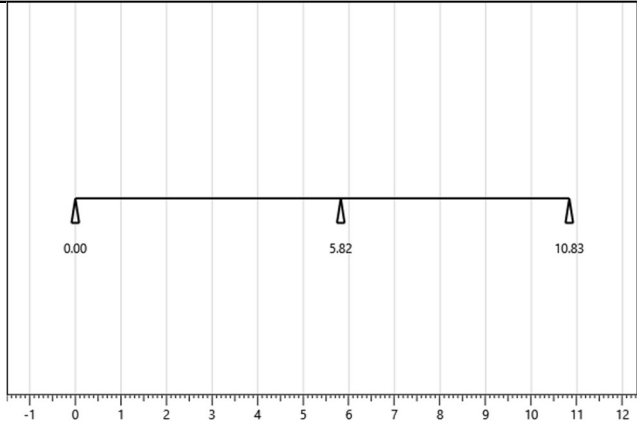
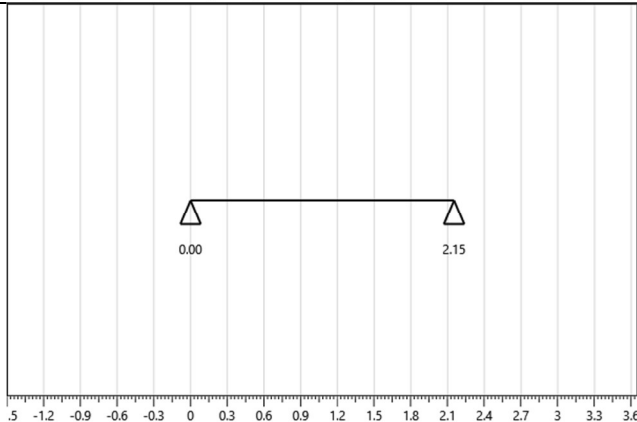
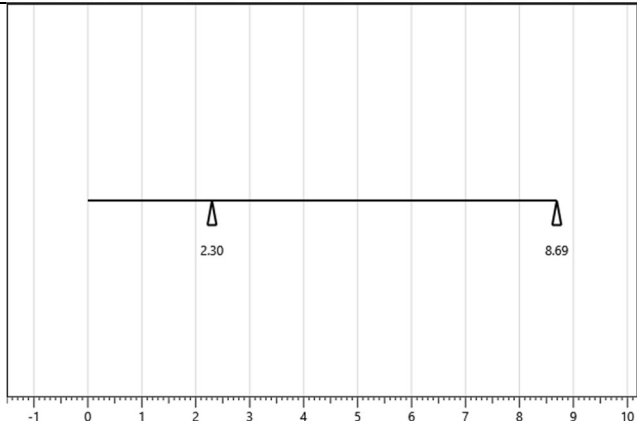
l_{ref} is the reference length of the span;

G_R is the rolling shear modulus (mean value).

The reference length of the spans (l_{ref}) is taken, depending on the static scheme, as reported in the following table.

Structural scheme	Reference length of the span
Simply supported beam	$l_{ref} = l$
Span of a continuous beam	$l_{ref} = 0.8 l$
Internal support of a continuous beam	$l_{ref} = 0.8 l_{min}$
Cantilever	$l_{ref} = 2 l$

The following table shows, for each floor and relatively to the different spans, the values of the reference lengths of the spans, the effective moment of inertia of the cross-sections of the CLT floor and the structural scheme adopted.

Floor name	Calculation width of the strip of floor [m]	Reference length l_{ref} [m]	Jeff [mm ⁴]	Structural scheme
Floor 9	0.5	4.66 4.01	9.763E7 9.512E7	
Floor 11	0.5	2.15	7.665E7	
Floor 13	0.5	4.60 5.11	3.403E8 3.508E8	

Bending strength

The checks are conducted according to § 6.1.6 of EN 1995-1-1. The following expression shall be satisfied:

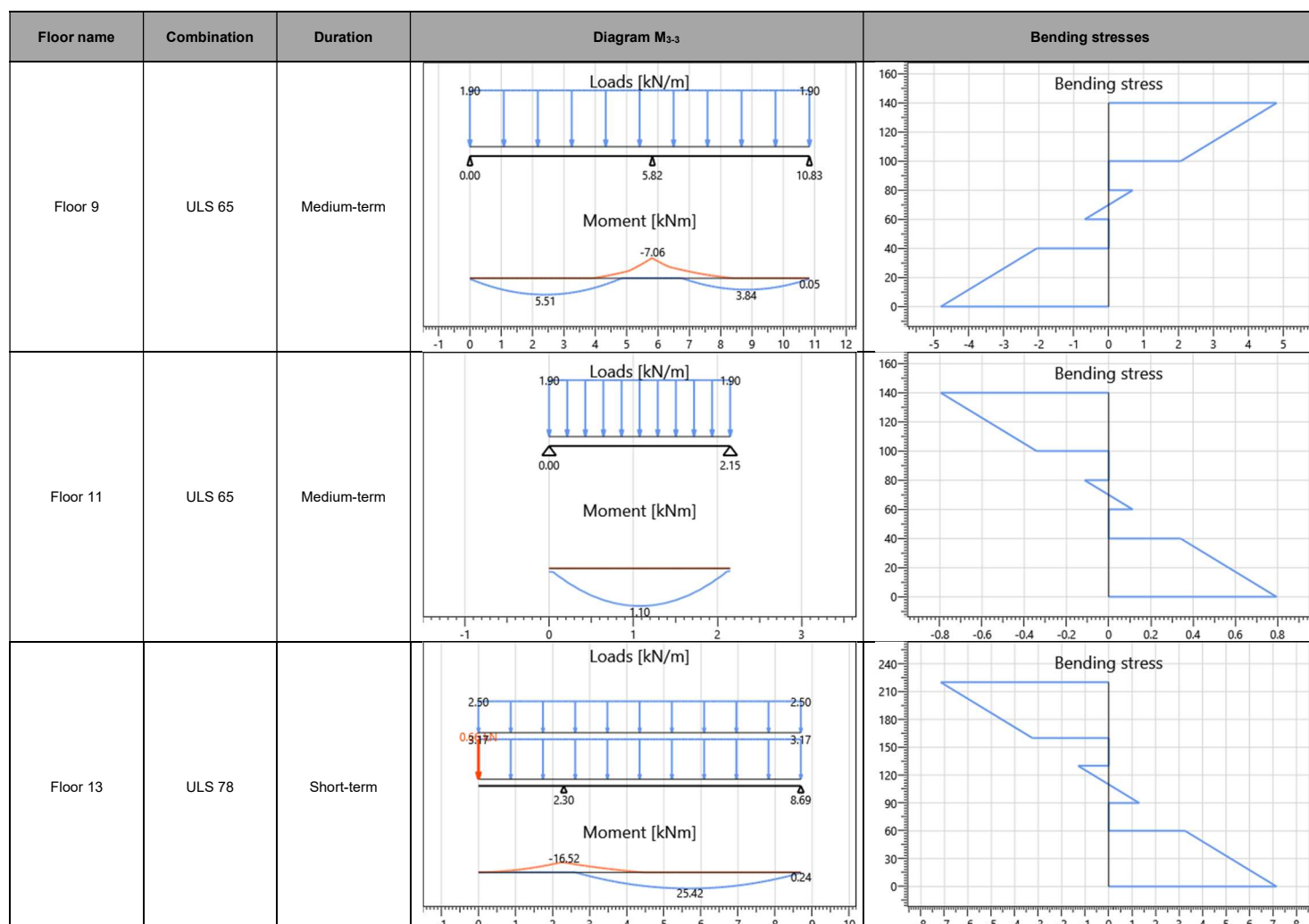
$$\frac{\sigma_{m,d}}{f_{m,d}} \leq 1$$

where:

$\sigma_{m,d}$ is the design bending stress;

$f_{m,d}$ is the design bending strength.

The following table illustrates the structural schemes and the envelopes of the diagram of the bending moment for the part of each floor with the most sever checks.



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Section	M ₃₋₃ [kNm]	J _{eff} [mm ⁴]	Comb.	Service class	k _{mod}	γ _M	f _{m,d} [MPa]	σ _{m,d} [MPa]	Check
Floor 9	BBS 140 5s	-7.07	95121955	ULS 65	1	0.8	1.25	15.36	4.81	31%
Floor 11	BBS 140 5s	1.10	76645676	ULS 65	1	0.8	1.25	15.36	0.80	5%
Floor 13	BBS 220 5s	25.42	350829570	ULS 78	1	0.9	1.25	17.28	7.16	41%

Shear strength

Shear strength of the layers parallel to the calculation direction

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_{v,d}}{f_{v,d}} \leq 1$$

where:

$\tau_{v,d}$ is the design shear stress;

$f_{v,d}$ is the design shear strength for the actual condition.

The maximum design shear stress in the longitudinal layers can be evaluated using the following expression:

$$\tau_{v,d} = \frac{V_d \cdot S_{max}}{J_{eff} \cdot b}$$

where:

V_d is the total shear force at the location in question;

S_{max} is the first moment of area;

J_{eff} is the effective moment of inertia of the CLT element cross section;

b is the width of the CLT element cross section ($k_{cr} = 1$).

Rolling shear strength of the transversal layers

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_{R,d}}{f_{v,R,d}} \leq 1$$

where:

$\tau_{R,d}$ the design rolling shear stress;

$f_{v,R,d}$ the design shear strength.

The maximum design shear stress in the transversal layers can be evaluated using the following expression:

$$\tau_{R,d} = \frac{V_d \cdot S_{R,max}}{J_{eff} \cdot b}$$

where:

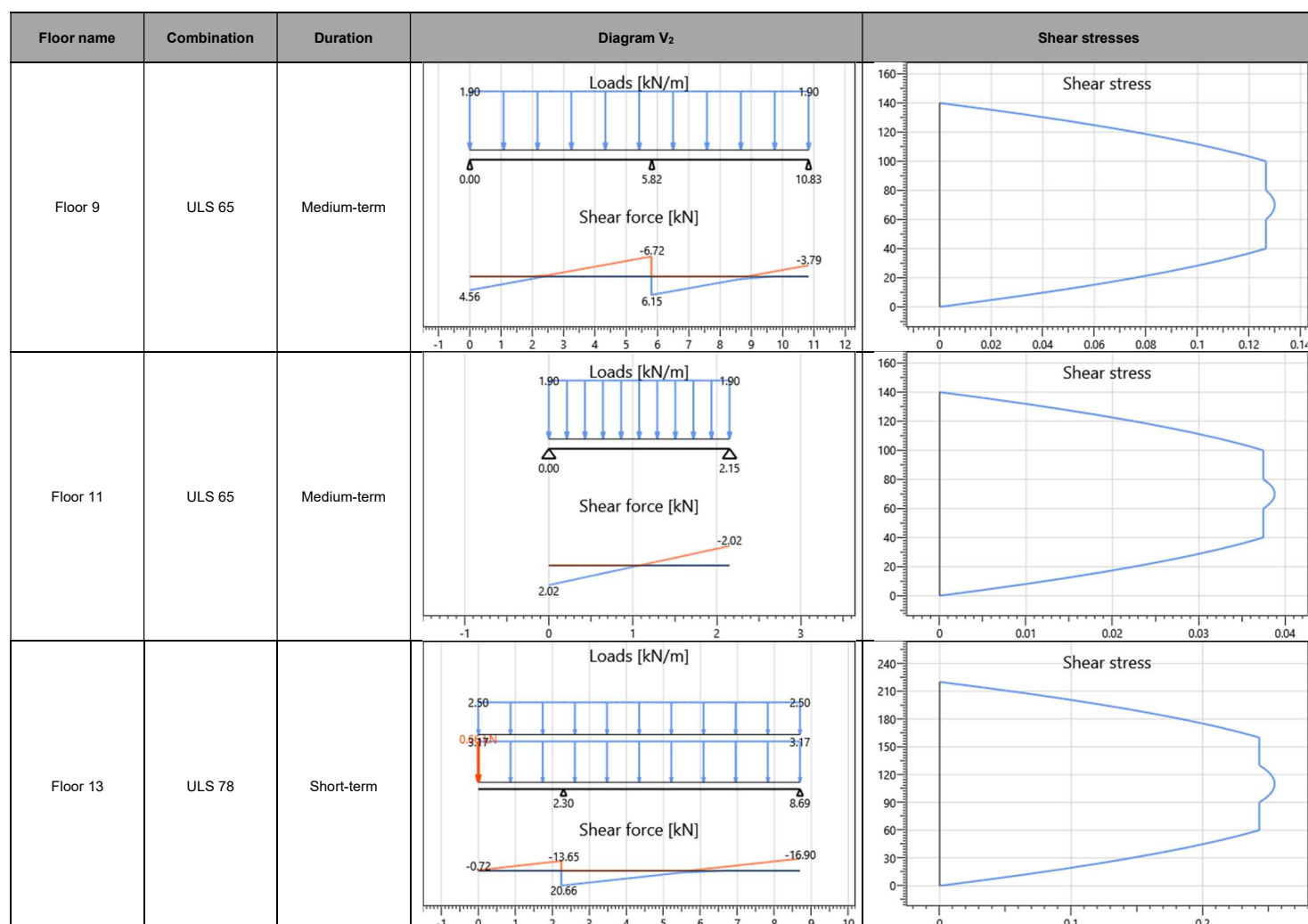
V_d is the total shear force at the location in question;

$S_{R,max}$ is the first moment of area;

J_{eff} is the effective moment of inertia of the CLT element cross section;

b is the width of the CLT element cross section ($k_{cr} = 1$).

The following table illustrates the structural schemes and the envelopes of the diagram of the shear force for the part of each floor with the most sever checks.



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Cross section	V ₂ [kN]	J _{eff} [mm ⁴]	Comb.	Service Class	k _{mod}	γ _M	f _{v,d} [MPa]	τ _{v,d} [MPa]	Check	f _{R,d} [MPa]	τ _{R,d} [MPa]	Check
Floor 9	BBS 140 5s	-6.72	97630603	ULS 65	1	0.8	1.25	2.56	0.13	5%	0.64	0.13	20%
Floor 11	BBS 140 5s	2.02	76645676	ULS 65	1	0.8	1.25	2.56	0.04	2%	0.64	0.04	6%
Floor 13	BBS 220 5s	20.66	350829570	ULS 78	1	0.9	1.25	2.88	0.25	9%	0.72	0.24	34%

Joist floors / solid wood floors

Bending strength

The checks are conducted according to § 6.3.2 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

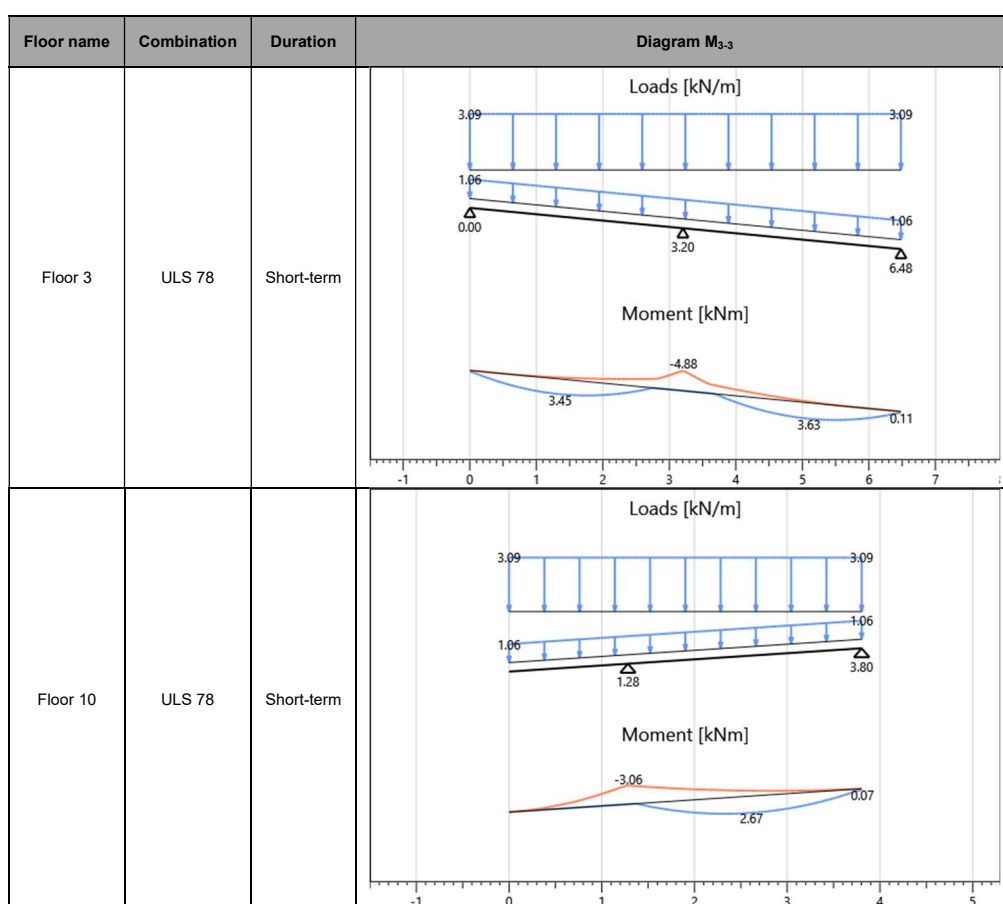
where:

$\sigma_{m,d}$ is the design bending stress;

$f_{m,d}$ is the design bending strength;

k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling.

k_{crit} is assumed equal to 1.0 for beams in which the lateral displacement of the compressed edge is prevented over the entire length and the torsional rotation is prevented at the supports.



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Section	M ₃₋₃ max [kNm]	W [mm ³]	k _{crit}	Comb.	Service Class	k _h	k _{mod}	γ _M	f _{m,d} [MPa]	σ _{m,d} [MPa]	Check
Floor 3	Spirovci 100x180 e=920	4.89	540000	1.00	ULS 78	1	1.00	0.9	1.3	16.62	9.06	55%
Floor 10	Spirovci 100x180 e=920	3.07	540000	1.00	ULS 78	1	1.00	0.9	1.3	16.62	5.68	34%

Shear strength

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

where:

τ_d is the design shear stress;

$f_{v,d}$ is the design shear strength for the actual condition.

For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member given as:

$$b_{ef} = k_{cr} \cdot b$$

where b is the width of the relevant section of the member.

The following value of k_{cr} are used

$k_{cr} = 0.67 (\leq 1)$ for solid timber

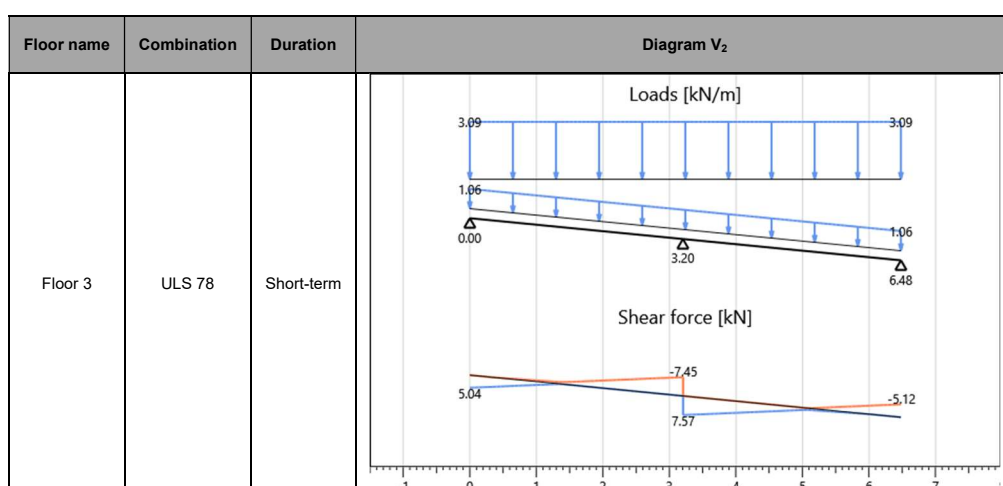
$k_{cr} = 0.83 (\leq 1)$ for glued laminated timber

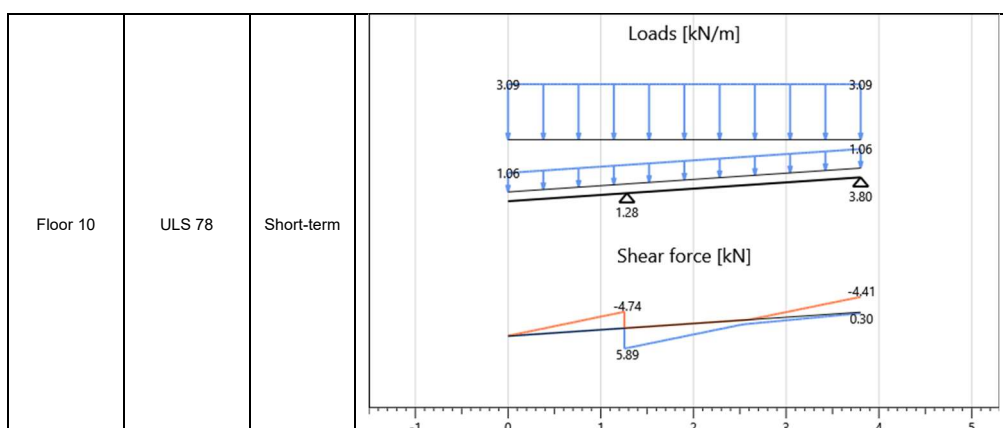
The maximum design shear stress in a rectangular cross section can be evaluated using the following expression:

$$\tau_d = \frac{3}{2} \cdot \frac{V_d}{k_{cr} \cdot A}$$

where A is the area of a joist cross section.

The following table illustrates the structural schemes and the envelopes of the shear force diagram for the joist of each floor with the most sever checks.





The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Section	$V_2 \text{ max}$ [kN]	Area [mm ²]	k_{cr}	Comb.	Service Class	k_{mod}	γ_M	$f_{v,d}$ [MPa]	$\tau_{2,d}$ [MPa]	Check
Floor 3	Spirovci 100x180 e=920	7.57	18000	0.67	ULS 78	1	0.9	1.3	2.77	0.94	34%
Floor 10	Spirovci 100x180 e=920	5.89	18000	0.67	ULS 78	1	0.9	1.3	2.77	0.73	26%

Floors deflections (SLS)

The deflection checks are carried out according to § 2.2.3 of EN 1995-1-1.

The net deflection below a straight line between the supports, $w_{net,fin}$, is taken as:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c$$

where:

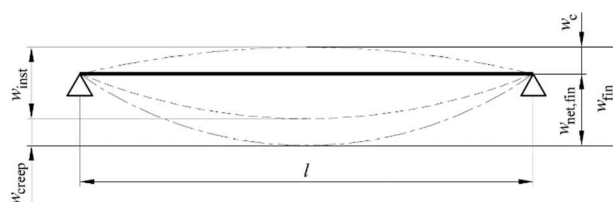
$w_{net,fin}$ is the net final deflection;

w_{inst} is the instantaneous deflection;

w_{creep} is the creep deflection;

w_c is the precamber (if applied);

w_{fin} is the final deflection.



The limiting values for deflections of floors are assumed as shown in the following table.

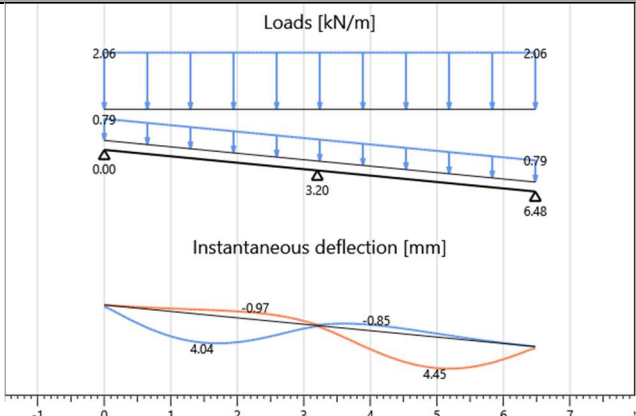
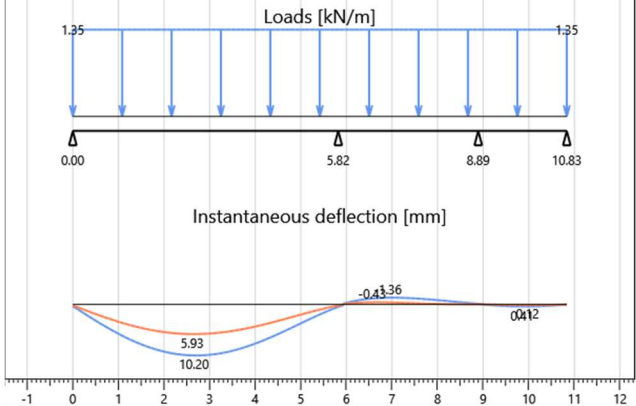
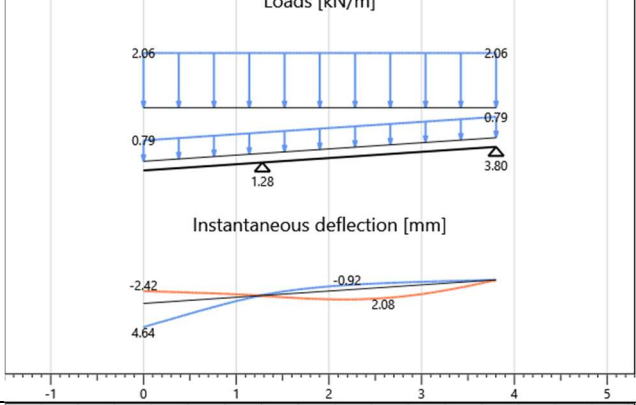
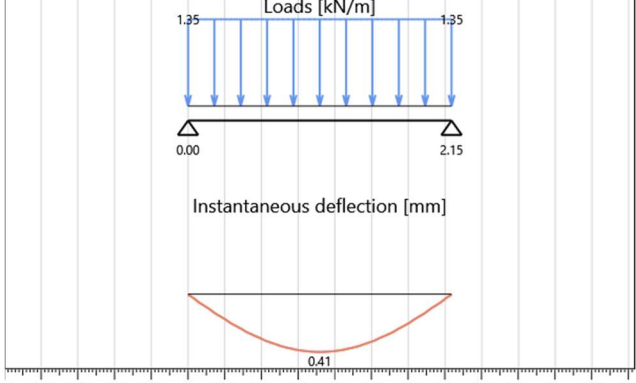
Deformation limits name	$w_{inst, span}$	$w_{net, fin, span}$	$w_{inst, overhang}$	$w_{net, fin, overhang}$	Neglect overhang check for deformation < 0
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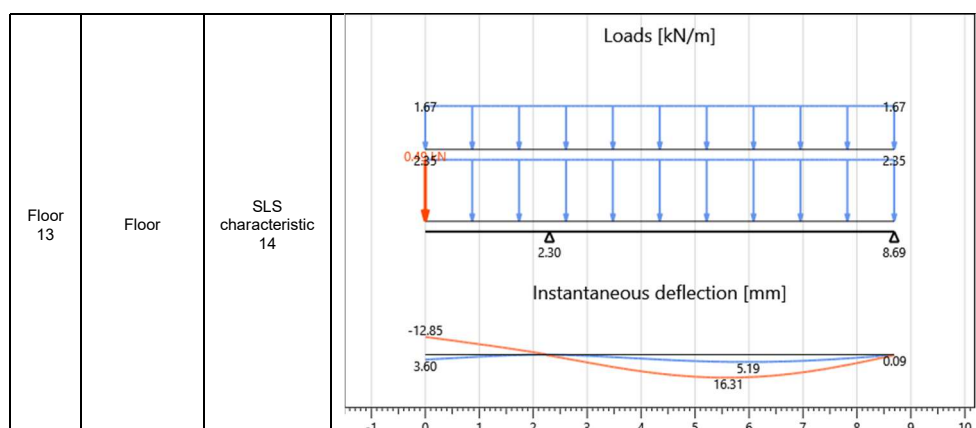
Floor	I/300	I/250	I/150	I/125	Yes
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Instantaneous deflection

The instantaneous deflection w_{inst} is calculated for the characteristic (rare) combination of actions.

The following table shows the deformation of each floor (relative to the element in which the deformation checks are the most severe).

Floor name	Deformation limits name	Combination	Instantaneous deflection
Floor 3	Floor	SLS characteristic 14	
Floor 9	Floor	SLS characteristic 4	
Floor 10	Floor	SLS characteristic 14	
Floor 11	Floor	SLS characteristic 4	



The table below shows the instantaneous deflection checks of the floor elements.

Floor name	Section	Combination	Most restrictive check	w_{inst} [mm]	w_{inst} limit [mm]	Deflection limit	Check
Floor 3	Spirovci 100x180 e=920	SLS characteristic 14	Internal span	4.45	10.92	$l/300$	41%
Floor 9	BBS 140 5s	SLS characteristic 4	Internal span	10.20	19.40	$l/300$	53%
Floor 10	Spirovci 100x180 e=920	SLS characteristic 14	Cantilever beams	4.64	8.55	$l/150$	54%
Floor 11	BBS 140 5s	SLS characteristic 4	Internal span	0.41	7.18	$l/300$	6%
Floor 13	BBS 220 5s	SLS characteristic 14	Internal span	16.31	21.31	$l/300$	77%

Final deflection

For structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations the final deformation, w_{fin} , may be taken as:

$$w_{fin} = w_{fin,G} + w_{fin,Q1} + \sum w_{fin,Qi}$$

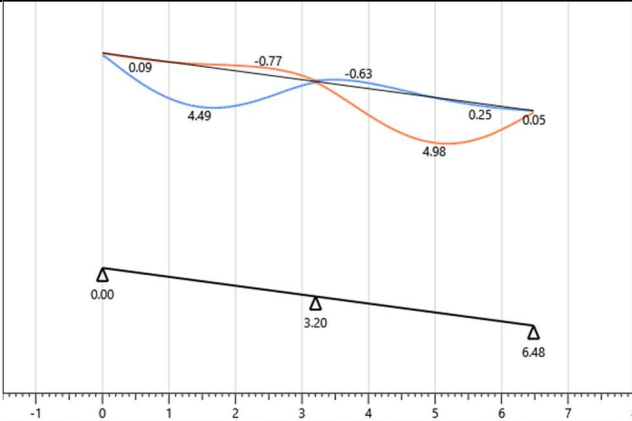
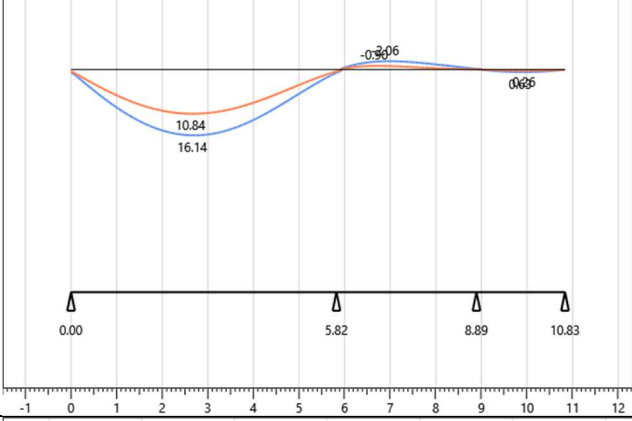
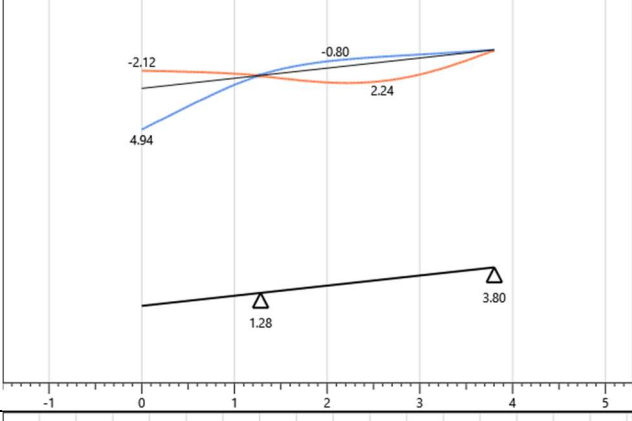
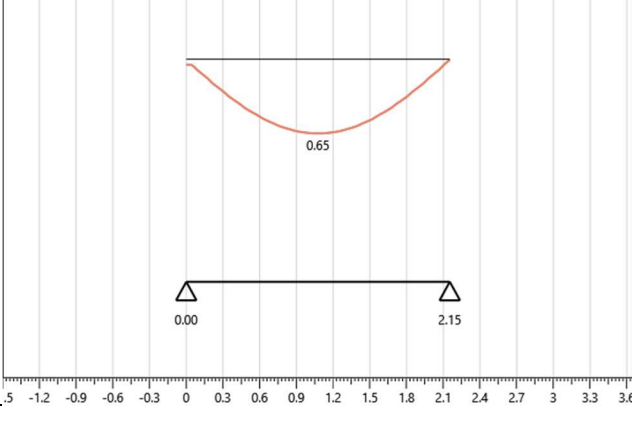
where:

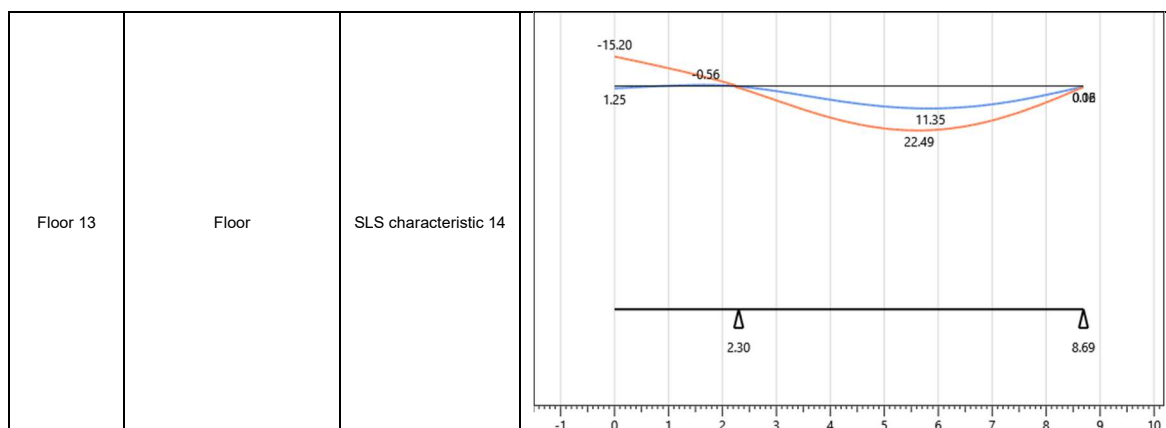
$$w_{fin,G} = w_{inst,G} \cdot (1 + k_{def}) \quad \text{for a permanent action, G}$$

$$w_{fin,Q,1} = w_{inst,Q,1} \cdot (1 + \Psi_{2,1} \cdot k_{def}) \quad \text{for the leading variable action, } Q_1$$

$$w_{fin,Q,i} = w_{inst,Q,i} \cdot (\Psi_{0,i} + \Psi_{2,1} \cdot k_{def}) \quad \text{for accompanying variable actions, } Q_i \text{ (i>1)}$$

The following table shows the deformation of each floor (relative to the element in which the deformation checks are the most severe).

Floor name	Deformation limits name	Combination	Final deflection
Floor 3	Floor	SLS characteristic 14	
Floor 9	Floor	SLS characteristic 4	
Floor 10	Floor	SLS characteristic 14	
Floor 11	Floor	SLS characteristic 4	



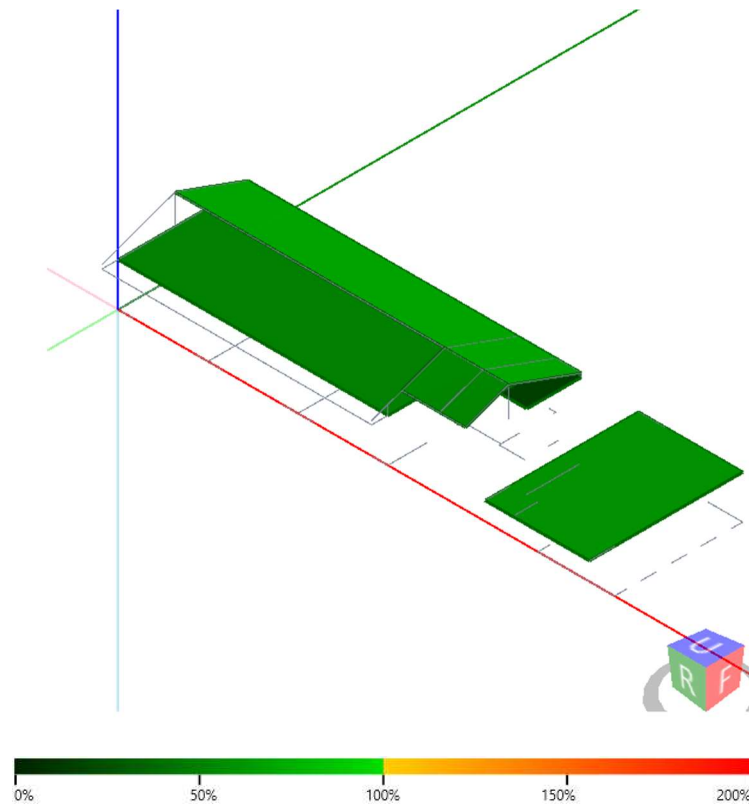
The table below shows the final deflection checks of the floor elements.

Floor name	Section	Combination	Service class	k_{def}	Most restrictive check	w_{fin} [mm]	w_{fin} limit [mm]	Deflection limit	Check
Floor 3	Spirovci 100x180 e=920	SLS characteristic 14	1	0.6	Internal span	4.98	13.10	$l/250$	38%
Floor 9	BBS 140 5s	SLS characteristic 4	1	0.8	Internal span	16.14	23.28	$l/250$	69%
Floor 10	Spirovci 100x180 e=920	SLS characteristic 14	1	0.6	Cantilever beams	4.94	10.26	$l/125$	48%
Floor 11	BBS 140 5s	SLS characteristic 4	1	0.8	Internal span	0.65	8.61	$l/250$	8%
Floor 13	BBS 220 5s	SLS characteristic 14	1	0.8	Internal span	22.49	25.57	$l/250$	88%

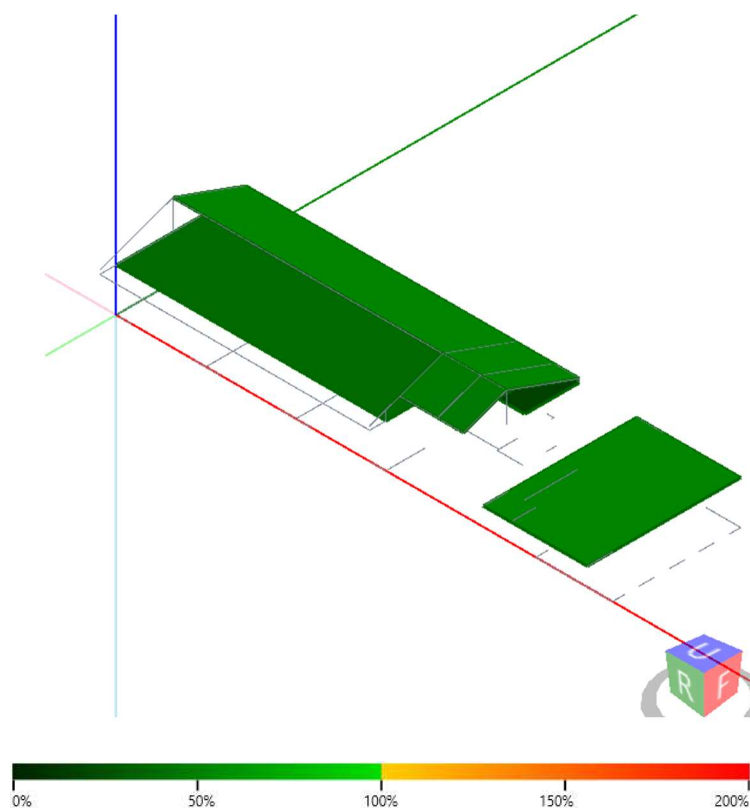
Verification percentage of floors

Ultimate Limit State (ULS)

Bending verifications

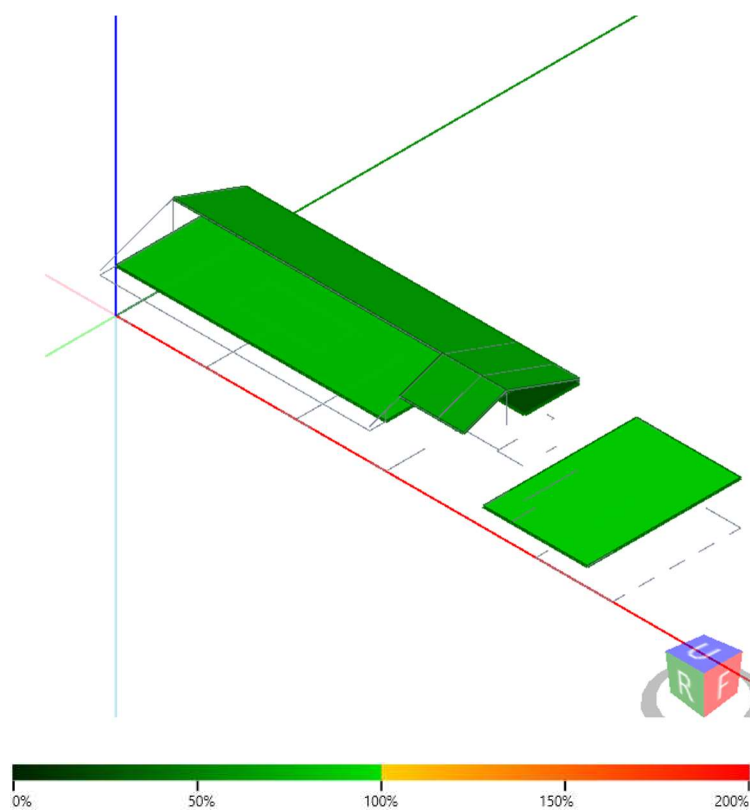


Shear verifications



Serviceability Limit State (SLS)

Deflection verifications



Timber beams

Bending strength

The checks are conducted according to § 6.3.2 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

where:

$\sigma_{m,d}$ is the design bending stress;

$f_{m,d}$ is the design bending strength;

k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling.

The factor k_{crit} is assumed equal to 1.0 for beams in which the lateral displacement of the compressed edge is prevented over the entire length and the torsional rotation is prevented at the supports. Otherwise, the factor is determined from the following expression:

$$k_{crit} = \begin{cases} 1 & \text{for } \lambda_{rel,m} \leq 0,75 \\ 1,56 - 0,75\lambda_{rel,m} & \text{for } 0,75 \leq \lambda_{rel,m} \leq 1,4 \\ \frac{1}{\lambda_{rel,m}^2} & \text{for } 1,4 < \lambda_{rel,m} \end{cases}$$

in which the relative slenderness for bending is taken as:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}}$$

and $\sigma_{m,crit}$, the critical bending stress calculated according to the classical theory of stability is taken as:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{l_{ef} W_y}$$

where:

$E_{0,05}$ is the fifth percentile value of modulus of elasticity parallel to grain;

$G_{0,05}$ is the fifth percentile value of shear modulus parallel to grain;

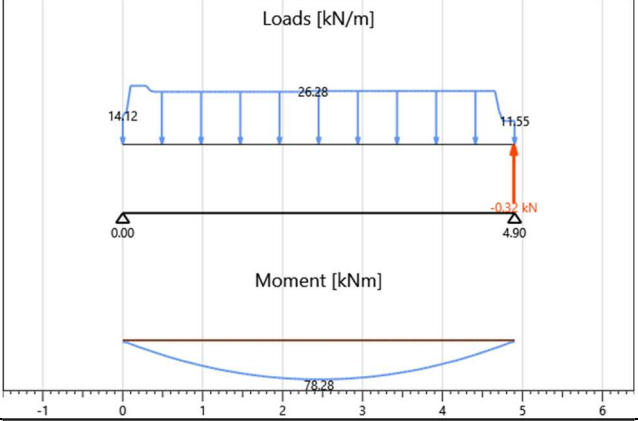
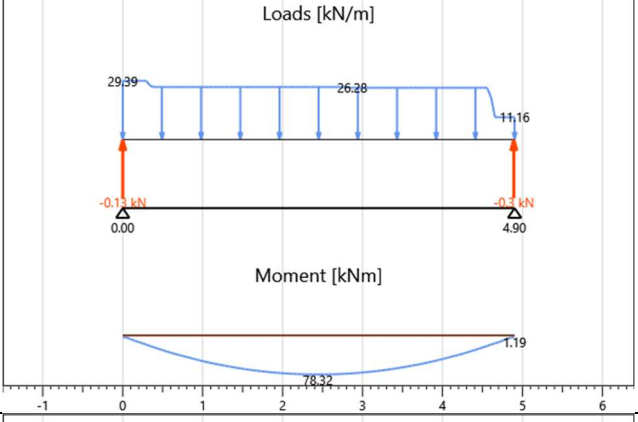
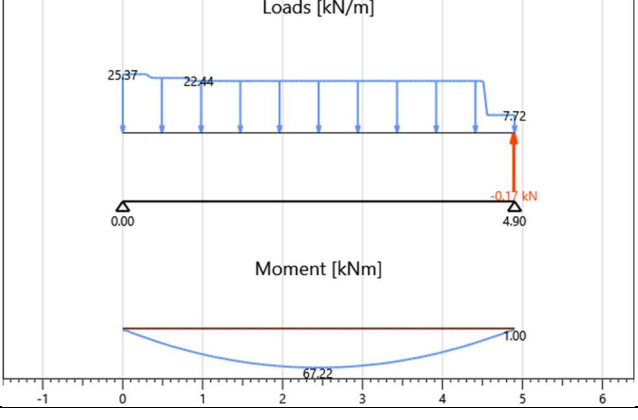
I_z is the second moment of area about the weak axis z;

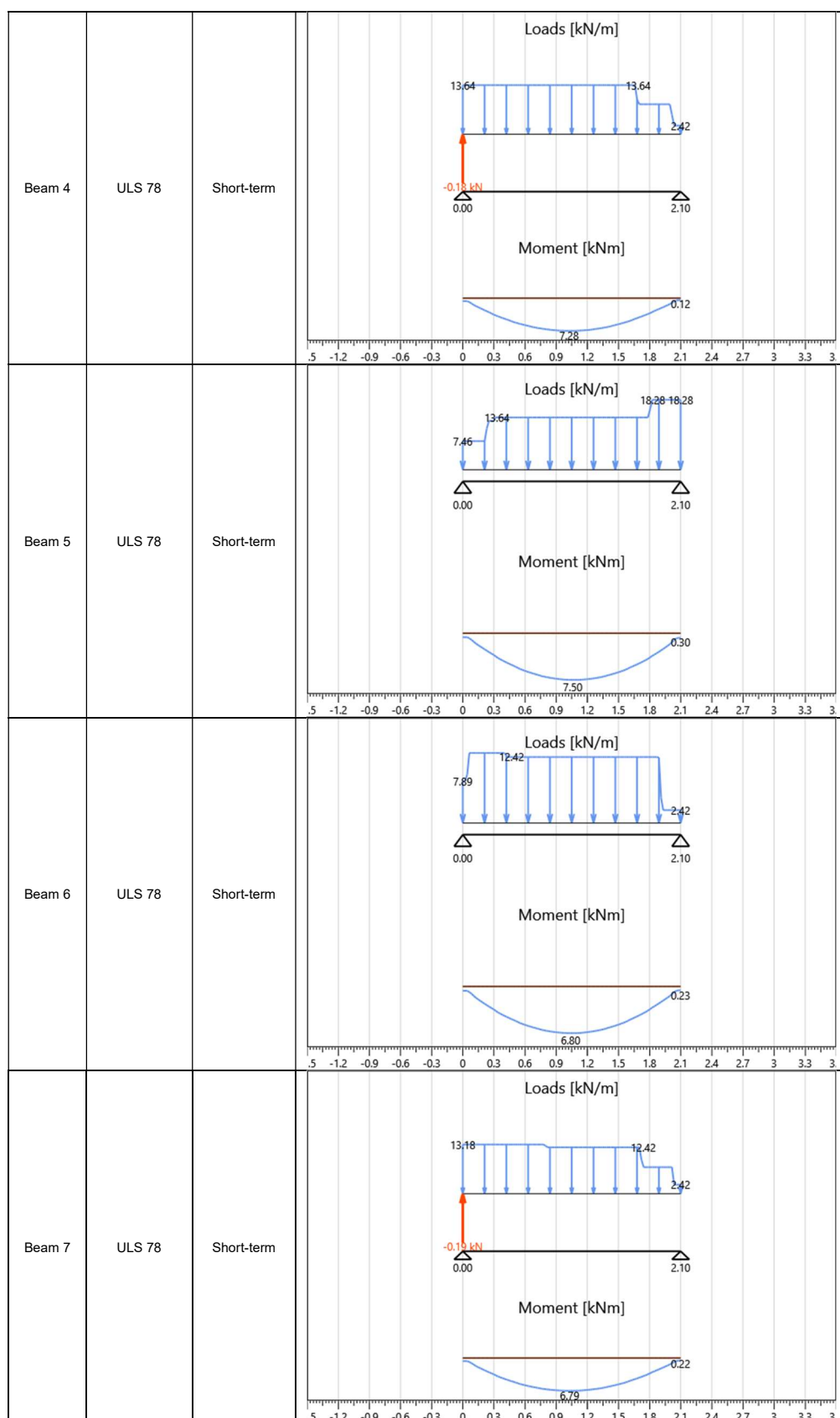
I_{tor} is the torsional moment of inertia;

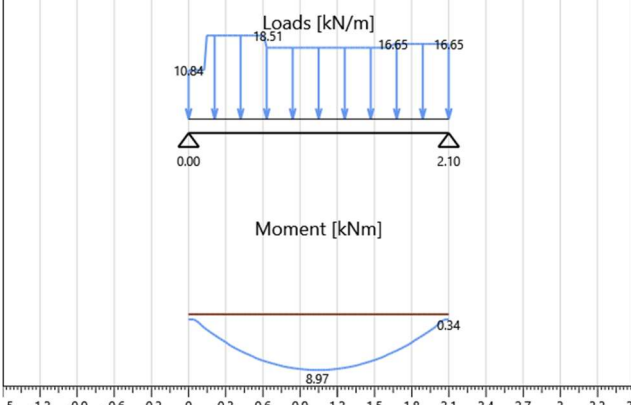
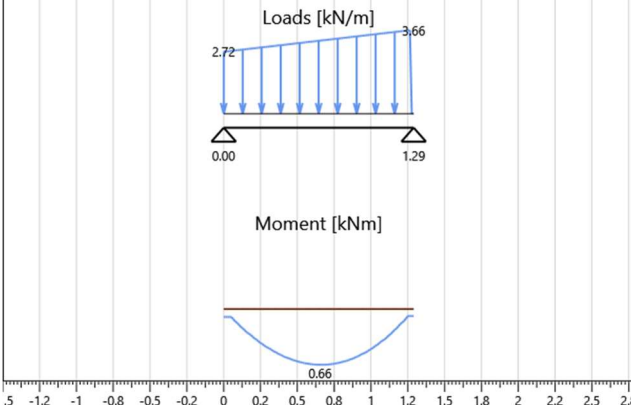
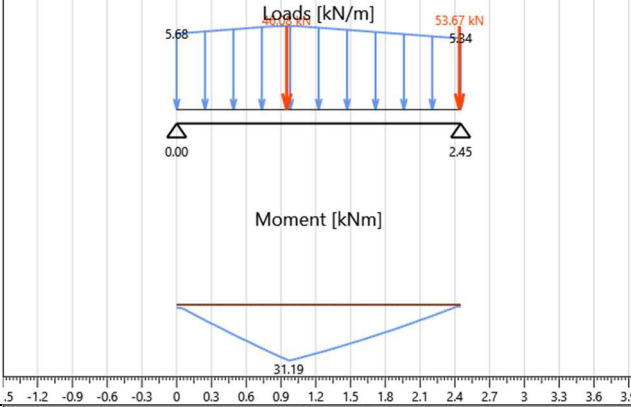
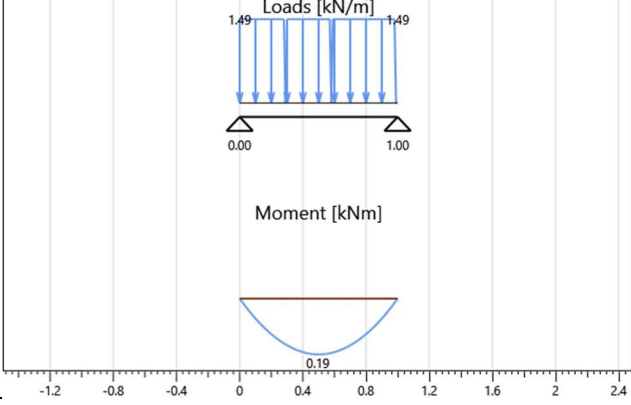
l_{ef} is the effective length of the beam, depending on the support conditions and the load configuration;

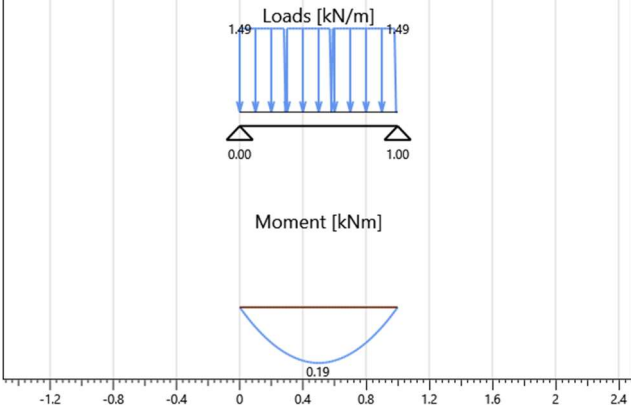
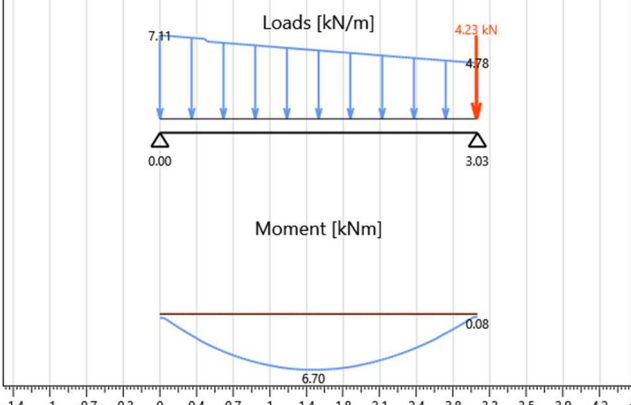
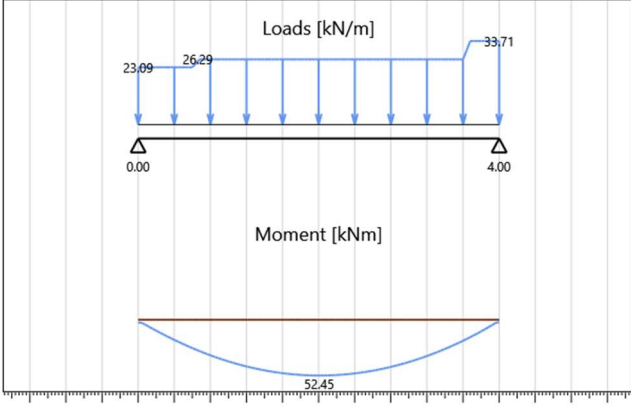
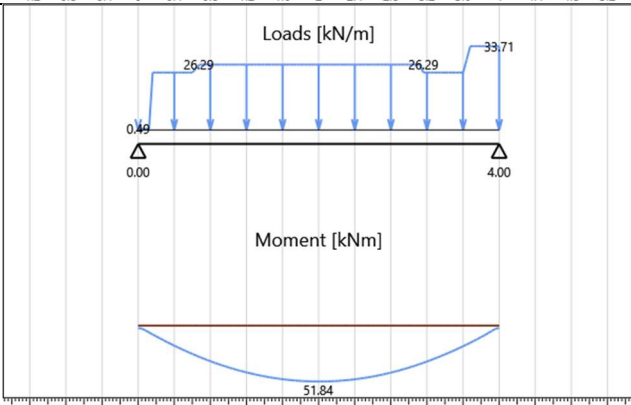
W_y is the section modulus about the strong axis y.

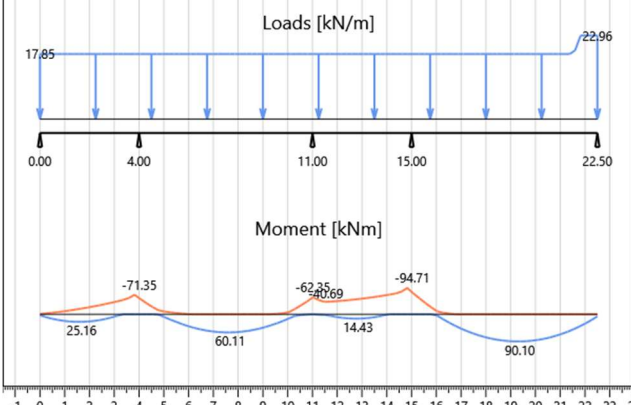
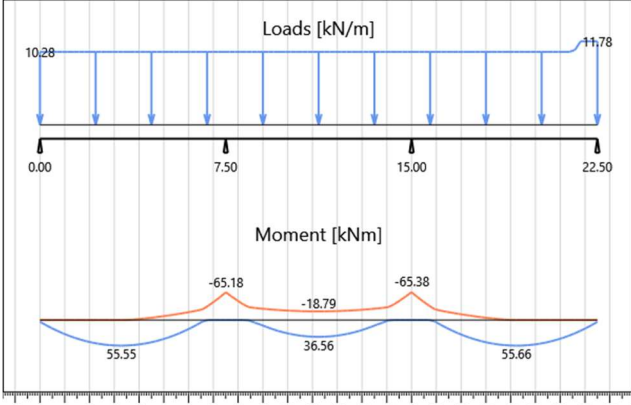
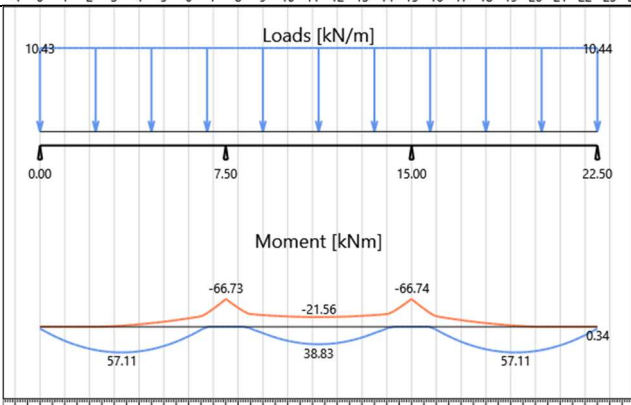
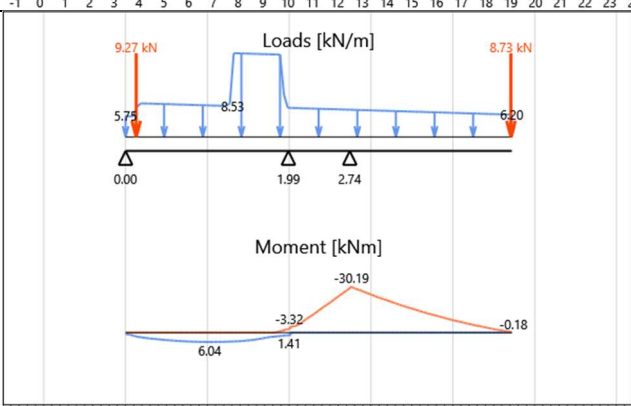
The following table shows, for each beam, the bending moment relating to the worst Ultimate Limit State combination.

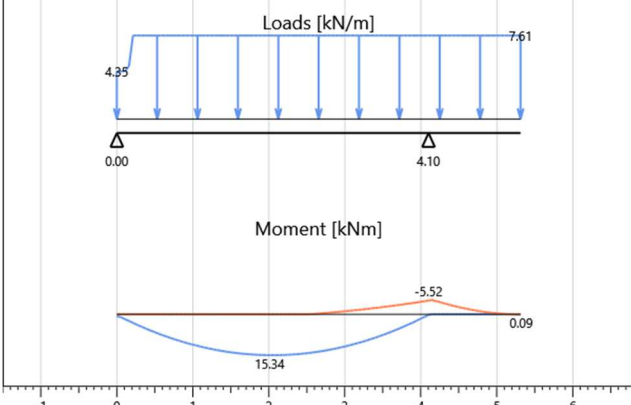
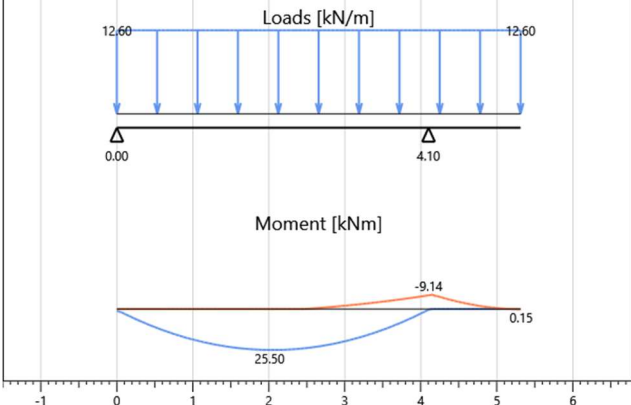
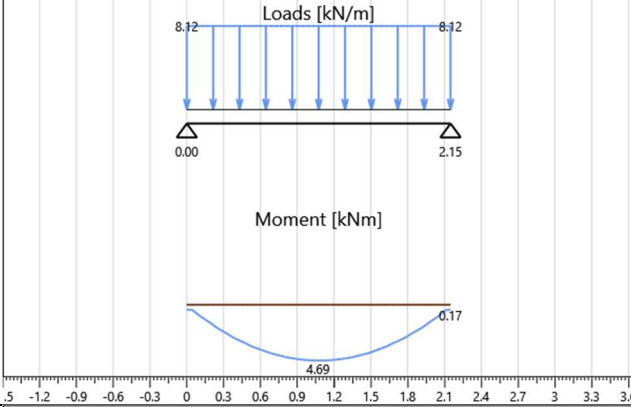
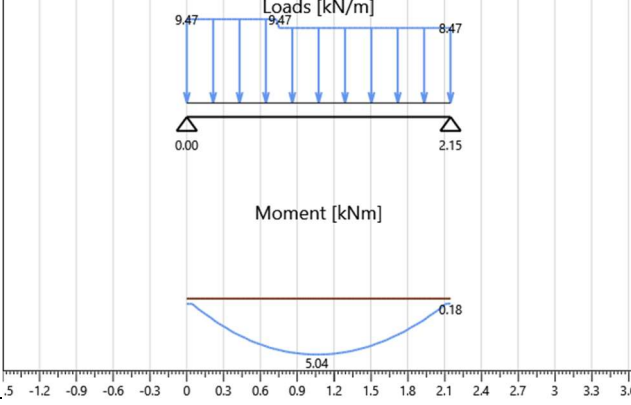
Beam	Combination	Duration	Diagram M_{3-3}
Beam 1	ULS 78	Short-term	
Beam 2	ULS 78	Short-term	
Beam 3	ULS 78	Short-term	

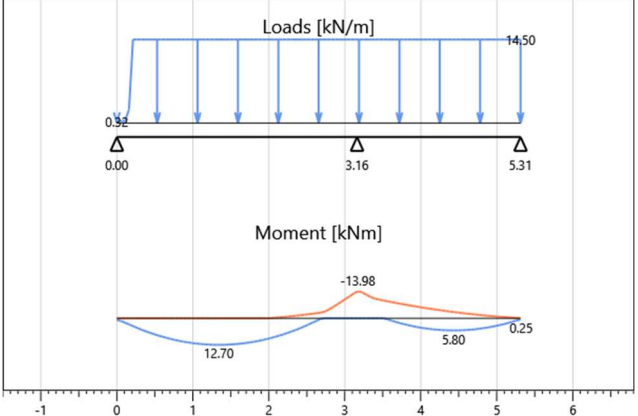
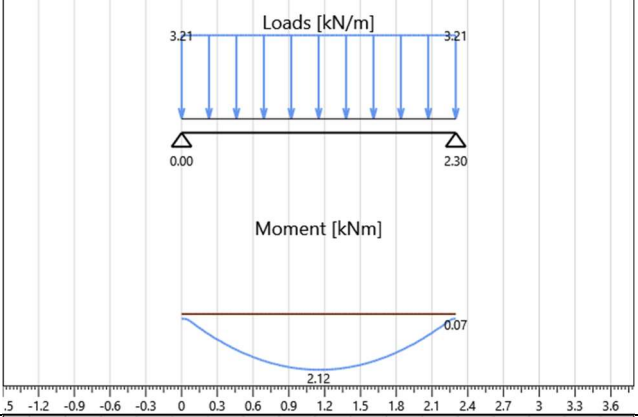
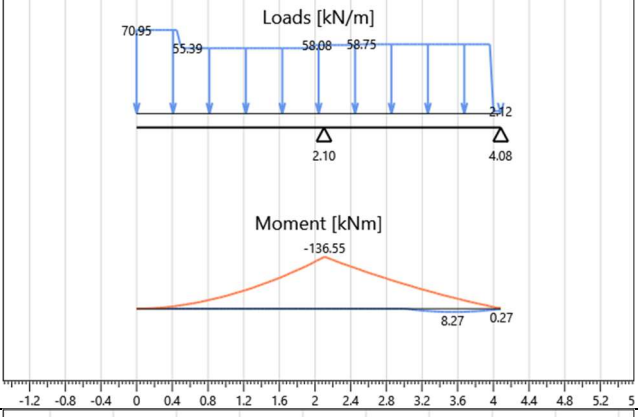
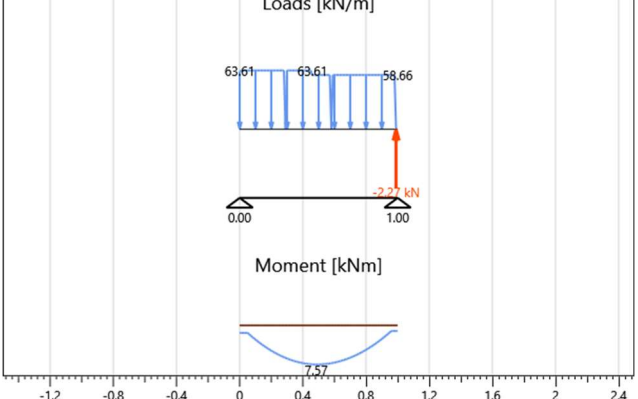


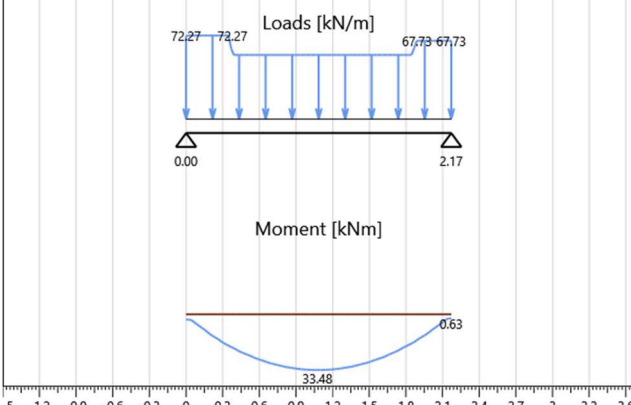
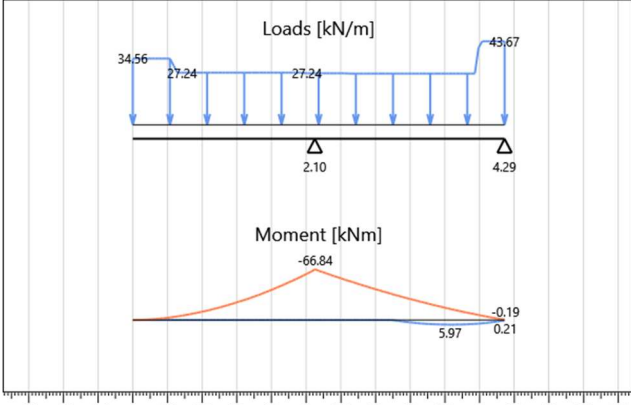
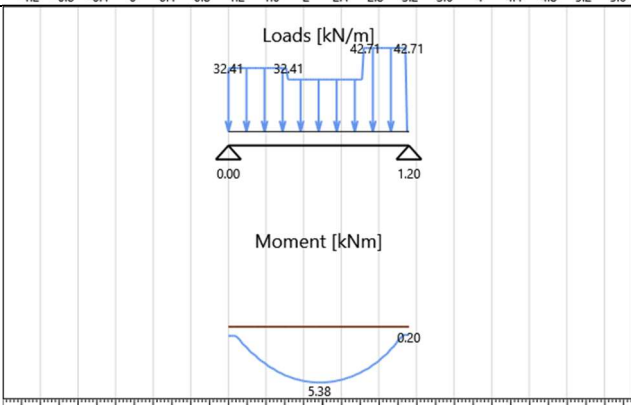
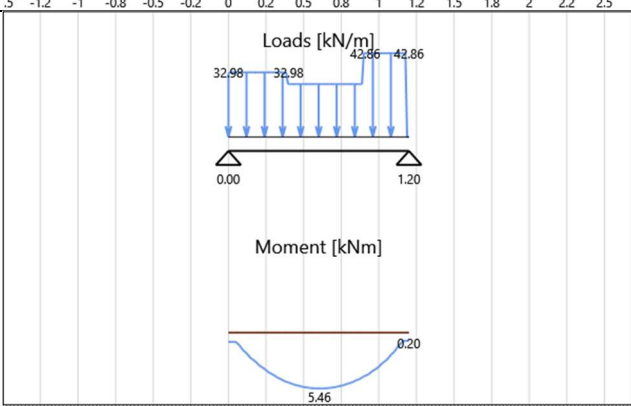
Beam 8	ULS 78	Short-term	
Beam 9	ULS 64	Permanent	
Beam 10	ULS 78	Short-term	
Beam 14	ULS 64	Permanent	

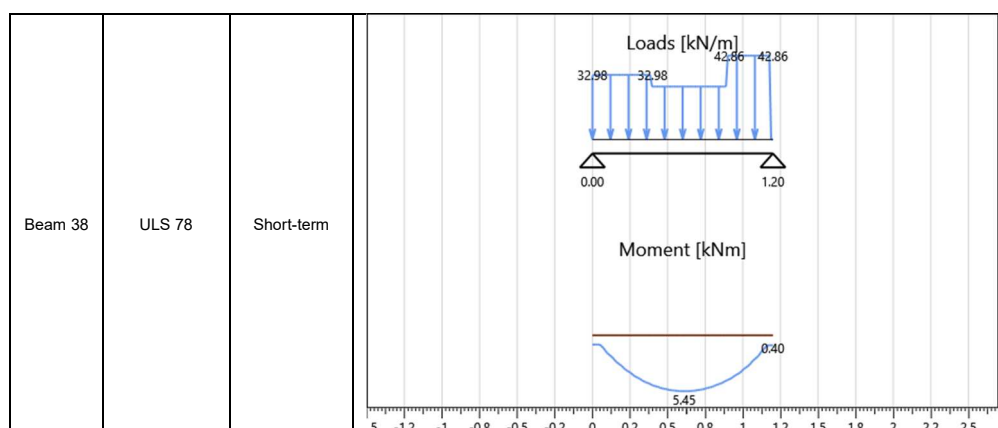
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Beam 16	ULS 64	Permanent	
Beam 17	ULS 65	Medium-term	
Beam 19	ULS 65	Medium-term	

Beam 20	ULS 78	Short-term	
Beam 21	ULS 78	Short-term	
Beam 22	ULS 78	Short-term	
Beam 25	ULS 78	Short-term	

Beam 26	ULS 78	Short-term	
Beam 27	ULS 78	Short-term	
Beam 28	ULS 78	Short-term	
Beam 29	ULS 78	Short-term	

Beam 30	ULS 78	Short-term	
Beam 31	ULS 64	Permanent	
Beam 32	ULS 78	Short-term	
Beam 33	ULS 78	Short-term	

Beam 34	ULS 78	Short-term	 Load diagram (kN/m): 72.27, 72.27, 67.73, 67.73. Span: 0.00 to 2.17. Moment diagram (kNm): 0.63, 33.48.
Beam 35	ULS 78	Short-term	 Load diagram (kN/m): 34.56, 27.24, 27.24, 43.67. Span: 2.10 to 4.29. Moment diagram (kNm): -66.84, -0.19, 5.97, 0.21.
Beam 36	ULS 78	Short-term	 Load diagram (kN/m): 32.41, 32.41, 42.71, 42.71. Span: 0.00 to 1.20. Moment diagram (kNm): 0.20, 5.38.
Beam 37	ULS 78	Short-term	 Load diagram (kN/m): 32.98, 32.98, 42.86, 42.86. Span: 0.00 to 1.20. Moment diagram (kNm): 0.20, 5.46.



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Beam name	Section	M _{3-3 max} [kNm]	W [mm ³]	Lateral restraints	σ _{m,crit} [MPa]	k _{crit}	Comb.	k _h	k _{mod}	γ _M	f _{m,d} [MPa]	σ _{m,d} [MPa]	Check
Beam 1	200x440 GL24h	78.28	6453333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	12.13	70%
Beam 2	200x440 GL24h	78.32	6453333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	12.14	70%
Beam 3	200x440 GL24h	67.22	6453333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	10.42	60%
Beam 4	200x200 GL24h	7.28	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	5.46	32%
Beam 5	200x200 GL24h	7.50	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	5.63	33%
Beam 6	200x200 GL24h	6.80	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	5.10	30%
Beam 7	200x200 GL24h	6.79	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	5.09	29%
Beam 8	200x200 GL24h	8.97	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	6.73	39%
Beam 9	200x200 GL24h	0.66	1333333	No torsional buckling	-	1.00	ULS 64	1.00	0.6	1.25	11.52	0.50	4%
Beam 10	200x240 GL24h	31.19	1920000	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	16.24	94%
Beam 14	140x80	0.19	149333	No torsional buckling	-	1.00	ULS 64	1.00	0.6	1.3	11.08	1.25	11%
Beam 15	140x80	0.19	149333	No torsional buckling	-	1.00	ULS 64	1.00	0.6	1.3	11.08	1.25	11%
Beam 16	200x200 GL24h	6.70	1333333	No torsional buckling	-	1.00	ULS 64	1.00	0.6	1.25	11.52	5.03	44%
Beam 17	260x280 GL28h	52.45	3397333	No torsional buckling	-	1.00	ULS 65	1.00	0.8	1.25	17.92	15.44	86%
Beam 19	260x280 GL28h	51.84	3397333	No torsional buckling	-	1.00	ULS 65	1.00	0.8	1.25	17.92	15.26	85%
Beam 20	200x400 GL28h	94.71	5333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	20.16	17.76	88%
Beam 21	200x400 GL28h	65.38	5333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	20.16	12.26	61%
Beam 22	200x400 GL28h	66.74	5333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	20.16	12.51	62%
Beam 25	200x400 GL28h	30.19	5333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	20.16	5.66	28%
Beam 26	200x240 GL24h	15.34	1920000	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	7.99	46%
Beam 27	200x240 GL24h	25.50	1920000	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	13.28	77%
Beam 28	200x200 GL24h	4.69	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	3.52	20%
Beam 29	200x200 GL24h	5.04	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	3.78	22%
Beam 30	200x240 GL24h	13.98	1920000	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	7.28	42%
Beam 31	200x100	2.12	333333	No torsional buckling	-	1.00	ULS 64	1.00	0.6	1.3	11.08	6.36	57%
Beam 32	200x520 GL24h	136.55	9013333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	15.15	88%
Beam 33	200x200 GL24h	7.57	1333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	5.68	33%
Beam 34	200x280 GL24h	33.48	2613333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	12.81	74%
Beam 35	200x520 GL24h	66.84	9013333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.25	17.28	7.42	43%
Beam 36	200x100	5.38	333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.3	16.62	16.13	97%
Beam 37	200x100	5.46	333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.3	16.62	16.37	99%
Beam 38	200x100	5.45	333333	No torsional buckling	-	1.00	ULS 78	1.00	0.9	1.3	16.62	16.36	98%

Shear strength

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

where:

τ_d is the design shear stress;

$f_{v,d}$ is the design shear strength for the actual condition.

For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member given as:

$$b_{ef} = k_{cr} \cdot b$$

where b is the width of the relevant section where the maximum shear stress is.

The following value of k_{cr} are used

$k_{cr} = 0.67 (\leq 1)$ for solid timber

$k_{cr} = 0.83 (\leq 1)$ for glued laminated timber

The maximum design shear stress in a generic cross section can be evaluated using the following expression:

$$\tau_d = \frac{V_d \cdot S}{k_{cr} \cdot b \cdot J_{y-y}}$$

V_d is the design shear force;

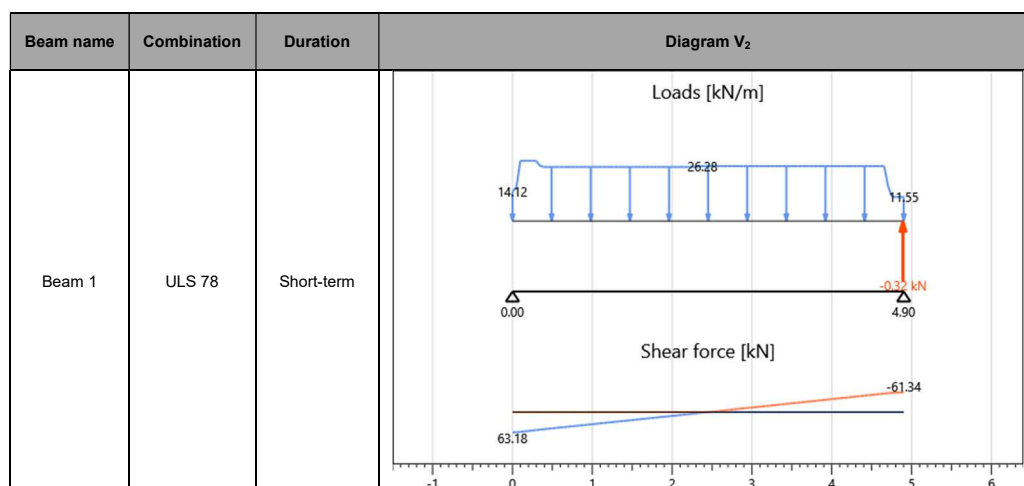
S is the static moment of the cross section cut by the considered chord b ;

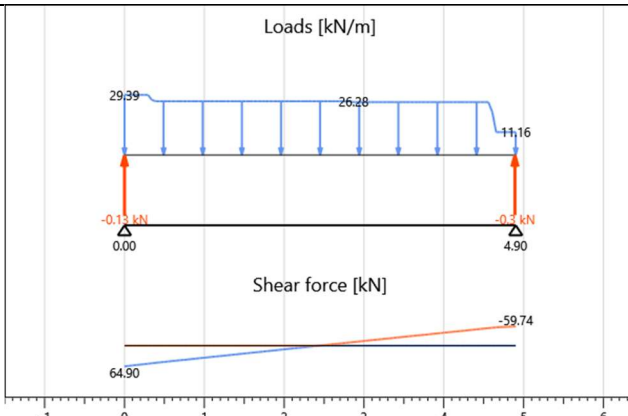
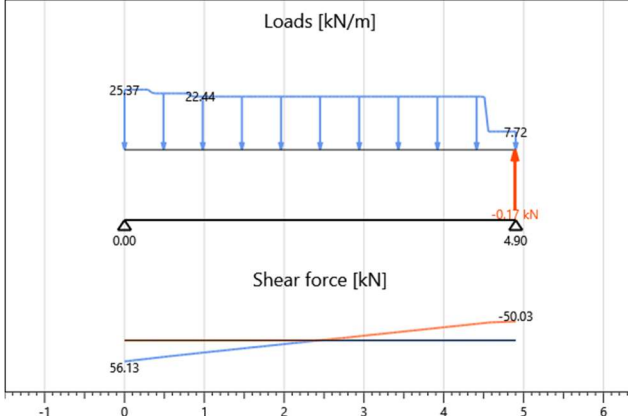
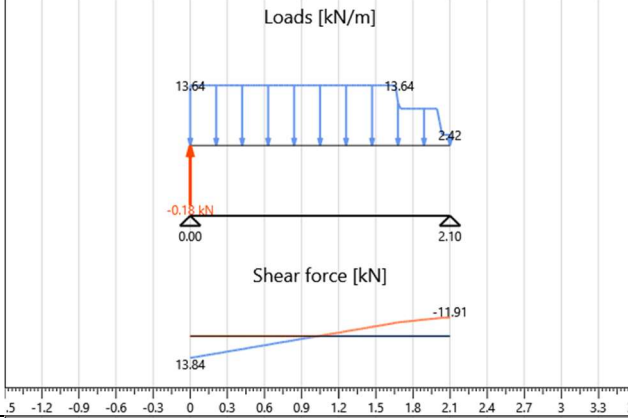
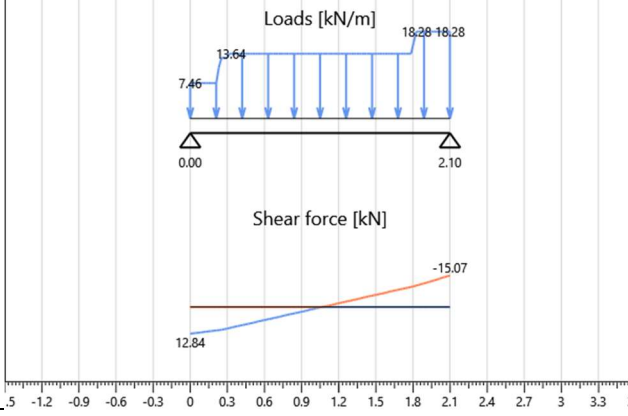
k_{cr} is the coefficient that takes into account the influence of cracks;

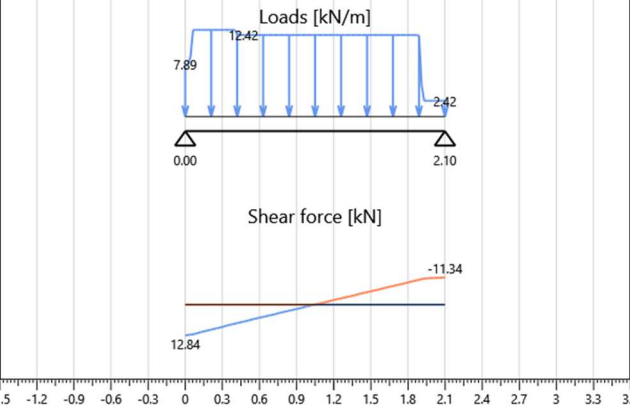
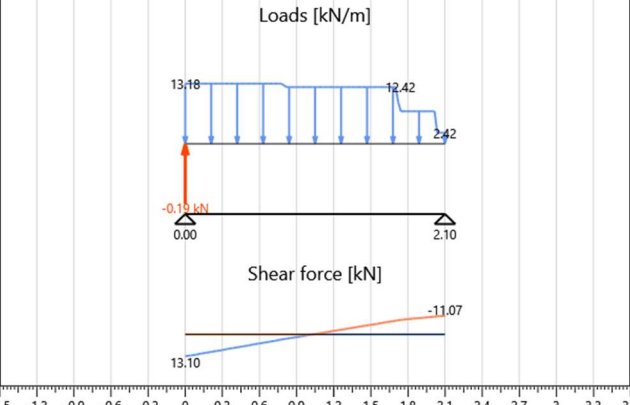
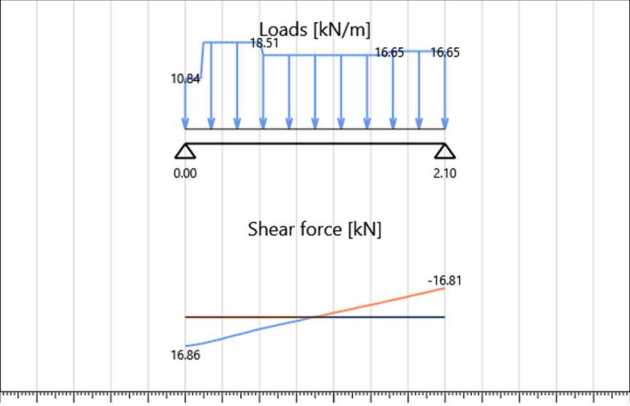
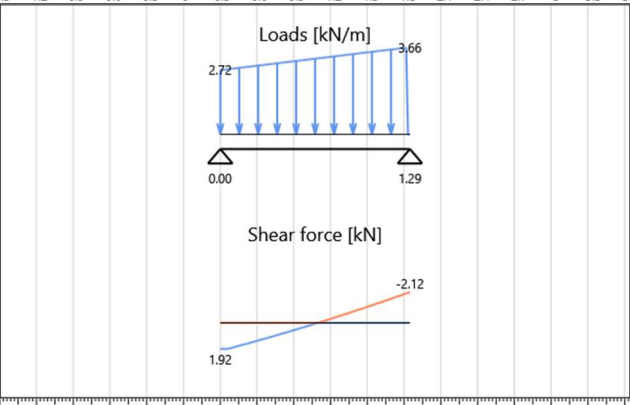
b is the chord where the shear stress is evaluated;

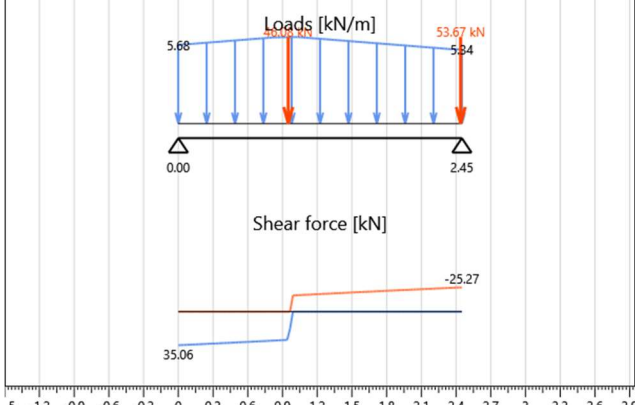
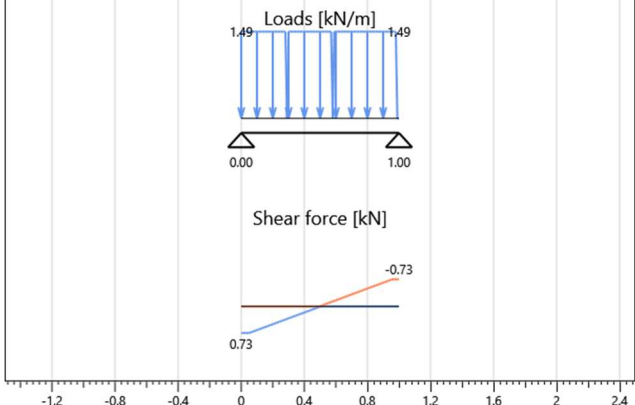
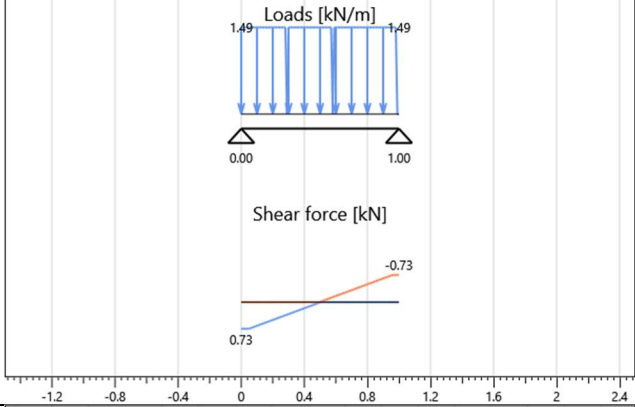
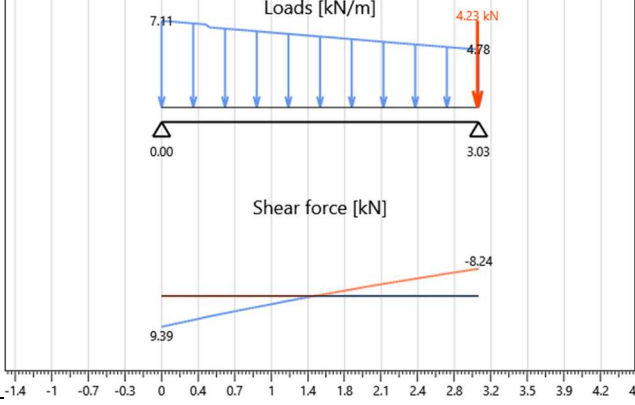
J_{y-y} is the moment of inertia with respect to the y axis.

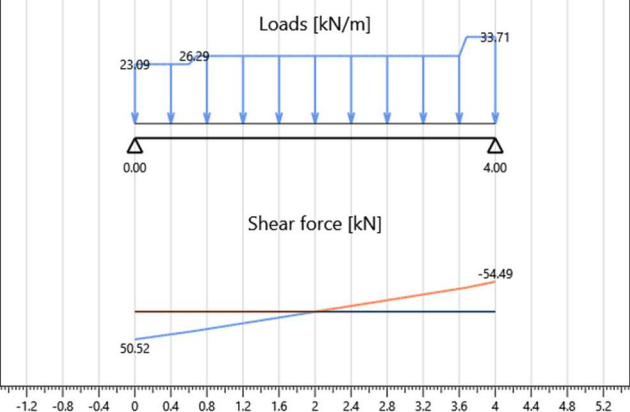
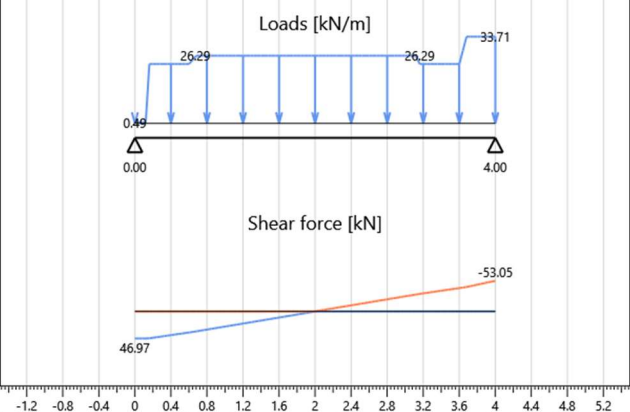
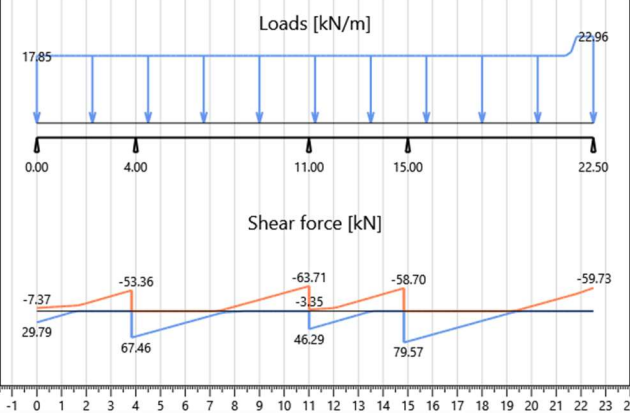
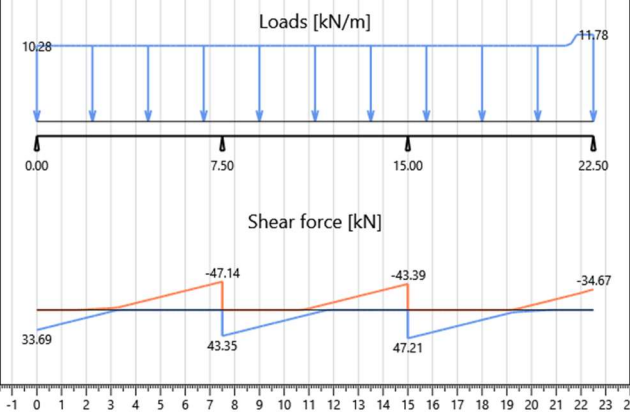
The following table illustrates the structural schemes and the envelopes of the shear force diagram for each beam.

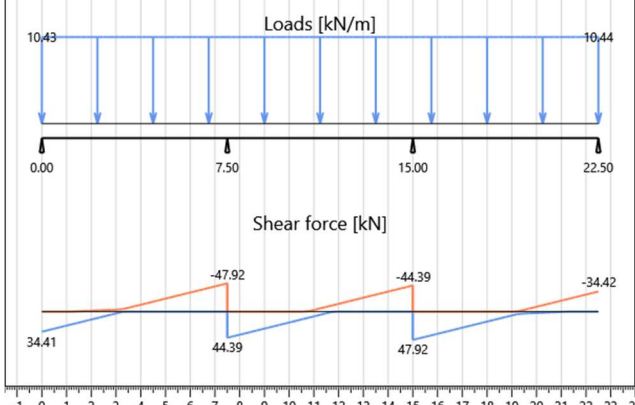
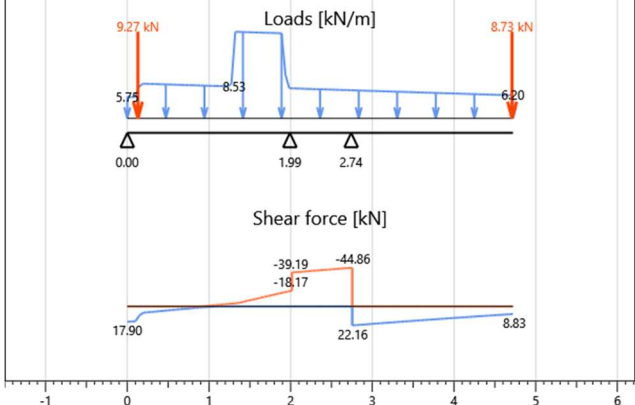
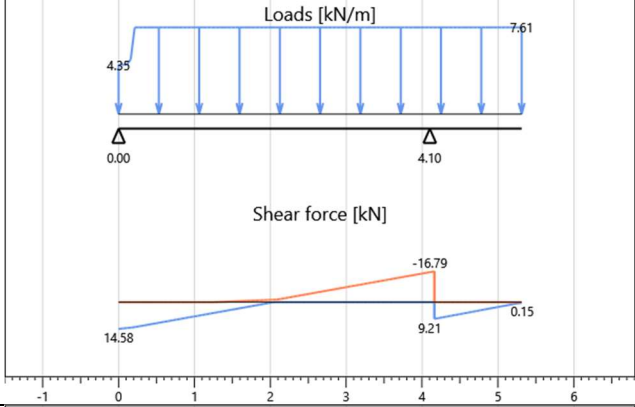
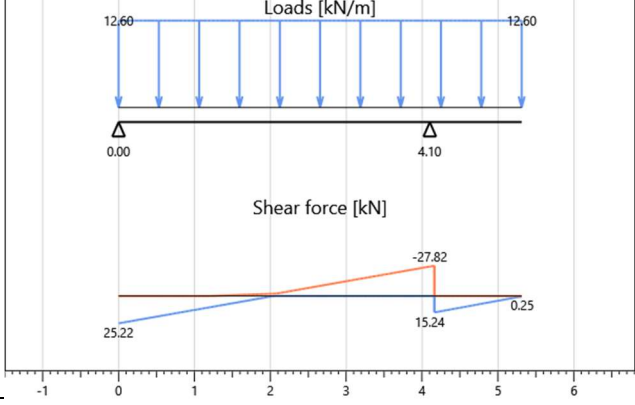


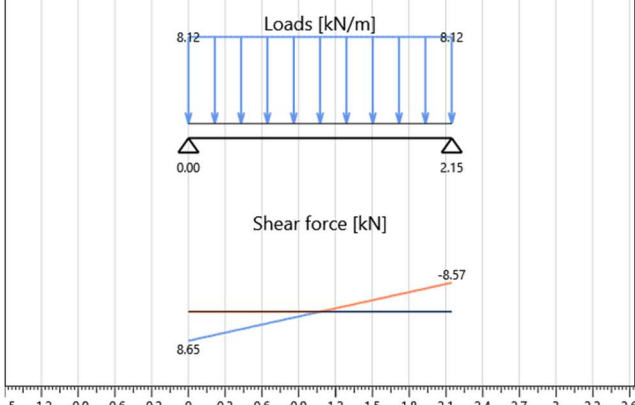
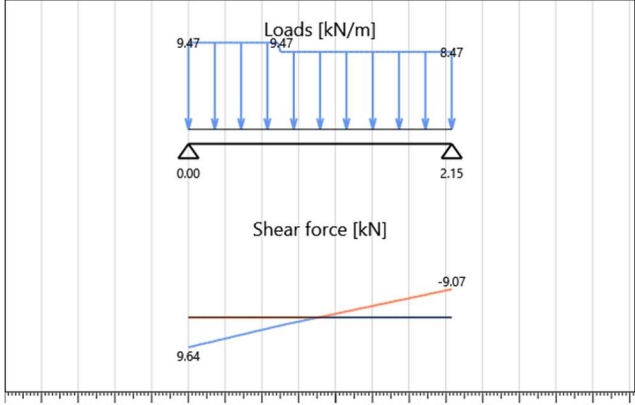
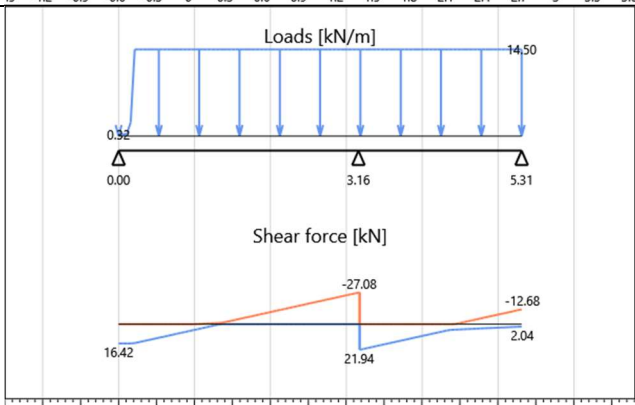
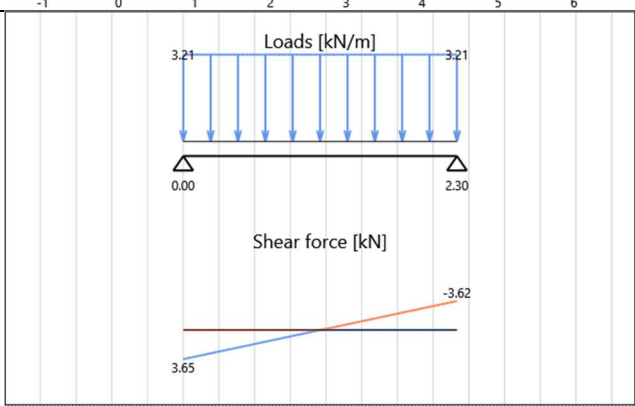
Beam 2	ULS 78	Short-term	
Beam 3	ULS 78	Short-term	
Beam 4	ULS 78	Short-term	
Beam 5	ULS 78	Short-term	

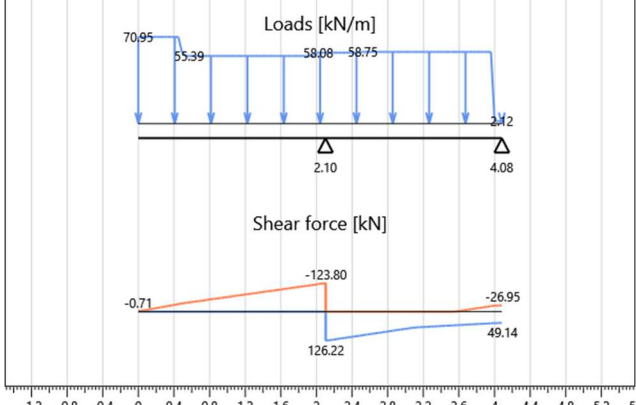
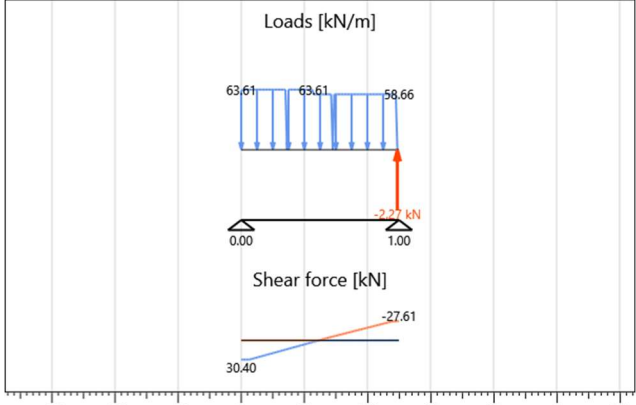
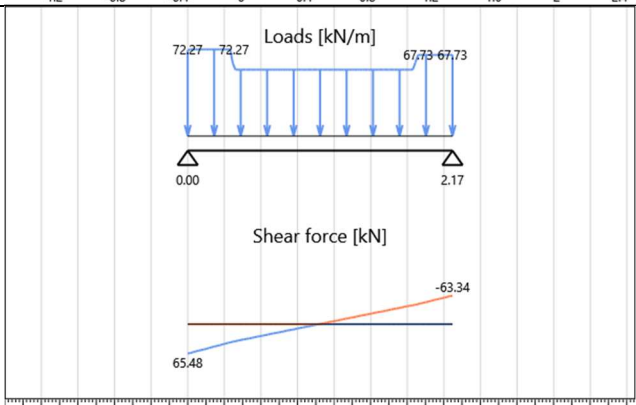
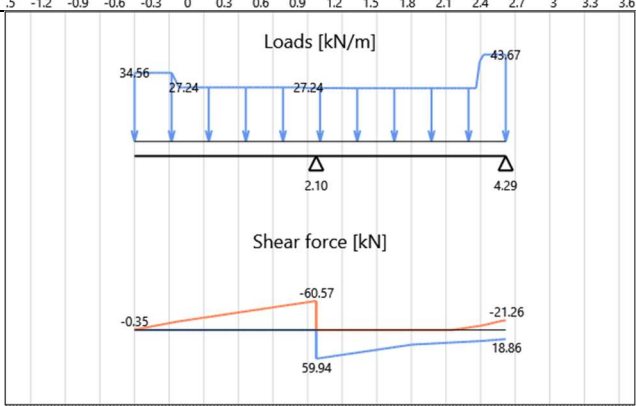
Beam 6	ULS 78	Short-term	
Beam 7	ULS 78	Short-term	
Beam 8	ULS 78	Short-term	
Beam 9	ULS 64	Permanent	

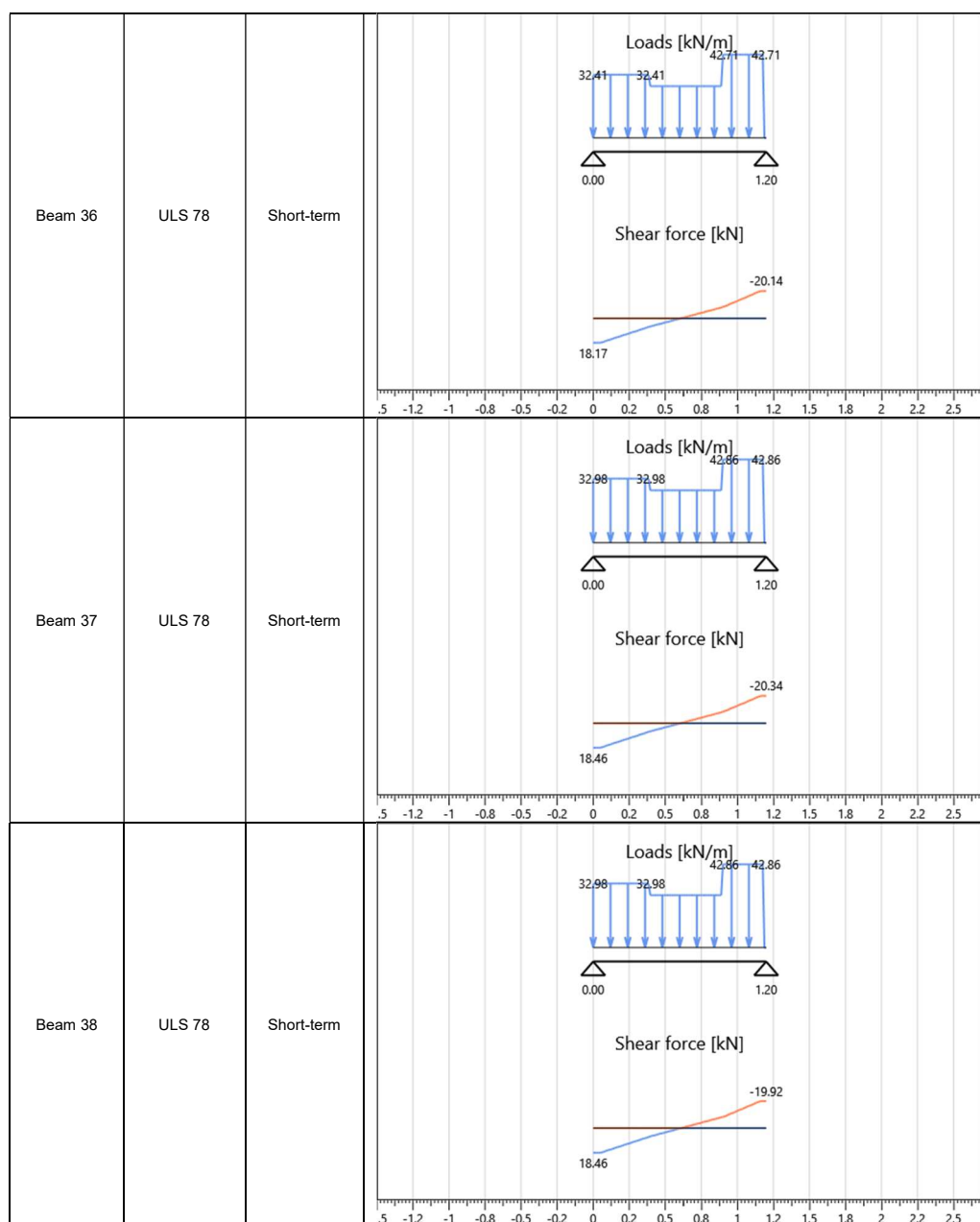
Beam 10	ULS 78	Short-term	
Beam 14	ULS 64	Permanent	
Beam 15	ULS 64	Permanent	
Beam 16	ULS 64	Permanent	

Beam 17	ULS 65	Medium-term	
Beam 19	ULS 65	Medium-term	
Beam 20	ULS 78	Short-term	
Beam 21	ULS 78	Short-term	

Beam 22	ULS 78	Short-term	 Load and Shear force diagrams for Beam 22. The load diagram shows a uniform distributed load of 10.43 kN/m over a 22.50 m span. The shear force diagram shows a linear decrease from 34.41 kN at the left support to -34.42 kN at the right support, with a zero-crossing at 11.25 m. Key values: 10.43, 0.00, 7.50, 15.00, 22.50, 34.41, -47.92, 44.39, -44.39, 47.92, -34.42.
Beam 25	ULS 78	Short-term	 Load and Shear force diagrams for Beam 25. The load diagram shows a uniform distributed load of 5.75 kN/m with point loads of 9.27 kN and 8.73 kN at 0.00 m and 6.20 m respectively. The shear force diagram shows a linear decrease from 17.90 kN to -8.83 kN, with a zero-crossing at 3.45 m. Key values: 9.27 kN, 5.75, 8.53, 1.99, 2.74, 6.20, 8.73 kN, 17.90, -39.19, -18.17, -44.86, 22.16, 8.83.
Beam 26	ULS 78	Short-term	 Load and Shear force diagrams for Beam 26. The load diagram shows a uniform distributed load of 4.35 kN/m over a 4.10 m span. The shear force diagram shows a linear decrease from 14.58 kN to -0.15 kN, with a zero-crossing at 2.05 m. Key values: 4.35, 0.00, 4.10, 7.61, 14.58, -16.79, 9.21, 0.15.
Beam 27	ULS 78	Short-term	 Load and Shear force diagrams for Beam 27. The load diagram shows a uniform distributed load of 12.60 kN/m over a 4.10 m span. The shear force diagram shows a linear decrease from 25.22 kN to -0.25 kN, with a zero-crossing at 2.05 m. Key values: 12.60, 0.00, 4.10, 12.60, 25.22, -27.82, 15.24, 0.25.

Beam 28	ULS 78	Short-term	
Beam 29	ULS 78	Short-term	
Beam 30	ULS 78	Short-term	
Beam 31	ULS 64	Permanent	

Beam 32	ULS 78	Short-term	
Beam 33	ULS 78	Short-term	
Beam 34	ULS 78	Short-term	
Beam 35	ULS 78	Short-term	



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Beam name	Section	V _{2,max} [kN]	b [mm]	S [mm ³]	k _{cr}	Comb.	Service class	k _{mod}	γ _M	f _{v,d} [MPa]	τ _{2,d} [MPa]	Check
Beam 1	200x440 GL24h	63.18	200.00	4840000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.30	51%
Beam 2	200x440 GL24h	64.90	200.00	4840000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.33	53%
Beam 3	200x440 GL24h	56.13	200.00	4840000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.15	46%
Beam 4	200x200 GL24h	13.84	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.63	25%
Beam 5	200x200 GL24h	15.07	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.68	27%
Beam 6	200x200 GL24h	12.84	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.58	23%
Beam 7	200x200 GL24h	13.10	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.59	23%
Beam 8	200x200 GL24h	16.86	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.76	30%
Beam 9	200x200 GL24h	2.12	200.00	1000000.00	0.83	ULS 64	1	0.6	1.25	1.68	0.10	6%
Beam 10	200x240 GL24h	35.06	200.00	1440000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.32	52%
Beam 14	140x80	0.73	140.00	112000.00	0.67	ULS 64	1	0.6	1.3	1.85	0.15	8%
Beam 15	140x80	0.73	140.00	112000.00	0.67	ULS 64	1	0.6	1.3	1.85	0.15	8%
Beam 16	200x200 GL24h	9.39	200.00	1000000.00	0.83	ULS 64	1	0.6	1.25	1.68	0.42	25%
Beam 17	260x280 GL28h	54.49	260.00	2548000.00	0.83	ULS 65	1	0.8	1.25	2.24	1.35	60%
Beam 19	260x280 GL28h	53.05	260.00	2548000.00	0.83	ULS 65	1	0.8	1.25	2.24	1.32	59%
Beam 20	200x400 GL28h	79.57	200.00	4000000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.80	71%
Beam 21	200x400 GL28h	47.21	200.00	4000000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.07	42%
Beam 22	200x400 GL28h	47.92	200.00	4000000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.08	43%
Beam 25	200x400 GL28h	44.86	200.00	4000000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.01	40%
Beam 26	200x240 GL24h	16.79	200.00	1440000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.63	25%
Beam 27	200x240 GL24h	27.82	200.00	1440000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.05	42%
Beam 28	200x200 GL24h	8.65	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.39	16%
Beam 29	200x200 GL24h	9.64	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	0.44	17%
Beam 30	200x240 GL24h	27.08	200.00	1440000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.02	40%
Beam 31	200x100	3.65	200.00	250000.00	0.67	ULS 64	1	0.6	1.3	1.85	0.41	22%
Beam 32	200x520 GL24h	126.22	200.00	6760000.00	0.83	ULS 78	2	0.9	1.25	2.52	2.19	87%
Beam 33	200x200 GL24h	30.40	200.00	1000000.00	0.83	ULS 78	1	0.9	1.25	2.52	1.37	55%
Beam 34	200x280 GL24h	65.48	200.00	1960000.00	0.83	ULS 78	1	0.9	1.25	2.52	2.11	84%
Beam 35	200x520 GL24h	60.57	200.00	6760000.00	0.83	ULS 78	2	0.9	1.25	2.52	1.05	42%
Beam 36	200x100	20.14	200.00	250000.00	0.67	ULS 78	1	0.9	1.3	2.77	2.25	81%
Beam 37	200x100	20.34	200.00	250000.00	0.67	ULS 78	1	0.9	1.3	2.77	2.28	82%
Beam 38	200x100	19.92	200.00	250000.00	0.67	ULS 78	1	0.9	1.3	2.77	2.23	81%

Beams deflections (SLS)

The deflection checks are carried out according to § 2.2.3 of EN 1995-1-1.

The net deflection below a straight line between the supports, $w_{net,fin}$, is taken as:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c$$

where:

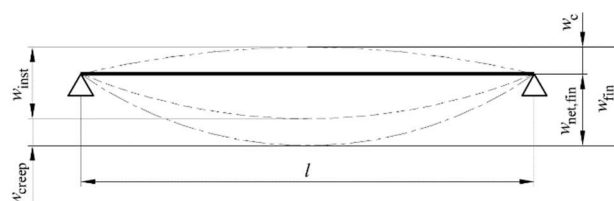
$w_{net,fin}$ is the net final deflection;

w_{inst} is the instantaneous deflection;

w_{creep} is the creep deflection;

w_c is the precamber (if applied);

w_{fin} is the final deflection.



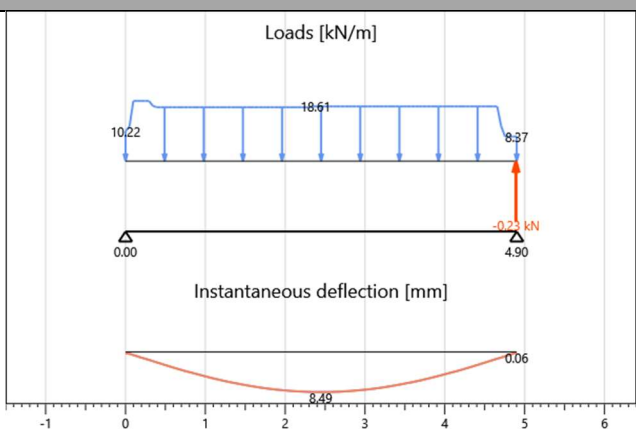
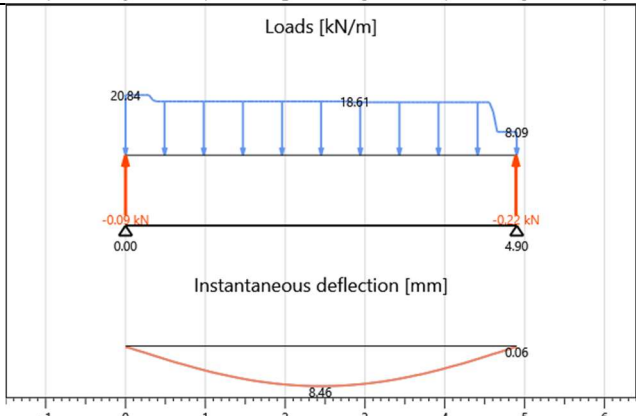
The limiting values for deflections of beams are assumed as shown in the following table.

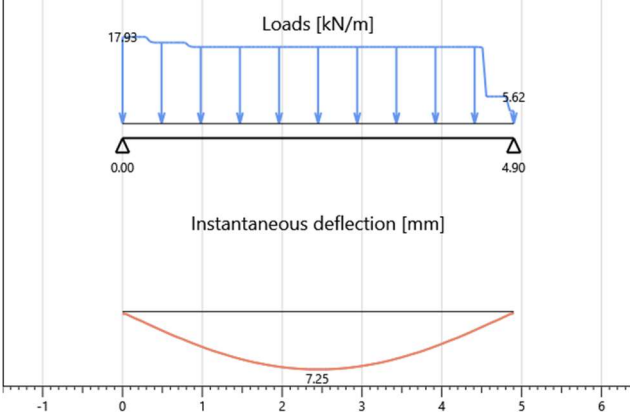
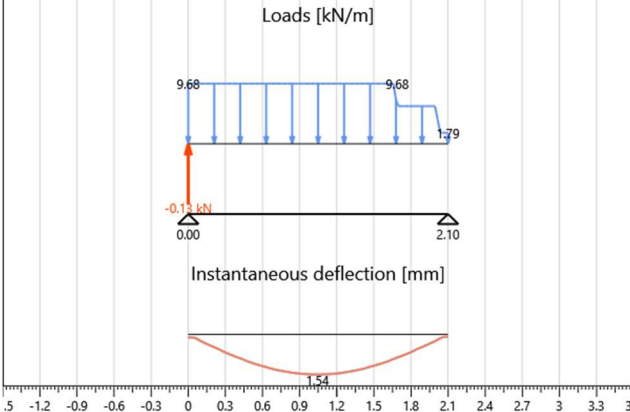
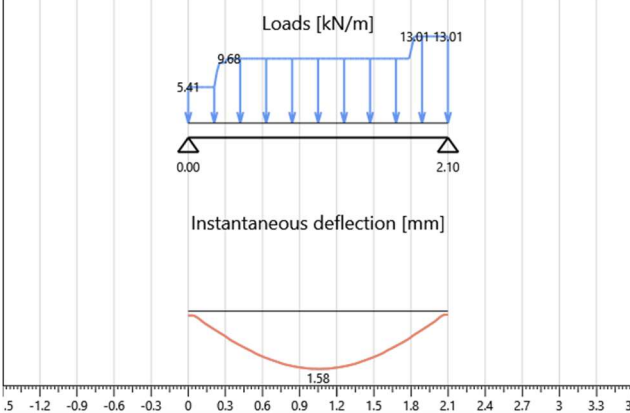
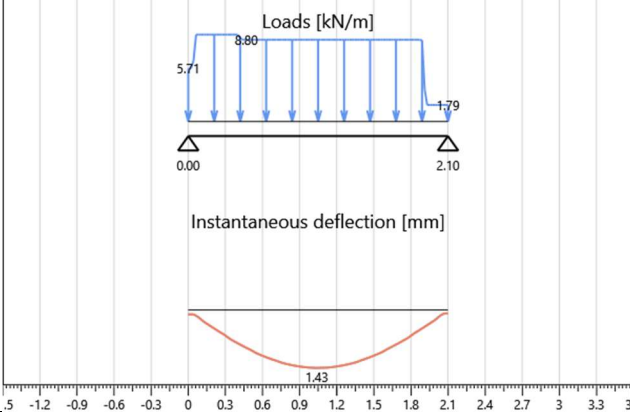
Deformation limits name	$w_{inst, span}$	$w_{net, fin, span}$	$w_{inst, overhang}$	$w_{net, fin, overhang}$	Neglect overhang check for deformation < 0
Timber beam	$l/300$	$l/250$	$l/150$	$l/125$	Yes

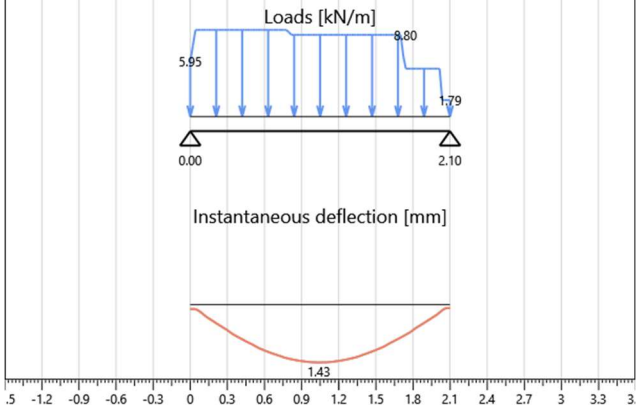
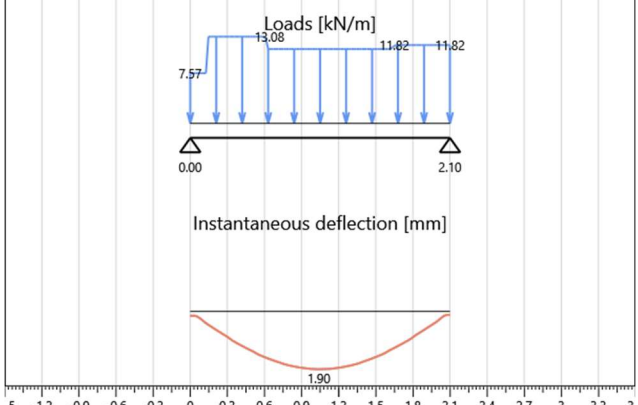
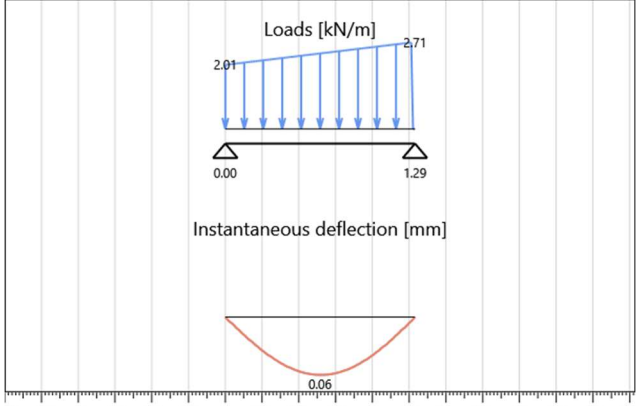
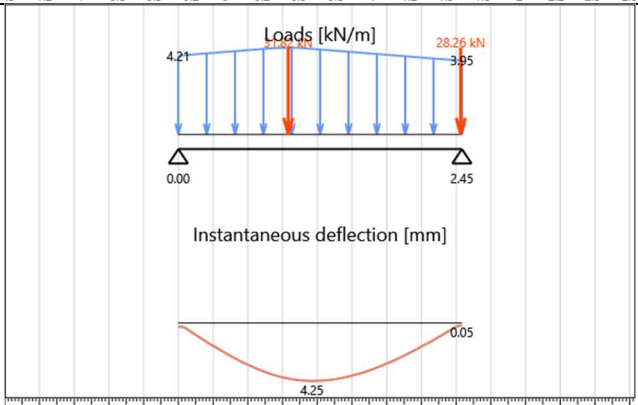
Instantaneous deflection

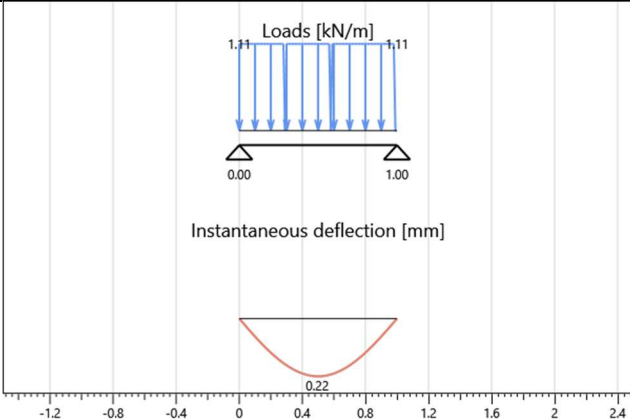
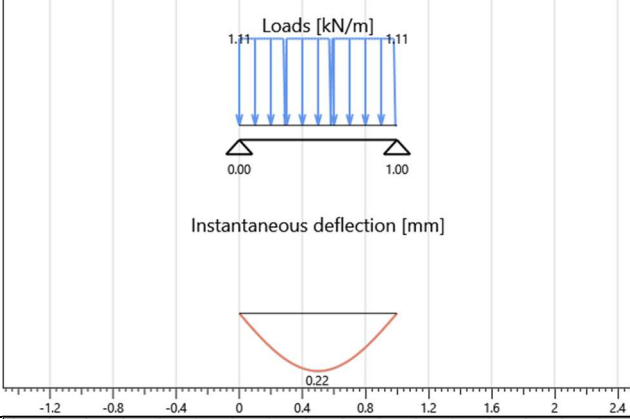
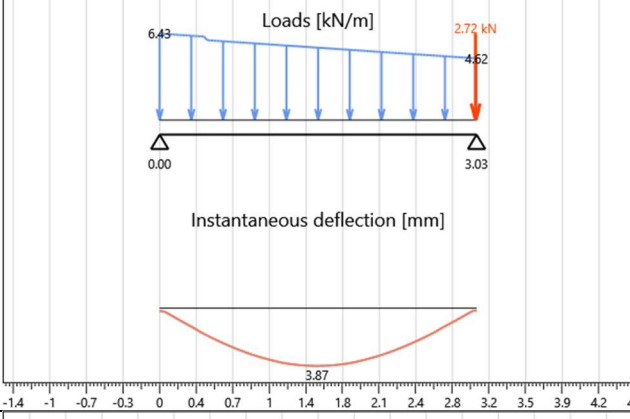
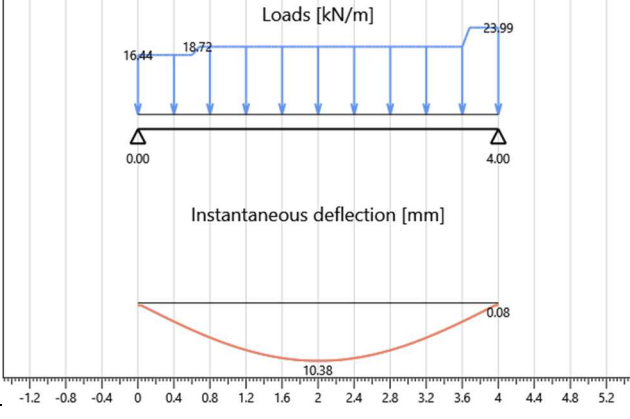
The instantaneous deflection w_{inst} is calculated for the characteristic (rare) combination of actions.

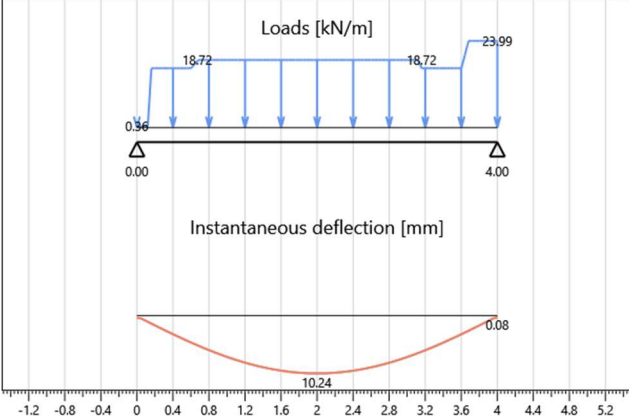
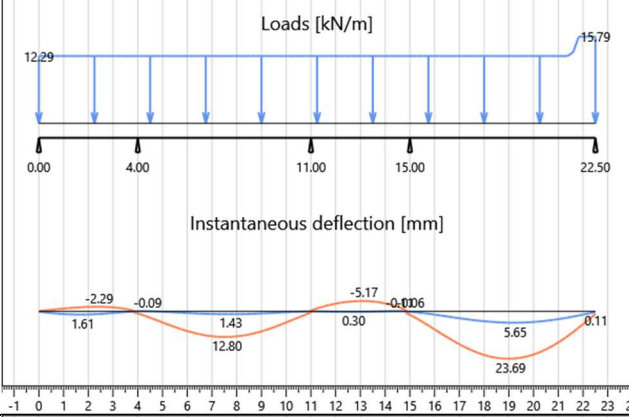
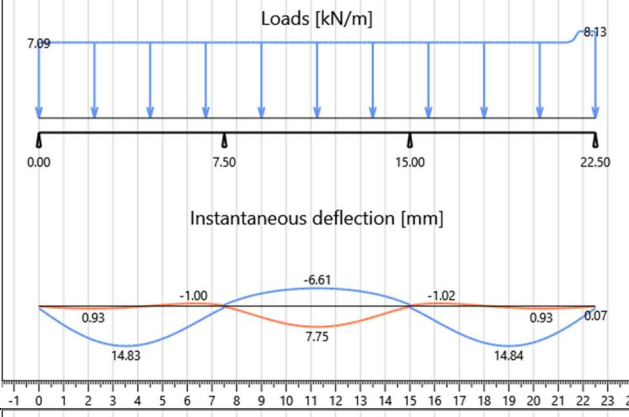
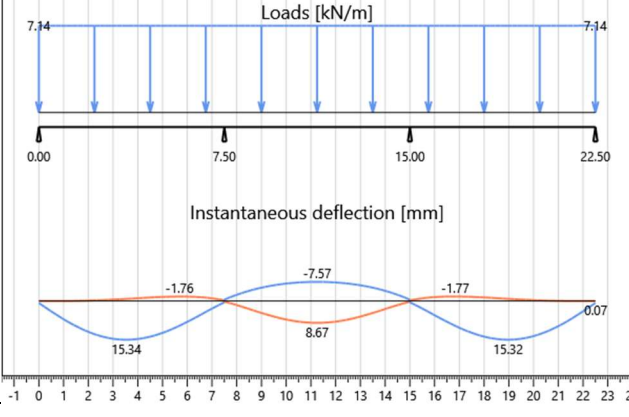
The following table shows the deformation of each beam.

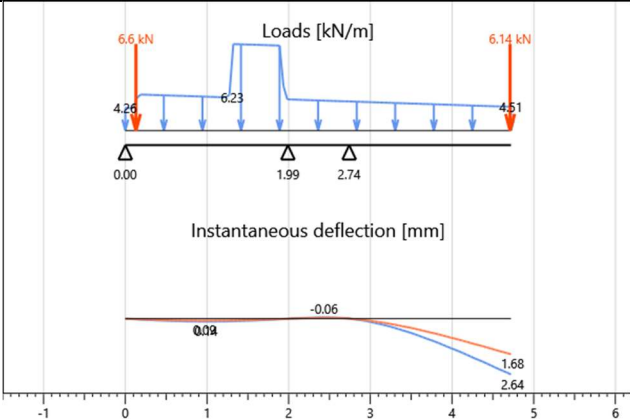
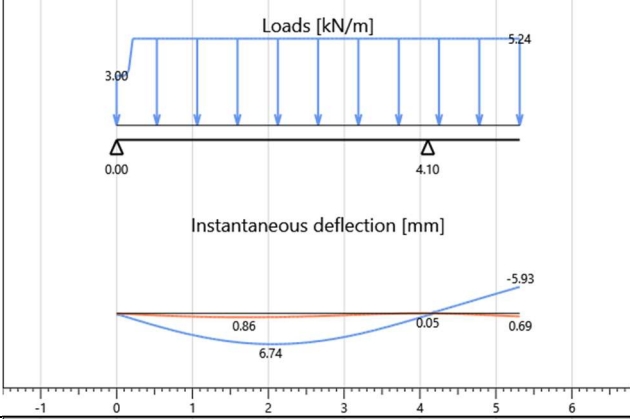
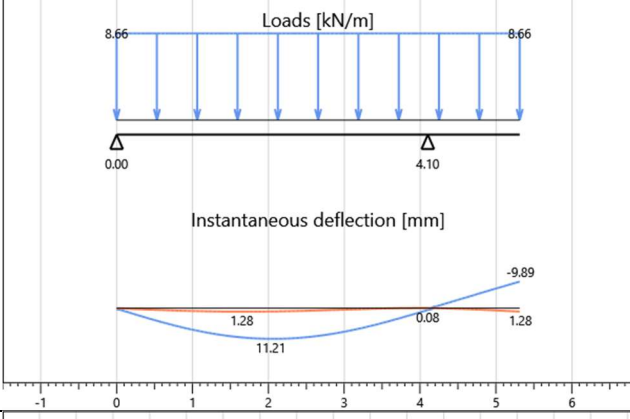
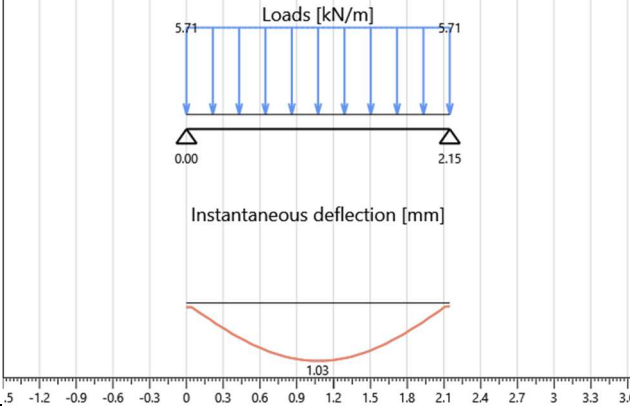
Beam name	Deformation limits name	Combination	Instantaneous deflection
Beam 1	Timber beam	SLS characteristic 15	
Beam 2	Timber beam	SLS characteristic 15	

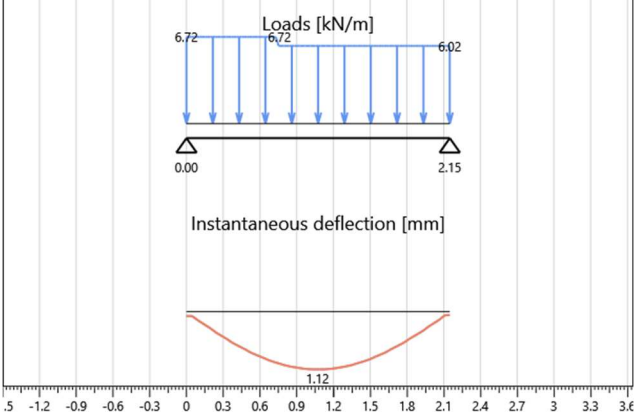
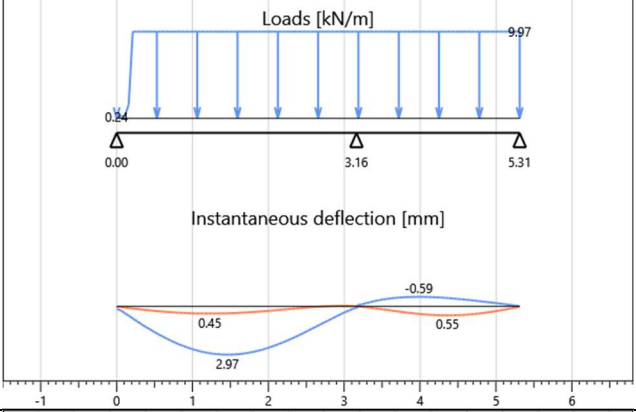
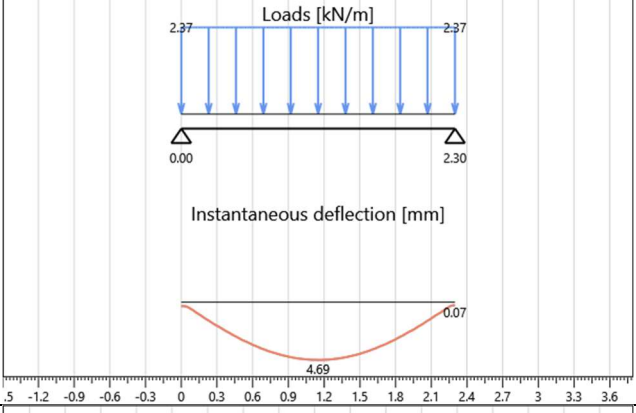
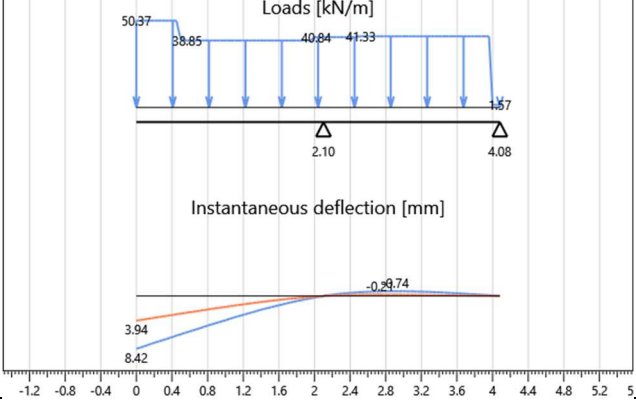
Beam 3	Timber beam	SLS characteristic 15	
Beam 4	Timber beam	SLS characteristic 15	
Beam 5	Timber beam	SLS characteristic 15	
Beam 6	Timber beam	SLS characteristic 15	

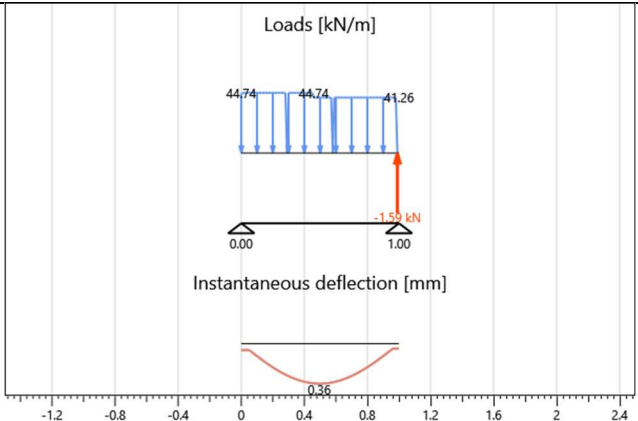
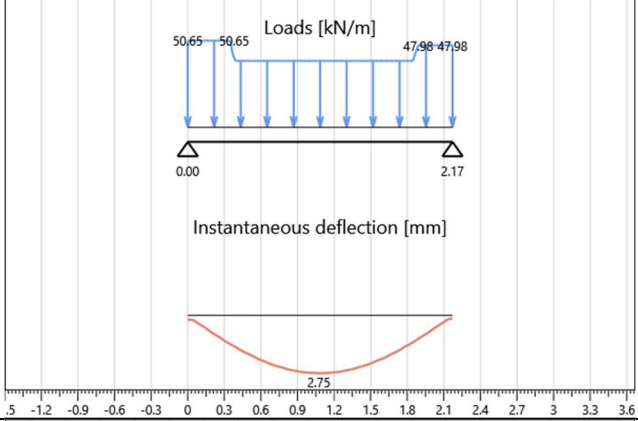
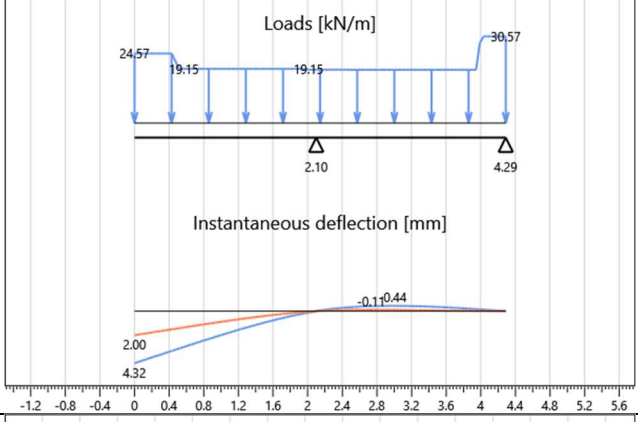
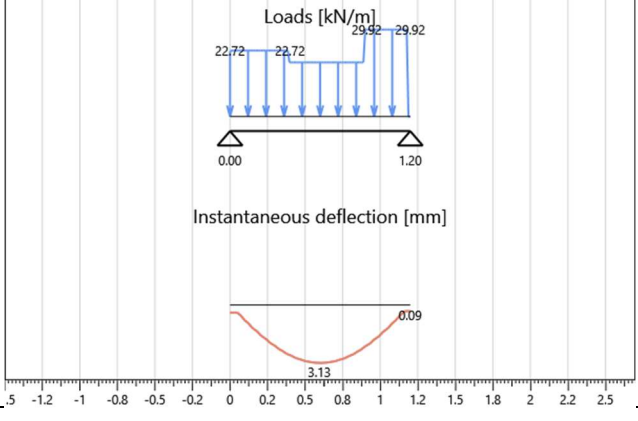
Beam 7	Timber beam	SLS characteristic 15	
Beam 8	Timber beam	SLS characteristic 15	
Beam 9	Timber beam	SLS characteristic 4	
Beam 10	Timber beam	SLS characteristic 14	

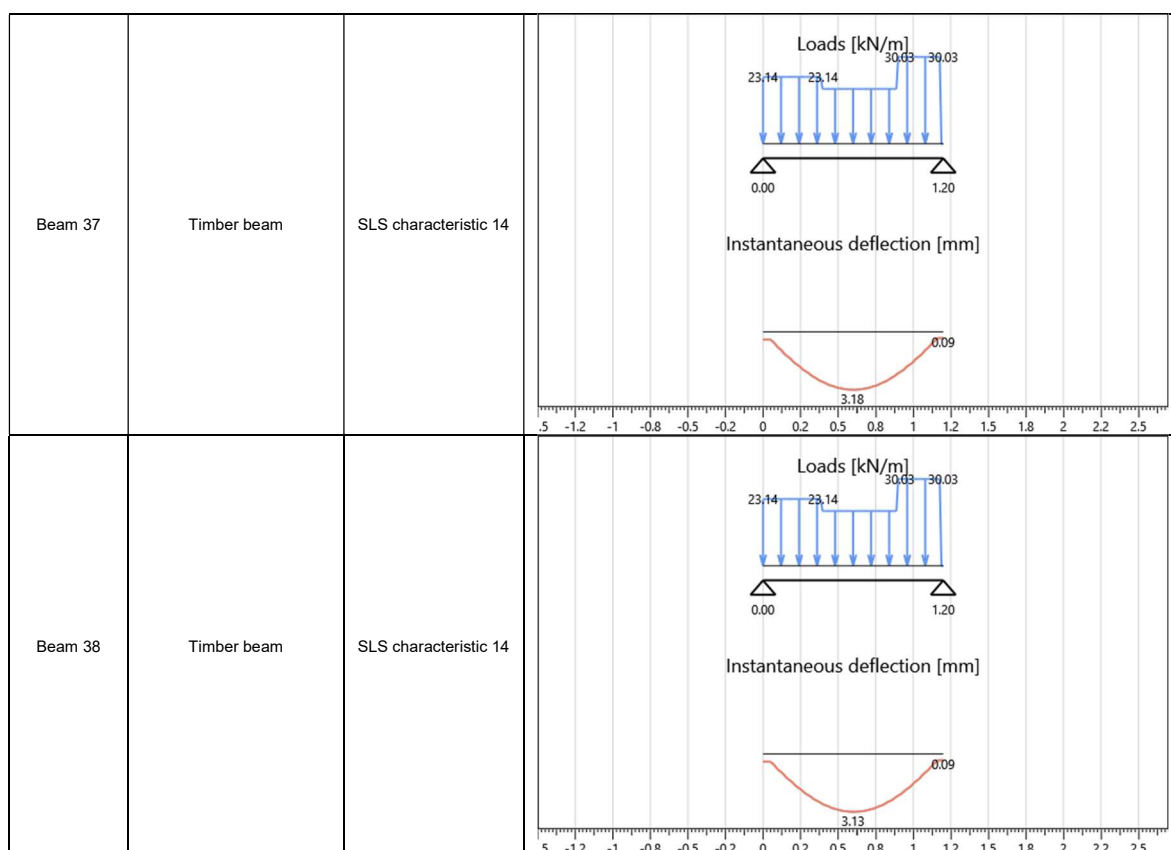
Beam 14	Timber beam	SLS characteristic 4	 Loads [kN/m] Instantaneous deflection [mm]
Beam 15	Timber beam	SLS characteristic 4	 Loads [kN/m] Instantaneous deflection [mm]
Beam 16	Timber beam	SLS characteristic 4	 Loads [kN/m] Instantaneous deflection [mm]
Beam 17	Timber beam	SLS characteristic 4	 Loads [kN/m] Instantaneous deflection [mm]

Beam 19	Timber beam	SLS characteristic 4	 Loads [kN/m] Instantaneous deflection [mm]
Beam 20	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 21	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 22	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]

Beam 25	Timber beam	SLS characteristic 15	 Diagram of Beam 25 showing loads and instantaneous deflection. The beam is supported at 0.00, 1.99, and 2.74. The load distribution is as follows: 6.6 kN point load at 0.00, 4.26 kN/m UDL from 0.00 to 1.99, 6.23 kN/m UDL from 1.99 to 2.74, and 6.14 kN point load at 2.74. The instantaneous deflection curve shows values of 0.02, -0.06, 1.68, and 2.64 mm at the respective points.
Beam 26	Timber beam	SLS characteristic 14	 Diagram of Beam 26 showing loads and instantaneous deflection. The beam is supported at 0.00 and 4.10. The load is a uniform distributed load (UDL) of 3.00 kN/m from 0.00 to 4.10. The instantaneous deflection curve shows values of 0.86, 6.74, 0.05, and -5.93 mm at the respective points.
Beam 27	Timber beam	SLS characteristic 14	 Diagram of Beam 27 showing loads and instantaneous deflection. The beam is supported at 0.00 and 4.10. The load is a uniform distributed load (UDL) of 8.66 kN/m from 0.00 to 4.10. The instantaneous deflection curve shows values of 1.28, 11.21, 0.08, and -9.89 mm at the respective points.
Beam 28	Timber beam	SLS characteristic 14	 Diagram of Beam 28 showing loads and instantaneous deflection. The beam is supported at 0.00 and 2.15. The load is a uniform distributed load (UDL) of 5.71 kN/m from 0.00 to 2.15. The instantaneous deflection curve shows a value of 1.03 mm at the midpoint.

Beam 29	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 30	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 31	Timber beam	SLS characteristic 4	 Loads [kN/m] Instantaneous deflection [mm]
Beam 32	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]

Beam 33	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 34	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 35	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]
Beam 36	Timber beam	SLS characteristic 14	 Loads [kN/m] Instantaneous deflection [mm]



The table below shows the instantaneous deflection checks of the beams.

Beam name	Section	Combination	Most restrictive check	W_{inst} [mm]	W_{inst} limit [mm]	Deflection limit	Check
Beam 1	200x440 GL24h	SLS characteristic 15	Internal span	8.49	16.33	$l/300$	52%
Beam 2	200x440 GL24h	SLS characteristic 15	Internal span	8.46	16.33	$l/300$	52%
Beam 3	200x440 GL24h	SLS characteristic 15	Internal span	7.25	16.33	$l/300$	44%
Beam 4	200x200 GL24h	SLS characteristic 15	Internal span	1.54	7.00	$l/300$	22%
Beam 5	200x200 GL24h	SLS characteristic 15	Internal span	1.58	7.00	$l/300$	23%
Beam 6	200x200 GL24h	SLS characteristic 15	Internal span	1.43	7.00	$l/300$	20%
Beam 7	200x200 GL24h	SLS characteristic 15	Internal span	1.43	7.00	$l/300$	20%
Beam 8	200x200 GL24h	SLS characteristic 15	Internal span	1.90	7.00	$l/300$	27%
Beam 9	200x200 GL24h	SLS characteristic 4	Internal span	0.06	4.30	$l/300$	1%
Beam 10	200x240 GL24h	SLS characteristic 14	Internal span	4.25	8.17	$l/300$	52%
Beam 14	140x80	SLS characteristic 4	Internal span	0.22	3.33	$l/300$	7%
Beam 15	140x80	SLS characteristic 4	Internal span	0.22	3.33	$l/300$	7%
Beam 16	200x200 GL24h	SLS characteristic 4	Internal span	3.87	10.10	$l/300$	38%
Beam 17	260x280 GL28h	SLS characteristic 4	Internal span	10.38	13.33	$l/300$	78%
Beam 19	260x280 GL28h	SLS characteristic 4	Internal span	10.24	13.33	$l/300$	77%
Beam 20	200x400 GL28h	SLS characteristic 14	Internal span	23.69	25.00	$l/300$	95%
Beam 21	200x400 GL28h	SLS characteristic 14	Internal span	14.84	25.00	$l/300$	59%
Beam 22	200x400 GL28h	SLS characteristic 14	Internal span	15.34	25.00	$l/300$	61%
Beam 25	200x400 GL28h	SLS characteristic 15	Cantilever beams	2.64	13.20	$l/150$	20%
Beam 26	200x240 GL24h	SLS characteristic 14	Internal span	6.74	13.67	$l/300$	49%
Beam 27	200x240 GL24h	SLS characteristic 14	Internal span	11.21	13.67	$l/300$	82%
Beam 28	200x200 GL24h	SLS characteristic 14	Internal span	1.03	7.17	$l/300$	14%
Beam 29	200x200 GL24h	SLS characteristic 14	Internal span	1.12	7.17	$l/300$	16%
Beam 30	200x240 GL24h	SLS characteristic 14	Internal span	2.97	10.53	$l/300$	28%
Beam 31	200x100	SLS characteristic 4	Internal span	4.69	7.67	$l/300$	61%
Beam 32	200x520 GL24h	SLS characteristic 14	Cantilever beams	8.42	14.00	$l/150$	60%
Beam 33	200x200 GL24h	SLS characteristic 14	Internal span	0.36	3.33	$l/300$	11%
Beam 34	200x280 GL24h	SLS characteristic 14	Internal span	2.75	7.23	$l/300$	38%
Beam 35	200x520 GL24h	SLS characteristic 14	Cantilever beams	4.32	14.00	$l/150$	31%
Beam 36	200x100	SLS characteristic 14	Internal span	3.13	4.00	$l/300$	78%
Beam 37	200x100	SLS characteristic 14	Internal span	3.18	4.00	$l/300$	79%
Beam 38	200x100	SLS characteristic 14	Internal span	3.13	4.00	$l/300$	78%

Final deflection

For structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations the final deformation, w_{fin} , may be taken as:

$$w_{fin} = w_{fin,G} + w_{fin,Q1} + \sum w_{fin,Qi}$$

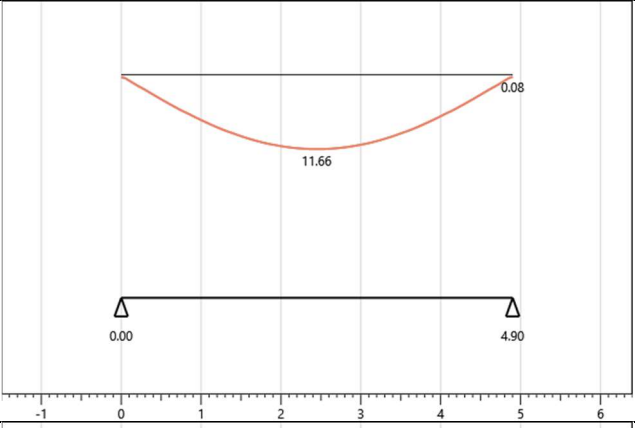
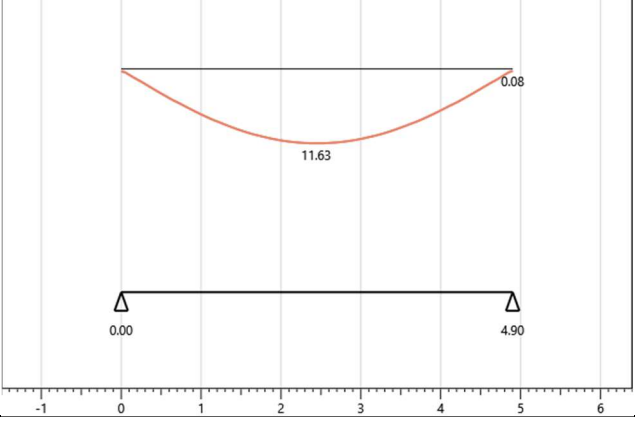
where:

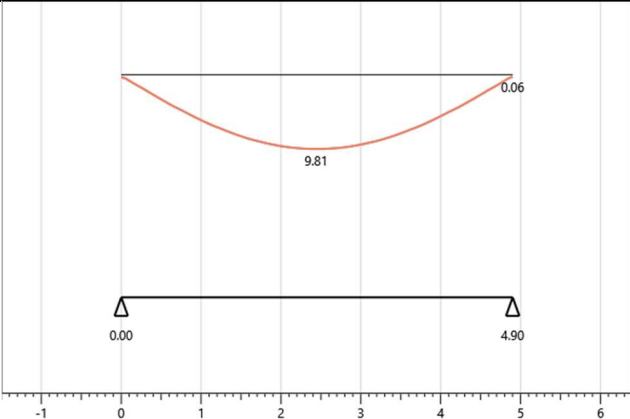
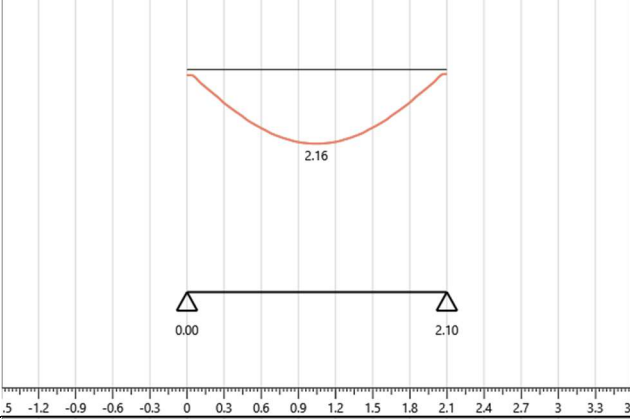
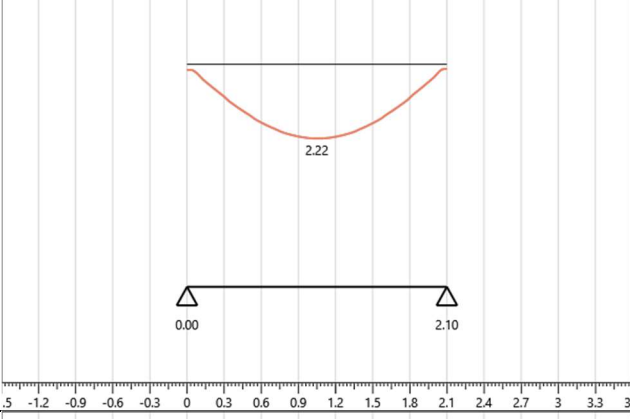
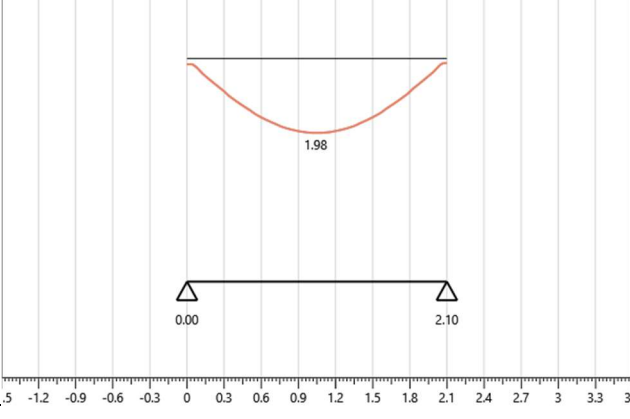
$$w_{fin,G} = w_{inst,G} \cdot (1 + k_{def}) \quad \text{for a permanent action, G}$$

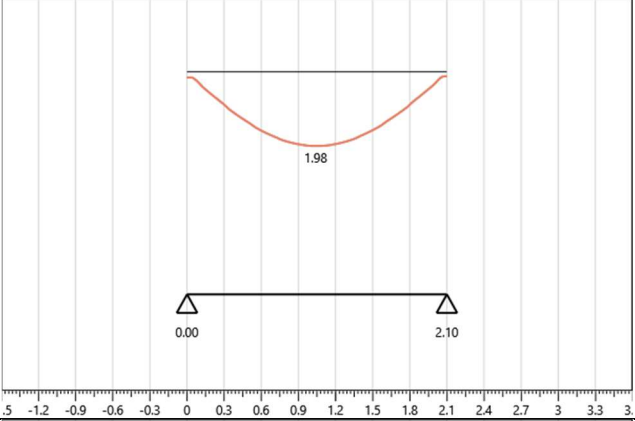
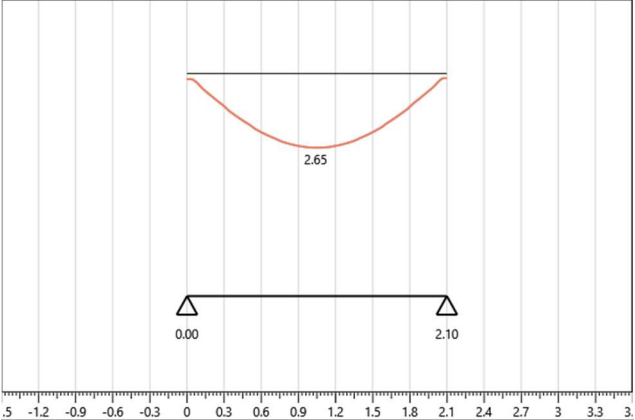
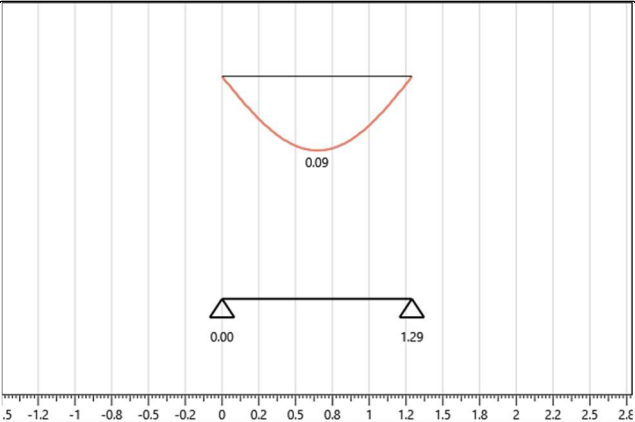
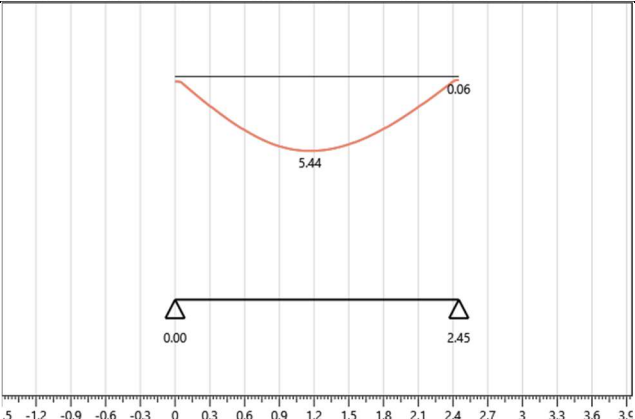
$$w_{fin,Q,1} = w_{inst,Q,1} \cdot (1 + \Psi_{2,1} \cdot k_{def}) \quad \text{for the leading variable action, } Q_1$$

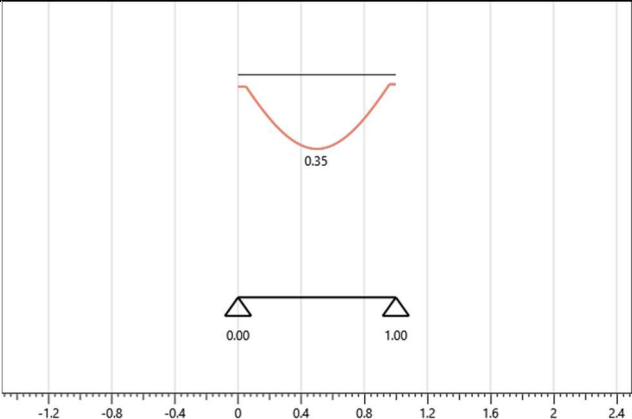
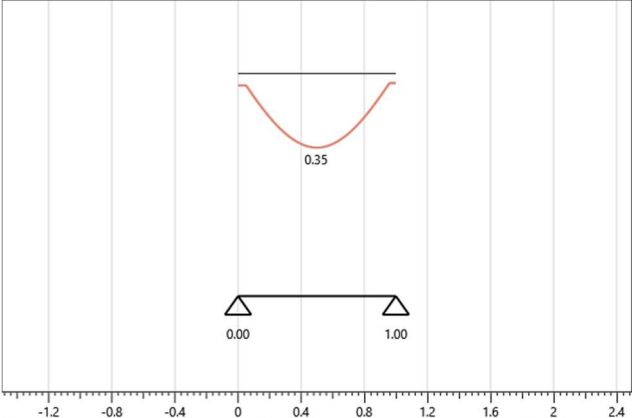
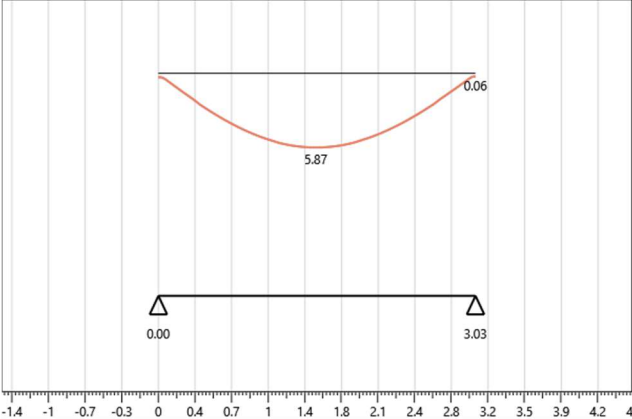
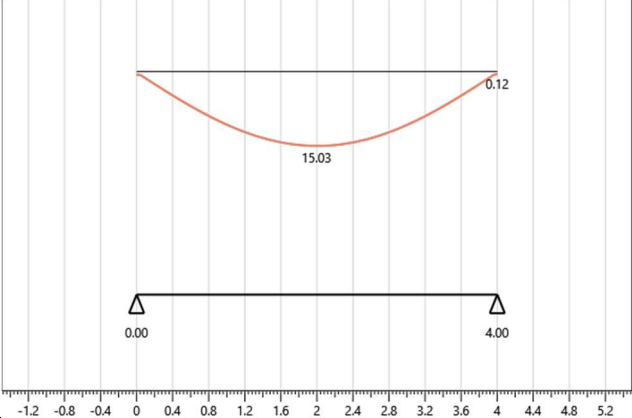
$$w_{fin,Q,i} = w_{inst,Q,i} \cdot (\Psi_{0,i} + \Psi_{2,1} \cdot k_{def}) \quad \text{for accompanying variable actions, } Q_i (i>1)$$

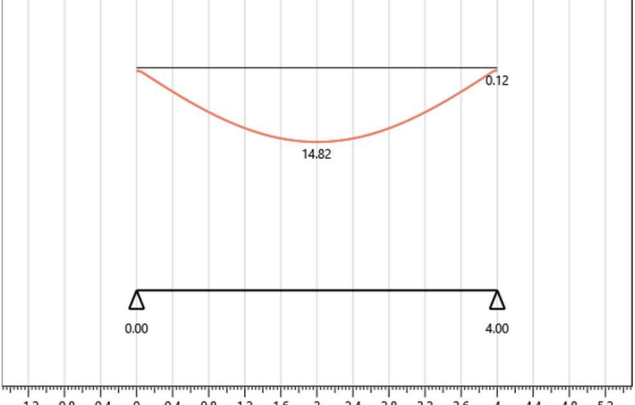
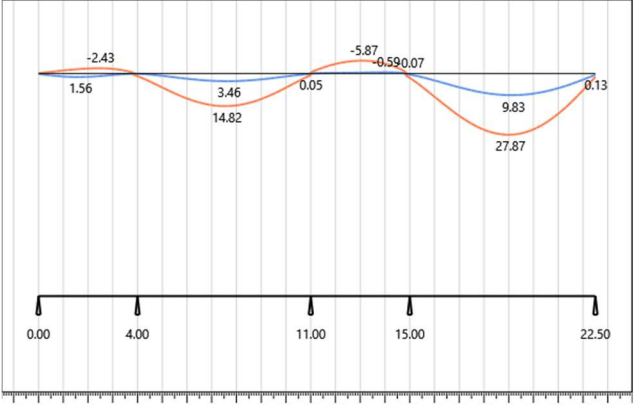
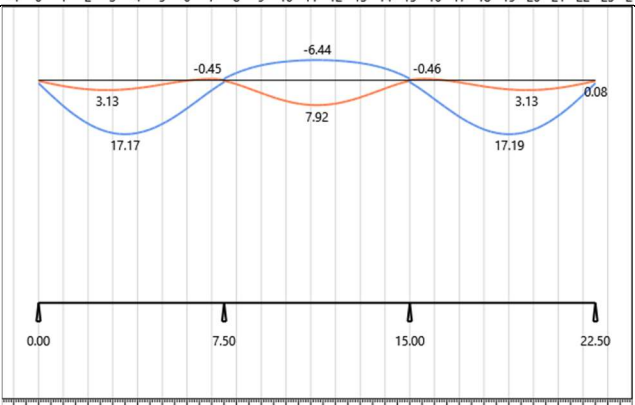
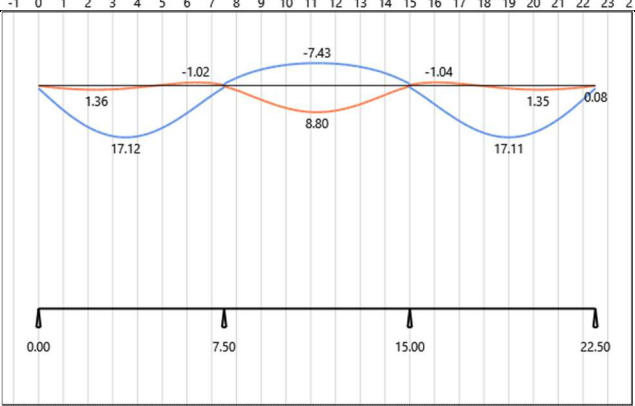
The following table shows the deformation of each floor (relative to the element in which the deformation checks are the most severe).

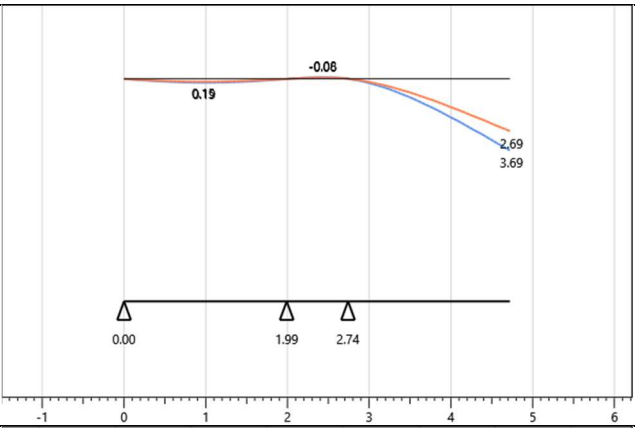
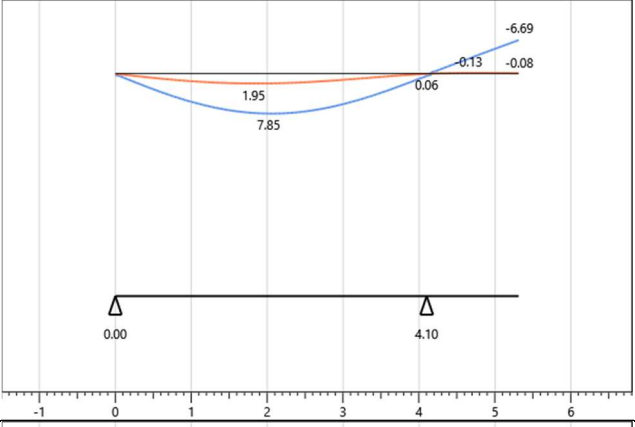
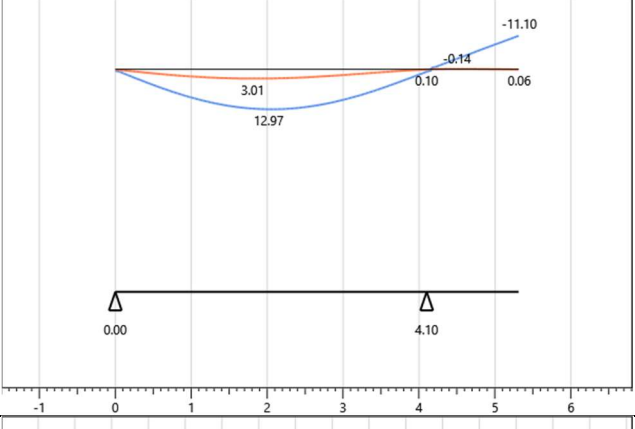
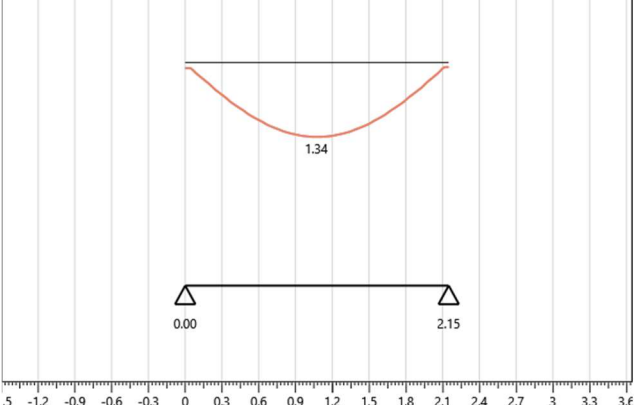
Beam name	Deformation limits name	Combination	Final deflection
Beam 1	Timber beam	SLS characteristic 15	
Beam 2	Timber beam	SLS characteristic 15	

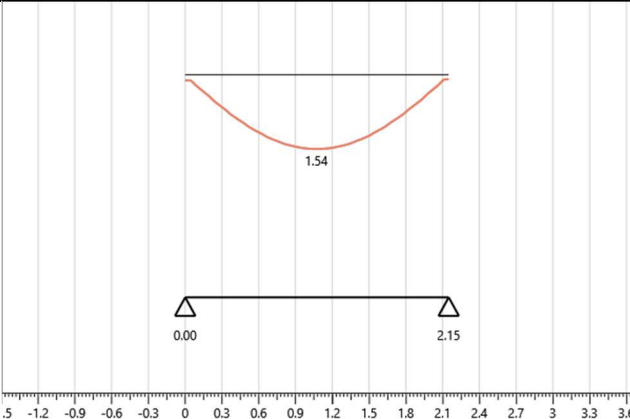
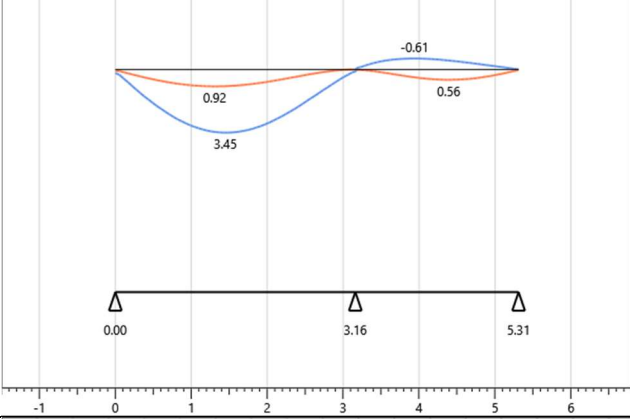
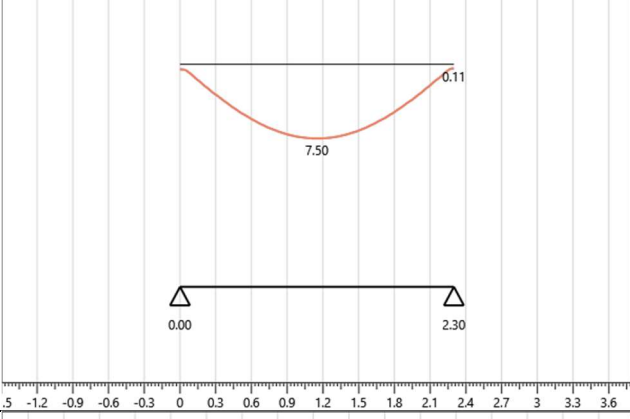
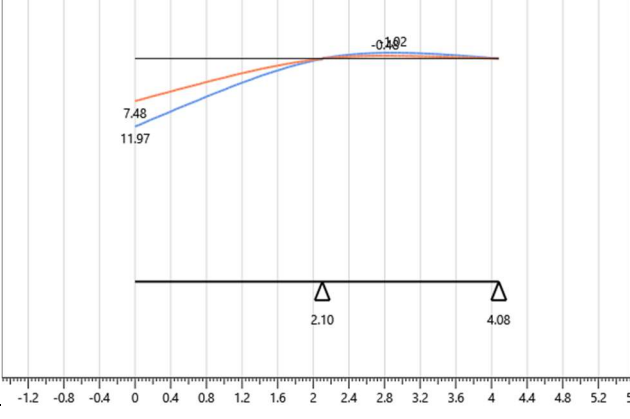
Beam 3	Timber beam	SLS characteristic 15	
Beam 4	Timber beam	SLS characteristic 15	
Beam 5	Timber beam	SLS characteristic 15	
Beam 6	Timber beam	SLS characteristic 15	

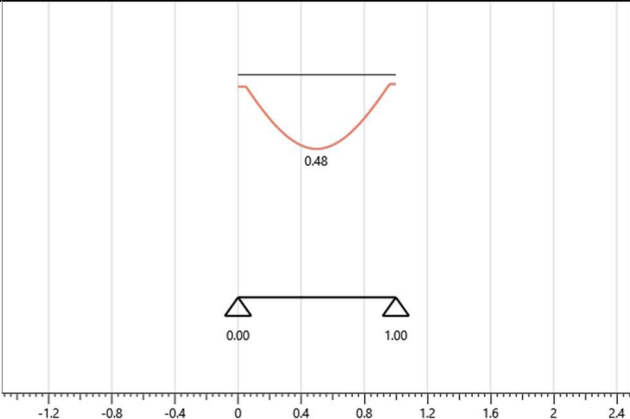
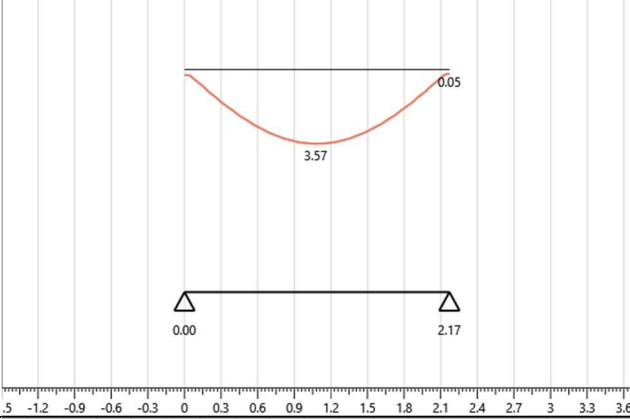
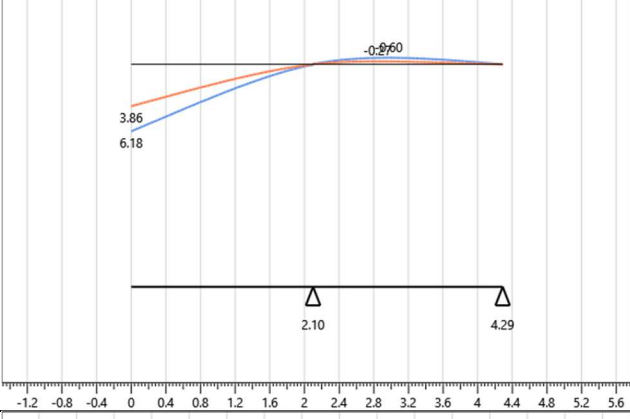
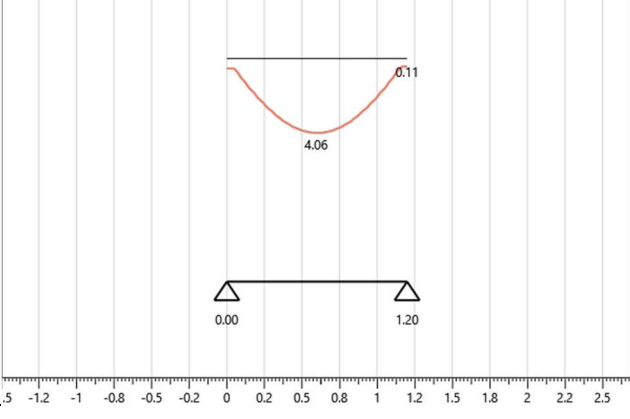
Beam 7	Timber beam	SLS characteristic 15	
Beam 8	Timber beam	SLS characteristic 15	
Beam 9	Timber beam	SLS characteristic 4	
Beam 10	Timber beam	SLS characteristic 14	

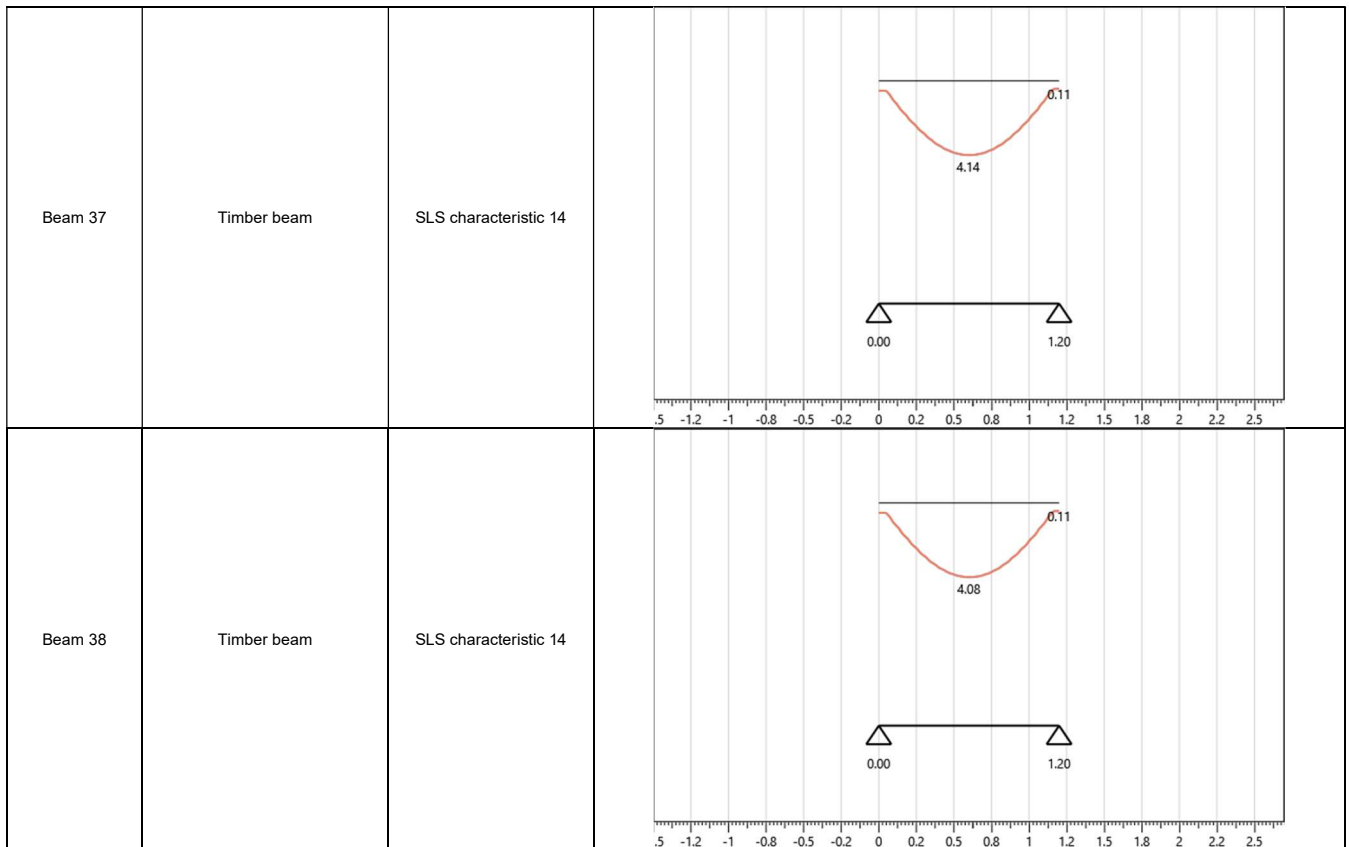
Beam 14	Timber beam	SLS characteristic 4	
Beam 15	Timber beam	SLS characteristic 4	
Beam 16	Timber beam	SLS characteristic 4	
Beam 17	Timber beam	SLS characteristic 4	

Beam 19	Timber beam	SLS characteristic 4	
Beam 20	Timber beam	SLS characteristic 14	
Beam 21	Timber beam	SLS characteristic 14	
Beam 22	Timber beam	SLS characteristic 14	

Beam 25	Timber beam	SLS characteristic 15	
Beam 26	Timber beam	SLS characteristic 14	
Beam 27	Timber beam	SLS characteristic 14	
Beam 28	Timber beam	SLS characteristic 14	

Beam 29	Timber beam	SLS characteristic 14	
Beam 30	Timber beam	SLS characteristic 14	
Beam 31	Timber beam	SLS characteristic 4	
Beam 32	Timber beam	SLS characteristic 14	

Beam 33	Timber beam	SLS characteristic 14	
Beam 34	Timber beam	SLS characteristic 14	
Beam 35	Timber beam	SLS characteristic 14	
Beam 36	Timber beam	SLS characteristic 14	



The table below shows the final deflection checks for every beam.

Beam name	Section	Combination	Service class	k_{def}	Most restrictive check	W_{fin} [mm]	W_{fin} limit [mm]	Deflection limit	Check
Beam 1	200x440 GL24h	SLS characteristic 15	1	0.6	Internal span	11.66	19.60	I/250	60%
Beam 2	200x440 GL24h	SLS characteristic 15	1	0.6	Internal span	11.63	19.60	I/250	59%
Beam 3	200x440 GL24h	SLS characteristic 15	1	0.6	Internal span	9.81	19.60	I/250	50%
Beam 4	200x200 GL24h	SLS characteristic 15	1	0.6	Internal span	2.16	8.40	I/250	26%
Beam 5	200x200 GL24h	SLS characteristic 15	1	0.6	Internal span	2.22	8.40	I/250	26%
Beam 6	200x200 GL24h	SLS characteristic 15	1	0.6	Internal span	1.98	8.40	I/250	24%
Beam 7	200x200 GL24h	SLS characteristic 15	1	0.6	Internal span	1.98	8.40	I/250	24%
Beam 8	200x200 GL24h	SLS characteristic 15	1	0.6	Internal span	2.65	8.40	I/250	32%
Beam 9	200x200 GL24h	SLS characteristic 4	1	0.6	Internal span	0.09	5.16	I/250	2%
Beam 10	200x240 GL24h	SLS characteristic 14	1	0.6	Internal span	5.44	9.80	I/250	55%
Beam 14	140x80	SLS characteristic 4	1	0.6	Internal span	0.35	4.00	I/250	9%
Beam 15	140x80	SLS characteristic 4	1	0.6	Internal span	0.35	4.00	I/250	9%
Beam 16	200x200 GL24h	SLS characteristic 4	1	0.6	Internal span	5.87	12.12	I/250	48%
Beam 17	260x280 GL28h	SLS characteristic 4	1	0.6	Internal span	15.03	16.00	I/250	94%
Beam 19	260x280 GL28h	SLS characteristic 4	1	0.6	Internal span	14.82	16.00	I/250	93%
Beam 20	200x400 GL28h	SLS characteristic 14	1	0.6	Internal span	27.87	30.00	I/250	93%
Beam 21	200x400 GL28h	SLS characteristic 14	1	0.6	Internal span	17.19	30.00	I/250	57%
Beam 22	200x400 GL28h	SLS characteristic 14	1	0.6	Internal span	17.12	30.00	I/250	57%
Beam 25	200x400 GL28h	SLS characteristic 15	1	0.6	Cantilever beams	3.69	15.84	I/125	23%
Beam 26	200x240 GL24h	SLS characteristic 14	1	0.6	Internal span	7.85	16.40	I/250	48%
Beam 27	200x240 GL24h	SLS characteristic 14	1	0.6	Internal span	12.97	16.40	I/250	79%
Beam 28	200x200 GL24h	SLS characteristic 14	1	0.6	Internal span	1.34	8.60	I/250	16%
Beam 29	200x200 GL24h	SLS characteristic 14	1	0.6	Internal span	1.54	8.60	I/250	18%
Beam 30	200x240 GL24h	SLS characteristic 14	1	0.6	Internal span	3.45	12.64	I/250	27%
Beam 31	200x100	SLS characteristic 4	1	0.6	Internal span	7.50	9.20	I/250	81%
Beam 32	200x520 GL24h	SLS characteristic 14	2	0.8	Cantilever beams	11.97	16.80	I/125	71%
Beam 33	200x200 GL24h	SLS characteristic 14	1	0.6	Internal span	0.48	4.00	I/250	12%
Beam 34	200x280 GL24h	SLS characteristic 14	1	0.6	Internal span	3.57	8.68	I/250	41%
Beam 35	200x520 GL24h	SLS characteristic 14	2	0.8	Cantilever beams	6.18	16.80	I/125	37%
Beam 36	200x100	SLS characteristic 14	1	0.6	Internal span	4.06	4.80	I/250	85%
Beam 37	200x100	SLS characteristic 14	1	0.6	Internal span	4.14	4.80	I/250	86%
Beam 38	200x100	SLS characteristic 14	1	0.6	Internal span	4.08	4.80	I/250	85%

Steel beams

Bending and shear checks

When the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected.

$$V_{Ed} \leq 0.5 V_{pl,Rd}$$

Otherwise the reduced moment resistance should be taken as the design resistance of the cross-section, calculated using a reduced yield strength for the shear area A_v :

$$(1 - \rho)f_y$$

where

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1 \right)^2$$

The reduced design plastic resistance moment allowing for the shear force may alternatively be obtained for I-cross-sections (class 1 or 2) with equal flanges and bending about the major axis as follows:

$$M_{c,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho \cdot A_w^2}{4 t_w} \right] f_{yk}}{\gamma_{M0}} \leq M_{c,Rd}$$

For circular hollow sections (class 1 or 2), the software uses a different equation that ensures higher safety standards:

$$M_{c,V,Rd} = \frac{W_{pl} f_{yk} (1 - \rho)}{\gamma_{M0}}$$

For all the class 3 cross-sections, the software uses a different equation, that ensures higher safety standards:

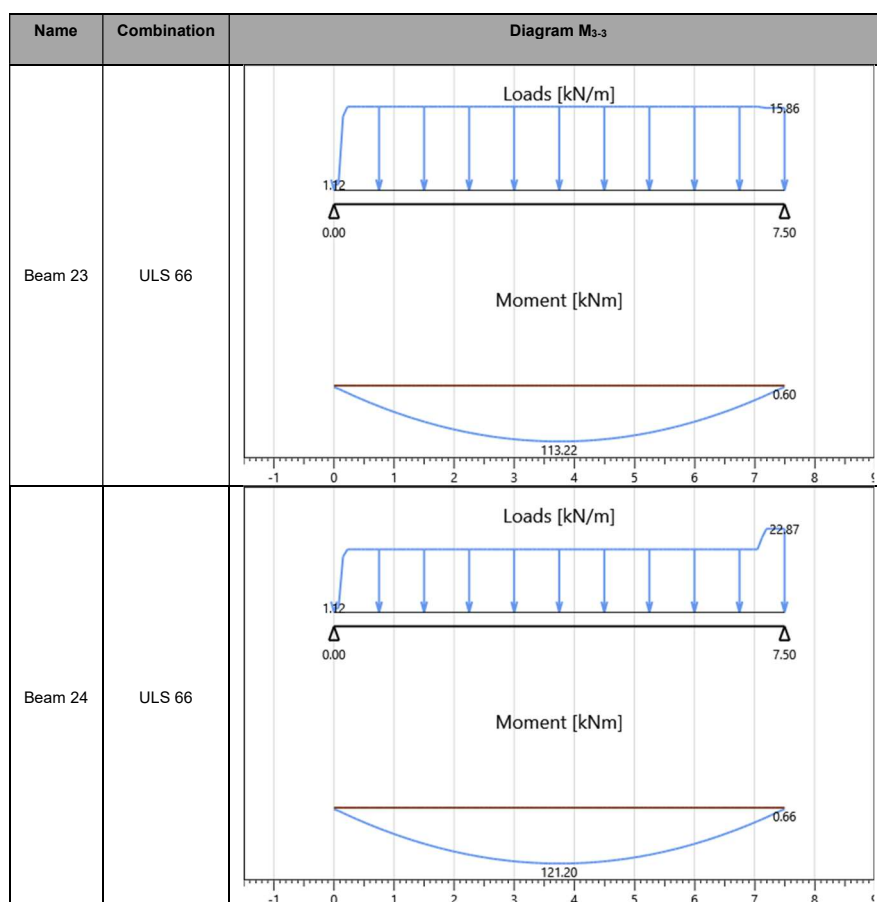
$$M_{c,V,Rd} = \frac{W_{el} f_{yk} (1 - \rho)}{\gamma_{M0}}$$

The partial factor for resistance of cross-section γ_{M0} is equal to 1.

It is assumed that the load is acting on the shear centre of the cross-section.

The following table provides, for each beam, the bending moment (the absolute value of the maximum bending moment M_{3-3} in the most stressed section) referring to the most severe ULS combination.

The table provides the structural schemes and the envelopes of the bending moment diagram for each beam.



Name	Profile	Steel grade	Cross section class	Combination	V ₂ [kN]	V ₂ /V _{pl,Rd} [%]	M ₃₋₃ max [kNm]	M _{c,V,Rd} [kNm]	Check
Beam 23	HE 240 B	S235 - EN 10025 -2	1	ULS 66	0.02	0%	113.22	247.46	46%
Beam 24	HE 240 B	S235 - EN 10025 -2	1	ULS 66	0.07	0%	121.20	247.46	49%

Shear strength

The checks are conducted according to § 6.2.6 of EN 1993-1-1. The design value of the shear force at each cross section should satisfy:

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 1$$

where:

V_{Ed} is the design value of the shear force;

$V_{pl,Rd}$ is the plastic shear resistance.

The design resistance, in absence of torsion, is given by:

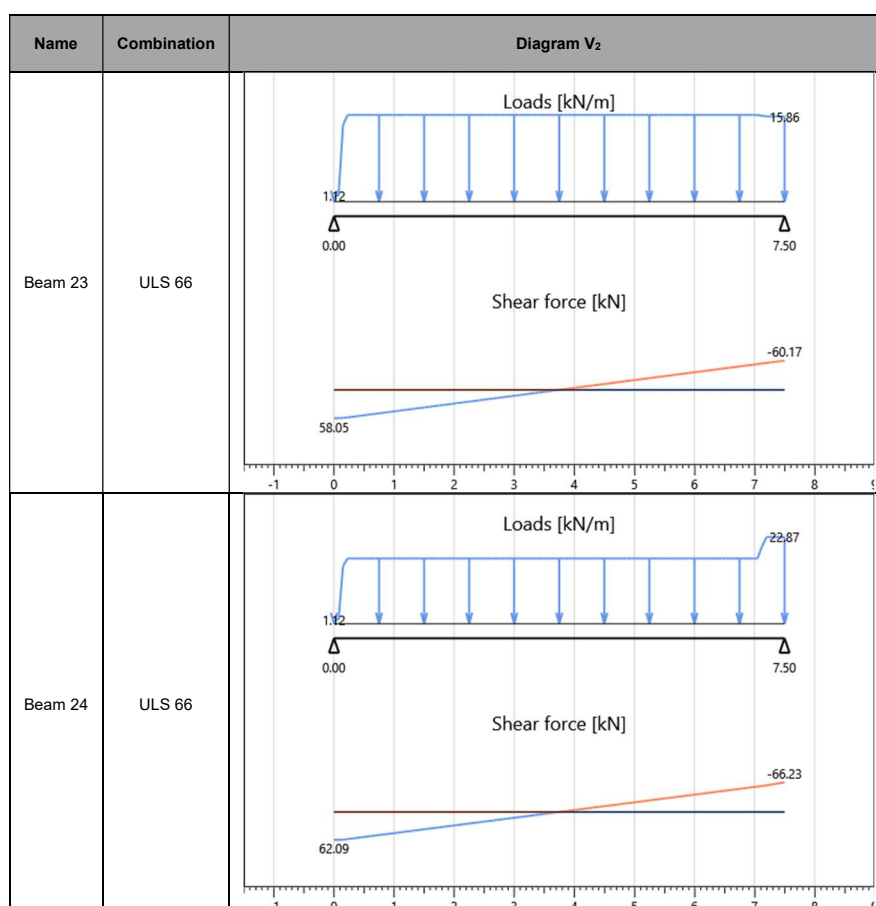
$$V_{pl,Rd} = \frac{A_v \cdot f_{yk}}{\sqrt{3} \cdot \gamma_{M0}}$$

where:

A_v is the shear area

γ_{M0} is the partial factor for resistance of cross-sections equal to 1.

The following table provides, for each beam, the structural schemes and the envelopes of the shear forces.



The shear checks are summarized below. The table reports the values in percent obtained from the calculations, relating to each verification.

Name	Profile	Steel grade	A_v $\times 10^2$ [mm ²]	Comb.	V_2 [kN]	$V_{pl,Rd}$ [kN]	Check
Beam 23	HE 240 B	S235 - EN 10025 -2	33	ULS 66	60.17	450.86	13%
Beam 24	HE 240 B	S235 - EN 10025 -2	33	ULS 66	66.23	450.86	15%

Steel beams deflections (SLE)

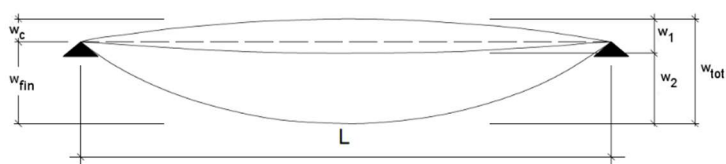
Structural members shall be designed to have adequate deflections limits.

The total deflection of the element is given by:

$$w_{tot} = w_1 + w_2$$

where:

w_c	precamber in the unloaded structural member;
w_1	deflection under permanents loads;
w_2	deflection under variable loads;
$w_{fin} = w_{tot} - w_c$	the final deflection not taking into account the precamber.

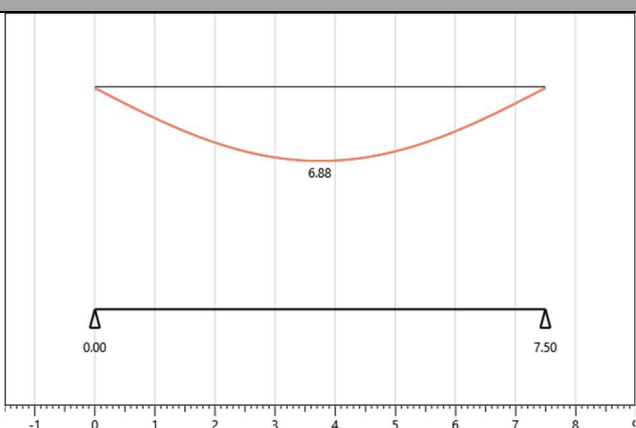
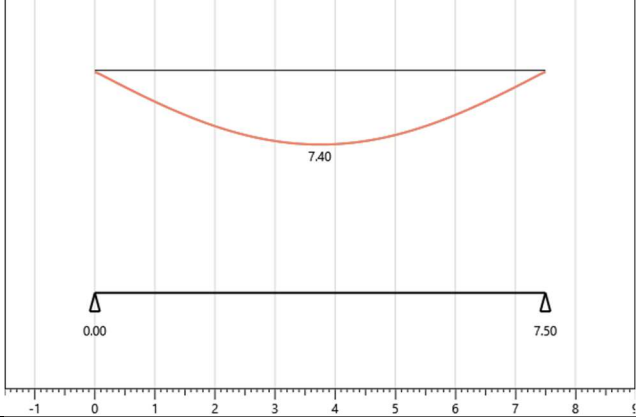


The following table provides the default limits of beams deflection where L is the effective beam span length or, in the case of cantilevering beams, the double value of the length.

Deformation limits name	$w_{2,span}$	$w_{fin,span}$	$w_{2,overhang}$	$w_{fin,overhang}$	Neglect overhang check for deformation < 0
Steel beam	$l/500$	$l/300$	$l/250$	$l/150$	Yes

Deflection under variable loads

The elastic deflection w_2 is calculated using the characteristic combination of the variable actions.

Name	Deformation limits name	Combination	Deflection under variable loads
Beam 23	Steel beam	SLS characteristic 4	
Beam 24	Steel beam	SLS characteristic 4	

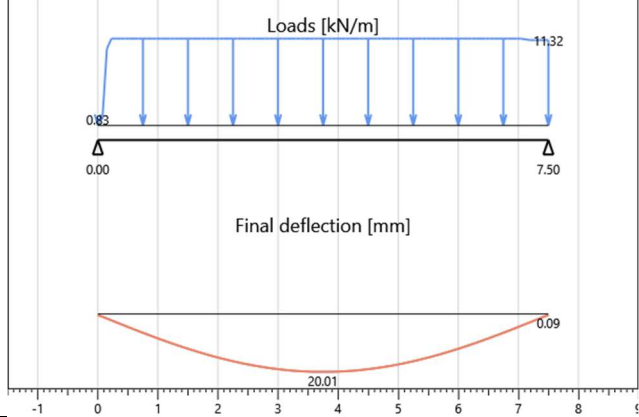
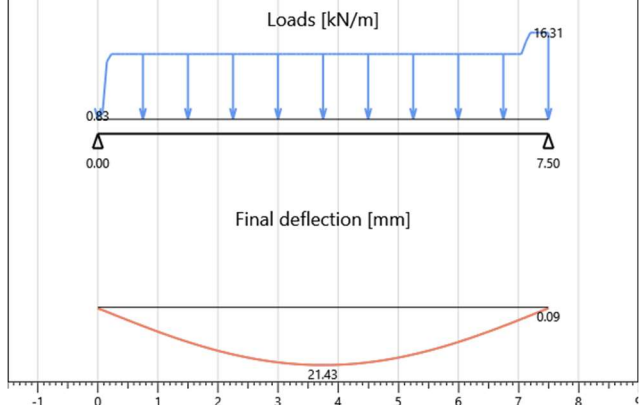
The following table provides the deflections checks of the steel beam under the variable loads.

Name	Profile	Combination	Most restrictive check	w_2 [mm]	$w_{2,limit}$ [mm]	Deflection limit	Check
Beam 23	HE 240 B	SLS characteristic 4	Internal span	6.88	15.00	$l/500$	46%
Beam 24	HE 240 B	SLS characteristic 4	Internal span	7.40	15.00	$l/500$	49%

Final elastic deflection

The final deflection w_{fin} is the deflection under the permanent loads (w_1) and the variable loads (w_2).

The following tables provides, for each beam, the final deformations.

Name	Deformation limits name	Combination	Final elastic deflection
Beam 23	Steel beam	SLS characteristic 4	
Beam 24	Steel beam	SLS characteristic 4	

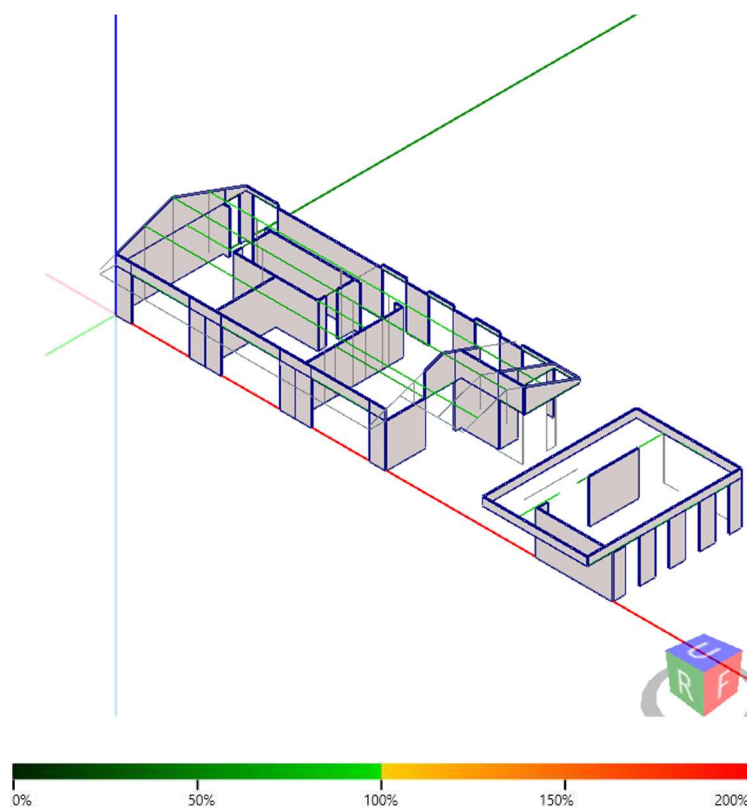
The following table provides the total deflection checks of the steel beams under the permanent and the variable loads.

Name	Profile	Combination	Most restrictive check	w_{fin} [mm]	$w_{fin,limit}$ [mm]	Deflection limit	Check
Beam 23	HE 240 B	SLS characteristic 4	Internal span	20.01	25.00	$l/300$	80%
Beam 24	HE 240 B	SLS characteristic 4	Internal span	21.43	25.00	$l/300$	86%

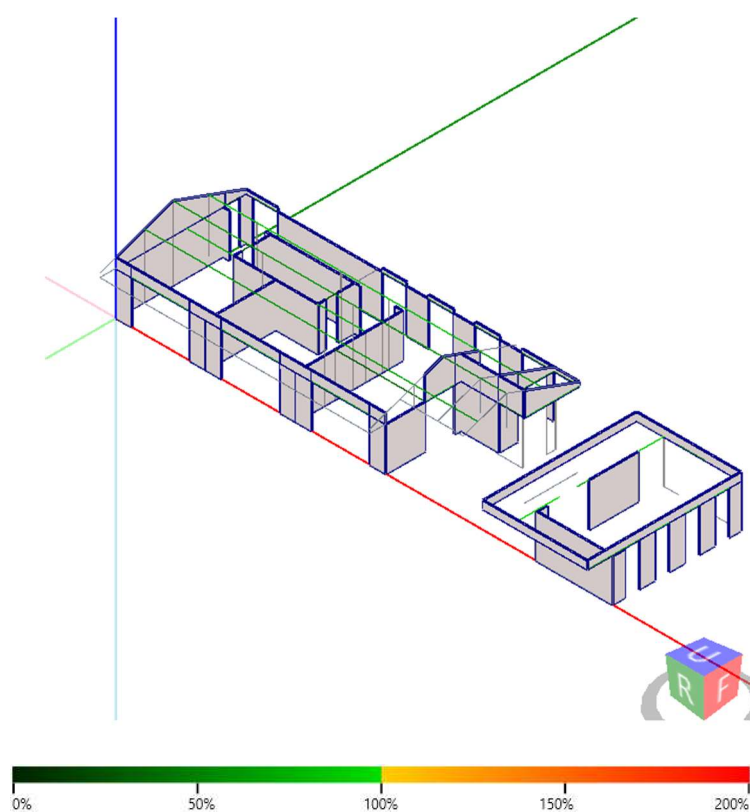
Verification percentage of beams

Ultimate Limit State (ULS)

Bending verifications

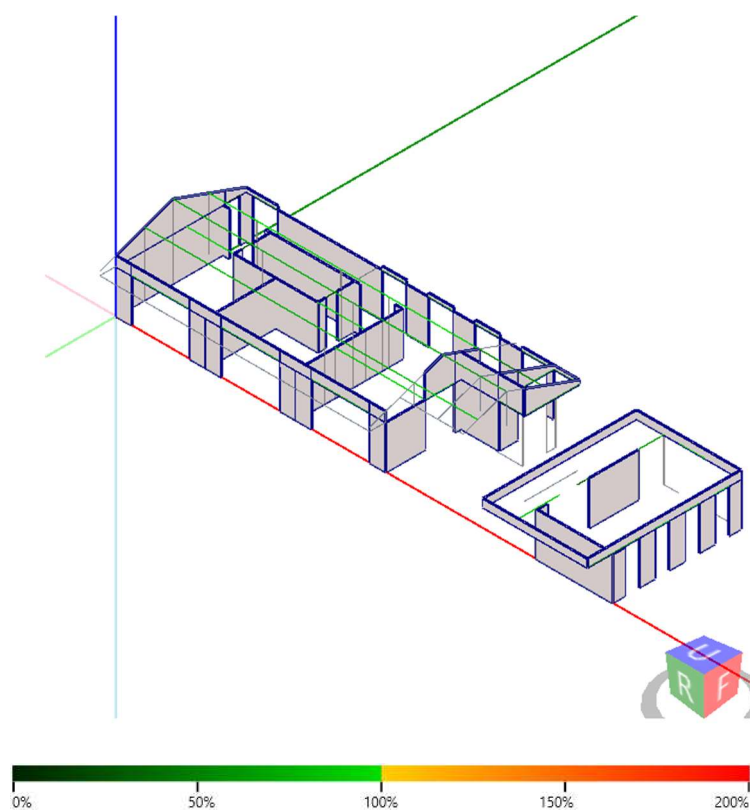


Shear verifications



Serviceability Limit State (SLS)

Deflection verifications



Timber columns

Stability of columns

The stability of columns subjected to compression is verified in accordance with § 6.3.2 of EN 1995-1-1.

The relative slenderness ratios should be taken as:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

and

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

where:

λ_y and $\lambda_{rel,y}$ are slenderness ratios corresponding to bending about the y-axis (deflection in the z-direction);

λ_z and $\lambda_{rel,z}$ are slenderness ratios corresponding to bending about the z-axis (deflection in the y-direction).

Where both $\lambda_{rel,z} \leq 0,3$ and $\lambda_{rel,y} \leq 0,3$, the stresses should satisfy the expressions (6.19) and (6.20) in 6.2.4 of EN 1995-1-1.

In all other cases the stresses, which will be increased due to deflection, should satisfy the following expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

where the symbols are defined as follows:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$$

$$k_y = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel,y} - 0,3) + \lambda_{rel,y}^2)$$

$$k_z = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel,z} - 0,3) + \lambda_{rel,z}^2)$$

where:

β_c is a factor for members within the straightness limits defined in Section 10 of EN 1995-1-1 and assumes the following values

$$\beta_c = \begin{cases} 0,2 & \text{for solid timber} \\ 0,1 & \text{for glued laminated timber and LVL} \end{cases}$$

The values of the actions in the tables below are related, for each pillar, to the most severe combination of load for the Ultimate Limit State of instability.

Comb.: The most severe combination of load

Dur.: Load duration

N: Axial force

V_2 : Shear force along the local axis 2

V_3 : Shear force along the local axis 3

M_{2-2} : Bending moment about local axis 2

M_{3-3} : Bending moment about local axis 3

Column name	Comb.	Dur.	N [kN]	V_2 [kN]	V_3 [kN]	M_{2-2} [kNm]	M_{3-3} [kNm]
Column 1	ULS 78	Short-term	31.52	0.00	0.00	0.00	0.00
Column 2	ULS 78	Short-term	85.44	0.00	0.00	0.00	0.00
Column 3	ULS 78	Short-term	85.59	0.00	0.00	0.00	0.00
Column 4	ULS 78	Short-term	46.08	0.00	0.00	0.00	0.00
Column 5	ULS 78	Short-term	19.02	0.00	0.00	0.00	0.00
Column 6	ULS 78	Short-term	118.96	0.00	0.00	0.00	0.00
Column 7	ULS 78	Short-term	84.69	0.00	0.00	0.00	0.00
Column 8	ULS 78	Short-term	125.41	0.00	0.00	0.00	0.00
Column 9	ULS 78	Short-term	74.76	0.00	0.00	0.00	0.00
Column 10	ULS 78	Short-term	49.69	0.00	0.00	0.00	0.00
Column 12	ULS 78	Short-term	31.74	0.00	0.00	0.00	0.00
Column 13	ULS 78	Short-term	86.51	0.00	0.00	0.00	0.00
Column 14	ULS 78	Short-term	86.51	0.00	0.00	0.00	0.00
Column 15	ULS 78	Short-term	55.32	0.00	0.00	0.00	0.00
Column 16	ULS 78	Short-term	126.22	0.00	0.00	0.00	0.00
Column 17	ULS 78	Short-term	86.40	0.00	0.00	0.00	0.00
Column 18	ULS 78	Short-term	87.33	0.00	0.00	0.00	0.00
Column 19	ULS 65	Medium-term	58.88	0.00	0.00	0.00	0.00
Column 20	ULS 65	Medium-term	54.21	0.00	0.00	0.00	0.00
Column 21	ULS 65	Medium-term	62.92	0.00	0.00	0.00	0.00
Column 22	ULS 65	Medium-term	47.79	0.00	0.00	0.00	0.00
Column 23	ULS 78	Short-term	87.33	0.00	0.00	0.00	0.00
Column 24	ULS 78	Short-term	86.26	0.00	0.00	0.00	0.00
Column 25	ULS 65	Medium-term	55.64	0.00	0.00	0.00	0.00
Column 26	ULS 78	Short-term	119.78	0.00	0.00	0.00	0.00
Column 27	ULS 78	Short-term	85.51	0.00	0.00	0.00	0.00
Column 28	ULS 78	Short-term	32.52	0.00	0.00	0.00	0.00
Column 29	ULS 78	Short-term	32.30	0.00	0.00	0.00	0.00
Column 30	ULS 78	Short-term	19.80	0.00	0.00	0.00	0.00
Column 31	ULS 65	Medium-term	73.92	0.00	0.00	0.00	0.00
Column 32	ULS 78	Short-term	56.09	0.00	0.00	0.00	0.00
Column 33	ULS 78	Short-term	36.00	0.00	0.00	0.00	0.00
Column 34	ULS 65	Medium-term	67.27	0.00	0.00	0.00	0.00
Column 35	ULS 78	Short-term	75.58	0.00	0.00	0.00	0.00
Column 36	ULS 78	Short-term	50.47	0.00	0.00	0.00	0.00
Column 42	ULS 78	Short-term	122.02	0.00	0.00	0.00	0.00
Column 43	ULS 78	Short-term	252.15	0.00	0.00	0.00	0.00
Column 45	ULS 78	Short-term	64.79	0.00	0.00	0.00	0.00
Column 46	ULS 78	Short-term	66.96	0.00	0.00	0.00	0.00

The following table summarizes the stability checks for the columns.

Sect.: Column cross section

h: Column height

Area: Cross sectional area of the column

J_y : Area moment of inertia with respect to the y axis

J_z : Area moment of inertia with respect to the z axis

Comb.: The most severe load combination

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

γ_M : Partial factor for a material property

$f_{c,0,d}$: Design compressive strength along the grain

$\sigma_{c,0,d}$: Design compressive stress along the grain

Column name	Sect.	h [m]	Area [mm ²]	J_y [mm ⁴]	J_z [mm ⁴]	$k_{c,y}$	$k_{c,z}$	Comb	Service Class	k_{mod}	γ_M	$f_{c,0,d}$	$\sigma_{c,0,d}$ [MPa]	Check
Column 1	200x200 GL24h	2.5	40000	1.33E8	1.33E8	0.93	0.93	ULS 78	1	0.9	1.25	17.28	0.79	5%
Column 2	200x200 GL24h	2.5	40000	1.33E8	1.33E8	0.93	0.93	ULS 78	1	0.9	1.25	17.28	2.14	13%
Column 3	200x200 GL24h	2.5	40000	1.33E8	1.33E8	0.93	0.93	ULS 78	1	0.9	1.25	17.28	2.14	13%
Column 4	200x200 GL24h	2.5	40000	1.33E8	1.33E8	0.93	0.93	ULS 78	1	0.9	1.25	17.28	1.15	7%
Column 5	200x200 GL24h	1.41 3	40000	1.33E8	1.33E8	0.99	0.99	ULS 78	1	0.9	1.25	17.28	0.48	3%
Column 6	200x200 GL24h	1.41 3	40000	1.33E8	1.33E8	0.99	0.99	ULS 78	1	0.9	1.25	17.28	2.97	17%
Column 7	200x200 GL24h	1.41 3	40000	1.33E8	1.33E8	0.99	0.99	ULS 78	1	0.9	1.25	17.28	2.12	12%
Column 8	200x200 GL24h	1.41 3	40000	1.33E8	1.33E8	0.99	0.99	ULS 78	1	0.9	1.25	17.28	3.14	18%
Column 9	200x200 GL24h	1.41 3	40000	1.33E8	1.33E8	0.99	0.99	ULS 78	1	0.9	1.25	17.28	1.87	11%
Column 10	200x200 GL24h	1.41 3	40000	1.33E8	1.33E8	0.99	0.99	ULS 78	1	0.9	1.25	17.28	1.24	7%
Column 12	200x200 GL24h	1.64 4	40000	1.33E8	1.33E8	0.98	0.98	ULS 78	1	0.9	1.25	17.28	0.79	5%
Column 13	200x200 GL24h	1.64 4	40000	1.33E8	1.33E8	0.98	0.98	ULS 78	1	0.9	1.25	17.28	2.16	13%
Column 14	200x200 GL24h	1.64 4	40000	1.33E8	1.33E8	0.98	0.98	ULS 78	1	0.9	1.25	17.28	2.16	13%
Column 15	200x200 GL24h	1.64 4	40000	1.33E8	1.33E8	0.98	0.98	ULS 78	1	0.9	1.25	17.28	1.38	8%
Column 16	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	3.76	69%
Column 17	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	2.57	47%
Column 18	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	2.60	48%
Column 19	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 65	1	0.8	1.3	12.92	1.75	36%
Column 20	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 65	1	0.8	1.3	12.92	1.61	33%
Column 21	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 65	1	0.8	1.3	12.92	1.87	39%
Column 22	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 65	1	0.8	1.3	12.92	1.42	29%
Column 23	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	2.60	48%
Column 24	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	2.57	47%
Column 25	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 65	1	0.8	1.3	12.92	1.66	34%
Column 26	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	3.56	65%
Column 27	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	2.54	47%
Column 28	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	1.02	15%
Column 29	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	1.01	15%
Column 30	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	0.62	9%
Column 31	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 65	1	0.8	1.3	12.92	2.31	38%
Column 32	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	1.75	26%
Column 33	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	1.13	16%
Column 34	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 65	1	0.8	1.3	12.92	2.00	41%
Column 35	140x240	3.6	33600	1.61E8	5.49E7	0.77	0.37	ULS 78	1	0.9	1.3	14.54	2.25	41%
Column 36	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	1.58	23%
Column 42	200x200 GL24h	3.6	40000	1.33E8	1.33E8	0.77	0.77	ULS 78	1	0.9	1.25	17.28	3.05	23%
Column 43	200x200 GL24h	3.6	40000	1.33E8	1.33E8	0.77	0.77	ULS 78	1	0.9	1.25	17.28	6.30	47%
Column 45	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	2.02	30%
Column 46	160x200	3.6	32000	1.07E8	6.83E7	0.65	0.47	ULS 78	1	0.9	1.3	14.54	2.09	31%

Steel columns

Buckling resistance

The buckling checks of the steel pillars are conducted according to the § 6.3.1 of EN 1993-1-1.

It is assumed that no torsional buckling of columns is occurring.

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1$$

where:

N_{Ed} is the design value of the compression force;

$N_{b,Rd}$ is the design buckling resistance of the compression member:

$$N_{b,Rd} = \frac{\chi A f_{yk}}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross – sections}$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_{yk}}{\gamma_{M1}} \quad \text{for Class 4 cross – sections}$$

where γ_{M1} is the partial factor for resistance of members to instability equal to 1, A and A_{eff} are the gross area and the effective area of the cross section (the effective area is calculated in accordance to § 4.4 of UNI EN 1993-1-5) and χ is the reduction factor for the relevant buckling mode, given by:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{con } \chi \leq 1$$

where:

$$\Phi = 0.5 \left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right]$$

α is an imperfection factor (Table 6.1 UNI EN 1993-1-1) corresponding to the appropriate buckling curve obtained from the Table 6.2 (UNI EN 1993-1-1);

$\bar{\lambda}$ is the non-dimensional slenderness in the case of flexural buckling given by:

$$\bar{\lambda} = \sqrt{\frac{A f_{yk}}{N_{cr}}} = \frac{L_{cr}}{i} \frac{1}{\lambda_l} \quad \text{for Class 1, 2 or 3 cross – sections}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_{yk}}{N_{cr}}} = \frac{L_{cr}}{i} \sqrt{\frac{A_{eff}}{A}} \frac{1}{\lambda_l} \quad \text{for Class 4 cross – sections}$$

where:

N_{cr} is the elastic critical force for flexural buckling;

L_{cr} is the buckling length in the buckling plane considered;

i is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section;

$\lambda_l = \pi \sqrt{\frac{E}{f_{yk}}}$ slenderness value to determine the relative slenderness.

The following table provides, for each column, the buckling checks referring to the most severe ULS combination.

Profile: The steel cross-section profile

Steel: Steel grade in accordance with the EN 10025-2

Class: Class of the steel cross-section

A: Gross or effective cross-section area depending on the section class

J_{y-y} : The cross-section moment of inertia about the y-y axis

J_{z-z} : The cross-section moment of inertia about the z-z axis

Comb.: The most severe load combination corresponding to the considered column

N: Axial force

$N_{b,Rd}$: The minimum design buckling resistance in the two principal directions

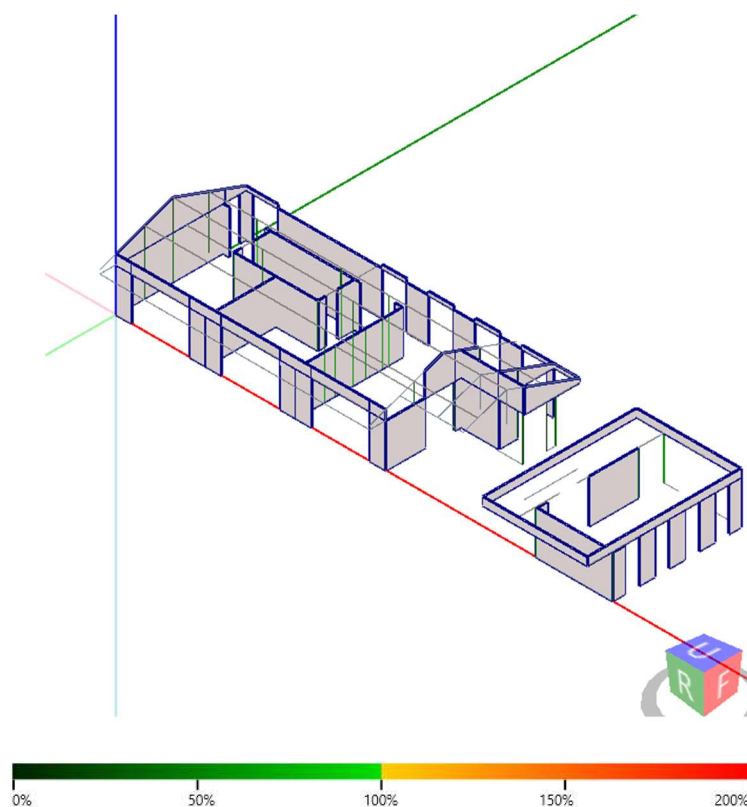
Check: Check in percent

Column name	Profile	Height [m]	Steel	Class	A $\times 10^2$ [mm ²]	J_{y-y} $\times 10^4$ [mm ⁴]	J_{z-z} $\times 10^4$ [mm ⁴]	Comb.	N [kN]	$N_{b,Rd}$ [kN]	Check
Column 37	SHS CF 150 5.0	3.60	S235 - EN 10025 -2	1	28.4	982	982	ULS 18	-5.96	503.76	-1%
Column 38	SHS CF 150 5.0	3.60	S235 - EN 10025 -2	1	28.4	982	982	ULS 78	64.97	503.76	13%
Column 39	RHS CF 150x100 5.0	3.60	S235 - EN 10025 -2	1	23.4	719	384	ULS 78	18.82	314.25	6%

Verification percentage of columns

Ultimate Limit State (ULS)

Buckling verifications



Framed walls

Stability of the studs

The stability of the studs subjected to compression is verified in accordance with § 6.3.2 of EN 1995-1-1. Specifically, the checked elements are the internal and the external studs which are the most loaded. These elements (stud or column in a sheathed wall) are braced against buckling in the in-plane direction therefore checks are performed only in the orthogonal direction.

Where both $\lambda_{rel,z} \leq 0,3$ and $\lambda_{rel,y} \leq 0,3$, the stresses should satisfy the expressions (6.19) and (6.20) in 6.2.4 of EN 1995-1-1.

In all other cases the stresses, which will be increased due to deflection, should satisfy the following expression:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1$$

The values of the actions in the tables below are related, for each pillar, to the most severe combination of load for the Ultimate Limit State of instability.

Wall name	Length [m]	Comb.	Dur.	N [kN]	M2-2 [kNm]
Wall 1	1.30	ULS 78	Short-term	104.17	0.00
Wall 2	2.60	ULS 78	Short-term	208.65	0.00
Wall 3	2.60	ULS 78	Short-term	194.50	0.00
Wall 4	1.30	ULS 78	Short-term	91.10	0.00
Wall 5	3.26	ULS 82	Instantaneous	29.67	3.88
Wall 6	0.55	ULS 78	Short-term	47.10	0.00
Wall 7	1.79	ULS 78	Short-term	61.83	0.00
Wall 8	1.79	ULS 78	Short-term	56.73	0.00
Wall 9	1.79	ULS 78	Short-term	57.31	0.00
Wall 10	8.61	ULS 78	Short-term	164.82	0.00
Wall 11	0.63	ULS 78	Short-term	24.15	0.00
Wall 12	0.53	ULS 82	Instantaneous	6.12	0.63
Wall 13	9.01	ULS 65	Medium-term	137.70	0.00
Wall 14	0.55	ULS 82	Instantaneous	6.09	0.65
Wall 15	3.16	ULS 84	Instantaneous	63.62	3.76
Wall 16	1.54	ULS 82	Instantaneous	34.14	1.83
Wall 17	0.63	ULS 64	Permanent	6.12	0.00
Wall 18	7.86	ULS 64	Permanent	72.87	0.00
Wall 19	0.62	ULS 64	Permanent	7.27	0.00
Wall 20	8.89	ULS 64	Permanent	50.61	0.00
Wall 21	5.82	ULS 64	Permanent	33.27	0.00
Wall 22	1.43	ULS 64	Permanent	9.26	0.00
Wall 23	1.43	ULS 64	Permanent	9.26	0.00
Wall 24	0.64	ULS 64	Permanent	4.60	0.00
Wall 25	0.64	ULS 64	Permanent	4.60	0.00
Wall 26	7.00	ULS 65	Medium-term	197.80	0.00
Wall 27	1.49	ULS 64	Permanent	13.04	0.00
Wall 29	2.73	ULS 78	Short-term	14.27	0.00
Wall 31	2.10	ULS 78	Short-term	11.35	0.00
Wall 33	2.10	ULS 78	Short-term	11.36	0.00
Wall 35	2.10	ULS 78	Short-term	11.36	0.00
Wall 37	4.80	ULS 78	Short-term	32.60	0.00
Wall 40	1.30	ULS 78	Short-term	22.56	0.00
Wall 41	1.30	ULS 78	Short-term	26.20	0.00
Wall 42	4.90	ULS 78	Short-term	75.89	0.00
Wall 43	2.60	ULS 78	Short-term	44.88	0.00
Wall 44	4.90	ULS 78	Short-term	75.89	0.00
Wall 45	2.60	ULS 78	Short-term	45.01	0.00
Wall 46	4.90	ULS 78	Short-term	75.89	0.00
Wall 47	9.01	ULS 82	Instantaneous	27.62	2.35
Wall 48	1.82	ULS 64	Permanent	2.36	0.00
Wall 50	2.45	ULS 82	Instantaneous	10.09	1.16
Wall 51	0.55	ULS 82	Instantaneous	2.23	0.21
Wall 52	3.16	ULS 84	Instantaneous	17.12	1.10
Wall 53	1.54	ULS 82	Instantaneous	4.91	0.40
Wall 54	3.03	ULS 64	Permanent	5.05	0.00
Wall 55	4.72	ULS 78	Short-term	19.39	0.00
Wall 56	1.49	ULS 64	Permanent	4.22	0.00
Wall 57	2.15	ULS 84	Instantaneous	12.73	0.75
Wall 58	0.98	ULS 82	Instantaneous	6.50	1.10
Wall 60	4.08	ULS 78	Short-term	267.98	0.00
Wall 64	0.99	ULS 78	Short-term	47.07	0.00

Wall 65	1.30	ULS 78	Short-term	73.56	0.00
Wall 66	1.30	ULS 78	Short-term	73.36	0.00
Wall 67	1.31	ULS 78	Short-term	55.50	0.00
Wall 68	1.00	ULS 82	Instantaneous	6.61	1.12
Wall 69	6.39	ULS 82	Instantaneous	40.79	7.60
Wall 70	12.79	ULS 64	Permanent	16.84	0.00
Wall 71	8.69	ULS 64	Permanent	11.44	0.00
Wall 72	12.79	ULS 64	Permanent	16.84	0.00
Wall 73	8.69	ULS 64	Permanent	11.44	0.00

The following table summarizes the stability checks for the studs of the framed walls.

Section: Type of cross-section of the stud

h_{stud} : Stud height

A_{stud} : Cross sectional area of the stud

J_{stud} : Cross sectional moment of inertia of the stud

Comb.: The most severe load combination

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

γ_M : Partial factor for a material property

$f_{c,0,k}$: Characteristic compressive strength along the grain

$f_{m,k}$: Design bending strength

$\sigma_{c,0,d}$: Design compressive stress along the grain

$\sigma_{m,d}$: Design bending stress about the principal axis

Wall name	Section	Stud	h_{stud} [m]	A_{stud} [mm ²]	J_{stud} [mm ⁴]	k_{Cstud}	Comb.	Service Class	k_{mod}	γ_M	$f_{c,0,k}$	$f_{m,k}$	N [kN]	$\sigma_{c,0,d}$ [MPa]	$\sigma_{m,d}$ [MPa]	Check
Wall 1	Okvir 200_ext=160	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	18.53	1.16	0.00	12%
Wall 1	Okvir 200_ext=160	External	3.6	32000	1.07E8	0.65	ULS 78	1	0.9	1.3	21.00	24.00	74.57	2.33	0.00	25%
Wall 2	Okvir 200_ext=160	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	19.65	1.23	0.00	13%
Wall 2	Okvir 200_ext=160	External	3.6	32000	1.07E8	0.65	ULS 78	1	0.9	1.3	21.00	24.00	76.41	2.39	0.00	25%
Wall 3	Okvir 200_ext=160	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	19.93	1.25	0.00	13%
Wall 3	Okvir 200_ext=160	External	3.6	32000	1.07E8	0.65	ULS 78	1	0.9	1.3	21.00	24.00	72.81	2.28	0.00	24%
Wall 4	Okvir 200_ext=160	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	17.74	1.11	0.00	12%
Wall 4	Okvir 200_ext=160	External	3.6	32000	1.07E8	0.65	ULS 78	1	0.9	1.3	21.00	24.00	62.95	1.97	0.00	21%
Wall 5	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	6.06	0.38	1.39	10%
Wall 5	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	2.69	0.17	0.74	5%
Wall 6	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	24.34	1.52	0.00	16%
Wall 7	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	11.52	0.72	0.00	8%
Wall 7	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	23.20	1.45	0.00	15%
Wall 8	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	11.19	0.70	0.00	7%
Wall 8	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	18.43	1.15	0.00	12%
Wall 9	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	11.13	0.70	0.00	7%
Wall 9	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	18.36	1.15	0.00	12%
Wall 10	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	12.92	0.81	0.00	9%
Wall 10	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	18.15	1.13	0.00	12%
Wall 11	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	19.00	1.19	0.00	13%
Wall 12	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	3.76	0.24	0.59	5%
Wall 13	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 65	1	0.8	1.3	21.00	24.00	51.81	3.24	0.00	39%
Wall 13	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 65	1	0.8	1.3	21.00	24.00	4.88	0.30	0.00	4%
Wall 14	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	3.07	0.19	0.61	5%
Wall 15	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 84	1	1.1	1.3	21.00	24.00	12.32	0.77	1.39	13%
Wall 15	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 84	1	1.1	1.3	21.00	24.00	9.93	0.62	0.78	9%
Wall 16	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	8.18	0.51	1.35	11%
Wall 16	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	6.98	0.44	0.74	7%
Wall 17	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	3.08	0.27	0.00	8%
Wall 18	Okvir 140	Internal	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	5.76	0.51	0.00	14%
Wall 18	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	2.78	0.25	0.00	7%
Wall 19	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	3.79	0.34	0.00	9%
Wall 20	Okvir 140	Internal	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	3.56	0.32	0.00	9%
Wall 20	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	1.89	0.17	0.00	5%
Wall 21	Okvir 140	Internal	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	3.57	0.32	0.00	9%
Wall 21	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	1.90	0.17	0.00	5%
Wall 22	Okvir 140	Internal	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	3.60	0.32	0.00	9%

Wall 22	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	2.72	0.24	0.00	7%
Wall 23	Okvir 140	Internal	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	3.60	0.32	0.00	9%
Wall 23	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	2.72	0.24	0.00	7%
Wall 24	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	2.67	0.24	0.00	7%
Wall 25	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	2.67	0.24	0.00	7%
Wall 26	Okvir 100 e=417mm	Internal	3.6	8000	6.67E6	0.20	ULS 65	1	0.8	1.3	21.00	24.00	11.78	1.47	0.00	56%
Wall 26	Okvir 100 e=417mm	External	3.6	8000	6.67E6	0.20	ULS 65	1	0.8	1.3	21.00	24.00	7.15	0.89	0.00	34%
Wall 27	Okvir 140	Internal	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	5.26	0.47	0.00	13%
Wall 27	Okvir 140	External	3.6	11200	1.83E7	0.37	ULS 64	1	0.6	1.3	21.00	24.00	2.79	0.25	0.00	7%
Wall 29	Okvir 200	Internal	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	3.77	0.24	0.00	2%
Wall 29	Okvir 200	External	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	2.00	0.13	0.00	1%
Wall 31	Okvir 200	Internal	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	3.77	0.24	0.00	2%
Wall 31	Okvir 200	External	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	0.80	0.05	0.00	0%
Wall 33	Okvir 200	Internal	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	3.77	0.24	0.00	2%
Wall 33	Okvir 200	External	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	1.66	0.10	0.00	1%
Wall 35	Okvir 200	Internal	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	4.01	0.25	0.00	2%
Wall 35	Okvir 200	External	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	2.26	0.14	0.00	1%
Wall 37	Okvir 200	Internal	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	4.70	0.29	0.00	2%
Wall 37	Okvir 200	External	0.3	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	3.04	0.19	0.00	1%
Wall 40	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	10.72	0.67	0.00	5%
Wall 40	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	6.41	0.40	0.00	3%
Wall 41	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	12.12	0.76	0.00	5%
Wall 41	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	8.26	0.52	0.00	4%
Wall 42	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	10.18	0.64	0.00	4%
Wall 42	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	4.05	0.25	0.00	2%
Wall 43	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	11.28	0.70	0.00	5%
Wall 43	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	6.53	0.41	0.00	3%
Wall 44	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	10.18	0.64	0.00	4%
Wall 44	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	5.29	0.33	0.00	2%
Wall 45	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	11.76	0.73	0.00	5%
Wall 45	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	7.29	0.46	0.00	3%
Wall 46	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	10.53	0.66	0.00	5%
Wall 46	Okvir 200	External	0.8	16000	5.33E7	1.00	ULS 78	1	0.9	1.3	21.00	24.00	5.85	0.37	0.00	3%
Wall 47	Okvir 200	Internal	2.5	16000	5.33E7	0.87	ULS 82	1	1.1	1.3	21.00	24.00	2.77	0.17	0.31	3%
Wall 47	Okvir 200	External	1.0	16000	5.33E7	1.00	ULS 82	1	1.1	1.3	21.00	24.00	0.61	0.04	0.16	1%
Wall 48	Okvir 200	Internal	0.8	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.88	0.06	0.00	1%
Wall 48	Okvir 200	External	1.0	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.57	0.04	0.00	0%
Wall 50	Okvir 200	Internal	2.4	16000	5.33E7	0.87	ULS 82	1	1.1	1.3	21.00	24.00	2.73	0.17	0.55	4%
Wall 50	Okvir 200	External	2.0	16000	5.33E7	0.93	ULS 82	1	1.1	1.3	21.00	24.00	1.22	0.08	0.29	2%
Wall 51	Okvir 200	External	2.1	16000	5.33E7	0.91	ULS 82	1	1.1	1.3	21.00	24.00	1.14	0.07	0.20	1%
Wall 52	Okvir 200	Internal	2.0	16000	5.33E7	0.93	ULS 84	1	1.1	1.3	21.00	24.00	3.54	0.22	0.41	3%
Wall 52	Okvir 200	External	2.0	16000	5.33E7	0.93	ULS 84	1	1.1	1.3	21.00	24.00	1.63	0.10	0.23	2%
Wall 53	Okvir 200	Internal	1.7	16000	5.33E7	0.95	ULS 82	1	1.1	1.3	21.00	24.00	1.98	0.12	0.29	2%
Wall 53	Okvir 200	External	2.0	16000	5.33E7	0.93	ULS 82	1	1.1	1.3	21.00	24.00	1.20	0.07	0.16	1%
Wall 54	Okvir 200	Internal	1.2	16000	5.33E7	0.99	ULS 64	1	0.6	1.3	21.00	24.00	1.39	0.09	0.00	1%
Wall 54	Okvir 200	External	1.4	16000	5.33E7	0.97	ULS 64	1	0.6	1.3	21.00	24.00	0.87	0.05	0.00	1%
Wall 55	Okvir 200	Internal	1.3	16000	5.33E7	0.98	ULS 78	1	0.9	1.3	21.00	24.00	7.24	0.45	0.00	3%
Wall 55	Okvir 200	External	2.0	16000	5.33E7	0.92	ULS 78	1	0.9	1.3	21.00	24.00	1.22	0.08	0.00	1%
Wall 56	Okvir 140	Internal	1.7	11200	1.83E7	0.87	ULS 64	1	0.6	1.3	21.00	24.00	1.75	0.16	0.00	2%
Wall 56	Okvir 140	External	2.0	11200	1.83E7	0.81	ULS 64	1	0.6	1.3	21.00	24.00	1.06	0.09	0.00	1%
Wall 57	Okvir 200	Internal	2.0	16000	5.33E7	0.93	ULS 84	1	1.1	1.3	21.00	24.00	3.60	0.22	0.41	3%
Wall 57	Okvir 200	External	2.0	16000	5.33E7	0.93	ULS 84	1	1.1	1.3	21.00	24.00	2.07	0.13	0.22	2%
Wall 58	Okvir 200	Internal	3.5	16000	5.33E7	0.67	ULS 82	1	1.1	1.3	21.00	24.00	2.98	0.19	0.95	6%
Wall 58	Okvir 200	External	3.5	16000	5.33E7	0.67	ULS 82	1	1.1	1.3	21.00	24.00	2.20	0.14	0.70	5%
Wall 60	Okvir 200 ext=160	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	37.42	2.34	0.00	25%
Wall 60	Okvir 200 ext=160	External	3.6	32000	1.07E8	0.65	ULS 78	1	0.9	1.3	21.00	24.00	50.63	1.58	0.00	17%
Wall 64	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	21.84	1.36	0.00	15%
Wall 64	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	15.16	0.95	0.00	10%
Wall 65	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	19.31	1.21	0.00	13%
Wall 65	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	27.32	1.71	0.00	18%
Wall 66	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	19.31	1.21	0.00	13%
Wall 66	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	27.12	1.70	0.00	18%
Wall 67	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	19.18	1.20	0.00	13%
Wall 67	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 78	1	0.9	1.3	21.00	24.00	26.52	1.66	0.00	18%
Wall 68	Okvir 200	Internal	3.5	16000	5.33E7	0.67	ULS 82	1	1.1	1.3	21.00	24.00	3.04	0.19	0.97	6%
Wall 68	Okvir 200	External	3.5	16000	5.33E7	0.67	ULS 82	1	1.1	1.3	21.00	24.00	2.20	0.14	0.70	5%
Wall 69	Okvir 200	Internal	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	3.99	0.25	1.39	9%
Wall 69	Okvir 200	External	3.6	16000	5.33E7	0.65	ULS 82	1	1.1	1.3	21.00	24.00	2.12	0.13	0.74	5%
Wall 70	Okvir 200	Internal	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.82	0.05	0.00	1%
Wall 70	Okvir 200	External	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.44	0.03	0.00	0%
Wall 71	Okvir 200	Internal	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.82	0.05	0.00	1%
Wall 71	Okvir 200	External	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.44	0.03	0.00	0%
Wall 72	Okvir 200	Internal	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.82	0.05	0.00	1%
Wall 72	Okvir 200	External	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.44	0.03	0.00	0%
Wall 73	Okvir 200	Internal	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.82	0.05	0.00	1%
Wall 73	Okvir 200	External	0.7	16000	5.33E7	1.00	ULS 64	1	0.6	1.3	21.00	24.00	0.44	0.03	0.00	0%

Compression perpendicular to the grain

The studs are supported at the base by the bottom plate which is stressed by compression forces perpendicular to the grain.

The following expression shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90,d} \cdot f_{c,90,d}$$

with

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}}$$

where:

$\sigma_{c,90,d}$ is the design compressive stress in the effective contact area perpendicular to the grain;

$F_{c,90,d}$ is the design compressive load perpendicular to the grain;

A_{ef} is the effective contact area in compression perpendicular to the grain;

$f_{c,90,d}$ is the design compressive strength perpendicular to the grain;

$k_{c,90,d}$ is a factor taking into account the load configuration, the possibility of splitting and the degree of compressive deformation.

The effective contact area perpendicular to the grain, A_{ef} , should be determined taking into account an effective contact length parallel to the grain, where the actual contact length, l , at each side is increased by 30 mm, but not more than a , l or $l/2$, see Figure 6.2 of EN 1995-1-1.

The value of $k_{c,90}$ should be taken as 1,0 unless the conditions in the following paragraphs apply. For members on continuous supports, provided that $l_1 \geq 2h$ (see Figure 6.2a of EN 1995-1-1), the value of $k_{c,90}$ should be taken as:

$k_{c,90} = 1,25$ for solid softwood timber

$k_{c,90} = 1,5$ for glued laminated softwood timber

where h is the depth of the member and l is the contact length.

The values of the actions in the tables below are related, for each wall, to the most severe combination of load for the compression Ultimate Limit State.

Wall name	Length [m]	Comb.	Dur.	N [kN]
Wall 1	1.30	ULS 78	Short-term	104.17
Wall 2	2.60	ULS 78	Short-term	208.65
Wall 3	2.60	ULS 78	Short-term	194.50
Wall 4	1.30	ULS 78	Short-term	91.10
Wall 5	3.26	ULS 64	Permanent	29.67
Wall 6	0.55	ULS 78	Short-term	47.10
Wall 7	1.79	ULS 78	Short-term	61.83
Wall 8	1.79	ULS 78	Short-term	56.73
Wall 9	1.79	ULS 78	Short-term	57.31
Wall 10	8.61	ULS 78	Short-term	164.82
Wall 11	0.63	ULS 78	Short-term	24.15
Wall 12	0.53	ULS 64	Permanent	6.12
Wall 13	9.01	ULS 65	Medium-term	137.70

Wall 14	0.55	ULS 64	Permanent	6.09
Wall 15	3.16	ULS 64	Permanent	53.38
Wall 16	1.54	ULS 64	Permanent	31.16
Wall 17	0.63	ULS 64	Permanent	6.12
Wall 18	7.86	ULS 64	Permanent	72.87
Wall 19	0.62	ULS 64	Permanent	7.27
Wall 20	8.89	ULS 64	Permanent	50.61
Wall 21	5.82	ULS 64	Permanent	33.27
Wall 22	1.43	ULS 64	Permanent	9.26
Wall 23	1.43	ULS 64	Permanent	9.26
Wall 24	0.64	ULS 64	Permanent	4.60
Wall 25	0.64	ULS 64	Permanent	4.60
Wall 26	7.00	ULS 65	Medium-term	197.80
Wall 27	1.49	ULS 64	Permanent	13.04
Wall 29	2.73	ULS 78	Short-term	14.27
Wall 31	2.10	ULS 78	Short-term	11.35
Wall 33	2.10	ULS 78	Short-term	11.36
Wall 35	2.10	ULS 78	Short-term	11.36
Wall 37	4.80	ULS 78	Short-term	32.60
Wall 40	1.30	ULS 78	Short-term	22.56
Wall 41	1.30	ULS 78	Short-term	26.20
Wall 42	4.90	ULS 78	Short-term	75.89
Wall 43	2.60	ULS 78	Short-term	44.88
Wall 44	4.90	ULS 78	Short-term	75.89
Wall 45	2.60	ULS 78	Short-term	45.01
Wall 46	4.90	ULS 78	Short-term	75.89
Wall 47	9.01	ULS 64	Permanent	27.62
Wall 48	1.82	ULS 64	Permanent	2.36
Wall 50	2.45	ULS 64	Permanent	10.09
Wall 51	0.55	ULS 64	Permanent	2.23
Wall 52	3.16	ULS 78	Short-term	23.91
Wall 53	1.54	ULS 64	Permanent	4.91
Wall 54	3.03	ULS 64	Permanent	5.05
Wall 55	4.72	ULS 78	Short-term	19.39
Wall 56	1.49	ULS 64	Permanent	4.22
Wall 57	2.15	ULS 78	Short-term	18.39
Wall 58	0.98	ULS 64	Permanent	6.50
Wall 60	4.08	ULS 78	Short-term	267.98
Wall 64	0.99	ULS 78	Short-term	47.07
Wall 65	1.30	ULS 78	Short-term	73.56
Wall 66	1.30	ULS 78	Short-term	73.36
Wall 67	1.31	ULS 78	Short-term	55.50
Wall 68	1.00	ULS 64	Permanent	6.61
Wall 69	6.39	ULS 64	Permanent	40.79
Wall 70	12.79	ULS 64	Permanent	16.84
Wall 71	8.69	ULS 64	Permanent	11.44
Wall 72	12.79	ULS 64	Permanent	16.84
Wall 73	8.69	ULS 64	Permanent	11.44

The following table summarizes the compression (perpendicular to the grain) checks for plates of the framed walls.

Wall name: Wall ID

A_{eff} : Effective contact area in compression perpendicular to the grain of the bottom plate

$k_{c,90}$: Factor taking into account the load configuration, the possibility of splitting and the degree of compressive deformation

Comb.: The most severe combination

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

γ_M : Partial factor for a material property

$f_{c,90,k}$: Characteristic compressive strength perpendicular to grain

$\sigma_{c,90,d}$: Design compressive stress perpendicular to grain

Wall name	Section	Stud	A_{eff} [mm ²]	$k_{c,90}$	Comb.	Service class	k_{mod}	γ_M	$f_{c,90,k}$ [MPa]	N [kN]	$\sigma_{c,90,d}$ [MPa]	Check
Wall 1	Okvir 200_ext=160	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	18.53	0.66	31%
Wall 1	Okvir 200_ext=160	External	38000.00	1.25	ULS 78	1	0.9	1.3	2.50	74.57	1.96	91%
Wall 2	Okvir 200_ext=160	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	19.65	0.70	32%

Wall 2	Okvir 200 ext=160	External	38000.00	1.25	ULS 78	1	0.9	1.3	2.50	76.41	2.01	93%
Wall 3	Okvir 200 ext=160	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	19.93	0.71	33%
Wall 3	Okvir 200 ext=160	External	38000.00	1.25	ULS 78	1	0.9	1.3	2.50	72.81	1.92	89%
Wall 4	Okvir 200 ext=160	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	17.74	0.63	29%
Wall 4	Okvir 200 ext=160	External	38000.00	1.25	ULS 78	1	0.9	1.3	2.50	62.95	1.66	77%
Wall 5	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	6.06	0.22	15%
Wall 5	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.69	0.12	8%
Wall 6	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	24.34	1.11	51%
Wall 7	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	11.52	0.41	19%
Wall 7	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	23.20	1.05	49%
Wall 8	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	11.19	0.40	18%
Wall 8	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	18.43	0.84	39%
Wall 9	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	11.13	0.40	18%
Wall 9	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	18.36	0.83	39%
Wall 10	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	12.92	0.46	21%
Wall 10	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	18.15	0.83	38%
Wall 11	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	19.00	0.86	40%
Wall 12	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	3.76	0.17	12%
Wall 13	Okvir 200	Internal	28000.00	1.25	ULS 65	1	0.8	1.3	2.50	51.81	1.85	96%
Wall 13	Okvir 200	External	22000.00	1.25	ULS 65	1	0.8	1.3	2.50	4.88	0.22	12%
Wall 14	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	3.07	0.14	10%
Wall 15	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	10.25	0.37	25%
Wall 15	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	8.38	0.38	26%
Wall 16	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	9.74	0.35	24%
Wall 16	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	7.07	0.32	22%
Wall 17	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	3.08	0.20	14%
Wall 18	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	5.76	0.29	20%
Wall 18	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	2.78	0.18	13%
Wall 19	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	3.79	0.25	17%
Wall 20	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	3.56	0.18	13%
Wall 20	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	1.89	0.12	9%
Wall 21	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	3.57	0.18	13%
Wall 21	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	1.90	0.12	9%
Wall 22	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	3.60	0.18	13%
Wall 22	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	2.72	0.18	12%
Wall 23	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	3.60	0.18	13%
Wall 23	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	2.72	0.18	12%
Wall 24	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	2.67	0.17	12%
Wall 25	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	2.67	0.17	12%
Wall 26	Okvir 100 e=417mm	Internal	14000.00	1.25	ULS 65	1	0.8	1.3	2.50	11.78	0.84	44%
Wall 26	Okvir 100 e=417mm	External	11000.00	1.25	ULS 65	1	0.8	1.3	2.50	7.15	0.65	34%
Wall 27	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	5.26	0.27	19%
Wall 27	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	2.79	0.18	13%
Wall 29	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	3.77	0.13	6%
Wall 29	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	2.00	0.09	4%
Wall 31	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	3.77	0.13	6%
Wall 31	Okvir 200	External	22000.00	1.00	ULS 78	1	0.9	1.3	2.50	0.80	0.04	2%
Wall 33	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	3.77	0.13	6%
Wall 33	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	1.66	0.08	3%
Wall 35	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	4.01	0.14	7%
Wall 35	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	2.26	0.10	5%
Wall 37	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	4.70	0.17	8%
Wall 37	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	3.04	0.14	6%
Wall 40	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	10.72	0.38	18%
Wall 40	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	6.41	0.29	13%
Wall 41	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	12.12	0.43	20%
Wall 41	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	8.26	0.38	17%
Wall 42	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	10.18	0.36	17%
Wall 42	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	4.05	0.18	9%
Wall 43	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	11.28	0.40	19%
Wall 43	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	6.53	0.30	14%
Wall 44	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	10.18	0.36	17%
Wall 44	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	5.29	0.24	11%
Wall 45	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	11.76	0.42	19%
Wall 45	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	7.29	0.33	15%
Wall 46	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	10.53	0.38	17%
Wall 46	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	5.85	0.27	12%
Wall 47	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.77	0.10	7%
Wall 47	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.61	0.03	2%
Wall 48	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.88	0.03	2%
Wall 48	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.57	0.03	2%
Wall 50	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.73	0.10	7%
Wall 50	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	1.23	0.06	4%
Wall 51	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	1.14	0.05	4%
Wall 52	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	5.07	0.18	8%
Wall 52	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	2.08	0.09	4%
Wall 53	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	1.98	0.07	5%
Wall 53	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	1.20	0.05	4%
Wall 54	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	1.39	0.05	3%
Wall 54	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.87	0.04	3%
Wall 55	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	7.24	0.26	12%
Wall 55	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	1.22	0.06	3%
Wall 56	Okvir 140	Internal	19600.00	1.25	ULS 64	1	0.6	1.3	2.50	1.75	0.09	6%
Wall 56	Okvir 140	External	15400.00	1.25	ULS 64	1	0.6	1.3	2.50	1.06	0.07	5%
Wall 57	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	5.39	0.19	9%
Wall 57	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	3.06	0.14	6%
Wall 58	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.98	0.11	7%
Wall 58	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.20	0.10	7%
Wall 60	Okvir 200 ext=160	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	37.42	1.34	62%
Wall 60	Okvir 200 ext=160	External	38000.00	1.25	ULS 78	1	0.9	1.3	2.50	50.63	1.33	62%
Wall 64	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	21.84	0.78	36%
Wall 64	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	15.16	0.69	32%
Wall 65	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	19.31	0.69	32%

Wall 65	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	27.32	1.24	57%
Wall 66	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	19.31	0.69	32%
Wall 66	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	27.12	1.23	57%
Wall 67	Okvir 200	Internal	28000.00	1.25	ULS 78	1	0.9	1.3	2.50	19.18	0.69	32%
Wall 67	Okvir 200	External	22000.00	1.25	ULS 78	1	0.9	1.3	2.50	26.52	1.21	56%
Wall 68	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	3.04	0.11	8%
Wall 68	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.20	0.10	7%
Wall 69	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	3.99	0.14	10%
Wall 69	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	2.12	0.10	7%
Wall 70	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.82	0.03	2%
Wall 70	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.44	0.02	1%
Wall 71	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.82	0.03	2%
Wall 71	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.44	0.02	1%
Wall 72	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.82	0.03	2%
Wall 72	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.44	0.02	1%
Wall 73	Okvir 200	Internal	28000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.82	0.03	2%
Wall 73	Okvir 200	External	22000.00	1.25	ULS 64	1	0.6	1.3	2.50	0.44	0.02	1%

Shear (load in-plane)

The values of the actions in the tables below are related, for each wall, to the most severe combination of load for the Ultimate Limit State.

Wall name	Length [m]	Comb.	Dur.	V2 [kN]
Wall 1	1.30	Dynamic SLV 4 ex+ ey+	Instantaneous	4.52
Wall 2	2.60	Dynamic SLV 4 ex+ ey+	Instantaneous	10.49
Wall 3	2.60	Dynamic SLV 4 ex+ ey+	Instantaneous	10.49
Wall 4	1.30	Dynamic SLV 4 ex+ ey+	Instantaneous	4.52
Wall 5	3.26	Dynamic SLV 8 ex+ ey+	Instantaneous	16.57
Wall 7	1.79	Dynamic SLV 4 ex+ ey+	Instantaneous	4.94
Wall 8	1.79	Dynamic SLV 4 ex+ ey+	Instantaneous	4.94
Wall 9	1.79	Dynamic SLV 4 ex+ ey+	Instantaneous	4.94
Wall 10	8.61	Dynamic SLV 4 ex+ ey+	Instantaneous	30.05
Wall 13	9.01	Dynamic SLV 8 ex- ey-	Instantaneous	44.99
Wall 15	3.16	Dynamic SLV 4 ex+ ey+	Instantaneous	10.31
Wall 16	1.54	Dynamic SLV 8 ex+ ey+	Instantaneous	8.14
Wall 18	7.86	Dynamic SLV 4 ex+ ey+	Instantaneous	43.29
Wall 20	8.89	Dynamic SLV 8 ex- ey-	Instantaneous	55.58
Wall 21	5.82	Dynamic SLV 8 ex- ey-	Instantaneous	33.47
Wall 22	1.43	Dynamic SLV 8 ex- ey-	Instantaneous	6.70
Wall 23	1.43	Dynamic SLV 8 ex- ey-	Instantaneous	5.83
Wall 26	7.00	Dynamic SLV 4 ex+ ey+	Instantaneous	33.10
Wall 27	1.49	Dynamic SLV 8 ex+ ey+	Instantaneous	11.16
Wall 40	1.30	Dynamic SLV 8 ex+ ey-	Instantaneous	3.76
Wall 41	1.30	Dynamic SLV 8 ex+ ey-	Instantaneous	3.76
Wall 43	2.60	Dynamic SLV 8 ex+ ey-	Instantaneous	7.76
Wall 45	2.60	Dynamic SLV 8 ex+ ey-	Instantaneous	7.76
Wall 47	9.01	Dynamic SLV 8 ex- ey-	Instantaneous	41.14
Wall 52	3.16	Dynamic SLV 4 ex+ ey+	Instantaneous	4.16
Wall 53	1.54	Dynamic SLV 8 ex- ey-	Instantaneous	7.74
Wall 56	1.49	Dynamic SLV 8 ex- ey-	Instantaneous	9.12
Wall 60	4.08	Dynamic SLV 8 ex- ey+	Instantaneous	24.15
Wall 64	0.99	Dynamic SLV 8 ex+ ey+	Instantaneous	4.42
Wall 65	1.30	Dynamic SLV 8 ex+ ey+	Instantaneous	6.07
Wall 66	1.30	Dynamic SLV 8 ex+ ey+	Instantaneous	6.07
Wall 67	1.31	Dynamic SLV 8 ex+ ey+	Instantaneous	6.12
Wall 69	6.39	Dynamic SLV 4 ex- ey-	Instantaneous	37.53

Lateral load-carrying capacity of metal fasteners

The design load-carrying capacity of a cantilevered panel secured against uplift is determined using the simplified method of analysis for walls proposed by EN 1995-1-1 9.2.4.2 “Simplified analysis of wall diaphragms – Method A”.

For a wall made up of several wall panels, the design racking load-carrying capacity of a wall should be calculated from:

$$F_{v,Rd} = \sum_i F_{i,v,Rd}$$

where:

$F_{i,v,Rd}$ is the design racking load-carrying capacity of the wall panel in accordance with 9.2.4.2(3) and 9.2.4.2(5) of EN 1995-1-1.

Wall panels which contain a door or window opening should not be considered to contribute to the racking load-carrying capacity.

The design racking load-carrying capacity of each wall panel, $F_{i,v,Rd}$, should be calculated from:

$$F_{i,v,Rd} = \frac{F_{t,Rd} \cdot b_i \cdot c_i}{s}$$

where:

- $F_{t,Rd}$ is the lateral design capacity of an individual fastener, modified by a factor of 1.2 in accordance with clause 9.2.4.2 (5);
- b_i is the wall panel width;
- s is the fastener spacing;
- c_i a coefficient dependent on the ratio between base and height of a single wall panel.

For wall panels with sheets on both sides the following rules apply:

- if the sheets and fasteners are of the same type and dimension then the total racking load carrying capacity of the wall should be taken as the sum of the racking load-carrying capacities of the individual sides;
- if different types of sheets are used, 75 % of the racking load-carrying capacity of the weaker side may, unless some other value is shown to be valid, be taken into consideration if fasteners with similar slip moduli are used. In other cases not more than 50 % should be taken into consideration.

Connector strength

The resistance of each connector is estimated according to the theory of Johansen presented in 8.2.2 EN 1995-1-1 for panel-to-timber connections (Single shear).

The characteristic load-carrying capacity for nails, staples, bolts, dowels and screws per shear plane per fastener, should be taken as the minimum value found from the following expressions

$$F_{v,Rk,a} = f_{h,1,k} \cdot t_1 \cdot d$$

$$F_{v,Rk,b} = f_{h,2,k} \cdot t_2 \cdot d$$

$$F_{v,Rk,c} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1 + \beta} \cdot \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right] + \beta^3 \left(\frac{t_2}{t_1} \right)^2 - \beta \left(1 + \frac{t_2}{t_1} \right)} \right] + \frac{F_{ax,Rk}}{4}$$

$$F_{v,Rk,d} = 1,05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2 + \beta} \cdot \left[\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta \right] + \frac{F_{ax,Rk}}{4}$$

$$F_{v,Rk,e} = 1,05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{1 + 2\beta} \cdot \left[\sqrt{2\beta^2(1 + \beta) + \frac{4\beta(1 + 2\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta \right] + \frac{F_{ax,Rk}}{4}$$

$$F_{v,Rk,f} = 1,15 \cdot \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$$

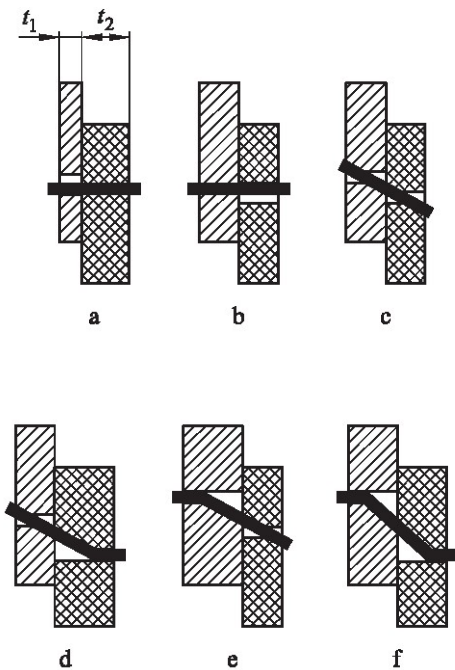


Figure: Failure modes for timber and panel connections.

In the expressions above, the first term on the right hand side is the load-carrying capacity according to the Johansen yield theory, whilst the second term $\frac{F_{ax,Rk}}{4}$ is the contribution from the rope effect.

Characteristic withdrawal capacity

The characteristic withdrawal capacity of nails, $F_{ax,Rk}$, should be taken as the smaller of the values found from the following expressions:

- For smooth nails:

$$F_{ax,Rk} = \begin{cases} f_{ax,k,tip} d t_{pen,frame} \\ f_{ax,k,head} d t + f_{head,k} d_h^2 \end{cases}$$

- For nails with improved adherence:

$$F_{ax,Rk} = \begin{cases} f_{ax,k,tip} d t_{pen,frame} = f_{ax,k,350} \left(\frac{\rho_{k,tip}}{350} \right)^{0.8} d t_{pen,frame} \\ f_{head,k} d_h^2 = f_{head,k,350} \left(\frac{\rho_{k,head}}{350} \right)^{0.8} d_h^2 \end{cases}$$

where:

$f_{ax,k,tip}$ is the characteristic pointside withdrawal strength;

$f_{ax,k,head}$ is the characteristic headside withdrawal strength;

$f_{head,k}$ is the characteristic headside pull-through strength;

d is the nail diameter;

d_h is the nail head diameter;

$t_{pen,frame}$ is the minimum value between the pointside penetration length and the length of the threaded part in the pointside member;

t is the thickness of the headside member.

In accordance with 8.3.2 (7) for smooth nails, the pointside penetration should be at least $8d$. For nails with a pointside penetration smaller than $12d$ the withdrawal capacity should be multiplied by $\frac{t_{pen}}{4d} - 2$.

For threaded nails, the pointside penetration should be at least $6d$. For nails with a pointside penetration smaller than $8d$ the withdrawal capacity should be multiplied by $\frac{t_{pen}}{2d} - 3$.

In the following table are reported the calculations for the characteristic pointside withdrawal strength ($F_{ax,k,tip}$) and for the characteristic headside pull-through strength ($F_{ax,k,head}$).

Section	Side	Fasteners	$\rho_{k,frame}$ [kg/m ³]	$f_{ax,k,tip}$ [MPa]	d [mm]	$t_{pen,frame}$ [mm]	$F_{ax,k,tip}$ [N]	$\rho_{k,panel}$ [kg/m ³]	$f_{ax,k,head}$ [MPa]	$f_{head,k}$ [MPa]	d_h [mm]	t [mm]	$F_{ax,k,head}$ [N]
Okvir 200_ext=160	1	Staples type Q 45 galv.	-	-	-	-	-	-	-	-	-	-	-

Okvir 200	1	Staples type Q 45 galv.	-	-	-	-	-	-	-	-	-	-	-
Okvir 140	1	Staples type Q 45 galv.	-	-	-	-	-	-	-	-	-	-	-
Okvir 140	2	Staples type Q 45 galv.	-	-	-	-	-	-	-	-	-	-	-
Okvir 100 e=417mm	1	Staples type Q 45 galv.	-	-	-	-	-	-	-	-	-	-	-
Okvir 100 e=417mm	2	Staples type Q 45 galv.	-	-	-	-	-	-	-	-	-	-	-

Lateral load-carrying capacity

The following table shows the resistances of the fasteners used to assemble the panels of the walls.

$F_{ax,Rk}$ is the characteristic axial withdrawal capacity of the fastener

Rope effect limit is the rope effect limited to a percentage of Johansen part

$F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener

Section	Side	Fasteners	K_{ser} [N/mm]	Failure mode	$F_{ax,Rk}$ [N]	Rope effect limit	$F_{v,Rk}$
Okvir 200 ext=160	1	Staples type Q 45 galv.	463	f	-	-	683
Okvir 200	1	Staples type Q 45 galv.	463	f	-	-	683
Okvir 140	1	Staples type Q 45 galv.	463	f	-	-	683
Okvir 140	2	Staples type Q 45 galv.	463	f	-	-	683
Okvir 100 e=417mm	1	Staples type Q 45 galv.	463	f	-	-	683
Okvir 100 e=417mm	2	Staples type Q 45 galv.	463	f	-	-	683

Check of the bearing capacity of the walls related to the lateral load-carrying capacity of metal fasteners

The following table summarizes the geometric characteristics of the wall panels and their load-bearing capacity $F_{i,v,Rk}$. This table also indicates whether the panels fulfill the geometrical requirements of 9.2.4.2(2) EN 1995-1-1:

- the spacing of fasteners is constant along the perimeter of every sheet;
- the width of each sheet is at least $h/4$.

Wall name	Section	Panel	b_i [mm]	N panels	c_i	Geometric check EN 1995-1-1 9.2.4.2 (2)	s [mm]	$F_{i,v,Rk}$ side 1 [kN]	$F_{i,v,Rk}$ side 2 [kN]
Wall 1	Okvir 200 ext=160	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 1	Okvir 200 ext=160	Junction	50.00	1	0.03	no	70	0.00	0.00
Wall 2	Okvir 200 ext=160	Full	1250.00	2	0.83	ok	70	12.19	0.00
Wall 2	Okvir 200 ext=160	Junction	100.00	1	0.07	no	70	0.00	0.00
Wall 3	Okvir 200 ext=160	Full	1250.00	2	0.83	ok	70	12.19	0.00
Wall 3	Okvir 200 ext=160	Junction	100.00	1	0.07	no	70	0.00	0.00
Wall 4	Okvir 200 ext=160	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 4	Okvir 200 ext=160	Junction	50.00	1	0.03	no	70	0.00	0.00
Wall 5	Okvir 200	Full	1250.00	2	0.83	ok	70	12.19	0.00
Wall 5	Okvir 200	Junction	760.00	1	0.51	ok	70	4.51	0.00
Wall 7	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 7	Okvir 200	Junction	540.00	1	0.36	no	70	0.00	0.00
Wall 8	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 8	Okvir 200	Junction	540.00	1	0.36	no	70	0.00	0.00
Wall 9	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 9	Okvir 200	Junction	540.00	1	0.36	no	70	0.00	0.00
Wall 10	Okvir 200	Full	1250.00	6	0.83	ok	70	12.19	0.00
Wall 10	Okvir 200	Junction	1110.00	1	0.74	ok	70	9.61	0.00
Wall 13	Okvir 200	Full	1250.00	7	0.83	ok	70	12.19	0.00
Wall 13	Okvir 200	Junction	260.00	1	0.17	no	70	0.00	0.00
Wall 15	Okvir 200	Full	1250.00	2	0.83	ok	70	12.19	0.00
Wall 15	Okvir 200	Junction	660.00	1	0.44	no	70	0.00	0.00
Wall 16	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 16	Okvir 200	Junction	290.00	1	0.19	no	70	0.00	0.00
Wall 18	Okvir 140	Full	1250.00	6	0.83	ok	70	12.19	12.19
Wall 18	Okvir 140	Junction	360.00	1	0.24	no	70	0.00	0.00
Wall 20	Okvir 140	Full	1250.00	7	0.83	ok	70	12.19	12.19
Wall 20	Okvir 140	Junction	140.00	1	0.09	no	70	0.00	0.00
Wall 21	Okvir 140	Full	1250.00	4	0.83	ok	70	12.19	12.19

Wall 21	Okvir 140	Junction	820.00	1	0.55	ok	70	5.24	5.24
Wall 22	Okvir 140	Full	1250.00	1	0.83	ok	70	12.19	12.19
Wall 22	Okvir 140	Junction	180.00	1	0.12	no	70	0.00	0.00
Wall 23	Okvir 140	Full	1250.00	1	0.83	ok	70	12.19	12.19
Wall 23	Okvir 140	Junction	180.00	1	0.12	no	70	0.00	0.00
Wall 26	Okvir 100 e=417mm	Full	834.00	8	0.56	ok	70	5.43	5.43
Wall 26	Okvir 100 e=417mm	Junction	328.00	1	0.22	no	70	0.00	0.00
Wall 27	Okvir 140	Full	1250.00	1	0.83	ok	70	12.19	12.19
Wall 27	Okvir 140	Junction	240.00	1	0.16	no	70	0.00	0.00
Wall 40	Okvir 200	Full	1250.00	1	1.00	ok	70	14.63	0.00
Wall 40	Okvir 200	Junction	50.00	1	0.13	no	70	0.00	0.00
Wall 41	Okvir 200	Full	1250.00	1	1.00	ok	70	14.63	0.00
Wall 41	Okvir 200	Junction	50.00	1	0.13	no	70	0.00	0.00
Wall 43	Okvir 200	Full	1250.00	2	1.00	ok	70	14.63	0.00
Wall 43	Okvir 200	Junction	100.00	1	0.25	no	70	0.00	0.00
Wall 45	Okvir 200	Full	1250.00	2	1.00	ok	70	14.63	0.00
Wall 45	Okvir 200	Junction	100.00	1	0.25	no	70	0.00	0.00
Wall 47	Okvir 200	Full	1250.00	7	1.00	ok	70	14.63	0.00
Wall 47	Okvir 200	Junction	260.00	1	0.31	no	70	0.00	0.00
Wall 52	Okvir 200	Full	1250.00	2	1.00	ok	70	14.63	0.00
Wall 52	Okvir 200	Junction	660.00	1	0.68	ok	70	5.23	0.00
Wall 53	Okvir 200	Full	1250.00	1	1.00	ok	70	14.63	0.00
Wall 53	Okvir 200	Junction	290.00	1	0.35	no	70	0.00	0.00
Wall 56	Okvir 140	Full	1250.00	1	1.00	ok	70	14.63	14.63
Wall 56	Okvir 140	Junction	240.00	1	0.29	no	70	0.00	0.00
Wall 60	Okvir 200 ext=160	Full	1250.00	3	0.83	ok	70	12.19	0.00
Wall 60	Okvir 200 ext=160	Junction	330.00	1	0.22	no	70	0.00	0.00
Wall 64	Okvir 200	Full	1250.00	0	0.83	ok	70	12.19	0.00
Wall 64	Okvir 200	Junction	990.00	1	0.66	ok	70	7.65	0.00
Wall 65	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 65	Okvir 200	Junction	50.00	1	0.03	no	70	0.00	0.00
Wall 66	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 66	Okvir 200	Junction	50.00	1	0.03	no	70	0.00	0.00
Wall 67	Okvir 200	Full	1250.00	1	0.83	ok	70	12.19	0.00
Wall 67	Okvir 200	Junction	60.00	1	0.04	no	70	0.00	0.00
Wall 69	Okvir 200	Full	1250.00	5	0.83	ok	70	12.19	0.00
Wall 69	Okvir 200	Junction	140.00	1	0.09	no	70	0.00	0.00

The following table instead shows checks with reference to the most significant load combinations.

Where a connection is constituted of two timber elements having different time-dependent behaviour, the calculation of the design load-carrying capacity should be made with the following modification factor $k_{mod,conn,i}$:

$$k_{mod,conn,i} = \sqrt{k_{mod,studs} \cdot k_{mod,side i}}$$

Wall name	Section	Comb.	Service class	Dur.	$k_{mod,studs}$	k_{mod1}	k_{mod2}	$k_{R,deg}$	γ_M	$F_{v,Rd}$ [kN]	$F_{v,Ed}$ [kN]	Check
Wall 1	Okvir 200 ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	4.52	44%
Wall 2	Okvir 200 ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	20.63	10.49	51%
Wall 3	Okvir 200 ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	20.63	10.49	51%
Wall 4	Okvir 200 ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	4.52	44%
Wall 5	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	24.44	16.57	68%
Wall 7	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	4.94	48%
Wall 8	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	4.94	48%
Wall 9	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	4.94	48%
Wall 10	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	70.01	30.05	43%
Wall 13	Okvir 200	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	72.19	44.99	62%
Wall 15	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	20.63	10.31	50%
Wall 16	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	8.14	79%
Wall 18	Okvir 140	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	123.76	43.29	35%
Wall 20	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	144.38	55.58	38%
Wall 21	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	91.38	33.47	37%
Wall 22	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	20.63	6.70	32%
Wall 23	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	20.63	5.83	28%
Wall 26	Okvir 100 e=417mm	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	73.45	33.10	45%
Wall 27	Okvir 140	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	20.63	11.16	54%
Wall 40	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	12.38	3.76	30%
Wall 41	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	12.38	3.76	30%
Wall 43	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	24.75	7.76	31%
Wall 45	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	24.75	7.76	31%
Wall 47	Okvir 200	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	86.63	41.14	47%
Wall 52	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	29.17	4.16	14%
Wall 53	Okvir 200	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	12.38	7.74	63%
Wall 56	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.1	1.0	1.3	24.75	9.12	37%
Wall 60	Okvir 200 ext=160	Dynamic SLV 8 ex- ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	30.94	24.15	78%
Wall 64	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	6.47	4.42	68%
Wall 65	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	6.07	59%
Wall 66	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	6.07	59%
Wall 67	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	-	1.0	1.3	10.31	6.12	59%
Wall 69	Okvir 200	Dynamic SLV 4 ex- ey-	1	Instantaneous	1.1	1.1	-	1.0	1.3	51.56	37.53	73%

Shear strength of the sheathing boards

The design load-carrying capacity of a cantilevered panel secured against uplift is determined using the simplified method of analysis for walls proposed by EN 1995-1-1 9.2.4.2 “Simplified analysis of wall diaphragms – Method A”.

For a wall made up of several wall panels, the design racking load-carrying capacity of a wall should be calculated from:

$$F_{v,Rd} = \sum_i F_{i,v,Rd}$$

where:

$F_{i,v,Rd}$ is the design racking load-carrying capacity of the wall panel in accordance with 9.2.4.2(3) and 9.2.4.2(5) of EN 1995-1-1.

The load-carrying capacity of a sheathing panel $F_{i,v,Rd}$ is:

$$F_{i,j,v,Rd} = f_{j,v,d} \cdot b_i \cdot t_{i,j}$$

where

$F_{i,j,v,Rd}$ is the shear strength of the single sheet, in which the first subscript indicates the panel of belonging and the second the side, external or internal;

$f_{j,v,d}$ is the shear strength of the single sheathing board;

b_i is the panel width;

$t_{i,j}$ is the thickness of the sheathing board.

The following table summarizes the load-bearing capacity $F_{i,v,Rk}$ of the wall panels. This table also indicates whether the panels fulfil the geometrical requirements of 9.2.4.2(2) EN 1995-1-1.

Wall name	Section	Panel	b_i [mm]	t_i side 1 [mm]	$f_{v,k}$ side 1 [MPa]	t_i side 2 [mm]	$f_{v,k}$ side 2 [MPa]	N of panels	Geometric check EN 1995- 1-1 9.2.4.2 (2)	$F_{i,v,Rk}$ side 1 [kN]	$F_{i,v,Rk}$ side 2 [kN]
Wall 1	Okvir 200 ext=160	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 1	Okvir 200 ext=160	Junction	50	15	6.8	-	-	1	no	0.00	0.00
Wall 2	Okvir 200 ext=160	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 2	Okvir 200 ext=160	Junction	100	15	6.8	-	-	1	no	0.00	0.00
Wall 3	Okvir 200 ext=160	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 3	Okvir 200 ext=160	Junction	100	15	6.8	-	-	1	no	0.00	0.00
Wall 4	Okvir 200 ext=160	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 4	Okvir 200 ext=160	Junction	50	15	6.8	-	-	1	no	0.00	0.00
Wall 5	Okvir 200	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 5	Okvir 200	Junction	760	15	6.8	-	-	1	ok	77.52	0.00
Wall 7	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 7	Okvir 200	Junction	540	15	6.8	-	-	1	no	0.00	0.00
Wall 8	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 8	Okvir 200	Junction	540	15	6.8	-	-	1	no	0.00	0.00
Wall 9	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 9	Okvir 200	Junction	540	15	6.8	-	-	1	no	0.00	0.00
Wall 10	Okvir 200	Full	1250	15	6.8	-	-	6	ok	127.50	0.00
Wall 10	Okvir 200	Junction	1110	15	6.8	-	-	1	ok	113.22	0.00

Wall 13	Okvir 200	Full	1250	15	6.8	-	-	7	ok	127.50	0.00
Wall 13	Okvir 200	Junction	260	15	6.8	-	-	1	no	0.00	0.00
Wall 15	Okvir 200	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 15	Okvir 200	Junction	660	15	6.8	-	-	1	no	0.00	0.00
Wall 16	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 16	Okvir 200	Junction	290	15	6.8	-	-	1	no	0.00	0.00
Wall 18	Okvir 140	Full	1250	15	6.8	15	6.8	6	ok	127.50	127.50
Wall 18	Okvir 140	Junction	360	15	6.8	15	6.8	1	no	0.00	0.00
Wall 20	Okvir 140	Full	1250	15	6.8	15	6.8	7	ok	127.50	127.50
Wall 20	Okvir 140	Junction	140	15	6.8	15	6.8	1	no	0.00	0.00
Wall 21	Okvir 140	Full	1250	15	6.8	15	6.8	4	ok	127.50	127.50
Wall 21	Okvir 140	Junction	820	15	6.8	15	6.8	1	ok	83.64	83.64
Wall 22	Okvir 140	Full	1250	15	6.8	15	6.8	1	ok	127.50	127.50
Wall 22	Okvir 140	Junction	180	15	6.8	15	6.8	1	no	0.00	0.00
Wall 23	Okvir 140	Full	1250	15	6.8	15	6.8	1	ok	127.50	127.50
Wall 23	Okvir 140	Junction	180	15	6.8	15	6.8	1	no	0.00	0.00
Wall 26	Okvir 100 e=417mm	Full	834	15	6.8	15	6.8	8	ok	85.07	85.07
Wall 26	Okvir 100 e=417mm	Junction	328	15	6.8	15	6.8	1	no	0.00	0.00
Wall 27	Okvir 140	Full	1250	15	6.8	15	6.8	1	ok	127.50	127.50
Wall 27	Okvir 140	Junction	240	15	6.8	15	6.8	1	no	0.00	0.00
Wall 40	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 40	Okvir 200	Junction	50	15	6.8	-	-	1	no	0.00	0.00
Wall 41	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 41	Okvir 200	Junction	50	15	6.8	-	-	1	no	0.00	0.00
Wall 43	Okvir 200	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 43	Okvir 200	Junction	100	15	6.8	-	-	1	no	0.00	0.00
Wall 45	Okvir 200	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 45	Okvir 200	Junction	100	15	6.8	-	-	1	no	0.00	0.00
Wall 47	Okvir 200	Full	1250	15	6.8	-	-	7	ok	127.50	0.00
Wall 47	Okvir 200	Junction	260	15	6.8	-	-	1	no	0.00	0.00
Wall 52	Okvir 200	Full	1250	15	6.8	-	-	2	ok	127.50	0.00
Wall 52	Okvir 200	Junction	660	15	6.8	-	-	1	ok	67.32	0.00
Wall 53	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 53	Okvir 200	Junction	290	15	6.8	-	-	1	no	0.00	0.00
Wall 56	Okvir 140	Full	1250	15	6.8	15	6.8	1	ok	127.50	127.50
Wall 56	Okvir 140	Junction	240	15	6.8	15	6.8	1	no	0.00	0.00
Wall 60	Okvir 200_ext=160	Full	1250	15	6.8	-	-	3	ok	127.50	0.00
Wall 60	Okvir 200_ext=160	Junction	330	15	6.8	-	-	1	no	0.00	0.00
Wall 64	Okvir 200	Full	1250	15	6.8	-	-	0	ok	127.50	0.00
Wall 64	Okvir 200	Junction	990	15	6.8	-	-	1	ok	100.98	0.00
Wall 65	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 65	Okvir 200	Junction	50	15	6.8	-	-	1	no	0.00	0.00
Wall 66	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 66	Okvir 200	Junction	50	15	6.8	-	-	1	no	0.00	0.00
Wall 67	Okvir 200	Full	1250	15	6.8	-	-	1	ok	127.50	0.00
Wall 67	Okvir 200	Junction	60	15	6.8	-	-	1	no	0.00	0.00
Wall 69	Okvir 200	Full	1250	15	6.8	-	-	5	ok	127.50	0.00
Wall 69	Okvir 200	Junction	140	15	6.8	-	-	1	no	0.00	0.00

The following table instead shows checks with reference to the most significant load combinations.

Wall name	Section	Comb.	Service Class	Dur.	k _{mod} side 1	k _{mod} side 2	γ _M	γ _{M2}	F _{V,Rd} [kN]	F _{V,Ed} [kN]	Check
Wall 1	Okvir 200_ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	4.52	4%
Wall 2	Okvir 200_ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	233.75	10.49	4%
Wall 3	Okvir 200_ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	233.75	10.49	4%
Wall 4	Okvir 200_ext=160	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	4.52	4%
Wall 5	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	304.81	16.57	5%
Wall 7	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	4.94	4%
Wall 8	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	4.94	4%
Wall 9	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	4.94	4%
Wall 10	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	805.04	30.05	4%
Wall 13	Okvir 200	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	-	1.2	-	818.13	44.99	5%
Wall 15	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	233.75	10.31	4%
Wall 16	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	8.14	7%
Wall 18	Okvir 140	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	1.2	1.2	1402.50	43.29	3%
Wall 20	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.2	1.2	1636.25	55.58	3%
Wall 21	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.2	1.2	1088.34	33.47	3%
Wall 22	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.2	1.2	233.75	6.70	3%
Wall 23	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.2	1.2	233.75	5.83	2%
Wall 26	Okvir 100 e=417mm	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	1.1	1.2	1.2	1247.66	33.10	3%
Wall 27	Okvir 140	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	1.1	1.2	1.2	233.75	11.16	5%
Wall 40	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	-	1.2	-	116.88	3.76	3%
Wall 41	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	-	1.2	-	116.88	3.76	3%
Wall 43	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	-	1.2	-	233.75	7.76	3%
Wall 45	Okvir 200	Dynamic SLV 8 ex+ ey-	1	Instantaneous	1.1	-	1.2	-	233.75	7.76	3%
Wall 47	Okvir 200	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	-	1.2	-	818.13	41.14	5%
Wall 52	Okvir 200	Dynamic SLV 4 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	295.46	4.16	1%
Wall 53	Okvir 200	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	-	1.2	-	116.88	7.74	7%
Wall 56	Okvir 140	Dynamic SLV 8 ex- ey-	1	Instantaneous	1.1	1.1	1.2	1.2	233.75	9.12	4%
Wall 60	Okvir 200_ext=160	Dynamic SLV 8 ex- ey+	1	Instantaneous	1.1	-	1.2	-	350.63	24.15	7%

Wall 64	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	92.57	4.42	5%
Wall 65	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	6.07	5%
Wall 66	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	6.07	5%
Wall 67	Okvir 200	Dynamic SLV 8 ex+ ey+	1	Instantaneous	1.1	-	1.2	-	116.88	6.12	5%
Wall 69	Okvir 200	Dynamic SLV 4 ex- ey-	1	Instantaneous	1.1	-	1.2	-	584.38	37.53	6%

Shear buckling of the sheet

According to 9.2.4.2 of EN 1995-1-1 shear buckling of the sheet may be disregarded, provided that

$$\frac{b_{net}}{t} \leq 100$$

where:

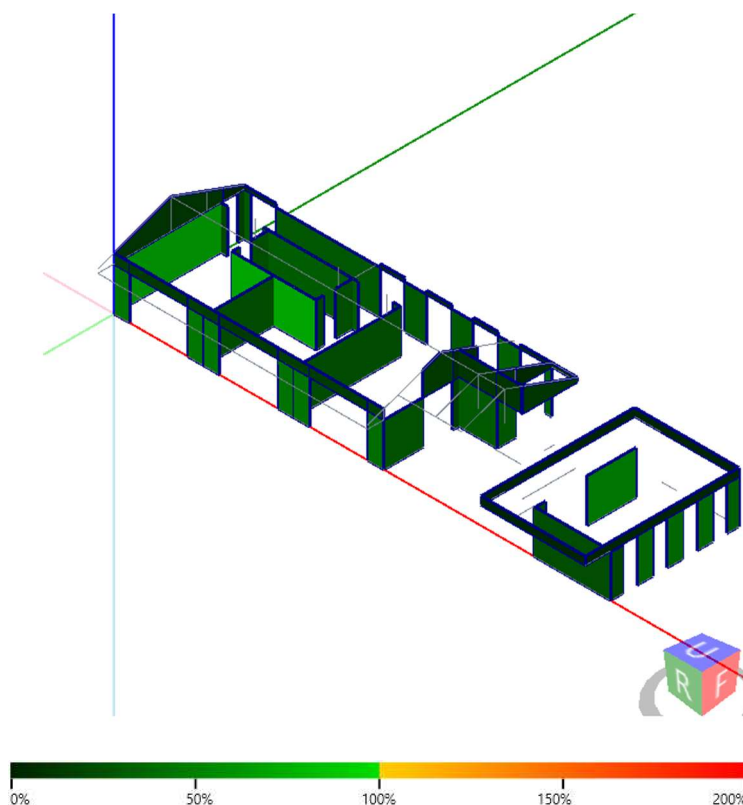
b_{net} is the clear distance between studs;

t is the thickness of the sheet.

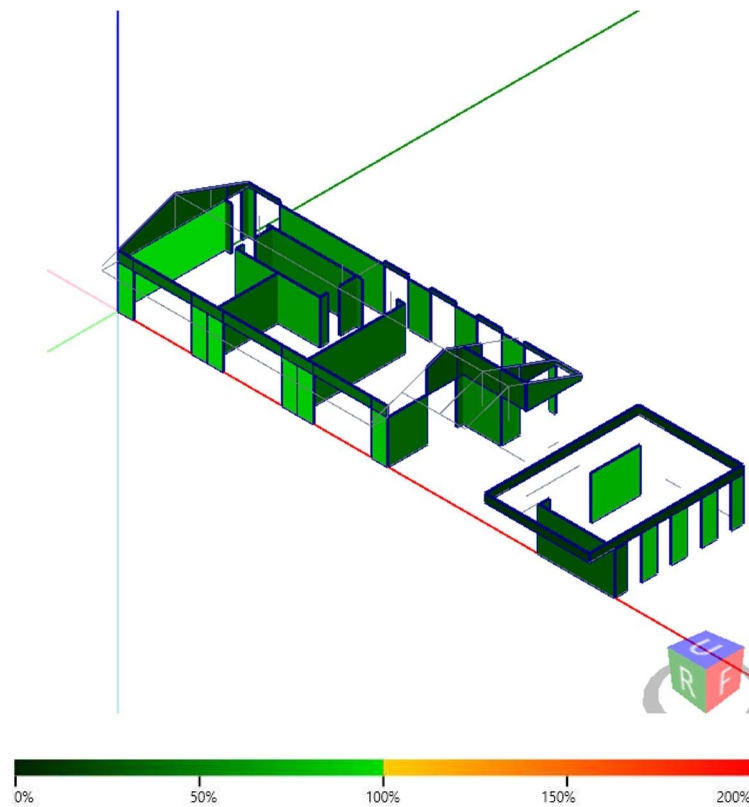
Verification percentage of walls

Ultimate Limit State (ULS)

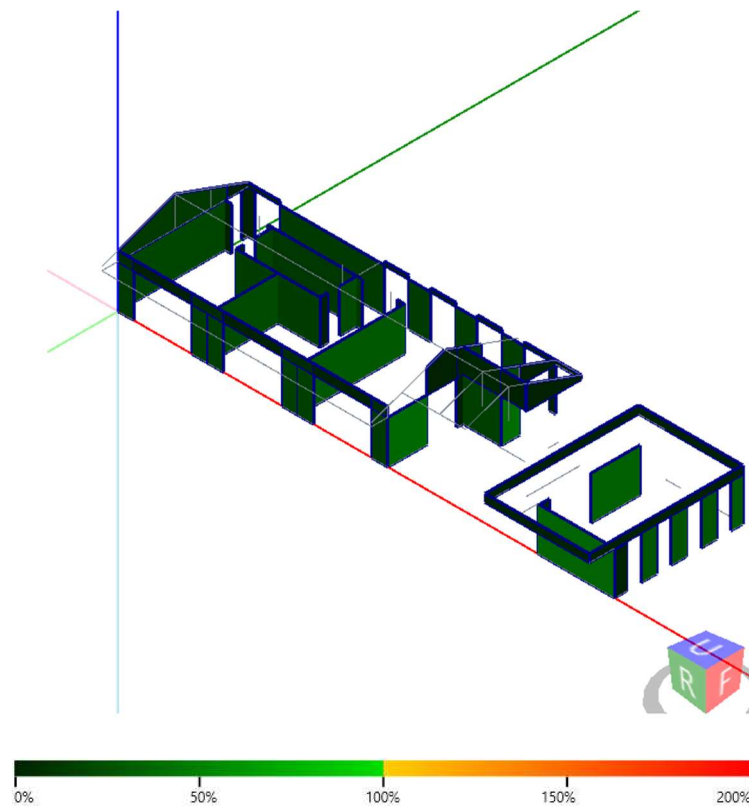
Buckling verifications



Compression perpendicular to the grain verifications

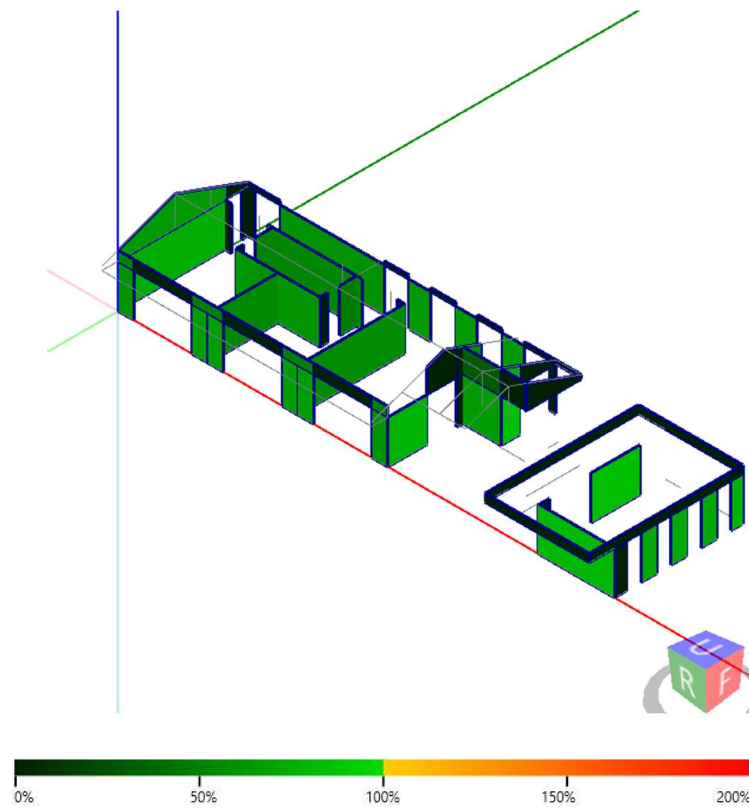


Shear verifications



Life Safety Limit State (SLV dynamic)

Shear verifications



Sections of the elements in the fire situation

According to clause 4.2.2 of EN 1995-1-2 an effective cross-section should be calculated by reducing the initial cross-section by the effective charring depth d_{ef}

$$d_{ef} = d_{char} + k_0 \cdot d_0$$

where:

d_{char} is the charring depth which, depending on the case, may be conventional ($d_{char,n}$) or one-dimensional ($d_{char,0}$);

$k_0 \cdot d_0$ is the thickness of the residual section, close to the char line, for which the material is assumed to have zero strength and stiffness, while the strength and stiffness properties of the remaining cross-section are assumed to be unchanged.

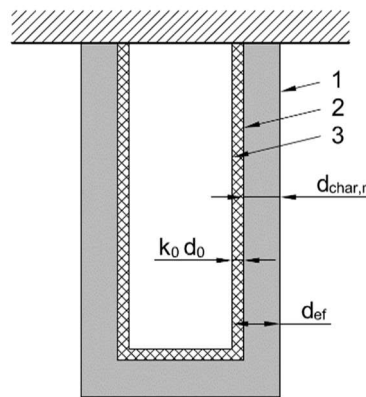


Figure: Definition of residual cross-section and effective cross-section. 1 - Initial surface of member; 2 - Border of residual cross-section; 3 - Border of effective cross-section

Charring depth

Unprotected surfaces – structural linear elements

The notional charring rate, the magnitude of which includes for the effect of corner rounding and fissures is taken as constant with time. The notional design charring depth is calculated as:

$$d_{char,n} = \beta_n \cdot t$$

where:

β_n is the notional design charring rate;

t is the time of fire exposure.

Unprotected surfaces – CLT

For CLT elements made with **fire resistant adhesives** the charring rate is taken as constant in the transition from one layer to another. The charring depth is calculated as:

$$d_{char,0} = \beta_{0,CLT} \cdot t$$

where:

β_0 is the CLT one-dimensional charring rate;

t is the time of fire exposure.

For CLT elements made with **adhesives not resistant to fire** the charring rate is considered to vary in the passage from one layer to the underlying one. In fact, it is considered that the heat-insulating char layer may fall off and, as the protective function of this layer is lost, an increase in the carbonization rate occurs. Only when the new carbonized layer reaches a thickness of 25 mm it can offer an effective protection and the charring rate decreases.

For the **first layer** the charring depth is calculated as:

$$d_{char,0} = \beta_{0,CLT} \cdot t$$

where:

$\beta_{0,CLT}$ is the CLT one-dimensional charring rate;

t is the time of fire exposure.

For the **second and subsequent layers**, until the charring depth of the individual layer exceeds 25 mm, the charring rate is:

$$\beta_{0,XLAM,k3} = k_3 \cdot \beta_{0,CLT}$$

where $k_3 = 2$.

For the **second and subsequent layers**, after the charring depth of the individual layer exceeds 25 mm, the charring rate is $\beta_{0,CLT}$.

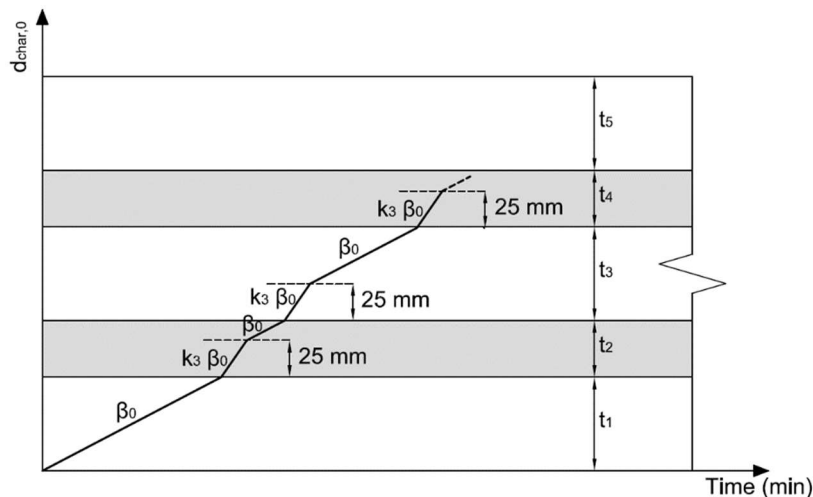


Figure: Diagram illustrating the variation of the charring rate in the passage from one layer to the underlying one of a CLT element with adhesives not resistant to fire.

Surfaces initially protected from fire exposure

For surfaces protected by fire protective sheathings, other protection materials or by other structural members, it should be taken into account that:

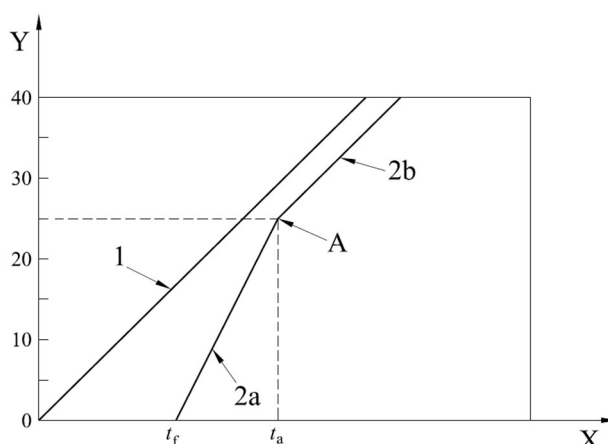
- the **start of charring** is delayed until time t_{ch} ;
- charring may commence prior to failure of the fire protection, but at a lower rate than the charring rate of the unprotected element until **failure time t_f** of the **fire protection**;
- after failure time t_f of the fire protection, the charring rate is increased above the values the charring rate of the unprotected element until the time t_a ;
- at the time t_a when the charring depth equals either the charring depth of the same member without fire protection or 25 mm whichever is the lesser, the charring rate reverts to the value of the unprotected element.

These rules are shown graphically in the following figures.

Variation of charring depth with time when $t_{ch} = t_f$ and the charring depth at time t_a is at least 25 mm

Key

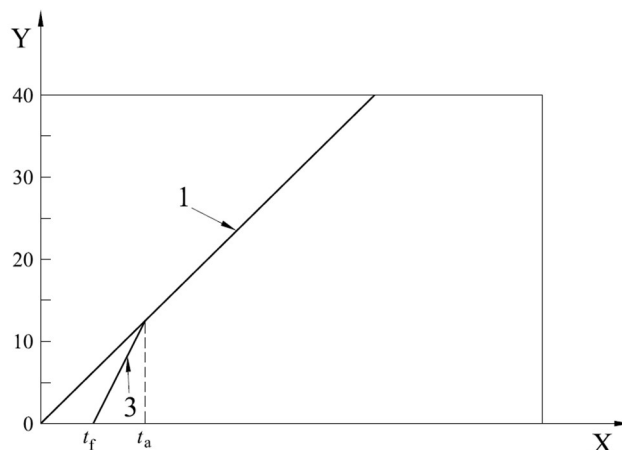
- | | |
|----|---|
| 1 | Relationship for members unprotected throughout the time of fire exposure for charring rate β_n (or β_0) |
| 2 | Relationship for initially protected members after failure of the fire protection |
| 2a | After the fire protection has fallen off, charring starts at increased rate |
| 2b | After char depth exceeds 25 mm charring rate reduces to the rate of the unprotected element |
| X | Time t |
| Y | Charring depth $d_{char,0}$ or $d_{char,n}$ in millimeters |
| A | $d_{char,0} = 25$ mm or $d_{char,n} = 25$ mm |



Variation of charring depth with time when $t_{ch} = t_f$ and the charring depth at time t_a is less than 25 mm

Key

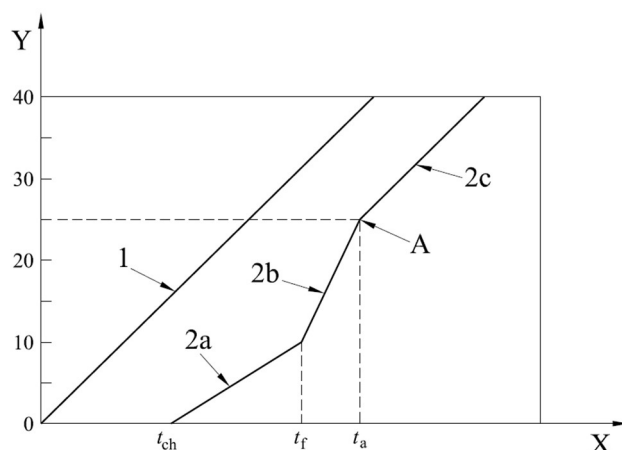
- 1 Relationship for members unprotected throughout the time of fire exposure
- 3 Relationship for initially protected members with failure times of fire protection t_f and time limit t_a smaller than given by expression $\frac{25}{k_3\beta_n} + t_f$



Variation of charring depth with time when $t_{ch} < t_f$

Key

- 1 Relationship for members unprotected throughout the time of fire exposure for charring rate β_n (or β_0)
- 2 Relationship for initially protected members where charring starts before failure of protection:
- 2a Charring starts at t_{ch} at a reduced rate when protection is still in place
- 2b After protection has fallen off, charring starts at increased rate
- 2c After char depth exceeds 25 mm charring rate reduces to the rate of the unprotected element
- X Time t
- Y Charring depth $d_{char,0}$ or $d_{char,n}$ in millimeters
- A $d_{char,0} = 25$ mm or $d_{char,n} = 25$ mm



Surfaces initially protected - CLT

For the protected CLT elements the same rules than the unprotected CLT elements are adopted, but for the first layer the rules of the protected elements shown in the previous figures are adopted.

Values of k_0 and d_0

The value of the depth of layer with assumed zero strength and stiffness is taken 7 mm.

For unprotected surfaces, k_0 should be determined from table below

ime	k_0
$t < 20$ minutes	$t/20$
$t \geq 20$ minutes	1,0

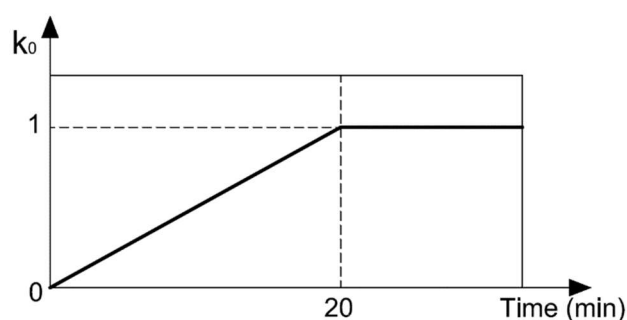


Figure: Variation of k_0 for unprotected members.

For protected surfaces with $t_{ch} > 20$ minutes, it should be assumed that k_0 varies linearly from 0 to 1 during the time interval from $t = 0$ to $t = t_{ch}$, see figure below. For protected surfaces with $t_{ch} \leq 20$ minutes the table of the unprotected elements applies.

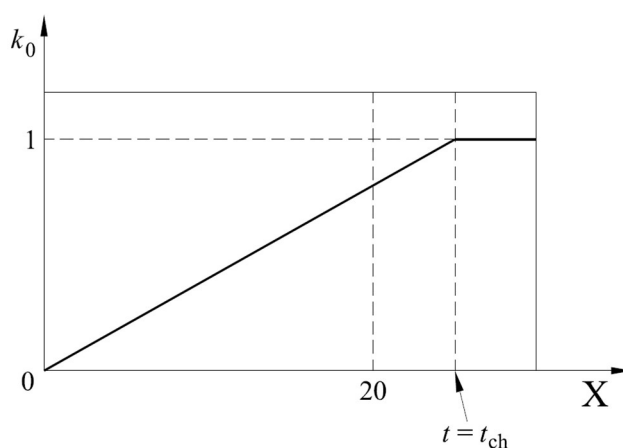


Figure: Variation of k_0 for protected members where $t_{ch} > 20$ minutes. X time (minutes)

Fire protection types

Gypsum Plasterboard

The following tables shows the characteristics of the Gypsum Plasterboard sheathings.

Fire protection name	Material	Protective sheathing	Internal layer thickness h_{p1} [mm]	External layer thickness h_{p2} [mm]	Unfilled gaps width >2 mm
GKF 15mm	Gypsum Plasterboard Type-F	Single	-	15	No

Calculation of the effective cross-sections

Timber beams

The following tables show the calculation of the effective cross-section of the beams in the fire situation.

Beam name	Fire data name	Fire resistance class R [min]	β_n	Fire protection name	k_2	t_{ch} [min]	t_r [min]	t_a [min]	$d_{char,n}$ [mm]
Beam 1	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 2	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 3	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 4	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 5	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 6	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 7	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 8	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 10	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 16	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 17	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 19	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 20	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 21	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 22	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 25	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 26	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 27	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 28	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 30	R30 nosilci	30	0.7	-	-	-	-	-	21
Beam 32	R60 nosilci	60	0.7	-	-	-	-	-	42
Beam 35	R60 nosilci	60	0.7	-	-	-	-	-	42

Beam name	Section name	$d_{char,n}$ [mm]	d_{eff} [mm]	b_{eff} [mm]	h_{eff} [mm]
Beam 1	200x440 GL24h	21	28	144	412
Beam 2	200x440 GL24h	21	28	144	412
Beam 3	200x440 GL24h	21	28	144	412
Beam 4	200x200 GL24h	21	28	144	172
Beam 5	200x200 GL24h	21	28	144	172
Beam 6	200x200 GL24h	21	28	144	172
Beam 7	200x200 GL24h	21	28	144	172
Beam 8	200x200 GL24h	21	28	144	172
Beam 10	200x240 GL24h	21	28	144	212
Beam 16	200x200 GL24h	21	28	144	172
Beam 17	260x280 GL28h	21	28	204	252
Beam 19	260x280 GL28h	21	28	204	252
Beam 20	200x400 GL28h	21	28	144	372
Beam 21	200x400 GL28h	21	28	144	372
Beam 22	200x400 GL28h	21	28	144	372
Beam 25	200x400 GL28h	21	28	144	372
Beam 26	200x240 GL24h	21	28	144	212
Beam 27	200x240 GL24h	21	28	144	212
Beam 28	200x200 GL24h	21	28	144	172
Beam 30	200x240 GL24h	21	28	144	212
Beam 32	200x520 GL24h	42	49	102	471
Beam 35	200x520 GL24h	42	49	102	471

Timber columns

The following tables show the calculation of the effective cross-section of the columns in the fire situation.

Column name	Fire data name	Fire resistance class R [min]	β_n	Fire protection name	k_2	t_{ch} [min]	t_r [min]	t_a [min]	$d_{char,n}$ [mm]
Column 2	R30 stebri	30	0.7	-	-	-	-	-	21
Column 3	R30 stebri	30	0.7	-	-	-	-	-	21
Column 6	R30 stebri	30	0.7	-	-	-	-	-	21
Column 7	R30 stebri	30	0.7	-	-	-	-	-	21
Column 8	R30 stebri	30	0.7	-	-	-	-	-	21
Column 13	R30 stebri	30	0.7	-	-	-	-	-	21
Column 14	R30 stebri	30	0.7	-	-	-	-	-	21

Column name	Section name	$d_{char,n}$ [mm]	d_{eff} [mm]	b_{eff} [mm]	h_{eff} [mm]
Column 2	200x200 GL24h	21	28	144	144
Column 3	200x200 GL24h	21	28	144	144
Column 6	200x200 GL24h	21	28	144	144
Column 7	200x200 GL24h	21	28	144	144
Column 8	200x200 GL24h	21	28	144	144
Column 13	200x200 GL24h	21	28	144	144
Column 14	200x200 GL24h	21	28	144	144

Joist floors

The following tables show the calculation of the effective cross-section of the elements of the joist floors in the fire situation.

Floor name	Fire data name	Fire resistance class R [min]	β_n	Fire protection name	k_2	t_{ch} sides up and down [min]	t_r sides up and down [min]	t_a sides up and down [min]	$d_{char,n}$ sides up and down [mm]	t_{ch} vertical sides [min]	t_r vertical sides [min]	t_a vertical sides [min]	$d_{char,n}$ vertical sides [mm]
Floor 3	R30 stropi	30	0.8	-	-	-	-	-	24	-	-	-	24
Floor 10	R30 stropi	30	0.8	-	-	-	-	-	24	-	-	-	24

Floor name	Section name	$d_{char,n}$ sides up and down [mm]	$d_{char,n}$ vertical sides [mm]	d_{eff} sides up and down [mm]	d_{eff} vertical sides [mm]	Cavities filled with non-combustible insulation	b_{eff} [mm]	h_{eff} [mm]
Floor 3	Spirovci 100x180 e=920	24	24	31	31	No	38	149
Floor 10	Spirovci 100x180 e=920	24	24	31	31	No	38	149

CLT Floors

The following tables show the calculation of the effective cross-section of the elements of the CLT floors in the fire situation.

Floor name	Fire data name	Glue line integrity maintained	Fire resistance class R [min]	β_0	Fire protection name	k_2	t_{ch} [min]	t_r [min]	t_s [min]	$d_{char,0}$ [mm]
Floor 9	R30 strop	No	30	0.65	-	-	-	-	-	20
Floor 11	R30 strop	No	30	0.65	-	-	-	-	-	20
Floor 13	R60 strop	No	60	0.65	-	-	-	-	-	39

Floor name	Section name	$d_{char,0}$ [mm]	d_{eff} [mm]	b_{eff} [mm]	h_{eff} [mm]	Fire stratigraphy
Floor 9	BBS 140 5s	20	27	500	114	40 - 20 - 20 - 20 - 14
Floor 11	BBS 140 5s	20	27	500	114	40 - 20 - 20 - 20 - 14
Floor 13	BBS 220 5s	39	46	500	174	60 - 30 - 40 - 30 - 14

Framed walls

The following tables show the calculation of the effective cross-section of the elements of the framed walls in the fire situation.

Wall name	Fire data name	Fire resistance class R [min]	β_n	Fire protection name	k_2	t_{ch} external sides [min]	t_r external sides [min]	t_a external sides [min]	$d_{char,n}$ external sides [mm]	t_{ch} internal sides [min]	t_r internal sides [min]	t_a internal sides [min]	$d_{char,n}$ internal sides [mm]
Wall 1	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 2	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 3	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 4	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 5	R60 zunanje stene	60	0.8	-	-	-	-	-	48	-	-	-	48
Wall 6	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 7	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 8	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 9	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 10	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 11	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 12	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 13	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 14	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 15	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 16	R60 zunanje stene	60	0.8	-	-	-	-	-	48	-	-	-	48
Wall 17	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 18	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 19	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 20	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 21	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 22	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 23	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 24	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 25	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 26	R30 notranje stene zascita	30	0.8	GKF 15mm	0.73	28	37	49	1	37	37	53	0
Wall 27	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 29	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 31	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 33	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 35	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 37	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 40	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 41	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 42	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 43	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 44	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 45	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 46	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 47	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 48	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 50	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 51	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 52	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 53	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 54	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 55	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 56	R30 notranje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 57	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 58	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 60	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 64	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 65	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 66	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 67	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 68	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24
Wall 69	R30 zunanje stene	30	0.8	-	-	-	-	-	24	-	-	-	24

Wall name	Section name	$d_{char,n}$ external sides [mm]	$d_{char,n}$ internal sides [mm]	d_{eff} external sides [mm]	d_{eff} internal sides [mm]	Cavities filled with non-combustible insulation	$b_{eff,internal}$ [mm]	$b_{eff,external}$ [mm]	h_{eff} [mm]
Wall 1	Okvir 200 ext=160	24	24	31	31	Yes	80	160	169
Wall 2	Okvir 200 ext=160	24	24	31	31	Yes	80	160	169
Wall 3	Okvir 200 ext=160	24	24	31	31	Yes	80	160	169
Wall 4	Okvir 200 ext=160	24	24	31	31	Yes	80	160	169
Wall 5	Okvir 200	48	48	55	55	Yes	80	80	145
Wall 6	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 7	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 8	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 9	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 10	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 11	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 12	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 13	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 14	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 15	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 16	Okvir 200	48	48	55	55	Yes	80	80	145
Wall 17	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 18	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 19	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 20	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 21	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 22	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 23	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 24	Okvir 140	24	24	31	31	Yes	80	80	78

Wall 25	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 26	Okvir 100 e=417mm	1	0	8	6	Yes	80	80	84
Wall 27	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 29	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 31	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 33	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 35	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 37	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 40	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 41	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 42	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 43	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 44	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 45	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 46	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 47	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 48	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 50	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 51	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 52	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 53	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 54	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 55	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 56	Okvir 140	24	24	31	31	Yes	80	80	78
Wall 57	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 58	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 60	Okvir 200 ext=160	24	24	31	31	Yes	80	160	169
Wall 64	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 65	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 66	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 67	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 68	Okvir 200	24	24	31	31	Yes	80	80	169
Wall 69	Okvir 200	24	24	31	31	Yes	80	80	169

Fire resistance checks

Design values of material properties and resistances

According to 2.3 of EN 1995-1-2, for verification of mechanical resistance, the design values of strength properties of the material shall be determined from

$$f_{d,fi} = k_{mod,fi} \cdot \frac{f_{20}}{\gamma_{M,fi}}$$

where:

$f_{d,fi}$ is the design strength in fire;

f_{20} is the 20% fractile of a strength property at normal temperature;

$k_{mod,fi}$ is the modification factor for fire;

$\gamma_{M,fi}$ is the partial safety factor for timber in fire.

The 20% fractile of a strength or a stiffness property is calculated as:

$$f_{20} = k_{fi} \cdot f_k$$

$$S_{20} = k_{fi} \cdot S_{05}$$

where k_{fi} is given in table below (table 2.1 of EN 1995-1-2).

Material	k_{fi}
Solid timber	1.25
Glued-laminated timber	1.15
Wood-based panels	1.15
LVL	1.10
LSL	1.10

Load combinations for structural fire design

For obtaining the relevant effect of action during fire exposure, the actions are combined with the combination for accidental design situations. The representative value of the variable action Q_1 may be considered as the quasi-permanent value $\psi_{2,1} Q_1$, or as an alternative the frequent value $\psi_{1,1} Q_1$:

$$G_1 + G_2 + P + (\psi_{11} \text{ or } \psi_{21}) \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \dots$$

The used combination is the Quasi permanent.

CLT floors checks in fire situation

Bending strength

The checks are conducted according to § 6.1.6 of EN 1995-1-1 using the effective cross section according to 4.2.2 of EN 1995-1-2. The following expression shall be satisfied:

$$\frac{\sigma_{m,d,fi}}{f_{m,d,fi}} \leq 1$$

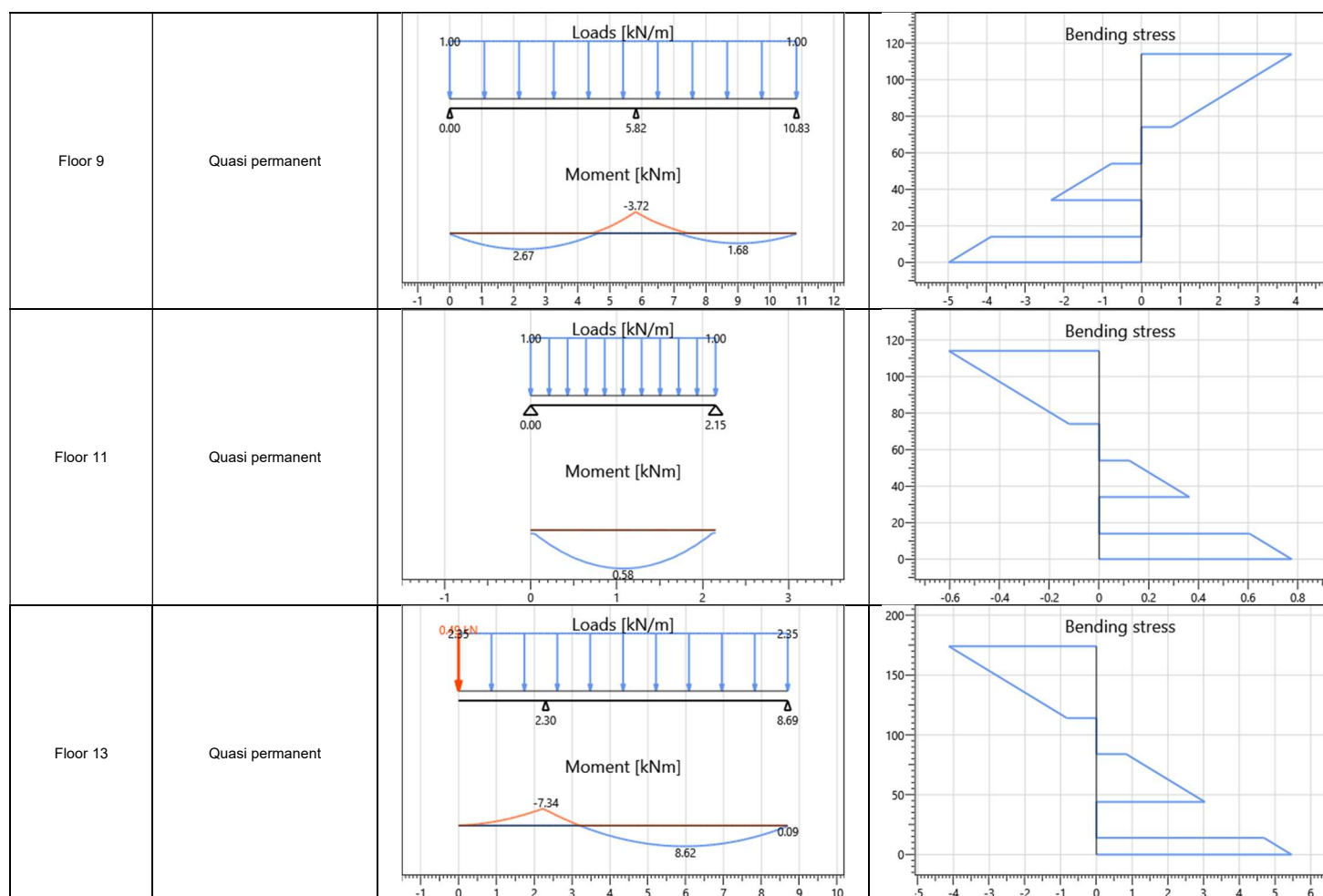
where:

$\sigma_{m,d,fi}$ the design bending stress in the fire situation;

$f_{m,d,fi}$ the design bending strength in the fire situation.

The following table illustrates the structural schemes and the envelopes of the diagram of the bending moment for the part of each floor with the most sever checks.

Floor name	Combination	Diagram M ₃₋₃	Bending stresses
------------	-------------	--------------------------	------------------



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Section	J_{eff} [mm ⁴]	M_{3-3} [kNm]	$k_{mod,fi}$	$\gamma_{M,fi}$	k_{fi}	$f_{m,d,fi}$ [MPa]	$\sigma_{m,d,fi}$ [MPa]	Check
Floor 9	BBS 140 5s	47857306	-3.72	1	1	1.15	27.60	4.98	18%
Floor 11	BBS 140 5s	47857306	0.58	1	1	1.15	27.60	0.77	3%
Floor 13	BBS 220 5s	156278368	8.62	1	1	1.15	27.60	5.47	20%

Shear strength

Shear strength of the layers parallel to the calculation direction

The checks are conducted according to § 6.1.7 of EN 1995-1-1 using the effective cross section according to 4.2.2 of EN 1995-1-2. The following expression shall be satisfied:

$$\frac{\tau_{v,d,fi}}{f_{v,d,fi}} \leq 1$$

being:

$\tau_{v,d,fi}$ the design shear stress in the fire situation;

$f_{v,d,fi}$ the design shear strength in fire.

The maximum design shear stress in the longitudinal layers can be evaluated using the following expression:

$$\tau_{v,d,fi} = \frac{V_{d,fi} \cdot S_{max,eff}}{J_{eff} \cdot b}$$

where:

$V_{d,fi}$ is the total shear force at the location in question in the fire situation;

$S_{max,eff}$ is the static moment of area of the effective CLT cross section;

J_{eff} is the moment of inertia of the effective CLT element cross section;

b is the width of the CLT element cross section ($k_{cr} = 1$).

Rolling shear strength of the transversal layers

The checks are conducted according to § 6.1.7 of EN 1995-1-1 using the effective cross section according to 4.2.2 of EN 1995-1-2. The following expression shall be satisfied:

$$\frac{\tau_{R,d,fi}}{f_{v,R,d,fi}} \leq 1$$

where:

$\tau_{R,d,fi}$ the design rolling shear stress in the fire situation;

$f_{v,R,d,fi}$ the design shear strength in fire.

The maximum design shear stress in the transversal layers can be evaluated using the following expression:

$$\tau_{R,d,fi} = \frac{V_{d,fi} \cdot S_{R,max,eff}}{J_{eff} \cdot b}$$

where:

$V_{d,fi}$ is the total shear force at the location in the fire situation;

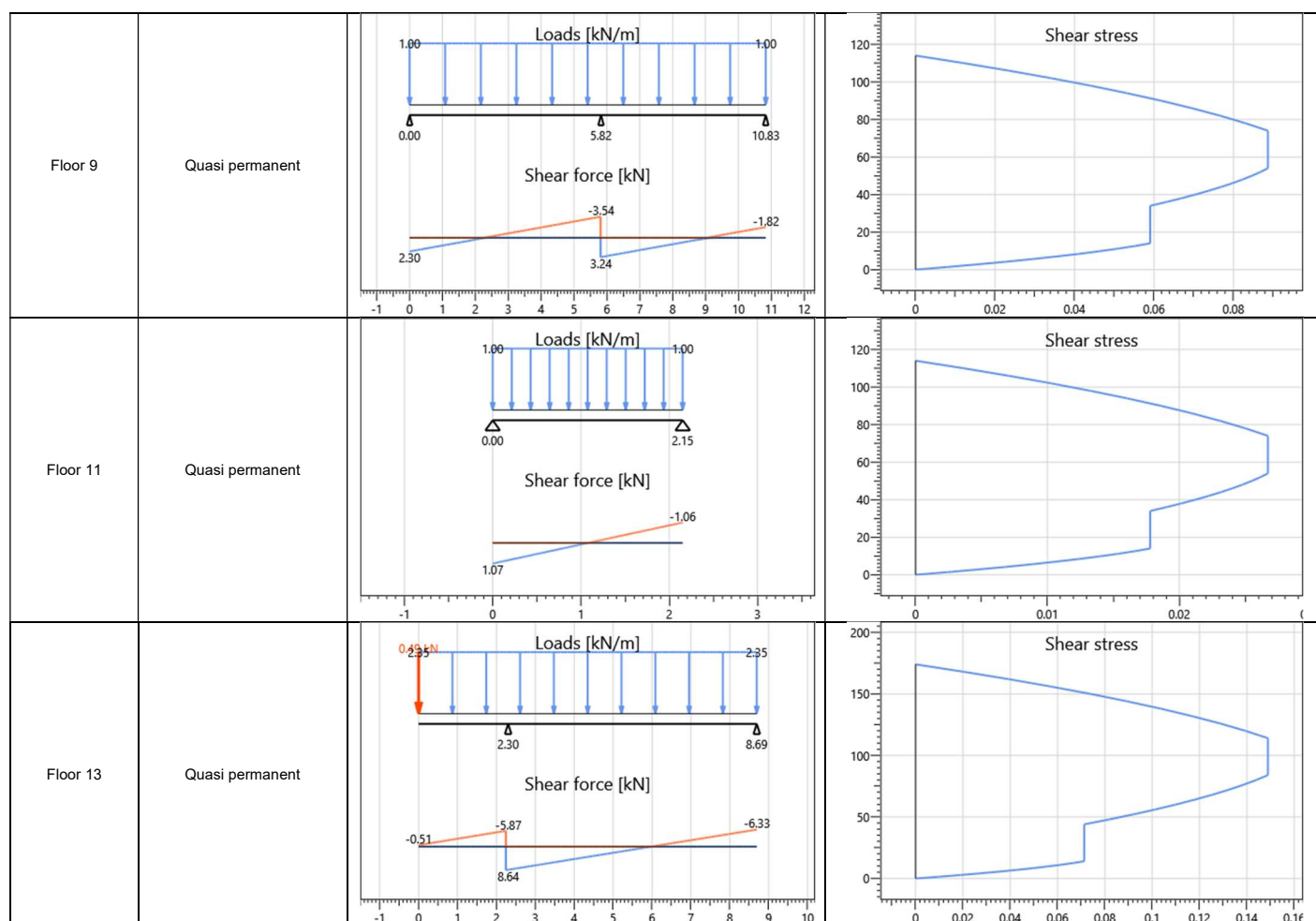
$S_{R,max,fi}$ is the first moment of area;

J_{eff} is the moment of inertia of the effective CLT element cross section;

b is the width of the CLT element cross section ($k_{cr} = 1$).

The following table illustrates the structural schemes and the envelopes of the diagram of the shear force for the part of each floor with the most sever checks.

Floor name	Combination	Diagram V_2	Shear stresses
------------	-------------	---------------	----------------



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Cross section	J_{eff} [mm ⁴]	V_2 [kN]	$k_{mod,fi}$	$\gamma_{M,fi}$	k_{fi}	$f_{v,d,fi}$ [MPa]	$\tau_{v,d,fi}$ [MPa]	Check	$f_{R,d,fi}$ [MPa]	$\tau_{R,d,fi}$ [MPa]	Check
Floor 9	BBS 140 5s	47857306	3.54	1	1	1.15	4.60	0.09	2%	1.15	0.09	8%
Floor 11	BBS 140 5s	47857306	1.07	1	1	1.15	4.60	0.03	1%	1.15	0.03	2%
Floor 13	BBS 220 5s	156278368	8.64	1	1	1.15	4.60	0.15	3%	1.15	0.15	13%

Joist floors / Solid wood floors checks in fire situation

Bending strength

The checks are conducted according to § 6.3.2 of EN 1995-1-1 using the effective cross section according to 4.2.2 of EN 1995-1-2. The following expression shall be satisfied:

$$\frac{\sigma_{m,d,fi}}{k_{crit,fi} \cdot f_{m,d,fi}} \leq 1$$

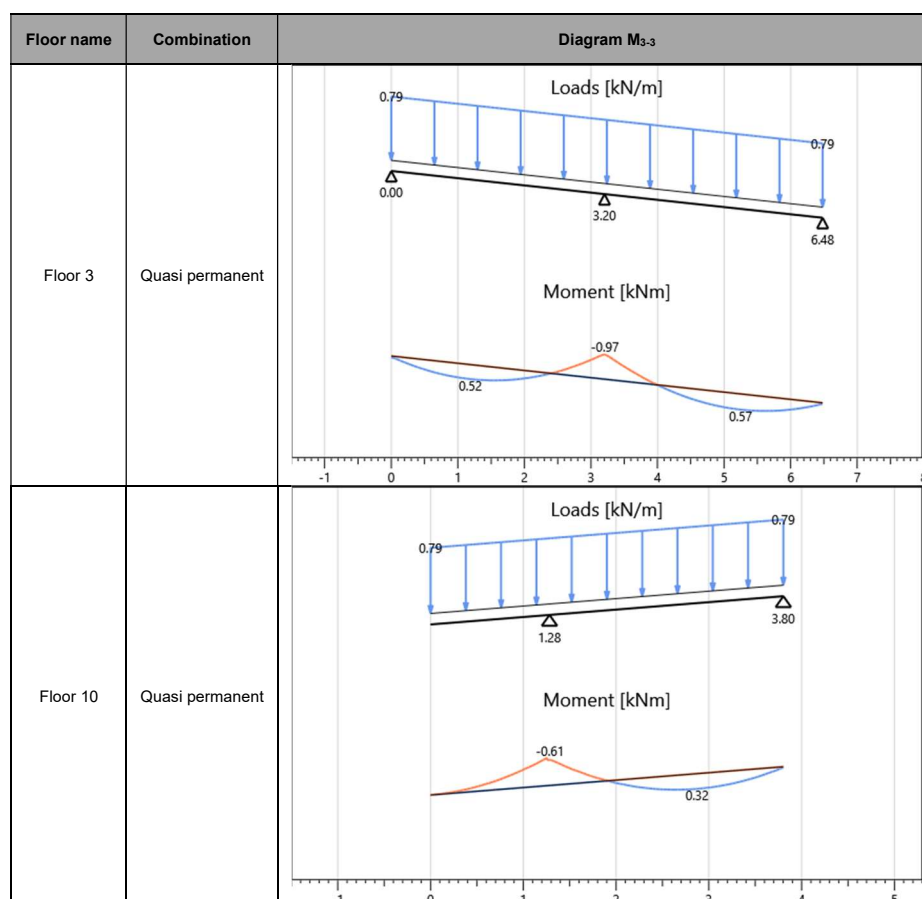
where:

$\sigma_{m,d,fi}$ is the design bending stress in the fire situation;

$f_{m,d,fi}$ is the design bending strength in fire;

k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling.

k_{crit} is assumed equal to 1.0 for beams in which the lateral displacement of the compressed edge is prevented over the entire length and the torsional rotation is prevented at the supports.



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Section	M ₃₋₃ max [kNm]	W _{y,fi} [mm ³]	k _{crit,fi}	k _h	k _{mod,fi}	γ _{M,fi}	k _{fi}	f _{m,d,fi} [MPa]	σ _{m,d,fi} [MPa]	Check
Floor 3	Spirovci 100x180 e=920	0.97	140606	1.00	1	1	1	1.25	30.00	6.89	23%
Floor 10	Spirovci 100x180 e=920	0.61	140606	1.00	1	1	1	1.25	30.00	4.32	14%

Shear strength

The checks are conducted according to § 6.1.7 of EN 1995-1-1 using the effective cross section according to 4.2.2 of EN 1995-1-2. The following expression shall be satisfied:

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} \leq 1$$

where:

$\tau_{d,fi}$ is the design shear stress in the fire situation;

$f_{v,d,fi}$ is the design shear strength in fire.

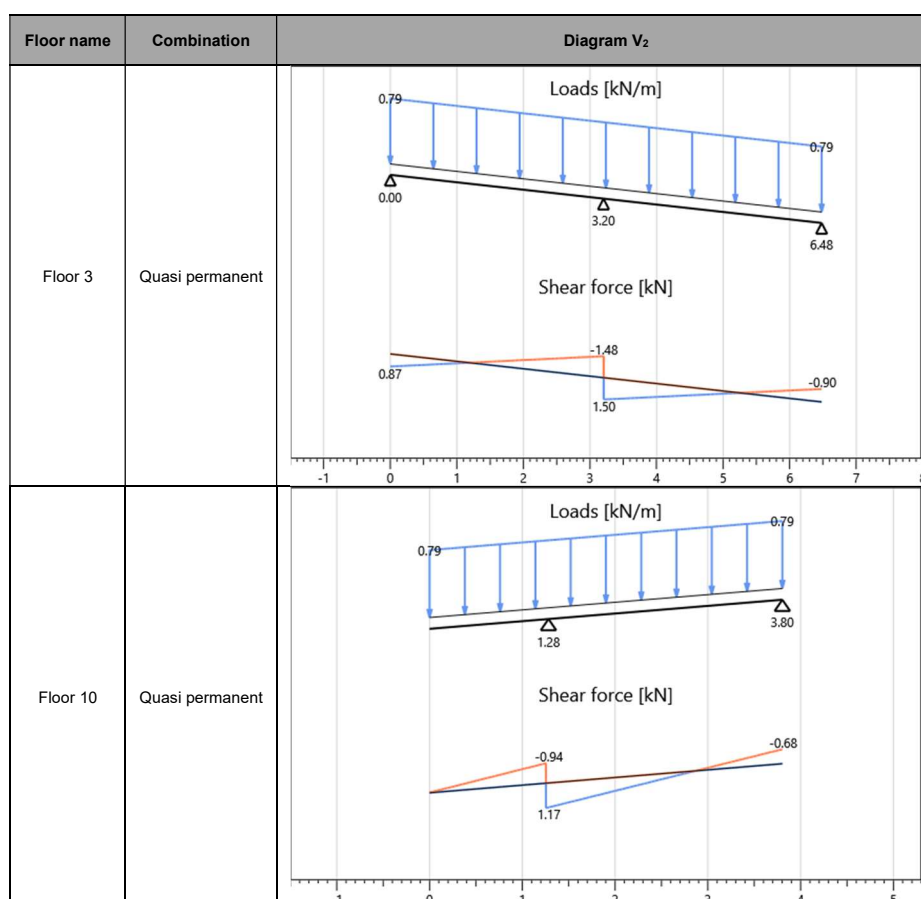
For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member $b_{ef} = k_{cr} \cdot b$.

The maximum design shear stress in a rectangular cross section can be evaluated using the following expression:

$$\tau_{d,fi} = \frac{3}{2} \cdot \frac{V_{d,fi}}{k_{cr} \cdot A_{eff}}$$

where A_{eff} is the area of a joist effective cross section.

The following table illustrates the structural schemes and the envelopes of the shear force diagram for the joist of each floor with the most sever checks.

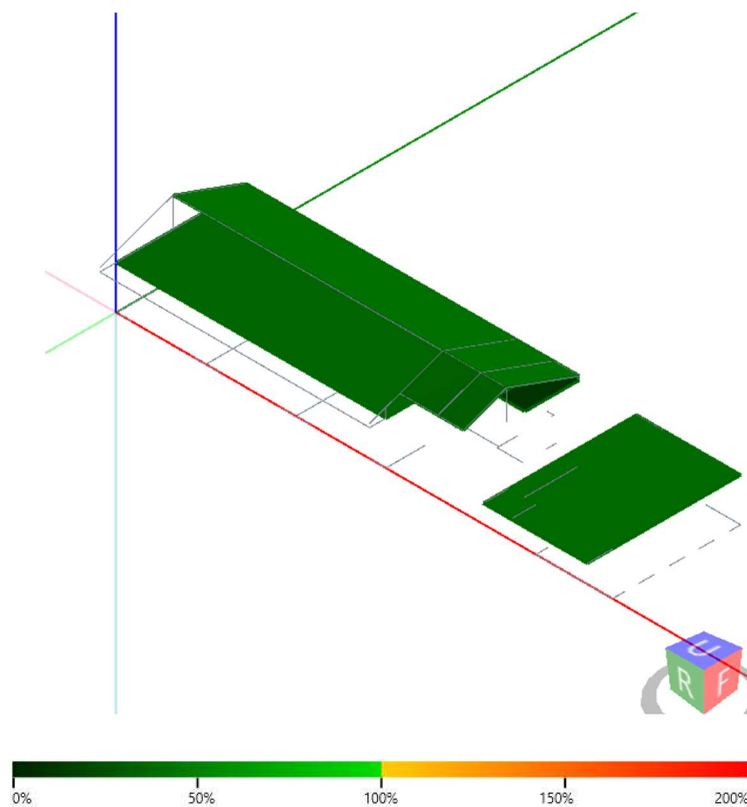


The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

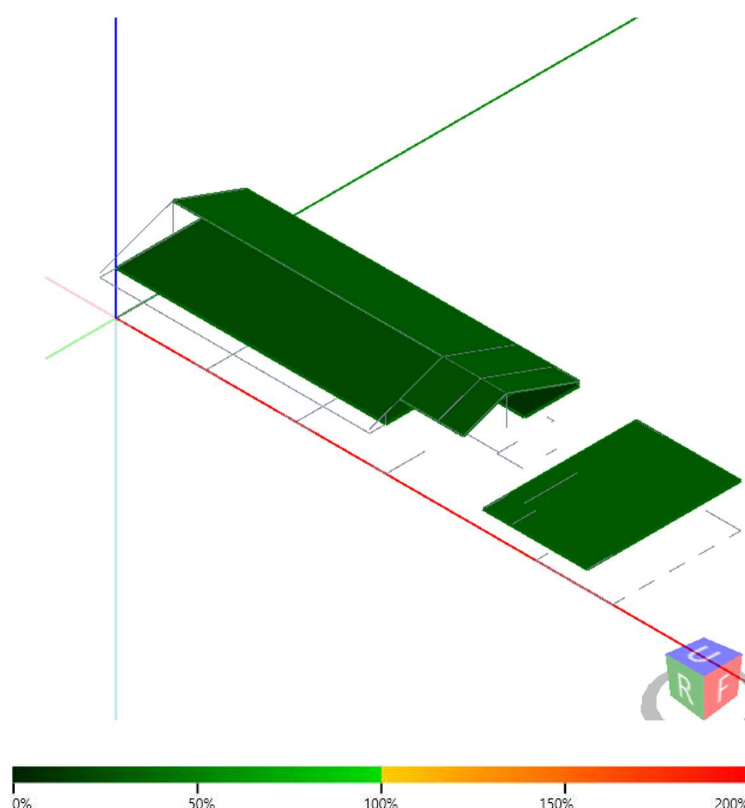
Floor name	Section	V ₂ max [kN]	A _{eff} [mm ²]	k _{cr}	k _{mod,fi}	γ _{M,fi}	k _{fi}	f _{v,d,fi} [MPa]	τ _{2,d,fi} [MPa]	Check
Floor 3	Spirovci 100x180 e=920	1.50	5662	0.67	1	1	1.25	5.00	0.59	12%
Floor 10	Spirovci 100x180 e=920	1.17	5662	0.67	1	1	1.25	5.00	0.46	9%

Verification percentage of floors in fire situation

Bending verifications



Shear verifications



Timber beams checks in fire situation

Bending strength

The checks are conducted according to § 6.3.2 of EN 1995-1-1. The following expression shall be satisfied using the effective cross sections of the elements according to 4.2.2 of EN 1995-1-2:

$$\frac{\sigma_{m,d,fi}}{k_{crit,fi} \cdot f_{m,d,fi}} \leq 1$$

where:

$\sigma_{m,d,fi}$ is the design bending stress in the fire situation;

$f_{m,d,fi}$ is the design bending strength in fire;

k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling.

The factor k_{crit} is assumed equal to 1.0 for beams in which the lateral displacement of the compressed edge is prevented over the entire length and the torsional rotation is prevented at the supports. Otherwise, the factor is determined from the following expression:

$$k_{crit,fi} = \begin{cases} 1 & \text{for } \lambda_{rel,m,fi} \leq 0,75 \\ 1,56 - 0,75\lambda_{rel,m,fi} & \text{for } 0,75 \leq \lambda_{rel,m,fi} \leq 1,4 \\ \frac{1}{\lambda_{rel,m,fi}^2} & \text{for } 1,4 < \lambda_{rel,m,fi} \end{cases}$$

in which the relative slenderness for bending is taken as:

$$\lambda_{rel,m,fi} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit,fi}}}$$

and $\sigma_{m,crit,fi}$, the critical bending stress calculated according to the classical theory of stability is taken as:

$$\sigma_{m,crit,fi} = \frac{M_{y,crit,fi}}{W_{y,fi}} = \frac{\pi \sqrt{E_{0,20} I_{z,fi} G_{0,20} I_{tor,fi}}}{l_{ef,fi} W_{y,fi}}$$

where:

$E_{0,20}$ 20% fractile of modulus of elasticity parallel to grain;

$G_{0,20}$ 20% fractile of shear modulus parallel to grain;

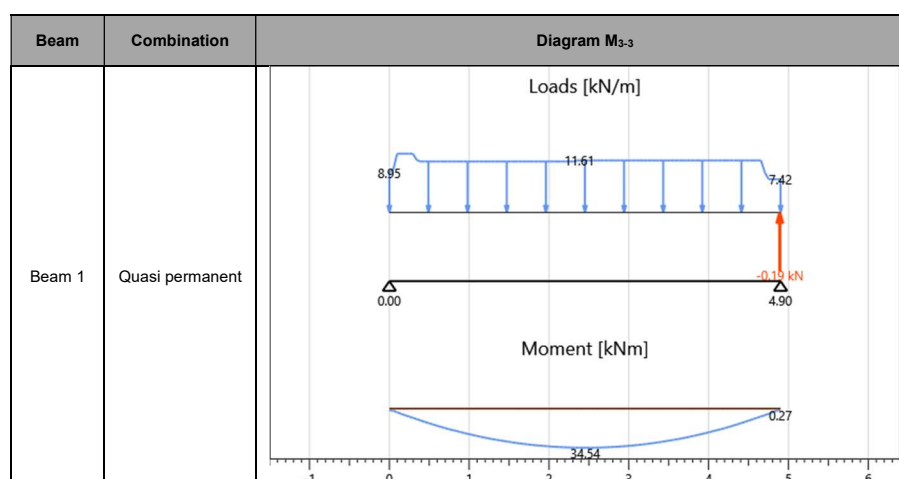
$I_{z,fi}$ is the second moment of area about the weak axis z in fire;

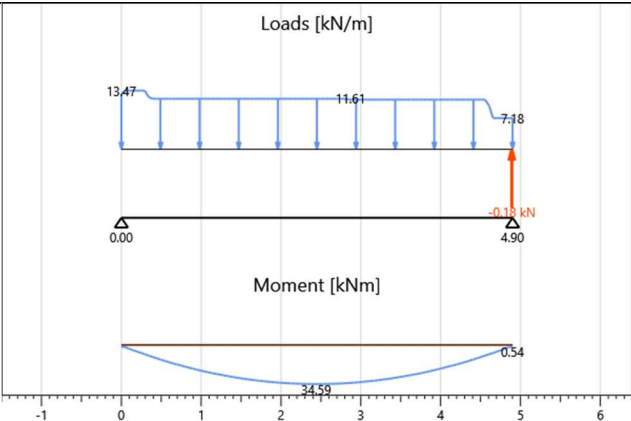
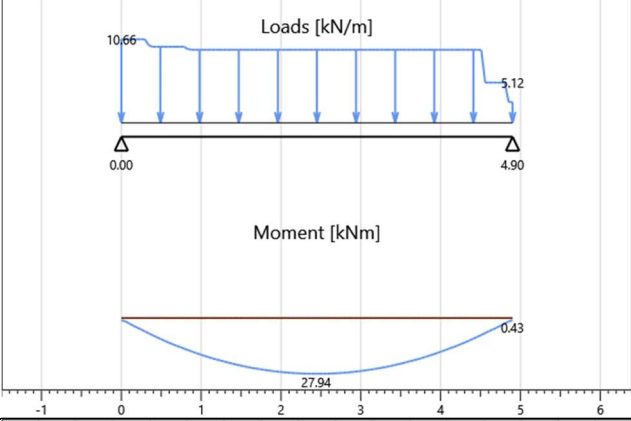
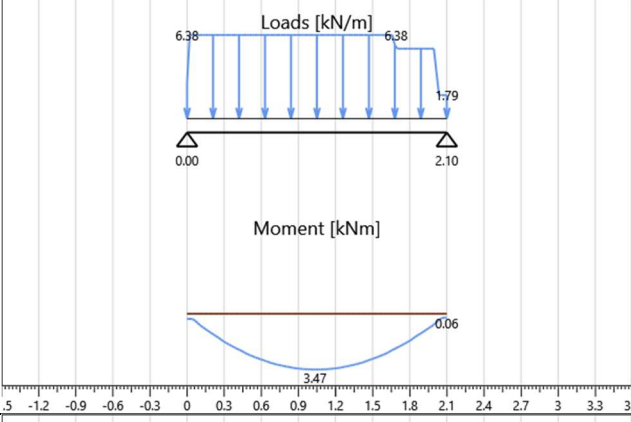
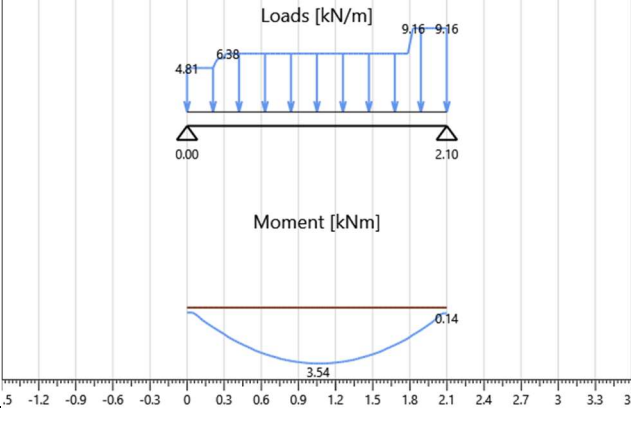
$I_{tor,fi}$ is the torsional moment of inertia in fire;

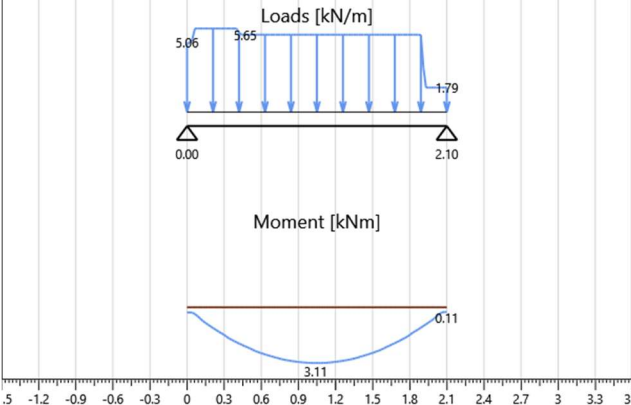
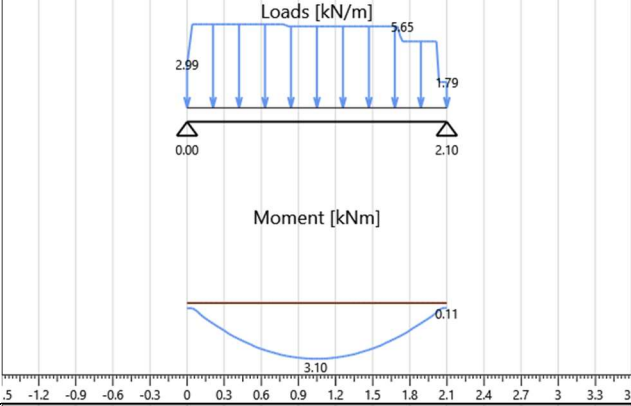
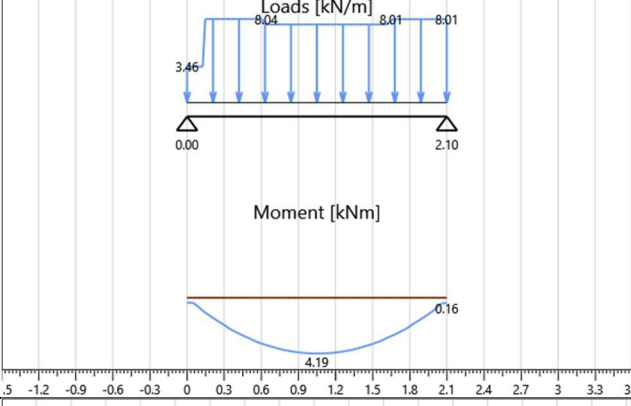
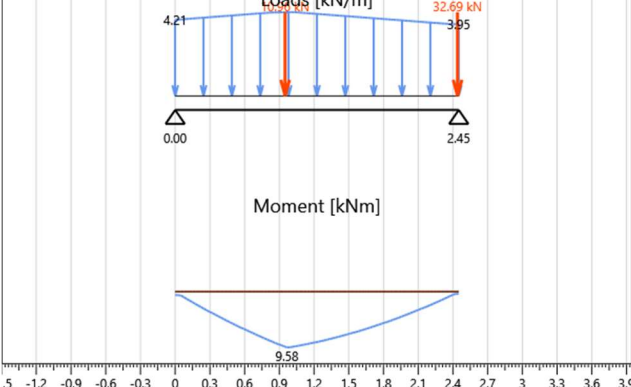
$l_{ef,fi}$ is the effective length of the beam, depending on the support conditions and the load configuration, in the fire situation;

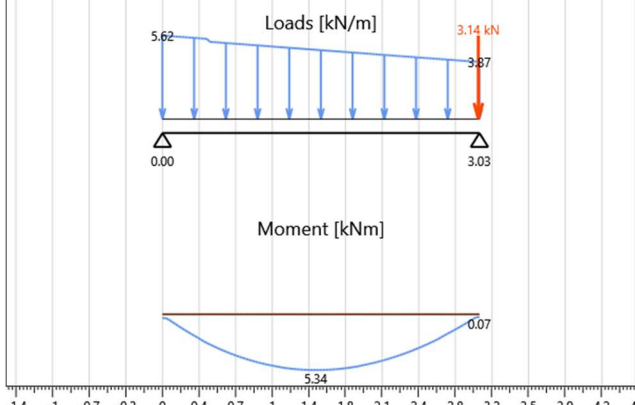
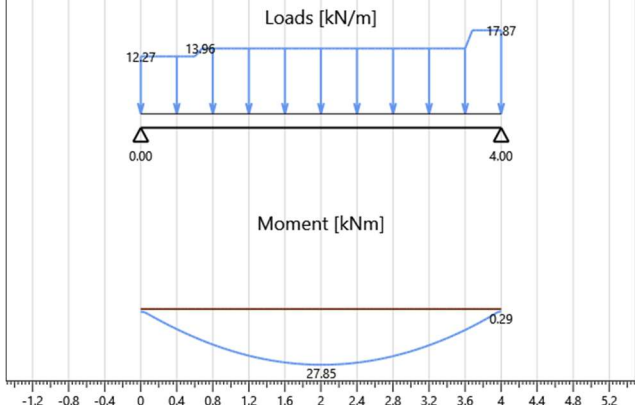
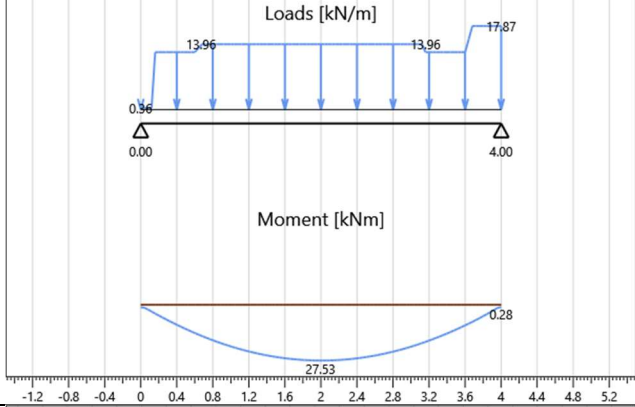
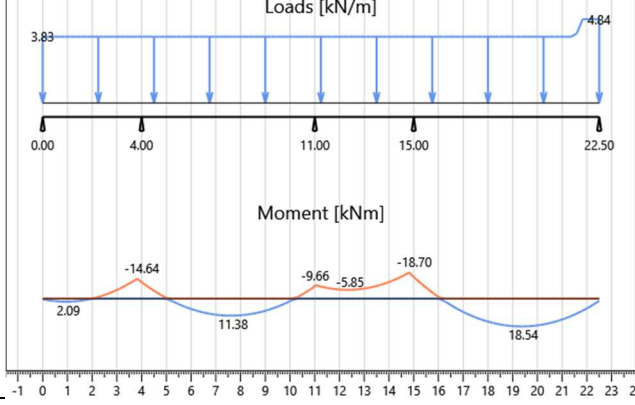
$W_{y,fi}$ is the section modulus about the strong axis y in the fire situation.

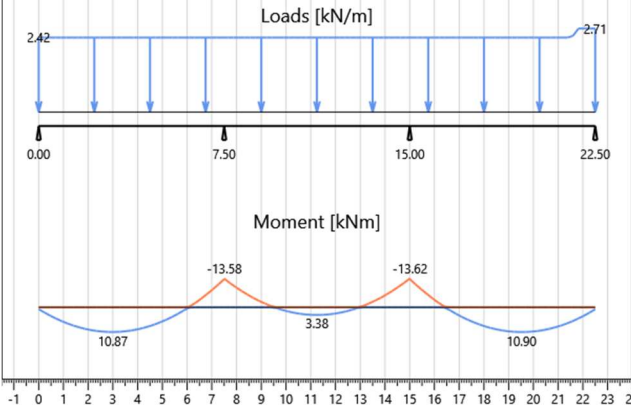
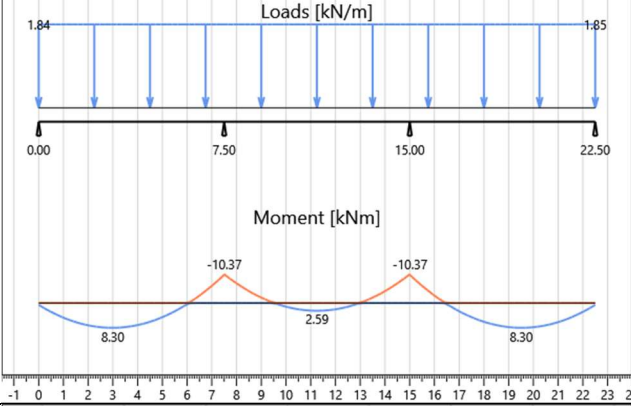
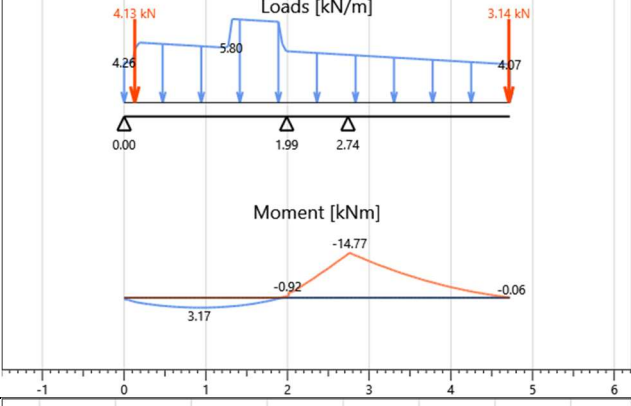
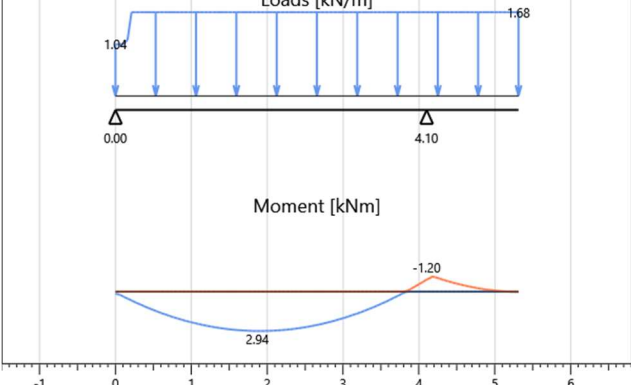
The following table shows, for each beam, the bending moment relating to the fire situation.

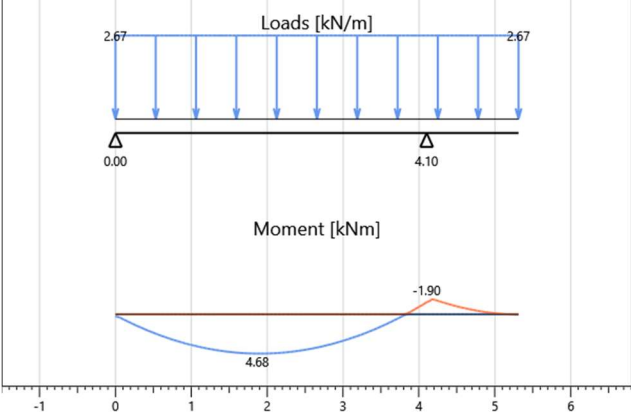
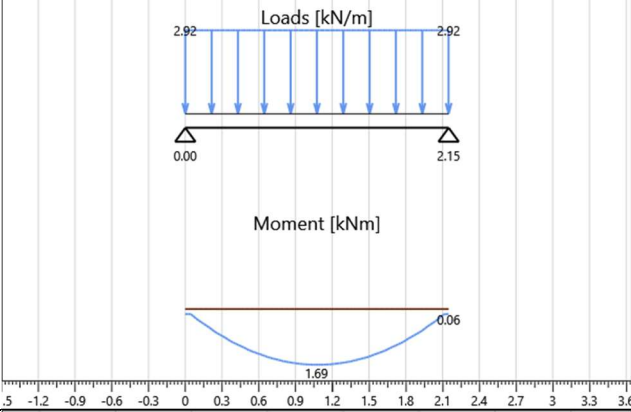
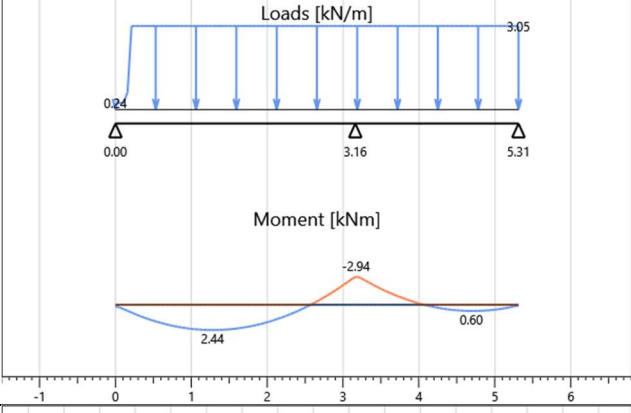
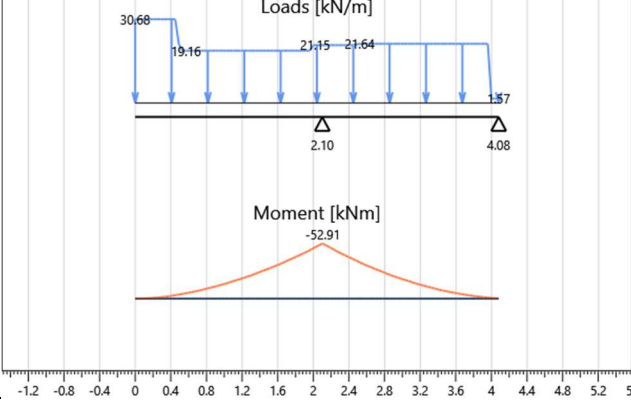


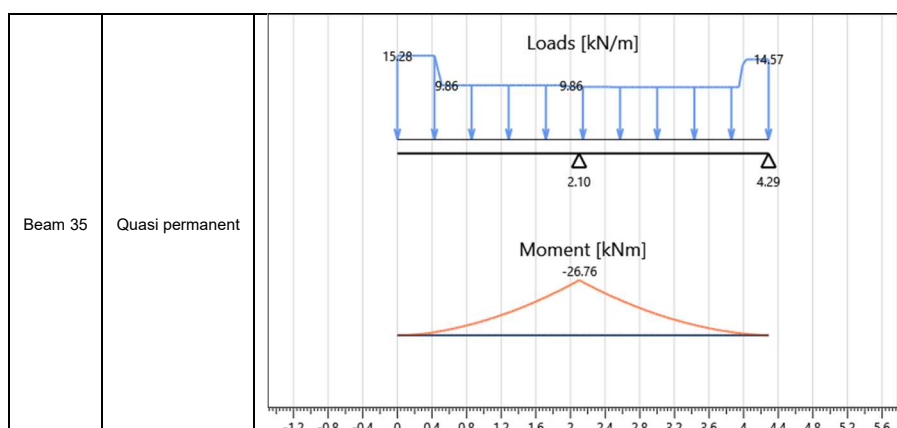
Beam 2	Quasi permanent	
Beam 3	Quasi permanent	
Beam 4	Quasi permanent	
Beam 5	Quasi permanent	

Beam 6	Quasi permanent	
Beam 7	Quasi permanent	
Beam 8	Quasi permanent	
Beam 10	Quasi permanent	

Beam 16	Quasi permanent	
Beam 17	Quasi permanent	
Beam 19	Quasi permanent	
Beam 20	Quasi permanent	

Beam 21	Quasi permanent	
Beam 22	Quasi permanent	
Beam 25	Quasi permanent	
Beam 26	Quasi permanent	

Beam 27	Quasi permanent	
Beam 28	Quasi permanent	
Beam 30	Quasi permanent	
Beam 32	Quasi permanent	



The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Beam name	Section	M _{3-3 max} [kNm]	W _{y,fi} [mm ³]	Lateral restraints	σ _{m,crit,fi} [MPa]	k _{crit,fi}	k _h	k _{mod,fi}	γ _{M,fi}	k _{fi}	f _{m,d,fi} [MPa]	σ _{m,d,fi} [MPa]	Check
Beam 1	200x440 GL24h	34.54	4073856	No torsional buckling	-	1.00	1	1	1	1.15	27.60	8.48	31%
Beam 2	200x440 GL24h	34.59	4073856	No torsional buckling	-	1.00	1	1	1	1.15	27.60	8.49	31%
Beam 3	200x440 GL24h	27.94	4073856	No torsional buckling	-	1.00	1	1	1	1.15	27.60	6.86	25%
Beam 4	200x200 GL24h	3.47	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	4.88	18%
Beam 5	200x200 GL24h	3.54	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	4.99	18%
Beam 6	200x200 GL24h	3.11	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	4.38	16%
Beam 7	200x200 GL24h	3.10	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	4.37	16%
Beam 8	200x200 GL24h	4.19	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	5.90	21%
Beam 10	200x240 GL24h	9.58	1078656	No torsional buckling	-	1.00	1	1	1	1.15	27.60	8.89	32%
Beam 16	200x200 GL24h	5.34	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	7.52	27%
Beam 17	260x280 GL28h	27.85	2159136	No torsional buckling	-	1.00	1	1	1	1.15	32.20	12.90	40%
Beam 19	260x280 GL28h	27.53	2159136	No torsional buckling	-	1.00	1	1	1	1.15	32.20	12.75	40%
Beam 20	200x400 GL28h	18.70	3321216	No torsional buckling	-	1.00	1	1	1	1.15	32.20	5.63	17%
Beam 21	200x400 GL28h	13.62	3321216	No torsional buckling	-	1.00	1	1	1	1.15	32.20	4.10	13%
Beam 22	200x400 GL28h	10.37	3321216	No torsional buckling	-	1.00	1	1	1	1.15	32.20	3.12	10%
Beam 25	200x400 GL28h	14.77	3321216	No torsional buckling	-	1.00	1	1	1	1.15	32.20	4.45	14%
Beam 26	200x240 GL24h	2.94	1078656	No torsional buckling	-	1.00	1	1	1	1.15	27.60	2.73	10%
Beam 27	200x240 GL24h	4.68	1078656	No torsional buckling	-	1.00	1	1	1	1.15	27.60	4.34	16%
Beam 28	200x200 GL24h	1.69	710016	No torsional buckling	-	1.00	1	1	1	1.15	27.60	2.37	9%
Beam 30	200x240 GL24h	2.94	1078656	No torsional buckling	-	1.00	1	1	1	1.15	27.60	2.73	10%
Beam 32	200x520 GL24h	52.91	3771297	No torsional buckling	-	1.00	1	1	1	1.15	27.60	14.03	51%
Beam 35	200x520 GL24h	26.76	3771297	No torsional buckling	-	1.00	1	1	1	1.15	27.60	7.10	26%

Shear strength

The checks are conducted according to § 6.1.7 of EN 1995-1-1 using the effective cross section according to 4.2.2 of EN 1995-1-2. The following expression shall be satisfied:

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} \leq 1$$

where:

$\tau_{d,fi}$ is the design shear stress in the fire situation;

$f_{v,d,fi}$ is the design shear strength in fire.

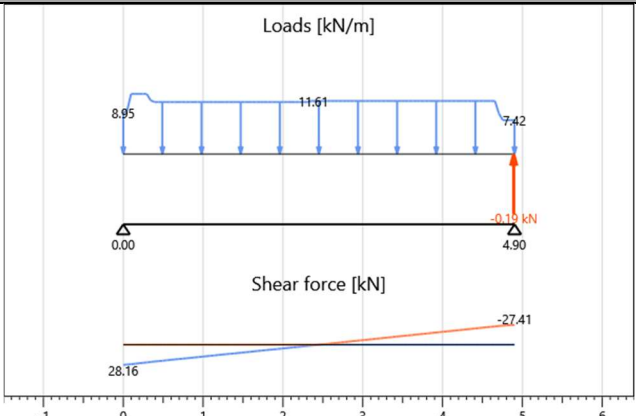
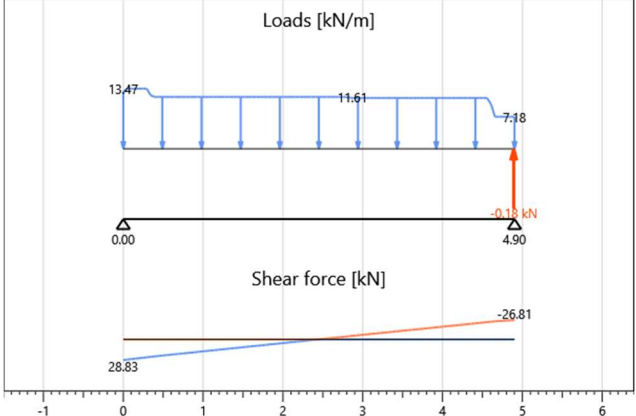
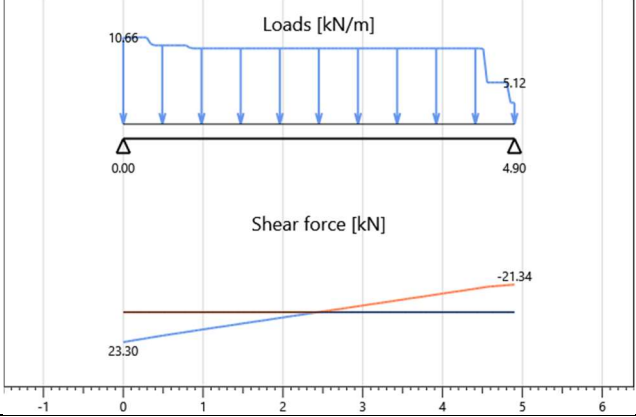
For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member $b_{ef} = k_{cr} \cdot b$.

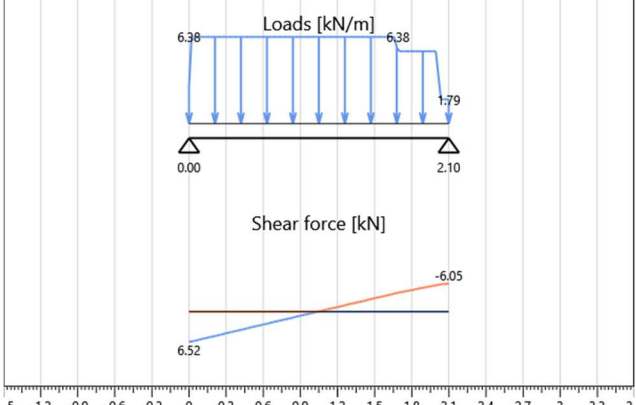
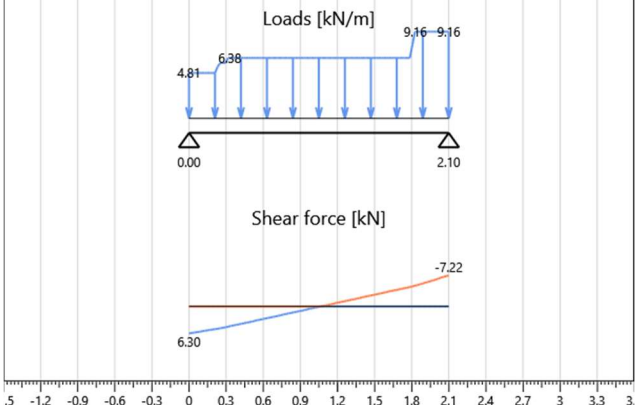
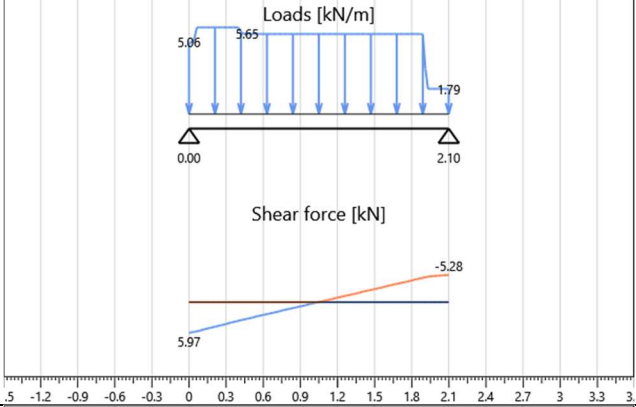
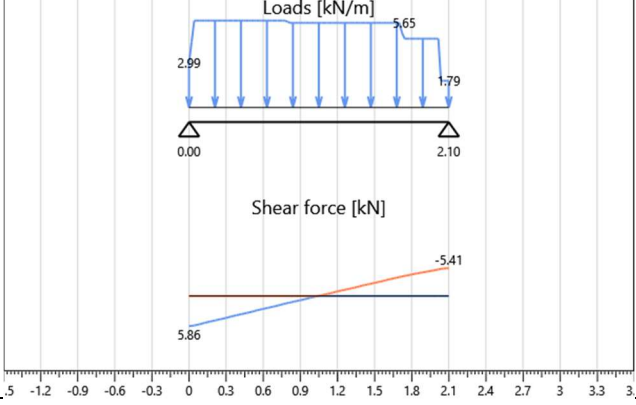
The maximum design shear stress in a rectangular cross section can be evaluated using the following expression:

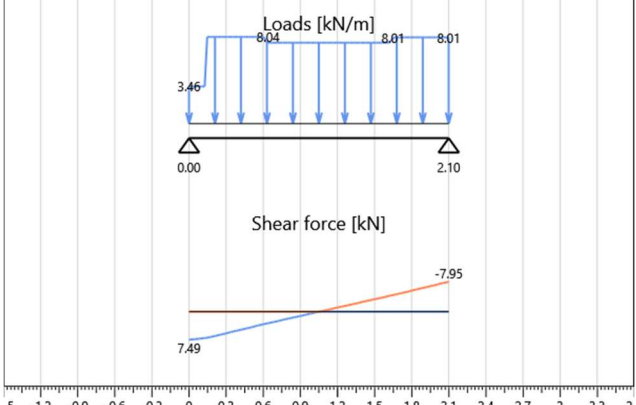
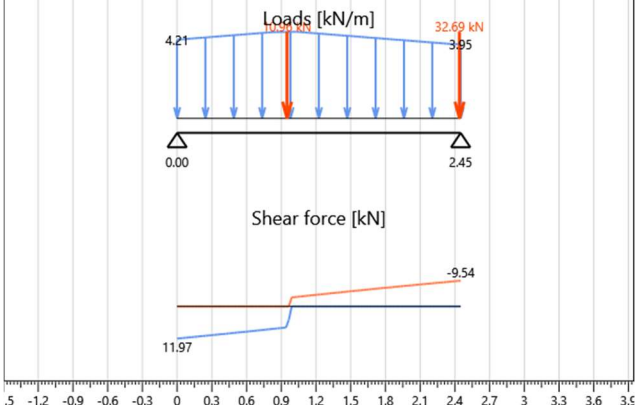
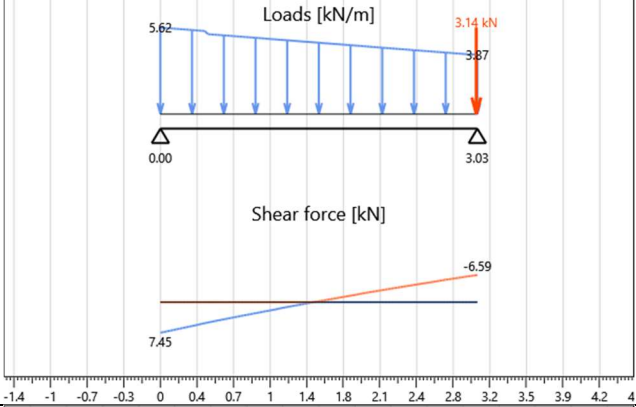
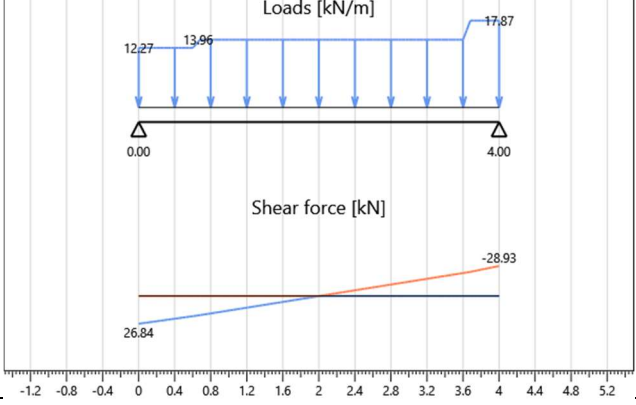
$$\tau_{d,fi} = \frac{3}{2} \cdot \frac{V_{d,fi}}{k_{cr} \cdot A_{eff}}$$

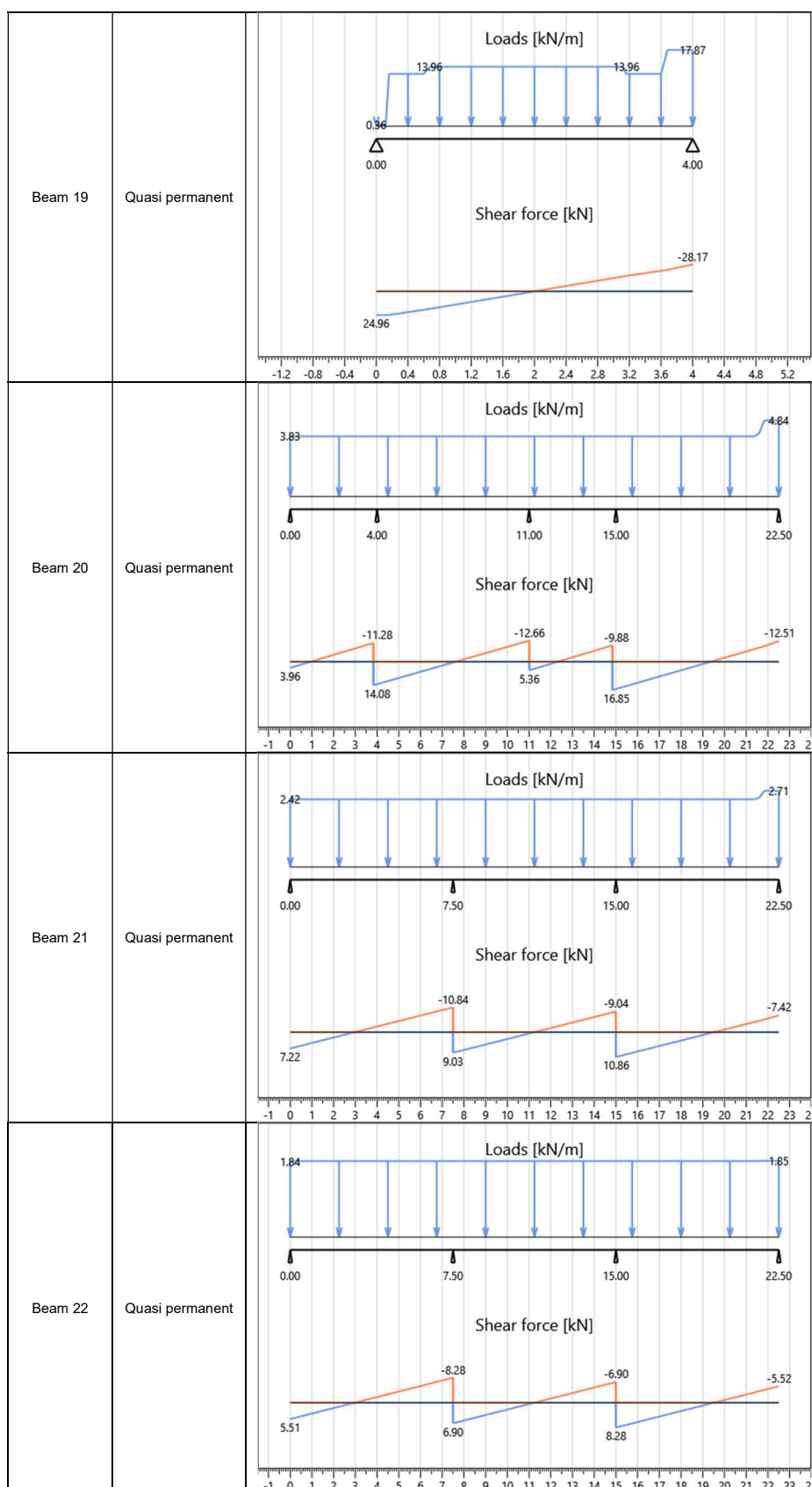
where A_{eff} is the area of the beam effective cross section.

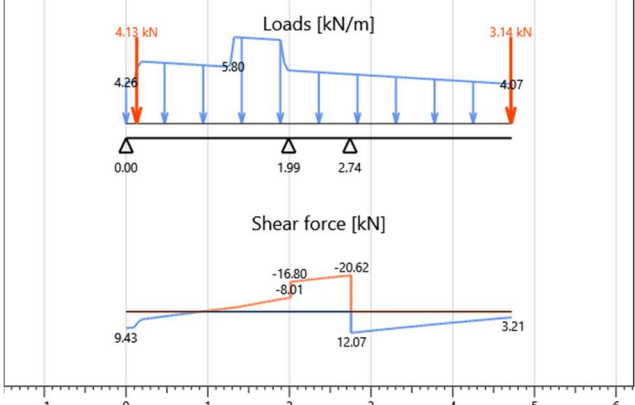
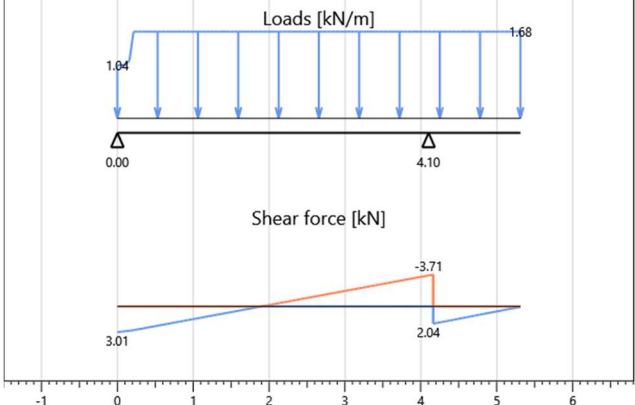
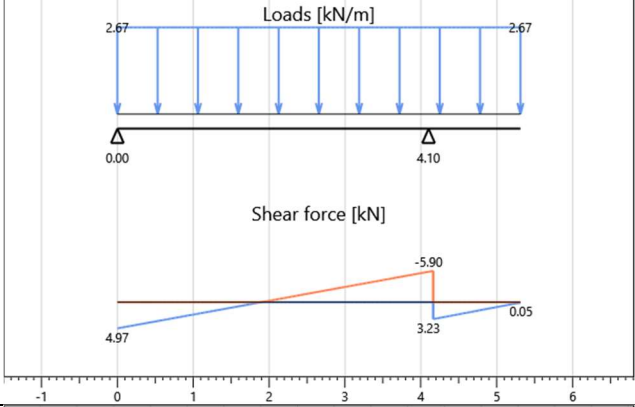
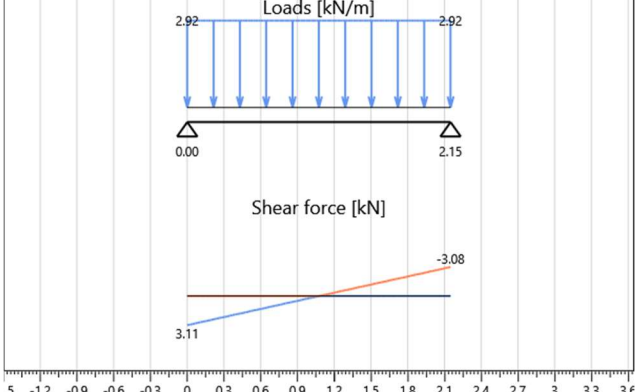
The following table illustrates the structural schemes and the envelopes of the shear force diagram for each beam.

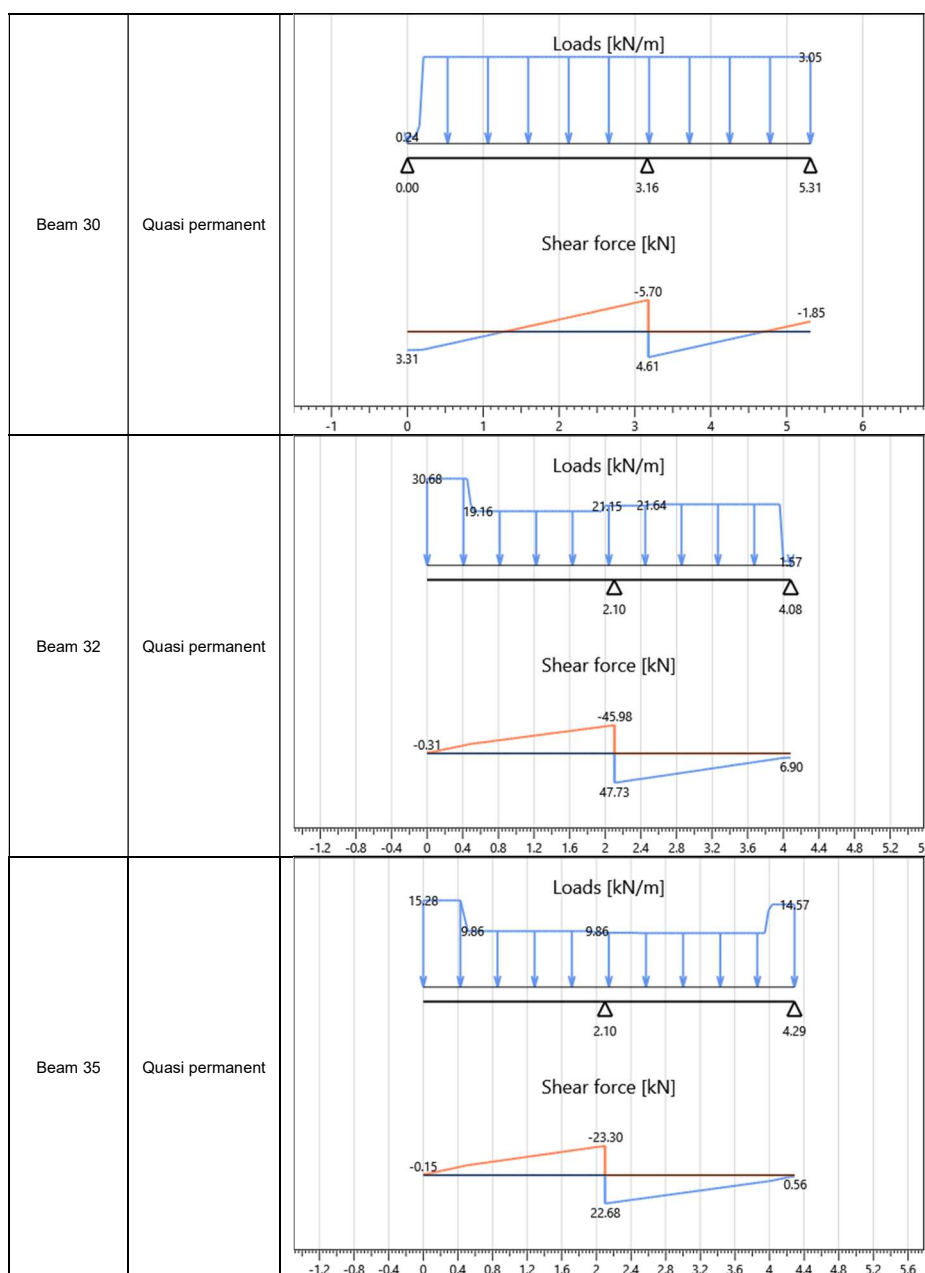
Beam name	Combination	Diagram V ₂
Beam 1	Quasi permanent	 Loads [kN/m] 8.95 11.61 7.42 0.00 4.90 Shear force [kN] 28.16 -27.41
Beam 2	Quasi permanent	 Loads [kN/m] 13.47 11.61 7.18 0.00 4.90 Shear force [kN] 28.83 -26.81
Beam 3	Quasi permanent	 Loads [kN/m] 10.66 5.12 0.00 4.90 Shear force [kN] 23.30 -21.34

Beam 4	Quasi permanent	
Beam 5	Quasi permanent	
Beam 6	Quasi permanent	
Beam 7	Quasi permanent	

Beam 8	Quasi permanent	
Beam 10	Quasi permanent	
Beam 16	Quasi permanent	
Beam 17	Quasi permanent	



Beam 25	Quasi permanent	 Loads [kN/m] Shear force [kN]
Beam 26	Quasi permanent	 Loads [kN/m] Shear force [kN]
Beam 27	Quasi permanent	 Loads [kN/m] Shear force [kN]
Beam 28	Quasi permanent	 Loads [kN/m] Shear force [kN]

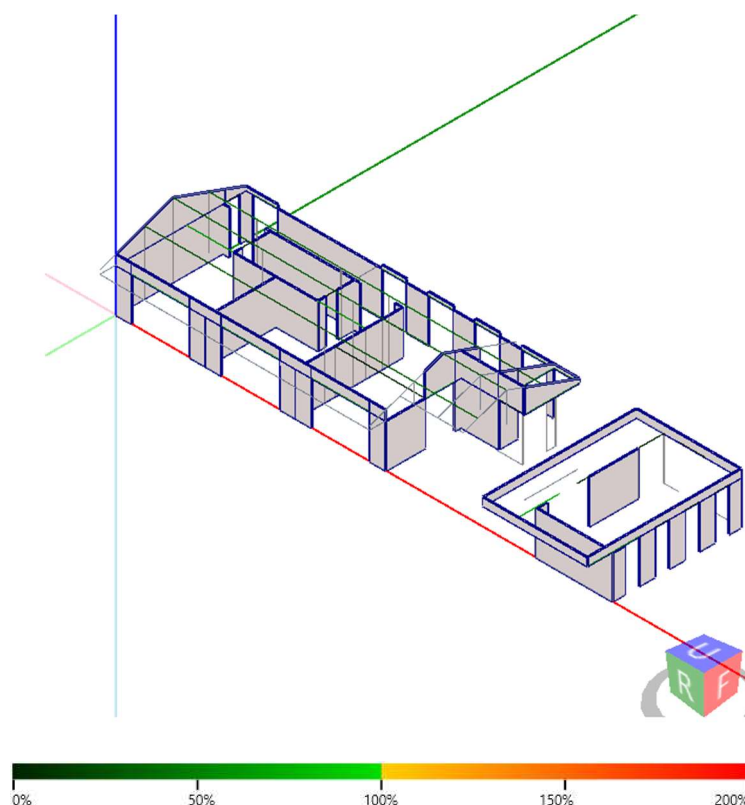


The checks are summarized below. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

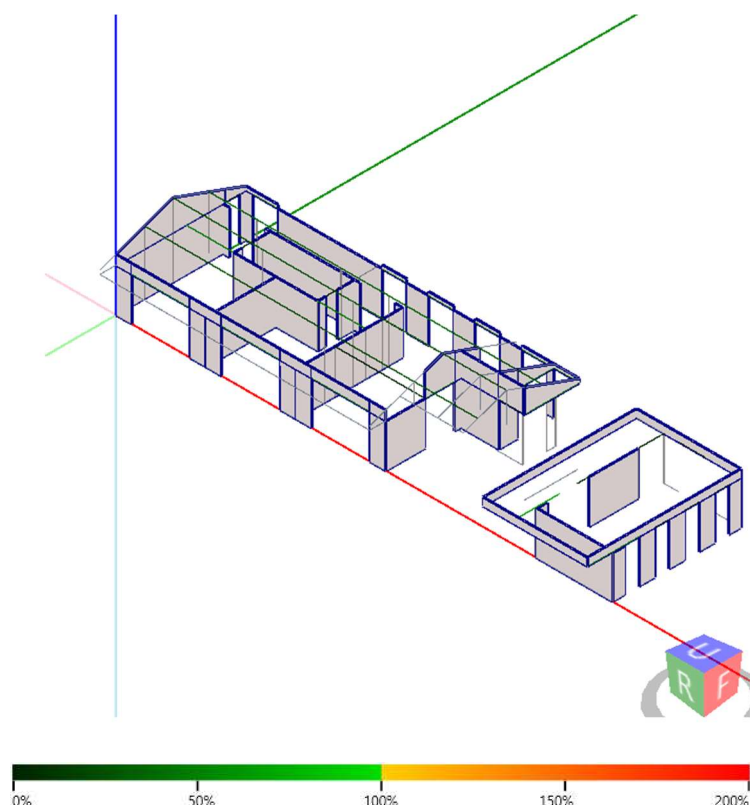
Beam name	Section	$V_{2,max}$ [kN]	A_{eff} [mm ²]	$k_{cr,fi}$	$k_{mod,fi}$	$\gamma_{M,fi}$	k_{fi}	$f_{v,d,fi}$ [MPa]	$\tau_{2,d,fi}$ [MPa]	Check
Beam 1	200x440 GL24h	28.16	59328	0.83	1	1	1.15	4.03	0.86	21%
Beam 2	200x440 GL24h	28.83	59328	0.83	1	1	1.15	4.03	0.88	22%
Beam 3	200x440 GL24h	23.30	59328	0.83	1	1	1.15	4.03	0.71	18%
Beam 4	200x200 GL24h	6.52	24768	0.83	1	1	1.15	4.03	0.48	12%
Beam 5	200x200 GL24h	7.22	24768	0.83	1	1	1.15	4.03	0.53	13%
Beam 6	200x200 GL24h	5.97	24768	0.83	1	1	1.15	4.03	0.44	11%
Beam 7	200x200 GL24h	5.86	24768	0.83	1	1	1.15	4.03	0.43	11%
Beam 8	200x200 GL24h	7.95	24768	0.83	1	1	1.15	4.03	0.58	14%
Beam 10	200x240 GL24h	11.97	30528	0.83	1	1	1.15	4.03	0.71	18%
Beam 16	200x200 GL24h	7.45	24768	0.83	1	1	1.15	4.03	0.54	14%
Beam 17	260x280 GL28h	28.93	51408	0.83	1	1	1.15	4.03	1.02	25%
Beam 19	260x280 GL28h	28.17	51408	0.83	1	1	1.15	4.03	0.99	25%
Beam 20	200x400 GL28h	16.85	53568	0.83	1	1	1.15	4.03	0.57	14%
Beam 21	200x400 GL28h	10.86	53568	0.83	1	1	1.15	4.03	0.37	9%
Beam 22	200x400 GL28h	8.28	53568	0.83	1	1	1.15	4.03	0.28	7%
Beam 25	200x400 GL28h	20.62	53568	0.83	1	1	1.15	4.03	0.70	17%
Beam 26	200x240 GL24h	3.71	30528	0.83	1	1	1.15	4.03	0.22	5%
Beam 27	200x240 GL24h	5.90	30528	0.83	1	1	1.15	4.03	0.35	9%
Beam 28	200x200 GL24h	3.11	24768	0.83	1	1	1.15	4.03	0.23	6%
Beam 30	200x240 GL24h	5.70	30528	0.83	1	1	1.15	4.03	0.34	8%
Beam 32	200x520 GL24h	47.73	48042	0.83	1	1	1.15	4.03	1.80	45%
Beam 35	200x520 GL24h	23.30	48042	0.83	1	1	1.15	4.03	0.88	22%

Verification percentage of beams in fire situation

Bending verifications



Shear verifications



Timber columns checks in fire situation

Stability of columns

The stability of columns subjected to compression is verified in accordance with § 6.3.2 of EN 1995-1-1 using the effective cross sections of the elements according to 4.2.2 of EN 1995-1-2.

The relative slenderness ratios should be taken as:

$$\lambda_{rel,y,fi} = \frac{\lambda_{y,fi}}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,20}}}$$

and

$$\lambda_{rel,z,fi} = \frac{\lambda_{z,fi}}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,20}}}$$

where:

$\lambda_{y,fi}, \lambda_{rel,y,fi}$ are slenderness ratios corresponding to bending about the y-axis (deflection in the z-direction) in the fire situation;

$\lambda_{z,fi}, \lambda_{rel,z,fi}$ are slenderness ratios corresponding to bending about the z-axis (deflection in the y-direction) in the fire situation.

Where both $\lambda_{rel,z} \leq 0,3$ and $\lambda_{rel,y} \leq 0,3$, the stresses should satisfy the expressions (6.19) and (6.20) in 6.2.4 of EN 1995-1-1.

In all other cases the stresses, which will be increased due to deflection, should satisfy the following expressions:

$$\frac{\sigma_{c,0,d,fi}}{k_{c,y,fi} \cdot f_{c,0,d,fi}} + \frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} + k_m \cdot \frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}} \leq 1$$

$$\frac{\sigma_{c,0,d,fi}}{k_{c,z,fi} \cdot f_{c,0,d,fi}} + k_m \cdot \frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} + \frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}} \leq 1$$

where the symbols are defined as follows:

$$k_{c,y,fi} = \frac{1}{k_{y,fi} + \sqrt{k_{y,fi}^2 - \lambda_{rel,y,fi}^2}}$$

$$k_{c,z,fi} = \frac{1}{k_{z,fi} + \sqrt{k_{z,fi}^2 - \lambda_{rel,z,fi}^2}}$$

$$k_{y,fi} = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel,y,fi} - 0,3) + \lambda_{rel,y,fi}^2)$$

$$k_{z,fi} = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel,z,fi} - 0,3) + \lambda_{rel,z,fi}^2)$$

where:

β_c is a factor for members within the straightness limits defined in Section 10 of EN 1995-1-1 and assumes the following values

$$\beta_c = \begin{cases} 0,2 & \text{for solid timber} \\ 0,1 & \text{for glued laminated timber and LVL} \end{cases}$$

The values of the actions in the tables below are related, for each pillar, to the fire situation.

- N: Axial force in the fire situation
- V₂: Shear force along the local axis 2 in the fire situation
- V₃: Shear force along the local axis 3 in the fire situation
- M₂₋₂: Bending moment about local axis 2 in the fire situation
- M₃₋₃: Bending moment about local axis 3 in the fire situation

Column name	Comb.	N [kN]	V2 [kN]	V3 [kN]	M2-2 [kNm]	M3-3 [kNm]
Column 2	Quasi permanent	20.42	0.00	0.00	0.00	0.00
Column 3	Quasi permanent	20.45	0.00	0.00	0.00	0.00
Column 6	Quasi permanent	25.72	0.00	0.00	0.00	0.00
Column 7	Quasi permanent	18.37	0.00	0.00	0.00	0.00
Column 8	Quasi permanent	27.10	0.00	0.00	0.00	0.00
Column 13	Quasi permanent	15.54	0.00	0.00	0.00	0.00

Column 14	Quasi permanent	15.54	0.00	0.00	0.00	0.00
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The following table summarizes the stability checks for the columns.

Sect.: Column cross section

h: Column height

A_{eff} : Effective cross sectional area of the column

$J_{y,fi}$: Area moment of inertia with respect to the y axis of the effective cross section

$J_{z,fi}$: Area moment of inertia with respect to the z axis of the effective cross section

$k_{mod,fi}$: Modification factor for fire

$\gamma_{M,fi}$: Partial factor for timber in fire

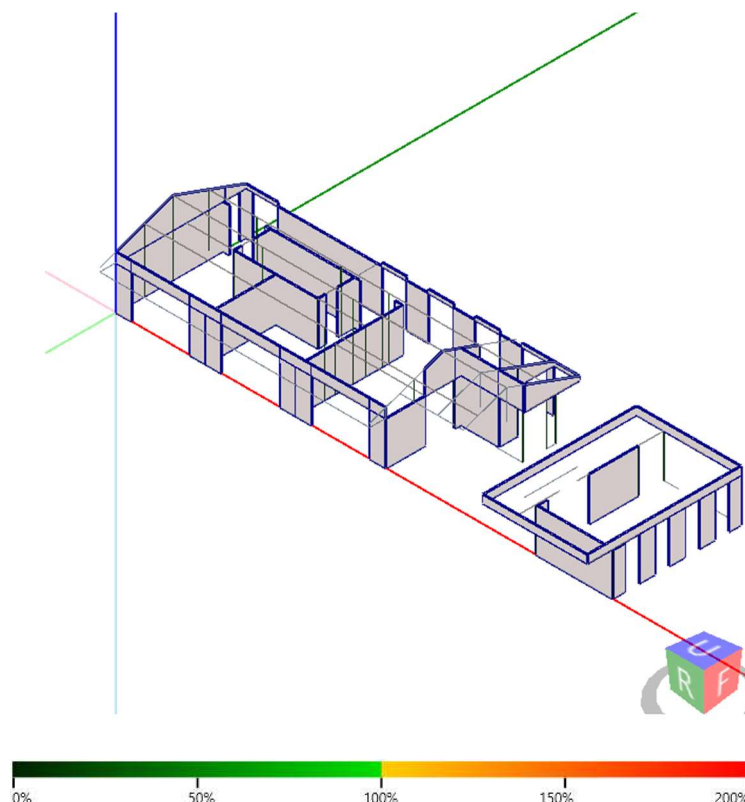
$f_{c,0,k,fi}$: Design compressive strength along the grain in fire

$\sigma_{c,0,d,fi}$: Design compressive stress along the grain for the fire situation

Column name	Sect.	h [m]	A_{eff} [mm ²]	$J_{y,fi}$ [mm ⁴]	$J_{z,fi}$ [mm ⁴]	$k_{c,y,fi}$	$k_{c,z,fi}$	$k_{mod,fi}$	$\gamma_{M,fi}$	k_{fi}	$f_{c,0,d,fi}$	$\sigma_{c,0,d,fi}$ [MPa]	Check
Column 2	200x200 GL24h	2.5	20736	3.58E7	3.58E7	0.85	0.85	1	1	1.15	27.60	0.98	4%
Column 3	200x200 GL24h	2.5	20736	3.58E7	3.58E7	0.85	0.85	1	1	1.15	27.60	0.99	4%
Column 6	200x200 GL24h	1.413	20736	3.58E7	3.58E7	0.97	0.97	1	1	1.15	27.60	1.24	5%
Column 7	200x200 GL24h	1.413	20736	3.58E7	3.58E7	0.97	0.97	1	1	1.15	27.60	0.89	3%
Column 8	200x200 GL24h	1.413	20736	3.58E7	3.58E7	0.97	0.97	1	1	1.15	27.60	1.31	5%
Column 13	200x200 GL24h	1.644	20736	3.58E7	3.58E7	0.96	0.96	1	1	1.15	27.60	0.75	3%
Column 14	200x200 GL24h	1.644	20736	3.58E7	3.58E7	0.96	0.96	1	1	1.15	27.60	0.75	3%

Verification percentage of columns in fire situation

Buckling verifications



Framed walls checks in fire situation

Stability of the studs

The stability of the studs subjected to compression is verified in accordance with § 6.3.2 of EN 1995-1-1 using the effective cross sections of the elements according to 4.2.2 of EN 1995-1-2. Specifically, the checked elements are the internal and the external studs which are the most loaded. Depending on whether the sheathing sheets are considered capable of preventing the lateral displacements of the studs in the wall plane or not, the buckling checks are performed only in the direction orthogonal to the plane of the wall or in both directions.

The relative slenderness ratio should be taken as:

$$\lambda_{rel,fi} = \frac{\lambda_{fi}}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,20}}}$$

where:

λ_{fi} , $\lambda_{rel,fi}$ are slenderness ratios corresponding to bending direction in the fire situation.

Where both $\lambda_{rel,z} \leq 0,3$ and $\lambda_{rel,y} \leq 0,3$, the stresses should satisfy the expressions (6.19) and (6.20) in 6.2.4 of EN 1995-1-1.

In all other cases the stresses, which will be increased due to deflection, should satisfy the following expression:

$$\frac{\sigma_{c,0,d,fi}}{k_{c,fi} \cdot f_{c,0,d,fi}} + \frac{\sigma_{m,d,fi}}{f_{m,d,fi}} \leq 1$$

where $k_{c,fi}$ is

$$k_{c,fi} = \frac{1}{k_{fi} + \sqrt{k_{fi}^2 - \lambda_{rel,fi}^2}}$$

$$k_{fi} = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel,fi} - 0,3) + \lambda_{rel,fi}^2)$$

The values of the actions in the tables below are related, for each pillar, to the most severe combination of load for the Ultimate Limit State of instability.

Wall name	Length [m]	Comb.	N [kN]	M2-2 [kNm]
Wall 1	1.30	Quasi permanent	47.87	0.00
Wall 2	2.60	Quasi permanent	95.78	0.00
Wall 3	2.60	Quasi permanent	87.32	0.00
Wall 4	1.30	Quasi permanent	39.95	0.00
Wall 5	3.26	Quasi permanent	21.98	0.00
Wall 6	0.55	Quasi permanent	23.94	0.00
Wall 7	1.79	Quasi permanent	31.06	0.00
Wall 8	1.79	Quasi permanent	28.35	0.00
Wall 9	1.79	Quasi permanent	28.65	0.00
Wall 10	8.61	Quasi permanent	85.14	0.00
Wall 11	0.63	Quasi permanent	12.01	0.00
Wall 12	0.53	Quasi permanent	4.53	0.00
Wall 13	9.01	Quasi permanent	91.36	0.00
Wall 14	0.55	Quasi permanent	4.51	0.00
Wall 15	3.16	Quasi permanent	41.60	0.00
Wall 16	1.54	Quasi permanent	24.06	0.00
Wall 17	0.63	Quasi permanent	4.84	0.00
Wall 18	7.86	Quasi permanent	57.66	0.00
Wall 19	0.62	Quasi permanent	5.84	0.00
Wall 20	8.89	Quasi permanent	37.49	0.00
Wall 21	5.82	Quasi permanent	24.64	0.00
Wall 22	1.43	Quasi permanent	6.86	0.00
Wall 23	1.43	Quasi permanent	6.86	0.00
Wall 24	0.64	Quasi permanent	3.41	0.00
Wall 25	0.64	Quasi permanent	3.41	0.00
Wall 26	7.00	Quasi permanent	112.71	0.00
Wall 27	1.49	Quasi permanent	9.66	0.00
Wall 29	2.73	Quasi permanent	3.91	0.00
Wall 31	2.10	Quasi permanent	3.08	0.00
Wall 33	2.10	Quasi permanent	3.08	0.00
Wall 35	2.10	Quasi permanent	3.08	0.00
Wall 37	4.80	Quasi permanent	8.34	0.00
Wall 40	1.30	Quasi permanent	7.36	0.00
Wall 41	1.30	Quasi permanent	8.39	0.00
Wall 42	4.90	Quasi permanent	25.09	0.00
Wall 43	2.60	Quasi permanent	14.63	0.00
Wall 44	4.90	Quasi permanent	25.09	0.00
Wall 45	2.60	Quasi permanent	14.67	0.00
Wall 46	4.90	Quasi permanent	25.09	0.00
Wall 47	9.01	Quasi permanent	20.46	0.00
Wall 48	1.82	Quasi permanent	1.74	0.00
Wall 50	2.45	Quasi permanent	7.47	0.00
Wall 51	0.55	Quasi permanent	1.65	0.00
Wall 52	3.16	Quasi permanent	10.83	0.00
Wall 53	1.54	Quasi permanent	3.64	0.00
Wall 54	3.03	Quasi permanent	3.74	0.00
Wall 55	4.72	Quasi permanent	9.54	0.00
Wall 56	1.49	Quasi permanent	3.13	0.00
Wall 57	2.15	Quasi permanent	7.89	0.00
Wall 58	0.98	Quasi permanent	4.81	0.00
Wall 60	4.08	Quasi permanent	100.72	0.00
Wall 64	0.99	Quasi permanent	18.55	0.00

Wall 65	1.30	Quasi permanent	28.70	0.00
Wall 66	1.30	Quasi permanent	28.55	0.00
Wall 67	1.31	Quasi permanent	20.56	0.00
Wall 68	1.00	Quasi permanent	4.89	0.00
Wall 69	6.39	Quasi permanent	30.22	0.00

The following table summarizes the stability checks for the studs of the framed walls.

Section: type of cross-section of the stud

h_{stud} : Stud height

$A_{stud,eff}$: Effective cross sectional area of the stud

J_{fi} : Cross sectional moment of inertia of the stud

$k_{mod,fi}$: Modification factor taking into account the effect of the duration of load and moisture content

$\gamma_{M,fi}$: Partial factor for a material property in fire

$f_{c,0,d,fi}$: Compressive strength along the grain in fire

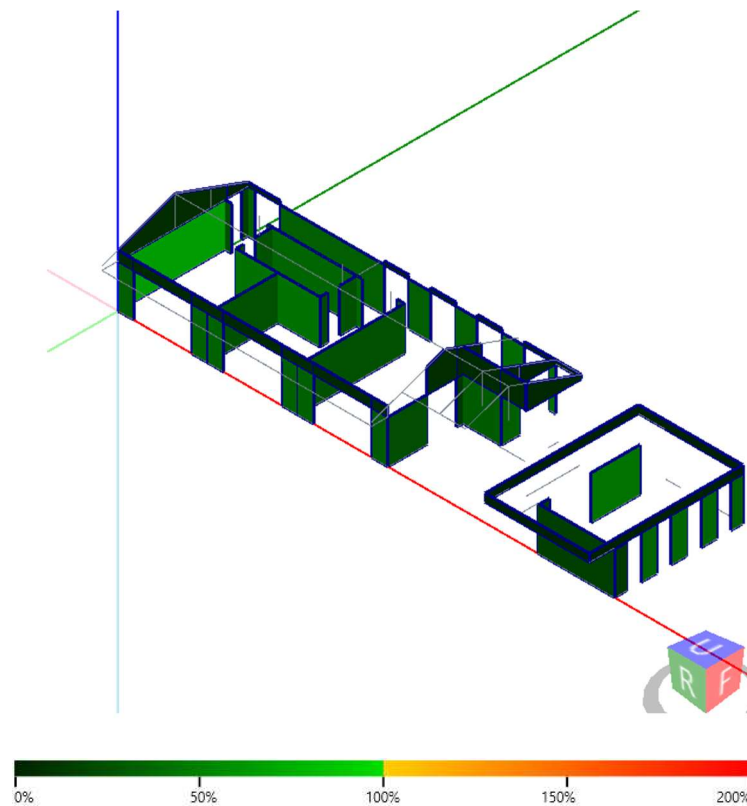
$\sigma_{c,0,d,fi}$: Design compressive stress along the grain in the fire situation

Wall name	Section	Stud	h_{mont} [m]	$A_{mont,eff}$ [mm ²]	Studs braced against buckling in the wall plane	J_{fi} [mm ⁴]	$k_{c,fi}$	$k_{mod,fi}$	$\gamma_{M,fi}$	k_{fi}	$f_{c,0,k}$	$f_{m,k}$	N_{fi} [kN]	$\sigma_{c,0,d}$ [MPa]	$\sigma_{m,d}$ [MPa]	Check
Wall 1	Okvir 200 ext=160	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.93	0.66	0.00	15%
Wall 1	Okvir 200 ext=160	External	3.6	27040	No	5.77E7	0.56	1	1	1.25	21.00	24.00	33.51	1.24	0.00	8%
Wall 2	Okvir 200 ext=160	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	9.57	0.71	0.00	16%
Wall 2	Okvir 200 ext=160	External	3.6	27040	No	5.77E7	0.56	1	1	1.25	21.00	24.00	34.32	1.27	0.00	9%
Wall 3	Okvir 200 ext=160	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	9.34	0.69	0.00	16%
Wall 3	Okvir 200 ext=160	External	3.6	27040	No	5.77E7	0.56	1	1	1.25	21.00	24.00	32.80	1.21	0.00	8%
Wall 4	Okvir 200 ext=160	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.06	0.60	0.00	14%
Wall 4	Okvir 200 ext=160	External	3.6	27040	No	5.77E7	0.56	1	1	1.25	21.00	24.00	26.94	1.00	0.00	7%
Wall 5	Okvir 200	Internal	3.6	11600	No	6.19E6	0.16	1	1	1.25	21.00	24.00	4.49	0.39	0.00	9%
Wall 5	Okvir 200	External	3.6	11600	No	6.19E6	0.16	1	1	1.25	21.00	24.00	1.99	0.17	0.00	4%
Wall 6	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	12.71	0.94	0.00	22%
Wall 7	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	5.82	0.43	0.00	10%
Wall 7	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	11.52	0.85	0.00	20%
Wall 8	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	5.74	0.42	0.00	10%
Wall 8	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.91	0.66	0.00	15%
Wall 9	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	5.91	0.44	0.00	10%
Wall 9	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.97	0.66	0.00	15%
Wall 10	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	6.53	0.48	0.00	11%
Wall 10	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.86	0.66	0.00	15%
Wall 11	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	9.26	0.69	0.00	16%
Wall 12	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	2.79	0.21	0.00	5%
Wall 13	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	28.85	2.13	0.00	49%
Wall 13	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	3.61	0.27	0.00	6%
Wall 14	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	2.28	0.17	0.00	4%
Wall 15	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.02	0.59	0.00	14%
Wall 15	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	6.45	0.48	0.00	11%
Wall 16	Okvir 200	Internal	3.6	11600	No	6.19E6	0.16	1	1	1.25	21.00	24.00	7.59	0.65	0.00	15%
Wall 16	Okvir 200	External	3.6	11600	No	6.19E6	0.16	1	1	1.25	21.00	24.00	5.38	0.46	0.00	11%
Wall 17	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.44	0.39	0.00	9%
Wall 18	Okvir 140	Internal	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	4.55	0.73	0.00	18%
Wall 18	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.18	0.35	0.00	8%
Wall 19	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	3.06	0.49	0.00	12%
Wall 20	Okvir 140	Internal	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.64	0.42	0.00	10%
Wall 20	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	1.40	0.22	0.00	5%
Wall 21	Okvir 140	Internal	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.65	0.42	0.00	10%

Wall 21	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	1.41	0.23	0.00	5%
Wall 22	Okvir 140	Internal	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.67	0.43	0.00	10%
Wall 22	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.02	0.32	0.00	8%
Wall 23	Okvir 140	Internal	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.67	0.43	0.00	10%
Wall 23	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.02	0.32	0.00	8%
Wall 24	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	1.98	0.32	0.00	8%
Wall 25	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	1.98	0.32	0.00	8%
Wall 26	Okvir 100 e=417mm	Internal	3.6	6693	No	3.57E6	0.16	1	1	1.25	21.00	24.00	6.71	1.00	0.00	23%
Wall 26	Okvir 100 e=417mm	External	3.6	6693	No	3.57E6	0.16	1	1	1.25	21.00	24.00	3.99	0.60	0.00	14%
Wall 27	Okvir 140	Internal	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	3.89	0.62	0.00	15%
Wall 27	Okvir 140	External	3.6	6240	No	3.16E6	0.16	1	1	1.25	21.00	24.00	2.07	0.33	0.00	8%
Wall 29	Okvir 200	Internal	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.99	0.07	0.00	0%
Wall 29	Okvir 200	External	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.53	0.04	0.00	0%
Wall 31	Okvir 200	Internal	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.99	0.07	0.00	0%
Wall 31	Okvir 200	External	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.27	0.02	0.00	0%
Wall 33	Okvir 200	Internal	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.99	0.07	0.00	0%
Wall 33	Okvir 200	External	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.46	0.03	0.00	0%
Wall 35	Okvir 200	Internal	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	1.03	0.08	0.00	0%
Wall 35	Okvir 200	External	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.58	0.04	0.00	0%
Wall 37	Okvir 200	Internal	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	1.17	0.09	0.00	0%
Wall 37	Okvir 200	External	0.3	13520	No	7.21E6	1.00	1	1	1.25	21.00	24.00	0.73	0.05	0.00	0%
Wall 40	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.49	0.26	0.00	1%
Wall 40	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	2.08	0.15	0.00	1%
Wall 41	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.89	0.29	0.00	1%
Wall 41	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	2.59	0.19	0.00	1%
Wall 42	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.34	0.25	0.00	1%
Wall 42	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	1.39	0.10	0.00	0%
Wall 43	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.64	0.27	0.00	1%
Wall 43	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	2.10	0.16	0.00	1%
Wall 44	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.34	0.25	0.00	1%
Wall 44	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	1.74	0.13	0.00	1%
Wall 45	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.78	0.28	0.00	1%
Wall 45	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	2.31	0.17	0.00	1%
Wall 46	Okvir 200	Internal	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	3.43	0.25	0.00	1%
Wall 46	Okvir 200	External	0.8	13520	No	7.21E6	0.94	1	1	1.25	21.00	24.00	1.90	0.14	0.00	1%
Wall 47	Okvir 200	Internal	2.5	13520	No	7.21E6	0.33	1	1	1.25	21.00	24.00	2.05	0.15	0.00	2%
Wall 47	Okvir 200	External	1.0	13520	No	7.21E6	0.91	1	1	1.25	21.00	24.00	0.46	0.03	0.00	0%
Wall 48	Okvir 200	Internal	0.8	13520	No	7.21E6	0.95	1	1	1.25	21.00	24.00	0.65	0.05	0.00	0%
Wall 48	Okvir 200	External	1.0	13520	No	7.21E6	0.91	1	1	1.25	21.00	24.00	0.42	0.03	0.00	0%
Wall 50	Okvir 200	Internal	2.4	13520	No	7.21E6	0.34	1	1	1.25	21.00	24.00	2.02	0.15	0.00	2%
Wall 50	Okvir 200	External	2.1	13520	No	7.21E6	0.42	1	1	1.25	21.00	24.00	0.91	0.07	0.00	1%
Wall 51	Okvir 200	External	2.1	13520	No	7.21E6	0.42	1	1	1.25	21.00	24.00	0.85	0.06	0.00	1%
Wall 52	Okvir 200	Internal	2.0	13520	No	7.21E6	0.49	1	1	1.25	21.00	24.00	2.21	0.16	0.00	1%
Wall 52	Okvir 200	External	2.0	13520	No	7.21E6	0.49	1	1	1.25	21.00	24.00	1.09	0.08	0.00	1%
Wall 53	Okvir 200	Internal	1.7	13520	No	7.21E6	0.59	1	1	1.25	21.00	24.00	1.47	0.11	0.00	1%
Wall 53	Okvir 200	External	2.0	13520	No	7.21E6	0.49	1	1	1.25	21.00	24.00	0.89	0.07	0.00	1%
Wall 54	Okvir 200	Internal	1.2	13520	No	7.21E6	0.84	1	1	1.25	21.00	24.00	1.03	0.08	0.00	0%
Wall 54	Okvir 200	External	1.4	13520	No	7.21E6	0.75	1	1	1.25	21.00	24.00	0.65	0.05	0.00	0%
Wall 55	Okvir 200	Internal	1.6	13520	No	7.21E6	0.67	1	1	1.25	21.00	24.00	2.08	0.15	0.00	1%
Wall 55	Okvir 200	External	2.0	13520	No	7.21E6	0.47	1	1	1.25	21.00	24.00	0.91	0.07	0.00	1%
Wall 56	Okvir 140	Internal	1.7	6240	No	3.16E6	0.57	1	1	1.25	21.00	24.00	1.30	0.21	0.00	1%
Wall 56	Okvir 140	External	2.0	6240	No	3.16E6	0.47	1	1	1.25	21.00	24.00	0.78	0.13	0.00	1%
Wall 57	Okvir 200	Internal	2.0	13520	No	7.21E6	0.49	1	1	1.25	21.00	24.00	2.26	0.17	0.00	1%
Wall 57	Okvir 200	External	2.0	13520	No	7.21E6	0.49	1	1	1.25	21.00	24.00	1.26	0.09	0.00	1%
Wall 58	Okvir 200	Internal	3.5	13520	No	7.21E6	0.17	1	1	1.25	21.00	24.00	2.21	0.16	0.00	4%
Wall 58	Okvir 200	External	3.5	13520	No	7.21E6	0.17	1	1	1.25	21.00	24.00	1.63	0.12	0.00	3%
Wall 60	Okvir 200 ext=160	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	14.05	1.04	0.00	24%
Wall 60	Okvir 200 ext=160	External	3.6	27040	No	5.77E7	0.56	1	1	1.25	21.00	24.00	18.84	0.70	0.00	5%
Wall 64	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.61	0.64	0.00	15%
Wall 64	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	5.91	0.44	0.00	10%
Wall 65	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.01	0.59	0.00	14%
Wall 65	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	10.45	0.77	0.00	18%
Wall 66	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	8.01	0.59	0.00	14%
Wall 66	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	10.30	0.76	0.00	18%
Wall 67	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	7.87	0.58	0.00	13%
Wall 67	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	9.94	0.74	0.00	17%
Wall 68	Okvir 200	Internal	3.5	13520	No	7.21E6	0.17	1	1	1.25	21.00	24.00	2.25	0.17	0.00	4%
Wall 68	Okvir 200	External	3.5	13520	No	7.21E6	0.17	1	1	1.25	21.00	24.00	1.63	0.12	0.00	3%
Wall 69	Okvir 200	Internal	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	2.96	0.22	0.00	5%
Wall 69	Okvir 200	External	3.6	13520	No	7.21E6	0.16	1	1	1.25	21.00	24.00	1.57	0.12	0.00	3%

Verification percentage of walls in fire situation

Buckling verifications



Connections

Hold down – Connection at the base of the structure

The design resistance R_d of the hold-downs is determined as the minimum value among the resistances relating to the following failure modes:

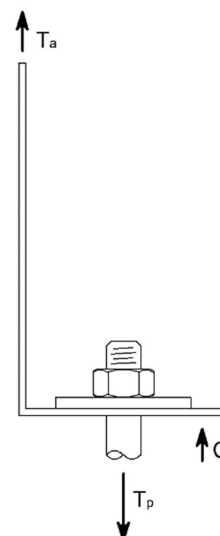
- nailing failure;
- hold-downs steel failure;
- failure of the concrete anchors.

Tensile force

The tensile force acting on the hold down (T_a) is evaluated as described in paragraph “Model Description”.

The tensile force acting on the concrete anchors is calculated taking into account the additional moment due to the non-alignment between the external force acting on the vertical flange of the hold down and the anchors themselves using a coefficient, indicated as k_t .

$$T_p = T_a \cdot k_t$$



Wall name	Length [m]	Connection name	N° of anchors at each wall end	Comb.	Dur.	N [kN]	M _{s-3} [kNm]	T _a [kN]	k _t	T _p [kN]
Wall 1	1.30	WKR 530	1	Horizontal ULS 1	Instantaneous	45.27	1.75	0.00	1.45	0.00
Wall 2	2.60	WKR 530	1	Horizontal ULS 1	Instantaneous	90.45	4.03	0.00	1.45	0.00
Wall 3	2.60	WKR 530	1	Horizontal ULS 1	Instantaneous	83.26	4.03	0.00	1.45	0.00
Wall 4	1.30	WKR 530	1	Horizontal ULS 1	Instantaneous	38.55	1.75	0.00	1.45	0.00
Wall 5	3.26	WKR 530	1	Dynamic SLV 8 ex+ ey+	Instantaneous	21.98	81.73	16.87	1.45	24.46
Wall 7	1.79	WKR 530	1	Horizontal ULS 1	Instantaneous	29.03	1.73	0.00	1.45	0.00
Wall 8	1.79	WKR 530	1	Horizontal ULS 1	Instantaneous	26.71	1.73	0.00	1.45	0.00
Wall 9	1.79	WKR 530	1	Horizontal ULS 1	Instantaneous	26.96	1.73	0.00	1.45	0.00
Wall 10	8.61	WKR 530	1	Horizontal ULS 1	Instantaneous	81.38	10.43	0.00	1.45	0.00
Wall 13	9.01	WKR 530	1	Horizontal ULS 1	Instantaneous	87.43	4.22	0.00	1.45	0.00
Wall 15	3.16	WKR 530	1	Horizontal ULS 1	Instantaneous	39.54	4.21	0.00	1.45	0.00
Wall 16	1.54	WKR 530	1	Dynamic SLV 8 ex+ ey+	Instantaneous	24.06	40.80	17.41	1.45	25.24
Wall 18	7.86	WKR 530	1	Horizontal ULS 1	Instantaneous	53.98	15.45	0.00	1.45	0.00
Wall 20	8.89	WKR 530	1	Dynamic SLV 8 ex- ey-	Instantaneous	37.49	200.09	6.26	1.45	9.08
Wall 21	5.82	WKR 530	1	Dynamic SLV 8 ex- ey-	Instantaneous	24.64	120.48	10.68	1.45	15.49
Wall 22	1.43	WKR 530	1	Dynamic SLV 8 ex- ey-	Instantaneous	6.86	24.11	15.30	1.45	22.19
Wall 23	1.43	WKR 530	1	Dynamic SLV 8 ex- ey-	Instantaneous	6.86	20.98	12.87	1.45	18.67
Wall 26	7.00	WKR 530	1	Horizontal ULS 1	Instantaneous	100.21	12.25	0.00	1.45	0.00
Wall 27	1.49	2x WKR 530	2	Dynamic SLV 8 ex+ ey+	Instantaneous	9.66	53.22	17.43	1.45	25.27
Wall 60	4.08	WKR 530	1	Horizontal ULS 1	Instantaneous	100.72	0.79	0.00	1.45	0.00
Wall 64	0.99	WKR 530	1	Dynamic SLV 8 ex+ ey+	Instantaneous	18.55	15.93	8.60	1.45	12.47
Wall 65	1.30	WKR 530	1	Dynamic SLV 8 ex+ ey+	Instantaneous	28.70	21.84	4.32	1.45	6.26
Wall 66	1.30	WKR 530	1	Dynamic SLV 8 ex+ ey+	Instantaneous	28.55	21.84	4.39	1.45	6.37
Wall 67	1.31	WKR 530	1	Dynamic SLV 8 ex+ ey+	Instantaneous	20.56	22.02	8.40	1.45	12.18
Wall 69	6.39	WKR 530	1	Dynamic SLV 4 ex- ey-	Instantaneous	30.22	135.12	8.39	1.45	12.16

Nailing resistance

The design value of the load-bearing capacity of the nailing is given by the following expression

$$R_{c,d} = \frac{k_{mod} \cdot R_{c,k,dens}}{\gamma_M}$$

where:

- $R_{c,k,dens}$ is the characteristic value of the nailing resistance. This value is reduced by the k_{dens} factor when the density of the material used is less than 350 kg/m^3 . k_{dens} can be evaluated using the formula $R_{c,k,dens} = R_{c,k} \cdot \left(\frac{\rho_k}{350}\right)^2$;
- k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content;
- γ_M is the is the partial factor for the connections.

Steel resistance

The tensile design strength of the hold-down can be evaluated according to the formula

$$R_{s,d} = \frac{R_{s,k}}{\gamma_{M2}}$$

where:

- $R_{s,k}$ is the characteristic value of the resistance of the hold-down;
- γ_{M2} is the partial factor for resistance of cross-sections in tension to fracture.

Concrete anchors resistance

The tension resistance of the concrete anchors can be evaluated according to the formula

$$R_{p,d} = \frac{R_{p,k}}{\gamma}$$

where:

- $R_{p,k}$ is the characteristic value of the resistance of the concrete anchors;
- γ is the safety factor.

The checks are summarized in the following table which shows the characteristic values of the resistances associated with collapse of the various components.

Name:	Name of the connection in which the hold-down is used
Comb.:	The most severe combination of load
$T_{a,d}$:	Design value of the tensile force acting on the hold down
$T_{p,d}$:	Design value of the tensile force acting on the concrete anchors

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

$k_{R,deg}$: Resistance degradation coefficient due to cyclic actions

γ_M : Partial safety factor

$R_{a,d}$: Design value of the hold down resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms associated with it

$R_{p,d}$: Design value of the concrete anchors resistance

$$T_{a,d} \leq R_{a,d} = \min(R_{c,d}; R_{s,d})$$

$$T_{p,d} \leq R_{p,d}$$

Wall name	Connection name	Comb.	Service class	$T_{a,d}$ [kN]	$R_{c,k,dens}$ [kN]	$R_{s,k}$ [kN]	k_{mod}	$k_{R,deg}$	γ_M	γ_{M2}	$R_{a,d}$ [kN]	$T_{p,d}$ [kN]	$R_{p,k}$ [kN]	γ	$R_{p,d}$ [kN]	Failure mode	Check
Wall 1	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 2	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 3	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 4	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 5	WKR 530	Dynamic SLV 8 ex+ ey+	1	16.87	42.60	26	1.1	1	1.3	1.25	20.80	24.46	42	1.5	28.00	Tension: anchors	87%
Wall 7	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 8	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 9	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 10	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 13	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 15	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 16	WKR 530	Dynamic SLV 8 ex+ ey+	1	17.41	42.60	26	1.1	1	1.3	1.25	20.80	25.24	42	1.5	28.00	Tension: anchors	90%
Wall 18	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 20	WKR 530	Dynamic SLV 8 ex- ey-	1	6.26	42.60	26	1.1	1	1.3	1.25	20.80	9.08	42	1.5	28.00	Tension: anchors	32%
Wall 21	WKR 530	Dynamic SLV 8 ex- ey-	1	10.68	42.60	26	1.1	1	1.3	1.25	20.80	15.49	42	1.5	28.00	Tension: anchors	55%
Wall 22	WKR 530	Dynamic SLV 8 ex- ey-	1	15.30	42.60	26	1.1	1	1.3	1.25	20.80	22.19	42	1.5	28.00	Tension: anchors	79%
Wall 23	WKR 530	Dynamic SLV 8 ex- ey-	1	12.87	42.60	26	1.1	1	1.3	1.25	20.80	18.67	42	1.5	28.00	Tension: anchors	67%
Wall 26	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 27	2x WKR 530	Dynamic SLV 8 ex+ ey+	1	17.43	42.60	26	1.1	1	1.3	1.25	20.80	25.27	42	1.5	28.00	Tension: anchors	90%
Wall 60	WKR 530	Horizontal ULS 1	1	0.00	42.60	26	1.1	1	1.3	1.25	-	0.00	42	1.5	-	-	0%
Wall 64	WKR 530	Dynamic SLV 8 ex+ ey+	1	8.60	42.60	26	1.1	1	1.3	1.25	20.80	12.47	42	1.5	28.00	Tension: anchors	45%
Wall 65	WKR 530	Dynamic SLV 8 ex+ ey+	1	4.32	42.60	26	1.1	1	1.3	1.25	20.80	6.26	42	1.5	28.00	Tension: anchors	22%
Wall 66	WKR 530	Dynamic SLV 8 ex+ ey+	1	4.39	42.60	26	1.1	1	1.3	1.25	20.80	6.37	42	1.5	28.00	Tension: anchors	23%
Wall 67	WKR 530	Dynamic SLV 8 ex+ ey+	1	8.40	42.60	26	1.1	1	1.3	1.25	20.80	12.18	42	1.5	28.00	Tension: anchors	44%
Wall 69	WKR 530	Dynamic SLV 4 ex- ey-	1	8.39	42.60	26	1.1	1	1.3	1.25	20.80	12.16	42	1.5	28.00	Tension: anchors	43%

Hold down – Upper levels connection

The design resistance R_d of the tie downs is determined as the minimum value among the resistances relating to the following failure modes:

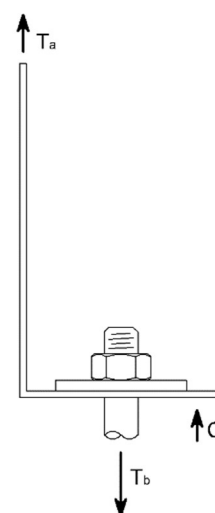
- nailing failure;
- hold-downs steel failure;
- failure of the bolt.

Tensile force

The tensile force acting on the hold down (T_a) is evaluated as described in paragraph “Model Description”.

The tensile force acting on the single bolt is calculated taking into account the numbers of bolts and additional moment due to the non-alignment between the external force acting on the vertical flange of the hold down and the bolt itself using a coefficient, indicated as k_t .

$$T_b = \frac{T_a \cdot k_t}{n_b}$$



Wall name	Length [m]	Connection name	N° of anchors at each wall end	Comb.	Dur.	N [kN]	M ₃₋₃ [kNm]	T _a [kN]	k _t	n _b	T _b [kN]
Wall 52	3.16	WKR 285 - WKR 285	1	Horizontal ULS 1	Instantaneous	10.83	0.39	0.00	1.45	1	0.00
Wall 56	1.49	WKR 285 - WKR 285	1	Dynamic SLV 8 ex- ey-	Instantaneous	3.13	15.35	9.88	1.45	1	14.3 3

Nailing resistance

The design value of the load-bearing capacity of the nailing is given by the following expression

$$R_{c,d} = \frac{k_{mod} \cdot R_{c,k,dens}}{\gamma_M}$$

where:

$R_{c,k,dens}$ is the characteristic value of the nailing resistance. This value is reduced by the k_{dens} factor when the density of the material used is less than 350 kg/m³. k_{dens} can be evaluated using the formula $R_{c,k,dens} = R_{c,k} \cdot \left(\frac{\rho_k}{350}\right)^2$;

k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content;

γ_M is the is the partial factor for the connections.

Steel resistance

The tensile design strength of the hold-down can be evaluated according to the formula

$$R_{s,d} = \frac{R_{s,k}}{\gamma_{M2}}$$

where:

$R_{s,k}$ is the characteristic value of the resistance of the tie-down;

γ_{M2} is the partial factor for resistance of cross-sections in tension to fracture.

Bolt resistance

The tension resistance of the bolt is evaluated as reported in the table 3.4 of EN 1993-1-8 by the following formula

$$R_{b,d} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$$

where:

f_{ub} is the ultimate tensile strength of the bolt;

A_s the resistant area of the threaded portion of the shank of the anchor;

γ_{M2} is the partial factor for resistance of cross-sections in tension to fracture.

The checks are summarized in the following table which shows the characteristic values of the resistances associated with collapse of the various components.

Name:	Name of the connection in which the hold-down is used
Comb.:	The most severe combination of load
$T_{a,d}$:	Design value of the tensile force acting on the hold down
$T_{b,d}$:	Design value of the tensile force acting on the bolt
k_{mod} :	Modification factor taking into account the effect of the duration of load and moisture content
$k_{R,deg}$:	Resistance degradation coefficient due to cyclic actions
γ_M :	Partial safety factor
$R_{a,d}$:	Design value of the hold down resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms associated with it

$R_{b,d}$: Design value of the bolt resistance

$$T_{a,d} \leq R_{a,d} = \min(R_{c,d}; R_{s,d})$$

$$T_{b,d} \leq R_{b,d}$$

Wall name	Connection name	Comb.	Service Class	$T_{a,d}$ [kN]	$R_{c,k,dens}$ [kN]	$R_{s,k}$ [kN]	k_{mod}	$k_{R,deg}$	γ_M	γ_{M2}	$R_{a,d}$ [kN]	$T_{b,d}$ [kN]	$R_{b,k}$ [kN]	$R_{b,d}$ [kN]	Failure mode	Check
Wall 52	WKR 285 - WKR 285	Horizontal ULS 1	1	0.00	37.30	26	1.1	1	1.3	1.25	-	0.00	30.3	-	-	0%
Wall 56	WKR 285 - WKR 285	Dynamic SLV 8 ex- ey-	1	9.88	37.30	26	1.1	1	1.3	1.25	20.80	14.33	30.3	24.24	Tension: bolt	59%

Timber to timber tensile plate – Upper levels connection

The design resistance R_d of a punched strap is determined as the minimum value among the resistances relating to the following failure modes:

- nailing failure;
- punched strap steel failure.

Tensile force

The tensile force acting on the punched metal plate (T_a) is evaluated as described in paragraph “Model Description”.

Wall name	Length [m]	Connection name	N° of connections at each wall end	Comb.	Dur.	N [kN]	M_{3-3} [kNm]	T_a [kN]
Wall 40	1.30	Kovinski trak 60mm	1	Horizontal ULS 1	Instantaneous	7.36	0.14	0.00
Wall 41	1.30	Kovinski trak 60mm	1	Horizontal ULS 1	Instantaneous	8.39	0.14	0.00
Wall 43	2.60	Kovinski trak 60mm	1	Horizontal ULS 1	Instantaneous	14.63	0.28	0.00
Wall 45	2.60	Kovinski trak 60mm	1	Horizontal ULS 1	Instantaneous	14.67	0.28	0.00
Wall 47	9.01	Kovinski trak 60mm	1	Horizontal ULS 1	Instantaneous	20.46	0.43	0.00
Wall 53	1.54	Kovinski trak 60mm	1	Dynamic SLV 8 ex- ey-	Instantaneous	3.64	12.95	7.53

Steel resistance

The tensile resistance of the punched element is evaluated on the basis of the indications of 6.2.3 EN 1993-1-1. For sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smaller of the design plastic resistance of the gross cross-section and the design ultimate resistance of the net cross-section at holes for fasteners.

The design plastic resistance of the gross cross-section is calculated as

$$R_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$$

where:

A is the area of the gross cross-section;

f_y is the nominal values of the yield strength of steel;

γ_{M0} is the partial factor for resistance of cross-sections.

The net cross section resistance can be evaluated using the following expression:

$$R_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}}$$

where:

A_{net} is the cross sectional net area;

f_u is the ultimate strength of the yield strength of steel;

γ_{M2} is the partial factor for resistance of cross-sections in tension to fracture.

Nailing resistance

The characteristic resistance of the connection was calculated as the product between the effective number of fasteners inserted and bearing capacity of the single fastener

$$R_{c,k} = n_{ef} \cdot R_{k,conn}$$

where the bearing capacity of the single fastener $R_{k,conn,k}$ is evaluated using Johansen theory and the effective number of fasteners is evaluated in accordance with 8.3.1.1 (8) and 8.5.1.1 (4) – EN 1995-1-1.

Wall name	Connection name	Number of rows	Number of fasteners in a row	Fasteners spacing in one row [mm]	Effective number of fasteners	$R_{conn,k}$ [kN]	$R_{c,k}$ [kN]
Wall 40	Kovinski trak 60mm	3	6	40	13.76	1.99	27.36
Wall 41	Kovinski trak 60mm	3	6	40	13.76	1.99	27.36
Wall 43	Kovinski trak 60mm	3	6	40	13.76	1.99	27.36
Wall 45	Kovinski trak 60mm	3	6	40	13.76	1.99	27.36
Wall 47	Kovinski trak 60mm	3	6	40	13.76	1.99	27.36
Wall 53	Kovinski trak 60mm	3	6	40	13.76	1.99	27.36

The design value of the load-bearing capacity is given by

$$R_{c,d} = \frac{k_{mod} \cdot R_{c,k}}{\gamma_M}$$

where:

$R_{c,k}$ is the characteristic resistance of the fastener;

k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content;

γ_M is the partial factor for connections.

The checks are summarized in the following table which shows the characteristic values of resistance associated with the different failure modes of the components.

Name: Name of the connection in which the punched strap is used

Comb.: The most severe combination of load

$T_{a,d}$: Design force acting on the connection

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

$k_{R,deg}$: Resistance degradation coefficient due to cyclic actions

γ_M : Partial safety factor

R_d : Design value of the resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms considered

$$T_{a,d} \leq \min(R_{pl,Rd}; R_{u,d}; R_{c,d})$$

Wall name	Connection name	Comb.	Service Class	$T_{a,d}$ [kN]	$R_{c,k}$ [kN]	$R_{pl,k}$ [kN]	$R_{u,k}$ [kN]	k_{mod}	$k_{R,deg}$	γ_M	γ_{M0}	γ_{M2}	R_d [kN]	Failure mode	Check
Wall 40	Kovinski trak 60mm	Horizontal ULS 1	1	0.00	27.36	31.5	25.5	1.1	1	1.3	1	1.25	-	-	0%
Wall 41	Kovinski trak 60mm	Horizontal ULS 1	1	0.00	27.36	31.5	25.5	1.1	1	1.3	1	1.25	-	-	0%
Wall 43	Kovinski trak 60mm	Horizontal ULS 1	1	0.00	27.36	31.5	25.5	1.1	1	1.3	1	1.25	-	-	0%
Wall 45	Kovinski trak 60mm	Horizontal ULS 1	1	0.00	27.36	31.5	25.5	1.1	1	1.3	1	1.25	-	-	0%
Wall 47	Kovinski trak 60mm	Horizontal ULS 1	1	0.00	27.36	31.5	25.5	1.1	1	1.3	1	1.25	-	-	0%
Wall 53	Kovinski trak 60mm	Dynamic SLV 8 ex-ey-	1	7.53	27.36	31.5	25.5	1.1	1	1.3	1	1.25	20.40	Tension: net steel section	37%

Timber to concrete angle bracket – Connection at the base of the structure

The design resistance R_d of an angle bracket is determined as the minimum value among the resistances relating to the following failure modes:

- shear failure of the angle and/or of the group of fasteners of the connection;
- shear failure of the anchors connecting the concrete.

Shear force

The shear force acting on the single angle bracket is calculated by dividing the total shear force V_2 by the number of angle brackets present in the wall (taking into account the possible presence of angle brackets on both sides of the structural element).

$$V_a = \frac{V_2}{n_{anc}}$$

where:

V_2 is the design shear force on the considered wall;

n_{anc} is the number of shear connections present in the wall.

The shear force acting on the concrete anchors of each angle bracket is equal to V_a .

Wall name	Length [m]	Connection name	N of connections	Comb.	Dur.	V_2 [kN]	V_a [kN]
Wall 1	1.30	Sidni vijak M16	1	Dynamic SLV 4 ex+ ey+	Instantaneo us	4.52	4.52
Wall 2	2.60	Sidni vijak M16	2	Dynamic SLV 4 ex+ ey+	Instantaneo us	10.49	5.25
Wall 3	2.60	Sidni vijak M16	2	Dynamic SLV 4 ex+ ey+	Instantaneo us	10.49	5.25
Wall 4	1.30	Sidni vijak M16	1	Dynamic SLV 4 ex+ ey+	Instantaneo us	4.52	4.52
Wall 5	3.26	Sidni vijak M16	2	Dynamic SLV 8 ex+ ey+	Instantaneo us	16.57	8.28
Wall 7	1.79	Sidni vijak M16	1	Dynamic SLV 4 ex+ ey+	Instantaneo us	4.94	4.94
Wall 8	1.79	Sidni vijak M16	1	Dynamic SLV 4 ex+ ey+	Instantaneo us	4.94	4.94
Wall 9	1.79	Sidni vijak M16	1	Dynamic SLV 4 ex+ ey+	Instantaneo us	4.94	4.94
Wall 10	8.61	Sidni vijak M16	6	Dynamic SLV 4 ex+ ey+	Instantaneo us	30.05	5.01
Wall 13	9.01	Sidni vijak M16	7	Dynamic SLV 8 ex- ey-	Instantaneo us	44.99	6.43
Wall 15	3.16	Sidni vijak M16	2	Dynamic SLV 4 ex+ ey+	Instantaneo us	10.31	5.16
Wall 16	1.54	Sidni vijak M16	1	Dynamic SLV 8 ex+ ey+	Instantaneo us	8.14	8.14
Wall 18	7.86	Sidni vijak M16	6	Dynamic SLV 4 ex+ ey+	Instantaneo us	43.29	7.22
Wall 20	8.89	Sidni vijak M16	7	Dynamic SLV 8 ex- ey-	Instantaneo us	55.58	7.94
Wall 21	5.82	Sidni vijak M16	4	Dynamic SLV 8 ex- ey-	Instantaneo us	33.47	8.37
Wall 22	1.43	Sidni vijak M16	1	Dynamic SLV 8 ex- ey-	Instantaneo us	6.70	6.70
Wall 23	1.43	Sidni vijak M16	1	Dynamic SLV 8 ex- ey-	Instantaneo us	5.83	5.83
Wall 26	7.00	Sidni vijak M16	5	Dynamic SLV 4 ex+ ey+	Instantaneo us	33.10	6.62
Wall 27	1.49	Sidni vijak M16	1	Dynamic SLV 8 ex+ ey+	Instantaneo us	11.16	11.16

Wall 60	4.08	Sidni vijak M16	3	Dynamic SLV 8 ex- ey+	Instantaneo us	24.15	8.05
Wall 64	0.99	Sidni vijak M16	1	Dynamic SLV 8 ex+ ey+	Instantaneo us	4.42	4.42
Wall 65	1.30	Sidni vijak M16	1	Dynamic SLV 8 ex+ ey+	Instantaneo us	6.07	6.07
Wall 66	1.30	Sidni vijak M16	1	Dynamic SLV 8 ex+ ey+	Instantaneo us	6.07	6.07
Wall 67	1.31	Sidni vijak M16	1	Dynamic SLV 8 ex+ ey+	Instantaneo us	6.12	6.12
Wall 69	6.39	Sidni vijak M16	5	Dynamic SLV 4 ex- ey-	Instantaneo us	37.53	7.51

Angle bracket resistance

The design value of the shear bearing capacity of the angle bracket can be estimated from the characteristic value by means of the following expression

$$R_{a,d} = \frac{k_{mod} \cdot R_{a,k,dens}}{\gamma_M}$$

where:

$R_{a,k,dens}$ is the characteristic value of the nailing resistance. This value is reduced by the k_{dens} factor when the density of the material used is less than 350 kg/m^3 . k_{dens} can be evaluated using the formula $R_{a,k,dens} = R_{a,k} \cdot \left(\frac{\rho_k}{350}\right)^2$.

Concrete anchors resistance

The shear resistance of the concrete anchors is evaluated by the following formula

$$R_{p,d} = \frac{R_{p,k}}{\gamma}$$

where:

$R_{p,k}$ is the characteristic value of the shear resistance of the concrete anchors;

γ is the safety factor.

The checks are summarized in the following table which illustrates the characteristic values of the resistances associated to the different components and their design values.

Name: Name of the connection in which the angle bracket is used

Comb.: The most severe combination of load

$V_{a,d}$: Shear force acting on the angle bracket and on the concrete anchors

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

$k_{R,deg}$: Resistance degradation coefficient due to cyclic actions

γ_M : Partial safety factor

$R_{a,d}$: Design value of the angle bracket resistance

$R_{p,d}$: Design value of the concrete anchors resistance

$$V_{a,d} \leq R_{a,d}$$

$$V_{a,d} \leq R_{p,d}$$

Wall name	Connection name	Comb.	Service class	$V_{a,d}$ [kN]	$R_{a,k,dens}$ [kN]	k_{mod}	$k_{R,deg}$	γ_M	$R_{a,d}$ [kN]	$R_{p,k}$ [kN]	γ	$R_{p,d}$ [kN]	Failure mode	Check
Wall 1	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	4.52	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	37%
Wall 2	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	5.25	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	43%
Wall 3	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	5.25	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	43%
Wall 4	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	4.52	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	37%
Wall 5	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	8.28	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	68%
Wall 7	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	4.94	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	41%
Wall 8	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	4.94	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	41%
Wall 9	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	4.94	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	41%
Wall 10	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	5.01	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	41%
Wall 13	Sidrni vijak M16	Dynamic SLV 8 ex- ey-	1	6.43	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	53%
Wall 15	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	5.16	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	43%
Wall 16	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	8.14	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	67%
Wall 18	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	7.22	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	60%
Wall 20	Sidrni vijak M16	Dynamic SLV 8 ex- ey-	1	7.94	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	66%
Wall 21	Sidrni vijak M16	Dynamic SLV 8 ex- ey-	1	8.37	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	69%
Wall 22	Sidrni vijak M16	Dynamic SLV 8 ex- ey-	1	6.70	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	55%
Wall 23	Sidrni vijak M16	Dynamic SLV 8 ex- ey-	1	5.83	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	48%
Wall 26	Sidrni vijak M16	Dynamic SLV 4 ex+ ey+	1	6.62	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	55%
Wall 27	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	11.16	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	92%
Wall 60	Sidrni vijak M16	Dynamic SLV 8 ex- ey+	1	8.05	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	67%
Wall 64	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	4.42	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	37%
Wall 65	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	6.07	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	50%
Wall 66	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	6.07	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	50%

Wall 67	Sidrni vijak M16	Dynamic SLV 8 ex+ ey+	1	6.12	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	51%
Wall 69	Sidrni vijak M16	Dynamic SLV 4 ex- ey-	1	7.51	14.30	1.1	1	1.3	12.1	14.3	1.00	14.3	Shear: connection element	62%

Timber to timber angle bracket

The design resistance R_d of an angle bracket is determined as the resistance of the following failure mode:

- shear failure of the angle and/or of the group of fasteners of the connection.

Shear force

The shear force acting on the single angle bracket is calculated by dividing the total shear force V_2 by the number of angle brackets present in the wall (taking into account the possible presence of angle brackets on both sides of the structural element).

$$V_a = \frac{V_2}{n_{anc}}$$

where:

V_2 is the design shear force on the considered wall;

n_{anc} is the number of shear connections present in the wall.

Wall name	Length [m]	Connection name	N of connections	Comb.	Dur.	V2 [kN]	Va [kN]
Wall 40	1.30	Vijaki 8mm	4	Dynamic SLV 8 ex+ ey-	Instantaneous	3.76	0.94
Wall 41	1.30	Vijaki 8mm	4	Dynamic SLV 8 ex+ ey-	Instantaneous	3.76	0.94
Wall 43	2.60	Vijaki 8mm	8	Dynamic SLV 8 ex+ ey-	Instantaneous	7.76	0.97
Wall 45	2.60	Vijaki 8mm	8	Dynamic SLV 8 ex+ ey-	Instantaneous	7.76	0.97
Wall 47	9.01	Vijaki 8mm	30	Dynamic SLV 8 ex- ey-	Instantaneous	41.14	1.37
Wall 52	3.16	Vijaki 8mm	10	Dynamic SLV 4 ex+ ey+	Instantaneous	4.16	0.42
Wall 53	1.54	Vijaki 8mm	5	Dynamic SLV 8 ex- ey-	Instantaneous	7.74	1.55
Wall 56	1.49	Vijaki 8mm	4	Dynamic SLV 8 ex- ey-	Instantaneous	9.12	2.28

Angle bracket resistance

The design value of the shear strength of the anchor is evaluated as

$$R_{a,d} = \frac{k_{mod} \cdot R_{a,k,dens}}{\gamma_M}$$

where:

$R_{a,k,dens}$ is the characteristic value of the nailing resistance. This value is reduced by the k_{dens} factor when the density of the material used is less than 350 kg/m^3 . k_{dens} can be evaluated using

$$\text{the formula } R_{a,k,dens} = R_{a,k} \cdot \left(\frac{\rho_k}{350} \right)^2;$$

k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content;

γ_M is the partial factor for connections.

The checks are summarized in the following table which illustrates the characteristic values of the resistance associated to the angle brackets and their design values. The following expression shall be satisfied:

$$V_{a,d} \leq R_{a,d}$$

Name: Name of the connection in which the angle bracket is used

Comb.: The most severe combination of load

$V_{a,d}$: Design value of the force acting on the single angular

k_{mod} : Modification factor taking into account the effect of the duration of load and moisture content

$k_{R,deg}$: Resistance degradation coefficient due to cyclic actions

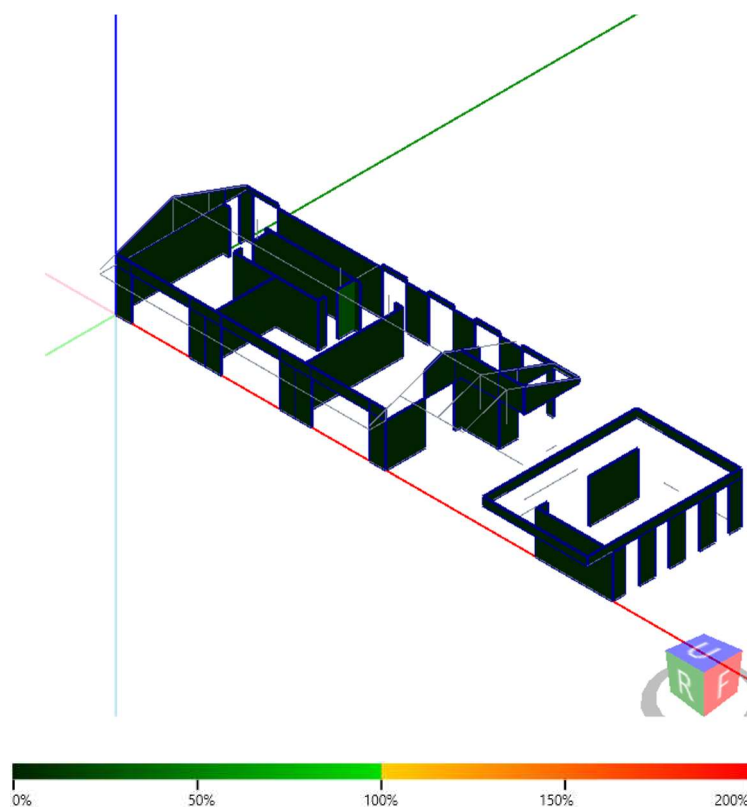
γ_M : Partial safety factor

Wall name	Connection name	Comb.	Service Class	$V_{a,d}$ [kN]	$R_{a,k,dens}$ [kN]	k_{mod}	$k_{R,deg}$	γ_M	$R_{a,d}$ [kN]	Check
Wall 40	Vijaki 8mm	Dynamic SLV 8 ex+ ey-	1	0.94	3.00	1.1	1	1.3	2.54	37%
Wall 41	Vijaki 8mm	Dynamic SLV 8 ex+ ey-	1	0.94	3.00	1.1	1	1.3	2.54	37%
Wall 43	Vijaki 8mm	Dynamic SLV 8 ex+ ey-	1	0.97	3.00	1.1	1	1.3	2.54	38%
Wall 45	Vijaki 8mm	Dynamic SLV 8 ex+ ey-	1	0.97	3.00	1.1	1	1.3	2.54	38%
Wall 47	Vijaki 8mm	Dynamic SLV 8 ex- ey-	1	1.37	3.00	1.1	1	1.3	2.54	54%
Wall 52	Vijaki 8mm	Dynamic SLV 4 ex+ ey+	1	0.42	3.00	1.1	1	1.3	2.54	16%
Wall 53	Vijaki 8mm	Dynamic SLV 8 ex- ey-	1	1.55	3.00	1.1	1	1.3	2.54	61%
Wall 56	Vijaki 8mm	Dynamic SLV 8 ex- ey-	1	2.28	3.00	1.1	1	1.3	2.54	90%

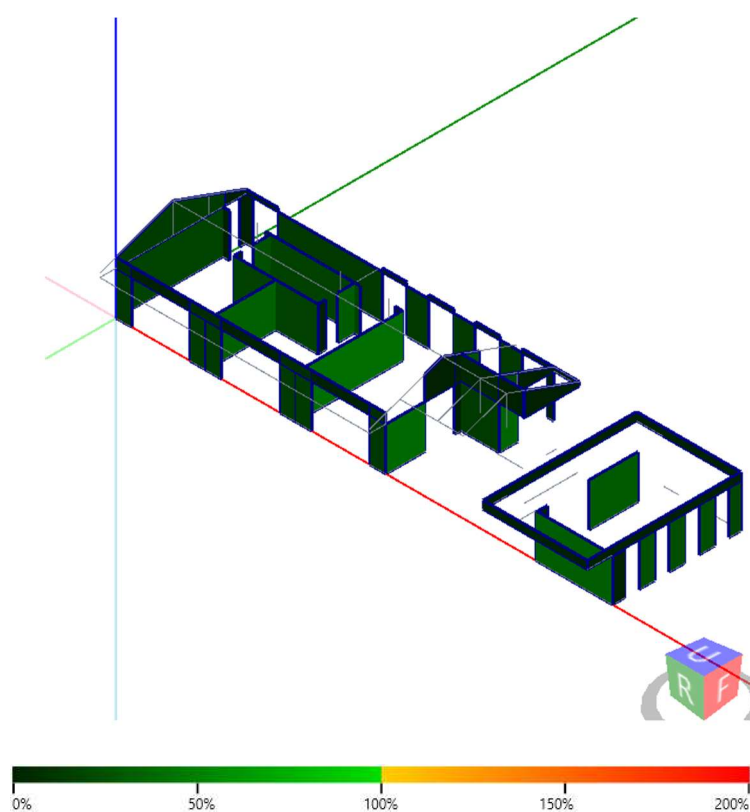
Verification percentage of connections

Ultimate Limit State (ULS)

Tensile connection elements verifications

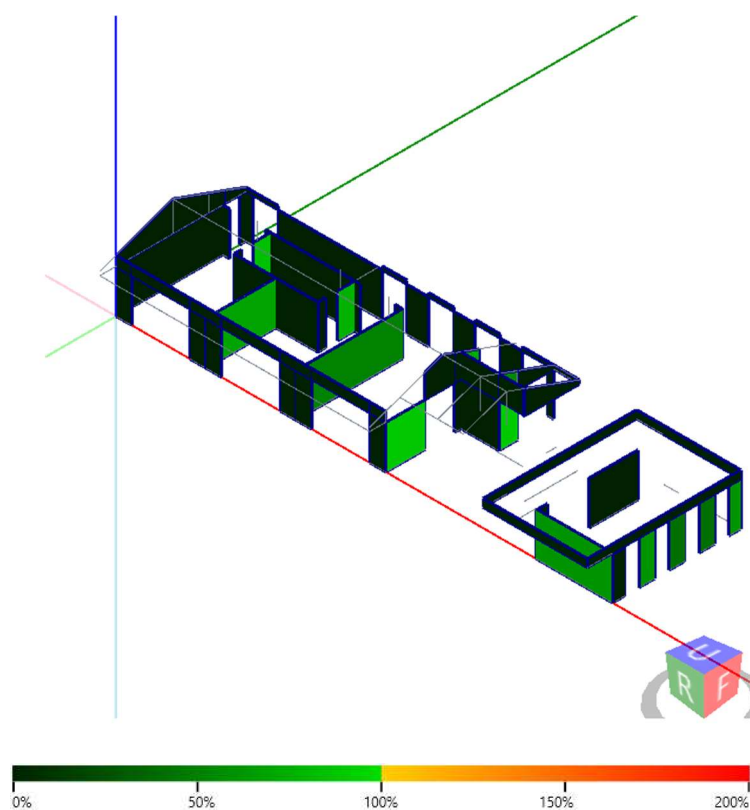


Shear connection elements verifications

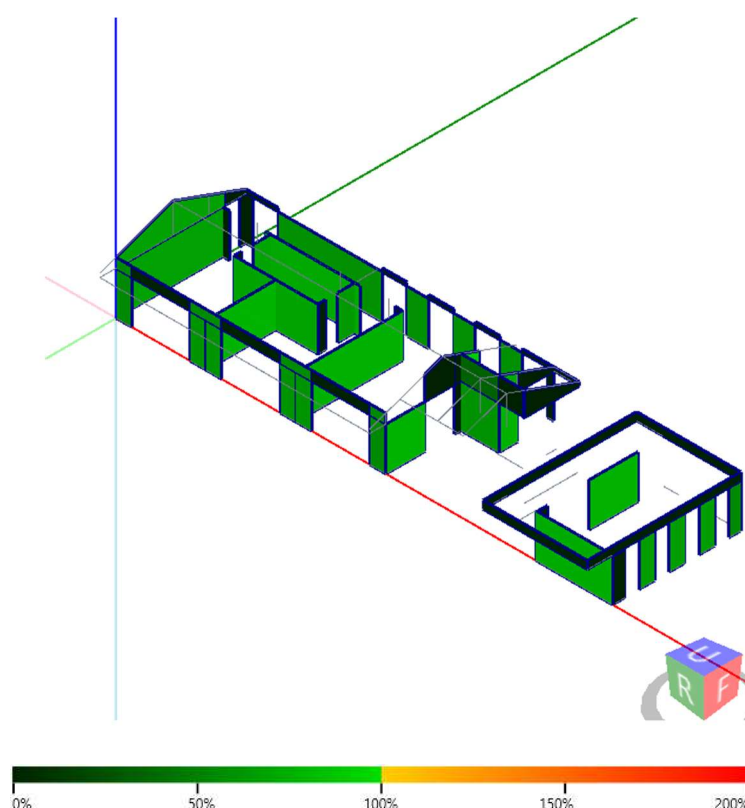


Life Safety Limit State (SLV dynamic)

Tensile connection elements verifications



Shear connection elements verifications



Damage Limitation - DLS

The “damage limitation requirement” is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the “no-collapse requirement” the interstorey drifts are limited in accordance with the following equation:

$$d_r < d_{r,lim} = 0.005 h$$

where:

d_r is the interstorey drift obtained applying the reduction factor $\nu = 0.4$ to the design ground acceleration a_g on type A ground;

h is the storey height.

The table below shows the seismic checks for the Damage Limit State.

Wall name: Wall ID

h: Storey height

Comb.: The most severe combination of load

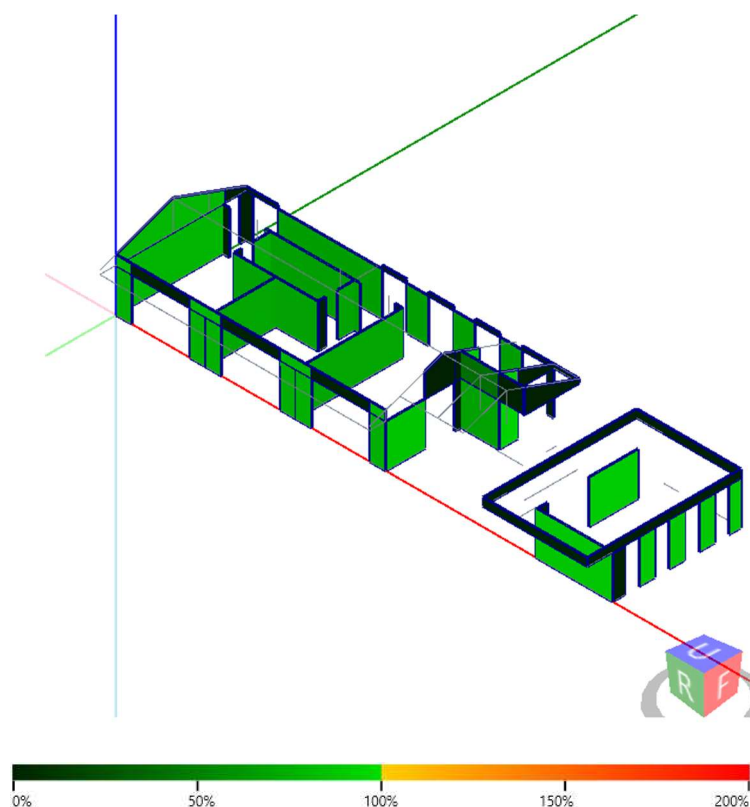
d_r : Evaluated interstorey drift

$d_{r,lim}$: Interstorey drift limit

The table provides the Damage Limit State checks in the case of Dynamic Linear Analysis.

Wall	h [m]	Comb.	d _r [mm]	d _{lim} [mm]	Verifica
Wall 1	3.60	Dynamic SLD 4 ex+ ey+	11.69	18.00	65%
Wall 2	3.60	Dynamic SLD 4 ex+ ey+	11.69	18.00	65%
Wall 3	3.60	Dynamic SLD 4 ex+ ey+	11.69	18.00	65%
Wall 4	3.60	Dynamic SLD 4 ex+ ey+	11.69	18.00	65%
Wall 5	3.60	Dynamic SLD 5 ex+ ey+	15.01	18.00	83%
Wall 7	3.60	Dynamic SLD 4 ex+ ey+	9.20	18.00	51%
Wall 8	3.60	Dynamic SLD 4 ex+ ey+	9.20	18.00	51%
Wall 9	3.60	Dynamic SLD 4 ex+ ey+	9.20	18.00	51%
Wall 10	3.60	Dynamic SLD 4 ex+ ey+	9.20	18.00	51%
Wall 13	3.60	Dynamic SLD 8 ex- ey-	12.79	18.00	71%
Wall 15	3.60	Dynamic SLD 4 ex+ ey+	9.60	18.00	53%
Wall 16	3.60	Dynamic SLD 5 ex+ ey+	17.58	18.00	98%
Wall 18	3.60	Dynamic SLD 4 ex+ ey+	9.21	18.00	51%
Wall 20	3.60	Dynamic SLD 8 ex- ey-	10.22	18.00	57%
Wall 21	3.60	Dynamic SLD 8 ex- ey-	10.30	18.00	57%
Wall 22	3.60	Dynamic SLD 8 ex- ey-	11.31	18.00	63%
Wall 23	3.60	Dynamic SLD 8 ex- ey-	9.84	18.00	55%
Wall 26	3.60	Dynamic SLD 4 ex+ ey+	9.70	18.00	54%
Wall 27	3.60	Dynamic SLD 5 ex+ ey+	15.01	18.00	83%
Wall 40	0.79	Dynamic SLD 5 ex+ ey-	2.78	3.94	71%
Wall 41	0.79	Dynamic SLD 5 ex+ ey-	2.78	3.94	71%
Wall 43	0.79	Dynamic SLD 5 ex+ ey-	2.78	3.94	71%
Wall 45	0.79	Dynamic SLD 5 ex+ ey-	2.78	3.94	71%
Wall 47	1.68	Dynamic SLD 8 ex- ey-	6.05	8.42	72%
Wall 52	1.95	Dynamic SLD 4 ex+ ey+	2.13	9.75	22%
Wall 53	1.67	Dynamic SLD 8 ex- ey-	7.63	8.36	91%
Wall 56	1.68	Dynamic SLD 8 ex- ey-	7.31	8.41	87%
Wall 60	3.60	Dynamic SLD 8 ex- ey+	16.31	18.00	91%
Wall 64	3.60	Dynamic SLD 5 ex+ ey+	15.68	18.00	87%
Wall 65	3.60	Dynamic SLD 5 ex+ ey+	15.68	18.00	87%
Wall 66	3.60	Dynamic SLD 5 ex+ ey+	15.68	18.00	87%
Wall 67	3.60	Dynamic SLD 5 ex+ ey+	15.68	18.00	87%
Wall 69	3.60	Dynamic SLD 4 ex- ey-	15.34	18.00	85%

The verification percentages of walls are shown below.

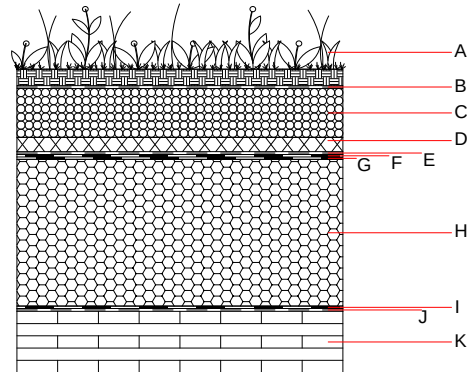


Flachdach/ flachgeneigtes Dach - fdmogo01-01

Flachdach/ flachgeneigtes Dach, Holzmassivbau, nicht hinterlüftet, ohne Installationsebene, ohne, Holz sichtbar

Bauphysikalische Bewertung

Brandschutz	REI	60
max. Spannweite = 5 m; max. Last $E_{d,fi}$ = 5 kN/m ² Klassifizierung durch HFA		
Wärmeschutz	U	0,10 W/(m ² K)
	Diffusionsverhalten	geeignet
Berechnung durch IBO		
Schallschutz	R_w (C_c; C_{tr})	55(-4;-9) dB
	L_{n,w} (C_i)	
Bewertung durch HFA		
Flächenbezogene Masse	m	241,00 kg/m ²



Baustoffangaben zur Konstruktion, Schichtaufbau (von außen nach innen, Maße in mm)

	Dicke	Baustoff	Wärmeschutz				Brandverhaltensklasse EN
			λ	μ min – max	ρ	c	
A	80,0	Extensive Begrünung			1700		
B	0,3	Filtervlies (Polypropylen)			300		
C	120,0	Perliteschüttung (Bläherlite lose) 100 kg/m ³ , feucht			100	1,000	
D	35,0	Drän- und Wasserspeicherelement (PE)			20		
E	0,3	Trennvlies (PP-Schutzvlies für Abdichtung und Trennlage)					
F	1,8	Abdichtungsbahn EPDM (wurzelfest)	0,170		1200		E
G		Glasvlies (werkseitig auf EPDM aufkaschiert)					
H	260,0	EPS-W plus 30 Aufdachdämmung im Gefälle verlegt; druckfest	0,035	60	28	1,450	E
I	4,0	Dampfsperrebahn - Bitumenbahn mit Aluminiumeinlage sd-Wert 1500m	0,230		1100		E
J	0,2	Glasvliesgewebe					
K	150,0	Brettspertholz (verklebt) 5-lagig Decklage mind. 40mm	0,130	50	500	1,600	D

Ökologische Bewertung (pro m² Konstruktionsfläche)

Datenbasis ecoinvent

Verbaute Menge an NAWAROS	kg	70,181
Anteil NAWAROS am Gesamtgewicht	%	29,26
Einsatz an Primärenergie	MJ	1711,486
Erneuerbarer Primärenergieanteil	%	3,98
$\Sigma \Delta OI3$		133,0

Berechnung durch IBO

Ökologische Bewertung im Detail

Datenbasis Datenbank ecoinvent

Lebenszyklus (Phasen)	GWP _{Fossil} [kg CO ₂ Äqv.]	GWP _{Biogen} [kg CO ₂ Äqv.]	GWP _{Total} [kg CO ₂ Äqv.]	AP [kg SO ₂ Äqv.]	EP [kg PO ₄ Äqv.]	ODP [kg R11 Äqv.]	POCP [kg Ethen Äqv.]
A1 - A3	101,445	-109,899	-8,454	0,398	0,111	9,66E-6	0,153

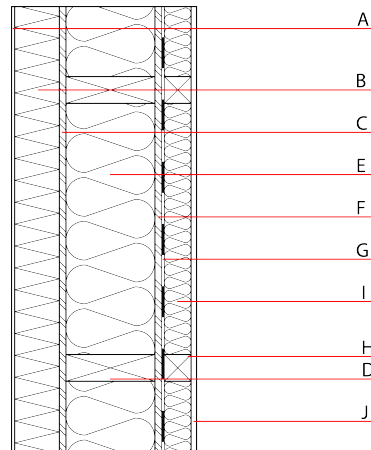
Lebenszyklus (Phasen)	PERE [MJ]	PERM [MJ]	PERT [MJ]	PENRE [MJ]	PENRM [MJ]	PENRT [MJ]
A1 - A3	68,160	1026,000	1094,160	1643,326	795,873	2439,199

Aussenwand - awropi31a-00

Aussenwand, Holzrahmen/Holztafel, nicht hinterlüftet, mit Installationsebene, geputzt, Gipsplatte

Bauphysikalische Bewertung

Brandschutz	REI von innen REI von außen	60 90
max. Wandhöhe = 3 m; max. Last $E_{d,fi}$ = 32,0 kN/m Klassifizierung durch HFA		
Wärmeschutz	U Diffusionsverhalten	0,12 W/(m ² K) geeignet
Berechnung durch HFA		
Schallschutz	R_w (C;C_{tr}) L_{n,w} (C_i)	46(-2;-8) dB
[C ₅₀₋₃₁₅₀ ; C _{tr,50-3150}] = [-8; -19] dB Bewertung durch HFA		
Flächenbezogene Masse	m	71,30 kg/m ²



Baustoffangaben zur Konstruktion, Schichtaufbau (von außen nach innen, Maße in mm)

	Dicke	Baustoff	Wärmeschutz				Brandverhaltensklasse EN
			λ	μ min – max	ρ	c	
A	7,0	Putzsystem	1,000	10 - 35	2000	1,130	A1
B	100,0	MW-PT [040; ≥100]	0,040	1	100	1,030	A1
C	15,0	OSB	0,130	200	600	1,700	D
D	200,0	Konstruktionsholz (60/..; e=625)	0,120	50	450	1,600	D
E	200,0	Mineralwolle [040; ≥30; ≥1000°C]	0,040	1	30	1,030	A1
F	15,0	OSB	0,130	200	600	1,700	D
G		Dampfbremse sd ≥ 14m					
H	60,0	Holz Fichte - Lattung (60/60) vertikal (a=625)	0,120	50	450	1,600	D
I	60,0	Mineralwolle [035; 20; ≥1000°C]	0,035	1	20	1,030	A1
J	12,5	Gipsplatte Typ DF (GKF) oder	0,250	10	800	1,050	A2
J	12,5	Gipsfaserplatte	0,320	21	1000	1,100	A2

Ökologische Bewertung (pro m² Konstruktionsfläche)

Datenbasis ecoinvent

Verbaute Menge an NAWAROS	kg	30,491
Anteil NAWAROS am Gesamtgewicht	%	41,53
Einsatz an Primärenergie	MJ	832,879
Erneuerbarer Primärenergieanteil	%	14,33
ΣΔO13		66,6

Berechnung durch HFA

Ökologische Bewertung im Detail

Datenbasis Datenbank ecoinvent

Lebenszyklus (Phasen)	GWP _{Fossil} [kg CO ₂ Äqv.]	GWP _{Biogen} [kg CO ₂ Äqv.]	GWP _{Total} [kg CO ₂ Äqv.]	AP [kg SO ₂ Äqv.]	EP [kg PO ₄ Äqv.]	ODP [kg R11 Äqv.]	POCP [kg Ethen Äqv.]
A1 - A3	51,206	-50,513	0,692	0,313	0,095	3,04E-6	0,115

Lebenszyklus (Phasen)	PERE [MJ]	PERM [MJ]	PERT [MJ]	PENRE [MJ]	PENRM [MJ]	PENRT [MJ]
A1 - A3	119,384	521,844	641,228	713,494	28,482	741,976

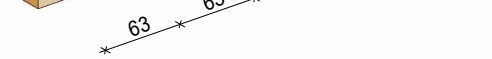
2.6	TEHNIČNI PRIKAZI	
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1.	KARAKTERISTIČEN PRIKAZ LESENIH STEN IN SIDRANJA	M 1:50
2.	VRTEC - OPAŽNI NAČRT TEMELJNE PLOŠČE	M 1:50
3.	VRTEC - ARMATURNI NAČRT TEMELJNE PLOŠČE	M 1:50
	Izvleček in specifikacija armature	
4.	VRTEC - ARMATURNI NAČRT NADTEMELJNEGA ZIDU	M 1:50
	Izvleček in specifikacija armature	
5.	VRTEC - TLORIS PRITLIČJA - NOSILNA KONSTRUKCIJA	M 1:50
6.	VRTEC - TLORIS PRITLIČJA - NOSILNA KONSTRUKCIJA - PREKLADE	M 1:50
7.	VRTEC - TLORIS PRITLIČJA - SIDRANJE	M 1:50
8.	VRTEC - TLORIS PODSTREŠJA - NOSILNA KONSTRUKCIJA	M 1:50
9.	VRTEC - TLORIS PODSTREŠJA - SIDRANJE	M 1:50
10.	VRTEC - TLORIS OSTREŠJA - NOSILNA KONSTRUKCIJA	M 1:50
11.	VRTEC - PREREZI	M 1:50
12.	VRTEC - 3D POGLEDI - NOSILNA KONSTRUKCIJA	M 1:50
13.	KUHINJA - OPAŽNI NAČRT TEMELJNE PLOŠČE	M 1:50
14.	KUHINJA - ARMATURNI NAČRT TEMELJNE PLOŠČE IN NADTEMELJNEGA ZIDU	M 1:50
	Izvleček in specifikacija armature	
15.	KUHINJA - TLORISI - NOSILNA KONSTRUKCIJA	M 1:50
16.	KUHINJA - TLORIS PRITLIČJA - SIDRANJE	M 1:50
17.	KUHINJA - PREREZI	M 1:50
18.	KUHINJA - 3D POGLEDI - NOSILNA KONSTRUKCIJA	M 1:50

KARAKTERISTIČEN PRIKAZ - STENA - OKVIR 100 e=417 mm



KARAKTERISTIČEN PRIKAZ - STENA - OKVIR 200



KARAKTERISTIČEN PRIKAZ - STENA - OKVIR 140



KARAKTERISTIČEN PRIKAZ - STENA - OKVIR 200_EXT=160



KARAKTERISTIČEN PRIKAZ - SIDRANJE - DVIŽNO IN STRIŽNO - BETON



KARAKTERISTIČEN PRIKAZ - SIDRANJE - DVOJNI DVIŽNI KOTNIK



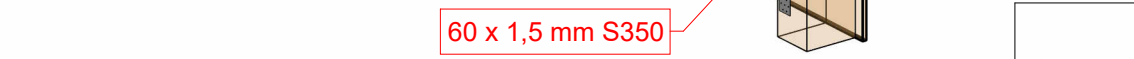
KARAKTERISTIČEN PRIKAZ - SIDRANJE - 2xDVIŽNI KOTNIK



KARAKTERISTIČEN PRIKAZ - SIDRANJE - OBEŠANJE - NAVOJNE PALICE



KARAKTERISTIČEN PRIKAZ - SIDRANJE - DVIŽNO IN STRIŽNO - LES



SIDRANJE - opisi in simboli

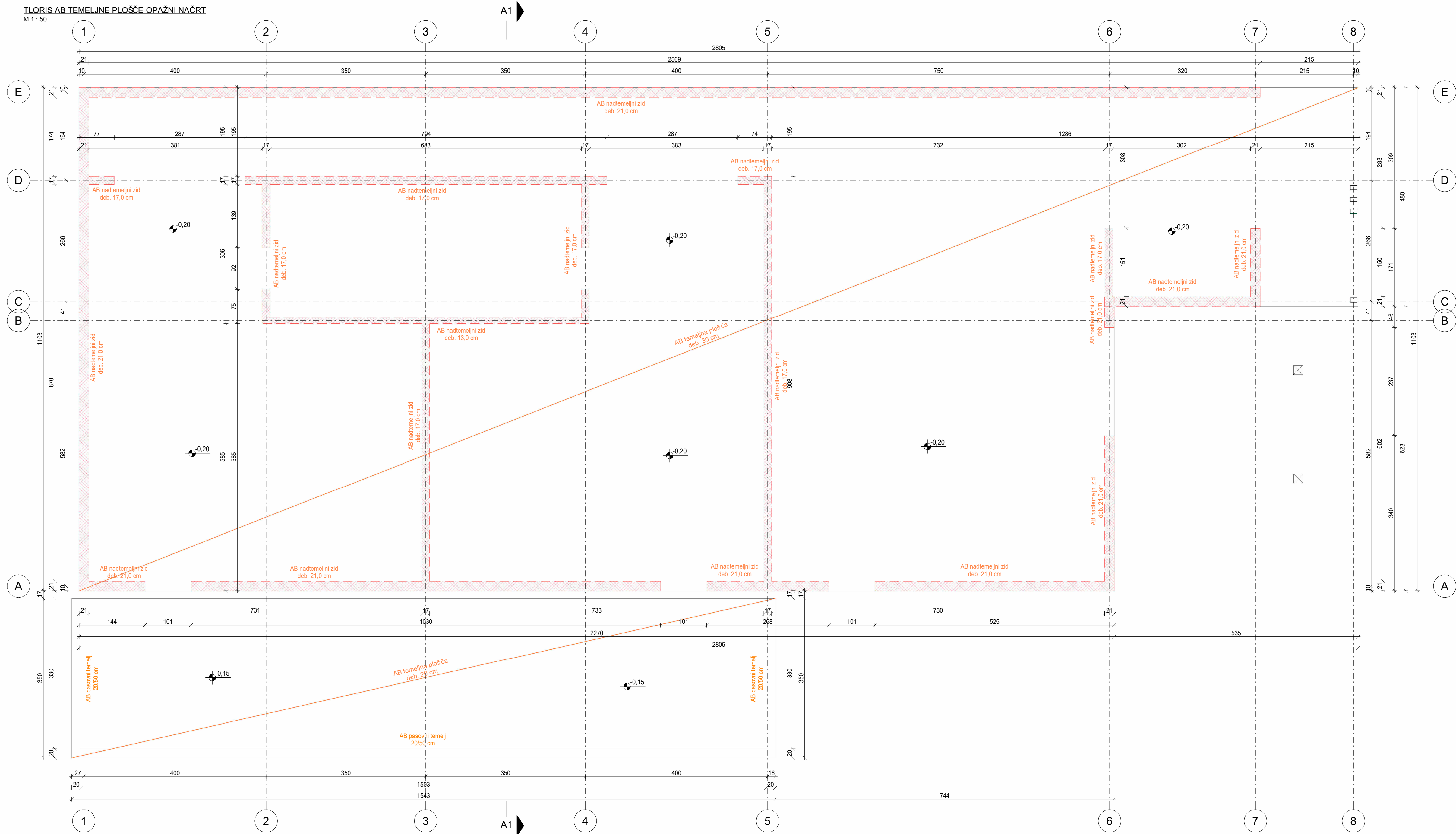
- ## DVIŽNI KOTNIK

TIPI LESENIH OKVIRNIH STEN Z OPISI	
Tip stene	Opis
Okvir 100 e=417mm	Pokončniki: C24 - notranji 80x100 mm; robni 80x100 mm Razmak: 417 mm Zgornja in spodnja pasnica: C24 - 80x100 mm Dvostranska poploščitev: OSB/3 deb. 15 mm Sponke: Haubold KG 745, razmak s=70 mm, nagib >30°
Okvir 140	Pokončniki: C24 - notranji 80x140 mm; robni 80x140 mm Razmak: 625 mm Zgornja in spodnja pasnica: C24 - 80x140 mm Dvostranska poploščitev: OSB/3 deb. 15 mm Sponke: Haubold KG 745, razmak s=70 mm, nagib >30°
Okvir 200	Pokončniki: C24 - notranji 80x200 mm; robni 160x200 mm Razmak: 625 mm Zgornja in spodnja pasnica: C24 - 80x200 mm Enostranska poploščitev: OSB/3 deb. 15 mm Sponke: Haubold KG 745, razmak s=70 mm, nagib >30°
Okvir 200_ext=160	Pokončniki: C24 - notranji 80x200 mm; robni 160x200 mm Razmak: 625 mm Zgornja in spodnja pasnica: C24 - 80x200 mm Enostranska poploščitev: OSB/3 deb. 15 mm Sponke: Haubold KG 745, razmak s=70 mm, nagib >30°

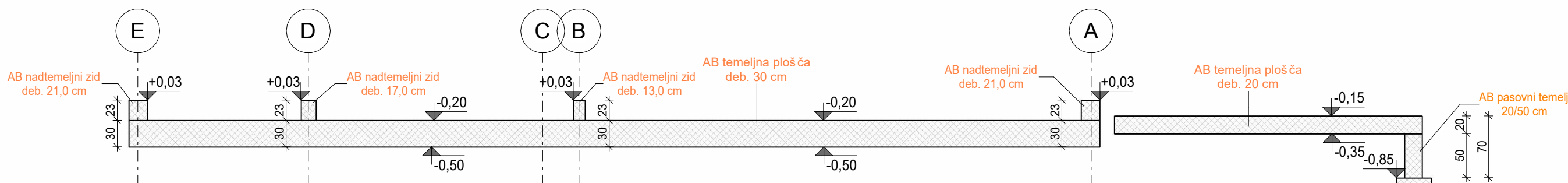
KARAKTERISTIČEN PRIKAZ LESENIH STEN IN SIDRANJA

Sprememba:		Datum spremembe:	
		Podpis:	
DOTES			
Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št.na5rta:	18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVIA	Št.projekta:	01/2025
Vsebina risbe			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:	Merilo: 1:50		
Datum:	Maj 2026		List št.: 1

TLORIS AB TEMELJNE PLOŠČE-OPAŽNI NAČRT
M 1 : 50



PREČNI PREREZ A1
M 1 : 50



OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. Samovoljno prilagajanje projekta ni dovoljeno

UPORABLJENI MATERIALI:

- BETON C25/30, XC2, Cl 0,2, d_{max} = 32, S3
- ARMATURNO JEKLO ZA PALICE B500 B
- ARMATURNO JEKLO ZA MREŽE B500 A

KROVNI SLOJI BETONA c_{nom}:

- AB PLOŠČE: 3 cm

LEGENDA MATERIALOV

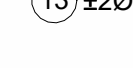
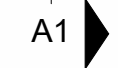
- ARMIRANI BETON
- AB STENE NAD PLOŠČO
- AB STENE IN NOSILCI POD PLOŠČO
- AB STEBRI, VV NAD PLOŠČO
- OPEČNE STENE NAD PLOŠČO

VRTEC - OPAŽNI NAČRT TEMELJNE PLOŠČE

Sprememba: Datum spremembe:
Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA	
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA	
Lokacija:		
Vrsta projekta:	PZI - projekt za izvedbo	Št. načrta: 18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADNENIŠTVA	Št. projekta: 01/2025
Vsebina risbe:		
Vodja projekta:	Jure Kotnik, mag.inž.arh. ZAPS 2309 PA	
Pooblaščen inženir:	Miha Oražem, u.d.l.g., IZS G-3801	
Projektant:		
Datum:	Maj 2026	Merilo: 1:50 List št.: 2



- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

- BETON C25/30, XC2, Cl 0,2, $d_{max} = 32$, S3

- ARMATURNO JEKLO ZA MREŽE B500 A

- AB PLOŠČE: 3 cm

	ARMIRANI BETON
	AB STENE NAD PLOŠČO
	AB STENE IN NOSILCI POD PLOŠČO
	AB STEBRI, VV NAD PLOŠČO
	OPEČNE STENE NAD PLOŠČO

Spremembra:	Datum spremembe:
	Podpis:



Projektanti:
Merilo: 1:50

Objekt: □

Št.risbe: 2□

Projekt

Objekt:

Die Längenangabe wird auf der Basis der Außenkontur berechnet.

Seznam količin

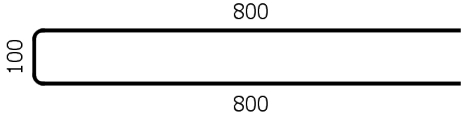
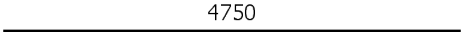
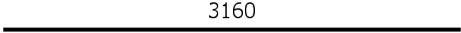
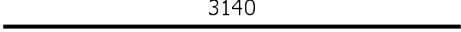
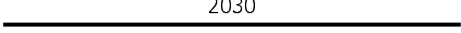
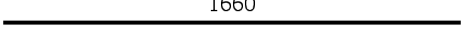
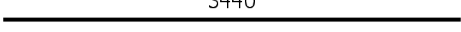
Material: Default									
Premer armaturnih palic		Tip elementov	Teža na m ¹		Skupaj dolžina		Skupaj teža		
8 mm		ø8, Dmin=4φ	0.41 kg/m		2490.16		1020.97 kg		
14 mm		ø14, Dmin=4φ	1.25 kg/m		860.37		1075.46 kg		
Skupaj:							2096.43 kg		

Izvleček armaturnih palic

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab-form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	1	8 mm	1.41	446	628.86		31	2 2	450	150	450	150		100 100	
	2	8 mm	1.80	515	927.00		21		800	200	800				
	3	8 mm	1.33	257	341.81		31	2 2	450	110	450	110		100 100	
	4	8 mm	2.50	4	10.00		00		2500						
Kommentar: ±															
	5	8 mm	1.73	93	160.89		31	2 2	140	620	140	620		100 100	

Objekt: ☐

Št.risbe: 2☐

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	6	8 mm	1.70	248	421.60		21		800	100	800				
															
	7	14 mm	4.75	4	19.00		00		4750						
Kommentar: ±															
	8	14 mm	3.16	4	12.64		00		3160						
Kommentar: ±															
	9	14 mm	3.14	4	12.56		00		3140						
Kommentar: ±															
	10	14 mm	2.03	4	8.12		00		2030						
Kommentar: ±															
	11	14 mm	1.66	4	6.64		00		1660						
Kommentar: ±															
	12	14 mm	3.44	18	61.92		00		3440						
Kommentar: ±															

Objekt: ☐

Št.risbe: 2☐

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	13	14 mm	6.00	78	468.00		00		6000						
Kommentar: ±									6000						
	14	14 mm	5.37	13	69.81		00		5370						
Kommentar: ±									5370						
	15	14 mm	6.01	16	96.16		00		6010						
Kommentar: ±									6010						
	16	14 mm	3.00	4	12.00		00		3000						
Kommentar: ±									3000						
	17	14 mm	5.98	4	23.92		00		5980						
Kommentar: ±									5980						
	18	14 mm	2.65	4	10.60		00		2650						
Kommentar: ±									2650						
	19	14 mm	2.21	4	8.84		00		2210						
Kommentar: ±									2210						

Objekt: ☐

Št.risbe: 2☐

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	20	14 mm	1.32	4	5.28		00		1320						
Kommentar: ±									1320						
	21	14 mm	4.01	4	16.04		00		4010						
Kommentar: ±									4010						
	22	14 mm	5.12	4	20.48		00		5120						
Kommentar: ±									5120						
	23	14 mm	2.09	4	8.36		00		2090						
Kommentar: ±									2090						

Seznam armaturnih mrež

Konst. element	Oznaka	Tip mreže	Teža/kos neto	Kom	Skupaj teža neto	Dolžina x Širina	Oblika	A	B	C	D	E / R	Rev
	1	Q503	103.00 kg	28	2884.00 kg	6000 x 2150	00						
Kommentar: ±													
	2	Q503	94.07 kg	14	1317.03 kg	5480 x 2150	00						
Kommentar: ±													
	3	Q503	51.50 kg	14	721.00 kg	3000 x 2150	00						
Kommentar: ±													
	4	Q503	51.16 kg	14	716.19 kg	2980 x 2150	00						
Kommentar: ±													
	5	Q503	29.46 kg	2	58.93 kg	3000 x 1230	00						
Kommentar: ±													
	6	Q503	82.06 kg	2	164.11 kg	4780 x 2150	00						
Kommentar: ±													
	7	Q503	39.14 kg	2	78.28 kg	2280 x 2150	00						
Kommentar: ±													

Objekt: 2

Št.risbe: 2

Konst. element	Oznaka	Tip mreže	Teža/kos neto	Kom	Skupaj teža neto	Dolžina x Širina	Oblika	A	B	C	D	E / R	Rev
	8	Q503	61.07 kg	2	122.13 kg	4780 x 1600	00						
Kommentar: ±													
	9	Q335	69.45 kg	4	277.80 kg	6000 x 2150	00						
Kommentar: ±													
	10	Q335	49.43 kg	2	98.85 kg	4270 x 2150	00						
Kommentar: ±													
	11	Q335	28.10 kg	2	56.21 kg	3000 x 1740	00						
Kommentar: ±													
	12	Q335	56.21 kg	4	224.82 kg	6000 x 1740	00						
Kommentar: ±													
	13	Q335	16.11 kg	2	32.22 kg	1720 x 1740	00						
Kommentar: ±													

Kosovnica armaturnih mrež

Tip mreže Q335	
Širina: 2150; Länge: 6000; B.teža: 69.45 kg; N.teža: 69.45 kg; Kom: 4; B. teža skupaj: 277.80 kg; N. teža skupaj: 277.80 kg;	
Širina: 2150; Länge: 6000; B.teža: 69.45 kg; N.teža: 56.21 kg; Kom: 4; B. teža skupaj: 277.80 kg; N. teža skupaj: 224.82 kg;	
Širina: 2150; Länge: 6000; B.teža: 69.45 kg; N.teža: 65.54 kg; Kom: 2; B. teža skupaj: 138.90 kg; N. teža skupaj: 131.08 kg;	
Širina: 2150; Länge: 6000; B.teža: 69.45 kg; N.teža: 56.21 kg; Kom: 1;	

Tip mreže Q335		
B. teža skupaj: 69.45 kg; N. teža skupaj: 56.21 kg;		
Skupaj	11	Bruto teža 763.95 kg Neto teža 689.91 kg

Tip mreže Q503	
Širina: 2150; Länge: 6000; B.teža: 103.00 kg; N.teža: 103.00 kg; Kom: 28; B. teža skupaj: 2884.00 kg; N. teža skupaj: 2884.00 kg;	
Širina: 2150; Länge: 6000; B.teža: 103.00 kg; N.teža: 94.07 kg; Kom: 14; B. teža skupaj: 1442.00 kg; N. teža skupaj: 1317.03 kg;	

Objekt: ☐

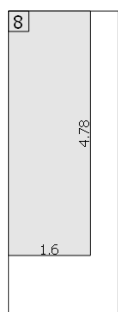
Št.risbe: 2☐

Tip mreže Q503

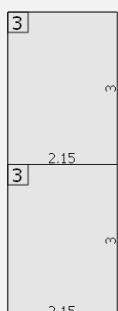
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 82.06 kg;
Kom: 2;
B. teža skupaj: 206.00 kg;
N. teža skupaj: 164.11 kg;



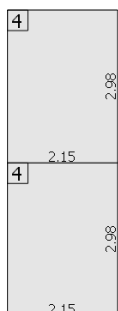
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 61.07 kg;
Kom: 2;
B. teža skupaj: 206.00 kg;
N. teža skupaj: 122.13 kg;



Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 103.00 kg;
Kom: 7;
B. teža skupaj: 721.00 kg;
N. teža skupaj: 721.00 kg;

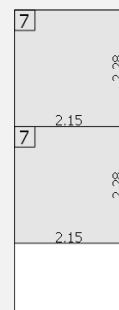


Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 102.31 kg;
Kom: 7;

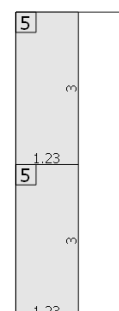


Tip mreže Q503

B. teža skupaj: 721.00 kg;
N. teža skupaj: 716.19 kg;
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 78.28 kg;
Kom: 1;
B. teža skupaj: 103.00 kg;
N. teža skupaj: 78.28 kg;



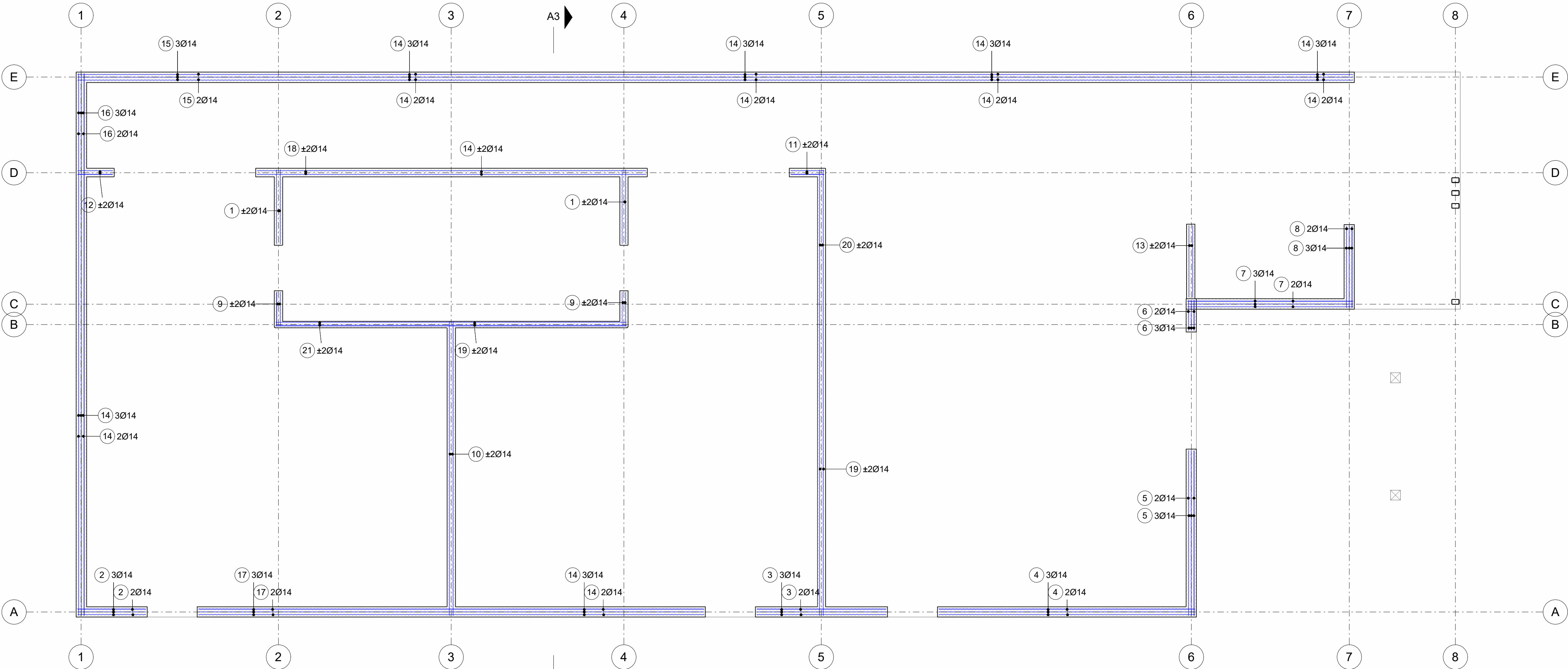
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 58.93 kg;
Kom: 1;
B. teža skupaj: 103.00 kg;
N. teža skupaj: 58.93 kg;



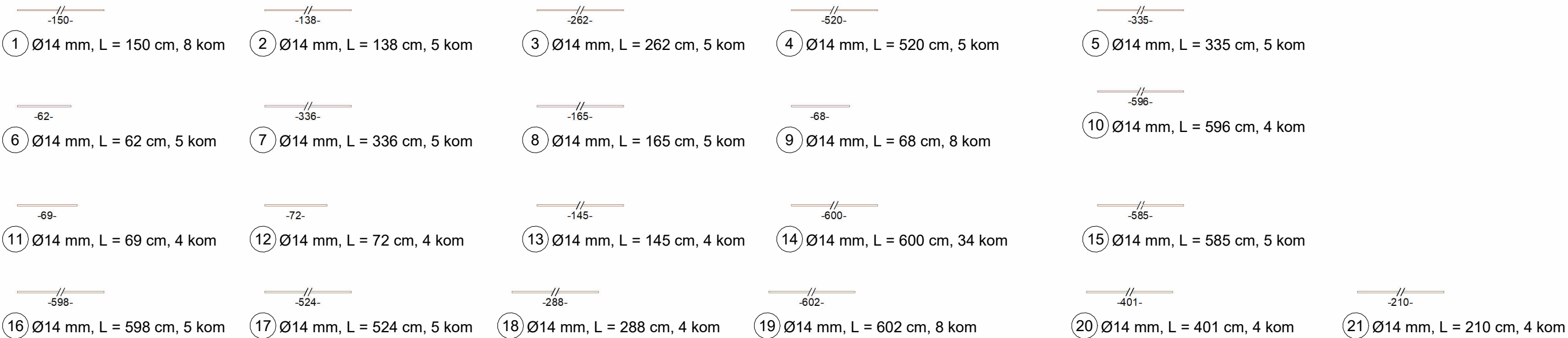
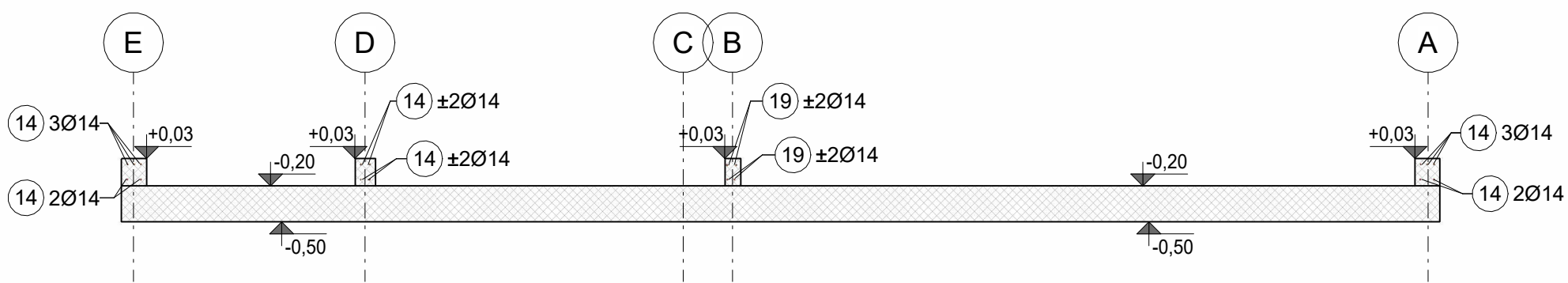
Skupaj

62

Bruto teža
6386.00 kg
Neto teža
6061.67 kg



PREČNI PREREZ A2
M 1 : 50



OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

UPORABLJENI MATERIALI:

- BETON C25/30, XC2, Cl 0.2, $d_{max} = 32$, S3
- ARMATURNO JEKLO ZA PALICE B500 B
- ARMATURNO JEKLO ZA MREŽE B500 A

KROVNI SLOJI BETONA c_{nom} :

- AB PLOŠČE: 3 cm

LEGENDA MATERIALOV

- ARMIRANI BETON
- AB STENE NAD PLOŠČO
- AB STENE IN NOSILCI POD PLOŠČO
- AB STEBRI, VV NAD PLOŠČO
- OPEČNE STENE NAD PLOŠČO

VRTEC - ARMATURNI NAČRT NADTEMELJNEGA ZIDU

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št. načrta:	18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVIA	Št. projekta:	01/2025
Vsebinska risba:			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:			
Datum:	Maj 2026	Merilo:	1:50
		List št.:	4

Objekt: □

Št.risbe: 3□

Projekt

Objekt:

Die Längenangabe wird auf der Basis der Außenkontur berechnet.

Seznam količin

Material: Default				
Premier armaturnih palic	Tip elementov	Teža na m ¹	Skupaj dolžina	Skupaj teža
14 mm	ø14, Dmin=4φ	1.25 kg/m	517.09	646.36 kg
Skupaj:				646.36 kg

Izvleček armaturnih palic

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab-form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	1	14 mm	1.50	8	12.00		00		1500						
Kommentar: ±									1500						
	2	14 mm	1.38	5	6.90		00		1380						
									1380						
	3	14 mm	2.62	5	13.10		00		2620						
									2620						
	4	14 mm	5.20	5	26.00		00		5200						
									5200						
	5	14 mm	3.35	5	16.75		00		3350						
									3350						

Objekt: □

Št.risbe: 3□

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	6	14 mm	0.62	5	3.10		00		620						
									620						
	7	14 mm	3.36	5	16.80		00		3360						
									3360						
	8	14 mm	1.65	5	8.25		00		1650						
									1650						
	9	14 mm	0.68	8	5.44		00		680						
Kommentar: ±									680						
	10	14 mm	5.96	4	23.84		00		5960						
Kommentar: ±									5960						
	11	14 mm	0.69	4	2.76		00		690						
Kommentar: ±									690						
	12	14 mm	0.72	4	2.88		00		720						
Kommentar: ±									720						

Objekt: ☐

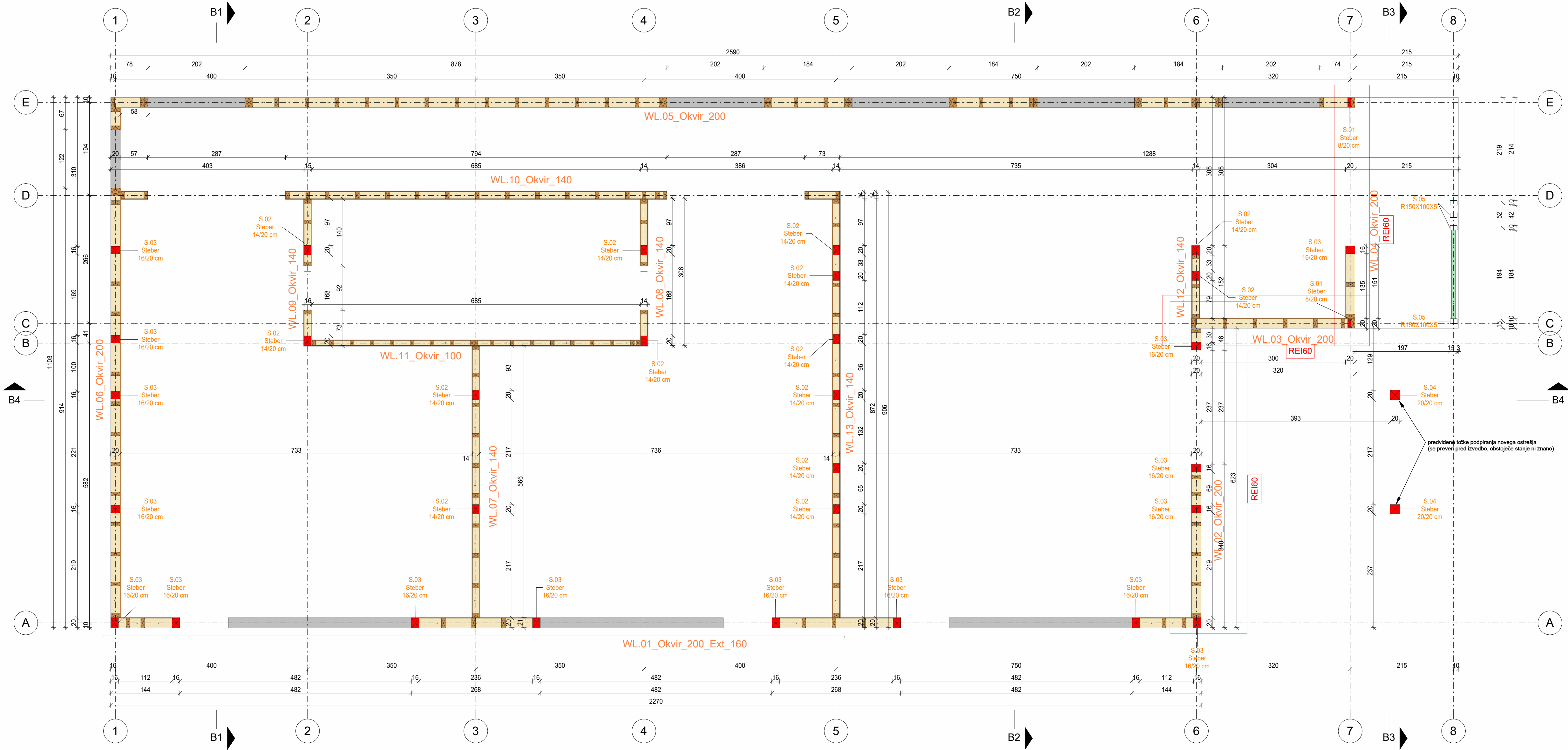
Št.risbe: 3☐

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	13	14 mm	1.45	4	5.80		00		1450						
Kommentar: ±									1450						
	14	14 mm	6.00	34	204.00		00		6000						
Kommentar: ±									6000						
	15	14 mm	5.85	5	29.25		00		5850						
									5850						
	16	14 mm	5.98	5	29.90		00		5980						
									5980						
	17	14 mm	5.24	5	26.20		00		5240						
									5240						
	18	14 mm	2.88	4	11.52		00		2880						
Kommentar: ±									2880						
	19	14 mm	6.02	8	48.16		00		6020						
Kommentar: ±									6020						

Objekt: □

Št.risbe: 3□

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	20	14 mm	4.01	4	16.04		00		4010						
Kommentar: ±									4010						
	21	14 mm	2.10	4	8.40		00		2100						
Kommentar: ±									2100						



POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z ngorljivo toplotno/zvo čno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

SPLOŠNA NAVODILA ZA VIJAČNJE LESENE KONSTRUKCIJE

- Vogalni in T-spoji sten se vijajo s samovreznimi lesnimi vijaki premera $\varnothing 8$ na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovce se vijajo v steno ali lego s samovreznimi lesnim vijakom premera $\varnothing 8$ in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera $\varnothing 8$ na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijajo z vijaki premera $\varnothing 8$ na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklopih medsebojno vijajo z vijaki premera $\varnothing 8$ na največji razdalji e=250mm,
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera $\varnothing 8$ na največji razdalji e=300mm,

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

Statične ojačitve v lesenih stenah (_SO = Statična ojačitev)			
Številka	Gradnik	Vrsta ojačitve	Material (_SO = Statična ojačitev)
S.01	DT_Steber_Les C24_8x20_SO	Steber	Les-C24_SO
S.02	DT_Steber_Les C24_14x20_SO	Steber	Les-C24_SO
S.03	DT_Steber_Les C24_16x20_SO	Steber	Les-C24_SO
S.04	DT_Steber_Les GL24_20x20_SO	Steber	Les-C24_SO

Jekleni profili				
Številka	Tip profila	Vrsta ojačitve	Material	Teža/m
N.04	RB 20	Palica	Jeklo - S235	19,76 kg/m
S.05	R150X100X5	Steber	Jeklo - S235	73,20 kg/m
				92,96 kg/m

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. Samovoljno prilagajanje projekta ni dovoljeno

LEGENDA MATERIALOV - LES

- LESENI ELEMENTI V POGLEDU
- LESENI ELEMENTI V PREREZU
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

LEGENDA MATERIALOV - JEKLO

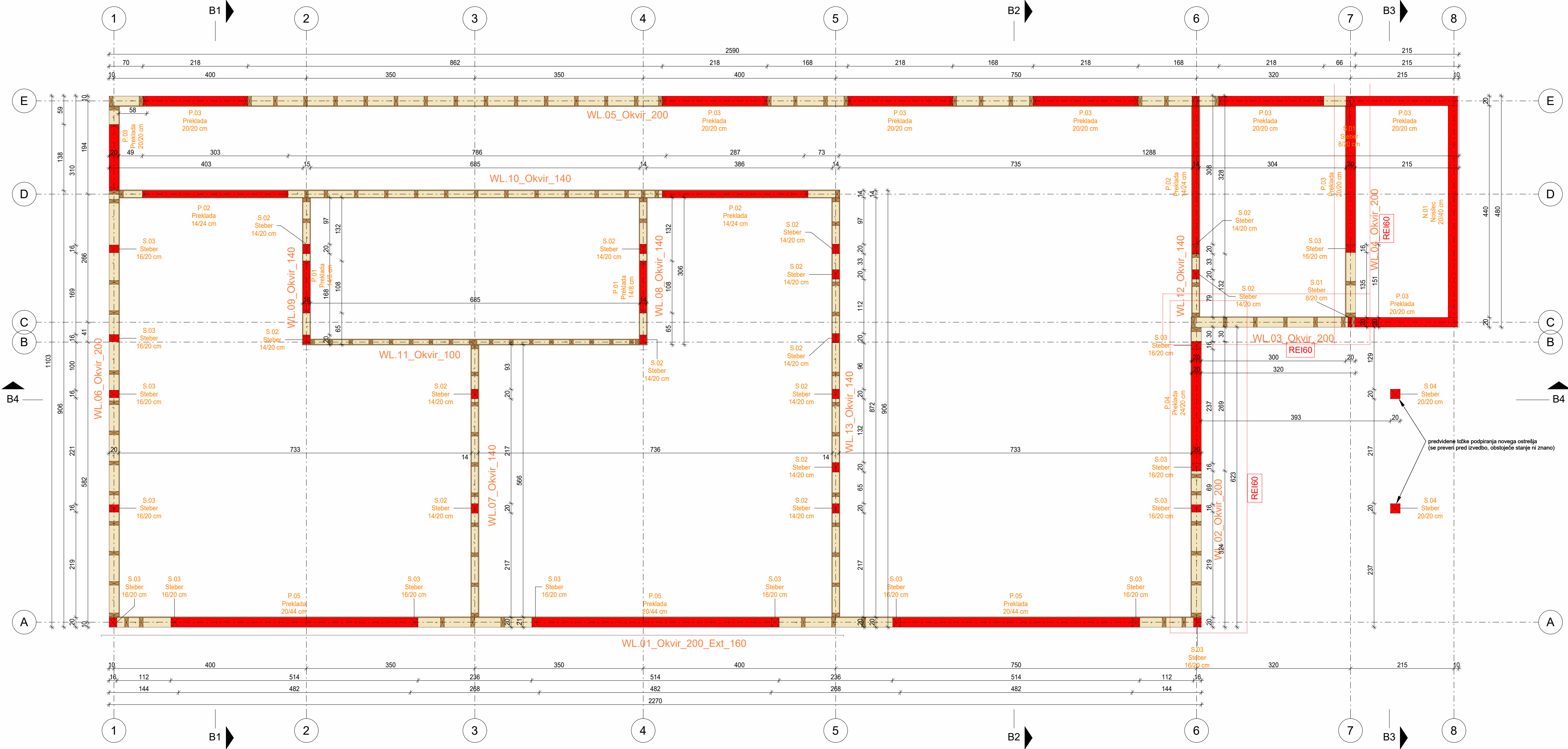
- JEKLO S235 - V POGLEDU
- JEKLENI ELEMENTI S235 - V PREREZU
- JEKLO S235 - SPOJI V POGLEDU IN PREREZU
- OZNAKA ZA DOLŽINSKI SPOJ

VRTEC - TLORIS PRITLIČJA - NOSILNA KONSTRUKCIJA

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št. načrta:	18/2026
Vrsta načrta:	NAČRTI S PODROČJA GRADBENIŠTVA	Št. projekta:	01/2025
Vsebinska risba			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:			
Datum:	Maj 2026	Merilo:	1:50
		List št.:	5



Statične ojačitve v lesenih stenah (_SO = Statična ojačitev)			
Številka	Gradnik	Vrsta ojačitve	Material (_SO = Statična ojačitev)
N.01	DT_Nosilec_Les GL24_20x40_SO	Nosilec	Les-GL24_SO
P.01	DT_Prekлада_Les C24_14x8_SO	Preklada	Les-C24_SO
P.02	DT_Prekлада_Les GL24_14x24_SO	Preklada	Les-GL24_SO
P.03	DT_Prekлада_Les GL24_20x20_SO	Preklada	Les-GL24_SO
P.04	DT_Prekлада_Les GL24_20x20_SO	Preklada	Les-GL24_SO
P.05	DT_Prekлада_Les GL24_20x44_SO	Preklada	Les-GL24_SO
S.01	DT_Steber_Les C24_8x20_SO	Steber	Les-C24_SO
S.02	DT_Steber_Les C24_14x20_SO	Steber	Les-C24_SO
S.03	DT_Steber_Les C24_16x20_SO	Steber	Les-C24_SO
S.04	DT_Steber_Les GL24_20x20_SO	Steber	Les-C24_SO

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

- LEGENDA MATERIALOV - LES**
- LESENI ELEMENTI V POGLEDU
 - LESENI ELEMENTI V PREREZU
 - LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
 - LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

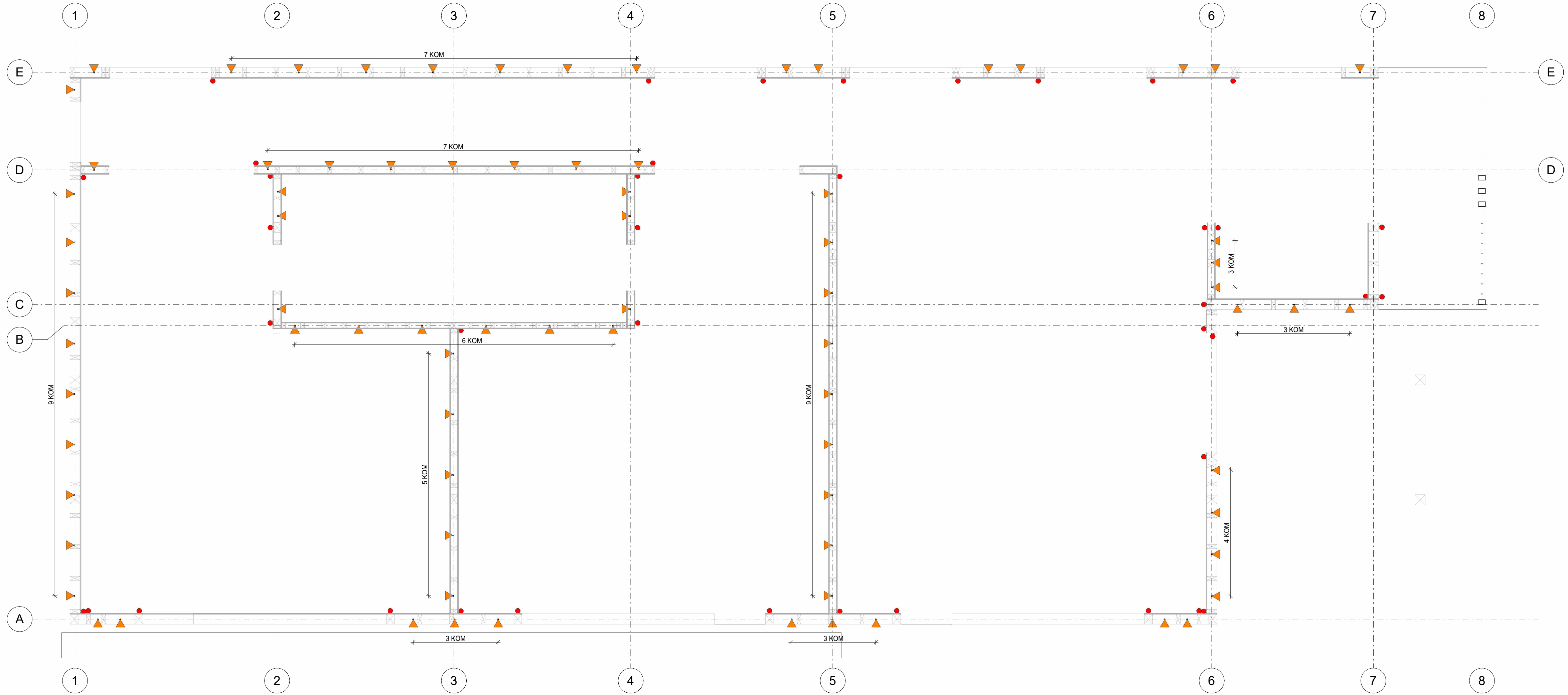
- LEGENDA MATERIALOV - JEKLO**
- JEKLO S235 - V POGLEDU
 - JEKLENI ELEMENTI S235 - V PREREZU
 - JEKLO S235 - SPOJI V POGLEDU IN PREREZU
 - OZNAKA ZA DOLŽINSKI SPOJ

VRTEC - TLOVIS PRITLIČJA - NOSILNA KONSTRUKCIJA - PREKLADE

Sprememba: Datum spremembe:
Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA	
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA	
Lokacija:		
Vrsta projekta:	PZI - projekt za izvedbo	Št. načrta: 18/2026
Vrsta načrta:	NAČRTI S PODROČJA GRADBENIŠTVJA	Št. projekta: 01/2025
Vsebina risbe		
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA	
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801	
Projektanti:		
Datum:	Maj 2026	Merilo: 1:50 List št.: 6



SIDRANJE - opisi in simboli

DVIŽNI KOTNIK
DVIŽNO SIDRANJE
Kotnik: WKR 53035
V steno: 16x žebelj LBA 4,0 x 60
Sidranje v AB ploščo: 1x navojna palica - 5.8 - M12x195 + kemična masa HYB-FIX

SIDRNI VIJAK M16x240
STRIŽNO SIDRANJE
SIDRNI VIJAKI SKOZI PASNICO V AB ZIDEC
Mehansko sidro M16 x 240
e_max = 1000 mm

PERFORIRAN TRAK
DVIŽNO SIDRANJE
Tip traku: 60 x 1,5 mm S350
Spoj zgoraj: 18 x žebelj LBA 4,0 X 60
Spoj spodaj: 18 x žebelj - LBA 4,0 X 60

2 DVOJNI DVIŽNI KOTNIK zgoraj/spodaj (stene)
DVIŽNO SIDRANJE
Kotnik: WKR 285
Spoj na steno zgoraj: 16 x žebelj LBA 4,0 x 60
Povezava prek medetažne plošče: svornik M12 (5.8)
Spoj na steno spodaj: 16 x žebelj LBA 4,0 x 60

2 x DVIŽNI KOTNIK (stebri, ki podpirajo lege)
2X WKR 285
žebliji 4x60
polno žebljanje
navzdol vijak VGS 11x125 + HUS10

LESNI VIJAK HBS 8x200
STRIŽNO SIDRANJE
LESNI VIJAKI SKOZI PASNICO V CLT PLOŠ ČO
HBS 8x200
e_max = 300 mm

LESNI VIJAK HBS 8x32C (os E)
STRIŽNO SIDRANJE
LESNI VIJAKI SKOZI KAPNO LEGO V CLT PLOŠ ČO
HBS 8x320
e_max = 300 mm

NAVOJNE PALICE M16 (8.8) + PODLOŽKE DIN 1052
OBEŠANJE e=300 (LES); 500mm (JEKLO)
NAVOJNE PALICE M16 (8.8) IN PODLOŽKE DIN 1052 !

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.
- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

SPLOŠNA NAVODILA ZA VIJAJČENJE LESENE KONSTRUKCIJE

- Vogalni in T-spoj sten se vijajo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovec se vijajo v steno ali lego s samovreznim lesnim vijakom premera Ø8 in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijajo z vijaki premera Ø8 na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklonih medsebojno vijajo z vijaki premera Ø8 na največji razdalji e=250mm,
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera Ø8 na največji razdalji e=300mm,

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

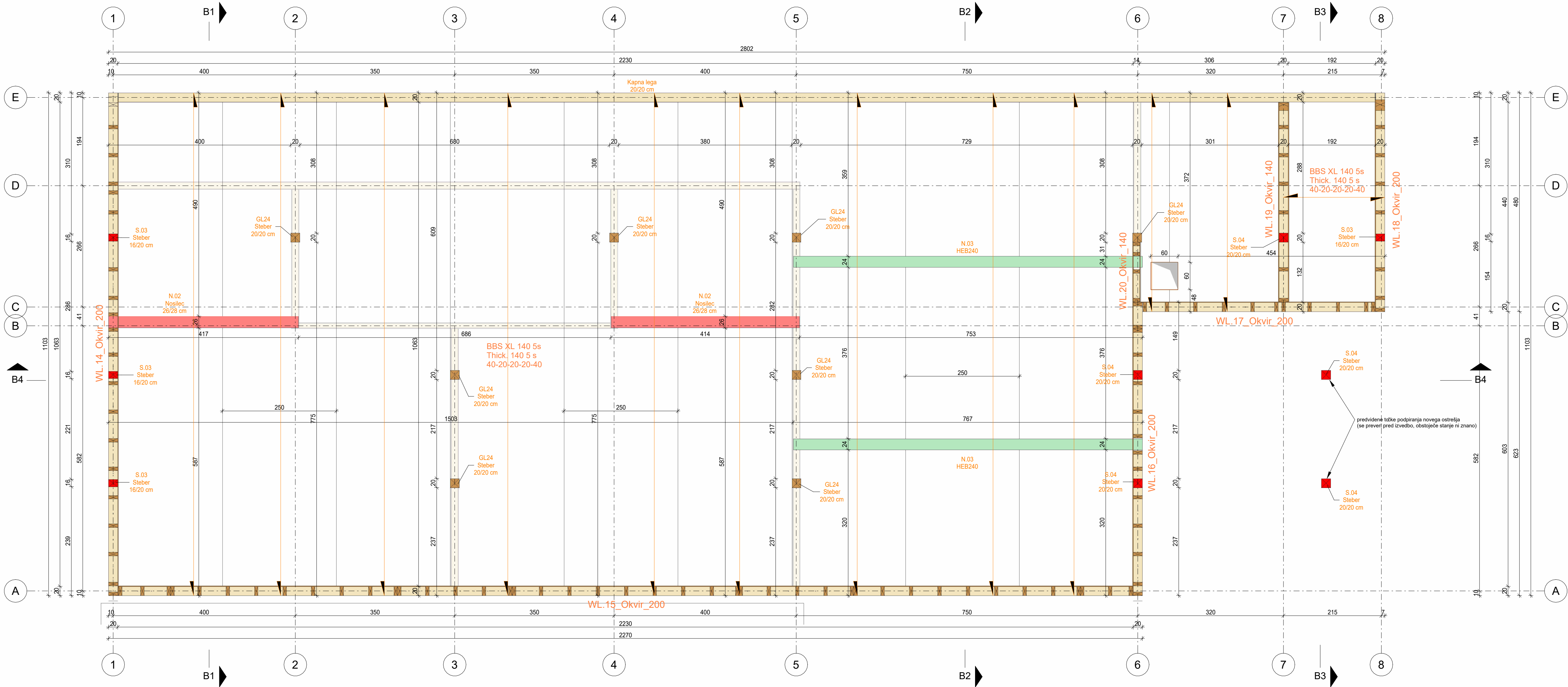
Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

VRTEC - TLORIS PRITLIČJA - SIDRANJE

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta:	18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta:	01/2025
Vsebina risbe			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.l.g., IZS G-3801		
Projektanti:			Merilo: 1:50
Datum:	Maj 2026	List št.:	7



Statične ojačitve v lesenih stenah (_SO = Statična ojačitev)			
Številka	Gradnik	Vrsta ojačitve	Material (_SO = Statična ojačitev)
N.01	DT_Nosilec_Les GL24_20x40_SO	Nosilec	Les-GL24_SO
N.02	DT_Nosilec_Les GL28_26x28_SO	Nosilec	Les-GL28_SO
P.01	DT_Prekлада_Les C24_14x8_SO	Preklada	Les-C24_SO
P.02	DT_Prekлада_Les GL24_14x24_SO	Preklada	Les-GL24_SO
P.03	DT_Prekлада_Les GL24_20x20_SO	Preklada	Les-GL24_SO
P.04	DT_Prekлада_Les GL24_20x24_SO	Preklada	Les-GL24_SO
P.05	DT_Prekлада_Les GL24_20x44_SO	Preklada	Les-GL24_SO
S.01	DT_Steber_Les C24_8x20_SO	Steber	Les-C24_SO
S.02	DT_Steber_Les C24_14x20_SO	Steber	Les-C24_SO
S.03	DT_Steber_Les C24_16x20_SO	Steber	Les-C24_SO
S.04	DT_Steber_Les GL24_20x20_SO	Steber	Les-C24_SO

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

LEGENDA MATERIALOV - LES

- LESENI ELEMENTI V POGLEDU
- LESENI ELEMENTI V PREREZU
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

LEGENDA MATERIALOV - JEKLO

- JEKLO S235 - V POGLEDU
- JEKLENI ELEMENTI S235 - V PREREZU
- JEKLO S235 - SPOJI V POGLEDU IN PREREZU
- OZNAKA ZA DOLŽINSKI SPOJ

VRTEC - TLORIS PODSTREŠJA - NOSILNA KONSTRUKCIJA

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št. načrta:	18/2026
Vrsta načrta:	NACRT S PODROČJA GRADBENIŠTVA	Št. projekta:	01/2025
Vsebina risbe			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:			
Datum:	Maj 2026	Merilo:	1:50
		List št.:	8

POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z ngorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

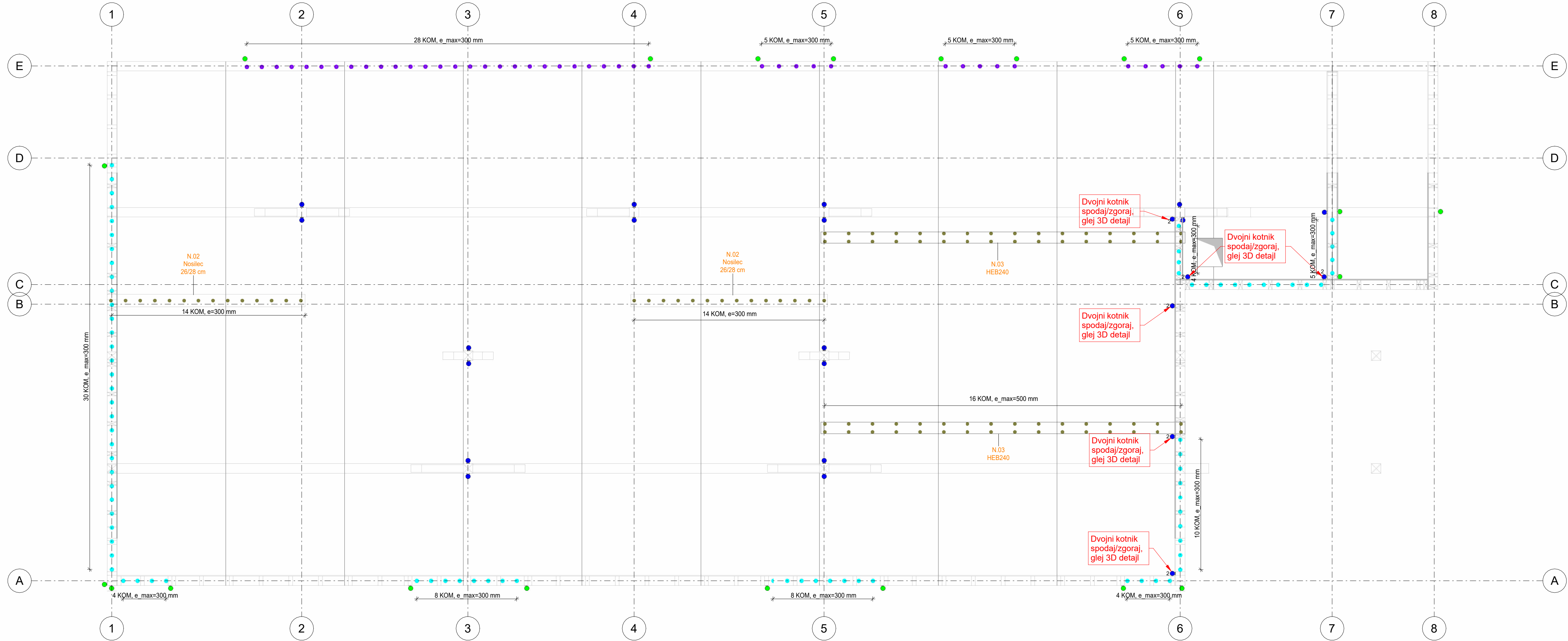
SPLOŠNA NAVODILA ZA VIJAČNJE LESENE KONSTRUKCIJE

- Vogalni in T-spoji sten se vijajo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovce se vijajo v steno ali lego s samovreznimi lesnim vijakom premera Ø8 in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijajo z vijaki premera Ø8 na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklolah medsebojno vijajo z vijaki premera Ø8 na največji razdalji e=250mm,
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera Ø8 na največji razdalji e=300mm,

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

Jekleni profili				
Številka	Tip profila	Vrsta ojačitve	Material	Teža/m
N.03	HEB240	Nosilec	Jeklo - S235	166,39 kg/m
S.05	R150X100X5	Steber	Jeklo - S235	18,30 kg/m
				184,69 kg/m



SIDRANJE - opisi in simboli

DVIŽNI KOTNIK
DVIŽNO SIDRANJE
Kotnik: WKR 53035
V steno: 16x žebelj LBA 4,0 x 60
Sidranje v AB plošč: 1x navojna palica - 5.8 - M12x195 + kemična masa HYB-FIX

SIDRNI VIJAK M16x240
STRIŽNO SIDRANJE
SIDRNI VIJAKI SKOZI PASNICO V AB ZIDEC
Mehansko sidro M16 x 240
e_max = 1000 mm

PERFORIRAN TRAK
DVIŽNO SIDRANJE
Tip traku: 60 x 1,5 mm S350
Spoj zgoraj: 18 x žebelj LBA 4,0 X 60
Spoj spodaj: 18 x žebelj - LBA 4,0 X 60

2 DVOJNI DVIŽNI KOTNIK zgoraj/spodaj (stene)
DVIŽNO SIDRANJE
Kotnik: WKR 285
Spoj na steno zgoraj: 16 x žebelj LBA 4,0 x 60
Povezava prek medetažne plošč: svornik M12 (5.8)
Spoj na steno spodaj: 16 x žebelj LBA 4,0 x 60

2 x DVIŽNI KOTNIK (stebri, ki podpirajo lege)
2X WKR 285
žebelji 4x60
polno žebljanje
navzdol vijak VGS 11x125 + HUS10

LESNI VIJAK HBS 8x200
STRIŽNO SIDRANJE
LESNI VIJAKI SKOZI PASNICO V CLT PLOŠ ČO
HBS 8x200
e_max = 300 mm

LESNI VIJAK HBS 8x32C (os E)
STRIŽNO SIDRANJE
LESNI VIJAKI SKOZI KAPNO LEGO V CLT PLOŠ ČO
HBS 8x320
e_max = 300 mm

NAVOJNE PALICE M16 (8.8) + PODLOŽKE DIN 1052
OBEŠANJE e=300 (LES); 500mm (JEKLO)
NAVOJNE PALICE M16 (8.8) IN PODLOŽKE DIN 1052 !

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

SPLOŠNA NAVODILA ZA VIJAČENJE LESENE KONSTRUKCIJE

- Vogalni in T-spoj stene se vijajo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm.
- vsak špirovce se vijajo v steno ali lego s samovreznimi lesnimi vijakom premera Ø8 in s povečano glavo ali podložko.
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm.
- medetažne križno lepljene plošče se v stene vijajo z vijaki premera Ø8 na največji razdalji e=250mm.
- medetažne križno lepljene plošče se na L-preklonih medsebojno vijajo z vijaki premera Ø8 na največji razdalji e=250mm.
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera Ø8 na največji razdalji e=300mm.

Dožina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

VRTEC - TLORIS PODSTREŠJA - SIDRANJE

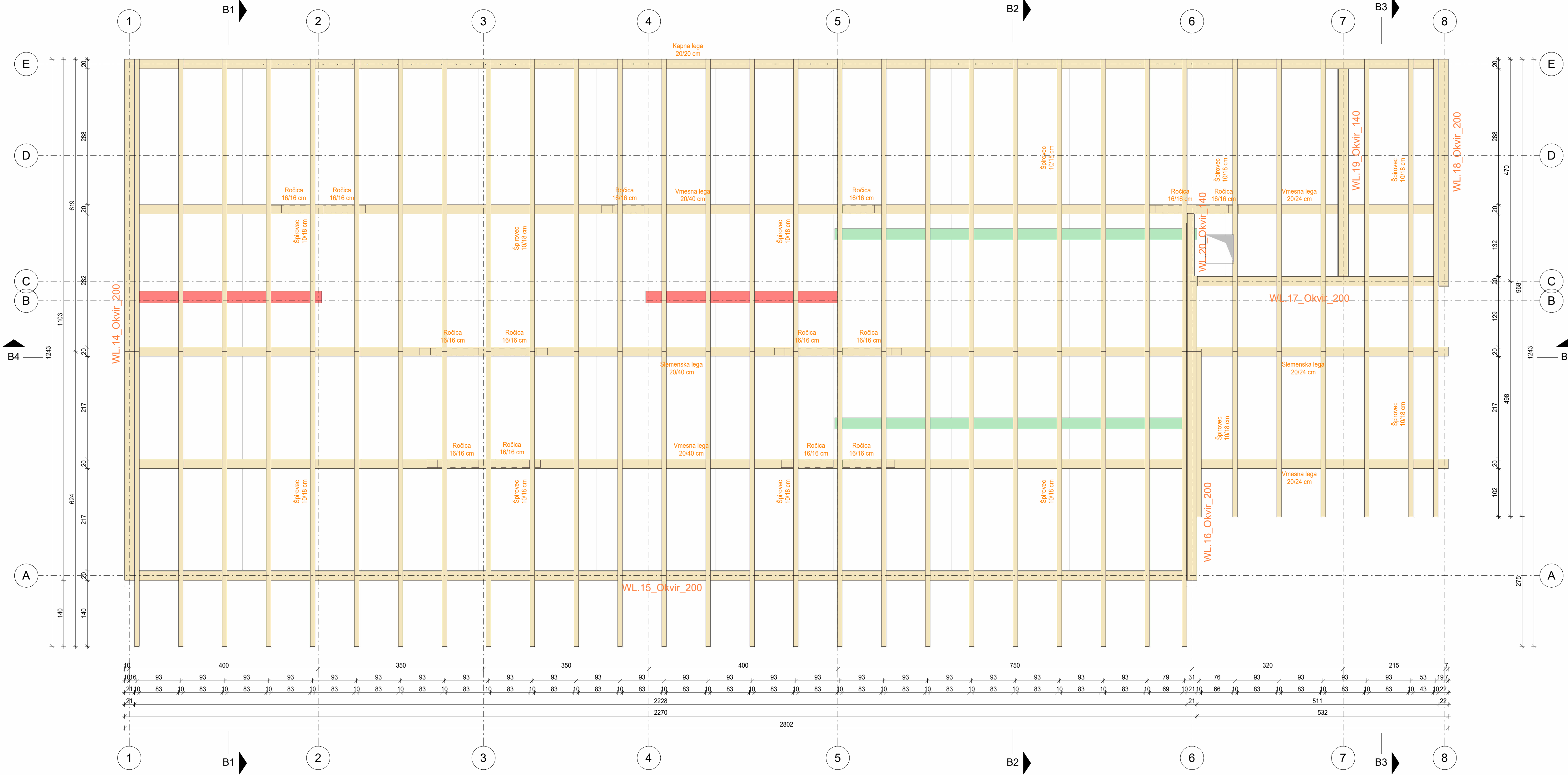
Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA

Lokacija:		
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta: 18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta: 01/2025

Vsebina risbe		
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA	
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801	
Projektanti:		Merilo: 1:50
Datum:	Maj 2026	List št.: 9



Elementi ostrešja		
Gradnik	Vrsta ojačitve	Material
DT_Kapna lega_Les C24_20x20	Kapna lega	Les-C24
DT_Ročica_Les C24_16x16	Ročica	Les-C24
DT_Slemenska lega_Les GL24_20x24	Slemenska lega	Les-GL24
DT_Slemenska lega_Les GL28_20x40	Slemenska lega	Les-GL28
DT_Steber_Les GL24_20x20	Steber	Les-GL24
DT_Vmesna lega_Les GL24_20x24	Vmesna lega	Les-GL24
DT_Vmesna lega_Les GL28_20x40	Vmesna lega	Les-GL28
DT_Špirovec_Les C24_10x18	Špirovec	Les-C24

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

- LEGENDA MATERIALOV - LES**
- LESENI ELEMENTI V POGLEDU
 - LESENI ELEMENTI V PREREZU
 - LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
 - LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

- LEGENDA MATERIALOV - JEKLO**
- JEKLO S235 - V POGLEDU
 - JEKLENI ELEMENTI S235 - V PREREZU
 - JEKLO S235 - SPOJI V POGLEDU IN PREREZU
 - OZNAKA ZA DOLŽINSKI SPOJ

VRTEC - TLORIS OSTREŠJA - NOSILNA KONSTRUKCIJA

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA	
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA	
Lokacija:		
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta: 18/2026
Vrsta načrta:	NACRT S PODROČJA GRADBENIŠTVIA	Št.projekta: 01/2025
Vsebinska risba		
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA	
Poblaščen inženir:	Mina Oražem, u.d.i.g., IZS G-3801	
Projektanti:		Merilo: 1:50
Datum:	Maj 2026	List št.: 10

POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.
- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.
- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

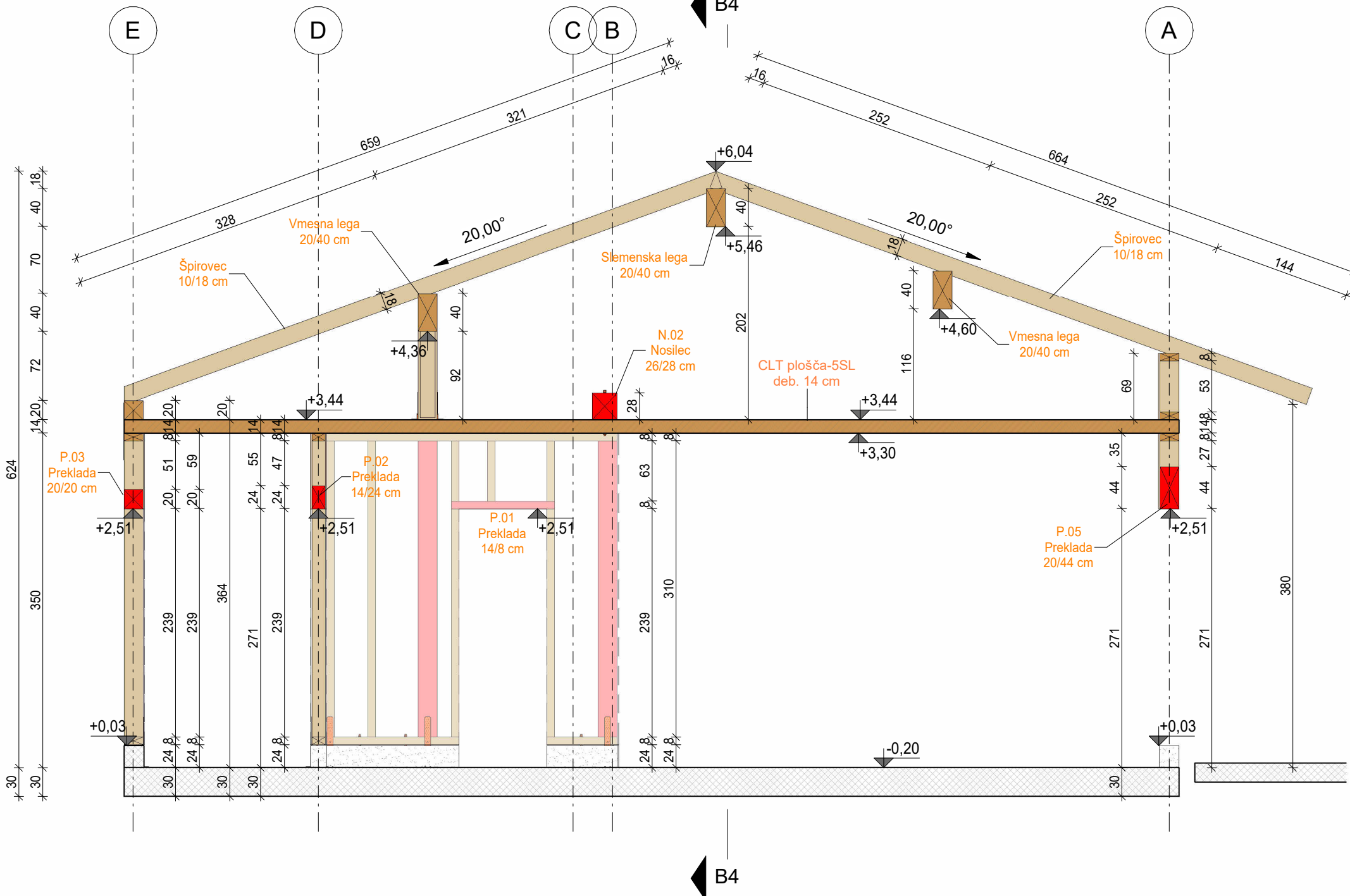
SPLOŠNA NAVODILA ZA VIJAČENJE LESENE KONSTRUKCIJE

- Vogalni in T-spoli sten se vijačijo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovec se vijači v steno ali lego s samovreznim lesnim vijakom premera Ø8 in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijači v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijačijo z vijaki premera Ø8 na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklopih medsebojno vijačijo z vijaki premera Ø8 na največji razdalji e=250mm,
- stebri in ojačitve ter vsi konstrukcijski elem. se vijačijo z vijaki premera Ø8 na največji razdalji e=300mm,

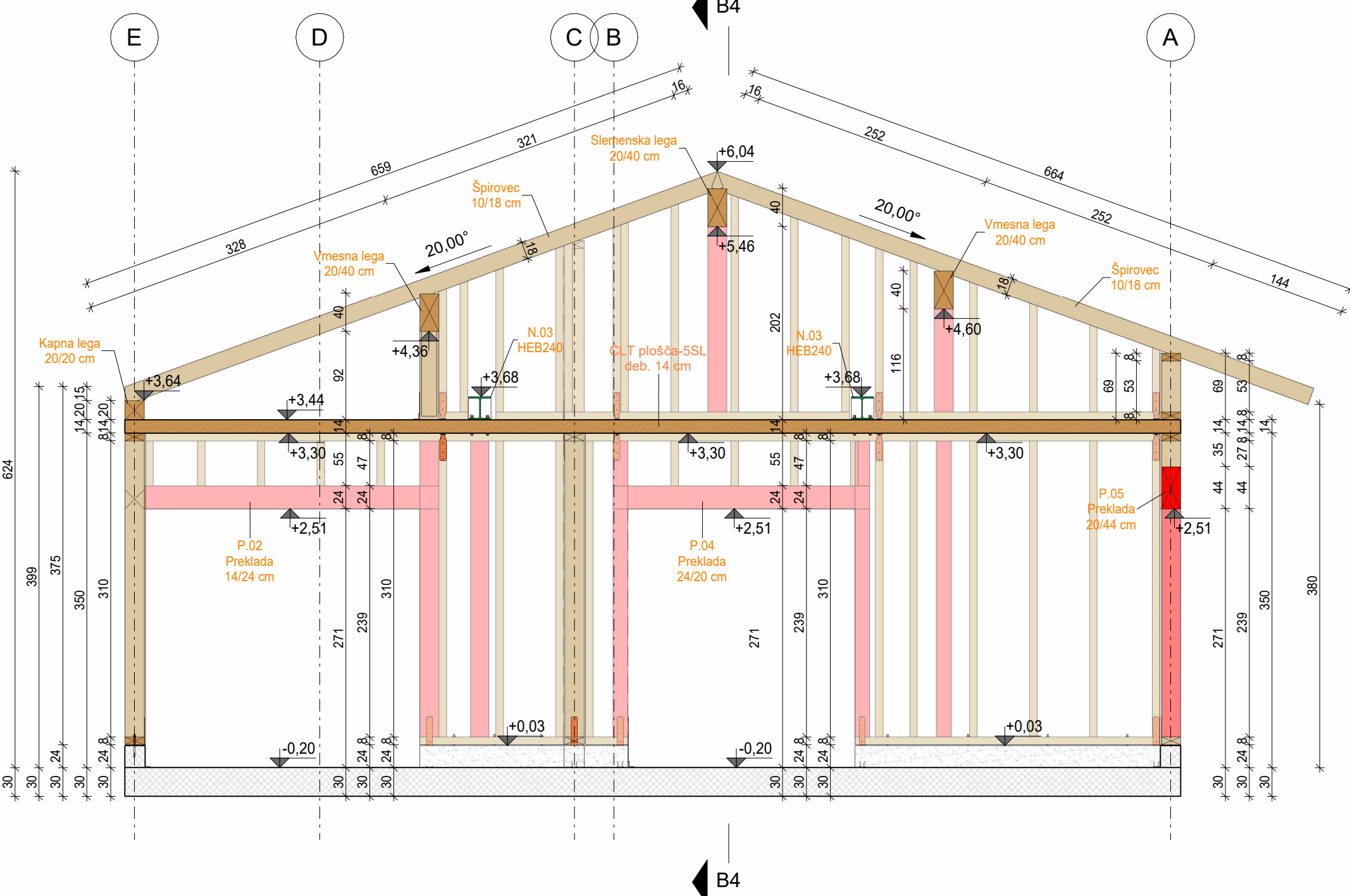
Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

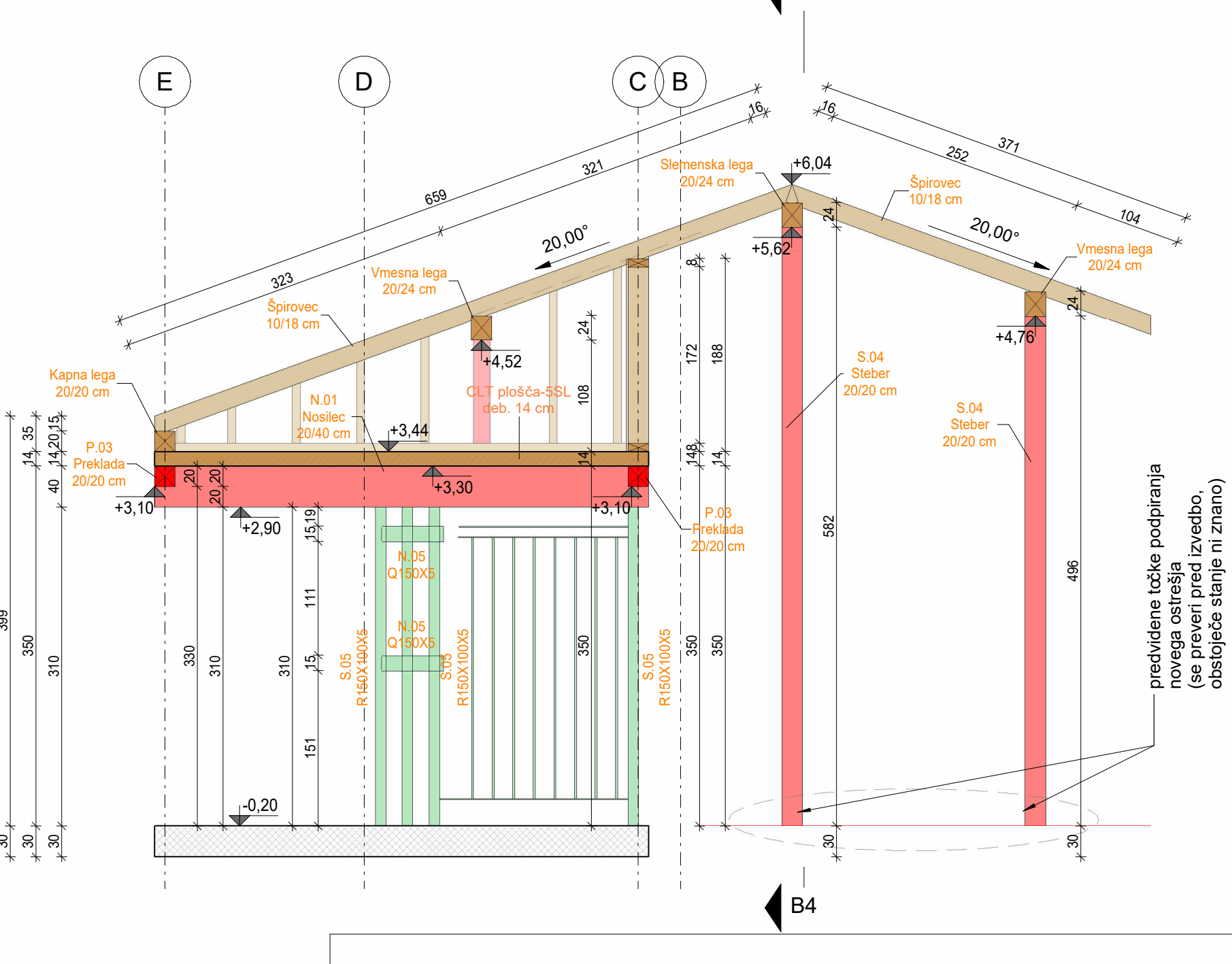
PREČNI PREREZ B1
M 1 : 50



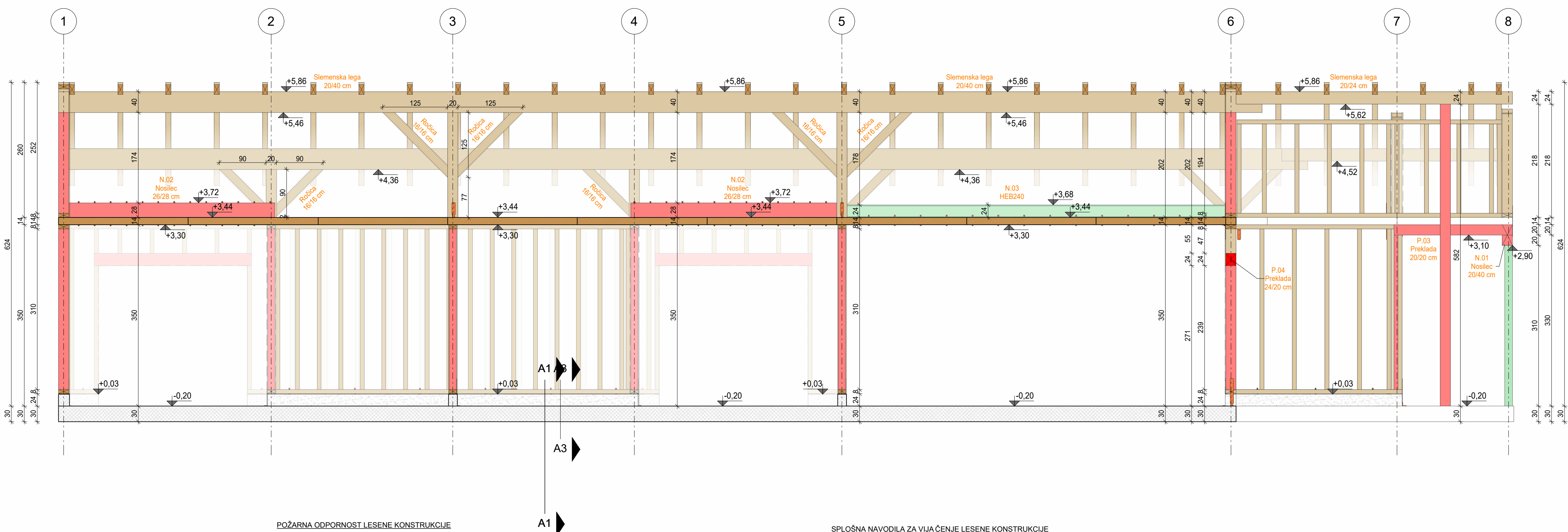
PREČNI PREREZ B2
M 1 : 50



PREČNI PREREZ B3
M 1 : 50



VZDOLŽNI PREREZ B4
M 1 : 50



POŽARNA ODORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapojljene z negorljivo toplotnoizolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.
- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.
- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

SPOŠNA NAVODILA ZA VIJAČENJE LESENE KONSTRUKCIJE

- Vogalni in T-spoji sten se vijačijo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovce se vijači v steno ali lego s samovreznim lesnim vijakom premera Ø8 in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijači v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijačijo z vijaki premera Ø8 na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklopih medsebojno vijačijo z vijaki premera Ø8 na največji razdalji e=250mm,
- stebi in ojačitve ter vsi konstrukcijski elem. se vijačijo z vijaki premera Ø8 na največji razdalji e=300mm,
- Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. Samovoljno prilagajanje projekta ni dovoljeno

LEGENDA MATERIALOV - LES

- LESENI ELEMENTI V POGLEDU
- LESENI ELEMENTI V PREREZU
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

LEGENDA MATERIALOV - JEKLO

- JEKLO S235 - V POGLEDU
- JEKLENI ELEMENTI S235 - V PREREZU
- JEKLO S235 - SPOJI V POGLEDU IN PREREZU
- OZNAKA ZA DOLŽINSKI SPOJ

VRTEC - PREREZI

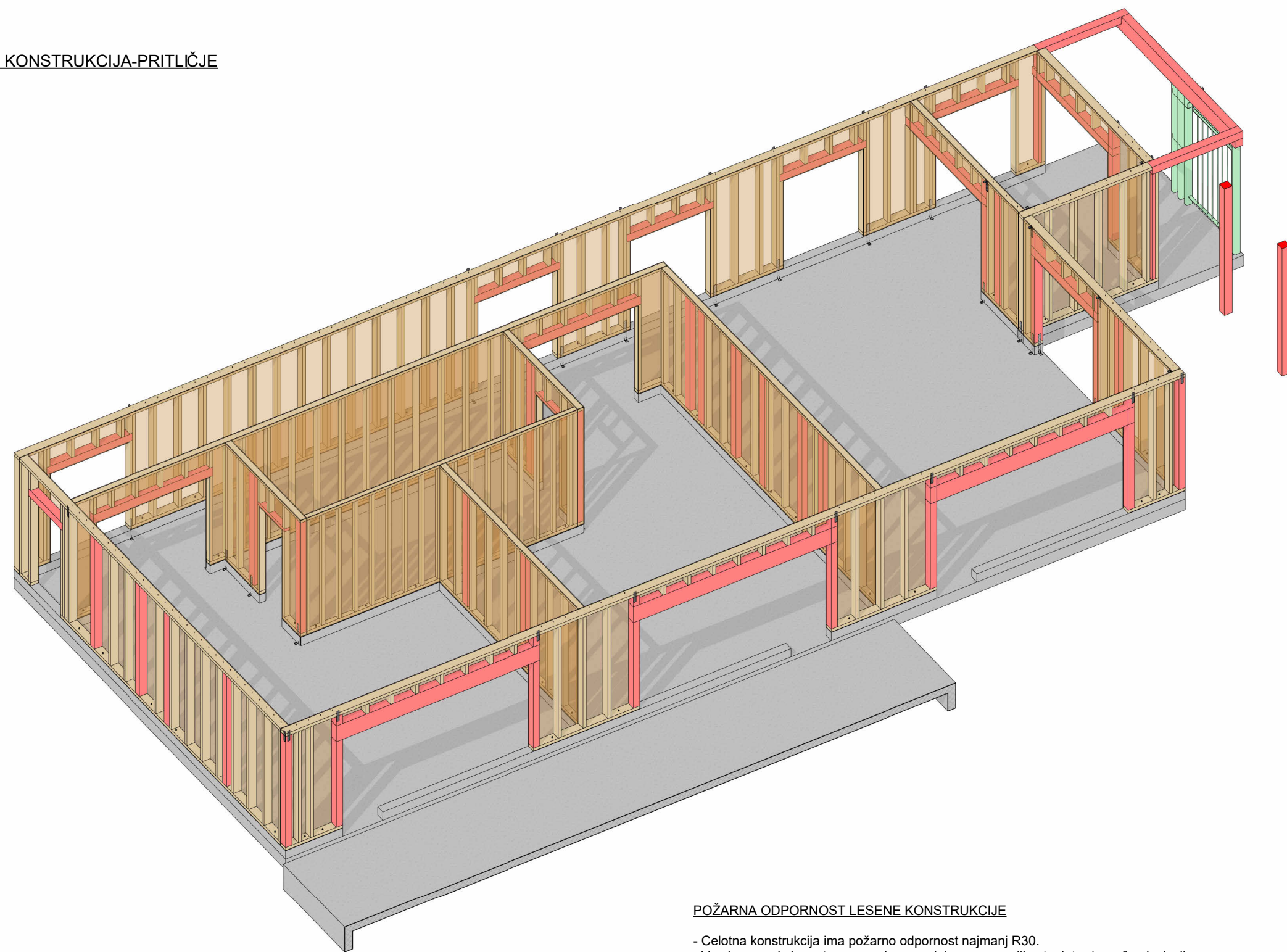
Sprememba: Datum spremembe: Podpis:



Investitor: OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA
Objekt: OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA

Lokacija: Vrsta projekta: PZI - projekt za izvedbo Vrsta načrta: NAČRT S PODROČJA GRADBENIŠTVA Št.č.načrta: 18/2026 Št.projekta: 01/2025
Vsebinska risba: Vodja projekta: Jure Kotnik, mag.inž.arh, ZAPS 2309 PA Pooblaščen inženir: Miha Oražem, u.d.i.g., IZS G-3801 Projektant: Merilo: 1:50 Datum: Maj 2026 List št.: 11

3D KONSTRUKCIJA-PRITLČJE



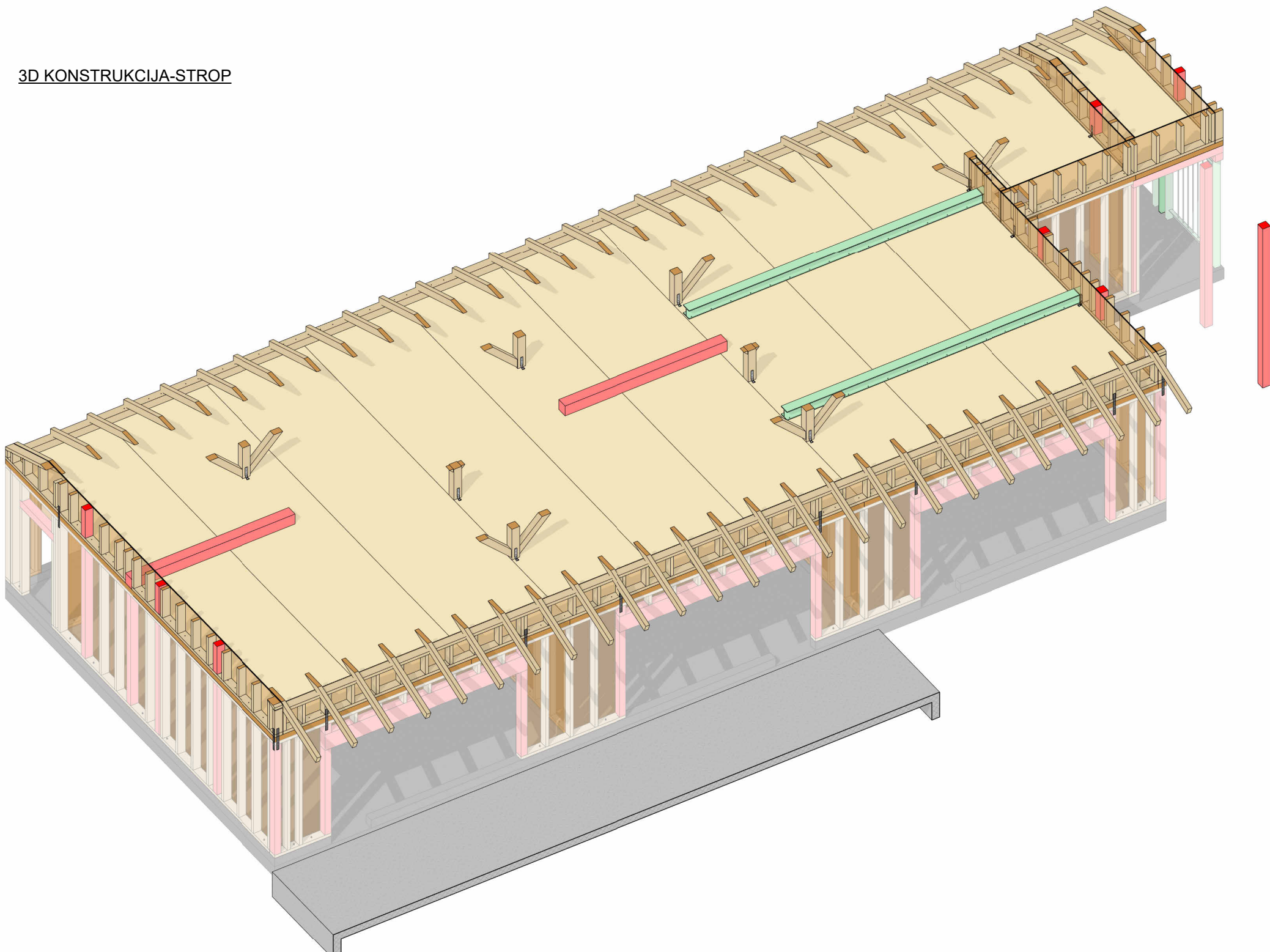
POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojstransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

3D KONSTRUKCIJA-STROP



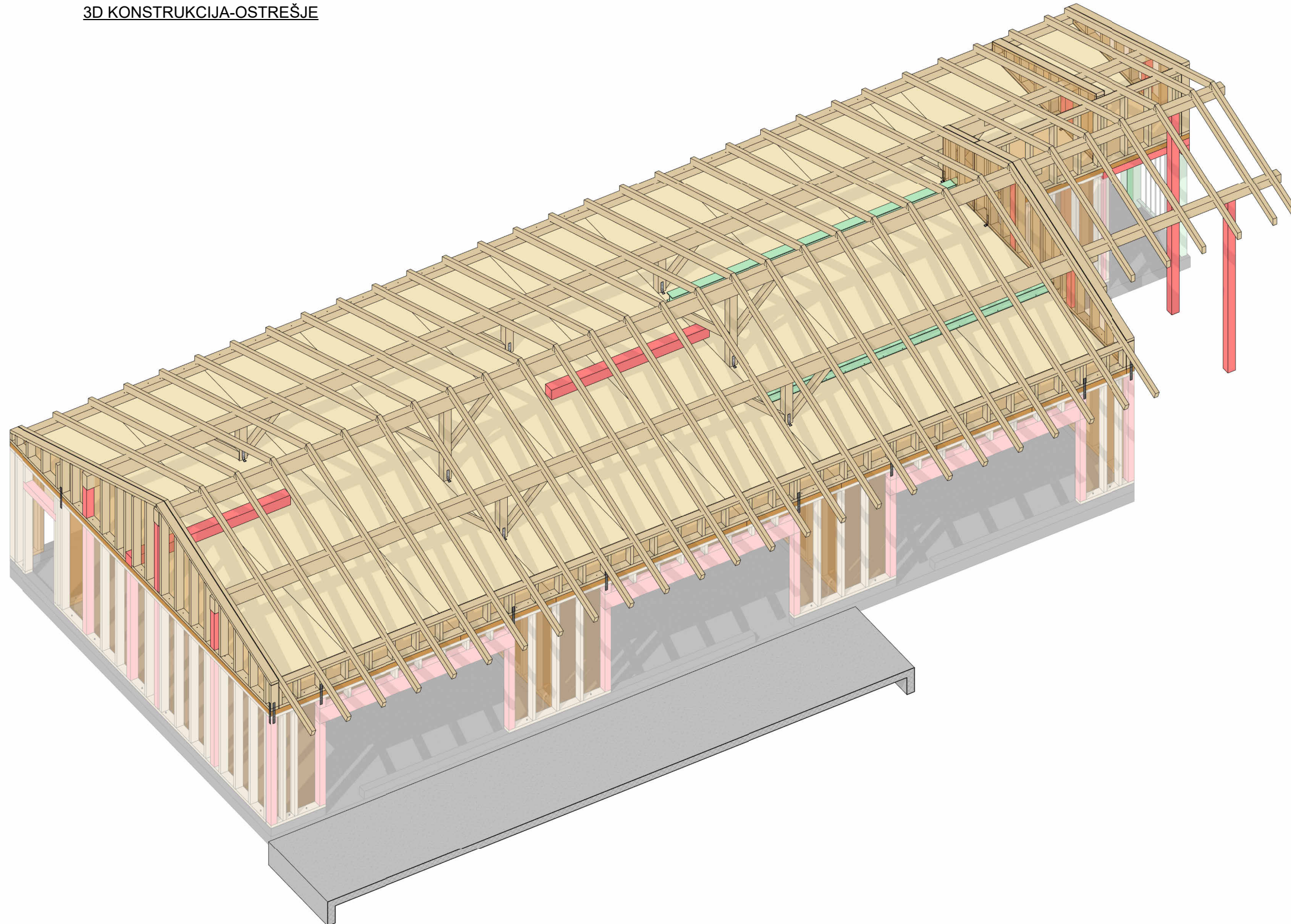
SPLOŠNA NAVODILA ZA VIJAČENJE LESENE KONSTRUKCIJE

- Vogalni in T-spoj stene se vijajo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm.
- vsak špirovec se vijajo v steno ali lego s samovreznim lesnim vijakom premera Ø8 in s povečano glavo ali podložko.
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm.
- medetažne križno lepljene plošče se v stene vijajo z vijaki premera Ø8 na največji razdalji e=250mm.
- medetažne križno lepljene plošče se na L-preklapih medsebojno vijajo z vijaki premera Ø8 na največji razdalji e=250mm.
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera Ø8 na največji razdalji e=300mm.

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

3D KONSTRUKCIJA-OSTREŠJE



OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. Samovoljno prilagajanje projekta ni dovoljeno

LEGENDA MATERIALOV - LES

- LESENI ELEMENTI V POGLEDU
- LESENI ELEMENTI V PREREZU
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

LEGENDA MATERIALOV - JEKLO

- JEKLO S235 - V POGLEDU
- JEKLENI ELEMENTI S235 - V PREREZU
- JEKLO S235 - SPOJI V POGLEDU IN PREREZU
- OZNAKA ZA DOLŽINSKI SPOJ

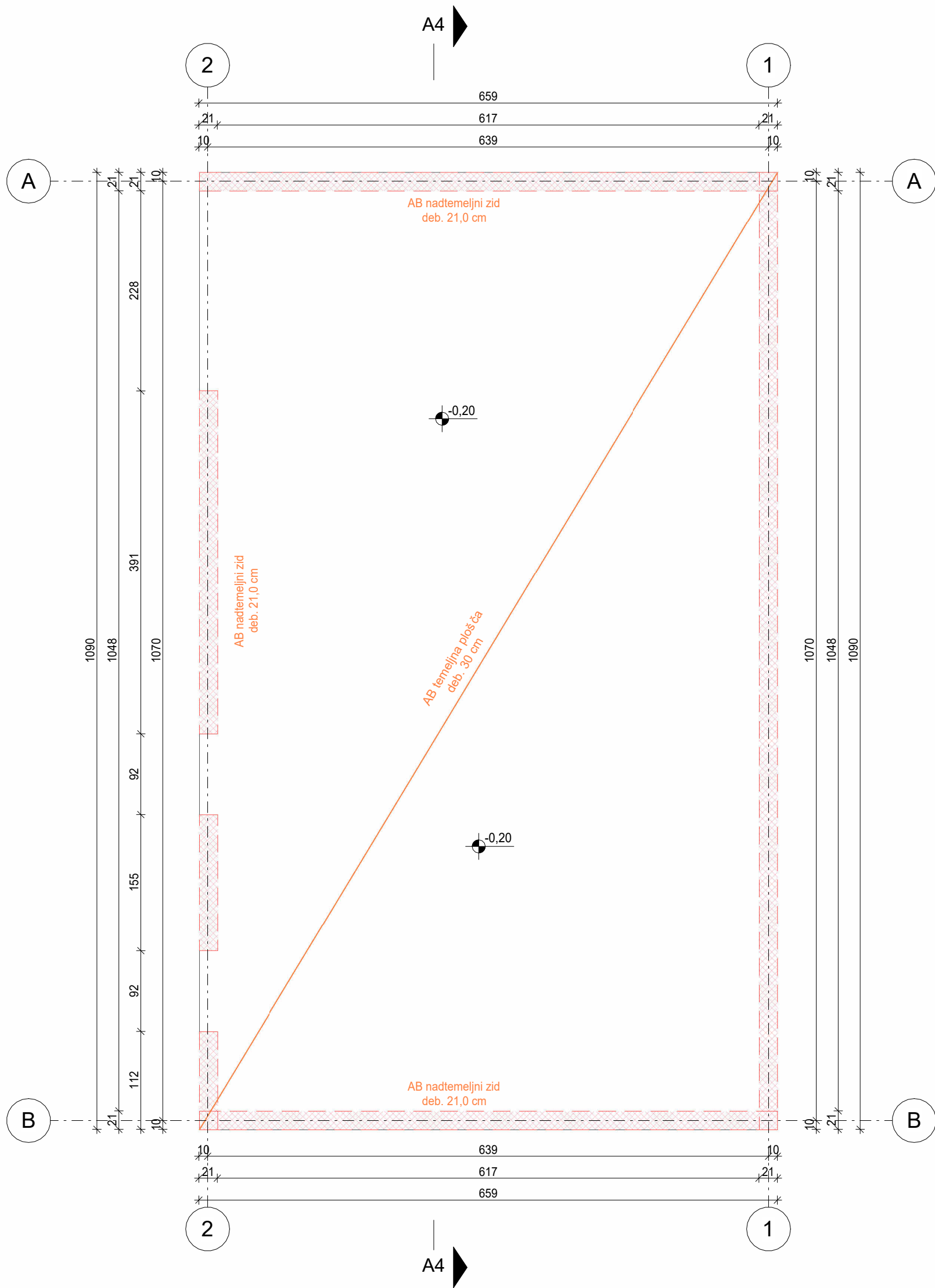
VRTEC - 3D POGLEDI - NOSILNA KONSTRUKCIJA

Sprememba:	Datum spremembe:
	Podpis:

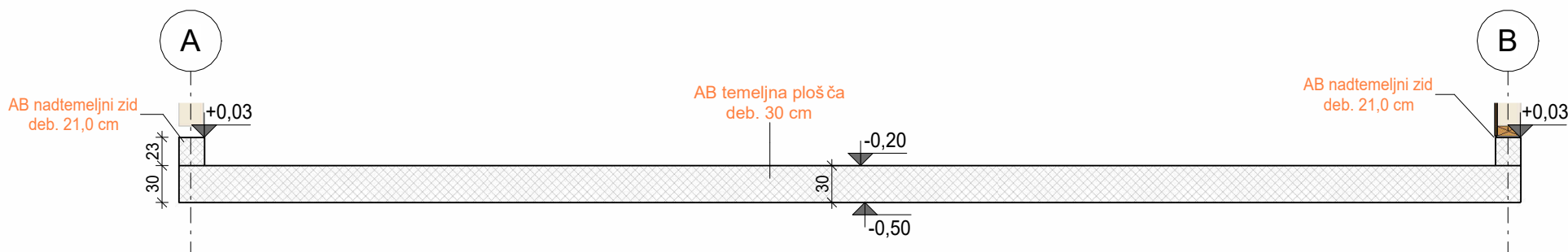


Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA	
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA	
Lokacija:		
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta: 18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta: 01/2025
Vsebina risbe:		
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA	
Pooblaščen inženir:	Miha Oražem, u.d.l.g., IZS G-3801	Merilo: 1:50
Projektanti:		
Datum:	Maj 2026	List št.: 12

TLORIS AB TEMELJNE PLOŠČE-OPAŽNI NAČRT
M 1 : 50



VZDOLŽNI PREREZ A1
M 1 : 50



OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

UPORABLJENI MATERIALI:

- BETON C25/30, XC2, CI 0.2, $d_{max} = 32$, S3
- ARMATURNO JEKLO ZA PALICE B500 B
- ARMATURNO JEKLO ZA MREŽE B500 A

KROVNI SLOJI BETONA c_{nom} :

- AB PLOŠČE: 3 cm

LEGENDA MATERIALOV

- ARMIRANI BETON
- AB STENE NAD PLOŠČO
- AB STENE IN NOSILCI POD PLOŠČO
- AB STEBRI, VV NAD PLOŠČO
- OPEČNE STENE NAD PLOŠČO

KUHINJA - OPAŽNI NAČRT TEMELJNE PLOŠČE

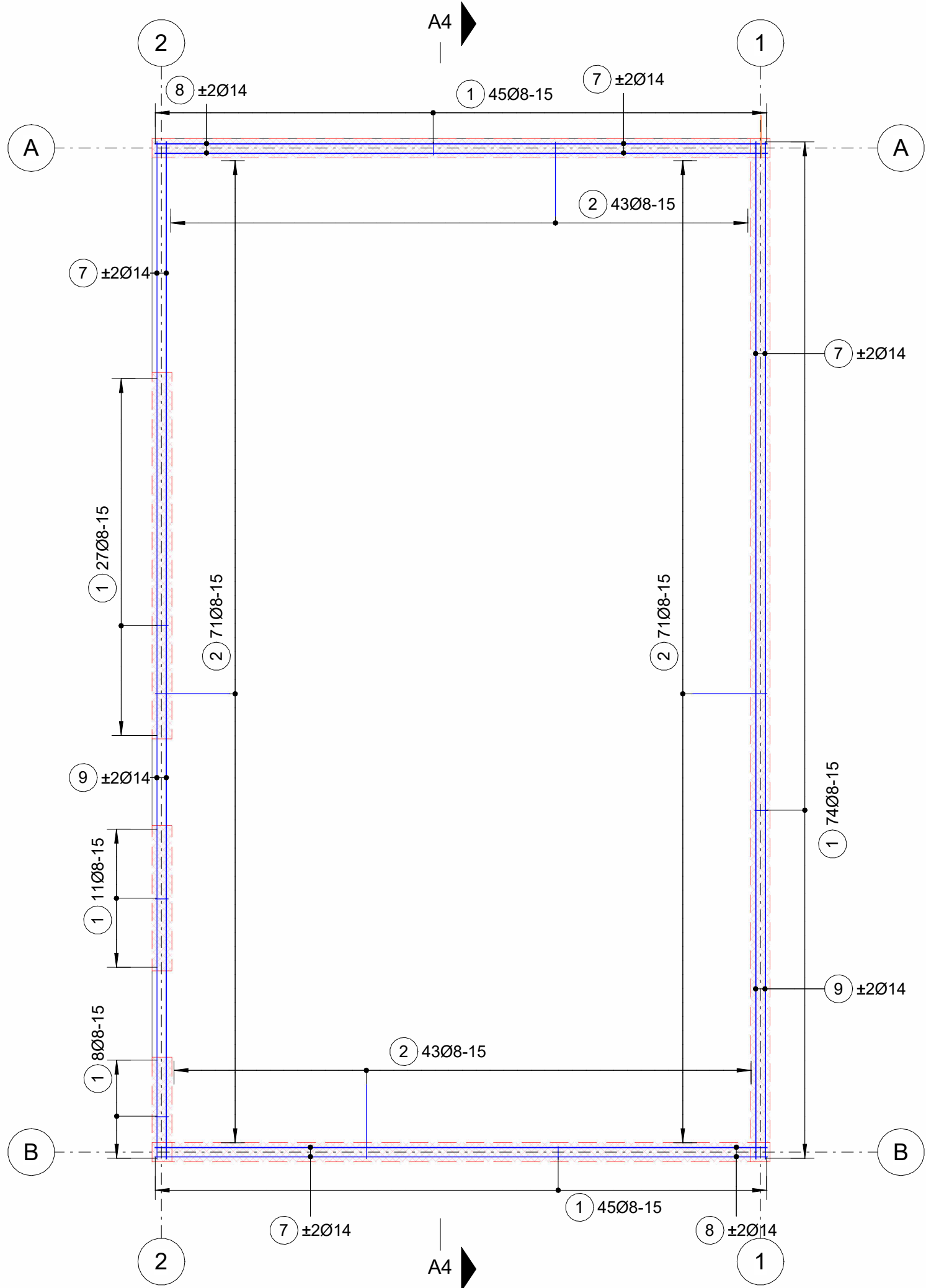
Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA	
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA	
Lokacija:		
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta: 18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta: 01/2025
Vsebina risbe:		
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA	
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801	
Projektanti:		
Datum:	Maj 2026	Merilo: 1:50 List št.: 13

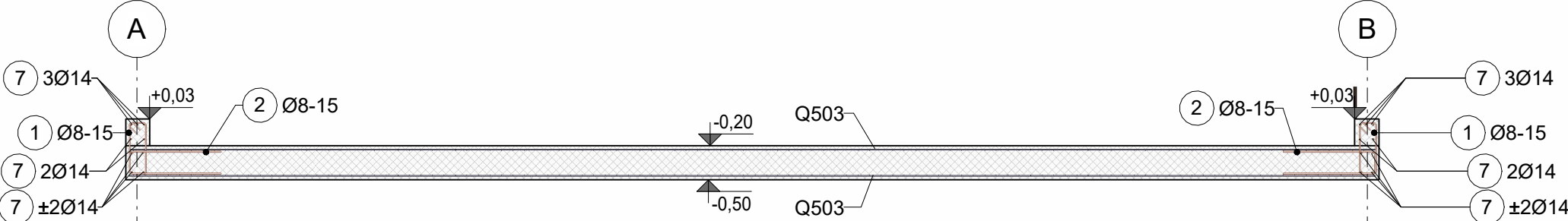
TLORIS AB TEMELJNE PLOŠČE-ARMATURNI NAČRT-PROSTI ROB

M 1 : 50



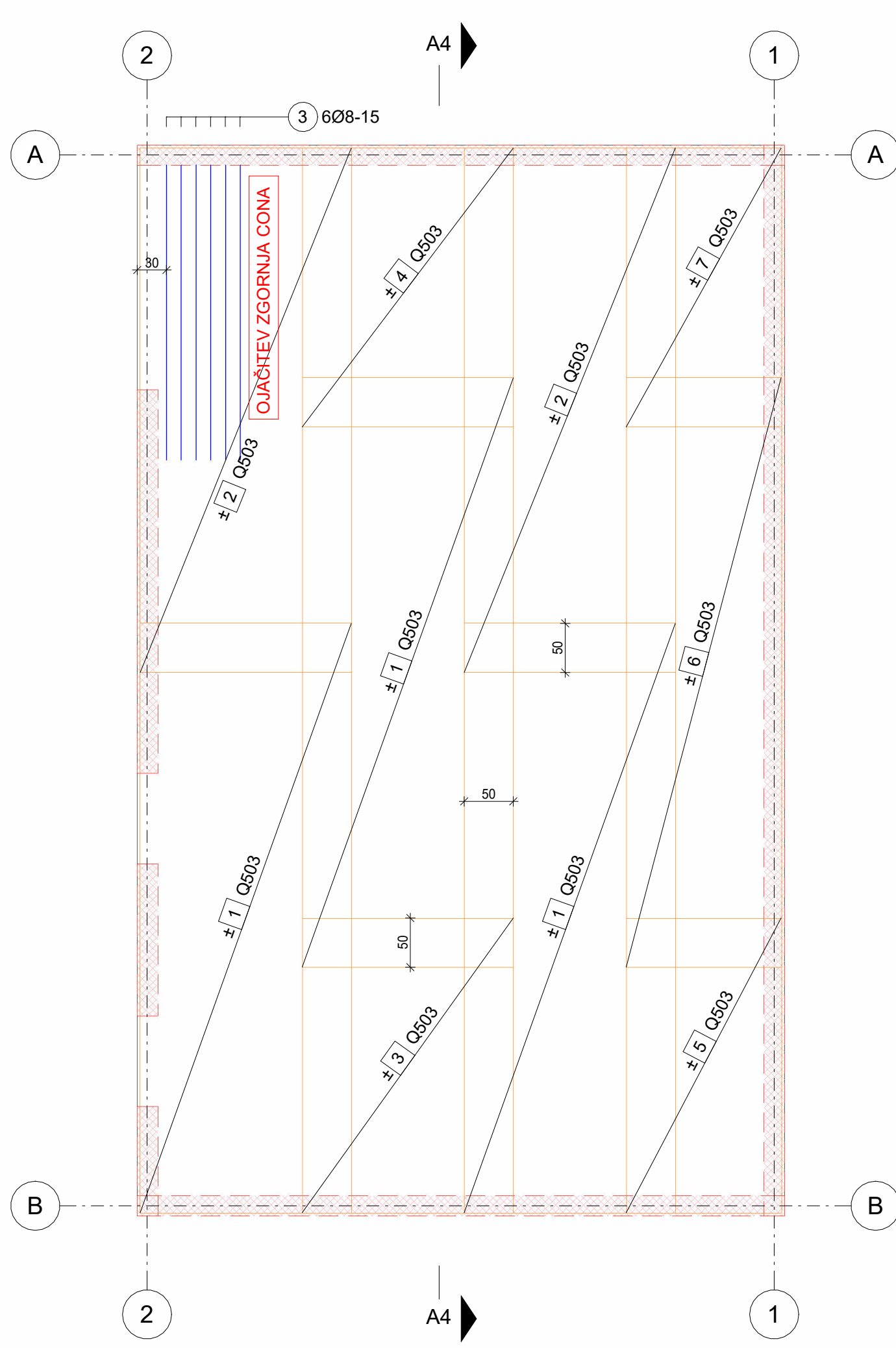
VZDOLŽNI PREREZ A1

M 1 : 50



TLORIS AB TEMELJNE PLOŠČE-ARMATURNI NAČRT-SPODNJA IN ZGORNJA CONA

M 1 : 50



1 Ø8 mm, L = 141 cm, 210 kom

5 Ø14 mm, L = 149 cm, 5 kom

9 Ø14 mm, L = 584 cm, 13 kom

2 Ø8 mm, L = 180 cm, 228 kom

6 Ø14 mm, L = 385 cm, 5 kom

3 Ø8 mm, L = 300 cm, 6 kom

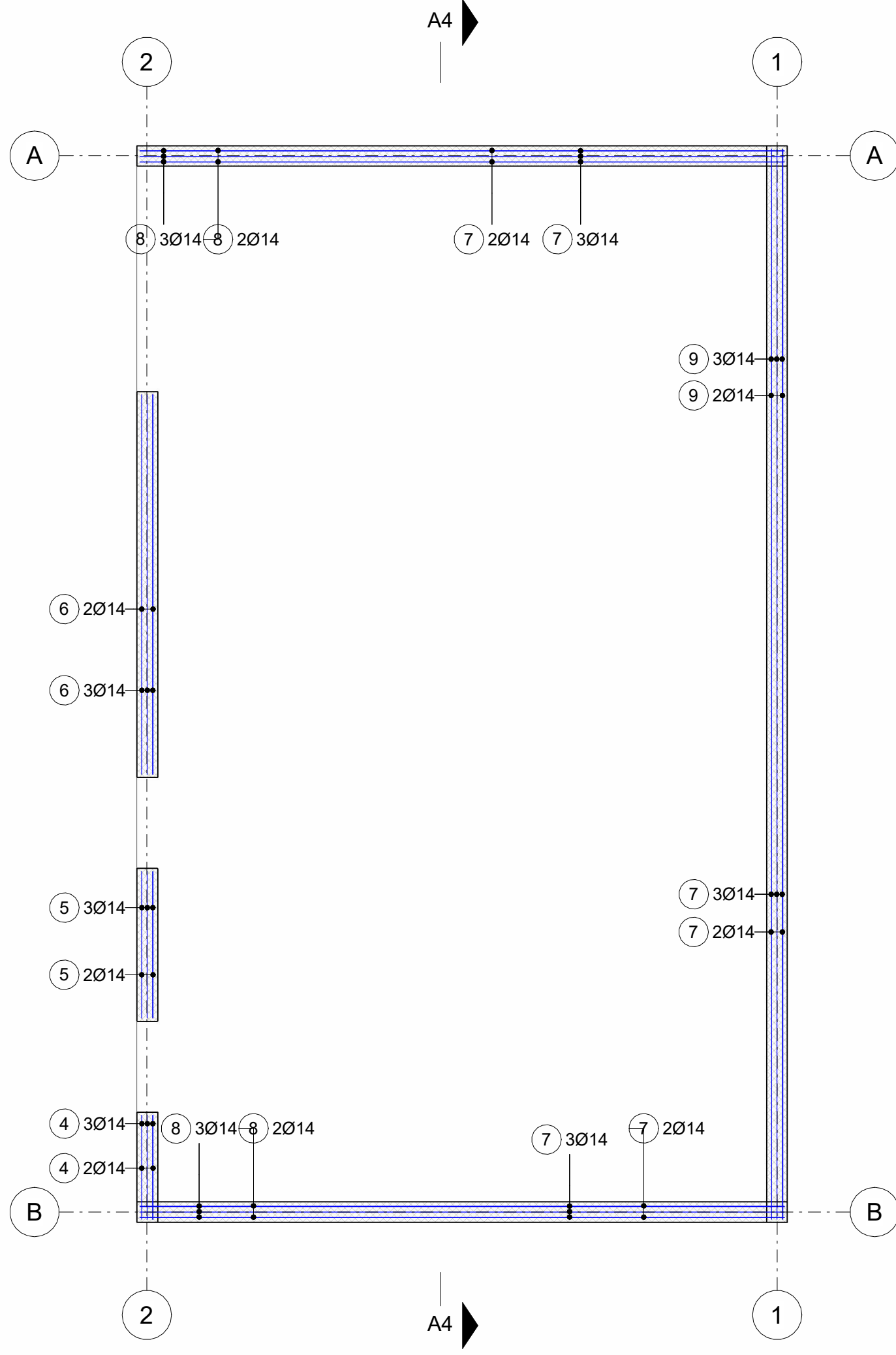
7 Ø14 mm, L = 600 cm, 31 kom

4 Ø14 mm, L = 106 cm, 5 kom

8 Ø14 mm, L = 153 cm, 18 kom

TLORIS AB TEMELJNE PLOŠČE-ARMATURNI NAČRT-NADTEMELJNI ZID

M 1 : 50



OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

UPORABLJENI MATERIALI:

- BETON C25/30, XC2, CI 0.2, $d_{max} = 32$, S3
- ARMATURNO JEKLO ZA PALIČE B500 B
- ARMATURNO JEKLO ZA MREŽE B500 A

KROVNI SLOJI BETONA c_{nom} :

- AB PLOŠČE: 3 cm

LEGENDA MATERIALOV

- ARMIRANI BETON
- AB STENE NAD PLOŠČO
- AB STENE IN NOSILCI POD PLOŠČO
- AB STEBRI, VV NAD PLOŠČO
- OPEČNE STENE NAD PLOŠČO

KUHINJA - ARMATURNI NAČRT TEM. PLOŠČE IN NADTEM. ZIDU

Sprememba: Datum spremembe: Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA	Št. načrta:	18/2026
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA	Št. projekta:	01/2025
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo		
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA		
Vsebina risbe:			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:			
Datum:	Maj 2026	Merilo:	1:50
		List št.:	14

Objekt: ☐

Št.risbe: 14☐

Projekt

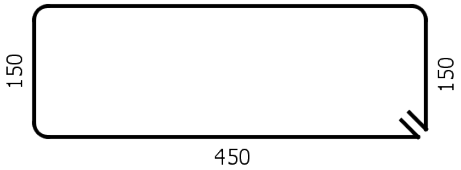
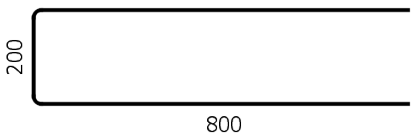
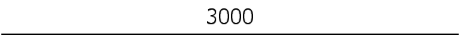
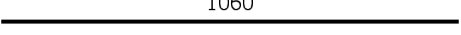
Objekt:

Die Längenangabe wird auf der Basis der Außenkontur berechnet.

Seznam količin

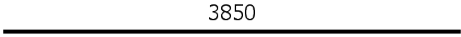
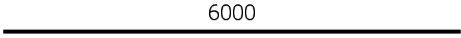
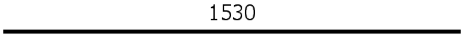
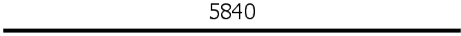
Material: Default				
Premjer armaturnih palic	Tip elementov	Teža na m ¹	Skupaj dolžina	Skupaj teža
8 mm	ø8, Dmin=4φ	0.41 kg/m	724.50	297.05 kg
14 mm	ø14, Dmin=4φ	1.25 kg/m	321.46	401.82 kg
Skupaj:				698.87 kg

Izveček armaturnih palic

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab-form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	1	8 mm	1.41	210	296.10		31	2 2	450	150	450	150		100 100	
															
	2	8 mm	1.80	228	410.40		21		800	200	800				
															
	3	8 mm	3.00	6	18.00		00		3000						
															
	4	14 mm	1.06	5	5.30		00		1060						
															
	5	14 mm	1.49	5	7.45		00		1490						

Objekt: ☐

Št.risbe: 14☐

Konst. element	Oznaka	Ø	Dolžina palice [m]	kom	Skupaj dolžina		Stab- form	Kljuka start/konec	A [mm]	B [mm]	C [mm]	D [mm]	E / R	H _{Start} / H _{Ende} [mm]	Rev
	6	14 mm	3.85	5	19.25		00		3850						
															
	7	14 mm	6.00	31	186.00		00		6000						
Kommentar: ±															
	8	14 mm	1.53	18	27.54		00		1530						
Kommentar: ±															
	9	14 mm	5.84	13	75.92		00		5840						
Kommentar: ±															

Seznam armaturnih mrež

Konst. element	Oznaka	Tip mreže	Teža/kos neto	Kom	Skupaj teža neto	Dolžina x Širina	Oblika	A	B	C	D	E / R	Rev
	1	Q503	103.00 kg	6	618.00 kg	6000 x 2150	00						
Kommentar: ±													
	2	Q503	91.67 kg	4	366.68 kg	5340 x 2150	00						
Kommentar: ±													
	3	Q503	51.50 kg	2	103.00 kg	3000 x 2150	00						
Kommentar: ±													
	4	Q503	48.75 kg	2	97.51 kg	2840 x 2150	00						
Kommentar: ±													
	5	Q503	37.85 kg	2	75.69 kg	3000 x 1580	00						
Kommentar: ±													
	6	Q503	75.69 kg	2	151.39 kg	6000 x 1580	00						
Kommentar: ±													
	7	Q503	35.83 kg	2	71.66 kg	2840 x 1580	00						
Kommentar: ±													

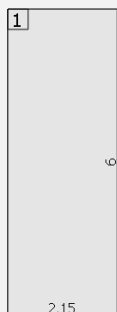
Objekt: ☐

Št.risbe: 14☐

Kosovnica armaturnih mrež

Tip mreže Q503

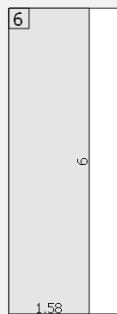
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 103.00 kg;
Kom: 6;
B. teža skupaj: 618.00 kg;
N. teža skupaj: 618.00 kg;



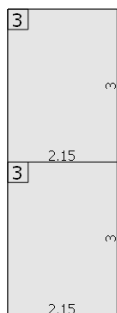
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 91.67 kg;
Kom: 4;
B. teža skupaj: 412.00 kg;
N. teža skupaj: 366.68 kg;



Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 75.69 kg;
Kom: 2;
B. teža skupaj: 206.00 kg;
N. teža skupaj: 151.39 kg;

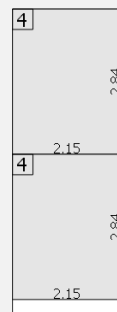


Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 103.00 kg;
Kom: 1;
B. teža skupaj: 103.00 kg;
N. teža skupaj: 103.00 kg;

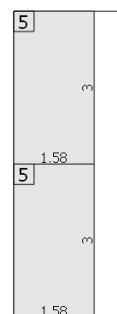


Tip mreže Q503

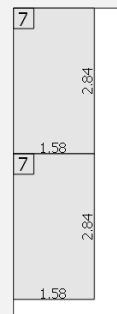
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 97.51 kg;
Kom: 1;
B. teža skupaj: 103.00 kg;
N. teža skupaj: 97.51 kg;



Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 75.69 kg;
Kom: 1;
B. teža skupaj: 103.00 kg;
N. teža skupaj: 75.69 kg;



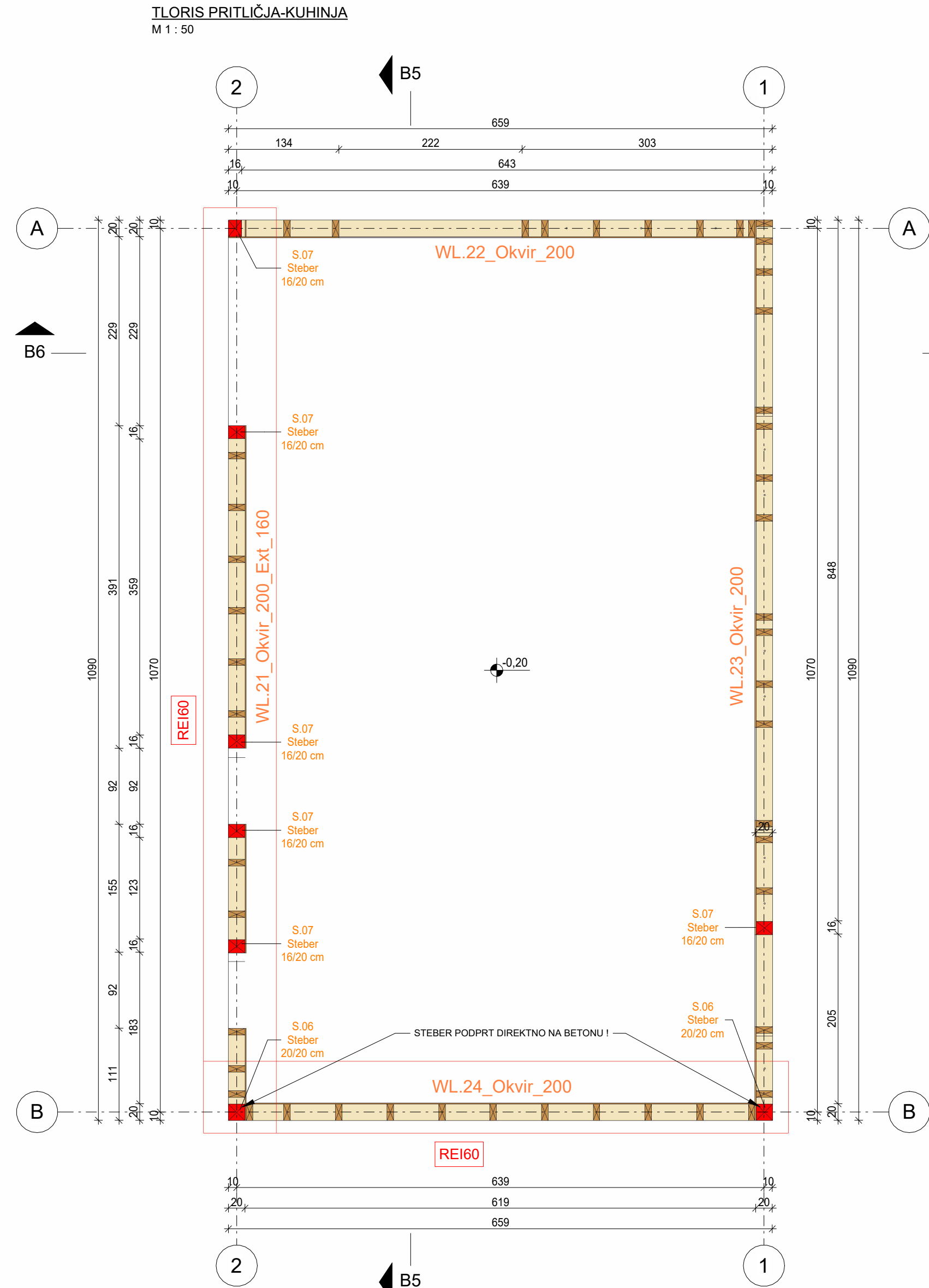
Širina: 2150;
Länge: 6000;
B.teža: 103.00 kg;
N.teža: 71.66 kg;
Kom: 1;
B. teža skupaj: 103.00 kg;
N. teža skupaj: 71.66 kg;



Skupaj

16

Bruto teža
1648.00 kg
Neto teža
1483.92 kg



POŽARNA ODPOORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zaščiteno steno izpolnjuje zahtevo REI60.

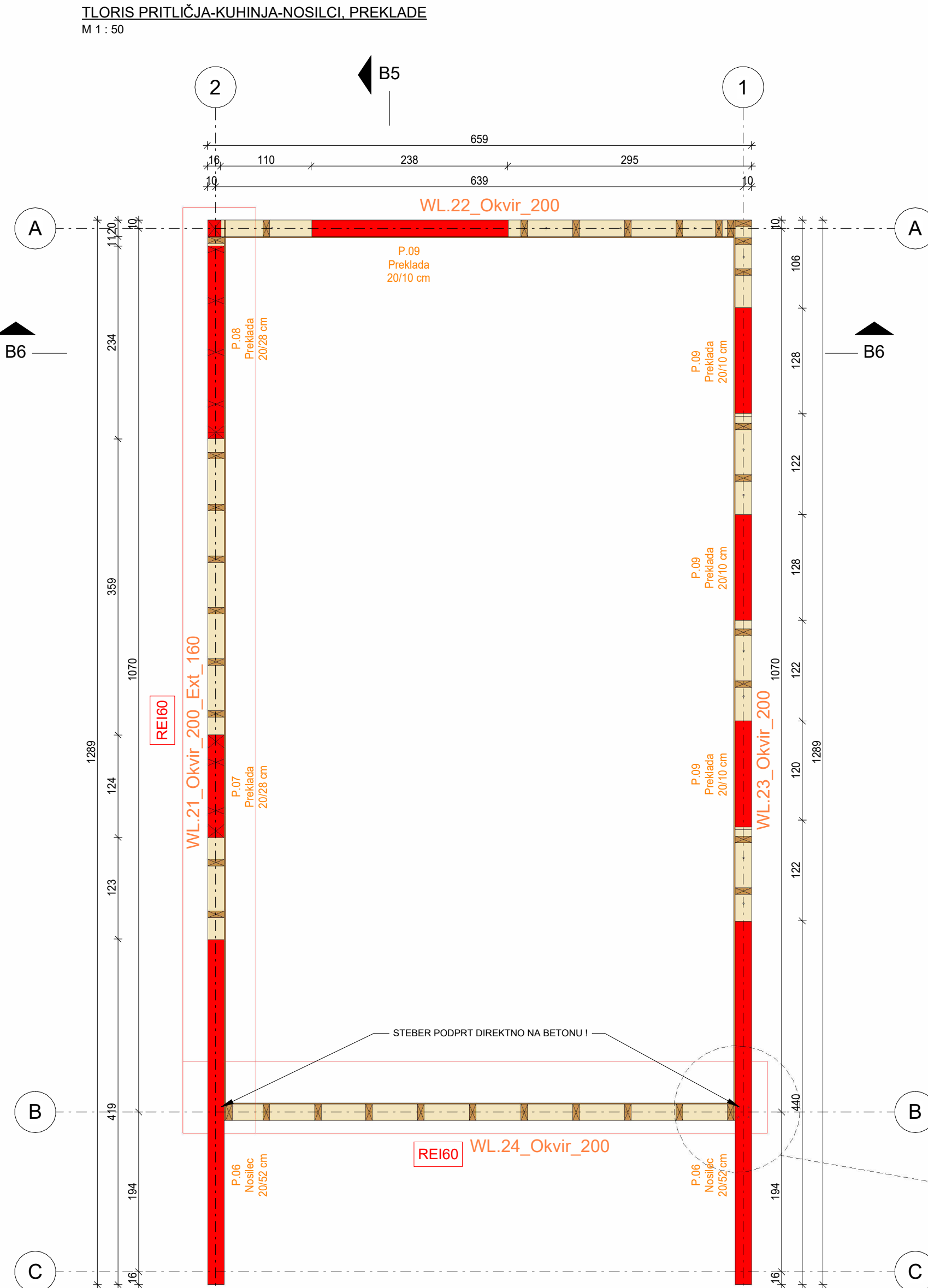
- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

SPLOŠNA NAVODILA ZA VIJAČENJE LESENE KONSTRUKCIJE

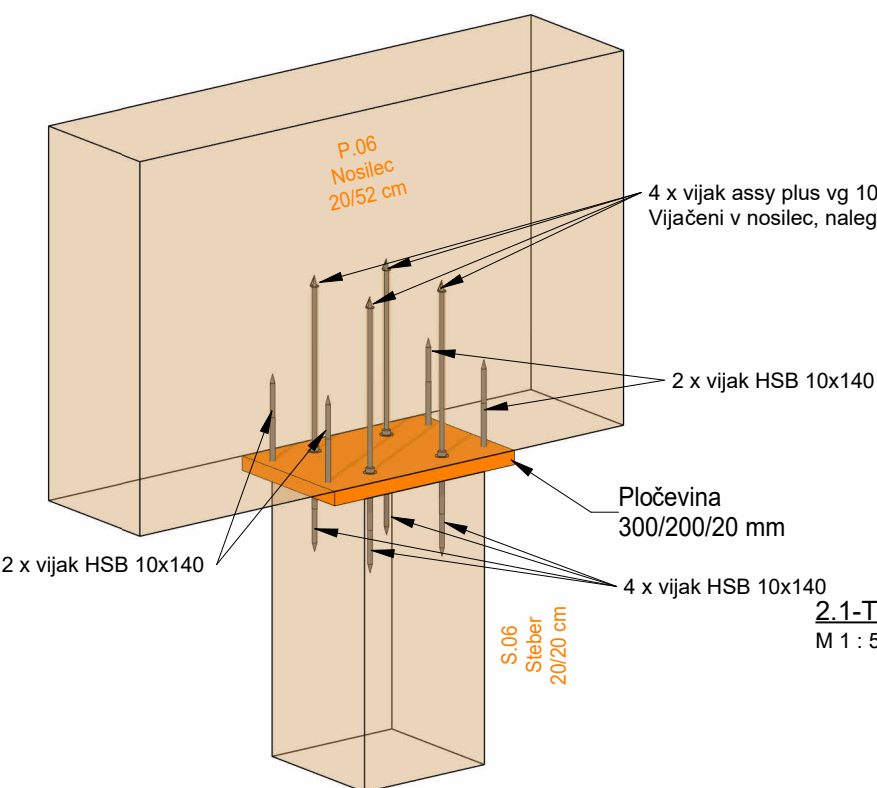
- Vogalni in T-spolji sten se vijajo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm.
- vsak špirovce se vijajo v steno ali lego s samovreznimi lesnimi vijaki premera Ø8 in s povečano glavo ali podložko.
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm.
- medetažne križno lepilne plošče se v stene vijajo z vijaki premera Ø8 na največji razdalji e=250mm.
- medetažne križno lepilne plošče se na L-preklopih medsebojno vijajo z vijaki premera Ø8 na največji razdalji e=250mm.
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera Ø8 na največji razdalji e=300mm.

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detalji povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

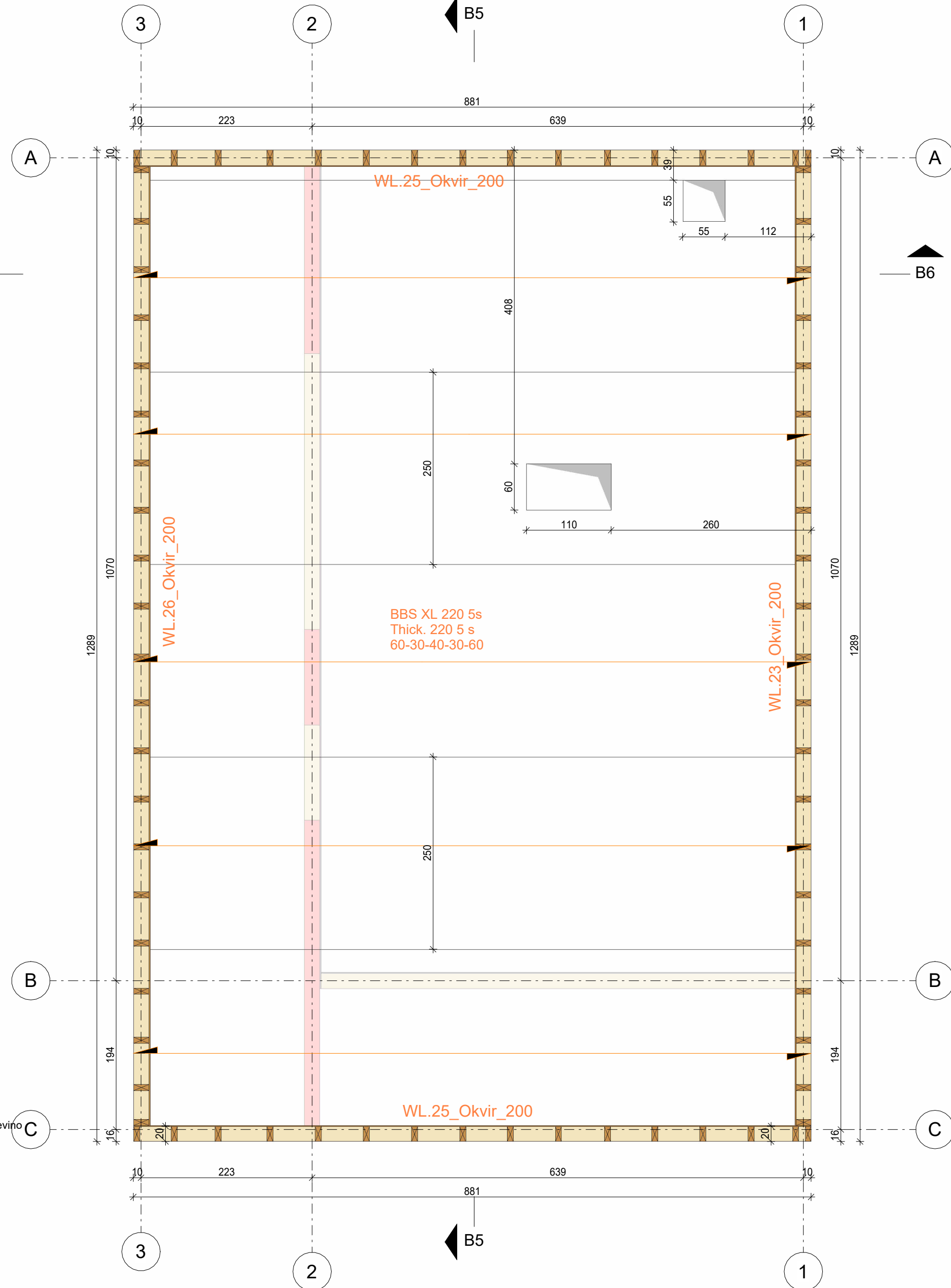


3D DETALJ STEBER-NOSILEC



2.1-TLORIS RAVNE STREHE-KUHINJA

M 1 : 50



Statične ojačitve v lesenih stenah (_SO = Statična ojačitev)			
Številka	Gradnik	Vrsta ojačitve	Material (_SO = Statična ojačitev)
P.06	DT_Prekлада_Les GL24_20x52_SO	Nosilec	Les-GL24_SO
P.07	DT_Prekлада_Les GL24_20x28_SO	Preklada	Les-GL24_SO
P.08	DT_Prekлада_Les GL24_20x28_SO	Preklada	Les-GL24_SO
P.09	DT_Prekлада_Les C24_20x10_SO	Preklada	Les-C24_SO
S.06	DT_Steber_Les GL24_20x20_SO	Steber	Les-C24_SO
S.07	DT_Steber_Les C24_16x20_SO	Steber	Les-C24_SO

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. Samovoljno prilagajanje projekta ni dovoljeno

LEGENDA MATERIALOV - LES

- LESENI ELEMENTI V POGLEDU
- LESENI ELEMENTI V PREREZU
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

LEGENDA MATERIALOV - JEKLO

- JEKLO S235 - V POGLEDU
- JEKLENI ELEMENTI S235 - V PREREZU
- JEKLO S235 - SPOJI V POGLEDU IN PREREZU
- OZNAKA ZA DOLŽINSKI SPOJ

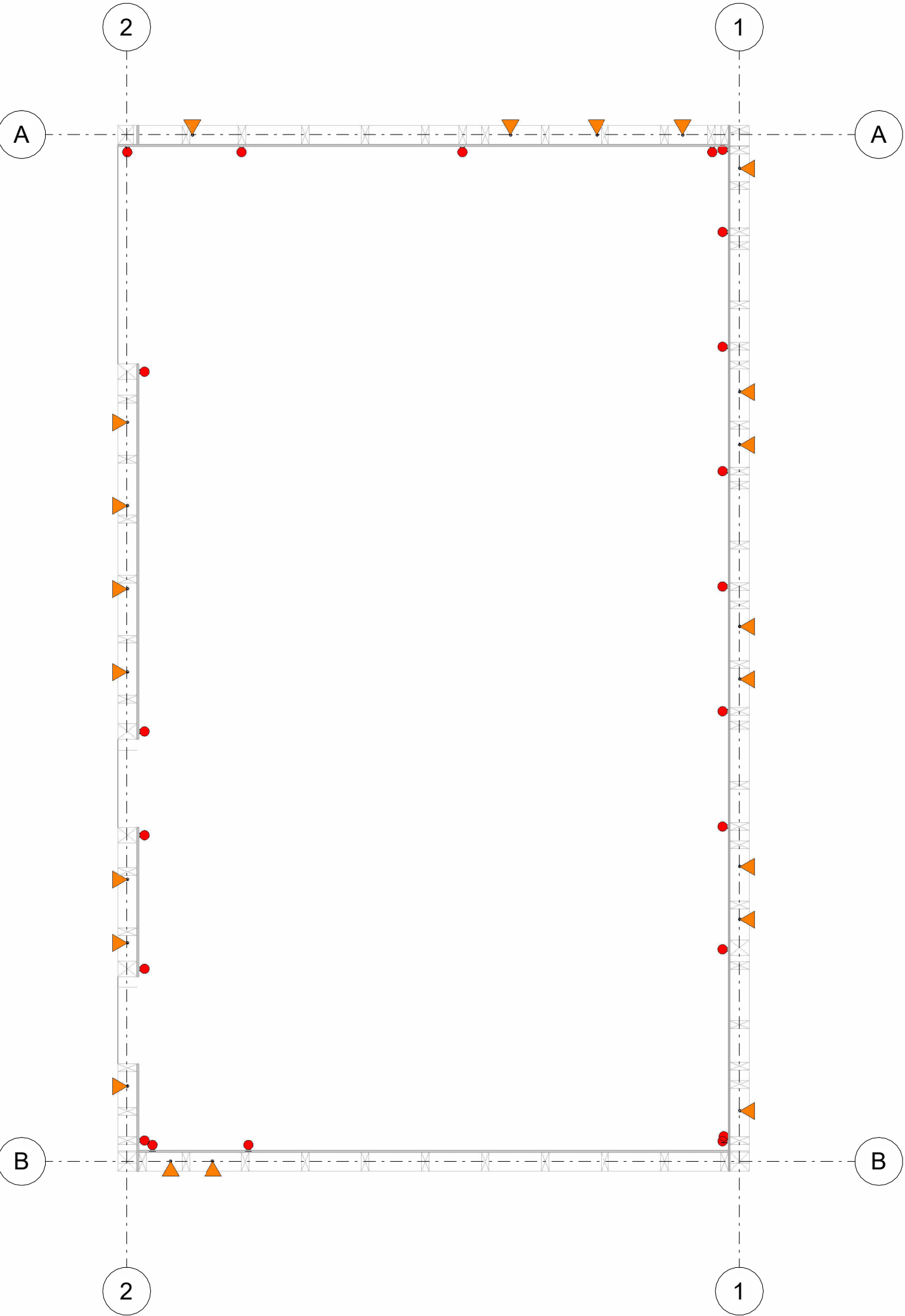
KUHINJA - TLORISI - NOSILNA KONSTRUKCIJA

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA

Lokacija:		
Vrsta projekta:	PZ1 - projekt za izvedbo	Št. načrta: 18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADNENIŠTVA	Št. projekta: 01/2025
Vsebinska risba:		
Vodja projekta:	Jure Kotnik, mag.inž.arh. ZAPS 2309 PA	
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801	
Projekant:		
Datum:	Maj 2026	Merilo: 1:50 List št.: 15



SIDRANJE - opisi in simboli

DVIŽNI KOTNIK
DVIŽNO SIDRANJE
Kotnik: WKR 53035
V steno: 16x žebelj LBA 4,0 x 60
Sidranje v AB ploščo: 1x navojna palica - 5.8 - M12x195 + kemična masa HYB-FIX

SIDRNI VIJAK M16x240
STRIŽNO SIDRANJE
SIDRNI VIJAKI SKOZI PASNICO V AB ZIDEC
Mehansko sidro M16 x 240
e_max = 1000 mm

PERFORIRAN TRAK
DVIŽNO SIDRANJE
Tip traku: 60 x 1,5 mm S350
Spoj zgoraj: 18 x žebelj LBA 4,0 X 60
Spoj spodaj: 18 x žebelj - LBA 4,0 X 60

2 DVOJNI DVIŽNI KOTNIK zgoraj/spodaj (stene)
DVIŽNO SIDRANJE
Kotnik: WKR 285
Spoj na steno zgoraj: 16 x žebelj LBA 4,0 x 60
Povezava prek medetažne ploščice: svornik M12 (5.8)
Spoj na steno spodaj: 16 x žebelj LBA 4,0 x 60

2 x DVIŽNI KOTNIK (stebri, ki podpirajo lege)
2X WKR 285
žebliji 4x60
polno žeblijanje
navzdol vijak VGS 11x125 + HUS10

LESNI VIJAK HBS 8x200
STRIŽNO SIDRANJE
LESNI VIJAKI SKOZI PASNICO V CLT PLOŠČO
HBS 8x200
e_max = 300 mm

LESNI VIJAK HBS 8x320 (os E)
STRIŽNO SIDRANJE
LESNI VIJAKI SKOZI KAPNO LEGO V CLT PLOŠČO
HBS 8x320
e_max = 300 mm

NAVOJNE PALICE M16 (8.8) + PODLOŽKE DIN 1052
OBEŠANJE e=300 (LES); 500mm (JEKLO)
NAVOJNE PALICE M16 (8.8) IN PODLOŽKE DIN 1052 !

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

POŽARNA ODPORNOST LESENE KONSTRUKCIJE

- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z negorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

SPLOŠNA NAVODILA ZA VIJAČENJE LESENE KONSTRUKCIJE

- Vogalni in T-spoji sten se vijačijo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovce se vijači v steno ali lego s samovreznim lesnim vijakom premera Ø8 in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijači v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijačijo z vijaki premera Ø8 na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklopih medsebojno vijačijo z vijaki premera Ø8 na največji razdalji e=250mm,
- stebri in ojačitve ter vsi konstrukcijski elem. se vijačijo z vijaki premera Ø8 na največji razdalji e=300mm,

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

KUHINJA - TLORIS PRITLIČJA - SIDRANJE

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta:	18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta:	01/2025
Vsebina risbe			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:	Merilo: 1:50		
Datum:	Maj 2026	List št.:	16

M 1 : 50



- Celotna konstrukcija ima požarno odpornost najmanj R30.
- Vse lesene okvirne stene so polno zapolnjene z nergorljivo toplotno/zvočno izolacijo.
- Notranja stena OKVIR 100 obojestransko obložena z mavčno kartonskimi ploščami tipa F deb. 15mm.
- Označene stene proti obstoječemu objektu izkazujejo požarno odpornost najmanj R60.
- Stropne plošče nad kuhinjo izkazujejo požarno odpornost najmanj R60.
- Jekleni deli konstrukcije zaščiteni s protipožarnim premazom ali oblogami za zagotovitev odp. najmanj R30.

- Sestava ravne strehe nad kuhinjo -> referenčna oz. ekvivalentna sestava v prilogi.
- Predvidena CLT strešna plošča s požarno zatesnjenimi stiki izpolnjuje zahtevo REI60.

- Sestava zunanje stene proti obstoječemu objektu -> referenčna oz. ekvivalentna sestava v prilogi.
Predvidena sestava zunanje stene izpolnjuje zahtevo REI60.

M 1 : 50



- Vogalni in T-spoj steni se vijajo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm ali z navojnimi palicami M12 (5.8) na največji razdalji e=1000mm,
- vsak špirovce se vijajo v steno ali lego s samovreznimi lesnimi vijakom premera Ø8 in s povečano glavo ali podložko,
- vsaka strešna lega lega, nosilec ali preklada, ki je vzporedna steni, se vijajo v steno in medetažno ploščo s samovreznimi lesnimi vijaki premera Ø8 na največji razdalji e=300mm,
- medetažne križno lepljene plošče se v stene vijajo z vijaki premera Ø8 na največji razdalji e=250mm,
- medetažne križno lepljene plošče se na L-preklopih medsebojno vijajo z vijaki premera Ø8 na največji razdalji e=250mm,
- stebri in ojačitve ter vsi konstrukcijski elem. se vijajo z vijaki premera Ø8 na največji razdalji e=300mm.

Dolžina lesnih vijakov se prilagodi glede na debelino priključenih elementov.

Vsi detajli povezovanja lesene konstrukcije in delavniški načrti morajo biti pregledani in potrjeni s strani projektanta!

OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

LEGENDA MATERIALOV - LES

- | | |
|---|--|
|  | LESENI ELEMENTI V POGLEDU |
|  | LESENI ELEMENTI V PREREZU |
|  | LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED |
|  | LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ |

LEGENDA MATERIALOV - JEKLO

- | | |
|---|---|
|  | JEKLO S235 - V POGLEDU |
|  | JEKLENI ELEMENTI S235 - V PREREZU |
|  | JEKLO S235 - SPOJI V POGLEDU IN PREREZU |
|  | OZNAKA ZA DOLŽINSKI SPOJ |

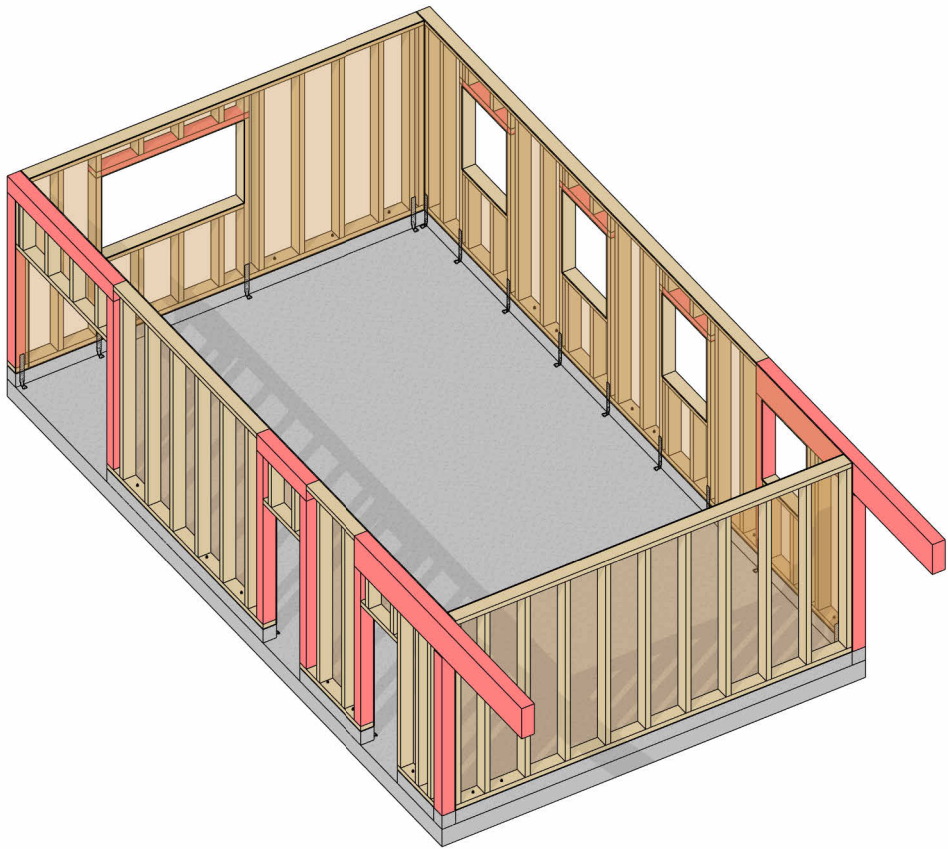
KUHINJA - PREREZI

Sprememba:	Datum spremembe:
	Podpis:

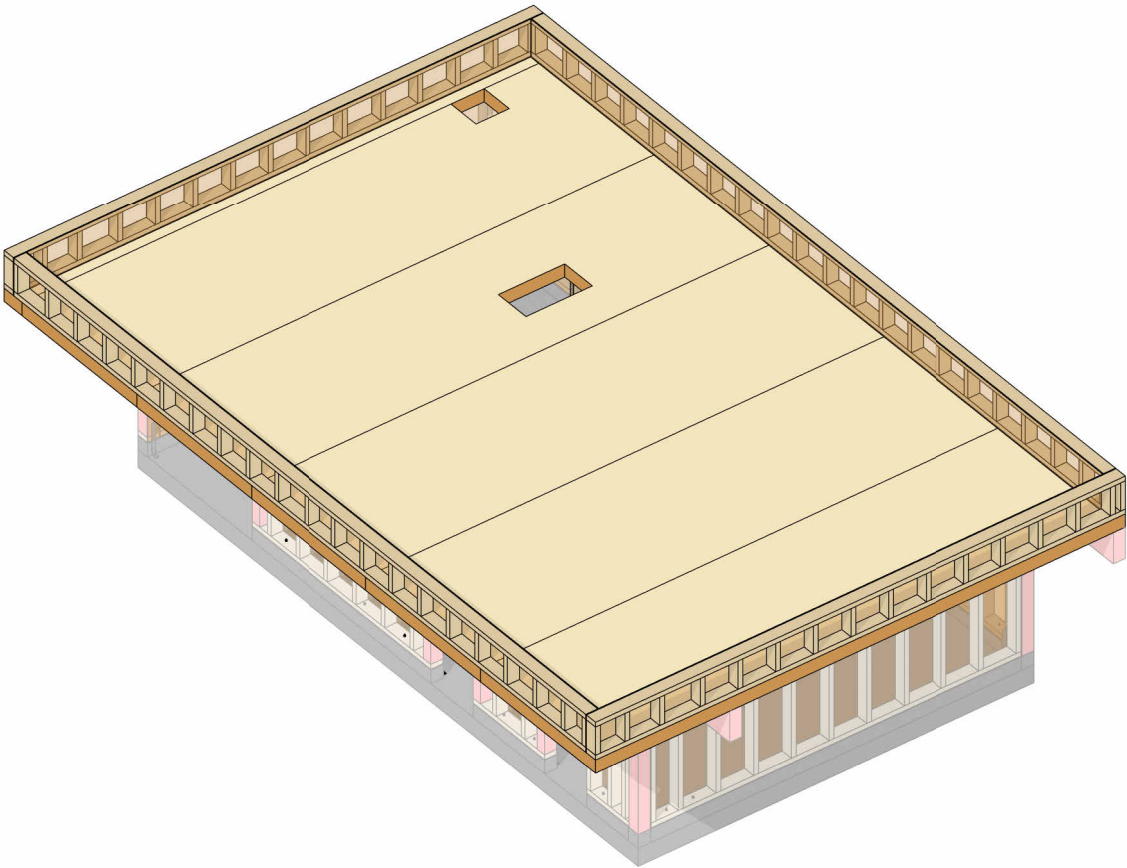


Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta:	18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta:	01/2025
Vsebina risbe:			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:	Merilo: 1:50		
Datum:	Maj 2026	List št.:	17

3D KONSTRUKCIJA-PRITLIČJE



3D KONSTRUKCIJA-STREHA



OPOMBE:

- vse mere preveriti na licu mesta
- armaturo prilagoditi prebojem
- morebitna neskladja med posameznimi na črti je potrebno takoj sporočiti odgovornemu vodji projekta. **Samovoljno prilagajanje projekta ni dovoljeno**

LEGENDA MATERIALOV - LES

- LESENI ELEMENTI V POGLEDU
- LESENI ELEMENTI V PREREZU
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - POGLED
- LESENI ELEMENTI - STATIČNA OJAČITEV V STENI - PREREZ

LEGENDA MATERIALOV - JEKLO

- JEKLO S235 - V POGLEDU
- JEKLENI ELEMENTI S235 - V PREREZU
- JEKLO S235 - SPOJI V POGLEDU IN PREREZU
- OZNAKA ZA DOLŽINSKI SPOJ

KUHINJA - 3D POGLEDI - NOSILNA KONSTRUKCIJA

Sprememba:	Datum spremembe:
	Podpis:



Investitor:	OBČINA MEŽICA TRG SVOBODE 1, 2393 MEŽICA		
Objekt:	OŠ MEŽICA VRTEC - PRIZIDEK IN REKONSTRUKCIJA		
Lokacija:			
Vrsta projekta:	PZI - projekt za izvedbo	Št.načrta:	18/2026
Vrsta načrta:	NAČRT S PODROČJA GRADBENIŠTVA	Št.projekta:	01/2025
Vsebina risbe:			
Vodja projekta:	Jure Kotnik, mag.inž.arh, ZAPS 2309 PA		
Pooblaščen inženir:	Miha Oražem, u.d.i.g., IZS G-3801		
Projektanti:			Merilo: 1:50
Datum:	Maj 2026	List št.:	18