

NASLOVNA STRAN S KLJUČNIMI PODATKI

2 Načrt gradbeništva
2.1 Načrt zaščite gradbene jame

INVESTITOR

Ministrstvo za vzgojo in izobraževanje
Masarykova cesta 16, 1000 Ljubljana

OBJEKT

ŠPORTNA DVORANA - ŠOLSKI CENTER NOVO MESTO

VRSTA PROJEKTNE DOKUMENTACIJE

PZI

ZA GRADNJO

Nova gradnja

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028-23

ŠTEVILKA NAČRTA

028-23-21

IZVOD

1 2 A

KRAJ IN DATUM IZDELAVE

Ajdovščina, maj 2024

KAZALO VSEBINE NAČRTA ŠT. 028-23-21

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1 TEHNIČNO POROČILO

1.1 Splošno

Za naročnika Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana, se je skladno s ponudbo izdelal načrt varovanja gradbene jame za izvedbo objekta ŠPORTNA DVORANA - ŠOLSKI CENTER NOVO MESTO na lokaciji Šolskega centra v Novem mestu.

1.2 Podloge za načrt

Pri izdelavi načrta smo kot projektne osnove uporabili naslednje podloge:

- Načrt arhitekture, št. 30/2023, SPINA d.o.o
- Geološko geomehanski elaborat št. 028-23-201, AC&P inženirski biro d.o.o
- Arhivske podatke že obstoječih objektov, Šolski center NM:

1.3 Podloge projektov (povzetek)

1.3.1 Zasnova objektov

Predvidena je novogradnja športne dvorane na lokaciji Šolskega centra v Novem mestu. Nova dvorana se nahaja na lokaciji obstoječega parkirišča, v atriju objektov MIC, delavnic, športne dvorane Leona Štuklja in objekta šole. Predviden objekt je pravokotne oblike s poševno zahodno steno, ki se na vogalih zaključi z vogali obstoječih objektov. Z obstoječo dvorano Leona Štuklja bo nova športna dvorana povezana preko hodnika na jugovzhodni strani.

Nosilna konstrukcija objekta je praktično v celoti armirano betonska. Nosilno AB konstrukcijo sestavljajo temeljne grede, talna, medetažne in strešna plošča, vertikalni okvirji ter stene. Okvirji, ki so sestavljeni iz stebrov in prečk, v sodelovanju s stenami, ki obdajajo stopniščne in dvigalne jaške, prevzemajo tako vertikalno, kot tudi horizontalno obtežbo.

1.3.2 Geološko – geomorfološki opis območja in karakteristične vrednosti materialov

Obravnavana lokacija se nahaja na JZ delu Novega mesta, ob športni dvorani Leona Štuklja.

Geološko (hribinsko) podlago širšega območja sestavlja bel/siv plastovit apnenec z redkimi plastmi dolomita in oolitni apnenec ($J_3^{2,3}$). Nad apnencem se pojavlja plast rjave pliokvartarne gline spremenljive debeline in sicer med 1,0 – 12,0 metri. V spodnjem delu, na kontaktu z apnencem se pojavlja plast glinastega gruščja, ki nakazuje prehod v trdo hribinsko podlago. Glede na rezultate geomehanskih preiskav ugotavljamo, da se globina do trdne podlage generalno veča od zahoda proti vzhodu.

Na osnovi izvedenih preiskav vzorcev iz obravnavanega območja in arhivskih podatkov smo izbrali karakteristične podatke o strižnih karakteristikah zemljin. Pri izboru smo upoštevali, poleg povprečnih vrednosti tudi variabilnost.

MATERIAL	Prostorn. teža	Enoosna tlačna trdnost	Nedrenirana strižna trdnost	Kohezija	Strižni kot	Modul elastičnosti
	γ	q_u	s_u	C	φ	E
	[kN/m ³]	[kPa]	[kPa]	[kPa]	[°]	[kPa]
Umetno nasutje (GC)	18,5	250	125	5,0	28	12.000
Rjava, težko gnetna meljasta glina z občasnim gruščem (CL-ML) – ilovica	17,0	150	75	10,0	23	6.000
Rjav, glinast, srednje do debelozrnat grušč (GC), mestoma bolj glinast	19,5	-	-	2,0	30	50.000
Kompaktna hribinska podlaga (J_3^{2+3})	25,0	-	-	40,0	40	550.000

Tabela 1: Karakteristične vrednosti geomehanskih parametrov

1.4 Opis podpornih ukrepov

1.4.1 Splošno

Za potrebe izgradnje objektov smo zasnovali začasno varovanje gradbene jame z različnimi ukrepi. Ti sledijo ekonomskim, geološko-geomehanskim, morfološkim, tehnološkim in drugim zahtevam. Varovanje izkopa gradbene jame bodo tvorili:

- širok izkop pod ustreznimi nakloni,
- sidrana in dvakrat sidrana pilotna stena,
- berlinska stena iz zabitih timic.

Celotna izvedba varovanja bo omogočala varne izkope, odvoze materiala ter izgradnjo objektov. Poleg tega je prilagojena tako, da bo minimalno vplivala na učne procese v šolskem centru.

Dostop v gradbeno jamo je predviden iz jugozahodne strani. Prav tako bo ves bistveni gradbiščni transport potekal tu.

1.4.2 Standardi

Vsi stabilnostni preračuni so bili izvedeni po MKE s programom Phase II, skladno z navodili EC7, DA1. Rezultati stabilnostnih in statičnih izračunov so predstavljeni v prilogah.

Gradbena jama je računana po pravilih in smernicah standardov iz družine Evrocode.

- | | |
|---|------|
| • osnove izračuna in obtežbe | EC 1 |
| • dimenzioniranje betonskih konstrukcij | EC 2 |
| • dimenzioniranje jeklenih konstrukcij | EC 3 |
| • geomehanske konstrukcije | EC 7 |

1.4.3 Široki izkop

Povsod tam, kjer ni prostorskih omejitev je predviden izkop gradbene jame pod naklonom, ki omogoča varno izvedbo. Široki izkopi so predvideni deloma na severni, deloma na vzhodni, deloma na južni in deloma na zahodni brežini objekta. Glede na stabilnostne analize so predvideni naslednji začasni nakloni izkopov:

- | | |
|---|-----|
| 1. Umetno nasutje: rjav glinast drobno do srednjezrnat grušč (GC), | 1:1 |
| 2. Rjava, težko gnetna meljasta glina z občasnim gruščem (CL-ML) – ilovica, | 1:1 |
| 3. Rjav, glinast, srednje do debelozrnat grušč (GC), mestoma bolj glinast, | 1:1 |
| 4. Kompaktna hribinska podlaga (J_3^{2+3}). | 3:1 |

1.4.4 Berlinska stena

Za namen začasnega varovanja odseka na severovzhodnem obodu jame smo predvideli t.i. berlinsko steno, ki je sestavljena iz tirnic S49 dolžine 4,0m. Tirnice se izvede na razmaku 75cm in se jih založi s lesenimi plohi. Iz pripravljenega platoja se z bagrom zabije oz. vtisne tirnico do predvidene globine, na razmaku 75cm. Tirnice se v primeru, da ne bo zagotovljeno zadostno vpetje (orientacijsko 1/3 dolžina tirnice pod dnom izkopa) poveže z jekleno gredo dimenzij 30x30cm, skladno s projektom. Ves material (tirnice in grede) se po končani gradnji odstrani.

1.4.5 Sidrana pilotna stena sever – PS-1

1.4.5.1 Splošno

Po severnem obodu gradbene jame je predvidena enkrat sidrana pilotna stena. Zgornja AB vezna greda je prereza 70/70cm. Predvidena dolžina pilotov je od 4,0 do 6,3m. Piloti so premera $\Phi 50$ cm in bodo uvtani 1,25m v apnenčasto podlago. Za potrebe dimenzioniranja smo izvedli stabilnostne analize, ki jih prilagamo v posebnem razdelku.

1.4.5.2 Piloti

- Piloti: $\phi 50$ cm/1,0m
- Jeklo: B500
- Beton in prerez : C30/37
- Stremena: $\phi 12/15$ cm
- Vzдолžna armatura: 8 $\phi 16$
- Zaščitni sloj: 8cm

POZ.	Y-KOOR	X-KOOR	Z-KOOR vrha pilota	DOLŽINA PILOTOV (m)	PREMER (cm)
S1	72856.0410	512304.7724	196,26	4,2	0.5
S2	72856.0455	512305.7724	196,26	4,4	0.5
S3	72856.0501	512306.7724	196,26	4,7	0.5
S4	72856.0547	512307.7724	196,26	4,9	0.5
S5	72856.0592	512308.7724	196,26	5,1	0.5
S6	72856.0638	512309.7724	196,26	5,1	0.5
S7	72856.0684	512310.7724	196,26	5,2	0.5
S8	72856.0729	512311.7724	196,26	5,3	0.5
S9	72856.0775	512312.7724	196,26	5,4	0.5
S10	72856.0821	512313.7724	196,26	5,5	0.5
S11	72856.0866	512314.7723	196,26	5,6	0.5
S12	72856.0912	512315.7723	196,26	5,7	0.5
S13	72856.0958	512316.7723	196,26	5,8	0.5
S14	72856.1003	512317.7723	196,26	5,9	0.5
S15	72856.1049	512318.7723	196,26	6,0	0.5
S16	72856.1095	512319.7723	196,26	6,1	0.5
S17	72856.1140	512320.7723	196,26	6,2	0.5
S18	72856.1186	512321.7723	196,26	6,3	0.5
S19	72856.1232	512322.7723	196,26	6,4	0.5
S20	72856.1277	512323.7722	196,26	6,5	0.5
S21	72856.1323	512324.7722	196,00	6,3	0.5
S22	72856.1369	512325.7722	195,51	6,0	0.5
S23	72856.1414	512326.7722	195,00	5,5	0.5
S24	72856.1460	512327.7722	194,50	5,2	0.5
S25	72856.1506	512328.7722	194,00	4,8	0.5

Tabela 2: Tabela dispozicije pilotov za pilotno steno PS-1

1.4.5.3 Sidrna vezna greda

Zgornja vezna greda povezuje Benotto pilote. Je kvadratnega prereza dimenzij $b \times h = 70 \times 70 \text{ cm}$. Armatura vezne grede je rebrasta S500. Vezna greda je iz armiranega betona C25/30. V vezni gredi se na predpisanih pozicijah izvede zobe pod naklonom 35° od vertikale. Dimenzije poševne ploskve znašajo $30/30 \text{ cm}$. V središče poševne ploskve in pravokotno na njeno površino se vgradi cev PVC DN160, skozi katero se izvede sidro. Na vsako stran cevi se vgradi po 2 stremeni s prirezano stranico.

- Jeklo: S500
- Beton: C25/30
- Stremena: $\phi 12/20 \text{ cm}$
- Vz dolžna armatura: 20 $\phi 16$
- Zaščitni sloj: 5cm

1.4.5.4 Sidra

Uporabi se tri vrвна trajna prednapeta geotehnična sidra. Sidra bodo uvrtna min. $6,0 \text{ m}$ v apnenčasto osnovo. Sidra se napne s predvideno silo, ko injekcijska masa doseže 75% trdnosti.

- Sidra: $3 \times 6''/251 \text{ kN}$, grede pod kotom 35°
- Jeklo / nosilnost: $f_{pk} = 1860 \text{ MPA}$
- Nosilnost sidra: $P_{\max} = 586 \text{ kN}$
- Sila prednapetja: $P_0 = 251 \text{ kN}$
- Dimenzije: $3 \times 0,62''$ ($3 \times 150 \text{ mm}^2$)
- Vezna dolžina: $6,0 \text{ m}$

1.4.6 Sidrana pilotna stena jug - PS-2

1.4.6.1 Splošno

Po južnem obodu gradbene jame je predvidena enkrat sidrana pilotna stena. Zgornja AB vezna greda je prereza $50/50 \text{ cm}$. Predvidena dolžina pilotov je od $2,7$ do $4,9 \text{ m}$. Piloti so premera $\phi 40 \text{ cm}$ in bodo uvrtni $1,0 \text{ m}$ v apnenčasto podlago.

Piloti bodo predvidoma izvedeni nad koto gradbene jame. Za namene sidranja nog pilotov se uporabi palična pasivna IBO sidra R32/250 dolžine $6,0 \text{ m}$.

Za potrebe dimenzioniranja smo izvedli stabilnostne analize, ki jih prilagamo v posebnem razdelku.

1.4.6.2 Piloti

- Piloti: $\phi 40 \text{ cm}/0,80 \text{ m}$
- Jeklo: B500
- Beton in prerez : C30/37
- Stremena: $\phi 12/15 \text{ cm}$
- Vz dolžna armatura: $8 \phi 16$
- Zaščitni sloj: 8cm

POZ.	Y-KOOR	X-KOOR	Z-KOOR vrha pilota	DOLŽINA PILOTOV (m)	PREMER (cm)
J1	72803.8231	512307.3798	197,25	3,5	40
J2	72803.8265	512308.0722	197,25	3,5	40
J3	72803.8291	512308.8722	197,25	3,6	40
J4	72803.8326	512309.6721	197,25	3,6	40
J5	72803.8360	512310.4721	197,25	3,7	40
J6	72803.8395	512311.2721	197,25	3,7	40
J7	72803.8429	512312.0721	197,25	3,7	40
J8	72803.8463	512312.8721	197,25	3,8	40
J9	72803.8498	512313.6721	197,25	3,8	40
J10	72803.8532	512314.4721	197,25	3,9	40
J11	72803.8567	512315.2721	197,25	3,9	40
J12	72803.8601	512316.0721	197,25	4,0	40
J13	72803.8635	512316.8721	197,25	4,0	40
J14	72803.8670	512317.6721	197,25	4,1	40
J15	72803.8704	512318.4721	197,25	4,1	40
J16	72803.8739	512319.2721	197,25	4,2	40
J17	72803.8773	512320.0721	197,25	4,2	40
J18	72803.8808	512320.8720	197,25	4,2	40
J19	72803.8842	512321.6720	197,25	4,3	40
J20	72803.8876	512322.4720	197,25	4,3	40
J21	72803.8911	512323.2720	197,25	4,4	40
J22	72803.8945	512324.0720	197,25	4,4	40
J23	72803.8980	512324.8720	197,25	4,5	40
J24	72803.9014	512325.6720	197,25	4,5	40
J25	72803.9048	512326.4720	197,25	4,6	40
J26	72803.9083	512327.2720	197,25	4,6	40
J27	72803.9117	512328.0720	197,25	4,6	40
J28	72803.9152	512328.8720	197,25	4,7	40
J29	72803.9186	512329.6720	197,25	4,7	40
J30	72803.9220	512330.4720	197,25	4,8	40
J31	72803.9255	512331.2719	197,25	4,8	40
J32	72803.9289	512332.0719	197,25	4,9	40
J33	72803.9324	512332.8719	197,25	4,9	40
J34	72803.9358	512333.6719	197,07	4,8	40
J35	72803.9392	512334.4719	196,73	4,5	40
J36	72803.9427	512335.2719	196,38	4,2	40
J37	72803.9461	512336.0719	196,04	3,9	40
J38	72803.9496	512336.8719	195,69	3,6	40
J39	72803.9530	512337.6719	195,34	3,3	40
J40	72803.9564	512338.4719	195,00	3,0	40
J41	72803.9599	512339.2719	194,65	2,7	40

Tabela 3: Dispozicija pilotov za pilotno steno PS-2

1.4.6.3 Sidrna vezna greda

Zgornja vezna greda povezuje Benotto pilote. Je kvadratnega prereza dimenzij $b \times h = 50 \times 50 \text{ cm}$. Armatura vezne grede je rebrasta S500. Vezna greda je iz armiranega betona C25/30. V vezni gredi se na predpisanih pozicijah izvede zobe pod naklonom 35° od vertikale. Dimenzije poševne ploskve znašajo $30/30 \text{ cm}$. V središče poševne ploskve in pravokotno na njeno površino se vgradi cev PVC DN160, skozi katero se izvede sidro. Na vsako stran cevi se vgradi po 2 stremeni s prirezano stranico.

- Jeklo: S500
- Beton: C25/30
- Stremena: $\phi 12/20 \text{ cm}$
- Vz dolžna armatura: 12 $\phi 14$
- Zaščitni sloj: 5cm

1.4.6.4 Sidra

Predvidena je vgradnja začasnih pasivnih sider IBO R32/250 na razmaku 3,2 m, dolžine 12,0m. Sidra se vgradi pod naklonom 35° glede na horizontalo. Izvede se minimalno prednapetje, da se zagotovi naleganje sidrne glave na površino grede.

- Sidra: IBO R32/250
- Nosilnost sidra: $P_{\min} = 250 \text{ kN}$
- Sila prednapetja: $P_0 = 10 \text{ kN}$

1.5 IZVEDBA DEL IN TEHNOLOGIJA GRADNJE

1.5.1 Priprava gradbišča

Za potrebe izvedbe gradbene jame je potrebno najprej urediti in zavarovati gradbišče in urediti dostopne poti. Dostop do gradbišča se bo izvedlo iz jugozahodne strani. Po glavni dostopni cesti se bo odvijal vsa gradbiščni promet. Vse površine je potrebno ustrezno zavarovati in označiti, postaviti je potrebno gradbiščne provizorije. Pred pričetkom zemeljskih del je potrebno elemente gradbene jame zakoličiti. Zakoličiti je potrebno tudi osi objekta in preveriti razdalje med osmi objekta in osmi pilotne stene.

1.5.2 Komunalni vodi

Na območju so obstoječi komunalni vodi, ki se premaknejo v fazi priprave gradbišča. Prestavitev vodov je predmet drugega načrta.

1.5.3 Opis konstrukcijskih elementov in izvedba

Varovanje gradbene jame bomo izvajali z naslednjimi konstrukcijskimi elementi:

1.5.3.1 Sidrana pilotna stena PS-1

Mesta pilotov se zakoliči, zakoličiti je potrebno tudi osi. Na zakoličenih mestih se izdelajo pilote. Pilote se izvaja po Benotto metodi. Betoniranje pilotov se izvaja kontraktorsko.

Na območju je najprej potrebno izvesti generalni izkop do predvidene kote delovnega platoja za izvedbo pilotov.

Piloti fi 50cm bodo uvrtni 1,25m v matično hribinsko podlago ali do ustrezne globine pod dnom gradbene jame. Pilote se izvede do predvidene kote delovnega platoja. Prevzem izkopa pilota izvede geomehanik. Po izdelavi pilotov sledi znižanje platoja na koto dna vezne grede, kjer se položi podložni beton. Na podložni beton se postavi opaz in armaturo za gredo dimenzij 70x70cm.

Po izdelavi sidrne vezne grede se skozi plastične cevi in jeklene konuse vgradi začasna geotehnična sidra različnih dolžin. Nagib sider je 35° navzdol od horizontale. Sidra se vgradijo v predhodno izvrtane vrtnice Ømin 152 mm. Vrtine je potrebno ceviti v sloju gline in preperelega apnenca, v kompaktnem apnencu pa cevljenje ni potrebno. Potrebno je izvesti delovni plato za vrtno garnituro. K prednapenjanju sider se lahko pristopi, ko injektirna masa doseže 75% trdnosti. Zahtevana kvaliteta injektirne mase je C25/30. Na sidrih se mora izvesti preizkuse nosilnosti po standardu SIST EN 1537. Natančneje je potek sidranja obravnavan v elaboratu sidranja, ki se ga naredi v fazi izvedbe, ko bo izbrana tehnologija izvajalca. Pri izdelavi geotehničnih sider je potrebno slediti smernicam za izvedbo začasnih geotehničnih sider (SIST EN 1537).

Po montaži sidrne grede se sidra prednapne in zaklini. Na sidrih se mora izvesti preizkuse nosilnosti po standardu SIST EN 1537. Natančneje bo potek sidranja obravnavan v elaboratu sidranja.

Po izvedbi sidranja se lahko izvede izkop do dna gradbene jame. Izvede se temeljenje objekta in temeljno ploščo, kar ni del tega načrta. Glede na to, da pilotna stena ni izvedena kontaktno na objekt, razklinjanje sider ni potrebno. Sidra na pilotni steni se lahko razklini v fazi zasipa kletne etaže objekta.

1.5.3.2 Sidrana pilotna stena PS-2

Mesta pilotov se zakoliči, zakoličiti je potrebno tudi osi. Na zakoličenih mestih se izdelajo pilote. Pilote se izvaja po Benotto metodi. Betoniranje pilotov se izvaja kontraktorsko.

Na območju je najprej potrebno izvesti generalni izkop do predvidene kote delovnega platoja za izvedbo pilotov.

Piloti fi 40cm bodo uvrtni 1,0m v matično hribinsko podlago ali do ustrezne globine pod dnom gradbene jame. Pilote se izvede do predvidene kote delovnega platoja. Prevzem izkopa pilota izvede geomehanik. Po izdelavi pilotov sledi znižanje platoja na koto dna vezne grede, kjer se položi podložni beton. Na podložni beton se postavi opaz in armaturo za gredo dimenzij 50x50cm.

Po izdelavi sidrne vezne grede se skozi plastične cevi in jeklene konuse vgradi začasna pasivna IBOsidra R32/250. Sidra se vgradijo v predhodno izvrtane vrtnice Ømin 91 mm. Nagib sider je 35° navzdol od horizontale. Vrtine je potrebno ceviti v sloju gline in preperelega apnenca, v kompaktnem apnencu pa cevljenje ni potrebno. Potrebno je izvesti delovni plato za vrtno

garnituro. K prednapenjanju sider se lahko pristopi 8 dni po vgradnji oziroma, ko injektirna masa doseže 75% trdnosti. Zahtevana kvaliteta injektirne mase je C30/37. Na sidrih se mora izvesti preizkuse nosilnosti po standardu SIST EN 1537.

Sila zaklinjanja sider $P_0=50$ kN. Pri vgradnji pasivnih sider je potrebno slediti smernicam za izvedbo trajnih pasivnih sider SIST EN 14489 in smiselno tudi SIST EN 1537 ter SIA 267. Med preizkusom sider je potrebno voditi protokol napenjanja sider.

1.5.3.3 Berlinska stena

Predvidena je izvedba berlinske stene dolžine 23,5m na severovzhodnem delu gradbene jame. Tirnice tipa S49 dolžine 4,0m bodo od kote obstoječega terena vtisnjene v glino. Uvrtnje tirnic v kompaktno podlago ni potrebno. Tirnice se vtiskuje na razmaku 75cm in se jih založi z lesenimi plohi. V kolikor stena ne dosega primerne vpetja, se na vrhu berlinske stene izvede AB greda dimenzij 30x30cm.

1.5.4 Faznost izvedbe

Grobo bo potek izdelave gradbene jame potekal na naslednji način:

1. Ureditev gradbišča, ureditev dostopnih poti,
2. Generalni izkop obstoječega terena do nivoja platoja za pilotiranje,
3. Izvedba pilotov in veznih sidrnih gred,
4. Vgradnja in napenjanje sider,
5. Izvedba izkopa do dna gradbene jame oz. do pet pilotov (PS-2),
6. Sidranje pet pilotov (PS-2),
7. Izvedba temeljev objekta,

1.5.5 Protokol sidranja

Protokol sidranja bo obravnavan v elaboratu sidranja, ki bo izdelan, ko bo znana tehnologija izbranega izvajalca. Pred izvedbo sider je potrebno izvesti testna sidra. In sicer je predvidena izvedba 2 testov sider. Teste sider se izvede do porušitve ali do predpisane natezne sile, da se določi nosilnosti sider. Na podlagi izvedenih testov bo projektant podal nadaljnje korake glede izvedbe posameznih sider oziroma bodo podani nadaljnji potrebni ukrepi (morebitna dodatna sidra) skladno z ugotovljeno dejansko nosilnostjo posameznega sidra.

1.5.6 Odvodnjevanje

Za gradbeno jamo ni problematičen razpršen površinski dotok iz okoliških brežin, v primeru koncentriranih dotokov (iztoki meteorne kanalizacije, iztoki jarkov, iztoki cestiščne vode, hudournikov ipd.) pa je potrebno te iztoke zajeti višje in jih peljati okrog jame po utrjenih kanaletah ali jarkih, nikakor pa ni dopustno prelivanje koncentriranih iztokov v samo jamo. Za zajem površinskih vod, ki pritečejo v gradbeno jamo je potrebno na dnu jame ob robovih predvideti začasne mulde, črpališča ali primerljiv ukrep s katerim se vode zajame in kontrolirano odvede z dna jame.

1.5.7 Končne ureditve

Po končani izvedbi gradbene jame in objekta se po potrebi poruši vezna greda, da se pilote prilagodi končni ureditvi terena.

1.5.8 Zaključki

Pri izvedbi naj se izvaja geomehanski in projektantski nadzor, ki bo preveril primernost in ustreznost ukrepov. V primeru ugotavljanja neskladja med dejanskim stanjem in projektno dokumentacijo, je potrebno obvestiti projektanta in nadzornega inženirja. Projektant bo po potrebi podal dodatna navodila.

Vse izkope gradbene jame je potrebno izvajati pod geomehanskim nadzorom, temeljna tla morajo biti prevzeta s strani strokovnjaka geomehanika. V primeru neugodnih vremenskih vplivov je izkope potrebno zaščititi pred vremenskimi vplivi, da ne pride do zamakanja brežin.

2 PRILOGE

2.1 STABILNOSTNE ANALIZE IN STATIČNI IZRAČUNI

2.1.1 Uvod

V okviru izdelave projekta »ŠPORTNA DVORANA - ŠOLSKI CENTER NOVO MESTO« so bile kot osnova za dimenzioniranje izdelane stabilnostne analize predvidenih podpornih ukrepov. Analize so bile opravljene s programom RS2, po metodi končnih elementov (MKE). Izračuni so bili narejeni po standardu Evrokod 7, projektni pristop DA1.

Stabilnostne analize so bile izvedene za pilotno steno PS-1, PS-2 in berlinsko steno.

2.1.2 Pilotna stena PS-1

2.1.2.1 Faznost gradnje

Gradnjo sidrane pilotne stene smo zaradi zahtevnosti izvedbe modelirali v več bistvenih fazah:

1. Generalni izkop obstoječega terena do nivoja platoja za pilotiranje,
2. Izvedba pilotov in veznih sidrnih gred,
3. Vgradnja in napenjanje pasivnih sider,
4. Izvedba izkopa do dna gradbene jame,
5. Redukcija strižnih karakteristik, $F=1,25$.

V prilogah so prikazani rezultati stabilnostnih analiz za fazo 5.

2.1.2.2 Vhodni podatki in robni pogoji

V analizi smo uporabili materiale s karakteristikami, ki so bile povzete po geološko-geomehanskem poročilu v sklopu projektne dokumentacije in na podlagi povratnih analiz v nadaljevanju. Karakteristike materialov in konstrukcijskih elementov, uporabljenih v analizi, so prikazane v preglednici 1:

Material Name	Material Color	Initial Element Loading	Account for Moisture Content in Unit Weight	Unit Weight (kN/m ³)	Porosity Value	Elastic Type	Poisson's Ratio	Young's Modulus (kPa)	Use Residual Young's Modulus	Failure Criterion	Material Type	Peak Tensile Strength (kPa)	Peak Friction Angle (degrees)	Peak Cohesion (kPa)	Residual Tensile Strength (kPa)	Residual Friction Angle (degrees)	Residual Cohesion (kPa)	Dilation Angle (degrees)	Apply SSR (Shear Strength Reduction)	Unsaturated Behavior	Material Behaviour	Fluid Bulk Modulus (kPa)	Use Biot's Coefficient for calculating Effective Stress	Static Water Mode	Ru Value
Mejasta glina z občasnim gručem (CL)		Field Stress and Body Force	No	17.5	0.5	Isotropic	0.3	6000	No	Mohr-Coulomb	Plastic	0	23	5	0	23	5	0	Yes	None	Drained	2.2e+06	No	Ru	0
Glinast gruč		Field Stress and Body Force	No	19.5	0.5	Isotropic	0.3	20000	No	Mohr-Coulomb	Plastic	0	33	0.1	0	33	0.1	0	Yes	None	Drained	2.2e+06	No	Ru	0
Apnenec		Field Stress and Body Force	No	25	0.5	Isotropic	0.3	250000	No	Mohr-Coulomb	Plastic	0	40	40	0	40	40	0	Yes	None	Drained	2.2e+06	No	Ru	0

Bolt Name	Bolt Color	Bolt Type	Bolt Diameter (mm)	Bolt Modulus, E (kPa)	Bolt Model	Out-of-Plane Spacing (m)	Bond Shear Stiffness (kN/m/m)	Joint Shear	Pre-Tensioning Force (kN)	Constant Pre-tensioning Force in Install Stage	Face Plates	Bond Length (m)
Sidro 3x6"/251kN_1		Tieback	26.39	1.95e+08	Yes	3	100000	No	251	Yes	Attached	6

Liner Name	Liner Color	Liner Type	Beam Element Formulation	Axial Strain (+Expansion)	Young's Modulus (kPa)	Poisson's Ratio	Material Type	Method	Area (m ²)	Moment of Inertia (m ⁴)
PILOT fi50/1.0m		Standard Beam	Timoshenko	0	3e+07	0.2	Elastic	Area	0.196	0.00306641

Slika 1: Vhodni podatki o karakteristikah materialov in podpornih ukrepov za PS-1

2.1.2.3 Splošno

Po delu severnega oboda gradbene jame, je predvidena enkrat sidrana pilotna stena. Piloti so premera $\Phi 50\text{cm}$ in bodo uvrtni vsaj $0,9\text{m}$ pod dno gradbene jame oziroma $1,25\text{m}$ v matično hribinsko podlago. Povezuje jih AB sidrna vezna greda dimenzije $b/h = 70 \times 70\text{cm}$.

Za potrebe dimenzioniranja smo izvedli stabilnostne analize, ki so prikazane v nadaljevanju.

2.1.2.4 Rezultati stabilnostnega preračuna in dimenzioniranje v osi pilotne stene PS-1

SIDRANA PILOTNA STENA PS-1

$$\begin{aligned} M_{sd1} &= 1,35 \times M'_{sd1} \times L & P_{d1} &= 1,35 \times P'_{d1} \times 1,0 \\ M_{sd2} &= 1,00 \times M'_{sd2} \times L & P_{d2} &= 1,00 \times P'_{d2} \times 1,0 \\ M_{sd} &= \max(M_{sd1}, M_{sd2}) & P_d &= \max(P'_{d1}, P'_{d2}, P_0) \end{aligned}$$

PILOT

L=	1.00 m	osni razmek med piloti
M_{sd1} =	69.50 kNm/m	moment v pilotu pri $F=1,00$
M_{sd2} =	69.70 kNm/m	moment v pilotu pri $F=1,25$
M_{sd}=	93.8 kNm	računski moment v pilotu

1. VRSTA SIDRO (geotehnično sidro $3 \times 6''/251\text{kN}$)

L=	3.20 m	osni razmek med sidri
P_p =	251.00 kN	sila prednapetja
P_{d1} =	278.80 kN	sila v sidru pri $F=1,00$
P_{d2} =	278.80 kN	sila v sidru $F=1,25$
P_d=	376.4 kN	računska sila v sidru

1. SIDRNA VEZNA GRED

L=	3.00 m	osni razmek med sidri
α =	35 °	kot sidra
P_d =	376 kN	računska sila v sidru
$P_{d,h}$ =	308 kN	računska horizontalna sila
$P_{d,v}$ =	216 kN	računska vertikalna sila
$M_{sd,y}$=	231 kNm	računski moment,y
$M_{sd,z}$=	162 kNm	računski moment,z
$V_{sd,z}$=	154 kN	računska prečna sila, z
$V_{sd,y}$=	108 kN	računska prečna sila,y

2.1.2.5 Določitev konstrukcijskih elementov

Izračun konstrukcijskih elementov podpornih objektov (piloti + greda) je bil izveden v programu betonexpress. Rezultate izračunov prilagamo v prilogah. V nadaljevanju prilagamo povzetke izračunov in dimenzioniranje elementov.

a Piloti

- Piloti: $\phi 50\text{cm}/1,0\text{m}$
- Jeklo: B500
- Beton in prerez : C30/37
- Stremena: $\phi 12/15\text{cm}$
- Vzdolžna armatura: $8\phi 16$
- Zaščitni sloj: 8cm

b Sidrna vezna greda

Vezna greda povezuje Benotto pilote. Je kvadratnega prereza, njene dimenzije so $b \times h = 70/70\text{cm}$. Armatura vezne grede je rebrasta S500. Vezna greda je iz armiranega betona C30/37.

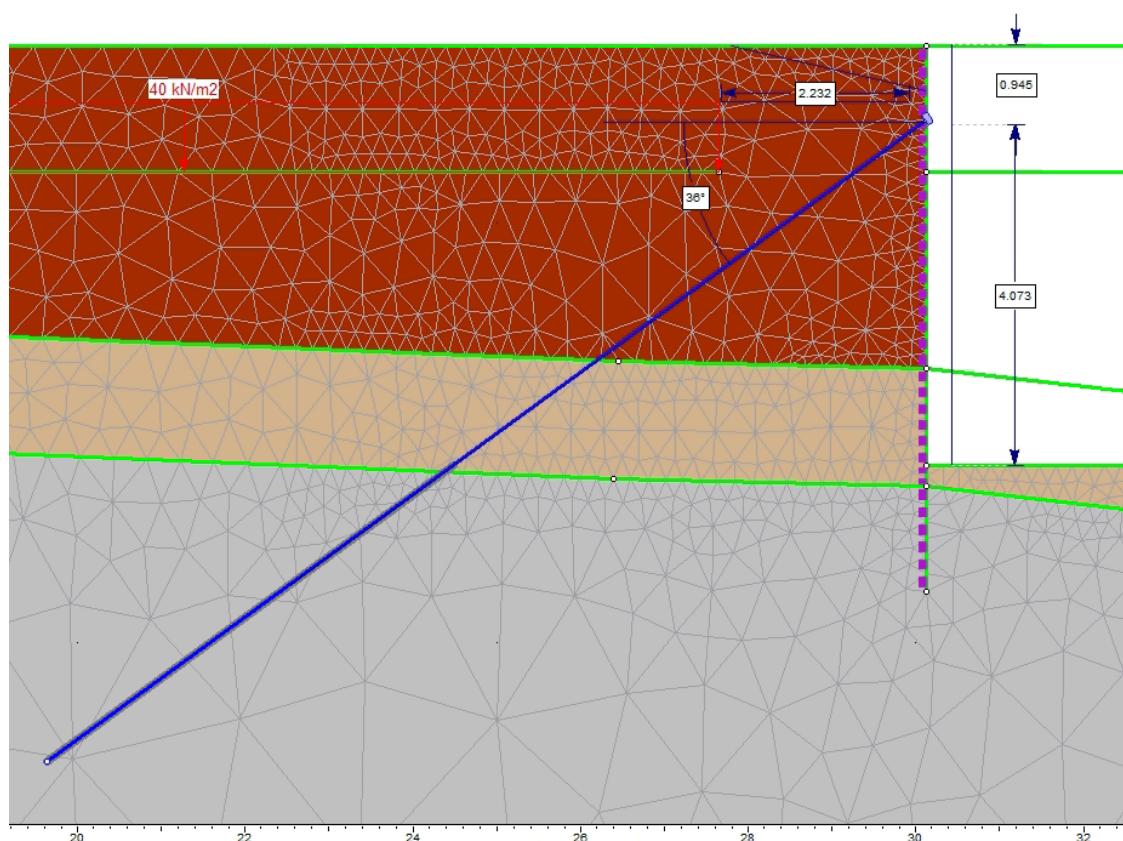
- Jeklo: S500
- Beton: C30/37
- Stremena: $\phi 12/15\text{cm}$
- Vzdolžna armatura: $20 \phi 14$
- Zaščitni sloj: 5cm

c Sidra

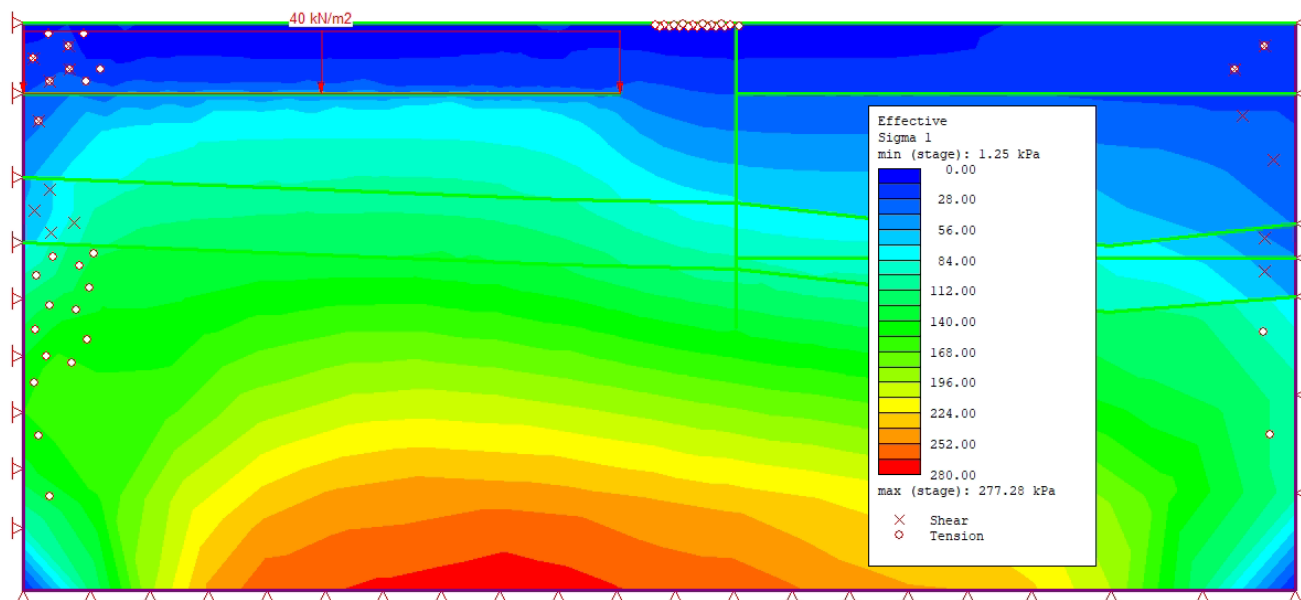
vezna dolžina za sidra v hribinah (po avtorju Littlejohn-u):	
$q_u = 40000 \text{ kPa}$	$\tau_u = (0,1 * q_u); \tau_u < 4,2 \text{ MPa}$ $F_d = \frac{0,5 \times \tau_u \times (\pi \times D \times L_{bond})}{F_s}$
$D_{vrine} = 91 \text{ mm}$	
$\tau_u = 4000 \text{ kPa}$	
$L_{bond} = 6.0 \text{ m}$	
$F_s = 1.4$	
$F_d = 2450 \text{ kN}$	

- Sidra: geotehnična sidra 3x6"
- Nosilnost sidra: $P_{min} = 586 \text{ kN}$
- Sila prednapetja: $P_0 = 251 \text{ kN}$
- Dolžina vpetja: $L = 6 \text{ m}$

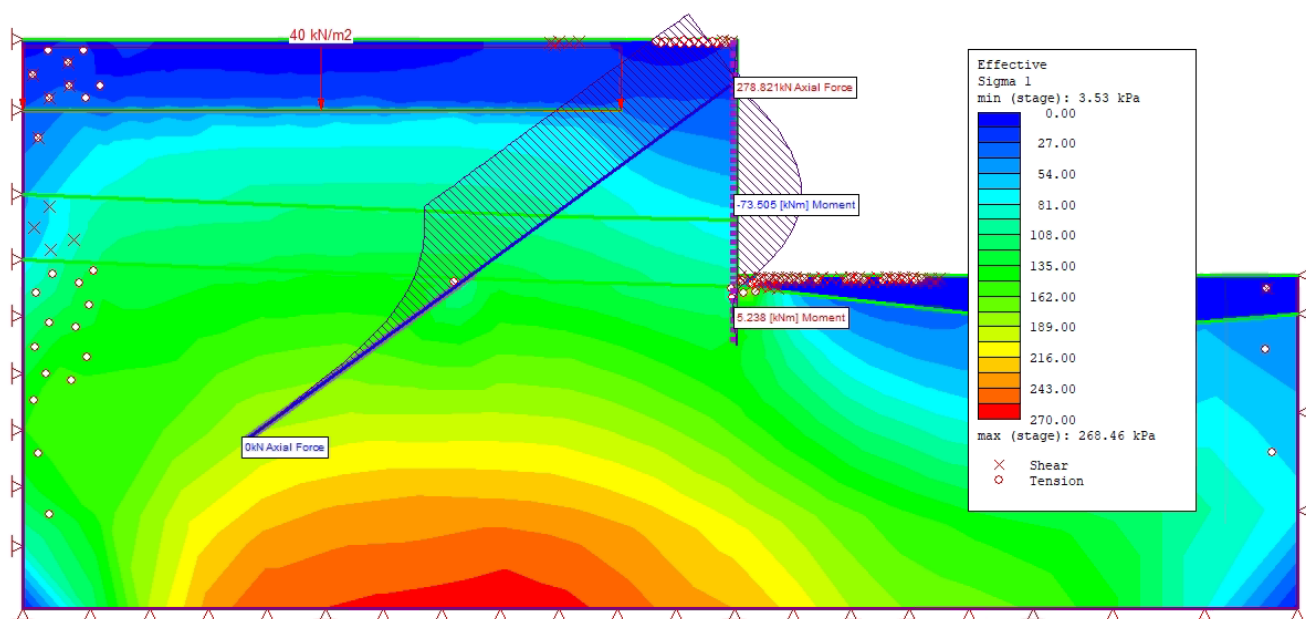
2.1.2.6 Izpis stabilnostnega preračuna v osi pilotne stene PS-1



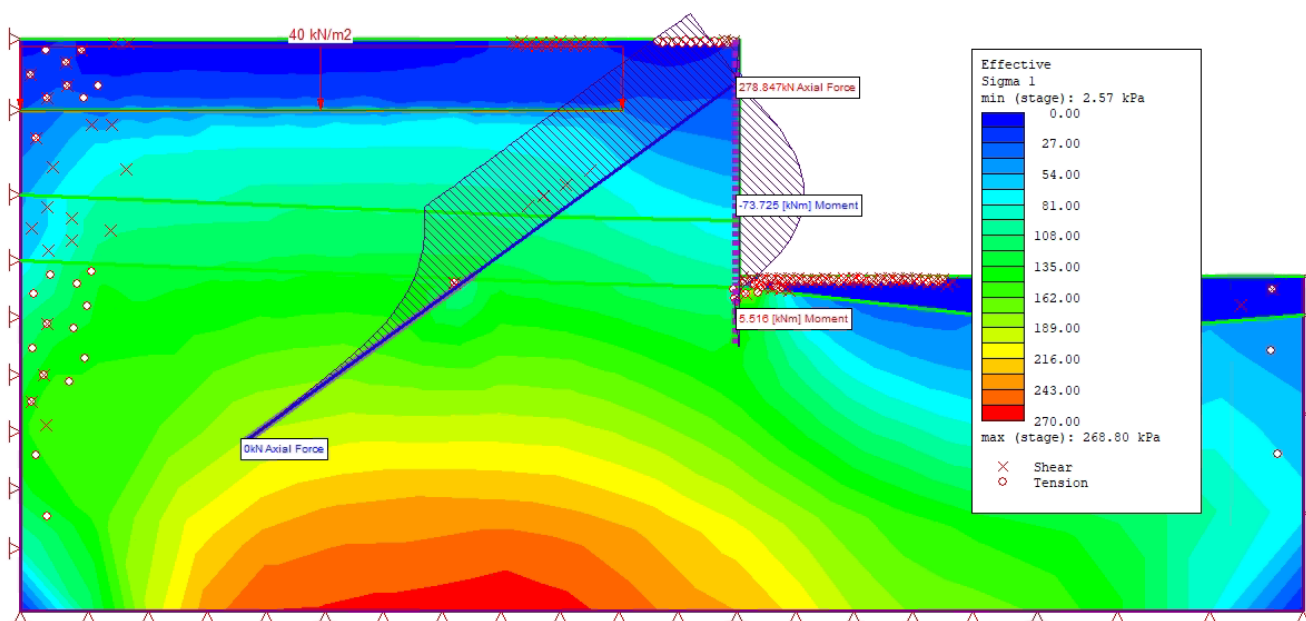
Slika 2: Prikaz računskega modela za končno stanje konstrukcije



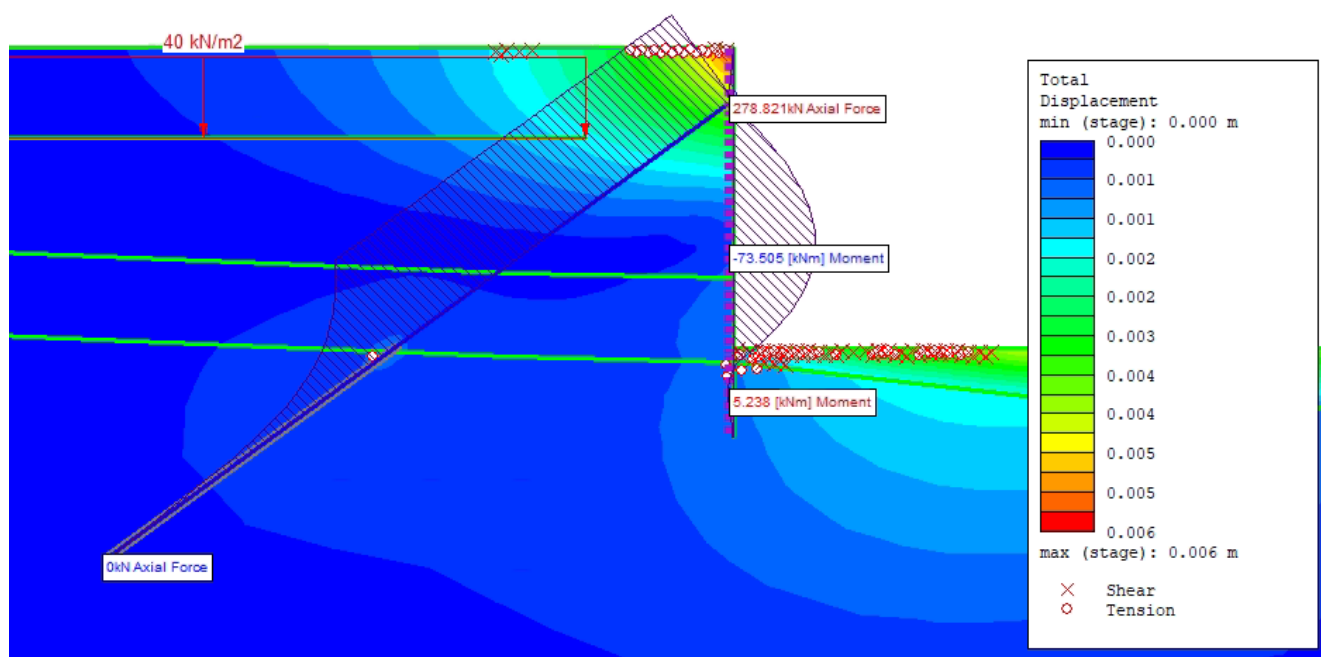
Slika 3: Začetno stanje - glavne napetosti



Slika 4: Izkop do dna gradbene jame - glavne napetosti, momenti v pilotu in osne sile v sidrih pri SRF=1,00



Slika 5: Izkop do dna gradbene jame - glavne napetosti, momenti v pilotu in osne sile v sidrih pri SRF=1,25



Slika 6: Premiki konstrukcije pri SRF=1,0

2.1.3 Pilotna stena PS-2

2.1.3.1 Faznost gradnje




Gradnjo sidrane pilotne stene smo zaradi zahtevnosti izvedbe modelirali v več bistvenih fazah:



6. Generalni izkop obstoječega terena do nivoja platoja za pilotiranje,
7. Izvedba pilotov in veznih sidrnih gred,
8. Vgradnja in napenjanje pasivnih sider,
9. Izvedba izkopa do dna gradbene jame,
10. Redukcija strižnih karakteristik, $F=1,25$.



V prilogah so prikazani rezultati stabilnostnih analiz za fazo 5.

2.1.3.2 Vhodni podatki in robni pogoji

V analizi smo uporabili materiale s karakteristikami, ki so bile povzete po geološko-geomehanskem poročilu v sklopu projektne dokumentacije in na podlagi povratnih analiz v nadaljevanju. Karakteristike materialov in konstrukcijskih elementov, uporabljenih v analizi, so prikazane v preglednici 1:

Material Name	Material Color	Initial Element Loading	Account for Moisture Content in Unit Weight	Unit Weight (kN/m ³)	Porosity Value	Elastic Type	Poisson's Ratio	Young's Modulus (kPa)	Use Residual Young's Modulus	Failure Criterion	Material Type	Peak Tensile Strength (kPa)	Peak Friction Angle (degrees)	Peak Cohesion (kPa)	Residual Tensile Strength (kPa)	Residual Friction Angle (degrees)	Residual Cohesion (kPa)	Dilation Angle (degrees)	Apply SSR (Shear Strength Reduction)	Unsaturated Behavior	Material Behaviour	Fluid Bulk Modulus (kPa)	Use Biot's Coefficient for calculating Effective Stress	Static Water Mode	Ru Value
Meljasta glina z občasnim gručcem (CL)		Field Stress and Body Force	No	17.5	0.5	Isotropic	0.3	6000	No	Mohr-Coulomb	Plastic	0	23	5	0	23	5	0	Yes	None	Drained	2.2e+06	No	Ru	0
Glinast gruč		Field Stress and Body Force	No	19.5	0.5	Isotropic	0.3	20000	No	Mohr-Coulomb	Plastic	0	33	0.1	0	33	0.1	0	Yes	None	Drained	2.2e+06	No	Ru	0
Apnenec		Field Stress and Body Force	No	25	0.5	Isotropic	0.3	250000	No	Mohr-Coulomb	Plastic	0	40	40	0	40	40	0	Yes	None	Drained	2.2e+06	No	Ru	0

Bolt Name	Bolt Color	Bolt Type	Bolt Diameter (mm)	Bolt Modulus, E (kPa)	Tensile Capacity (kN)	Residual Tensile Capacity (kN)	Out-of-Plane Spacing (m)	Pre-Tensioning Force (kN)	Constant Pre-tensioning Force in Install Stage
IBO R32/250 spodnji		End Anchored	51.54	1.6e+08	1200	1200	0.8	10	Yes
IBO R32/250		End Anchored	51.54	1.6e+08	1200	1200	3.2	50	Yes

Liner Name	Liner Color	Liner Type	Beam Element Formulation	Axial Strain (+Expansion)	Young's Modulus (kPa)	Poisson's Ratio	Material Type	Method	Area (m ²)	Moment of Inertia (m ⁴)	Unit Weight (kN/m ³)
PILOT f150/1.0m		Standard Beam	Timoshenko	0	3e+07	0.2	Elastic	Area	0.196	0.00306641	
Pilot 40/0,8m		Standard Beam	Timoshenko	0	3e+07	0.2	Elastic	Area	0.157	0.00157	24

Slika 7: Vhodni podatki o karakteristikah materialov in podpornih ukrepov za PS-2

2.1.3.3 Splošno

Po delu severnega oboda gradbene jame, je predvidena enkrat sidrana pilotna stena. Piloti so premera $\Phi 40\text{cm}$ in bodo uvrtni 1,00m v matično hribinsko podlago oz. bodo generalno nad dnom gradbene jame. Povezuje jih AB sidrna vezna greda dimenzije $b/h = 50 \times 50\text{cm}$, pete pilotov se stabilizira s pasivnimi IBO R23/250 sidri dolžine 6,0m

Za potrebe dimenzioniranja smo izvedli stabilnostne analize, ki so prikazane v nadaljevanju.

2.1.3.4 Rezultati stabilnostnega preračuna in dimenzioniranje v osi pilotne stene PS-2

SIDRANA PILOTNA STENA PS-2

$$\begin{aligned} M_{sd1} &= 1,35 \times M'_{sd1} \times L & P_{d1} &= 1,35 \times P'_{d1} \times 1,0 \\ M_{sd2} &= 1,00 \times M'_{sd2} \times L & P_{d2} &= 1,00 \times P'_{d2} \times 1,0 \\ M_{sd} &= \max(M_{sd1}, M_{sd2}) & P_d &= \max(P'_{d1}, P'_{d2}, P_0) \end{aligned}$$

PILOT

L=	0.80 m	osni razmek med piloti
M_{sd1} =	20.50 kNm/m	moment v pilotu pri $F=1,00$
M_{sd2} =	17.20 kNm/m	moment v pilotu pri $F=1,25$
M_{sd}=	22.1 kNm	računski moment v pilotu

1. VRSTA SIDRO (IBO pasivno sidro fi32/250)

L=	3.20 m	osni razmek med sidri
P_p =	50.00 kN	sila prednapetja
P_{d1} =	92.00 kN	sila v sidru pri $F=1,00$
P_{d2} =	89.00 kN	sila v sidru $F=1,25$
P_d=	124.2 kN	računska sila v sidru

1. SIDRNA VEZNA GREDA

L=	3.20 m	osni razmek med sidri
α =	35 °	kot sidra
P_d =	124 kN	računska sila v sidru
$P_{d,h}$ =	102 kN	računska horizontalna sila
$P_{d,v}$ =	71 kN	računska vertikalna sila
$M_{sd,y}$=	81 kNm	računski moment,y
$M_{sd,z}$=	57 kNm	računski moment,z
$V_{sd,z}$=	51 kN	računska prečna sila, z
$V_{sd,y}$=	36 kN	računska prečna sila,y

2.1.3.5 Določitev konstrukcijskih elementov

Izračun konstrukcijskih elementov podpornih objektov (piloti + greda) je bil izveden v programu betonexpress. Rezultate izračunov prilagamo v prilogah. V nadaljevanju prilagamo povzetke izračunov in dimenzioniranje elementov.

a Piloti

- Piloti: $\phi 40\text{cm}/0,80\text{m}$
- Jeklo: B500
- Beton in prerez : C30/37
- Stremena: $\phi 12/15\text{cm}$
- Vz dolžna armatura: $8\phi 16$
- Zaščitni sloj: 8cm

b Sidrna vezna greda

Vezna greda povezuje Benotto pilote. Je kvadratnega prereza, njene dimenzije so $b \times h = 70/70\text{cm}$. Armatura vezne grede je rebrasta S500. Vezna greda je iz armiranega betona C25/30.

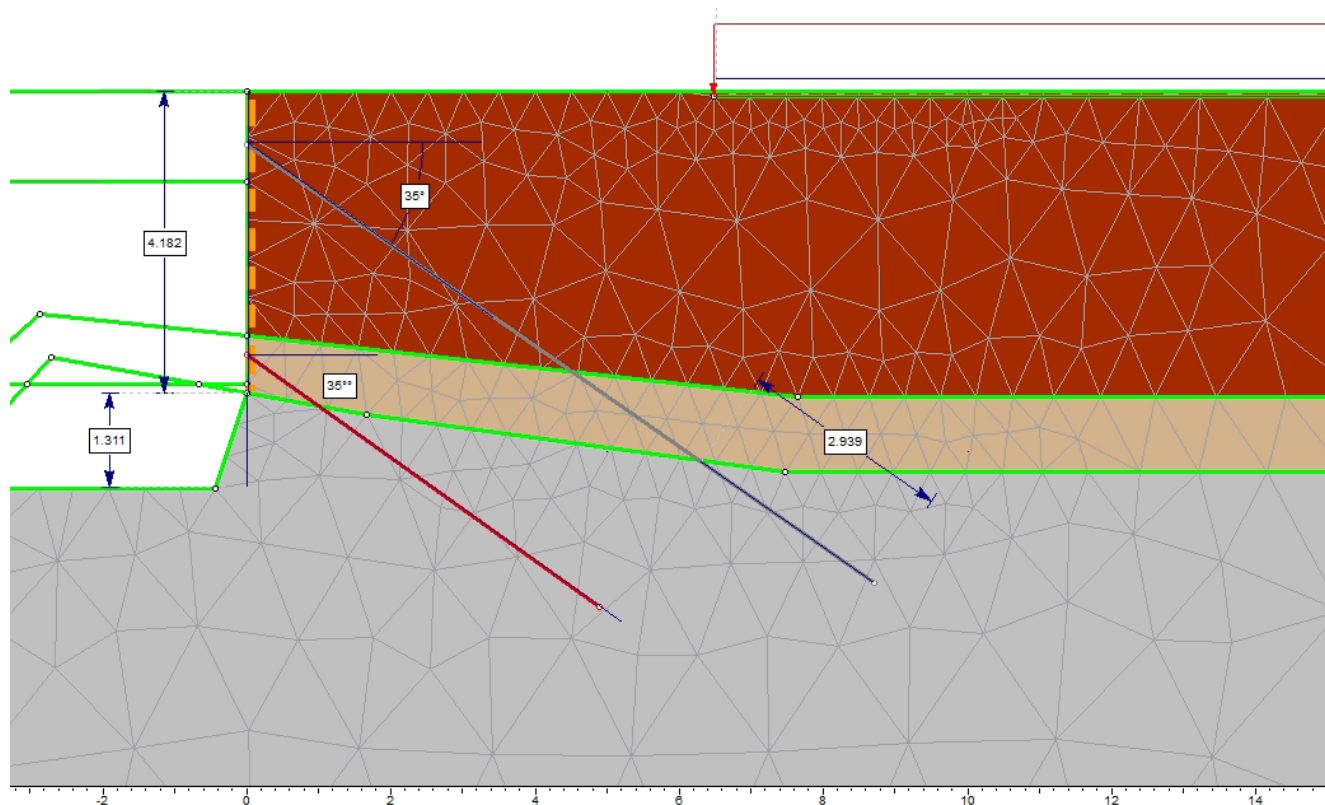
- Jeklo: S500
- Beton: C25/30
- Stremena: $\phi 12/15\text{cm}$
- Vz dolžna armatura: $12 \phi 14$
- Zaščitni sloj: 5cm

c Sidra

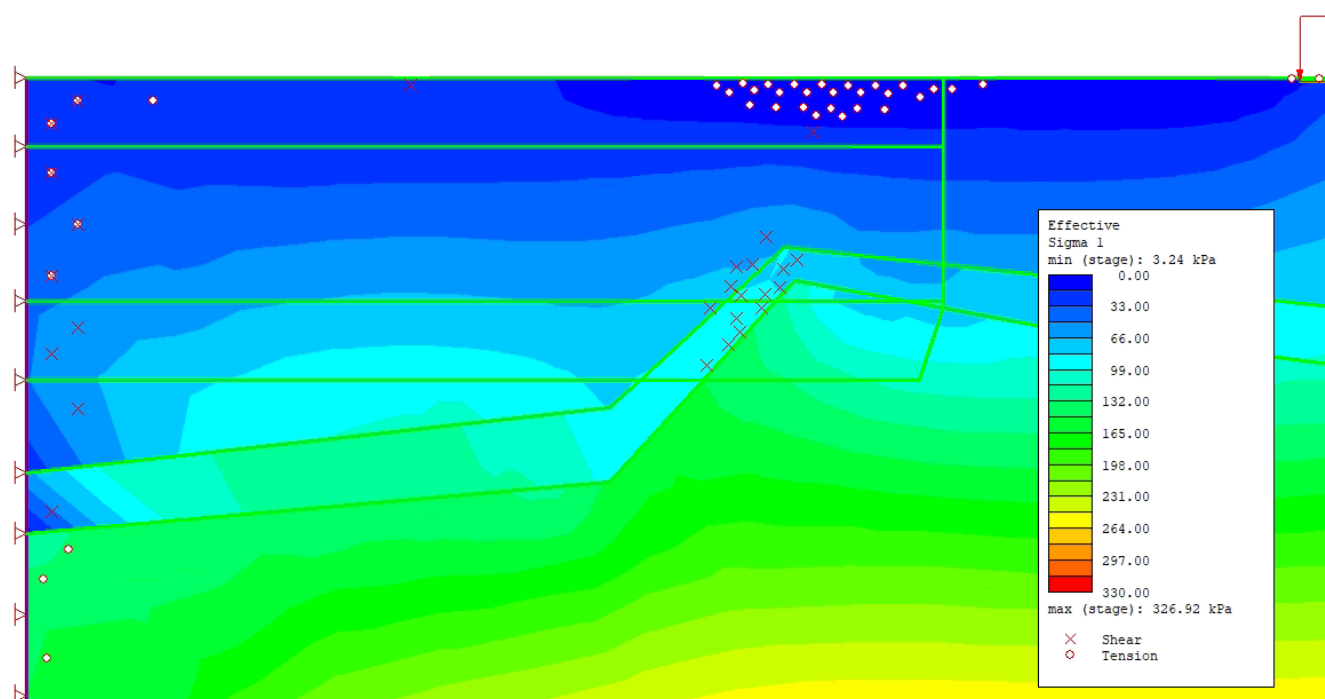
vezna dolžina za sidra v hribinah (po avtorju Littlejohn-u):	
$q_u = 40000 \text{ kPa}$	$\tau_u = (0,1 * q_u); \tau_u < 4,2 \text{ MPa}$ $F_d = \frac{0,5 \times \tau_u \times (\pi \times D \times L_{bond})}{F_s}$
$D_{vrine} = 91 \text{ mm}$	
$\tau_u = 4000 \text{ kPa}$	
$L_{bond} = 2.0 \text{ m}$	
$F_s = 1.4$	
$F_d = 817 \text{ kN}$	

- Sidra: pasivna IBO sidra R23/250
- Nosilnost sidra: $P_{min} = 250 \text{ kN}$
- Sila prednapetja: $P_0 = 251 \text{ kN}$
- Dolžina veznega dela v hribini: $L = 2,0 - 3,0 \text{ m}$

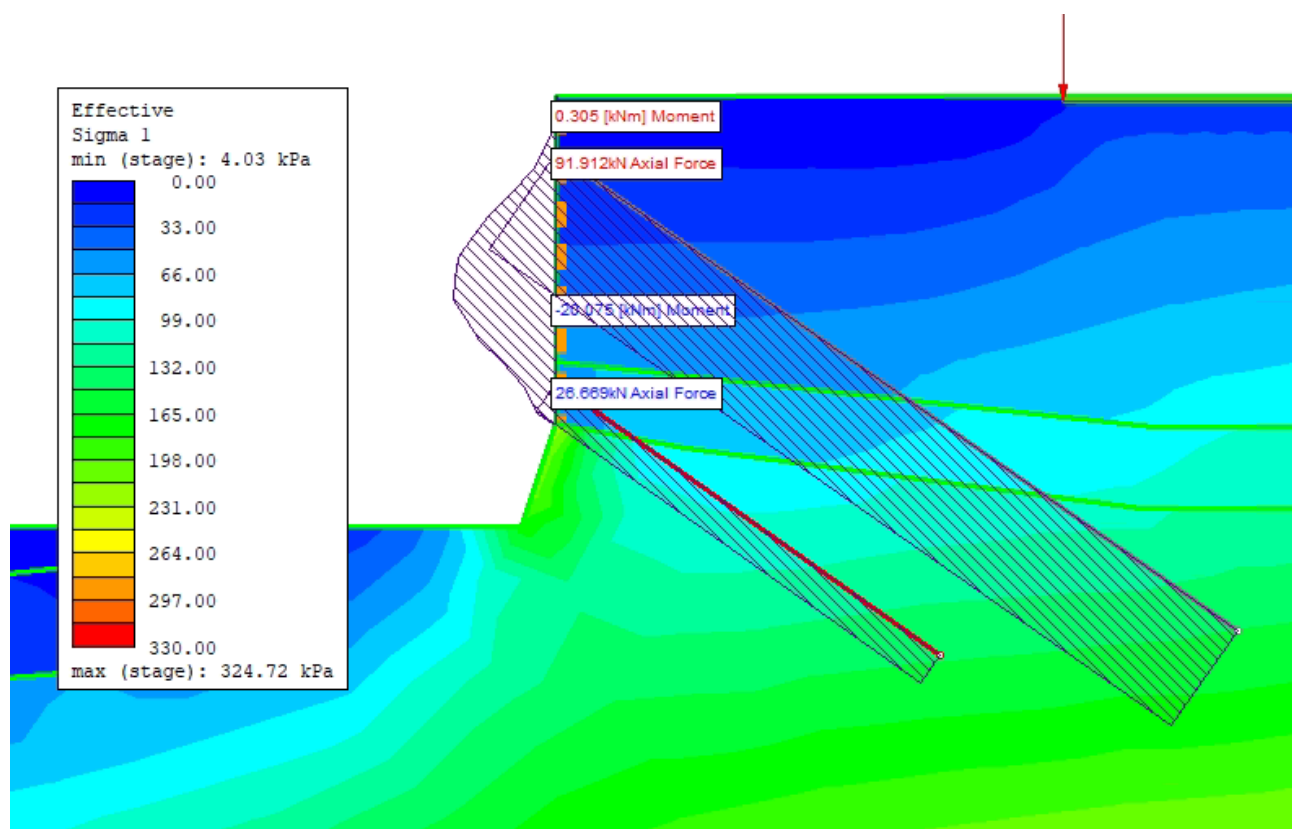
2.1.3.6 Izpis stabilnostnega preračuna v osi pilotne stene PS-1



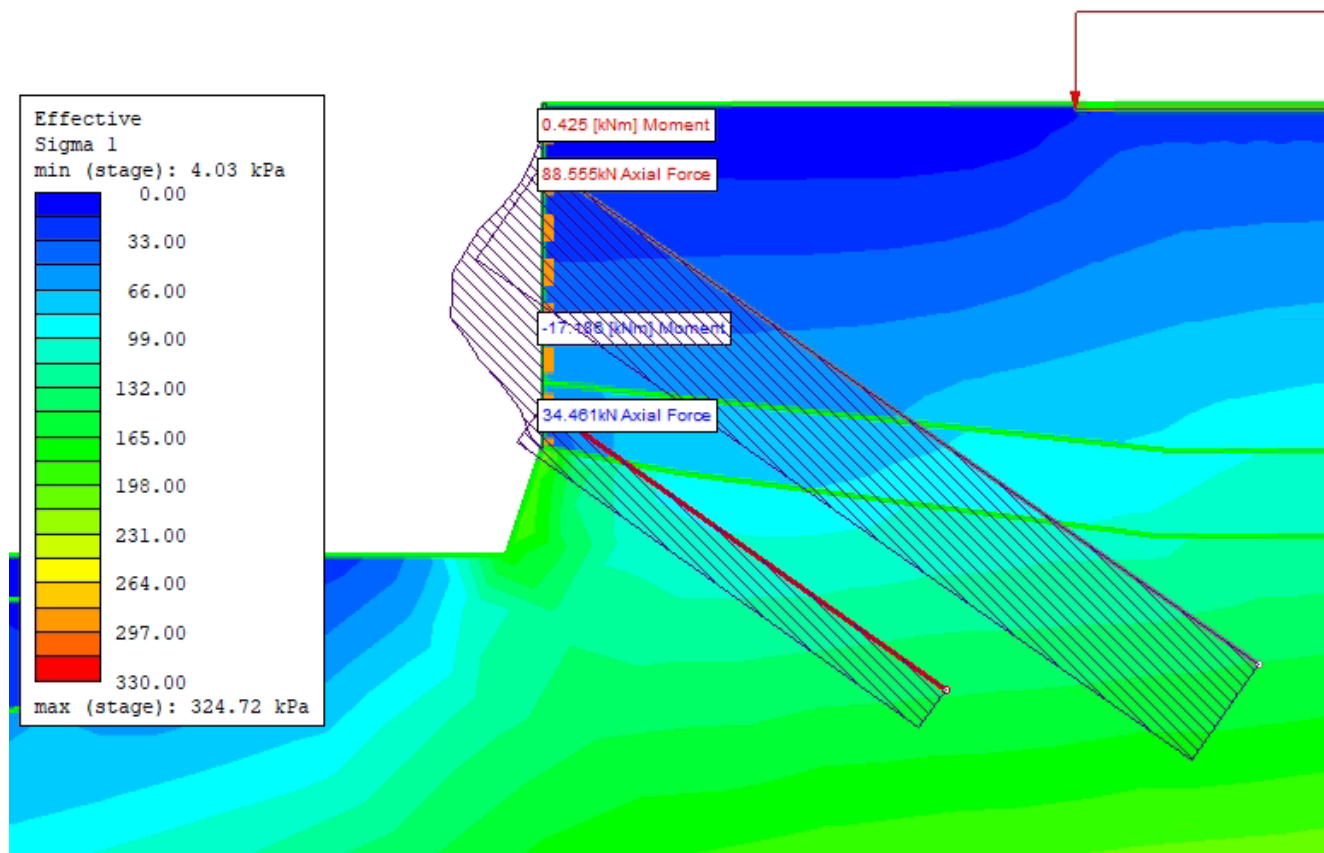
Slika 8: Prikaz računskega modela za končno stanje konstrukcije



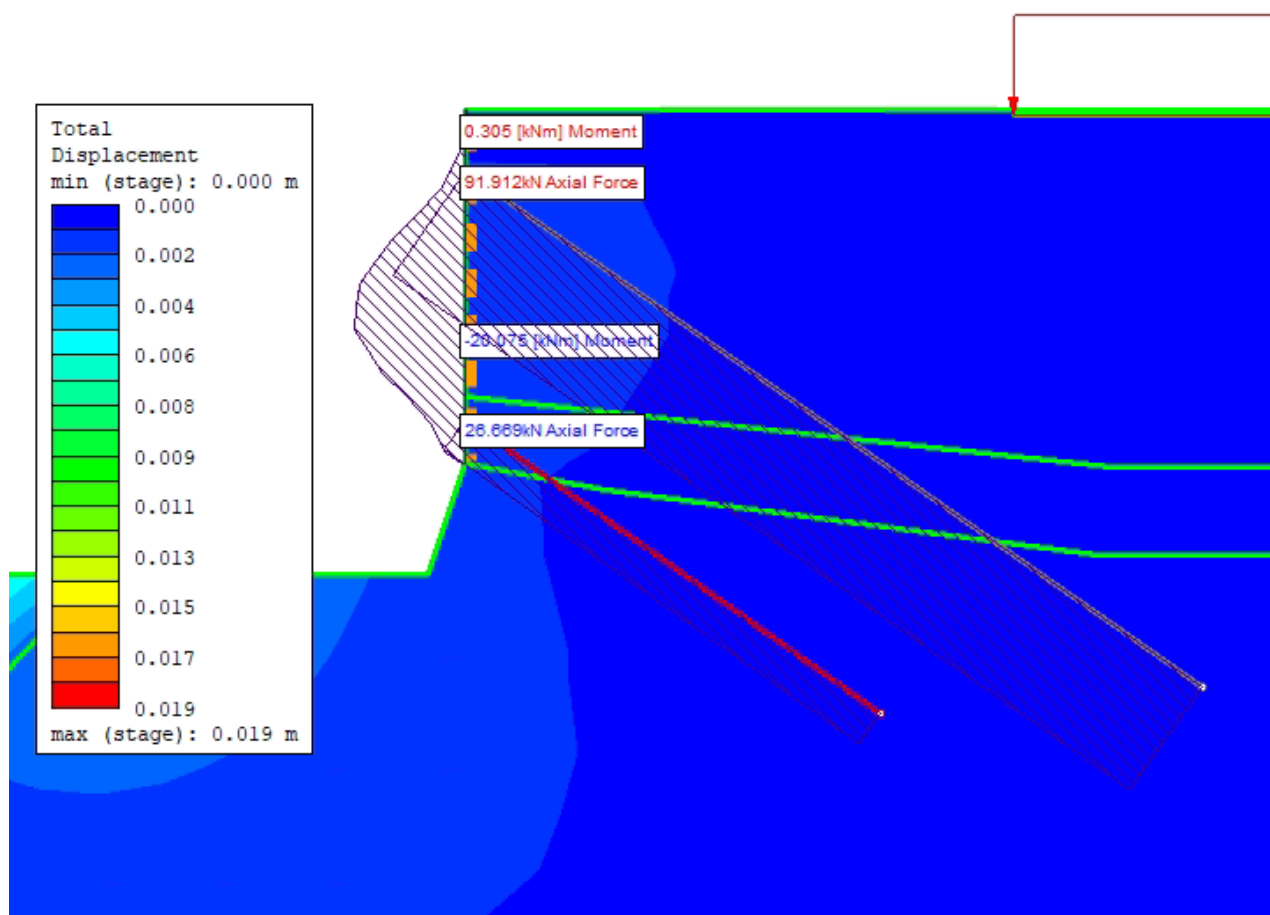
Slika 9: Začetno stanje - glavne napetosti



Slika 10: Izkop do dna gradbene jame - glavne napetosti, momenti v pilotu in osne sile v sidrih pri SRF=1,00



Slika 11: Izkop do dna gradbene jame - glavne napetosti, momenti v pilotu in osne sile v sidrih pri SRF=1,25



Slika 12: Premiki konstrukcije pri SRF=1,0

2.1.4 Berlinska stena

Berlinska stena se izvede iz vtisnjenih železniških tirnic S49, prostor med njimi se zapolni z lesenimi plohi. Predvidena je uporaba tirnic dolžine 4,0m. Tirnice so vtisnjene v plast plikvartarne gline. Največja dolžina vrhnjega konzolnega dela berlinske stene znaša 0,5m.

DIMENZIONIRANJE VERTIKAL BERLINSKE STENE				S49	
M [kNm]	F	W [cm ³]	D [m]	f _d [kN/cm ²]	f _y [kN/cm ²]
3.6	1.35	252.1	0.75	1.4	23.5

$$f_d = \frac{M \times F \times D}{W}$$

$$f_d < f_y$$

M	momentna obremenitev profila
F	varnostni faktor
W	odpornostni moment prereza
D	razmak med profili
f _d	računska napetost v profilu
f _y	natezna trdnost jekla

- Nosilne vertikale : S49; D = 0,75m
- Polnilo: Leseni plohi; d=5cm

2.2 PROJEKTANTSKI POPIS DEL S PREDRAČUNOM

POPIS DEL

Zap. št	Opis	EM	kol.	cena/enota	znesek EUR
PREDELA					
Geodetska dela					
1	Zakoličenje pilotov in grede ter zavarovanje profilov	pav	1.00		0.00
2	Zakoličenje delovnega platoja, gradbene jame ter točk za široki izkop in zavarovanje profilov	pav	1.00		0.00
Ostala preddela					
4	Organizacija gradbišča za potrebe geomehanskih del - postavitve začasnih objektov, zavarovanje gradbišča, varovanje gr. jame, varovanje pred prahom in hrupom,...	pav	1.00		0.00
Transporti					
5	Transport geomehanske mehanizacije (vrtalna garnitura za izdelavo pilotov fi50) na gradbišče ter odvoz z gradbišča	pav	1.00		0.00
6	Transport geomehanske mehanizacije (vrtalna garnitura za izdelavo pilotov fi40 in sider) na gradbišče ter odvoz z gradbišča	pav	1.00		0.00
PREDELA SKUPAJ					0.00
ZEMELJSKA DELA					
Izkopi					
7	Izkop vezljivih zemljin do 3. kategorije z vgradnjo v nasip, brez transporta. (ureditev delovnega platoja)	m3	0.00		0.00
8	Izkop vezljive zemljine - 3. kategorije skupaj z nakladanjem (poglobitev za vezno gredo)	m3	370.00		0.00
9	Širok izkop vezljive zemljine - 3. kategorije skupaj z nakladanjem (generalni izkop gradbene jame)	m3	6,230.00		0.00
10	Širok izkop kompaktne hribine - 5. kategorije skupaj z nakladanjem (generalni izkop gradbene jame)	m3	1,660.00		0.00
11	Ureditev planuma temeljnih tal mehke zemljine - 2. in 3. kategorije (pilota greda)	m2	60.00		0.00
Nasipi, zasipi					
12	Izdelava nasutja za objektom iz zrnate kamnine 0/128 z dobavo iz kamnoloma.	m3	1,650.00		0.00
Koli in vodnjaki					
13	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 40 cm, izkop v glinah in gruščih. Dolžina pilotov je 2,5 m do 4 m. Število pilotov je 41.	m1	120.00		0.00
14	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 40 cm, izkop v apnencih do globine 2,5m v podlago. Dolžina pilotov je 2,5 m do 4 m. Število pilotov je 41.	m1	41.00		0.00
15	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 50 cm, izkop v glinah in gruščih. Dolžina pilotov je 4 m do 6.3 m. Število pilotov je 25.	m1	108.00		0.00
16	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 50 cm, izkop v apnencih do globine 2,0m v podlago. Dolžina pilotov je 4 m do 6.3 m. Število pilotov je 25.	m1	31.00		0.00
17	Odbijanje glav pilota premera 50cm z nakladanjem in odvozom porušenega betona (odbije se zgornjih 50cm pilota oz. do kote za izvedbo grede).	kos	25.00		0.00

POPIS DEL

Zap. št	Opis	EM	kol.	cena/enoto	znesek EUR
18	Odbijanje glav pilota premera 40cm z nakladanjem in odvozom porušenega betona (odbije se zgornjih 50cm pilota oz. do kote za izvedbo grede).	kos	41.00		0.00
19	Ročni izkop okoli pilotov v zemljini 2. in 3 kategorije	m3	15.00		0.00
	Berlinska stena				
21	Dobava, vgraditev in vzdrževanje jeklene zagatne stene tirnice: S49, L=4.0m na razmaku 0,75m	50.00	m ²		0.00

POPIS DEL

Zap. št	Opis	EM	kol.	cena/enota	znesek EUR
Prevozi, razprostiranje in ureditev deponij materiala					
22	Odvoz zemljin do 3. ktg na trajno deponijo, vključno z deponiranjem. Postavka vključuje vse stroške, povezane z deponiranjem materiala (upoštevani vsi upravni postopki, raziskave materialov, ipd..)	m3	6,600.00		0.00
23	Odvoz hribin 4., 5. in 6. ktg v predelavo (drobljenje) ali na trajno deponijo. Postavka vključuje vse stroške, povezane z deponiranjem ali drobljenjem materiala (upoštevani vsi upravni postopki, raziskave materialov, ipd..)	m3	1,660.00		0.00
Sidranje					
24	Dobava, vrtanje, vgraditev in injektiranje trajnega pasivnega sidra IBO R32-250 dolžine L=6-12m; (v ceni postavke upoštevati vse stroške in material za izvedbo sidra vključno z dobavo in vgradnjo sidrišč, protikorozijsko zaščito, ustrezni certifikati, napenjalni preizkusi...)	m1	366.00		0.00
25	Dobava, vrtanje, vgraditev in injektiranje 7 kos začasnih geotehničnih sider 3x0,62" vezne dolžine Lv=6m, Lp=6-9m; upoštevati še dolžino potrebno za napenjanje (v ceni postavke upoštevati vse stroške in material za izvedbo sidra vključno z dobavo, vgradnjo, ustreznimi certifikati...)	m1	431.25		0.00
26	Dobava in vgradnja dodatne injekcijske mase za sidra IBO R32-250, ki presegajo osnovno porabo. Za meter sidra pri Wc=0,45 poraba cementa znaša 25 kg. OCENA	kg	1,500.00		0.00
27	Dobava in vgraditev rezervnih sidrišč, z brezšivno cevjo in naležno jekleno ploščo	kos	21.00		0.00
ZEMELJSKA DELA SKUPAJ					0.00
GRADBENA IN OBRTNIŠKA DELA					
Dela s cementnim betonom					
28	Dobava in vgraditev podložnega cementnega betona C12/16 v prerez do 0,15 m3/m2	m3	60.00		0.00
29	Dobava in vgraditev ojačenega cementnega betona C30/37 XC4; XD3; PVII v prerez od 0,31 do 0,50 m3/m1 - AB greda 50/50 in 70/70cm.	m3	21.10		0.00
30	Dobava in vgraditev cementnega betona C30/37 XC4; XD3; PVII, v prerez od 0,11 do 0,20 m3/m1 - piloti fi 40cm in fi 50cm.	m3	47.37		0.00
31	Izravnavna ostenja gradbene jame z vidnim cementnim betonom C25/30, XC4; XD3; VB-II, debeline 10-15cm, kompletno z mrežo Q196, pritrjeno z jeklenimi sidri v pilote oziroma zaledje izkopa. Upoštevan je preklon	m2	0.00		0.00

POPIS DEL

Zap. št	Opis	EM	kol.	cena/enota	znesek EUR
	Dela z jeklom za ojačitev				
32	Dobava in postavitve armaturnih košev iz rebrastih palic (fi 8-16mm) iz visokovrednega naravno trdega jekla B500 B za srednje zahtevno ojačitev (piloti fi40).	kg	4,777.13		0.00
33	Dobava in postavitve armaturnih košev iz rebrastih palic (fi 10-16mm) iz visokovrednega naravno trdega jekla B500 B za srednje zahtevno ojačitev (piloti fi50).	kg	4,386.88		0.00
34	Dobava in postavitve rebrastih palic iz visokovrednega naravno trdega jekla B500 B s premerom 14 mm in večjim, za srednje zahtevno ojačitev (AB grede)	kg	1,495.52		0.00
35	Dobava in postavitve rebrastih žic iz visokovrednega naravno trdega jekla B500 B s premerom do 12 mm, za srednje zahtevno ojačitev (AB grede)	kg	1,120.00		0.00
	Tesarska dela				
36	Izdelava dvostranskega vezanega opaža AB gred in zidov.	m2	72.00		0.00
	Dilatacija				
37	Izdelava dilatacije po detajlu v prilogi.	kos	13.00		0.00
	GRADBENA IN OBRTNIŠKA DELA SKUPAJ				0.00
	TUJE STORITVE				
	Preskusi, nadzor in tehnična dokumentacija				
38	Preizkus zveznosti pilota	kos	15.00		0.00
39	Dobava in vgraditev reperjev za monitoring objektov	kos	6.00		0.00
40	Izvedba začetne meritve reperjev.	kpl	1.00		0.00
41	Geomehanski nadzor	ur	100.00		0.00
42	Projektantski nadzor	ur	100.00		0.00
43	Izdelava projekta izvedenih del PID	kpl	1.00		0.00
	TUJE STORITVE SKUPAJ				0.00
	REKAPITULACIJA				
	PREDEDELA SKUPAJ:				0.00
	ZEMELJSKA DELA SKUPAJ				0.00
	GRADBENA IN OBRTNIŠKA DELA SKUPAJ				0.00
	TUJE STORITVE SKUPAJ				0.00
	NEPREDVIDENA DELA 10%				0.00
	SKUPAJ				0.00
	DDV 22%:				0.00
	SKUPAJ Z DDV:				0.00

PROJEKTANTSKI PREDRAČUN

Zap. št	Opis	EM	kol.	cena/enota	znesek EUR
PREDDELA					
Geodetska dela					
1	Zakoličenje pilotov in grede ter zavarovanje profilov	pav	1.00	500.00	500.00
2	Zakoličenje delovnega platoja, gradbene jame ter točk za široki izkop in zavarovanje profilov	pav	1.00	500.00	500.00
Ostala predдела					
4	Organizacija gradbišča za potrebe geomehanskih del - postavitve začasnih objektov, zavarovanje gradbišča, varovanje gr. jame, varovanje pred prahom in hrupom,...	pav	1.00	1,500.00	1,500.00
Transporti					
5	Transport geomehanske mehanizacije (vrtlana garnitura za izdelavo pilotov fi50) na gradbišče ter odvoz z gradbišča	pav	1.00	5,750.00	5,750.00
6	Transport geomehanske mehanizacije (vrtlana garnitura za izdelavo pilotov fi40 in sider) na gradbišče ter odvoz z gradbišča	pav	1.00	5,000.00	5,000.00
PREDDELA SKUPAJ					13,250.00
ZEMELJSKA DELA					
Izkopi					
7	Izkop vezljivih zemljin do 3. kategorije z vgradnjo v nasip, brez transporta. (ureditev delovnega platoja)	m3	0.00	6.50	0.00
8	Izkop vezljive zemljine - 3. kategorije skupaj z nakladanjem (poglobitev za vezno gredo)	m3	370.00	5.50	2,035.00
9	Širok izkop vezljive zemljine - 3. kategorije skupaj z nakladanjem (generalni izkop gradbene jame)	m3	6,230.00	4.50	28,035.00
10	Širok izkop kompaktne hribine - 5. kategorije skupaj z nakladanjem (generalni izkop gradbene jame)	m3	1,660.00	12.50	20,750.00
11	Ureditev planuma temeljnih tal mehke zemljine - 2. in 3. kategorije (pilotna greda)	m2	60.00	1.25	75.00
Nasipi, zasipi					
12	Izdelava nasutja za objektom iz zrnate kamnine 0/128 z dobavo iz kamnoloma.	m3	1,650.00	20.00	33,000.00
Koli in vodnjaki					
13	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 40 cm, izkop v glinah in gruščih. Dolžina pilotov je 2,5 m do 4 m. Število pilotov je 41.	m1	120.00	75.00	9,000.00
14	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 40 cm, izkop v apnencih do globine 2,5m v podlago. Dolžina pilotov je 2,5 m do 4 m. Število pilotov je 41.	m1	41.00	90.00	3,690.00
15	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 50 cm, izkop v glinah in gruščih. Dolžina pilotov je 4 m do 6.3 m. Število pilotov je 25.	m1	108.00	86.25	9,315.00
16	Vrtanje s cevovijo za izdelavo uvrtenih kolov iz ojačenega cementnega betona, sistema Benotto, premera 50 cm, izkop v apnencih do globine 2,0m v podlago. Dolžina pilotov je 4 m do 6.3 m. Število pilotov je 25.	m1	31.00	103.50	3,208.50
17	Odbijanje glav pilota premera 50cm z nakladanjem in odvozom porušenega betona (odbije se zgornjih 50cm pilota oz. do kote za izvedbo grede).	kos	25.00	28.75	718.75

PROJEKTANTSKI PREDRAČUN

Zap. št	Opis	EM	kol.	cena/enoto	znesek EUR
18	Odbijanje glav pilota premera 40cm z nakladanjem in odvozom porušenega betona (odbije se zgornjih 50cm pilota oz. do kote za izvedbo grede).	kos	41.00	25.00	1,025.00
19	Ročni izkop okoli pilotov v zemljini 2. in 3 kategorije	m3	15.00	65.00	975.00
	Berlinska stena				
21	Dobava, vgraditev in vzdrževanje jeklene zagatne stene				
	tirnice: S49, L=4.0m na razmaku 0,75m	50.00	m ²	70.00	3,500.00

PROJEKTANTSKI PREDRAČUN

Zap. št	Opis	EM	kol.	cena/enota	znesek EUR
Prevozi, razprostiranje in ureditev deponij materiala					
22	Odvoz zemljin do 3. ktg na trajno deponijo, vključno z deponiranjem. Postavka vključuje vse stroške, povezane z deponiranjem materiala (upoštevani vsi upravni postopki, raziskave materialov, ipd..)	m3	6,600.00	5.50	36,300.00
23	Odvoz hribin 4., 5. in 6. ktg v predelavo (drobljenje) ali na trajno deponijo. Postavka vključuje vse stroške, povezane z deponiranjem ali drobljenjem materiala (upoštevani vsi upravni postopki, raziskave materialov, ipd..)	m3	1,660.00	6.50	10,790.00
Sidranje					
24	Dobava, vrtanje, vgraditev in injektiranje trajnega pasivnega sidra IBO R32-250 dolžine L=6-12m; (v ceni postavke upoštevati vse stroške in material za izvedbo sidra vključno z dobavo in vgradnjo sidrišč, protikorozijsko zaščito, ustrezni certifikati, napenjalni preizkusi...)	m1	366.00	82.00	30,012.00
25	Dobava, vrtanje, vgraditev in injektiranje 7 kos začasnih geotehničnih sider 3x0,62" vezne dolžine Lv=6m, Lp=6-9m; upoštevati še dolžino potrebno za napenjanje (v ceni postavke upoštevati vse stroške in material za izvedbo sidra vključno z dobavo, vgradnjo, ustreznimi certifikati...)	m1	431.25	93.00	40,106.25
26	Dobava in vgradnja dodatne injekcijske mase za sidra IBO R32-250, ki presegajo osnovno porabo. Za meter sidra pri Wc=0,45 poraba cementa znaša 25 kg. OCENA	kg	1,500.00	0.25	375.00
27	Dobava in vgraditev rezervnih sidrišč, z brezšivno cevjo in naležno jekleno ploščo	kos	21.00	45.00	945.00
ZEMELJSKA DELA SKUPAJ					233,855.50
GRADBENA IN OBRTNIŠKA DELA					
Dela s cementnim betonom					
28	Dobava in vgraditev podložnega cementnega betona C12/16 v prerez do 0,15 m3/m2	m3	60.00	95.00	5,700.00
29	Dobava in vgraditev ojačenega cementnega betona C30/37 XC4; XD3; PVII v prerez od 0,31 do 0,50 m3/m1 - AB greda 50/50 in 70/70cm.	m3	21.10	105.00	2,215.50
30	Dobava in vgraditev cementnega betona C30/37 XC4; XD3; PVII, v prerez od 0,11 do 0,20 m3/m1 - piloti fi 40cm in fi 50cm.	m3	47.37	108.00	5,115.85
31	Izravnavna ostenja gradbene jame z vidnim cementnim betonom C25/30, XC4; XD3; VB-II, debeline 10-15cm, kompletno z mrežo Q196, pritrjeno z jeklenimi sidri v pilote oziroma zaledje izkopa. Upoštevan je preklap	m2	0.00	45.00	0.00

PROJEKTANTSKI PREDRAČUN

Zap. št	Opis	EM	kol.	cena/enoto	znesek EUR
	Dela z jeklom za ojačitev				
32	Dobava in postavitve armaturnih košev iz rebrastih palic (fi 8-16mm) iz visokovrednega naravno trdega jekla B500 B za srednje zahtevno ojačitev (piloti fi40).	kg	4,777.13	1.40	6,687.98
33	Dobava in postavitve armaturnih košev iz rebrastih palic (fi 10-16mm) iz visokovrednega naravno trdega jekla B500 B za srednje zahtevno ojačitev (piloti fi50).	kg	4,386.88	1.40	6,141.63
34	Dobava in postavitve rebrastih palic iz visokovrednega naravno trdega jekla B500 B s premerom 14 mm in večjim, za srednje zahtevno ojačitev (AB grede)	kg	1,495.52	1.40	2,093.73
35	Dobava in postavitve rebrastih žic iz visokovrednega naravno trdega jekla B500 B s premerom do 12 mm, za srednje zahtevno ojačitev (AB grede)	kg	1,120.00	1.40	1,568.00
	Tesarska dela				
36	Izdelava dvostranskega vezanega opaža AB gred in zidov.	m2	72.00	20.00	1,440.00
	Dilatacija				
37	Izdelava dilatacije po detajlu v prilogi.	kos	13.00	50.00	650.00
GRADBENA IN OBRTNIŠKA DELA SKUPAJ					31,612.69
TUJE STORITVE					
	Preskusi, nadzor in tehnična dokumentacija				
38	Preizkus zveznosti pilota	kos	15.00	35.00	525.00
39	Dobava in vgraditev reperjev za monitoring objektov	kos	6.00	85.00	510.00
40	Izvedba začetne meritve reperjev.	kpl	1.00	300.00	300.00
41	Geomehanski nadzor	ur	100.00	55.00	5,500.00
42	Projektantski nadzor	ur	100.00	55.00	5,500.00
43	Izdelava projekta izvedenih del PID	kpl	1.00	4,000.00	4,000.00
TUJE STORITVE SKUPAJ					16,335.00
REKAPITULACIJA					
PREDEDELA SKUPAJ:					13,250.00
ZEMELJSKA DELA SKUPAJ					233,855.50
GRADBENA IN OBRTNIŠKA DELA SKUPAJ					31,612.69
TUJE STORITVE SKUPAJ					16,335.00
NEPREDVIDENA DELA 10%					29,505.32
SKUPAJ					324,558.51
DDV 22%:					71,402.87
SKUPAJ Z DDV:					395,961.39

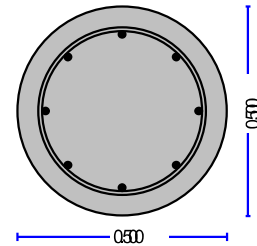
2.3 STATIČNI IZRAČUN PILOTOV IN GRED

Šolski center NM 27/05/2024**1. Piloti fi50/1,0m PS-1****Isolated column single bending**

(EC2 EN1992-1-1:2004, EC0 EN1990:2002,)

D =0.500 m, Ned = 50.00 kN**Med yy = 95.00 kNm, Med zz = 0.00 kNm****Reinforced concrete design**

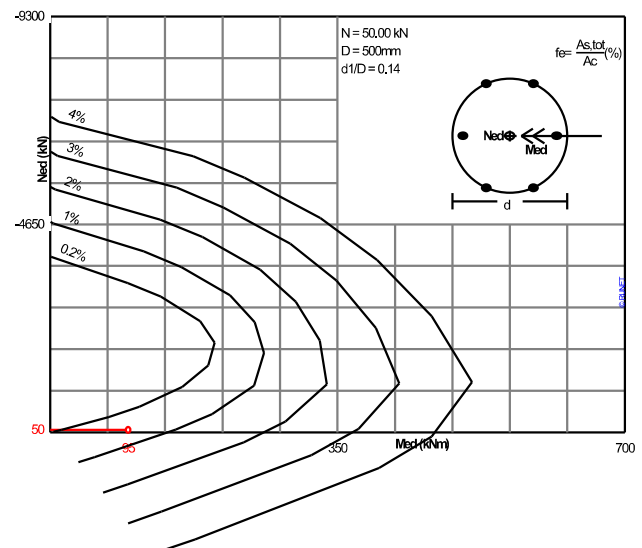
Concrete-Steel class: C30/37-B500C (EC2 §3)
 Environmental class : XC1 (EC2 §4.4.1)
 Concrete cover : Cnom=50 mm (EC2 §4.4.1)
 Concrete weight : 25.0 kN/m³
 $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N)
 $f_{cd}=\alpha_{cc} \cdot f_{ck}/\gamma_c=1.00 \times 30/1.50=20.00$ MPa (EC2 §3.1.6)
 $f_{ctd}=\alpha_{ct} \cdot f_{ctk0.05}/\gamma_c=1.00 \times 2.0/1.50=1.33$ MPa (EC2 §3.1.6)
 $f_{yd}=f_{yk}/\gamma_s=500/1.15=435$ MPa (EC2 §3.2.7)
 Modulus of elasticity of concrete $E_{cm}=33.0$ GPa

**2. Dimensions and loads**Circular column with diameter $D=0.500$ m, column length $L=6.500$ mLoads, Axial load $N_{ed}=50.00$ kN (compression), moments $M_{edyy}=95.00$ kNm, $M_{edzz}=0.00$ kNmEffective length direction z-z : $L_{cz}=1.00 \times L=6.500$ mEffective length direction y-y : $L_{cy}=1.00 \times L=6.500$ mEffective depth of cross section $d=h-d_1$, $d_1=d_2=C_{nom}+\phi_s+\phi/2=50+12+20/2=72$ mm, $d=428$ mm**3. Design for compression with small eccentricity (ULS)**

(EC2 §6.1, §9.2.1)

 $N_{ed}=50.00$ kN, $M_{ed,yy}=95.00$ kNm, $M_{ed,zz}=0.00$ kNm**Design using numerical integration**

Design chart for single bending and axial force
 obtained from numerical integration of the
 concrete and steel forces over the cross-section
 $N_{ed}=50.00$ kN (compression), $M_{ed}=95.00$ kNm
 C30/37-B500C

 $D=500$ mm $d=428$ mm, $d_1=72$ mm, $d_2=72$ mm, $d_1/D=0.144$ $e=M_{ed}/N_{ed}=95.00/50.00=1.900$ m=1900 mm $z_s=h/2-d_1=500/2-72=178$ mm, $e=1900$ mm $>$ $z_s=178$ mm $A_{s1}=A_{s2}=736$ mm², $(A_{s1}+A_{s2})/A_c=0.75\%$ $\epsilon_{c2}/\epsilon_{s1}=-3.50/10.74$ **$A_{s,tot}=14.73$ cm²**

Minimum longitudinal reinforcement, $A_s \geq 0.0020 A_c$, $\varnothing_s \geq 8$, $A_{s, \min} = 6 \varnothing 10$ (4.71 cm²) (EC2 §9.5.2.2)
 Maximum longitudinal reinforcement, $A_s \leq 0.04 A_c$, ($A_{s, \max} = 78.54$ cm²) (EC2 §9.5.2.3)
 Transverse reinforcement, links with minimum \varnothing_s at maximum spacing $S_{cl, t}$ at column heights from 0.50m to H-0.50m: Links $\varnothing_s \geq 6$, $S_{cl, t} \leq 20 \times 18 = 3$ (EC2 §9.5.3)
 at regions 0 to 0.50m and H-0.50m to H: Links $\varnothing_s \geq 6$, $S_{cl, t} \leq 12 \times 18 = 2$
 Basic required anchorage length $L_{bd} = 400 \text{ mm} = 0.400 \text{ m}$ (EC2 Eq.8.3)

Longitudinal reinforcement: $8 \varnothing 16$ (16.08 cm²)

Transverse reinforcement: Links $\varnothing 12/320$ [h:0.50m~H-0.50m], $\varnothing 12/220$ [h:0~0.50m, H-0.50m~H]

4. Design for second order effects (EC2 EN1992-1-1:2004, §5.8.3)

Final creep coefficient $\varphi(\infty, t_0) = 2.50$ (EC2 §3.1.4, Annex B)
 Effective creep coefficient $\varphi_{ef} = \varphi(\infty, t_0) \cdot (M_{oEqp}/M_{oEd}) = 2.50 \times 0.50 = 1.25$ (EC2 §5.8.4)
 Modulus of elasticity of concrete $E_{cd} = E_{cm}/\gamma_{ce} = 1000 \times 33.0 / 1.20 = 27.50 \text{ GPa} = 27500 \text{ MPa}$ (EC2 Eq.5.20)
 Modulus of elasticity of steel $E_s = 200 \text{ GPa} = 200000 \text{ MPa}$
 Reinforcement ratio $\rho = A_s / (b \cdot d) = 1608 / (500 \times 500) = 0.0064$

4.1. Slenderness criterion for isolated members (EC2 EN1992-1-1:2004, §5.8.3.1)

$\lambda, \lim = 20 \cdot A \cdot B \cdot C / \sqrt{n}$ (Eq.5.13N)
 $\omega = A_s \cdot f_{yd} / (A_c \cdot f_{cd}) = 1608 \times 435 / (500 \times 500 \times 20.00) = 0.14$
 $n = N_{ed} / (A_c \cdot f_{cd}) = 50000 / (500 \times 500 \times 20.00) = 0.010$
 $A = 1 / (1 + 0.2 \cdot \varphi_{ef}) = 1 / (1 + 0.2 \times 1.25) = 0.80$
 $B = \sqrt{1 + 2.0 \cdot \omega} = \sqrt{1 + 2.0 \cdot 0.14} = 1.13$
 $C = 1.70 - r_m = 0.70$, ($r_m = M_{01}/M_{02} = 1.0$)
 $\lambda, \lim = 20 \times 0.80 \times 1.13 \times 0.70 / \sqrt{0.010} = 126.70$

4.2. Slenderness and effective length, _direction z-z (EC2 EN1992-1-1:2004, §5.8.3.2)

Effective length $L_o = \beta \cdot L = 1.00 \times 6.500 = 6.500 \text{ m}$
 Slenderness ratio $\lambda = L_o / i$, $i = 0.250 \times 500 \text{ mm}$, $\lambda = 6500 / 125 = 52.00$ (Eq.5.14)
 $\lambda = 52.00 \leq \lambda, \lim = 126.70$, **second order effects may be ignored**

4.3. Nominal Stiffness (EC2 EN1992-1-1:2004, §5.8.7.2)

$EI = K_c \cdot E_{cd} \cdot I_c + K_s \cdot E_s \cdot I_s$ (EC2 Eq.5.21)
 $\rho = A_s / A_c = 0.006$, $E_{cd} = 27500 \text{ MPa}$, $E_s = 200000 \text{ MPa}$
 $n = N_{ed} / (A_c \cdot f_{cd}) = 50000 / (500 \times 500 \times 20.00) = 0.010$
 $K_s = 1$, $K_c = k_1 \cdot k_2 / (1 + \varphi_{ef})$, $\varphi_{ef} = 1.25$ (EC2 Eq.5.22)
 $k_1 = \sqrt{(f_{ck} / 20) \text{ MPa}} = \sqrt{(30 / 20)} = 1.22 \text{ MPa}$ (EC2 Eq.5.23)
 $k_2 = n \cdot \lambda / 170 \leq 0.20$, $n = 0.010$, $\lambda = 52.00$, $k_2 = 0.003$ (EC2 Eq.5.24)
 $K_c = 1.225 \times 0.003 / (1 + 1.25) = 0.002$
 $EI = 0.002 \times 27500 \times 500 \times 500^3 / 12 + 1.0 \times 200000 \times 2 \times 603 \times (356 / 2)^2 = 7.88 \cdot 10^{12} \text{ Nmm}^2 = 7881 \text{ kNm}^2$

4.4. Moment magnification factor (EC2 EN1992-1-1:2004, §5.8.7.3)

$M_{ed} = M_{oed} [1 + \beta / ((N_b / N_{ed}) - 1)]$, $N_b = \pi^2 \cdot EI / L_o^2$ (EC2 Eq.5.28)
 $\beta = \pi^2 / \alpha_0$, $\alpha_0 = 9.6$, $\beta = 1.03$ (EC2 Eq.5.29)
 $N_b = 3.14^2 \times 7881 / 6.500^2 = 1840.92 \text{ kN}$
 $M_{ed} / M_{oed} = 1 + 1.03 / (1840.92 / 50.00 - 1) = 1.03$, **Med=97.73kNm**

4.5. Nominal curvature

(EC2 EN1992-1-1:2004, §5.8.8)

$Med = Moed + M2$, $M2 = Ned \cdot e2$, $e2 = (1/r) l_0^2 / c$, $1/r = Kr \cdot K\phi \cdot (1/r_0)$ (EC2 Eq.5.31, 5.33, 5.34)
 $Kr = (n_u - n) / (n_u - n_{bal}) \leq 1$, $K\phi = 1 + \beta \cdot \phi_{ef} \geq 1$, $1/r_0 = e_{yd} / (0.45 \cdot d)$ (EC2 Eq.5.34, 5.36, 5.37)
 $n = Ned / (Ac \cdot f_{cd}) = 0.010$, $n_u = 1 + \omega = 1 + 0.140 = 1.140$, $n_{bal} = 0.4$
 $Kr = (1.140 - 0.010) / (1.140 - 0.4) = 1.527 \leq 1 = 1.000$
 $e_{yd} = f_{yd} / E_s = 435.00 / 200000.00 = 0.00218$
 $\beta = 0.35 + f_{ck} / 200 - \lambda / 150 = 0.35 + 30.00 / 200 - 52.00 / 150 = 0.153$, $K\phi = 1 + \beta \cdot \phi_{ef} = 1 + 0.153 \times 1.25 = 1.192$
 $d = 0.428 \text{ m}$, $1/r_0 = 0.00218 / (0.45 \times 0.428) = 0.01129$, $1/r = 1.00 \times 1.192 \times 0.01129 = 0.01346 \text{ 1/m}$
 $e2 = (1/r) l_0^2 / c$, $e2 = 0.01346 \times 6.500^2 / 3.14^2 = 0.05761$
 $M2 = 50.00 \times 0.05761 = 2.88 \text{ kNm}$, $Med = 95.00 + 2.88 = 97.88$, **Med = 97.88 kNm**

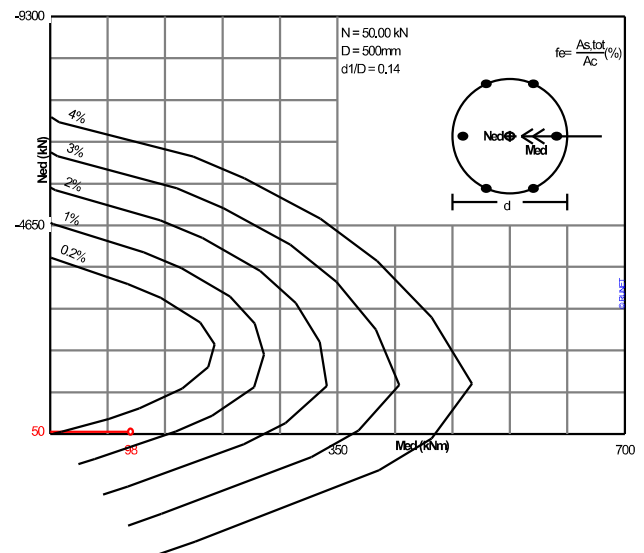
5. Design for compression with small eccentricity (ULS)

(EC2 §6.1, §9.2.1)

Ned = 50.00 kN, Med,yy = 97.88 kNm, Med,zz = 0.00 kNmDesign using numerical integration

Design chart for single bending and axial force
 obtained from numerical integration of the
 concrete and steel forces over the cross-section
 Ned = 50.00 kN (compression), Med = 97.88 kNm
 C30/37-B500C
 D = 500 mm
 d = 428 mm, d1 = 72 mm, d2 = 72 mm, d1/D = 0.144
 $e = Med / Ned = 97.88 / 50.00 = 1.958 \text{ m} = 1958 \text{ mm}$
 $z_s = h/2 - d1 = 500/2 - 72 = 178 \text{ mm}$, $e = 1958 \text{ mm} > z_s = 178 \text{ mm}$

$As1 = As2 = 736 \text{ mm}^2$, $(As1 + As2) / Ac = 0.75\%$
 $\epsilon_{c2} / \epsilon_{s1} = -3.50 / 10.74$

As,tot = 14.73 cm²

Minimum longitudinal reinforcement, $As \geq 0.0020 Ac$, $\phi_s \geq 8$, $As, min = 6\phi 10$ (4.71 cm²) (EC2 §9.5.2.2)
 Maximum longitudinal reinforcement, $As \leq 0.04 Ac$, ($As, max = 78.54 \text{ cm}^2$) (EC2 §9.5.2.3)
 Transverse reinforcement, links with minimum ϕ_s at maximum spacing $S_{cl,t}$ (EC2 §9.5.3)
 at column heights from 0.50 m to H-0.50 m: Links $\phi_s \geq 6$, $S_{cl,t} \leq 20 \times 18 = 3$
 at regions 0 to 0.50 m and H-0.50 m to H: Links $\phi_s \geq 6$, $S_{cl,t} \leq 12 \times 18 = 2$
 Basic required anchorage length $L_{bd} = 400 \text{ mm} = 0.400 \text{ m}$ (EC2 Eq.8.3)

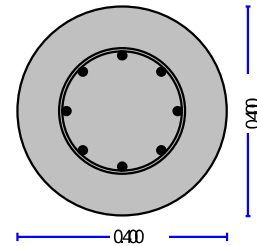
Longitudinal reinforcement: 8Ø16 (16.08 cm²)**Transverse reinforcement: Links Ø12/320 [h:0.50m~H-0.50m], Ø12/220 [h:0~0.50m, H-0.50m~H]**

Šolski center NM 27/05/2024**1. Piloti fi40/0,80m PS-2****Isolated column single bending**

(EC2 EN1992-1-1:2004, EC0 EN1990:2002,)

D =0.400 m, Ned = 32.00 kN**Med yy = 22.00 kNm, Med zz = 0.00 kNm****Reinforced concrete design**

Concrete-Steel class: C30/37-B500C (EC2 §3)
 Environmental class : XC1 (EC2 §4.4.1)
 Concrete cover : Cnom=80 mm (EC2 §4.4.1)
 Concrete weight : 25.0 kN/m³
 $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N)
 $f_{cd}=\alpha_{cc} \cdot f_{ck}/\gamma_c=1.00 \times 30/1.50=20.00$ MPa (EC2 §3.1.6)
 $f_{ctd}=\alpha_{ct} \cdot f_{ctk0.05}/\gamma_c=1.00 \times 2.0/1.50=1.33$ MPa (EC2 §3.1.6)
 $f_{yd}=f_{yk}/\gamma_s=500/1.15=435$ MPa (EC2 §3.2.7)
 Modulus of elasticity of concrete $E_{cm}=33.0$ GPa

**2. Dimensions and loads**

Circular column with diameter D=0.400 m, column length L=3.000 m

Loads, Axial load Ned=32.00kN (compression), moments Medyy=22.00kNm, Medzz=0.00kNm

Effective length direction z-z : Lcz= 1.00xL= 3.000m

Effective length direction y-y : Lcy= 1.00xL= 3.000m

Effective depth of cross section d=h-d1, d1=d2=Cnom+Øs+Ø/2=80+8+20/2=98mm, d=302mm

3. Design for compression with small eccentricity (ULS)

(EC2 §6.1, §9.2.1)

Ned=32.00kN, Med,yy=22.00kNm, Med,zz=0.00kNm**Design using numerical integration**

Design chart for single bending and axial force
 obtained from numerical integration of the
 concrete and steel forces over the cross-section
 Ned=32.00kN (compression), Med=22.00kNm
 C30/37-B500C

D=400mm

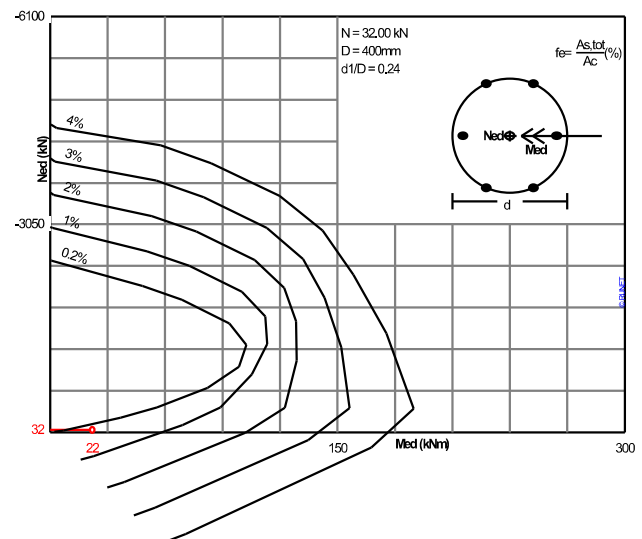
d=302mm, d1= 98mm, d2= 98mm, d1/D=0.245

e=Med/Ned=22.00/32.00=0.688m=688mm

zs=h/2-d1=400/2-98=102mm, e=688mm>sz=102mm

As1=As2=314mm², (As1+As2)/Ac=0.50%

εc2/εs1=-3.50/6.70

As,tot= 6.28cm²

Minimum longitudinal reinforcement, $A_s \geq 0.0020 A_c$, $\varnothing_s \geq 8$, $A_{s,min} = 6\varnothing 8$ (3.02 cm^2) (EC2 §9.5.2.2)
 Maximum longitudinal reinforcement, $A_s \leq 0.04 A_c$, ($A_{s,max} = 50.27 \text{ cm}^2$) (EC2 §9.5.2.3)
 Transverse reinforcement, links with minimum \varnothing_s at maximum spacing $S_{cl,t}$ at column heights from 0.40m to $H-0.40\text{m}$: Links $\varnothing_s \geq 6$, $S_{cl,t} \leq 20 \times 18 = 3$ (EC2 §9.5.3)
 at regions 0 to 0.40m and $H-0.40\text{m}$ to H : Links $\varnothing_s \geq 6$, $S_{cl,t} \leq 12 \times 18 = 2$
 Basic required anchorage length $L_{bd} = 400 \text{ mm} = 0.40 \text{ m}$ (EC2 Eq.8.3)

Longitudinal reinforcement: $8\varnothing 16$ (16.08 cm^2)

Transverse reinforcement: Links $\varnothing 8/320$ [$h: 0.40 \text{ m} \sim H-0.40 \text{ m}$], $\varnothing 8/220$ [$h: 0 \sim 0.40 \text{ m}$, $H-0.40 \text{ m} \sim H$]

4. Design for second order effects (EC2 EN1992-1-1:2004, §5.8.3)

Final creep coefficient $\varphi(\infty, t_0) = 2.50$ (EC2 §3.1.4, Annex B)
 Effective creep coefficient $\varphi_{ef} = \varphi(\infty, t_0) \cdot (M_{oEqp}/M_{oEd}) = 2.50 \times 0.50 = 1.25$ (EC2 §5.8.4)
 Modulus of elasticity of concrete $E_{cd} = E_{cm}/\gamma_{ce} = 1000 \times 33.0 / 1.20 = 27.50 \text{ GPa} = 27500 \text{ MPa}$ (EC2 Eq.5.20)
 Modulus of elasticity of steel $E_s = 200 \text{ GPa} = 200000 \text{ MPa}$
 Reinforcement ratio $\rho = A_s / (b \cdot d) = 1608 / (400 \times 400) = 0.0100$

4.1. Slenderness criterion for isolated members (EC2 EN1992-1-1:2004, §5.8.3.1)

$\lambda, \lim = 20 \cdot A \cdot B \cdot C / \sqrt{n}$ (Eq.5.13N)
 $\omega = A_s \cdot f_{yd} / (A_c \cdot f_{cd}) = 1608 \times 435 / (400 \times 400 \times 20.00) = 0.22$
 $n = N_{ed} / (A_c \cdot f_{cd}) = 32000 / (400 \times 400 \times 20.00) = 0.010$
 $A = 1 / (1 + 0.2 \cdot \varphi_{ef}) = 1 / (1 + 0.2 \times 1.25) = 0.80$
 $B = \sqrt{1 + 2.0 \cdot \omega} = \sqrt{1 + 2.0 \cdot 0.22} = 1.20$
 $C = 1.70 - r_m = 0.70$, ($r_m = M_{01}/M_{02} = 1.0$)
 $\lambda, \lim = 20 \times 0.80 \times 1.20 \times 0.70 / \sqrt{0.010} = 134.27$

4.2. Slenderness and effective length, _direction z-z (EC2 EN1992-1-1:2004, §5.8.3.2)

Effective length $L_0 = \beta \cdot L = 1.00 \times 3.000 = 3.000 \text{ m}$
 Slenderness ratio $\lambda = L_0 / i$, $i = 0.250 \times 400 \text{ mm}$, $\lambda = 3000 / 100 = 30.00$ (Eq.5.14)
 $\lambda = 30.00 \leq \lambda, \lim = 134.27$, **second order effects may be ignored**

4.3. Nominal Stiffness (EC2 EN1992-1-1:2004, §5.8.7.2)

$EI = K_c \cdot E_{cd} \cdot I_c + K_s \cdot E_s \cdot I_s$ (EC2 Eq.5.21)
 $\rho = A_s / A_c = 0.010$, $E_{cd} = 27500 \text{ MPa}$, $E_s = 200000 \text{ MPa}$
 $n = N_{ed} / (A_c \cdot f_{cd}) = 32000 / (400 \times 400 \times 20.00) = 0.010$
 $K_s = 1$, $K_c = k_1 \cdot k_2 / (1 + \varphi_{ef})$, $\varphi_{ef} = 1.25$ (EC2 Eq.5.22)
 $k_1 = \sqrt{(f_{ck} / 20) \text{ MPa}} = \sqrt{(30 / 20)} = 1.22 \text{ MPa}$ (EC2 Eq.5.23)
 $k_2 = n \cdot \lambda / 170 \leq 0.20$, $n = 0.010$, $\lambda = 30.00$, $k_2 = 0.002$ (EC2 Eq.5.24)
 $K_c = 1.225 \times 0.002 / (1 + 1.25) = 0.001$
 $EI = 0.001 \times 27500 \times 400 \times 400^3 / 12 + 1.0 \times 200000 \times 2 \times 603 \times (204 / 2)^2 = 2.57 \cdot 10^{12} \text{ Nmm}^2 = 2566 \text{ kNm}^2$

4.4. Moment magnification factor (EC2 EN1992-1-1:2004, §5.8.7.3)

$M_{ed} = M_{oed} [1 + \beta / ((N_b / N_{ed}) - 1)]$, $N_b = \pi^2 \cdot EI / L_0^2$ (EC2 Eq.5.28)
 $\beta = \pi^2 / \alpha$, $\alpha = 9.6$, $\beta = 1.03$ (EC2 Eq.5.29)
 $N_b = 3.14^2 \times 2566 / 3.000^2 = 2813.71 \text{ kN}$
 $M_{ed} / M_{oed} = 1 + 1.03 / (2813.71 / 32.00 - 1) = 1.01$, **$M_{ed} = 22.26 \text{ kNm}$**

4.5. Nominal curvature

(EC2 EN1992-1-1:2004, §5.8.8)

$M_{ed} = M_{oed} + M_2$, $M_2 = N_{ed} \cdot e_2$, $e_2 = (1/r) l_o^2 / c$, $1/r = K_r \cdot K_\phi \cdot (1/r_o)$ (EC2 Eq.5.31, 5.33, 5.34)
 $K_r = (n_u - n) / (n_u - n_{bal}) \leq 1$, $K_\phi = 1 + \beta \cdot \phi_{ef} \geq 1$, $1/r_o = e_{yd} / (0.45 \cdot d)$ (EC2 Eq.5.34, 5.36, 5.37)
 $n = N_{ed} / (A_c \cdot f_{cd}) = 0.010$, $n_u = 1 + \omega = 1 + 0.219 = 1.219$, $n_{bal} = 0.4$
 $K_r = (1.219 - 0.010) / (1.219 - 0.4) = 1.476 \leq 1 = 1.000$
 $e_{yd} = f_{yd} / E_s = 435.00 / 200000.00 = 0.00218$
 $\beta = 0.35 + f_{ck} / 200 - \lambda / 150 = 0.35 + 30.00 / 200 - 30.00 / 150 = 0.300$, $K_\phi = 1 + \beta \cdot \phi_{ef} = 1 + 0.300 \times 1.25 = 1.375$
 $d = 0.302 \text{ m}$, $1/r_o = 0.00218 / (0.45 \times 0.302) = 0.01600$, $1/r = 1.00 \times 1.375 \times 0.01600 = 0.02201 \text{ 1/m}$
 $e_2 = (1/r) l_o^2 / c$, $e_2 = 0.02201 \times 3.000^2 / 3.14^2 = 0.02007$
 $M_2 = 32.00 \times 0.02007 = 0.64 \text{ kNm}$, $M_{ed} = 22.00 + 0.64 = 22.64$, **$M_{ed} = 22.64 \text{ kNm}$**

5. Design for compression with small eccentricity (ULS)

(EC2 §6.1, §9.2.1)

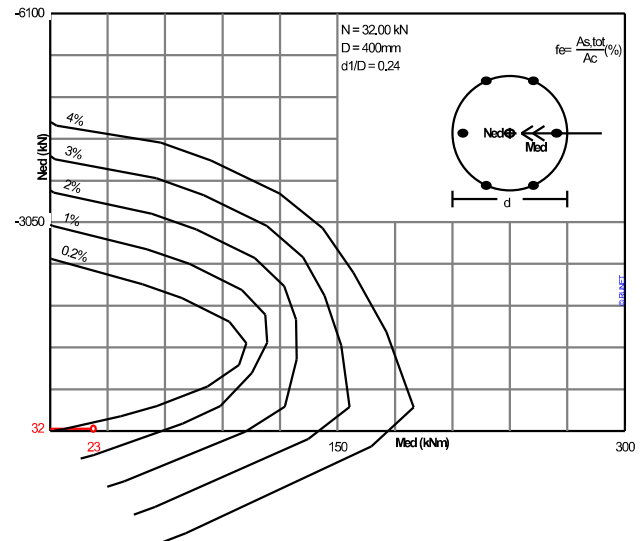
 $N_{ed} = 32.00 \text{ kN}$, $M_{ed,yy} = 22.64 \text{ kNm}$, $M_{ed,zz} = 0.00 \text{ kNm}$ Design using numerical integration

Design chart for single bending and axial force
 obtained from numerical integration of the
 concrete and steel forces over the cross-section
 $N_{ed} = 32.00 \text{ kN}$ (compression), $M_{ed} = 22.64 \text{ kNm}$
 C30/37-B500C
 $D = 400 \text{ mm}$
 $d = 302 \text{ mm}$, $d_1 = 98 \text{ mm}$, $d_2 = 98 \text{ mm}$, $d_1/D = 0.245$
 $e = M_{ed} / N_{ed} = 22.64 / 32.00 = 0.708 \text{ m} = 708 \text{ mm}$
 $z_s = h/2 - d_1 = 400/2 - 98 = 102 \text{ mm}$, $e = 708 \text{ mm} > z_s = 102 \text{ mm}$

$A_{s1} = A_{s2} = 314 \text{ mm}^2$, $(A_{s1} + A_{s2}) / A_c = 0.50\%$
 $\epsilon_{c2} / \epsilon_{s1} = -3.50 / 6.70$

 $A_{s,tot} = 6.28 \text{ cm}^2$

Minimum longitudinal reinforcement, $A_s \geq 0.0020 A_c$, $\phi_s \geq 8$, $A_{s,min} = 6\phi 8$ (3.02 cm^2) (EC2 §9.5.2.2)
 Maximum longitudinal reinforcement, $A_s \leq 0.04 A_c$, ($A_{s,max} = 50.27 \text{ cm}^2$) (EC2 §9.5.2.3)
 Transverse reinforcement, links with minimum ϕ_s at maximum spacing $S_{cl,t}$ (EC2 §9.5.3)
 at column heights from 0.40 m to $H - 0.40 \text{ m}$: Links $\phi_s \geq 6$, $S_{cl,t} \leq 20 \times 18 = 3$
 at regions 0 to 0.40 m and $H - 0.40 \text{ m}$ to H : Links $\phi_s \geq 6$, $S_{cl,t} \leq 12 \times 18 = 2$
 Basic required anchorage length $L_{bd} = 400 \text{ mm} = 0.40 \text{ m}$ (EC2 Eq.8.3)

Longitudinal reinforcement: $8\phi 16$ (16.08 cm^2)**Transverse reinforcement: Links $\phi 8/320$ [$h: 0.40 \text{ m} \sim H - 0.40 \text{ m}$], $\phi 8/220$ [$h: 0 \sim 0.40 \text{ m}$, $H - 0.40 \text{ m} \sim H$]**

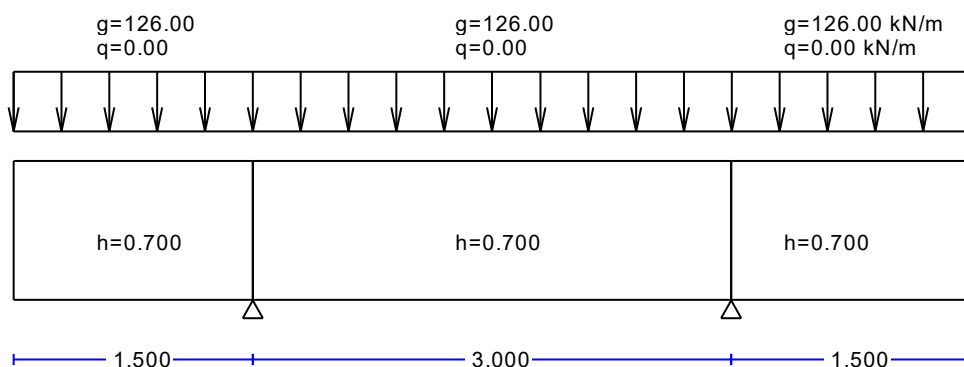
Šolski center NM 27/05/2024

1. Pilotna greda 70x70 PS-1

Continuous beam with distributed loads

(EC2 EN1992-1-1:2004, EC0 EN1990:2002,)

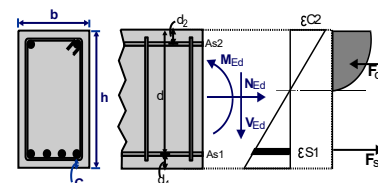
C30/37 - S500



Reinforced concrete design

Concrete-Steel class: C30/37-S500 (EC2 §3)

Environmental class : XC3 (EC2 §4.4.1)

Concrete cover : C_{nom}=50 mm (EC2 §4.4.1)Concrete weight : 25.0 kN/m³ $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N) $f_{cd}=\alpha_{cc} \cdot f_{ck} / \gamma_c = 1.00 \times 30 / 1.50 = 20.00$ MPa (EC2 §3.1.6) $f_{ctd}=\alpha_{ct} \cdot f_{ctk0.05} / \gamma_c = 1.00 \times 2.0 / 1.50 = 1.33$ MPa (EC2 §3.1.6) $f_{yd}=f_{yk} / \gamma_s = 500 / 1.15 = 435$ MPa (EC2 §3.2.7)Modulus of elasticity of concrete $E_{cm}=33.0$ GPa

2. Dimensions and loads

Continuous beam (rectangular section), number of spans=1

Partial safety factors for actions : $\gamma_G=1.35$, $\gamma_Q=1.50$ (EC0 Annex A1)Combination of variable actions : $\psi_0=0.70$, $\psi_1=0.60$, $\psi_2=0.30$ Effective depth of cross section $d=h-d_1$, $d_1=C_{nom}+\phi_s+0.5\phi=50+12+0.5 \times 16=70$ mmbeam self weight = $b_w \cdot h \times 25.0$ Spans, widths, thickness, load on spans (g =self weight +dead, q =live)Cant-1 $L=1.50$ m $b_w=0.700$ m $b_{eff}=0.700$ m $h=0.700$ m $g=12.25+126.00=138.25$ kN/m $q=0.00$ kN/mSpan-1 $L=3.00$ m $b_w=0.700$ m $b_{eff}=0.700$ m $h=0.700$ m $g=12.25+126.00=138.25$ kN/m $q=0.00$ kN/mCant-2 $L=1.50$ m $b_w=0.700$ m $b_{eff}=0.700$ m $h=0.700$ m $g=12.25+126.00=138.25$ kN/m $q=0.00$ kN/m

3. Ultimate limit state (ULS), design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Load (STR) Span-1 $q_{ed}=\gamma_G \cdot g + \gamma_Q \cdot q = 1.35g + 1.50q = 1.35 \times 138.25 + 1.50 \times 0.00 = 186.64$ kN/mCross section values (area A , moment of inertia I_{yy} , centroid z_c)Cant-1 $L=1.500$ m, $A=0.49000$ m² (4.90×10^{-5} m²), $I_{yy}=0.02001$ m⁴ (2.00×10^{-10} m⁴), $z_c=0.000$ m (0 mm)Span-1 $L=3.000$ m, $A=0.49000$ m² (4.90×10^{-5} m²), $I_{yy}=0.02001$ m⁴ (2.00×10^{-10} m⁴), $z_c=0.000$ m (0 mm)Cant-2 $L=1.500$ m, $A=0.49000$ m² (4.90×10^{-5} m²), $I_{yy}=0.02001$ m⁴ (2.00×10^{-10} m⁴), $z_c=0.000$ m (0 mm)

4. Shearing forces and bending moments

Maximum bending moments at spans for load combinations 1.35g+1.50q

Cant-1, Med=-209.97 kNm, $x_0=1.500$ m, $x_1=0.000$ m, $x_2=0.000$ m

Span-1, Med= 54.44 kNm, $x_0=1.500$ m, $x_1=0.736$ m, $x_2=0.736$ m

Cant-2, Med=-209.97 kNm, $x_0=0.000$ m, $x_1=0.000$ m, $x_2=0.000$ m

Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0, Med=-209.97 kNm, $x_1=1.500$ m, $x_2=0.000$ m

Support-1, Med=-209.97 kNm, $x_1=3.000$ m, $x_2=1.500$ m

Maximum shear forces for load combinations 1.35g+1.50q

Cant-1, Ved,A= 0.00 kN, Ved,B=-279.96 kN

Span-1, Ved,A= 279.96 kN, Ved,B=-279.96 kN

Cant-2, Ved,A= 279.96 kN, Ved,B= 0.00 kN

Maximum reactions due to dead and live loads (Rg and Rq)

Support-0, Rg($x_1.35$)= 559.91 kN, Rq($x_1.50$)= 0.00 kN

Support-1, Rg($x_1.35$)= 559.91 kN, Rq($x_1.50$)= 0.00 kN

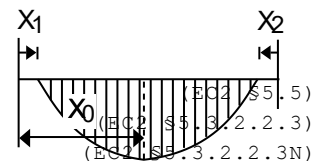


5. Design actions, shearing forces and bending moments

Design action values after moment redistribution by 0%

Reduction of support values to values at support faces ($b_{sup}=0.20$ m)

Check for minimum values, ($0.65q_{l2/8}$, $0.65q_{l2/12}$)



Maximum span bending moments and shear forces for load combinations 1.35g+1.50q

Cant-1, Med=-181.97 kNm, Ved,A= 18.66 kN, Ved,B=-261.29 kN

Span-1, Med= 54.44 kNm, Ved,A= 261.29 kN, Ved,B=-261.29 kN

Cant-2, Med=-181.97 kNm, Ved,A= 261.29 kN, Ved,B= 18.66 kN

Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0, Med=-181.97 kNm, $x_1=1.500$ m, $x_2=0.000$ m

Support-1, Med=-181.97 kNm, $x_1=3.000$ m, $x_2=1.500$ m

Maximum shear forces at distance d from support face 1.35g+1.50q

Cant-1, $b/2+d=0.730$ m, $1.35g+1.50q=186.64$ kN/m, VedA= 0.00 kN, VedB= 143.71 kN

Span-1, $b/2+d=0.730$ m, $1.35g+1.50q=186.64$ kN/m, VedA= 143.71 kN, VedB= 143.71 kN

Cant-2, $b/2+d=0.730$ m, $1.35g+1.50q=186.64$ kN/m, VedA= 143.71 kN, VedB= 0.00 kN

6. Ultimate limit state (ULS), design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.2.1)

6.1. Span-1 Ultimate limit state (ULS), design for bending

(EC2 §6.1, §9.2.1)

Effective depth of cross section $d_1=C_{nom}+\phi_s+0.5\phi=50+12+0.5\cdot 16=70$ mm, $d_2=70$ mm, $d=700-70=630$ mm

Reinforcement for bending (only tension reinforcement is needed)

Med= 54.44 kNm $b_w=700$ mm $d=630$ mm $K_d=7.14$ $x/d=0.03$ $\epsilon_{c2}/\epsilon_{s1}=-0.7/20.0$ $k_s=2.33$, **As1= 2.01 cm²**

Minimum longitudinal tension reinf., $A_s > 0.26 b d \cdot f_{ctm} / f_{yk}$, ($A_{s,min}= 6.65$ cm²) (EC2 §9.2.1.1.1)

Maximum tension or compression reinf., $A_s \leq 0.04 A_c$, ($A_{s,max}=196.0$ cm²) (EC2 §9.2.1.1.3)

Reinforcement for bending: 6Ø16 (12.06 cm²) (bottom), 6Ø16 (12.06 cm²) (top)

Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

$b=700$ mm, $h=700$ mm, $d=630$ mm, $A_{s1}=1206$ mm², $A_{s2}=1206$ mm²

$\epsilon_{c2}=-2.16\%$, $\epsilon_{s1}=19.93\%$, $A_{s1}/b \cdot d=0.00311$ (0.311%)

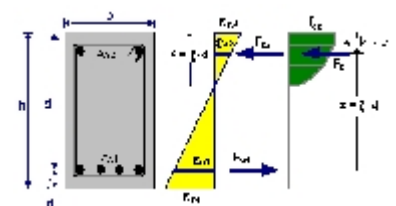
$x/d=\epsilon_{c2}/(\epsilon_{c2}+\epsilon_{s1})=2.16/(2.16+19.93)=0.098$, $x=61.6$ mm

$\alpha_r=0.691$, $k_a=0.380$, $F_c=\alpha_r \cdot b \cdot x \cdot f_{cd}=F_{s1}=596.25$ kN, $A_{s1}=F_{s1}/f_{yd}=1371$ mm²

$z=d-k_a \cdot x=(1-k_a \cdot \epsilon_{c2}/(\epsilon_{c2}+\epsilon_{s1}))d$, $z/d=1.0-0.380 \cdot 0.098=0.963$, $z=606.6$ mm,

$K_{d2}=1/(0.691 \cdot 0.098 \cdot 0.963 \cdot 20.00)=0.768$ mm²/N, $K_d=0.876$

Bending capacity $M_r=b \cdot d^2/K_{d2}=[10^{-6}] \times 700 \times 630^2/0.768=362.00$ kNm



6.2. Support-0 Ultimate limit state (ULS), design for bending

(EC2 §6.1, §9.2.1)

Effective depth of cross section $d_1 = C_{nom} + \varnothing s + 0.5\varnothing = 50 + 12 + 0.5 \times 16 = 70 \text{ mm}$, $d_2 = 70 \text{ mm}$, $d = 700 - 70 = 630 \text{ mm}$

Reinforcement for bending (only tension reinforcement is needed)

 $M_{ed} = -181.97 \text{ kNm}$, $b_w = 700 \text{ mm}$, $d = 630 \text{ mm}$, $K_d = 3.91$, $x/d = 0.06$, $\varepsilon_{c2}/\varepsilon_{s1} = -1.4/20.0$, $k_s = 2.35$, **$A_{s2} = 6.80 \text{ cm}^2$** Minimum longitudinal tension reinf., $A_s \geq 0.26 b_d \cdot f_{ctm} / f_{yk}$, ($A_{s,min} = 6.65 \text{ cm}^2$) (EC2 §9.2.1.1.1)Maximum tension or compression reinf., $A_s \leq 0.04 A_c$, ($A_{s,max} = 196.0 \text{ cm}^2$) (EC2 §9.2.1.1.3)**Reinforcement for bending: $6\varnothing 16$ (12.06 cm²) (top), $6\varnothing 16$ (12.06 cm²) (bottom)**

Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

 $b = 700 \text{ mm}$, $h = 700 \text{ mm}$, $d = 630 \text{ mm}$, $A_{s1} = 1206 \text{ mm}^2$, $A_{s2} = 1206 \text{ mm}^2$ $\varepsilon_{c2} = -2.16\%$, $\varepsilon_{s1} = 19.93\%$, $A_{s1}/b \cdot d = 0.00311$ (0.311%) $x/d = \varepsilon_{c2} / (\varepsilon_{c2} + \varepsilon_{s1}) = 2.16 / (2.16 + 19.93) = 0.098$, $x = 61.6 \text{ mm}$ $\alpha_r = 0.691$, $k_a = 0.380$, $F_c = \alpha_r \cdot b \cdot x \cdot f_{cd} = F_{s1} = 596.25 \text{ kN}$, $A_{s1} = F_{s1} / f_{yd} = 1371 \text{ mm}^2$ $z = d - k_a \cdot x = ([1 - k_a \cdot \varepsilon_{c2} / (\varepsilon_{c2} + \varepsilon_{s1})] d)$, $z/d = 1.0 - 0.380 \times 0.098 = 0.963$, $z = 606.6 \text{ mm}$, $K_d = 1 / (0.691 \cdot 0.098 \cdot 0.963 \cdot 20.00) = 0.768 \text{ mm}^2/\text{N}$, $K_d = 0.876$ Bending capacity $M_r = b \cdot d^2 / K_d = [10^{-6}] \times 700 \times 630^2 / 0.768 = 362.00 \text{ kNm}$ **6.3. Support-1 Ultimate limit state (ULS), design for bending**

(EC2 §6.1, §9.2.1)

Effective depth of cross section $d_1 = C_{nom} + \varnothing s + 0.5\varnothing = 50 + 12 + 0.5 \times 16 = 70 \text{ mm}$, $d_2 = 70 \text{ mm}$, $d = 700 - 70 = 630 \text{ mm}$

Reinforcement for bending (only tension reinforcement is needed)

 $M_{ed} = -181.97 \text{ kNm}$, $b_w = 700 \text{ mm}$, $d = 630 \text{ mm}$, $K_d = 3.91$, $x/d = 0.06$, $\varepsilon_{c2}/\varepsilon_{s1} = -1.4/20.0$, $k_s = 2.35$, **$A_{s2} = 6.80 \text{ cm}^2$** Minimum longitudinal tension reinf., $A_s \geq 0.26 b_d \cdot f_{ctm} / f_{yk}$, ($A_{s,min} = 6.65 \text{ cm}^2$)Maximum tension or compression reinf., $A_s \leq 0.04 A_c$, ($A_{s,max} = 196.0 \text{ cm}^2$)**Reinforcement for bending: $6\varnothing 16$ (12.06 cm²) (top), $6\varnothing 16$ (12.06 cm²) (bottom)**

Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

 $b = 700 \text{ mm}$, $h = 700 \text{ mm}$, $d = 630 \text{ mm}$, $A_{s1} = 1206 \text{ mm}^2$, $A_{s2} = 1206 \text{ mm}^2$ $\varepsilon_{c2} = -2.16\%$, $\varepsilon_{s1} = 19.93\%$, $A_{s1}/b \cdot d = 0.00311$ (0.311%) $x/d = \varepsilon_{c2} / (\varepsilon_{c2} + \varepsilon_{s1}) = 2.16 / (2.16 + 19.93) = 0.098$, $x = 61.6 \text{ mm}$ $\alpha_r = 0.691$, $k_a = 0.380$, $F_c = \alpha_r \cdot b \cdot x \cdot f_{cd} = F_{s1} = 596.25 \text{ kN}$, $A_{s1} = F_{s1} / f_{yd} = 1371 \text{ mm}^2$ $z = d - k_a \cdot x = ([1 - k_a \cdot \varepsilon_{c2} / (\varepsilon_{c2} + \varepsilon_{s1})] d)$, $z/d = 1.0 - 0.380 \times 0.098 = 0.963$, $z = 606.6 \text{ mm}$, $K_d = 1 / (0.691 \cdot 0.098 \cdot 0.963 \cdot 20.00) = 0.768 \text{ mm}^2/\text{N}$, $K_d = 0.876$ Bending capacity $M_r = b \cdot d^2 / K_d = [10^{-6}] \times 700 \times 630^2 / 0.768 = 362.00 \text{ kNm}$ **7. Ultimate limit state (ULS), Design for shear**

(EC2 EN1992-1-1:2004, §6.2, §9.2.2)

7.1. Span-0 right Ultimate limit state (ULS), Design for shear

(EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc}

(EC2 §6.2.2)

 $V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$, $V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$

(EC2 Eq.6.2.a,b)

 $C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120$, $f_{ck} = 30 \text{ MPa}$, $b_w = 700 \text{ mm}$, $d = 630 \text{ mm}$ $k = 1 + \sqrt{200/d} \leq 2$, $k = 1.56$, $k_1 = 0.15$ $V_{rdc,min} = 0.001 \times (0.37) \times 700 \times 630 = 163.17 \text{ kN}$, $v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.37 \text{ N/mm}^2$,

(EC2 Eq.6.3N)

 $\rho_1 = 1206 / (700 \times 630) = 0.0027$, $V_{rdc} = 0.001 \times [0.120 \times 1.56 \times (0.27 \times 30)^{0.33}] \times 700 \times 630 = 165.80 \text{ kN}$ $V_{ed} = 143.71 \text{ kN} \leq V_{rdc} = 165.80 \text{ kN}$, **$V_{ed} \leq V_{rdc}$ shear reinforcement is not needed**Concrete strut capacity V_{rdmax}

(EC2 §6.2.3 Eq.6.9)

 $V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta)$, $V_{ed} / \max(V_{rdmax}) = 0.12$, $\theta = 21.8^\circ$, $\cot \theta = 2.50$, $\tan \theta = 0.40$ $\alpha_{cw} = 1.00$, $z = 0.9d$, $f_{ck} = 30.0 \leq 60 \text{ MPa}$, $v_1 = 0.6 [1 - f_{ck}/250] = 0.6 [1 - 30/250] = 0.528$, $f_{cd} = 20.00 \text{ MPa}$ $V_{rdmax} = 0.001 \times 1.00 \times 700 \times 0.9 \times 630 \times 0.528 \times 20.00 / 2.90 = 1445.3 \text{ kN}$ $V_{ed} = 261.3 \text{ kN} < 1445.3 \text{ kN} = V_{rdmax}$, the check is verified

Minimum links for shear reinforcement (EC2 §9.2.2)
 Minimum shear reinforcement ratio $\rho_{w,min}$ (EC2 Eq.9.5N)
 $\rho_{w,min} = (0.08 \times f_{ck})^{0.5} / f_{yk}$, $f_{ck} = 30 \text{ N/mm}^2$, $f_{yk} = 500 \text{ N/mm}^2$, $\rho_{w,min} = 0.0009$
 $\min A_{sw}/s = 10 \times 0.0009 \times 700 \times \sin(90^\circ) = 6.30 \text{ cm}^2/\text{m}$
 Maximum longitudinal spacing of links $s_{lmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 470mm (EC2 §9.2.2.6, Eq.9.6N)
 Maximum transverse spacing of link legs $s_{tmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 470mm (§9.2.2.8, Eq.9.8N)
 Minimum shear reinforcement 2Ø12/47.0 ($A_{sw}/s = 9.62 \text{ cm}^2/\text{m}$)

Span-0 right Shear reinforcement: 2Ø12/47.0 ($A_{sw}/s = 9.62 \text{ cm}^2/\text{m}$)

7.2. Span-1 left Ultimate limit state (ULS), Design for shear (EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc} (EC2 §6.2.2)
 $V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$, $V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ (EC2 Eq.6.2.a,b)
 $C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120$, $f_{ck} = 30 \text{ MPa}$, $b_w = 700 \text{ mm}$, $d = 630 \text{ mm}$
 $k = 1 + \sqrt{(200/d)} \leq 2$, $k = 1.56$, $k_1 = 0.15$
 $V_{rd,c(min)} = 0.001 \times (0.37) \times 700 \times 630 = 163.17 \text{ kN}$, $v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.37 \text{ N/mm}^2$, (EC2 Eq.6.3N)
 $\rho_1 = 1206 / (700 \times 630) = 0.0027$, $V_{rdc} = 0.001 \times [0.120 \times 1.56 \times (0.27 \times 30)^{0.33}] \times 700 \times 630 = 165.80 \text{ kN}$
 $V_{ed} = 143.71 \text{ kN} \leq V_{rdc} = 165.80 \text{ kN}$, **Ved ≤ Vrdc shear reinforcement is not needed**

Concrete strut capacity V_{rdmax} (EC2 §6.2.3 Eq.6.9)
 $V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta)$, $V_{ed} / \max(V_{rdmax}) = 0.12$, $\theta = 21.8^\circ$ $\cot \theta = 2.50$ $\tan \theta = 0.40$
 $\alpha_{cw} = 1.00$ $z = 0.9d$, $f_{ck} = 30.0 \leq 60 \text{ MPa}$ $v_1 = 0.6 [1 - f_{ck}/250] = 0.6 [1 - 30/250] = 0.528$, $f_{cd} = 20.00 \text{ MPa}$
 $V_{rdmax} = 0.001 \times 1.00 \times 700 \times 0.9 \times 630 \times 0.528 \times 20.00 / 2.90 = 1445.3 \text{ kN}$
 $V_{ed} = 261.3 \text{ kN} < 1445.3 \text{ kN} = V_{rdmax}$, the check is verified

Minimum links for shear reinforcement (EC2 §9.2.2)
 Minimum shear reinforcement ratio $\rho_{w,min}$ (EC2 Eq.9.5N)
 $\rho_{w,min} = (0.08 \times f_{ck})^{0.5} / f_{yk}$, $f_{ck} = 30 \text{ N/mm}^2$, $f_{yk} = 500 \text{ N/mm}^2$, $\rho_{w,min} = 0.0009$
 $\min A_{sw}/s = 10 \times 0.0009 \times 700 \times \sin(90^\circ) = 6.30 \text{ cm}^2/\text{m}$
 Maximum longitudinal spacing of links $s_{lmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 470mm (EC2 §9.2.2.6, Eq.9.6N)
 Maximum transverse spacing of link legs $s_{tmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 470mm (§9.2.2.8, Eq.9.8N)
 Minimum shear reinforcement 2Ø12/47.0 ($A_{sw}/s = 9.62 \text{ cm}^2/\text{m}$)

Span-1 left Shear reinforcement: 2Ø12/47.0 ($A_{sw}/s = 9.62 \text{ cm}^2/\text{m}$)

7.3. Span-1 right Ultimate limit state (ULS), Design for shear (EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc} (EC2 §6.2.2)
 $V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$, $V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ (EC2 Eq.6.2.a,b)
 $C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120$, $f_{ck} = 30 \text{ MPa}$, $b_w = 700 \text{ mm}$, $d = 630 \text{ mm}$
 $k = 1 + \sqrt{(200/d)} \leq 2$, $k = 1.56$, $k_1 = 0.15$
 $V_{rd,c(min)} = 0.001 \times (0.37) \times 700 \times 630 = 163.17 \text{ kN}$, $v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.37 \text{ N/mm}^2$, (EC2 Eq.6.3N)
 $\rho_1 = 1206 / (700 \times 630) = 0.0027$, $V_{rdc} = 0.001 \times [0.120 \times 1.56 \times (0.27 \times 30)^{0.33}] \times 700 \times 630 = 165.80 \text{ kN}$
 $V_{ed} = 143.71 \text{ kN} \leq V_{rdc} = 165.80 \text{ kN}$, **Ved ≤ Vrdc shear reinforcement is not needed**

Concrete strut capacity V_{rdmax} (EC2 §6.2.3 Eq.6.9)
 $V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta)$, $V_{ed} / \max(V_{rdmax}) = 0.12$, $\theta = 21.8^\circ$ $\cot \theta = 2.50$ $\tan \theta = 0.40$
 $\alpha_{cw} = 1.00$ $z = 0.9d$, $f_{ck} = 30.0 \leq 60 \text{ MPa}$ $v_1 = 0.6 [1 - f_{ck}/250] = 0.6 [1 - 30/250] = 0.528$, $f_{cd} = 20.00 \text{ MPa}$
 $V_{rdmax} = 0.001 \times 1.00 \times 700 \times 0.9 \times 630 \times 0.528 \times 20.00 / 2.90 = 1445.3 \text{ kN}$
 $V_{ed} = 261.3 \text{ kN} < 1445.3 \text{ kN} = V_{rdmax}$, the check is verified

Minimum links for shear reinforcement (EC2 §9.2.2)
 Minimum shear reinforcement ratio $\rho_{w,min}$ (EC2 Eq.9.5N)
 $\rho_{w,min} = (0.08 \times f_{ck})^{0.5} / f_{yk}$, $f_{ck} = 30 \text{ N/mm}^2$, $f_{yk} = 500 \text{ N/mm}^2$, $\rho_{w,min} = 0.0009$
 $\min A_{sw}/s = 10 \times 0.0009 \times 700 \times \sin(90^\circ) = 6.30 \text{ cm}^2/\text{m}$
 Maximum longitudinal spacing of links $s_{lmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 470mm (EC2 §9.2.2.6, Eq.9.6N)
 Maximum transverse spacing of link legs $s_{tmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 470mm (§9.2.2.8, Eq.9.8N)
 Minimum shear reinforcement 2Ø12/47.0 ($A_{sw}/s = 9.62 \text{ cm}^2/\text{m}$)

Span-1 right Shear reinforcement: 2Ø12/47.0 ($A_{sw}/s = 9.62 \text{ cm}^2/\text{m}$)

7.4. Span-2 left Ultimate limit state (ULS), Design for shear

(EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc}

(EC2 §6.2.2)

$$V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d, \quad V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$$

(EC2 Eq.6.2.a,b)

$$C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120, \quad f_{ck} = 30 \text{ MPa}, \quad b_w = 700 \text{ mm}, \quad d = 630 \text{ mm}$$

$$k = 1 + \sqrt{(200/d)} \leq 2, \quad k = 1.56, \quad k_1 = 0.15$$

$$V_{rd,c}(\min) = 0.001 \times (0.37) \times 700 \times 630 = 163.17 \text{ kN}, \quad v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.37 \text{ N/mm}^2,$$

(EC2 Eq.6.3N)

$$\rho_1 = 1206 / (700 \times 630) = 0.0027, \quad V_{rdc} = 0.001 \times [0.120 \times 1.56 \times (0.27 \times 30)^{0.33}] \times 700 \times 630 = 165.80 \text{ kN}$$

$$V_{ed} = 143.71 \text{ kN} \leq V_{rdc} = 165.80 \text{ kN}, \quad \mathbf{V_{ed} \leq V_{rdc} \text{ shear reinforcement is not needed}}$$

Concrete strut capacity V_{rdmax}

(EC2 §6.2.3 Eq.6.9)

$$V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta), \quad V_{ed} / \max(V_{rdmax}) = 0.12, \quad \theta = 21.8^\circ \quad \cot \theta = 2.50 \quad \tan \theta = 0.40$$

$$\alpha_{cw} = 1.00 \quad z = 0.9d, \quad f_{ck} = 30.0 \leq 60 \text{ MPa} \quad v_1 = 0.6 [1 - f_{ck} / 250] = 0.6 [1 - 30 / 250] = 0.528, \quad f_{cd} = 20.00 \text{ MPa}$$

$$V_{rdmax} = 0.001 \times 1.00 \times 700 \times 0.9 \times 630 \times 0.528 \times 20.00 / 2.90 = 1445.3 \text{ kN}$$

$$V_{ed} = 261.3 \text{ kN} < 1445.3 \text{ kN} = V_{rdmax}, \text{ the check is verified}$$

Minimum links for shear reinforcement

(EC2 §9.2.2)

Minimum shear reinforcement ratio $\rho_{w,min}$

(EC2 Eq.9.5N)

$$\rho_{w,min} = (0.08 \times (f_{ck})^{0.5} / f_{yk}, \quad f_{ck} = 30 \text{ N/mm}^2, \quad f_{yk} = 500 \text{ N/mm}^2, \quad \rho_{w,min} = 0.0009$$

$$\min A_{sw}/s = 10 \times 0.0009 \times 700 \times \sin(90^\circ) = 6.30 \text{ cm}^2/\text{m}$$

$$\text{Maximum longitudinal spacing of links } s_{lmax} = 0.75d \quad (< 600 \text{ mm}) = 470 \text{ mm}$$

(EC2 §9.2.2.6, Eq.9.6N)

$$\text{Maximum transverse spacing of link legs } s_{tmax} = 0.75d \quad (< 600 \text{ mm}) = 470 \text{ mm}$$

(§9.2.2.8, Eq.9.8N)

$$\text{Minimum shear reinforcement } 2\emptyset 12/47.0 \quad (A_{sw}/s = 9.62 \text{ cm}^2/\text{m})$$

$$\mathbf{\text{Span-2 left Shear reinforcement: } 2\emptyset 12/47.0 \quad (A_{sw}/s = 9.62 \text{ cm}^2/\text{m})}$$

8. Serviceability limit state (SLS), Span-1

(EC2 EN1992-1-1:2004, §7)

$$L = 3.000 \text{ m}, \quad b = 0.700 \text{ m}, \quad h = 0.700 \text{ m}, \quad d = 0.630 \text{ m}$$

$$\text{Load (quasi-permanent combination)} \quad q_{ed} = g + \psi_2 \cdot q = 138.25 + 0.30 \times 0.00 = 138.25 \text{ kN/m}$$

$$L_{eff} = 3.000 \text{ m}, \quad M_{ed} = (138.25 / 186.64) \times 54.44 = 40.32 \text{ kNm}, \quad M_{ed}(SLS) = 40.32 \text{ kNm}$$

$$\text{Final creep coefficient } \phi(\infty, t_0) = 2.72$$

(EC2 §3.1.4, Annex B)

$$\text{Total shrinkage strain } \epsilon_{cs} = -0.30\%$$

$$\gamma_c = 1.00, \quad \gamma_s = 1.00$$

(EC2 §2.4.2.4.2)

$$\text{Modulus of elasticity of concrete } E_{cm} = 33.0 \text{ GPa}, \quad E_{eff} = 33.0 / (1 + 2.72) = 8.87 \text{ GPa} = 8870 \text{ MPa}$$

(EC2 Eq.7.20)

$$\text{Modulus of elasticity of steel } E_s = 200 \text{ GPa} = 200000 \text{ MPa}$$

$$\text{Modular ratio } E_s/E_c = 200/33.0 = 6.06, \quad \text{effective } E_s/E_{c,eff} = 200/8.87 = 22.55$$

$$\text{Tension reinforcement: } 6\emptyset 16 \quad (12.06 \text{ cm}^2), \quad \text{Compression reinforcement: } 6\emptyset 16 \quad (12.06 \text{ cm}^2)$$

$$\text{Reinforcement ratio } \rho = A_{s1} / (b \cdot d) = 1206 / (700 \times 630) = 0.0027, \quad \rho' = A_{s2} / (b \cdot d) = 1206 / (700 \times 630) = 0.0027$$

8.1. State I (uncracked section) (SLS), Span-1

$$\text{Bending stiffness of uncracked section, } EI = (200/22.55) \times (0.001 \times 25.223) = 223712 \text{ kNm}^2$$

$$A_i = A_c + (n-1)(A_{s1} + A_{s2}), \quad e = (n-1)(A_{s1} \cdot y_{1s} - A_{s2} \cdot y_{2s}) / A_i, \quad I = I_c + b \cdot h \cdot e^2 + (A_{s1} \cdot y_{1s}^2 + A_{s2} \cdot y_{2s}^2)(n-1)$$

$$S = A_s \cdot y_{2s} = (0.001) \times 2 \times 1206 \times 0.283 = (0.001) \times 0.342 \text{ m}^3, \quad y_2 = 353 \text{ mm}, \quad y_{2s} = y_2 - d = 353 - 70 = 283 \text{ mm} \quad (\text{EC2 Eq.7.21})$$

$$\text{Curvature due to moment } 1/r_M = 40.32 / 223712 = (0.001) \times 0.180 \quad (1/\text{m})$$

$$\text{Curvature due to shrinkage } 1/r_{cs} = (0.001 \times 0.30) \times 22.55 \times (0.342 / 25.223) = (0.001) \times 0.092 \quad (1/\text{m})$$

$$\text{Total curvature } 1/r = (0.001) \times 0.180 + (0.001) \times 0.092 = (0.001) \times 0.272 \quad (1/\text{m})$$

$$\text{Cracking moment, } M_{cr} = f_{ctm} \cdot (I / y_2) = 2.9 \times (25.223 / 0.353) = 207.01 \text{ kNm}$$

8.2. State II (fully cracked section) (SLS), Span-1

$$\rho = 0.0027, \quad \rho' = 0.0027, \quad \rho' / \rho = 1.000, \quad n = \alpha_e = 22.55, \quad n \cdot \rho = 0.061, \quad \xi = 0.669, \quad \alpha = 0.264, \quad x = \alpha \cdot d = 0.166 \text{ m}$$

$$\text{Bending stiffness of fully cracked section, } EI = \xi \cdot E_s \cdot A_s \cdot d^2 = 0.669 \times 200 \times 1206 \times 0.630^2 = 64076 \text{ kNm}^2$$

$$y_2 = (1 - \alpha) d = 464 \text{ mm}, \quad \epsilon_s = y_2 \cdot M / EI = (0.001) \times 464 \times 40.32 / 64076 = 0.29$$

$$S = A_s \cdot y_2 = (0.001) \times 2 \times 1206 \times 0.464 = (0.001) \times 0.559 \text{ m}^3$$

(EC2 Eq.7.21)

$$\text{Curvature due to moment } 1/r_M = 40.32 / 64076 = (0.001) \times 0.629 \quad (1/\text{m})$$

$$\text{Curvature due to shrinkage } 1/r_{cs} = (0.001 \times 0.30) \times 22.55 \times (0.559 / 7.224) = (0.001) \times 0.524 \quad (1/\text{m})$$

$$\text{Total curvature } 1/r = (0.001) \times 0.629 + (0.001) \times 0.524 = (0.001) \times 1.153 \quad (1/\text{m})$$

$$M_{ed} = 40.32 \text{ kNm}, \quad \epsilon_c / \epsilon_s = 0.10 / 0.29, \quad x = 166 \text{ mm}, \quad \sigma_s = 58 \text{ N/mm}^2$$

8.3. Checking deflections without calculation (SLS), Span-1

(EC2 §7.4.2)

$$1/d = K[11 + 1.5\sqrt{f_{ck}(\rho_o/\rho)} + 3.2\sqrt{f_{ck}(\rho_o/\rho - 1)}]^{3/2} = 68.93$$

(EC2 Eq.7.16a)

$$f_{ck} = 30.00 \text{ N/mm}^2, \rho_o = 0.001 \times \sqrt{30.00} = 0.0055, \rho = 0.0027, \rho' = 0.0014, \rho \leq \rho_o, K = 1.5$$

$$1/d = (310/\sigma_s) \times (1/d), \sigma_s = 58 \text{ N/mm}^2, 1/d = (310/58) \times 68.93 = 366.11$$

(EC2 Eq.7.17)

$$l_{eff}/d = 3.000/0.630 = 4.76 \leq 366.11, \text{ Span/depth under limits}$$

8.4. Checking deflections by calculation (SLS), Span-1

(EC2 §7.4.3)

$$M_{ed} = 40.32 < 0.70 \times M_{cr} = 0.70 \times 207.01 = 144.91 \text{ kNm}, \zeta = 0.00$$

(Eq.7.19)

$$\text{Final curvature } (1/r) = 0.00 \times (0.001 \times 1.153) + (1 - 0.00) \times (0.001 \times 0.272) = (0.001) \times 0.272 \text{ (1/m)}$$

(Eq.7.18)

$$\beta = (M_a + M_b)/M_c = (181.97 + 181.97)/54.44 = 6.69, k = 0.104(1 - 6.69/10) = 0.0345$$

$$f = k \cdot l_{eff}^2 \cdot (1/r) = 0.0345 \times 3.000^2 \times 0.272 = 0.08 \text{ mm}$$

$$f = 0.08 \text{ mm} \leq 1000 \times 3.000/250 = 12.00 \text{ mm}, \text{ Deflection under limits}$$

8.5. Minimum reinforcement areas (SLS)

(EC2 EN1992-1-1:2004, §7.3.2)

$$\text{Minimum reinforcement areas } A_{s,min} = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} / \sigma_s$$

(EC2 Eq.7.1)

$$b = 0.700 \text{ m}, b_{eff} = 0.700 \text{ m}, h = 0.700 \text{ m}, d = 0.630 \text{ m}, x = 0.166 \text{ m}, \varnothing = 16 \text{ mm}$$

$$N_{ed} = 0.00 \text{ kN}, \sigma_c = (N_{ed}/b h) = 0.00 \text{ N/mm}^2, \sigma_s = f_{yd} = 435 \text{ N/mm}^2$$

$$A_{ct} = (h - x) \cdot b = (700 - 166) \times 700 = 373601 \text{ mm}^2$$

$$\max(h, b_l) = 700 \text{ mm}, f_{ctm} = 2.90 \text{ N/mm}^2, A_{ct} = 373601 \text{ mm}^2, k = 0.72, k_c = 0.40, k_l = 1.50$$

$$\text{Minimum reinforcement, } A_{s,min} = 0.40 \times 0.72 \times 2.90 \times 373601 / 435 = 717 \text{ mm}^2$$

8.6. Calculation of crack width (SLS), Span-1

(EC2 EN1992-1-1:2004, §7.3.3)

$$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm})$$

(EC2 Eq.7.8)

$$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot (f_{ct,eff}/\rho_{eff}) (1 + \alpha_e \cdot \rho_{eff})] / E_s \geq 0.6 \sigma_s / E_s$$

(EC2 Eq.7.9)

$$\sigma_s = 58 \text{ N/mm}^2, \text{ short term loading: } E_s/E_c = 6.06, k_t = 0.6, \text{ long term loading: } E_s/E_c = 22.55, k_t = 0.4$$

$$2.5(h - d) = 175 \text{ mm}, (h - x)/3 = 178 \text{ mm}, h/2 = 350 \text{ mm}$$

$$A_{ceff} = 2.5(h - d)b = 2.5 \times (700 - 630) \times 700 = 122500 \text{ mm}^2$$

(\$7.3.2.3)

$$\rho_{eff} = A_s/A_{ceff} = 1206/122500 = 0.010$$

$$\epsilon_{sm} - \epsilon_{cm} = [58 - 0.4 \times (2.9/0.010) (1 + 22.55 \times 0.010)] / 200 = -0.43\% \geq 0.6 \times 58 / 200 = 0.18\%$$

$$s_{r,max} = k_3 \cdot (C_{nom} + \varnothing_s) + k_1 \cdot k_2 \cdot k_4 \cdot \varnothing / \rho_{eff}$$

(EC2 Eq.7.11)

$$\varnothing = 16 \text{ mm}, k_1 = 0.8, k_2 = (e_1 + e_2)/2e_1 = 0.5, k_3 = 3.4, k_4 = 0.425$$

$$s_{r,max} = 3.4 \times 62.00 + 0.8 \times 0.5 \times 0.425 \times 16 / 0.010 = 487.09 \text{ mm}$$

$$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = 487.09 \times 0.001 \times 0.18 = 0.09 \text{ mm}$$

$$w_k = 0.09 \text{ mm} \leq 0.30 \text{ mm} = w_{max}, \text{ Environmental class: XC3, Crack width under limit}$$

9. Serviceability limit state (SLS), Cant-Left

(EC2 EN1992-1-1:2004, §7)

$$L = 1.500 \text{ m}, b = 0.700 \text{ m}, h = 0.700 \text{ m}, d = 0.630 \text{ m}$$

$$\text{Load (quasi-permanent combination) } q_{ed} = g + \psi_2 \cdot q = 138.25 + 0.30 \times 0.00 = 138.25 \text{ kN/m}$$

$$l_{eff} = 1.500 \text{ m}, M_{ed} = (138.25/186.64) \times 181.97 = 134.79 \text{ kNm}, M_{ed}(SLS) = 134.79 \text{ kNm}$$

$$\text{Final creep coefficient } \phi(\infty, t_o) = 2.72$$

(EC2 §3.1.4, Annex B)

$$\text{Total shrinkage strain } \epsilon_{cs} = -0.30\%$$

$$\gamma_c = 1.00, \gamma_s = 1.00$$

(EC2 §2.4.2.4.2)

$$\text{Modulus of elasticity of concrete } E_{cm} = 33.0 \text{ GPa}, E_{ceff} = 33.0 / (1 + 2.72) = 8.87 \text{ GPa} = 8870 \text{ MPa}$$

(EC2 Eq.7.20)

$$\text{Modulus of elasticity of steel } E_s = 200 \text{ GPa} = 200000 \text{ MPa}$$

$$\text{Modular ratio } E_s/E_c = 200/33.0 = 6.06, \text{ effective } E_s/E_{c,eff} = 200/8.87 = 22.55$$

$$\text{Tension reinforcement: } 6\varnothing 16 \text{ (12.06 cm}^2\text{)}, \text{ Compression reinforcement: } 6\varnothing 16 \text{ (12.06 cm}^2\text{)}$$

$$\text{Reinforcement } 1/r = (0.001)/(b \cdot d) = 1206/(700 \times 630) = 0.0027, \rho' = A_{s2}/(b \cdot d) = 1206/(700 \times 630) = 0.0027$$

9.1. State I (uncracked section) (SLS), Cant-Left

$$\text{Bending stiffness of uncracked section, } EI = (200/22.55) \times (0.001 \times 25.223) = 223712 \text{ kNm}^2$$

$$A_i = A_c + (n - 1)(A_{s1} + A_{s2}), e = (n - 1)(A_{s1} \cdot y_{1s} - A_{s2} \cdot y_{2s})/A_i, I = I_c + b \cdot h \cdot e^2 + (A_{s1} \cdot y_{1s}^2 + A_{s2} \cdot y_{2s}^2)(n - 1)$$

$$S = A_s \cdot y_{2s} = (0.001) \times 2 \times 1206 \times 0.283 = (0.001) \times 0.342 \text{ m}^3, y_2 = 353 \text{ mm}, y_{2s} = y_2 - d = 353 - 70 = 283 \text{ mm}$$

(EC2 Eq.7.21)

$$\text{Curvature due to moment } 1/r_M = 134.79/223712 = (0.001) \times 0.603 \text{ (1/m)}$$

$$\text{Curvature due to shrinkage } 1/r_{cs} = (0.001 \times 0.30) \times 22.55 \times (0.342/25.223) = (0.001) \times 0.092 \text{ (1/m)}$$

$$\text{Total curvature } 1/r = (0.001) \times 0.603 + (0.001) \times 0.092 = (0.001) \times 0.694 \text{ (1/m)}$$

$$\text{Cracking moment, } M_{cr} = f_{ctm} \cdot (I/y_2) = 2.9 \times (25.223/0.353) = 207.01 \text{ kNm}$$

9.2. State II (fully cracked section) (SLS), Cant-Left

$\rho=0.0027$, $\rho'=0.0027$, $\rho'/\rho=1.000$, $n=\alpha_e=22.55$, $n\cdot\rho=0.061$, $\xi=0.669$, $\alpha=0.264$, $x=\alpha\cdot d=0.166m$
 Bending stiffness of fully cracked section, $EI=\xi\cdot E_s\cdot A_s\cdot d^2=0.669\times 200\times 1206\times 0.630^2=64076 \text{ kNm}^2$
 $y_2=(1-\alpha)d=464mm$, $\epsilon_s=y_2\cdot M/EI=(0.001)\times 464\times 134.79/64076=0.98$
 $S=A_s\cdot y_2=(0.001)\times 2\times 1206\times 0.464=(0.001)\times 0.559 \text{ m}^3$ (EC2 Eq.7.21)
 Curvature due to moment $1/r_M=134.79/64076=(0.001)\times 2.104 \text{ (1/m)}$
 Curvature due to shrinkage $1/r_{cs}=(0.001\times 0.30)\times 22.55\times (0.559/7.224)=(0.001)\times 0.524 \text{ (1/m)}$
 Total curvature $1/r=(0.001)\times 2.104+(0.001)\times 0.524=(0.001)\times 2.627 \text{ (1/m)}$
 $M_{ed}=134.79 \text{ kNm}$, $\epsilon_c/\epsilon_s=0.35/0.98$, $x=166mm$, $\sigma_s=195 \text{ N/mm}^2$

9.3. Checking deflections without calculation (SLS), Cant-Left

(EC2 §7.4.2)

$1/d=K[1+1.5\sqrt{f_{ck}(\rho_o/\rho)}+3.2\sqrt{f_{ck}(\rho_o/\rho-1)}]^{3/2}=18.38$ (EC2 Eq.7.16a)
 $f_{ck}=30.00N/mm^2$, $\rho_o=0.001\times\sqrt{30.00}=0.0055$, $\rho=0.0027$, $\rho'=0.0014$, $\rho\leq\rho_o$, $K=0.4$
 $1/d=(310/\sigma_s)\times(1/d)$, $\sigma_s=195 \text{ N/mm}^2$, $1/d=(310/195)\times 18.38=29.21$ (EC2 Eq.7.17)
 $l_{eff}/d=1.500/0.630=2.38 \leq 29.21$, **Span/depth under limits**

9.4. Checking deflections by calculation (SLS), Cant-Left

(EC2 §7.4.3)

$M_{ed}=134.79 < 0.70\times M_{cr}=0.70\times 207.01=144.91 \text{ kNm}$, $\zeta=0.00$ (Eq.7.19)
 Final curvature $(1/r)=(0.00\times(0.001\times 2.627)+(1-0.00)\times(0.001\times 0.694))=(0.001)\times 0.694 \text{ (1/m)}$ (Eq.7.18)
 $f=k\cdot l_{eff}^2\cdot(1/r)=0.2500\times 1.500^2\times 0.694=0.39 \text{ mm}$
 $f=0.39mm \leq 1000\times 1.500/250=6.00mm$, **Deflection under limits**

9.5. Minimum reinforcement areas (SLS)

(EC2 EN1992-1-1:2004, §7.3.2)

Minimum reinforcement areas $A_{s,min}=k_c\cdot k\cdot f_{ct,eff}\cdot A_{ct}/\sigma_s$ (EC2 Eq.7.1)
 $b=0.700m$, $b_{eff}=0.700m$, $h=0.700m$, $d=0.630m$, $x=0.166m$, $\varnothing=16mm$
 $N_{ed}=0.00kN$, $\sigma_c=(N_{ed}/bh)=0.0N/mm^2$, $\sigma_s=f_{yd}=435N/mm^2$
 $A_{ct}=(h-x)\cdot b=(700-166)\times 700=373601 \text{ mm}^2$
 $\max(h,b_l)=700mm$, $f_{ctm}=2.90N/mm^2$, $A_{ct}=373601mm^2$, $k=0.72$, $k_c=0.40$, $k_l=1.50$
 Minimum reinforcement, $A_{s,min}=0.40\times 0.72\times 2.90\times 373601/435=717mm^2$

9.6. Calculation of crack width (SLS), Cant-Left

(EC2 EN1992-1-1:2004, §7.3.3)

$w_k=s_r,\max\cdot(\epsilon_{sm}-\epsilon_{cm})$ (EC2 Eq.7.8)
 $\epsilon_{sm}-\epsilon_{cm}=[\sigma_s-k_t\cdot(f_{ct,eff}/\rho_{eff})(1+\alpha_e\cdot\rho_{eff})]/E_s \geq 0.6 \sigma_s/E_s$ (EC2 Eq.7.9)
 $\sigma_s=195N/mm^2$, short term loading: $E_s/E_c=6.06$, $k_t=0.6$, long term loading: $E_s/E_c=22.55$, $k_t=0.4$
 $2.5(h-d)=175 \text{ mm}$, $(h-x)/3=178 \text{ mm}$, $h/2=350 \text{ mm}$
 $A_{ceff}=2.5(h-d)b=2.5\times(700-630)\times 700=122500 \text{ mm}^2$ (§7.3.2.3)
 $\rho_{eff}=A_s/A_{ceff}=1206/122500=0.010$
 $\epsilon_{sm}-\epsilon_{cm}=[195-0.4\times(2.9/0.010)(1+22.55\times 0.010)]/200=0.26\% \geq 0.6\times 195/200=0.59\%$
 $s_r,\max=k_3\cdot(C_{nom}+\varnothing_s)+k_1\cdot k_2\cdot k_4\cdot \varnothing/\rho_{eff}$ (EC2 Eq.7.11)
 $\varnothing=16mm$, $k_1=0.8$, $k_2=(e_1+e_2)/2e_1=0.5$, $k_3=3.4$, $k_4=0.425$
 $s_r,\max=3.4\times 62.00+0.8\times 0.5\times 0.425\times 16/0.010=487.09 \text{ mm}$
 $w_k=s_r,\max\cdot(\epsilon_{sm}-\epsilon_{cm})=487.09\times 0.001\times 0.59=0.29 \text{ mm}$
 $w_k=0.29mm \leq 0.30mm=w_{max}$, Environmental class: XC3, **Crack width under limit**

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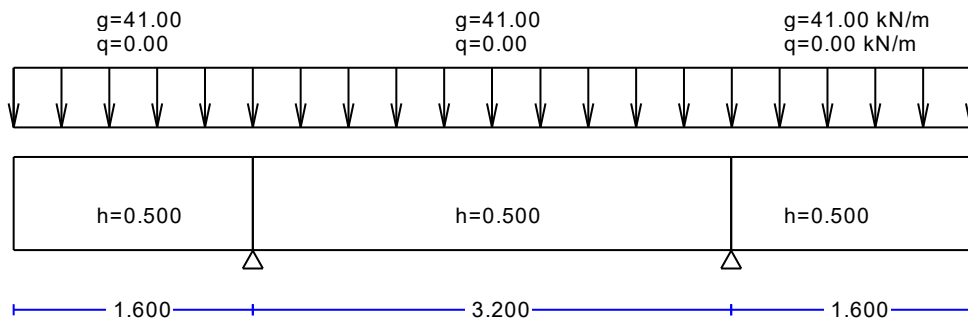
Šolski center NM 27/05/2024

1. Pilotna greda 50x50 PS-2

Continuous beam with distributed loads

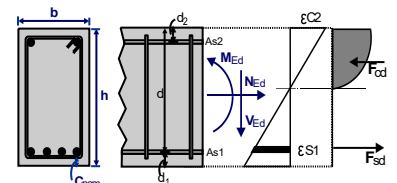
(EC2 EN1992-1-1:2004, EC0 EN1990:2002,)

C30/37 - S500



Reinforced concrete design

Concrete-Steel class: C30/37-S500 (EC2 §3)
 Environmental class : XC4 (EC2 §4.4.1)
 Concrete cover : $C_{nom}=50$ mm (EC2 §4.4.1)
 Concrete weight : 25.0 kN/m³
 $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N)
 $f_{cd}=\alpha_{cc} \cdot f_{ck} / \gamma_c = 1.00 \times 30 / 1.50 = 20.00$ MPa (EC2 §3.1.6)
 $f_{ctd}=\alpha_{ct} \cdot f_{ctk0.05} / \gamma_c = 1.00 \times 2.0 / 1.50 = 1.33$ MPa (EC2 §3.1.6)
 $f_{yd}=f_{yk} / \gamma_s = 500 / 1.15 = 435$ MPa (EC2 §3.2.7)
 Modulus of elasticity of concrete $E_{cm}=33.0$ GPa



2. Dimensions and loads

Continuous beam (rectangular section), number of spans=1
 Partial safety factors for actions : $\gamma_G=1.35$, $\gamma_Q=1.50$ (EC0 Annex A1)
 Combination of variable actions : $\psi_0=0.70$, $\psi_1=0.60$, $\psi_2=0.30$
 Effective depth of cross section $d=h-d_1$, $d_1=C_{nom}+\phi_s+0.5\phi_s=50+8+0.5 \times 16=66$ mm
 beam self weight = $b_w \cdot h \times 25.0$

Spans, widths, thickness, load on spans (g =self weight +dead, q =live)

Cant-1 $L=1.60$ m $b_w=0.500$ m $b_{eff}=0.500$ m $h=0.500$ m $g=6.25+41.00=47.25$ kN/m $q=0.00$ kN/m
 Span-1 $L=3.20$ m $b_w=0.500$ m $b_{eff}=0.500$ m $h=0.500$ m $g=6.25+41.00=47.25$ kN/m $q=0.00$ kN/m
 Cant-2 $L=1.60$ m $b_w=0.500$ m $b_{eff}=0.500$ m $h=0.500$ m $g=6.25+41.00=47.25$ kN/m $q=0.00$ kN/m

3. Ultimate limit state (ULS), design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Load (STR) Span-1 $q_{ed}=\gamma_G \cdot g + \gamma_Q \cdot q = 1.35g + 1.50q = 1.35 \times 47.25 + 1.50 \times 0.00 = 63.79$ kN/m

Cross section values (area A, moment of inertia Iyy, centroid zc)

Cant-1 $L=1.60$ m, $A=0.25000$ m² (2.50×10^5 mm²), $I_{yy}=0.00521$ m⁴ (5.21×10^9 mm⁴), $z_c=0.000$ m (0 mm)
 Span-1 $L=3.20$ m, $A=0.25000$ m² (2.50×10^5 mm²), $I_{yy}=0.00521$ m⁴ (5.21×10^9 mm⁴), $z_c=0.000$ m (0 mm)
 Cant-2 $L=1.60$ m, $A=0.25000$ m² (2.50×10^5 mm²), $I_{yy}=0.00521$ m⁴ (5.21×10^9 mm⁴), $z_c=0.000$ m (0 mm)

4. Shearing forces and bending moments

Maximum bending moments at spans for load combinations 1.35g+1.50q

Cant-1, Med= -81.65 kNm, $x_0=1.600$ m, $x_1=0.000$ m, $x_2=0.000$ m

Span-1, Med= 21.17 kNm, $x_0=1.600$ m, $x_1=0.785$ m, $x_2=0.785$ m

Cant-2, Med= -81.65 kNm, $x_0=0.000$ m, $x_1=0.000$ m, $x_2=0.000$ m

Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0, Med= -81.65 kNm, $x_1=1.600$ m, $x_2=0.000$ m

Support-1, Med= -81.65 kNm, $x_1=3.200$ m, $x_2=1.600$ m

Maximum shear forces for load combinations 1.35g+1.50q

Cant-1, Ved,A= 0.00 kN, Ved,B=-102.06 kN

Span-1, Ved,A= 102.06 kN, Ved,B=-102.06 kN

Cant-2, Ved,A= 102.06 kN, Ved,B= 0.00 kN

Maximum reactions due to dead and live loads (Rg and Rq)

Support-0, Rg($x_1.35$)= 204.12 kN, Rq($x_1.50$)= 0.00 kN

Support-1, Rg($x_1.35$)= 204.12 kN, Rq($x_1.50$)= 0.00 kN



5. Design actions, shearing forces and bending moments

Design action values after moment redistribution by 0%

Reduction of support values to values at support faces ($b_{sup}=0.20$ m)

Check for minimum values, ($0.65q_{l2/8}$, $0.65q_{l2/12}$)

Maximum span bending moments and shear forces for load combinations 1.35g+1.50q

Cant-1, Med= -71.44 kNm, Ved,A= 6.38 kN, Ved,B= -95.68 kN

Span-1, Med= 21.17 kNm, Ved,A= 95.68 kN, Ved,B= -95.68 kN

Cant-2, Med= -71.44 kNm, Ved,A= 95.68 kN, Ved,B= 6.38 kN

Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0, Med= -71.44 kNm, $x_1=1.600$ m, $x_2=0.000$ m

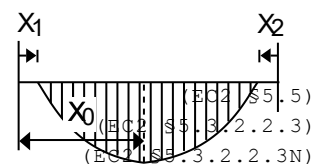
Support-1, Med= -71.44 kNm, $x_1=3.200$ m, $x_2=1.600$ m

Maximum shear forces at distance d from support face 1.35g+1.50q

Cant-1, $b/2+d=0.534$ m, $1.35g+1.50q=63.79$ kN/m, Ved,A= 0.00 kN, Ved,B= 68.00 kN

Span-1, $b/2+d=0.534$ m, $1.35g+1.50q=63.79$ kN/m, Ved,A= 68.00 kN, Ved,B= 68.00 kN

Cant-2, $b/2+d=0.534$ m, $1.35g+1.50q=63.79$ kN/m, Ved,A= 68.00 kN, Ved,B= 0.00 kN



6. Ultimate limit state (ULS), design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.2.1)

6.1. Span-1 Ultimate limit state (ULS), design for bending

(EC2 §6.1, §9.2.1)

Effective depth of cross section $d_1=C_{nom}+\phi_s+0.5\phi=50+8+0.5\cdot16=66$ mm, $d_2=66$ mm, $d=500-66=434$ mm

Reinforcement for bending (only tension reinforcement is needed)

Med= 21.17 kNm $b_w=500$ mm $d=434$ mm $K_d=6.67$ $x/d=0.04$ $\epsilon_{c2}/\epsilon_{s1}=-0.7/20.0$ $k_s=2.33$, **As1= 1.14 cm²**

Minimum longitudinal tension reinf., $A_s \geq 0.26 b d \cdot f_{ctm} / f_{yk}$, ($A_{s,min}=3.27$ cm²) (EC2 §9.2.1.1.1)

Maximum tension or compression reinf., $A_s \leq 0.04 A_c$, ($A_{s,max}=100.0$ cm²) (EC2 §9.2.1.1.3)

Reinforcement for bending: 4Ø14 (6.16 cm²) (bottom), 4Ø14 (6.16 cm²) (top)

Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

$b=500$ mm, $h=500$ mm, $d=434$ mm, $A_{s1}=616$ mm², $A_{s2}=616$ mm²

$\epsilon_{c2}=-2.60\%$, $\epsilon_{s1}=20.00\%$, $A_{s1}/b \cdot d=0.00393$ (0.393%)

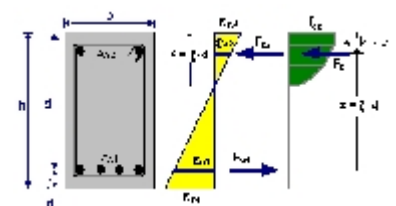
$x/d=\epsilon_{c2}/(\epsilon_{c2}+\epsilon_{s1})=2.60/(2.60+20.00)=0.115$, $x=49.9$ mm

$\alpha_r=0.744$, $k_a=0.394$, $F_c=\alpha_r \cdot b \cdot x \cdot f_{cd}=F_{s1}=371.27$ kN, $A_{s1}=F_{s1}/f_{yd}=853$ mm²

$z=d-k_a \cdot x=(1-k_a \cdot \epsilon_{c2}/(\epsilon_{c2}+\epsilon_{s1}))d$, $z/d=1.0-0.394 \cdot 0.115=0.955$, $z=414.3$ mm,

$K_{d2}=1/(0.744 \cdot 0.115 \cdot 0.955 \cdot 20.00)=0.612$ mm²/N, $K_d=0.782$

Bending capacity $M_r=b \cdot d^2/K_{d2}=[10^{-6}] \times 500 \times 434^2/0.612=154.00$ kNm



6.2. Support-0 Ultimate limit state (ULS), design for bending

(EC2 §6.1, §9.2.1)

Effective depth of cross section $d_1 = C_{nom} + \varnothing s + 0.5\varnothing = 50 + 8 + 0.5 \times 16 = 66 \text{ mm}$, $d_2 = 66 \text{ mm}$, $d = 500 - 66 = 434 \text{ mm}$
 Reinforcement for bending (only tension reinforcement is needed)
 $M_{ed} = -71.44 \text{ kNm}$ $b_w = 500 \text{ mm}$ $d = 434 \text{ mm}$ $K_d = 3.63$ $x/d = 0.07$ $\varepsilon_{c2}/\varepsilon_{s1} = -1.5/20.0$ $k_s = 2.36$, **As2= 3.88cm²**
 Minimum longitudinal tension reinf., $A_s \geq 0.26 b d \cdot f_{ctm}/f_{yk}$, ($A_{s,min} = 3.27 \text{ cm}^2$) (EC2 §9.2.1.1.1)
 Maximum tension or compression reinf., $A_s \leq 0.04 A_c$, ($A_{s,max} = 100.0 \text{ cm}^2$) (EC2 §9.2.1.1.3)
Reinforcement for bending: 4Ø14 (6.16cm²) (top), 4Ø14 (6.16cm²) (bottom)

Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

$b = 500 \text{ mm}$, $h = 500 \text{ mm}$, $d = 434 \text{ mm}$, $A_{s1} = 616 \text{ mm}^2$, $A_{s2} = 616 \text{ mm}^2$
 $\varepsilon_{c2} = -2.60\%$, $\varepsilon_{s1} = 20.00\%$, $A_{s1}/b \cdot d = 0.00393 (0.393\%)$
 $x/d = \varepsilon_{c2}/(\varepsilon_{c2} + \varepsilon_{s1}) = 2.60/(2.60 + 20.00) = 0.115$, $x = 49.9 \text{ mm}$
 $\alpha_r = 0.744$, $k_a = 0.394$, $F_c = \alpha_r \cdot b \cdot x \cdot f_{cd} = F_{s1} = 371.27 \text{ kN}$, $A_{s1} = F_{s1}/f_{yd} = 853 \text{ mm}^2$
 $z = d - k_a \cdot x = ([1 - k_a \cdot \varepsilon_{c2}/(\varepsilon_{c2} + \varepsilon_{s1})]d)$, $z/d = 1.0 - 0.394 \times 0.115 = 0.955$, $z = 414.3 \text{ mm}$,
 $K_d = 1/(0.744 \cdot 0.115 \cdot 0.955 \cdot 20.00) = 0.612 \text{ mm}^2/\text{N}$, $K_d = 0.782$
 Bending capacity $M_r = b \cdot d^2 / K_d = [10^{-6}] \times 500 \times 434^2 / 0.612 = 154.00 \text{ kNm}$

6.3. Support-1 Ultimate limit state (ULS), design for bending

(EC2 §6.1, §9.2.1)

Effective depth of cross section $d_1 = C_{nom} + \varnothing s + 0.5\varnothing = 50 + 8 + 0.5 \times 16 = 66 \text{ mm}$, $d_2 = 66 \text{ mm}$, $d = 500 - 66 = 434 \text{ mm}$
 Reinforcement for bending (only tension reinforcement is needed)
 $M_{ed} = -71.44 \text{ kNm}$ $b_w = 500 \text{ mm}$ $d = 434 \text{ mm}$ $K_d = 3.63$ $x/d = 0.07$ $\varepsilon_{c2}/\varepsilon_{s1} = -1.5/20.0$ $k_s = 2.36$, **As2= 3.88cm²**
 Minimum longitudinal tension reinf., $A_s \geq 0.26 b d \cdot f_{ctm}/f_{yk}$, ($A_{s,min} = 3.27 \text{ cm}^2$)
 Maximum tension or compression reinf., $A_s \leq 0.04 A_c$, ($A_{s,max} = 100.0 \text{ cm}^2$)
Reinforcement for bending: 4Ø14 (6.16cm²) (top), 4Ø14 (6.16cm²) (bottom)

Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

$b = 500 \text{ mm}$, $h = 500 \text{ mm}$, $d = 434 \text{ mm}$, $A_{s1} = 616 \text{ mm}^2$, $A_{s2} = 616 \text{ mm}^2$
 $\varepsilon_{c2} = -2.60\%$, $\varepsilon_{s1} = 20.00\%$, $A_{s1}/b \cdot d = 0.00393 (0.393\%)$
 $x/d = \varepsilon_{c2}/(\varepsilon_{c2} + \varepsilon_{s1}) = 2.60/(2.60 + 20.00) = 0.115$, $x = 49.9 \text{ mm}$
 $\alpha_r = 0.744$, $k_a = 0.394$, $F_c = \alpha_r \cdot b \cdot x \cdot f_{cd} = F_{s1} = 371.27 \text{ kN}$, $A_{s1} = F_{s1}/f_{yd} = 853 \text{ mm}^2$
 $z = d - k_a \cdot x = ([1 - k_a \cdot \varepsilon_{c2}/(\varepsilon_{c2} + \varepsilon_{s1})]d)$, $z/d = 1.0 - 0.394 \times 0.115 = 0.955$, $z = 414.3 \text{ mm}$,
 $K_d = 1/(0.744 \cdot 0.115 \cdot 0.955 \cdot 20.00) = 0.612 \text{ mm}^2/\text{N}$, $K_d = 0.782$
 Bending capacity $M_r = b \cdot d^2 / K_d = [10^{-6}] \times 500 \times 434^2 / 0.612 = 154.00 \text{ kNm}$

7. Ultimate limit state (ULS), Design for shear

(EC2 EN1992-1-1:2004, §6.2, §9.2.2)

7.1. Span-0 right Ultimate limit state (ULS), Design for shear

(EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc} (EC2 §6.2.2)
 $V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$, $V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ (EC2 Eq.6.2.a,b)
 $C_{rdc} = 0.18/\gamma_c = 0.18/1.50 = 0.120$, $f_{ck} = 30 \text{ MPa}$, $b_w = 500 \text{ mm}$, $d = 434 \text{ mm}$
 $k = 1 + \sqrt{(200/d)} \leq 2$, $k = 1.68$, $k_1 = 0.15$
 $V_{rdc}(min) = 0.001 \times (0.42) \times 500 \times 434 = 91.14 \text{ kN}$, $v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.42 \text{ N/mm}^2$, (EC2 Eq.6.3N)
 $\rho_1 = 616/(500 \times 434) = 0.0028$, $V_{rdc} = 0.001 \times [0.120 \times 1.68 \times (0.28 \times 30)^{0.33}] \times 500 \times 434 = 88.93$, $V_{rdc} = 91.14 \text{ kN}$
 $V_{ed} = 68.00 \text{ kN} \leq V_{rdc} = 91.14 \text{ kN}$, **Ved < Vrdc shear reinforcement is not needed**

Concrete strut capacity V_{rdmax}

(EC2 §6.2.3 Eq.6.9)

$V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta)$, $V_{ed}/\max(V_{rdmax}) = 0.09$, $\theta = 21.8^\circ$ $\cot \theta = 2.50$ $\tan \theta = 0.40$
 $\alpha_{cw} = 1.00$ $z = 0.9d$, $f_{ck} = 30.0 \leq 60 \text{ MPa}$ $v_1 = 0.6[1 - f_{ck}/250] = 0.6[1 - 30/250] = 0.528$, $f_{cd} = 20.00 \text{ MPa}$
 $V_{rdmax} = 0.001 \times 1.00 \times 500 \times 0.9 \times 434 \times 0.528 \times 20.00 / 2.90 = 711.2 \text{ kN}$
 $V_{ed} = 95.7 \text{ kN} < 711.2 \text{ kN} = V_{rdmax}$, the check is verified

Minimum links for shear reinforcement (EC2 §9.2.2)
 Minimum shear reinforcement ratio $\rho_{w,min}$ (EC2 Eq.9.5N)
 $\rho_{w,min} = (0.08 \times (f_{ck})^{0.5} / f_{yk}, f_{ck} = 30 \text{ N/mm}^2, f_{yk} = 500 \text{ N/mm}^2, \rho_{w,min} = 0.0009$
 $\min A_{sw}/s = 10 \times 0.0009 \times 500 \times \sin(90^\circ) = 4.50 \text{ cm}^2/\text{m}$
 Maximum longitudinal spacing of links $s_{lmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 325mm (EC2 §9.2.2.6, Eq.9.6N)
 Maximum transverse spacing of link legs $s_{tmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 325mm (§9.2.2.8, Eq.9.8N)
 Minimum shear reinforcement $2\emptyset 8/32.5$ ($A_{sw}/s = 6.19 \text{ cm}^2/\text{m}$)

Span-0 right Shear reinforcement: $2\emptyset 8/32.5$ ($A_{sw}/s = 6.19 \text{ cm}^2/\text{m}$)

7.2. Span-1 left Ultimate limit state (ULS), Design for shear (EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc} (EC2 §6.2.2)
 $V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d, V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ (EC2 Eq.6.2.a,b)
 $C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120, f_{ck} = 30 \text{ MPa}, b_w = 500 \text{ mm}, d = 434 \text{ mm}$
 $k = 1 + \sqrt{(200/d)} \leq 2, k = 1.68, k_1 = 0.15$
 $V_{rd,c(min)} = 0.001 \times (0.42) \times 500 \times 434 = 91.14 \text{ kN}, v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.42 \text{ N/mm}^2, (EC2 Eq.6.3N)$
 $\rho_1 = 616 / (500 \times 434) = 0.0028, V_{rdc} = 0.001 \times [0.120 \times 1.68 \times (0.28 \times 30)^{0.33}] \times 500 \times 434 = 88.93, V_{rdc} = 91.14 \text{ kN}$
 $V_{ed} = 68.00 \text{ kN} \leq V_{rdc} = 91.14 \text{ kN}, V_{ed} \leq V_{rdc}$ shear reinforcement is not needed

Concrete strut capacity V_{rdmax} (EC2 §6.2.3 Eq.6.9)
 $V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta), V_{ed} / \max(V_{rdmax}) = 0.09, \theta = 21.8^\circ \cot \theta = 2.50 \tan \theta = 0.40$
 $\alpha_{cw} = 1.00, z = 0.9d, f_{ck} = 30.0 \leq 60 \text{ MPa}, v_1 = 0.6 [1 - f_{ck} / 250] = 0.6 [1 - 30 / 250] = 0.528, f_{cd} = 20.00 \text{ MPa}$
 $V_{rdmax} = 0.001 \times 1.00 \times 500 \times 0.9 \times 434 \times 0.528 \times 20.00 / 2.90 = 711.2 \text{ kN}$
 $V_{ed} = 95.7 \text{ kN} < 711.2 \text{ kN} = V_{rdmax}$, the check is verified

Minimum links for shear reinforcement (EC2 §9.2.2)
 Minimum shear reinforcement ratio $\rho_{w,min}$ (EC2 Eq.9.5N)
 $\rho_{w,min} = (0.08 \times (f_{ck})^{0.5} / f_{yk}, f_{ck} = 30 \text{ N/mm}^2, f_{yk} = 500 \text{ N/mm}^2, \rho_{w,min} = 0.0009$
 $\min A_{sw}/s = 10 \times 0.0009 \times 500 \times \sin(90^\circ) = 4.50 \text{ cm}^2/\text{m}$
 Maximum longitudinal spacing of links $s_{lmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 325mm (EC2 §9.2.2.6, Eq.9.6N)
 Maximum transverse spacing of link legs $s_{tmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 325mm (§9.2.2.8, Eq.9.8N)
 Minimum shear reinforcement $2\emptyset 8/32.5$ ($A_{sw}/s = 6.19 \text{ cm}^2/\text{m}$)

Span-1 left Shear reinforcement: $2\emptyset 8/32.5$ ($A_{sw}/s = 6.19 \text{ cm}^2/\text{m}$)

7.3. Span-1 right Ultimate limit state (ULS), Design for shear (EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc} (EC2 §6.2.2)
 $V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d, V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ (EC2 Eq.6.2.a,b)
 $C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120, f_{ck} = 30 \text{ MPa}, b_w = 500 \text{ mm}, d = 434 \text{ mm}$
 $k = 1 + \sqrt{(200/d)} \leq 2, k = 1.68, k_1 = 0.15$
 $V_{rd,c(min)} = 0.001 \times (0.42) \times 500 \times 434 = 91.14 \text{ kN}, v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.42 \text{ N/mm}^2, (EC2 Eq.6.3N)$
 $\rho_1 = 616 / (500 \times 434) = 0.0028, V_{rdc} = 0.001 \times [0.120 \times 1.68 \times (0.28 \times 30)^{0.33}] \times 500 \times 434 = 88.93, V_{rdc} = 91.14 \text{ kN}$
 $V_{ed} = 68.00 \text{ kN} \leq V_{rdc} = 91.14 \text{ kN}, V_{ed} \leq V_{rdc}$ shear reinforcement is not needed

Concrete strut capacity V_{rdmax} (EC2 §6.2.3 Eq.6.9)
 $V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta), V_{ed} / \max(V_{rdmax}) = 0.09, \theta = 21.8^\circ \cot \theta = 2.50 \tan \theta = 0.40$
 $\alpha_{cw} = 1.00, z = 0.9d, f_{ck} = 30.0 \leq 60 \text{ MPa}, v_1 = 0.6 [1 - f_{ck} / 250] = 0.6 [1 - 30 / 250] = 0.528, f_{cd} = 20.00 \text{ MPa}$
 $V_{rdmax} = 0.001 \times 1.00 \times 500 \times 0.9 \times 434 \times 0.528 \times 20.00 / 2.90 = 711.2 \text{ kN}$
 $V_{ed} = 95.7 \text{ kN} < 711.2 \text{ kN} = V_{rdmax}$, the check is verified

Minimum links for shear reinforcement (EC2 §9.2.2)
 Minimum shear reinforcement ratio $\rho_{w,min}$ (EC2 Eq.9.5N)
 $\rho_{w,min} = (0.08 \times (f_{ck})^{0.5} / f_{yk}, f_{ck} = 30 \text{ N/mm}^2, f_{yk} = 500 \text{ N/mm}^2, \rho_{w,min} = 0.0009$
 $\min A_{sw}/s = 10 \times 0.0009 \times 500 \times \sin(90^\circ) = 4.50 \text{ cm}^2/\text{m}$
 Maximum longitudinal spacing of links $s_{lmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 325mm (EC2 §9.2.2.6, Eq.9.6N)
 Maximum transverse spacing of link legs $s_{tmax} = 0.75d$ ($\leq 600 \text{ mm}$) = 325mm (§9.2.2.8, Eq.9.8N)
 Minimum shear reinforcement $2\emptyset 8/32.5$ ($A_{sw}/s = 6.19 \text{ cm}^2/\text{m}$)

Span-1 right Shear reinforcement: $2\emptyset 8/32.5$ ($A_{sw}/s = 6.19 \text{ cm}^2/\text{m}$)

7.4. Span-2 left Ultimate limit state (ULS), Design for shear

(EC2 §6.2, §9.2.2)

Shear capacity without shear reinforcement V_{rdc}

(EC2 §6.2.2)

$$V_{rdc} = [C_{rdc} \cdot k \cdot (100 \rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d, \quad V_{rdc} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$$

(EC2 Eq.6.2.a,b)

$$C_{rdc} = 0.18 / \gamma_c = 0.18 / 1.50 = 0.120, \quad f_{ck} = 30 \text{ MPa}, \quad b_w = 500 \text{ mm}, \quad d = 434 \text{ mm}$$

$$k = 1 + \sqrt{(200/d)} \leq 2, \quad k = 1.68, \quad k_1 = 0.15$$

$$V_{rd,c}(\min) = 0.001 \times (0.42) \times 500 \times 434 = 91.14 \text{ kN}, \quad v_{min} = 0.0350 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.42 \text{ N/mm}^2,$$

(EC2 Eq.6.3N)

$$\rho_1 = 616 / (500 \times 434) = 0.0028, \quad V_{rdc} = 0.001 \times [0.120 \times 1.68 \times (0.28 \times 30)^{0.33}] \times 500 \times 434 = 88.93, \quad V_{rdc} = 91.14 \text{ kN}$$

$$V_{ed} = 68.00 \text{ kN} \leq V_{rdc} = 91.14 \text{ kN}, \quad \mathbf{V_{ed} \leq V_{rdc} \text{ shear reinforcement is not needed}}$$

Concrete strut capacity V_{rdmax}

(EC2 §6.2.3 Eq.6.9)

$$V_{rdmax} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta), \quad V_{ed} / \max(V_{rdmax}) = 0.09, \quad \theta = 21.8^\circ \quad \cot \theta = 2.50 \quad \tan \theta = 0.40$$

$$\alpha_{cw} = 1.00 \quad z = 0.9d, \quad f_{ck} = 30.0 \leq 60 \text{ MPa} \quad v_1 = 0.6 [1 - f_{ck}/250] = 0.6 [1 - 30/250] = 0.528, \quad f_{cd} = 20.00 \text{ MPa}$$

$$V_{rdmax} = 0.001 \times 1.00 \times 500 \times 0.9 \times 434 \times 0.528 \times 20.00 / 2.90 = 711.2 \text{ kN}$$

$$V_{ed} = 95.7 \text{ kN} < 711.2 \text{ kN} = V_{rdmax}, \text{ the check is verified}$$

Minimum links for shear reinforcement

(EC2 §9.2.2)

Minimum shear reinforcement ratio $\rho_{w,min}$

(EC2 Eq.9.5N)

$$\rho_{w,min} = (0.08 \times (f_{ck})^{0.5} / f_{yk}, \quad f_{ck} = 30 \text{ N/mm}^2, \quad f_{yk} = 500 \text{ N/mm}^2, \quad \rho_{w,min} = 0.0009$$

$$\min A_{sw}/s = 10 \times 0.0009 \times 500 \times \sin(90^\circ) = 4.50 \text{ cm}^2/\text{m}$$

$$\text{Maximum longitudinal spacing of links } s_{lmax} = 0.75d \quad (< 600 \text{ mm}) = 325 \text{ mm}$$

(EC2 §9.2.2.6, Eq.9.6N)

$$\text{Maximum transverse spacing of link legs } s_{tmax} = 0.75d \quad (< 600 \text{ mm}) = 325 \text{ mm}$$

(§9.2.2.8, Eq.9.8N)

$$\text{Minimum shear reinforcement } 2\emptyset 8/32.5 \quad (A_{sw}/s = 6.19 \text{ cm}^2/\text{m})$$

$$\mathbf{\text{Span-2 left Shear reinforcement: } 2\emptyset 8/32.5 \quad (A_{sw}/s = 6.19 \text{ cm}^2/\text{m})}$$

8. Serviceability limit state (SLS), Span-1

(EC2 EN1992-1-1:2004, §7)

$$L = 3.200 \text{ m}, \quad b = 0.500 \text{ m}, \quad h = 0.500 \text{ m}, \quad d = 0.434 \text{ m}$$

$$\text{Load (quasi-permanent combination)} \quad q_{ed} = g + \psi_2 \cdot q = 47.25 + 0.30 \times 0.00 = 47.25 \text{ kN/m}$$

$$L_{eff} = 3.200 \text{ m}, \quad M_{ed} = (47.25/63.79) \times 21.17 = 15.68 \text{ kNm}, \quad M_{ed}(SLS) = 15.68 \text{ kNm}$$

$$\text{Final creep coefficient } \phi(\infty, t_0) = 2.50$$

(EC2 §3.1.4, Annex B)

$$\text{Total shrinkage strain } \epsilon_{cs} = -0.30\%$$

$$\gamma_c = 1.00, \quad \gamma_s = 1.00$$

(EC2 §2.4.2.4.2)

$$\text{Modulus of elasticity of concrete } E_{cm} = 33.0 \text{ GPa}, \quad E_{eff} = 33.0 / (1 + 2.50) = 9.43 \text{ GPa} = 9430 \text{ MPa}$$

(EC2 Eq.7.20)

$$\text{Modulus of elasticity of steel } E_s = 200 \text{ GPa} = 200000 \text{ MPa}$$

$$\text{Modular ratio } E_s/E_c = 200/33.0 = 6.06, \quad \text{effective } E_s/E_{c,eff} = 200/9.43 = 21.21$$

$$\text{Tension reinforcement: } 4\emptyset 14 \quad (6.16 \text{ cm}^2), \quad \text{Compression reinforcement: } 4\emptyset 14 \quad (6.16 \text{ cm}^2)$$

$$\text{Reinforcement ratio } \rho = A_{s1}/(b \cdot d) = 616 / (500 \times 434) = 0.0028, \quad \rho' = A_{s2}/(b \cdot d) = 616 / (500 \times 434) = 0.0028$$

8.1. State I (uncracked section) (SLS), Span-1

$$\text{Bending stiffness of uncracked section, } EI = (200/21.21) \times (0.001 \times 6.405) = 60400 \text{ kNm}^2$$

$$A_i = A_c + (n-1)(A_{s1} + A_{s2}), \quad e = (n-1)(A_{s1} \cdot y_{1s} - A_{s2} \cdot y_{2s}) / A_i, \quad I = I_c + b \cdot h \cdot e^2 + (A_{s1} \cdot y_{1s}^2 + A_{s2} \cdot y_{2s}^2)(n-1)$$

$$S = A_s \cdot y_{2s} = (0.001) \times 2 \times 616 \times 0.187 = (0.001) \times 0.115 \text{ m}^3, \quad y_2 = 253 \text{ mm}, \quad y_{2s} = y_2 - d/2 = 253 - 66 = 187 \text{ mm}$$

(EC2 Eq.7.21)

$$\text{Curvature due to moment } 1/r_M = 15.68/60400 = (0.001) \times 0.260 \quad (1/\text{m})$$

$$\text{Curvature due to shrinkage } 1/r_{cs} = (0.001 \times 0.30) \times 21.21 \times (0.115/6.405) = (0.001) \times 0.114 \quad (1/\text{m})$$

$$\text{Total curvature } 1/r = (0.001) \times 0.260 + (0.001) \times 0.114 = (0.001) \times 0.374 \quad (1/\text{m})$$

$$\text{Cracking moment, } M_{cr} = f_{ctm} \cdot (I/y_2) = 2.9 \times (6.405/0.253) = 73.43 \text{ kNm}$$

8.2. State II (fully cracked section) (SLS), Span-1

$$\rho = 0.0028, \quad \rho' = 0.0028, \quad \rho'/\rho = 1.000, \quad n = \alpha_e = 21.21, \quad n \cdot \rho = 0.059, \quad \xi = 0.672, \quad \alpha = 0.262, \quad x = \alpha \cdot d = 0.114 \text{ m}$$

$$\text{Bending stiffness of fully cracked section, } EI = \xi \cdot E_s \cdot A_s \cdot d^2 = 0.672 \times 200 \times 616 \times 0.434^2 = 15590 \text{ kNm}^2$$

$$y_2 = (1 - \alpha)d = 320 \text{ mm}, \quad \epsilon_s = y_2 \cdot M / EI = (0.001) \times 320 \times 15.68 / 15590 = 0.32$$

$$S = A_s \cdot y_2 = (0.001) \times 2 \times 616 \times 0.320 = (0.001) \times 0.197 \text{ m}^3$$

(EC2 Eq.7.21)

$$\text{Curvature due to moment } 1/r_M = 15.68/15590 = (0.001) \times 1.006 \quad (1/\text{m})$$

$$\text{Curvature due to shrinkage } 1/r_{cs} = (0.001 \times 0.30) \times 21.21 \times (0.197/1.653) = (0.001) \times 0.760 \quad (1/\text{m})$$

$$\text{Total curvature } 1/r = (0.001) \times 1.006 + (0.001) \times 0.760 = (0.001) \times 1.765 \quad (1/\text{m})$$

$$M_{ed} = 15.68 \text{ kNm}, \quad \epsilon_c/\epsilon_s = 0.11/0.32, \quad x = 114 \text{ mm}, \quad \sigma_s = 64 \text{ N/mm}^2$$

8.3. Checking deflections without calculation (SLS), Span-1

(EC2 §7.4.2)

$$1/d = K[11 + 1.5\sqrt{f_{ck}(\rho/\rho)} + 3.2\sqrt{f_{ck}(\rho/\rho-1)}]^{3/2} = 65.19$$

(EC2 Eq.7.16a)

$$f_{ck} = 30.00 \text{ N/mm}^2, \rho_o = 0.001 \times \sqrt{30.00} = 0.0055, \rho = 0.0028, \rho' = 0.0014, \rho \leq \rho_o, K = 1.5$$

$$1/d = (310/\sigma_s) \times (1/d), \sigma_s = 64 \text{ N/mm}^2, 1/d = (310/64) \times 65.19 = 313.53$$

(EC2 Eq.7.17)

$$l_{eff}/d = 3.200/0.434 = 7.37 \leq 313.53, \text{ Span/depth under limits}$$

8.4. Checking deflections by calculation (SLS), Span-1

(EC2 §7.4.3)

$$M_{ed} = 15.68 < 0.70 \times M_{cr} = 0.70 \times 73.43 = 51.40 \text{ kNm}, \zeta = 0.00$$

(Eq.7.19)

$$\text{Final curvature } (1/r) = 0.00 \times (0.001 \times 1.765) + (1 - 0.00) \times (0.001 \times 0.374) = (0.001) \times 0.374 \text{ (1/m)}$$

(Eq.7.18)

$$\beta = (M_a + M_b) / M_c = (71.44 + 71.44) / 21.17 = 6.75, k = 0.104 (1 - 6.75/10) = 0.0338$$

$$f = k \cdot l_{eff}^2 \cdot (1/r) = 0.0338 \times 3.200^2 \times 0.374 = 0.13 \text{ mm}$$

$$f = 0.13 \text{ mm} \leq 1000 \times 3.200 / 250 = 12.80 \text{ mm}, \text{ Deflection under limits}$$

8.5. Minimum reinforcement areas (SLS)

(EC2 EN1992-1-1:2004, §7.3.2)

$$\text{Minimum reinforcement areas } A_{s,min} = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} / \sigma_s$$

(EC2 Eq.7.1)

$$b = 0.500 \text{ m}, b_{eff} = 0.500 \text{ m}, h = 0.500 \text{ m}, d = 0.434 \text{ m}, x = 0.114 \text{ m}, \varnothing = 14 \text{ mm}$$

$$N_{ed} = 0.00 \text{ kN}, \sigma_c = (N_{ed}/bh) = 0.00 \text{ N/mm}^2, \sigma_s = f_{yd} = 435 \text{ N/mm}^2$$

$$A_{ct} = (h - x) \cdot b = (500 - 114) \times 500 = 193211 \text{ mm}^2$$

$$\max(h, b_l) = 500 \text{ mm}, f_{ctm} = 2.90 \text{ N/mm}^2, A_{ct} = 193211 \text{ mm}^2, k = 0.86, k_c = 0.40, k_l = 1.50$$

$$\text{Minimum reinforcement, } A_{s,min} = 0.40 \times 0.86 \times 2.90 \times 193211 / 435 = 443 \text{ mm}^2$$

8.6. Calculation of crack width (SLS), Span-1

(EC2 EN1992-1-1:2004, §7.3.3)

$$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm})$$

(EC2 Eq.7.8)

$$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot (f_{ct,eff} / \rho_{eff}) (1 + \alpha_e \cdot \rho_{eff})] / E_s \geq 0.6 \sigma_s / E_s$$

(EC2 Eq.7.9)

$$\sigma_s = 64 \text{ N/mm}^2, \text{ short term loading: } E_s / E_c = 6.06, k_t = 0.6, \text{ long term loading: } E_s / E_c = 21.21, k_t = 0.4$$

$$2.5(h - d) = 165 \text{ mm}, (h - x) / 3 = 129 \text{ mm}, h / 2 = 250 \text{ mm}$$

$$A_{ceff} = 0.333(h - x)b = 0.333 \times (500 - 114) \times 500 = 64339 \text{ mm}^2$$

(§7.3.2.3)

$$\rho_{eff} = A_s / A_{ceff} = 616 / 64339 = 0.010$$

$$\epsilon_{sm} - \epsilon_{cm} = [64 - 0.4 \times (2.9 / 0.010) (1 + 21.21 \times 0.010)] / 200 = -0.41\% \geq 0.6 \times 64 / 200 = 0.19\%$$

$$s_{r,max} = k_3 \cdot (C_{nom} + \varnothing_s) + k_1 \cdot k_2 \cdot k_4 \cdot \varnothing / \rho_{eff}$$

(EC2 Eq.7.11)

$$\varnothing = 14 \text{ mm}, k_1 = 0.8, k_2 = (e_1 + e_2) / 2e_1 = 0.5, k_3 = 3.4, k_4 = 0.425$$

$$s_{r,max} = 3.4 \times 58.00 + 0.8 \times 0.5 \times 0.425 \times 14 / 0.010 = 445.78 \text{ mm}$$

$$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = 445.78 \times 0.001 \times 0.19 = 0.09 \text{ mm}$$

$$w_k = 0.09 \text{ mm} \leq 0.30 \text{ mm} = w_{max}, \text{ Environmental class: XC4, Crack width under limit}$$

9. Serviceability limit state (SLS), Cant-Left

(EC2 EN1992-1-1:2004, §7)

$$L = 1.600 \text{ m}, b = 0.500 \text{ m}, h = 0.500 \text{ m}, d = 0.434 \text{ m}$$

$$\text{Load (quasi-permanent combination) } q_{ed} = g + \psi_2 \cdot q = 47.25 + 0.30 \times 0.00 = 47.25 \text{ kN/m}$$

$$l_{eff} = 1.600 \text{ m}, M_{ed} = (47.25 / 63.79) \times 71.44 = 52.92 \text{ kNm}, M_{ed}(SLS) = 52.92 \text{ kNm}$$

$$\text{Final creep coefficient } \phi(\infty, t_o) = 2.50$$

(EC2 §3.1.4, Annex B)

$$\text{Total shrinkage strain } \epsilon_{cs} = -0.30\%$$

$$\gamma_c = 1.00, \gamma_s = 1.00$$

(EC2 §2.4.2.4.2)

$$\text{Modulus of elasticity of concrete } E_{cm} = 33.0 \text{ GPa}, E_{ceff} = 33.0 / (1 + 2.50) = 9.43 \text{ GPa} = 9430 \text{ MPa}$$

(EC2 Eq.7.20)

$$\text{Modulus of elasticity of steel } E_s = 200 \text{ GPa} = 200000 \text{ MPa}$$

$$\text{Modular ratio } E_s / E_c = 200 / 33.0 = 6.06, \text{ effective } E_s / E_c = 200 / 9.43 = 21.21$$

$$\text{Tension reinforcement: } 4\varnothing 14 \text{ (6.16 cm}^2\text{)}, \text{ Compression reinforcement: } 4\varnothing 14 \text{ (6.16 cm}^2\text{)}$$

$$\text{Reinforcement ratio } \rho = A_{s1} / (b \cdot d) = 616 / (500 \times 434) = 0.0028, \rho' = A_{s2} / (b \cdot d) = 616 / (500 \times 434) = 0.0028$$

9.1. State I (uncracked section) (SLS), Cant-Left

$$\text{Bending stiffness of uncracked section, } EI = (200 / 21.21) \times (0.001 \times 6.405) = 60400 \text{ kNm}^2$$

$$A_i = A_c + (n - 1)(A_{s1} + A_{s2}), e = (n - 1)(A_{s1} \cdot y_{1s} - A_{s2} \cdot y_{2s}) / A_i, I = I_c + b \cdot h \cdot e^2 + (A_{s1} \cdot y_{1s}^2 + A_{s2} \cdot y_{2s}^2) (n - 1)$$

$$S = A_s \cdot y_{2s} = (0.001) \times 2 \times 616 \times 0.187 = (0.001) \times 0.115 \text{ m}^3, y_2 = 253 \text{ mm}, y_{2s} = y_2 - d = 253 - 66 = 187 \text{ mm}$$

(EC2 Eq.7.21)

$$\text{Curvature due to moment } 1/r_M = 52.92 / 60400 = (0.001) \times 0.876 \text{ (1/m)}$$

$$\text{Curvature due to shrinkage } 1/r_{cs} = (0.001 \times 0.30) \times 21.21 \times (0.115 / 6.405) = (0.001) \times 0.114 \text{ (1/m)}$$

$$\text{Total curvature } 1/r = (0.001) \times 0.876 + (0.001) \times 0.114 = (0.001) \times 0.991 \text{ (1/m)}$$

$$\text{Cracking moment, } M_{cr} = f_{ctm} \cdot (I / y_2) = 2.9 \times (6.405 / 0.253) = 73.43 \text{ kNm}$$

9.2. State II (fully cracked section) (SLS), Cant-Left

$\rho=0.0028$, $\rho'=0.0028$, $\rho'/\rho=1.000$, $n=\alpha_e=21.21$, $n\cdot\rho=0.059$, $\xi=0.672$, $\alpha=0.262$, $x=\alpha\cdot d=0.114\text{m}$
 Bending stiffness of fully cracked section, $EI=\xi\cdot E_s\cdot A_s\cdot d^2=0.672\times 200\times 616\times 0.434^2=15590\text{ kNm}^2$
 $y_2=(1-\alpha)d=320\text{mm}$, $\epsilon_s=y_2\cdot M/EI=(0.001)\times 320\times 52.92/15590=1.09$
 $S=A_s\cdot y_2=(0.001)\times 2\times 616\times 0.320=(0.001)\times 0.197\text{ m}^3$ (EC2 Eq.7.21)
 Curvature due to moment $1/r_M=52.92/15590=(0.001)\times 3.394\text{ (1/m)}$
 Curvature due to shrinkage $1/r_{cs}=(0.001\times 0.30)\times 21.21\times (0.197/1.653)=(0.001)\times 0.760\text{ (1/m)}$
 Total curvature $1/r=(0.001)\times 3.394+(0.001)\times 0.760=(0.001)\times 4.154\text{ (1/m)}$
 $M_{ed}=52.92\text{ kNm}$, $\epsilon_c/\epsilon_s=0.39/1.09$, $x=114\text{mm}$, $\sigma_s=218\text{ N/mm}^2$

9.3. Checking deflections without calculation (SLS), Cant-Left

(EC2 §7.4.2)

$1/d=K[1+1.5\sqrt{f_{ck}(\rho_o/\rho)}+3.2\sqrt{f_{ck}(\rho_o/\rho-1)}]^{3/2}=17.38$ (EC2 Eq.7.16a)
 $f_{ck}=30.00\text{N/mm}^2$, $\rho_o=0.001\times\sqrt{30.00}=0.0055$, $\rho=0.0028$, $\rho'=0.0014$, $\rho\leq\rho_o$, $K=0.4$
 $1/d=(310/\sigma_s)\times(1/d)$, $\sigma_s=218\text{ N/mm}^2$, $1/d=(310/218)\times 17.38=24.77$ (EC2 Eq.7.17)
 $l_{eff}/d=1.600/0.434=3.69\leq 24.77$, **Span/depth under limits**

9.4. Checking deflections by calculation (SLS), Cant-Left

(EC2 §7.4.3)

$\zeta=1-0.50\cdot(M_{cr}/M_{ed})^2=1-0.50\times(73.43/52.92)^2=0.04$ (Eq.7.19)
 Final curvature $(1/r)=(0.04\times(0.001\times 4.154)+(1-0.04)\times(0.001\times 0.991))=(0.001)\times 1.109\text{ (1/m)}$ (Eq.7.18)
 $f=k\cdot l_{eff}^2\cdot(1/r)=0.2500\times 1.600^2\times 1.109=0.71\text{ mm}$
 $f=0.71\text{mm}\leq 1000\times 1.600/250=6.40\text{mm}$, **Deflection under limits**

9.5. Minimum reinforcement areas (SLS)

(EC2 EN1992-1-1:2004, §7.3.2)

Minimum reinforcement areas $A_{s,min}=k_c\cdot k\cdot f_{ct,eff}\cdot A_{ct}/\sigma_s$ (EC2 Eq.7.1)
 $b=0.500\text{m}$, $b_{eff}=0.500\text{m}$, $h=0.500\text{m}$, $d=0.434\text{m}$, $x=0.114\text{m}$, $\varnothing=14\text{mm}$
 $N_{ed}=0.00\text{kN}$, $\sigma_c=(N_{ed}/bh)=0.0\text{N/mm}^2$, $\sigma_s=f_{yd}=435\text{N/mm}^2$
 $A_{ct}=(h-x)\cdot b=(500-114)\times 500=193211\text{ mm}^2$
 $\max(h,b_l)=500\text{mm}$, $f_{ctm}=2.90\text{N/mm}^2$, $A_{ct}=193211\text{mm}^2$, $k=0.86$, $k_c=0.40$, $k_l=1.50$
 Minimum reinforcement, $A_{s,min}=0.40\times 0.86\times 2.90\times 193211/435=443\text{mm}^2$

9.6. Calculation of crack width (SLS), Cant-Left

(EC2 EN1992-1-1:2004, §7.3.3)

$w_k=s_r\cdot\max(\epsilon_{sm}-\epsilon_{cm})$ (EC2 Eq.7.8)
 $\epsilon_{sm}-\epsilon_{cm}=[\sigma_s-k_t\cdot(f_{ct,eff}/\rho_{eff})(1+\alpha_e\cdot\rho_{eff})]/E_s\geq 0.6\sigma_s/E_s$ (EC2 Eq.7.9)
 $\sigma_s=218\text{N/mm}^2$, short term loading: $E_s/E_c=6.06$, $k_t=0.6$, long term loading: $E_s/E_c=21.21$, $k_t=0.4$
 $2.5(h-d)=165\text{ mm}$, $(h-x)/3=129\text{ mm}$, $h/2=250\text{ mm}$
 $A_{ceff}=0.333(h-x)b=0.333\times(500-114)\times 500=64339\text{ mm}^2$ (§7.3.2.3)
 $\rho_{eff}=A_s/A_{ceff}=616/64339=0.010$
 $\epsilon_{sm}-\epsilon_{cm}=[218-0.4\times(2.9/0.010)(1+21.21\times 0.010)]/200=0.36\text{‰}\geq 0.6\times 218/200=0.65\text{‰}$
 $s_r\cdot\max=k_3\cdot(C_{nom}+\varnothing_s)+k_1\cdot k_2\cdot k_4\cdot\varnothing/\rho_{eff}$ (EC2 Eq.7.11)
 $\varnothing=14\text{mm}$, $k_1=0.8$, $k_2=(e_1+e_2)/2e_1=0.5$, $k_3=3.4$, $k_4=0.425$
 $s_r\cdot\max=3.4\times 58.00+0.8\times 0.5\times 0.425\times 14/0.010=445.78\text{ mm}$
 $w_k=s_r\cdot\max\cdot(\epsilon_{sm}-\epsilon_{cm})=445.78\times 0.001\times 0.65=0.29\text{ mm}$
 $w_k=0.29\text{mm}\leq 0.30\text{mm}=w_{max}$, Environmental class: XC4, **Crack width under limit**

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3 RISBE

	Vsebina	merilo	oznaka
1	Gradbena situacija	M 1 : 250	G.1
2	Zakoličbena situacija	M 1 : 250	G.2
3	Prečni prerez gradbene jame	M 1 : 100	G.3
4	Vzdolžni profili pilotnih sten	M 1 : 100	G.4
5	Armaturne risbe: PILOTI Ø50cm	M 1 : 50	G.5
6	Armaturne risbe: PILOTI Ø40cm	M 1 : 50	G.6
7	Armaturne risbe: PILOTNE GREDE 50x50cm	M 1 : 50	G.7
8	Armaturne risbe: PILOTNE GREDE 70x70cm	M 1 : 50	G.8

Temelji med polovico osi 6 do 7 stopničasti

SIDRANA PILOTNA STENA PS-1
L=25.50m, piloti ø0.5m/1.0m, AB GREDA 0.7x0.7m,
GEOTEHNIČNA SIDRA 3x6"/251kN,
dolžina kampade 5.1 m

temelji PR: 195.95

temelji K: 192.16

Dno gradbene jame,
odmik od objekta = 1,75m,
kota dna = 192,10 m n.v

192.10

BERLINSKA STENA
L=23.42m, TIRNICE S-49/0.75m, AB GREDA 0.3x0.3m

temelji vhod: 193.53

OPCIJSKA KONZOLNA PILOTNA STENA
pred izvedbo preveriti temeljna tla objekta
športna dvorana Leona Štuklja
L=8.50m, piloti ø0.5m/1.0m, AB GREDA 0.7x0.7m

temelji: 190.02

temelji K: 194.13

temelji PR: 197.63

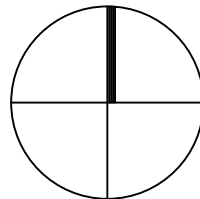
SIDRANA PILOTNA STENA PS-2
L=34.00m, piloti ø0.4m/0.8m, AB GREDA 0.5x0.5m,
IBO PASIVNA SIDRA ø32/250, L=12.0m
piloti izvedeni nad dnom gradbene jame in sidrani z
IBO PASIVNA SIDRA ø32/250, L=6.0m

Gradbiščna cesta, š=4m,
vzdolžni naklon=20%

Športna dvorana - Šolski center Novo mesto

02 GRADBENA SITUACIJA
Gradbena situacija zaščite gradbene jame

merilo: 1 : 250



Dispozicija sider:

POZICIJA	L prosti [m]	L vezni [m]	Tip sidra	Naklon
S1	6.00	6.00	Trivrvno 3x6"/251 kN	35°
S2	6.00	6.00	Trivrvno 3x6"/251 kN	35°
S3	6.00	7.00	Trivrvno 3x6"/251 kN	35°
S4	6.00	7.00	Trivrvno 3x6"/251 kN	35°
S5	6.00	8.00	Trivrvno 3x6"/251 kN	35°
S6	6.00	8.00	Trivrvno 3x6"/251 kN	35°
S7	6.00	9.00	Trivrvno 3x6"/251 kN	35°



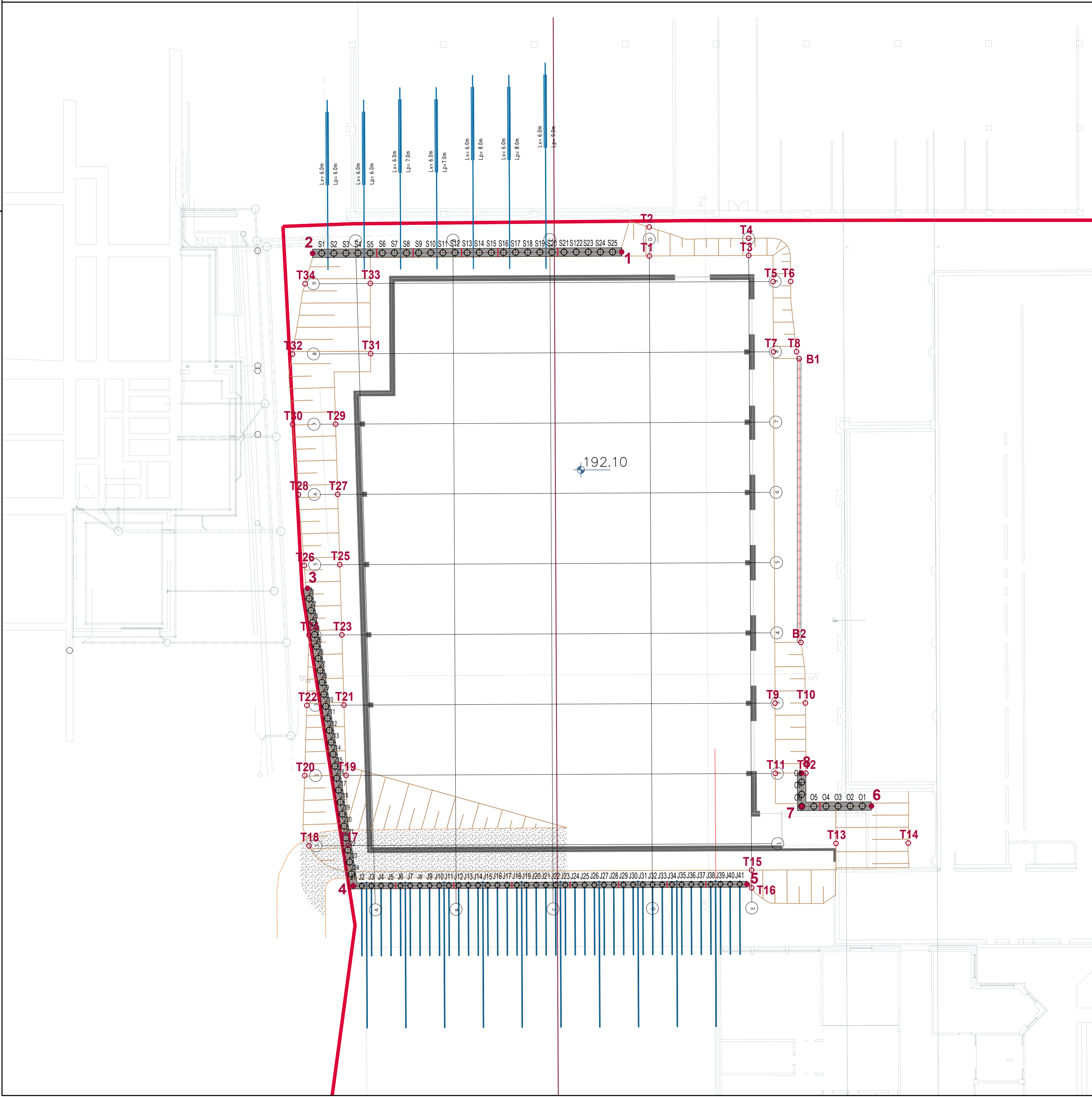
MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE,
MASARYKOVA CESTA 16, 1000 LJUBLJANA

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podjetje za geotehniko, infrastrukturo in raziskave
tovarniška c. 26, 5270 Ajdovščina | +386(0)5 8500740 | info@acap.si



vodja projekta:	MATEJ SOMRAK, univ.dipl.inž.arh.	ZAPS A-1746
pooblaščen inženir:	ANDRAŽ CEKET, univ.dipl.inž.grad.	IZS PI G-2435
izdelal:	ANDREJ BRICMAN, dipl.inž.geol. (UN)	
investitor:	Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana	
vrsta projekta:	PZI	
vrsta načrta:	2.1 Načrt zaščite gradbene jame	
naziv objekta:	Športna dvorana - Šolski center Novo mesto	
vsebina risbe:	02 GRADBENA SITUACIJA Gradbena situacija zaščite gradbene jame	

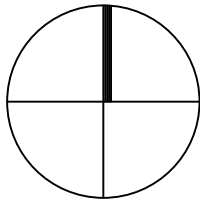
datum:	št. projekta:	št. načrta:	merilo:	št. risbe:
05.2024	028-23	028-23-21	1 : 250	G.1



Športna dvorana - Šolski center Novo mesto

06 ZAKOLIČBENA SITUACIJA
Zakoličbena situacija

merilo: 1 : 250



Zakoličba konstrukcij:

POZICIJA	X	Y	Z
1	512329.5222	72856.1566	193.91
2	512304.0224	72856.0375	196.46
3	512303.6044	72828.3868	196.84
4	512307.3798	72803.8231	197.25
5	512339.8719	72803.9627	194.44
6	512350.1338	72810.4141	193.57
7	512344.3857	72810.3875	193.57
8	512344.3728	72813.1374	193.57
B1	512344.1692	72847.34108	
B2	512344.3192	72823.92329	

Zakoličba izkopov in gradbene jame:

POZICIJA	X	Y	POZICIJA	X	Y
T1	512331.809	72855.8233	T18	512303.7263	72807.13007
T2	512331.7978	72858.21288	T19	512306.785	72812.94456
T3	512340.0089	72855.86206	T20	512303.3728	72812.92847
T4	512340.0023	72857.2793	T21	512306.6119	72818.75367
T5	512342.0186	72853.71115	T22	512303.5535	72818.72939
T6	512343.4711	72853.718	T23	512306.4394	72824.54306
T7	512342.046	72847.91122	T24	512303.736	72824.53031
T8	512343.9549	72847.92022	T25	512306.2666	72830.34231
T9	512342.1831	72818.91154	T26	512303.3434	72830.26597
T10	512344.6835	72818.92333	T27	512306.0939	72836.14156
T11	512342.2105	72813.11161	T28	512302.847	72836.12625
T12	512344.7218	72813.12345	T29	512305.9211	72841.94081
T13	512347.2041	72807.33509	T30	512302.3722	72841.92407
T14	512353.1723	72807.36323	T31	512308.8118	72847.7545
T15	512340.2482	72805.12853	T32	512302.3611	72847.72409
T16	512340.2551	72803.65586	T33	512308.7845	72853.55444
T17	512306.9575	72807.15517	T34	512303.4037	72853.52907



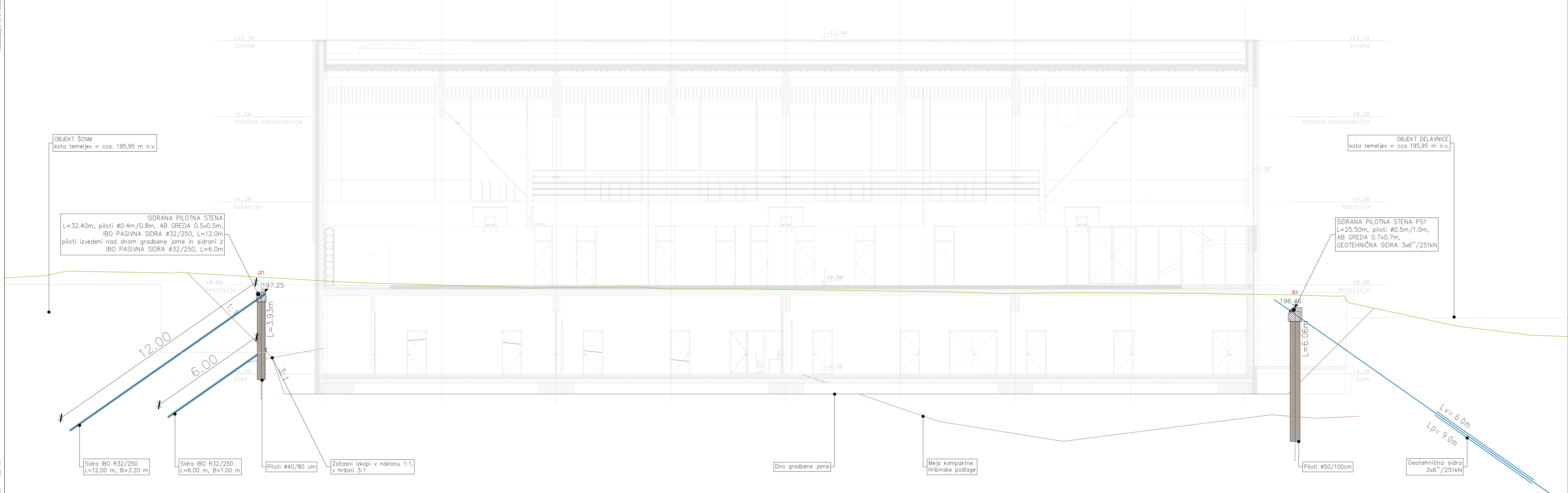
MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE,
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podjetje za geotehniko, infrastrukturo in raziskave
tovarniška c. 26, 5270 ajdovščina | +386(0)5 8500740 | info@acap.si



vodja projekta:	MATEJ SOMRAK, univ.dipl.inž.arh.	ZAPS A-1746
pooblaščen inženir:	ANDRAŽ CEKET, univ.dipl.inž.grad.	IZS PI G-2435
izdelal:	ANDREJ BRICMAN, dipl.inž.geol. (UN)	
investitor:	Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana	
vrsta projekta:	PZI	
vrsta načrta:	2.1 Načrt zaščite gradbene jame	
naziv objekta:	Športna dvorana - Šolski center Novo mesto	
vsebina risbe:	06 ZAKOLIČBENA SITUACIJA Zakoličbena situacija	



datum:	št. projekta:	št. načrta:	merilo:	št. risbe:
05.2024	028-23	028-23-21	1 : 250	G.2

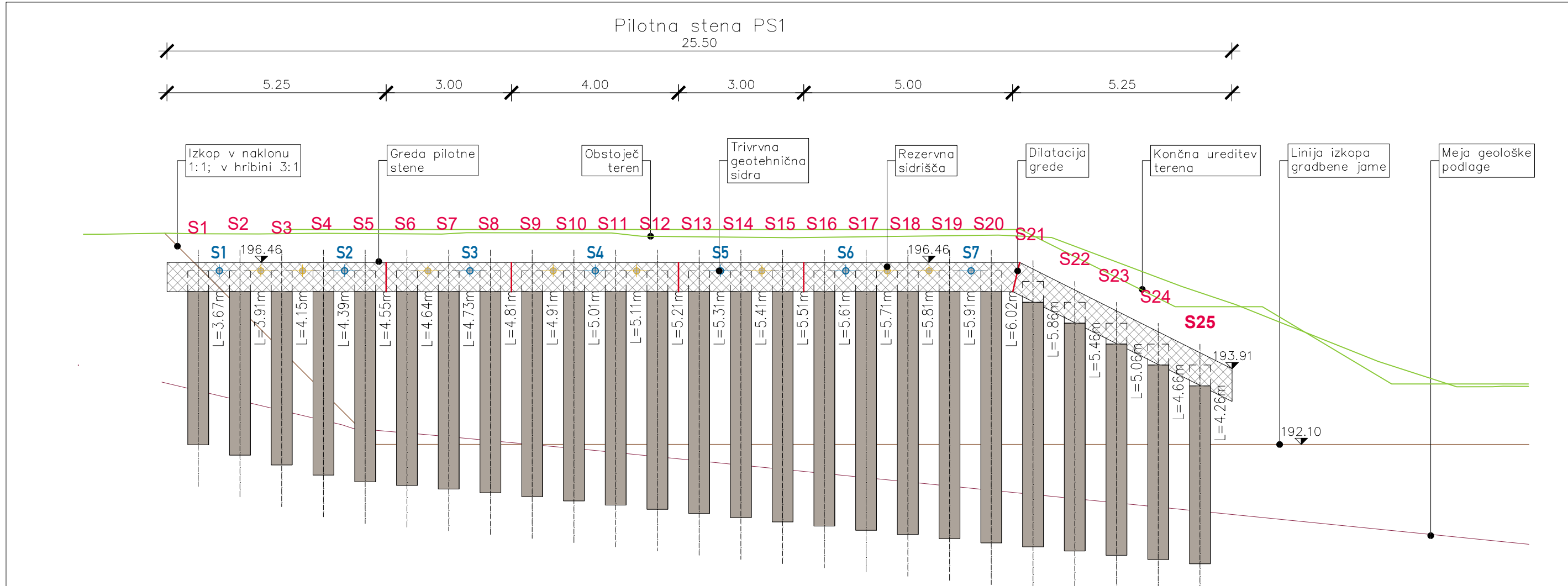
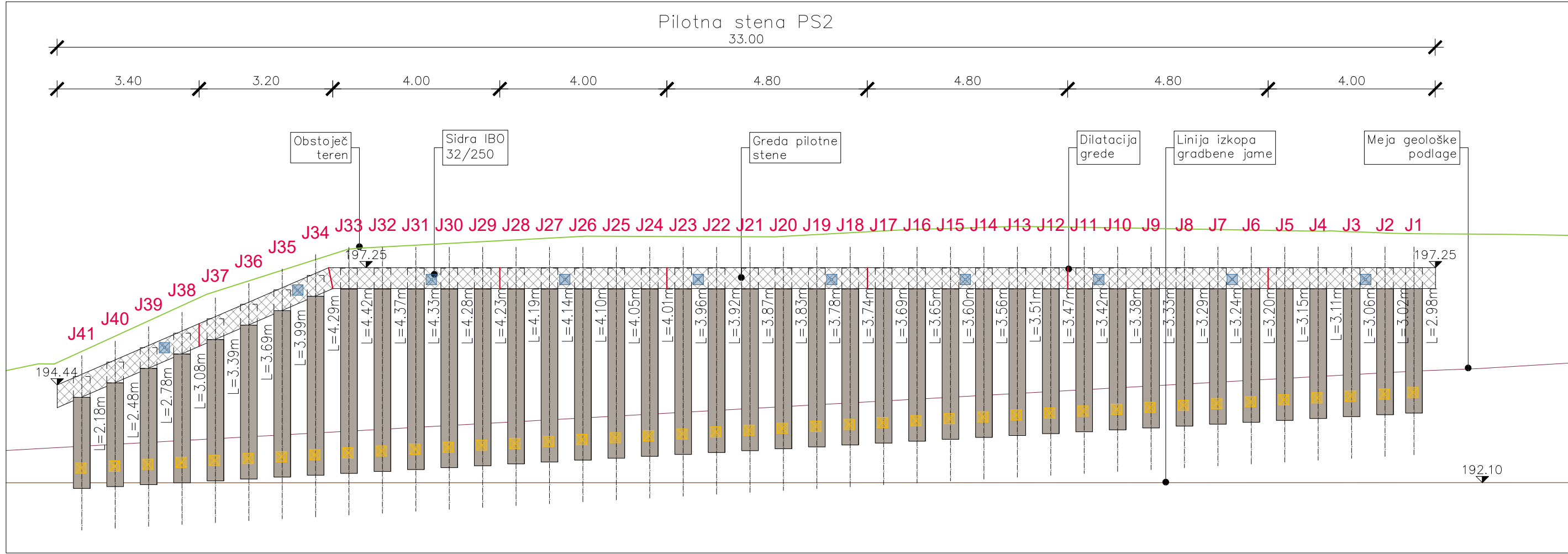
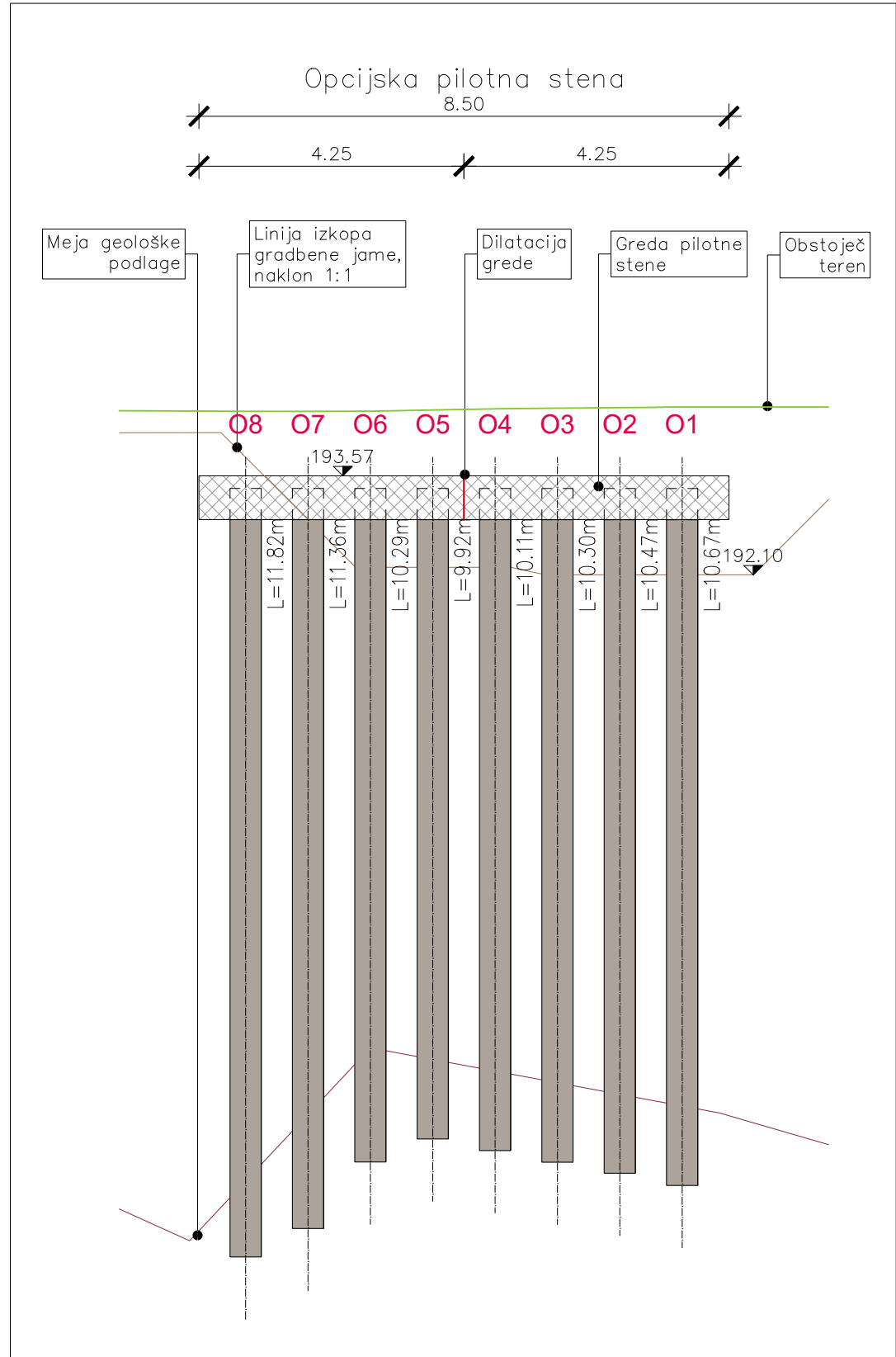


Športna dvorana - Šolski center Novo mesto

32 PREČNI PREREZI
Prečni prerez gradbene jame

merilo: 1 : 100/100

	MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE, MASARYKOVA CESTA 16, 1000 LJUBLJANA	AC&P inženirski biro d.o.o. podjetje za geotehniko, infrastrukturo in raziskave tovarniška c. 26, 5270 ajdovščina +386(0)5 8500740 info@acsp.si	
vodja projekta:	MATEJ SOMRAK, univ.dipl.inž.arh.	ZAPS A-1746	
pooblaščen inženir:	ANDRAŽ CEKET, univ.dipl.inž.grad.	IZS PI G-2435	
izdelal:	ANDREJ BRICMAN, dipl.inž.geol. (UN)		
investitor:	Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana		
vrsta projekta:	PZI		
vrsta načrta:	2.1 Načrt zaščite gradbene jame		
naziv objekta:	Športna dvorana - Šolski center Novo mesto		
vsebinska risba:	32 PREČNI PREREZI Prečni prerez gradbene jame		
datum:	št. projekta:	št. načrta:	merilo:
05.2024	028-23	028-23-21	1 : 100/100
			G.3



Športna dvorana - Šolski center Novo mesto

42 VZDOLŽNI PROFIL
Vzdolžni profili pilotnih sten

merilo: 1 : 100/100

- Sidra IBO R32/250, L=12.00 m, B=3.20 m
- Sidra IBO R32/250, L=6.00 m, B=1.00 m
- Geotehnična sidra 3x6"/251kN
- Rezervna sidrišča



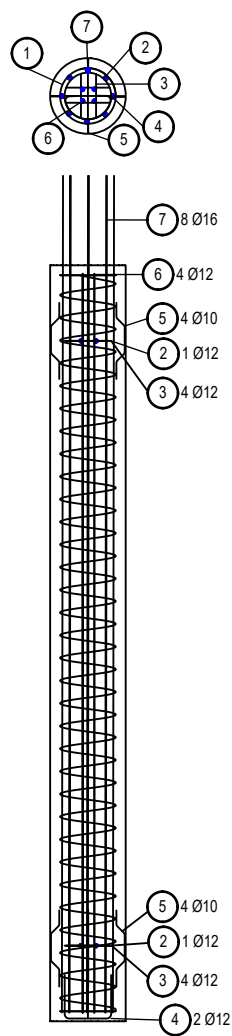
MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE,
MASARYKOVA CESTA 16, 1000 LJUBLJANA

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podjetje za geotekniko, infrastrukturo in raziskave
tovarniška c. 26, 5270 Ajdovščina | +386(0)5 8500740 | info@acap.si



vodja projekta:	MATEJ SOMRAK, univ.dipl.inž.arh.	ZAPS A-1746
pooblaščen inženir:	ANDRAŽ CEKET, univ.dipl.inž.grad.	IZS PI G-2435
izdelal:	ANDREJ BRICMAN, dipl.inž.geol. (UN)	
investitor:	Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana	
vrsta projekta:	PZI	
vrsta načrta:	2.1 Načrt zaščite gradbene jame	
naziv objekta:	Športna dvorana - Šolski center Novo mesto	
vsebina risbe:	42 VZDOLŽNI PROFIL Vzdolžni profili pilotnih sten	
datum:	05.2024	
št. projekta:	028-23	
št. načrta:	028-23-21	
merilo:	1 : 100/100	
št. risbe:	G.4	

Seznam palic - oblika krivljenja



Poz.	Kosov	Fi [mm]	Posam. dolžina [m]	Kotirana oblika krivljenja (ni v merilu)
1	1	12	Tek.m	
2	2	12	1.72	
3	8	12	0.30	
4	2	12	0.92	
5	8	10	0.58	
6	4	12	4.90	
7	8	16	5.55	

AB PILOT Φ50cm	št.	dolžina L (m)	Armatura poz. 1 (m)	Armatura poz. 2 (kos)	Armatura poz. 3 (kos)	Armatura poz. 4 (kos)	Armatura poz. 5 (kos)	Armatura poz. 6 (m)	Armatura poz. 7 (m)
1	1	4.0	35.82	2	8	2	8	15.60	45.50
2	2	4.5	81.88	4	16	4	16	35.20	101.00
3	6	5.0	272.04	12	48	12	48	117.60	333.00
4	6	5.5	298.44	12	48	12	48	129.60	363.00
5	6	6.0	324.90	12	48	12	48	141.60	393.00
6	4	6.5	234.20	8	32	8	32	102.40	282.00
skupaj	25		1247.28	50.00	200.00	50.00	200.00	542.00	1517.50
teža (kg/m)			0.92	0.92	0.92	0.92	0.65	0.92	1.64
skupna teža (kg)			1147.50	79.12	55.20	42.32	75.40	498.64	2488.70
								Skupaj (kg)	4386.88

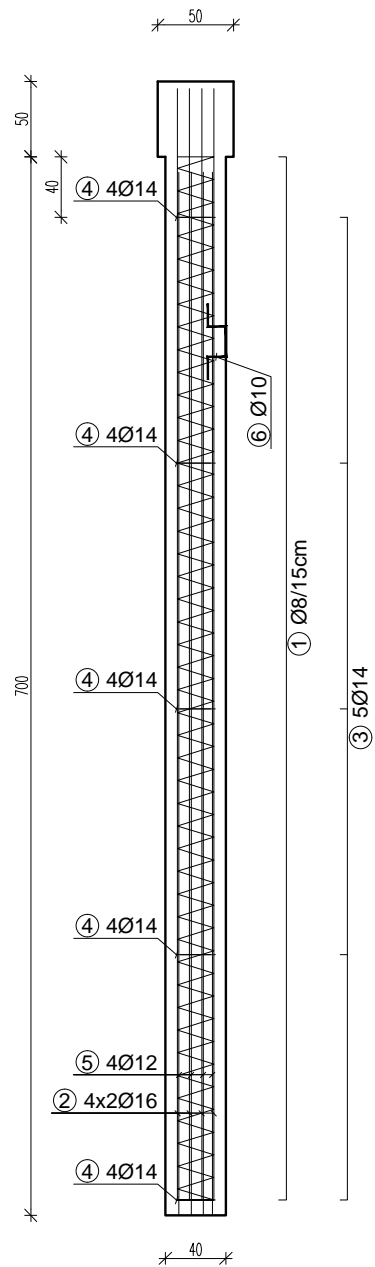
Športna dvorana - Šolski center Novo mesto

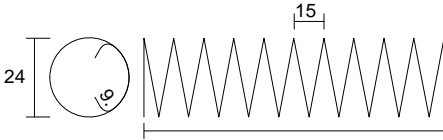

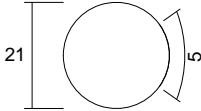
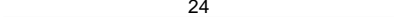

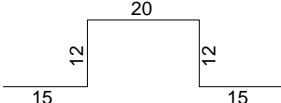
ARMATURNE RISBE

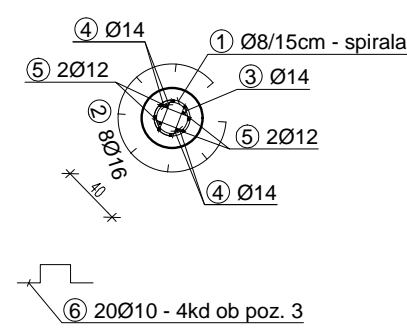
PILOTI fi50

MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE, Masarykova cesta 16, 1000 Ljubljana		AC&P inženirski biro d.o.o. podjetje za geotekniko, infrastrukturo in raziskave Tovarniška cesta 26, 5270 Ajdovščina +386(0)5 8500740 info@acap.si			
vodja projekta:		MATEJ SOMRAK, univ.dipl.inž.arh.		ZAPS A-1746	
pooblaščen inženir:		ANDRAŽ CEKET, univ.dipl.inž.grad.		IZS PI G-2435	
izdelal:		NIKO ROVTAR			
investitor:		Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana			
vrsta projekta:		PZI			
vrsta načrta:		2 Načrt gradbeništva - Načrt varovanja gradbene jame			
naziv objekta:		Športna dvorana - Šolski center Novo mesto			
vsebina risbe:		ARMATURNE RISBE PILOTI fi50			
datum:		št. projekta:	št. načrta:	merilo:	št. risbe:
05.2024		028-23	028-23-21	1 : 50	G.5

AB PILOT 40, L=3,0-5,0m, kd 59



Palice - specifikacija					
ozn	oblika in mere [cm]		Ø		
AB pilot 40					
1			8		
2			16		
3					
4					
5			12		
6			10		



AB PILOT Φ40cm	št.	dolžina L (m)	Armatura poz. 1 (m)	Armatura poz. 2 (kos)	Armatura poz. 3 (m)	Armatura poz. 4 (kos)	Armatura poz. 5 (kos)	Armatura poz. 6 (m)
1	2	3.0	30.14	48	3	4	23	8.88
2	9	3.5	158.31	252	12	17	122	39.96
3	13	4.0	261.30	416	17	25	203	57.72
4	12	4.5	271.32	432	24	35	211	79.92
5	5	5.0	125.65	200	10	14	98	33.30
skupaj	41		846.72	1,348	65.31	95.04	657.60	219.78
teža (kg/m)			0.41	1.64	1.25	1.25	0.92	0.65
skupna teža (kg)			347.16	3,802.44	24.49	109.30	350.90	142.86
Skupaj (kg)								4777.13

Športna dvorana - Šolski center Novo mesto

ARMATURNE RISBE

PILOTI fi40

MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE,
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1000 Ljubljana

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podjetje za geotehniko, infrastrukturo in raziskave
Tovarniška cesta 26, 5270 Ajdovščina | +386(0)5 8500740 | info@acap.si

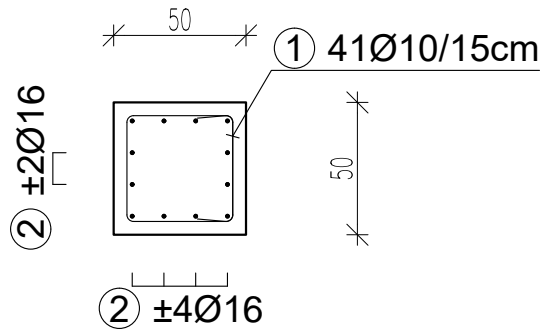


vodja projekta:	MATEJ SOMRAK, univ.dipl.inž.arh.	ZAPS A-1746
pooblaščen inženir:	ANDRAŽ CEKET, univ.dipl.inž.grad.	IZS PI G-2435
izdelal:	NIKO ROVTAR	
investitor:	Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana	
vrsta projekta:	PZI	
vrsta načrta:	2 Načrt gradbeništva - Načrt varovanja gradbene jame	
naziv objekta:	Športna dvorana - Šolski center Novo mesto	
vsebina risbe:	ARMATURNE RISBE PILOTI fi40	

datum:	št. projekta:	št. načrta:	merilo:	št. risbe:
05.2024	028-23	028-23-21	1 : 50	G.6

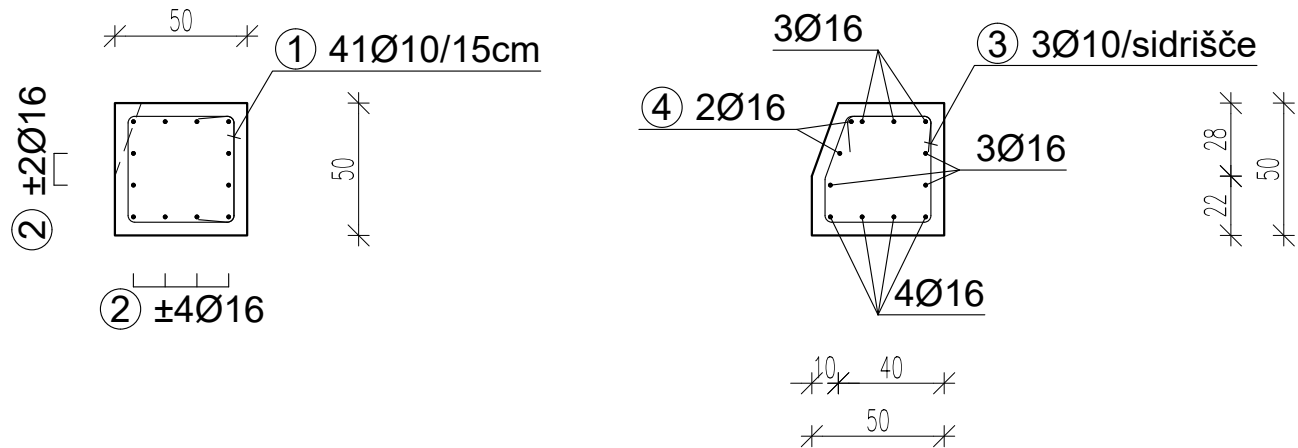
AB GREDA 50×50cm, kampada L=6.10m

PREREZ



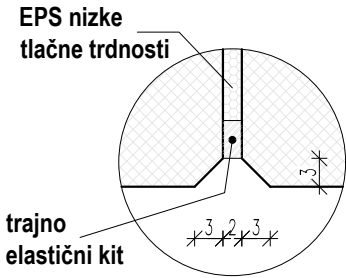
AB GREDA 50×50cm s sidrišči, kampada L=6.10m

PREREZ



Sidrišče na osnem razmaku 1,60m (3× na kampado)
Palico poz. 2 rezati na licu mesta, zaradi sidrišč !

Detajl dilatacije š=2cm



Dilatacija:
* armaturna palica fi20 dolžine 150cm - moznik
* polovica palice v PVC cevi, polovica pa v betonu
* 4× fi20 / dilatacijo

Športna dvorana - Šolski center Novo mesto

71 ARMATURNE RISBE
Armaturna risba grede 50x50

merilo: 1 : 50

Armaturna greda 50x50						
ARMATURNI IZVLEČEK						
poz.		Ø [mm]	Dolžina [m]	št. kosov	Teža na m [kg/m]	Skupaj [kg]
1		10	2.20	176.7	0.650	252.68
2		16	6.00	57	1.640	560.88
3		10	2.06	51.3	0.650	68.69
4		16	1.50	45.6	1.640	112.18
					Skupaj:	994.43

MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE,
MASARYKOVA CESTA 16, 1000 LJUBLJANA

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podjetje za geotehniko, infrastrukturo in raziskave
tovarniška c. 26, 5270 ajdovščina | +386(0)5 8500740 | info@acap.si



vodja projekta: MATEJ SOMRAK, univ.dipl.inž.arh. ZAPS A-1746

pooblaščen inženir: ANDRAŽ CEKET, univ.dipl.inž.grad. IZS PI G-2435

izdelal: ANDREJ BRICMAN, dipl.inž.geol. (UN)

investitor: Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana

vrsta projekta: PZI

vrsta načrta: 2.1 Načrt zaščite gradbene jame

naziv objekta: Športna dvorana - Šolski center Novo mesto

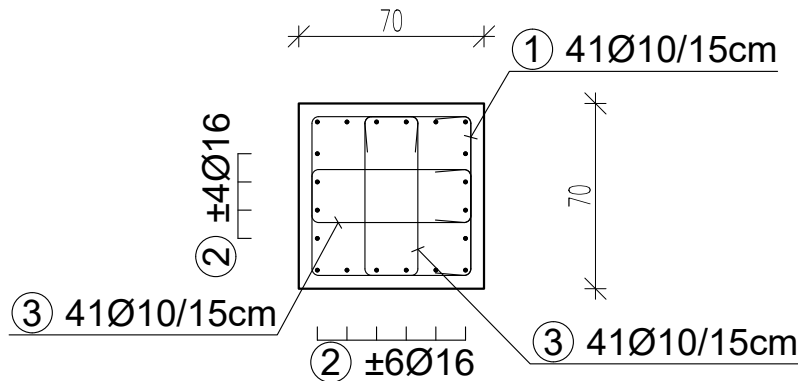
vsebina risbe: 71 ARMATURNE RISBE
Armaturna risba grede 50x50

datum: št. projekta: št. načrta: merilo: št. risbe:

05.2024 028-23 028-23-21 1 : 50 G.7

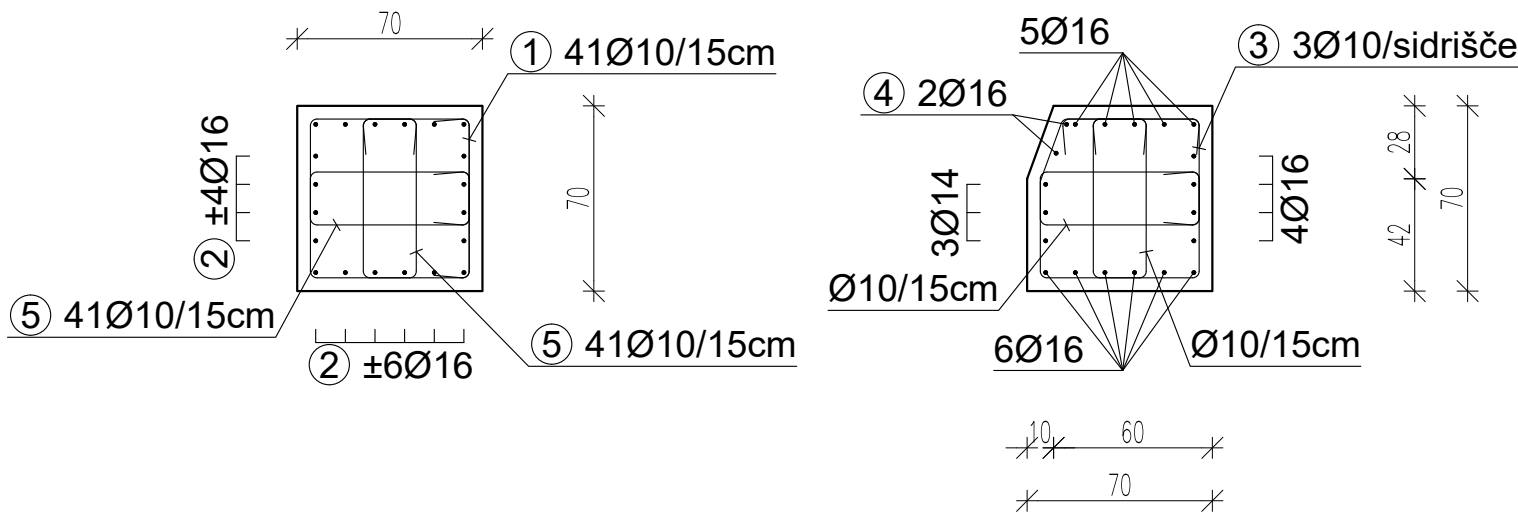
AB GREDA 70×70cm, kampada L=6.10m

PREREZ



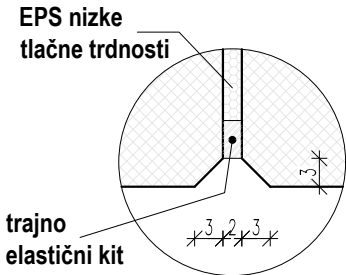
AB GREDA 70×70cm s sidrišči, kampada L=6.10m

PREREZ



Sidrišče na osnem razmaku 1,00m (5× na kampado)
Palici poz. 2 rezati na licu mesta, zaradi sidrišč !

Detajl dilatacije š=2cm



Dilatacija:
* armaturna palica fi20 dolžine 150cm - moznik
* polovica palice v PVC cevi, polovica pa v betonu
* 4× fi20 / dilatacijo

Športna dvorana - Šolski center Novo mesto

71 ARMATURNE RISBE
Armaturna risba grede 70x70

merilo: 1 : 50

Armaturna greda 70x70						
ARMATURNI IZVLEČEK						
poz.		Φ [mm]	Dolžina [m]	št. kosov	Teža na m [kg/m]	Skupaj [kg]
1		10	3.22	106.3	0.650	222.38
2		16	6.00	76.5	1.640	752.76
3		10	3.06	63.75	0.650	126.80
4		10	2.02	340	0.650	446.42
5		16	1.00	42.5	1.640	69.70
Skupaj:						1,618.06

MINISTRSTVO ZA VZGOJO IN IZOBRAŽEVANJE,
MASARYKOVA CESTA 16, 1000 LJUBLJANA

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podjetje za geotehniko, infrastrukturo in raziskave
tovarniška c. 26, 5270 ajdovščina | +386(0)5 8500740 | info@acap.si



vodja projekta: MATEJ SOMRAK, univ.dipl.inž.arh. ZAPS A-1746

pooblaščen inženir: ANDRAŽ CEKET, univ.dipl.inž.grad. IZS PI G-2435

izdelal: ANDREJ BRICMAN, dipl.inž.geol. (UN)

investitor: Ministrstvo za vzgojo in izobraževanje, Masarykova cesta 16, 1000 Ljubljana

vrsta projekta: PZI

vrsta načrta: 2.1 Načrt zaščite gradbene jame

naziv objekta: Športna dvorana - Šolski center Novo mesto

vsebina risbe: 71 ARMATURNE RISBE
Armaturna risba grede 70x70

datum: št. projekta: št. načrta: merilo: št. risbe:

05.2024 028-23 028-23-21 1 : 50 G.8