

RELACION TEKNIK

PROJEKT ZBATIM

OBJEKTI

*“Ndertim i Ujesjellesit Mançurisht +
Plase”*

Bashkia Maliq

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1. DETYRA E PROJEKTIMIT

Detyra e projektimit eshte hartuar nga Bashkia Maliq si me poshte:

Objekti:

Objekti i detyrës së projektimit është hartimi i projektit të " Ndërtim i ujësjellësit Mancurisht + Plase" , sipas planimetrisë bashkangjitur kësaj detyre projektimi.

- 1. Linja e Furnizimit Ujesjellesit Stacion Pompimi – Depo Plase (200m³) – Rrjeti Kryesor*
- 2. Linja Stacioni i Pompomit Mancurisht – Depo 500 m³ – Linje Dergimi*
- 3. Linja e dergimit ujesjellesit Depo Uji – Lidhjet me Rrjetin, Rrjet shperndares egzistues te fshatrave*
- 4. Mates uji komplet + Lidhje per familjaret*

Burimi i marjes se ujit :

Uji do te meret ne Stacion Pompimi Plase dhe ne Stacionin e Pompomit Mancurisht

Te dhenat Kryesore:

1. Popullsia aktuale është rreth 4870 banorë.
2. Shtesa natyrore e popullsisë është rreth 2.5% në vit.
3. Perspektiva e ujësjellësit të parashikohet për 25 vjet.
4. Norma për frymë të popullatës të llogaritet 150 l/dite per banor sipas normave në fuqi.
5. Materiali i tubacionit të jetë HDPE i çertifikuar.
6. Lidhja me rrjetet egzistuese te fshatrave.
7. Planimetria e rrjetit egzistues.

2. Te pergjitheshme

Fshatrat Burimas, Kreshpanj, Rrembec, Orman ndodhen në Njësinë Administrative Pojan.

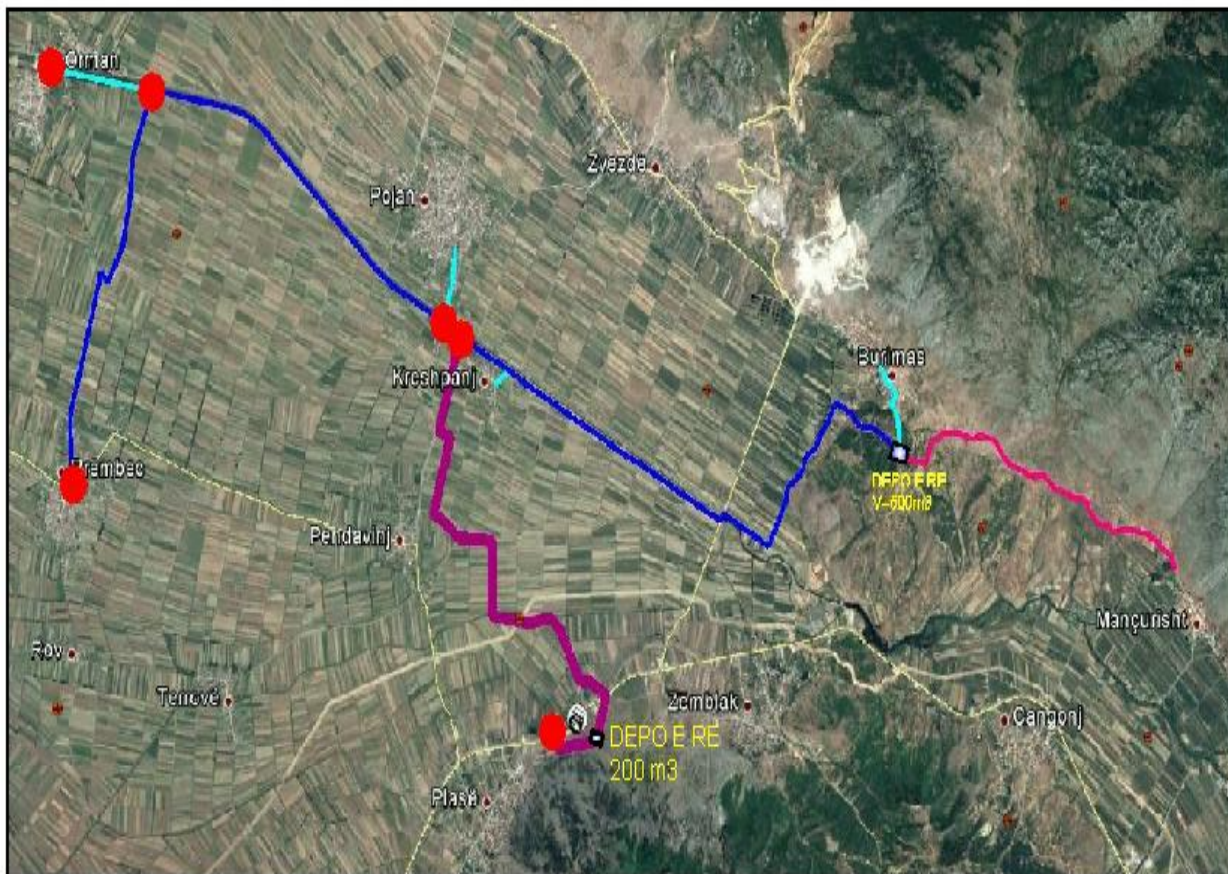
Aktualisht furnizimi me uje kryhet nga burimet e Mançurishtit, zone e cila ndodhet nen administrimin e Bashkise Devoll. Burimet e Mançurishtit kane nje debit prej rreth 30 l/sek. Nga kjo sasi meret rreth 15 l/sek. Ne afersi te burimit eshte ndertuar Stacioni i Pompave. Ky stacion eshte i amortizuar dhe ka nevoje per rikonstruksion. Deri ne afersi te fshatit Burimas uji ngjitet nepermjet pompave dhe me pas me rrjedhje te lire shperndahet ne tre fshatra, Burimas, Kreshpanj, dhe Rrembec.

Ujesjellesi (projekti) nga Stacioni i Pompave ne Plase deri mbi fshatin Burimas eshte me tubo metalike, te korroduara e te demtuara pjeserisht. Ndersa pjesa tjeter eshte me tubo eterniti. Keto tubo jane demtuar dhe duhet te theksohet se eterniti eshte nje material shume i rezikshem per shendetin e njeriut.

Fshati Burimas ka nje depo qe sherben per furnizimin e fshatit.

Ne vazhdimesi nepermjet ketij ujesjellesi do te furnizohet edhe fshati Orman, fshat ky qe ka probleme me cilesine e ujit qe perdor nga burimet e Shengjergjit.

Horografia e Skemes Ujesjellesit



3. Popullsia

Zona ka nje popullsi rreth 4870 banore, numer i cili ndryshon ne menyre te vazhdueshme.

Aktivitetet kryesore ne kete zone jane bujqesia ne zonen ne afersi me qytetin dhe zonat perreth , blegtoria dhe tregtia e materialeve te ndryshme , te sherbimeve industriale dhe sociale

4. Gjendja ekzistuese e furnizimit me uje

Fshatrat Burimas, Kreshpanj, Rrembec, Orman ndodhen në Njësinë Administrative Pojan.

Aktualisht furnizimi me uje kryhet nga burimet e Mançurishtit, zone e cila ndodhet nen administrimin e Bashkise Devoll. Burimet e Mançurishtit kane nje debit prej rreth 30 l/sek. Nga kjo sasi meret rreth 15 l/sek.Ne afersi te burimit eshte ndertuar Stacioni i Pompave. Ky stacion eshte i amortizuar dhe ka nevojte per rikonstruksion. Deri ne afersi te fshatit Burimas uji ngjitet nepermjet pompave dhe me pas me rrjedhje te lire shperndahet ne tre fshatra, Burimas, Kreshpanj, dhe Rrembec.

Ujesjellesi nga Stacioni i Pompave ne Plase deri mbi fshatin Burimas eshte me tubo metalike, te korroduara e te demtuara pjeserisht.Ndersa pjesa tjeter eshte me tubo eterniti.Keto tubo jane demtuar dhe duhet te theksohet se eterniti eshte nje material shume i rezikshem per shendetin e njeriut.

Fshati Burimas ka nje depo qe sherben per furnizimin e fshatit.

Ne vazhdimesi nepermjet ketij ujesjellesi do te furnizohet edhe fshati Orman, fshat ky qe ka probleme me cilesine e ujit qe perdor nga burimet e Shengjergjit.

5. Depo e ujesjellesit V=500m³.

a. Vend vendosja.

Mbeshtetur ne studimin e terenit, afersise, kushteve gjeologjike inxhinierike qe duhet te plotesohen per vendosjen e depos se ujesjellesit u percaktua qe Depoja e ujesjellesit te vendoset ne kodren me te larte te Fshatit Burimas.

Depoja e ujesjellesit do te vendoset ne majen e nje kodre me kuote 910 m mbi nivelin e detit, ne nje teren te qendrueshem.

Kuota e vendosjes se depos ploteson parametrat teknike te projektit te ujesjellesit.

b. Kapaciteti.

Depoja e ujesjellesit do te llogaritet per te perballuar furnizimin me uje te linjes se Dergimit, ujesjellesin "Plase - Burimas-Kreshpanj-Rrembec-Orman".

Kapaciteti i depos qe duhet te projektohet dhe te perballoje per periudhen 20 vjecare te parashikimit te plotesimit te nevojave te popullates se ketyre zonave me uje eshte 500 m³.

6. Skema e pergjitheshme e ndertimit te ujesjellesit.

Pas shqyrtimit te hollesisem te disa varianteve te skemes se ndertimit te ujesjellesit, evidentimin dhe studimin e vendeve te mundeshme te vendosjes dhe ndertimit te depos qe ploteson kerkesat e sigurimit te parametrave teknike te projektit te furnizimit me uje te ujesjellesit "Plase - Burimas-Kreshpanj-Rrembec-Orman" skema me ekonomike dhe me efektive qe ploteson ne kompleks furnizimin me uje ujesjellesit dhe te perspektives se furnizimit me uje te ujesjellesit paraqitet si me poshte:

1.LLOGARITJET HIDRAULIKE TE PRURJES SE KERKUAR

1.Llogaritja e Prurjes per Fshatin Burimas

	Njesia	Llogaritja e prurjes				
		2017	2022	2027	2032	2037
Popullsia	Banore	1,036	1,144	1,263	1,394	1,539
BURIMAS		1036	1,144	1,263	1,394	1,539
Norma mesatare per person (sipas VKM Nr 722, date 19.11.1998, I azhornuar me 19.11.2008).	l/b/dite	150.0	150.0	150.0	150.0	150.0
konsumatoret e pritshem (20-30% e normes) biznese, sociale, shkolla, spitale etj	l/dite	20.0	22.0	25.0	28.0	30.0
Norma totale	l/b/dite	170.0	172.0	175.0	178.0	180.0
Prurja mesatare ditore per popullsine	m ³ /dite	176.1	196.7	221.0	248.2	277.1
Humbjet, rrjedhje, lidhje te paligjeshme etj.	%	8.0	12.0	16.0	20.0	20.0
Prurja mesatare ditore per popullsine duke perfshire humbjet rrjedhjet lidhjet e paligjeshme etj.	m ³ /dite	14.1	23.6	35.4	49.6	55.4

Kerkesa ditore totale	m3/dite	190.2	220.3	256.4	297.8	332.5
Norma mesatare per person	l/b/dite	183.60	192.64	203.00	213.60	216.00
Prurja ne rrjet (me koeficient orar K=2.6)	l/s	5.72	6.63	7.71	8.96	10.01
Prurja e kerkuar nga burimi	l/s	2.20	2.55	2.97	3.45	3.85

2.Llogaritja e Prurjes per Fshatin Kreshpanj

	Njesia	Llogaritja e prurjes				
		2017	2022	2027	2032	2037
Popullsia	Banore	371	410	452	499	551
KRESHPANJ		371	410	452	499	551
Norma mesatare per person (sipas VKM Nr 722, date 19.11.1998, I azhornuar me 19.11.2008).	l/b/dite	150.0	150.0	150.0	150.0	150.0
konsumatoret e pritshem (20-30% e normes) biznese, sociale, shkolla, spitale etj	l/dite	20.0	22.0	25.0	28.0	30.0
Norma totale	l/b/dite	170.0	172.0	175.0	178.0	180.0
Prurja mesatare ditore per popullsine	m3/dite	63.1	70.5	79.1	88.9	99.2
Humbjet, rrjedhje, lidhje te paligjeshme etj.	%	8.0	12.0	16.0	20.0	20.0
Prurja mesatare ditore per popullsine duke perfshire humbjet rrjedhjet lidhjet e paligjeshme etj.	m3/dite	5.0	8.5	12.7	17.8	19.8
Kerkesa ditore totale	m3/dite	68.1	78.9	91.8	106.7	119.1
Norma mesatare per person	l/b/dite	183.60	192.64	203.00	213.60	216.00
Prurja ne rrjet (me koeficient orar K=2.6)	l/s	2.05	2.37	2.76	3.21	3.58
Prurja e kerkuar nga burimi	l/s	0.79	0.91	1.06	1.23	1.38

3.Llogaritja e Prurjes per Fshatin Rembec

	Njesia	Llogaritja e prurjes				
		2017	2022	2027	2032	2037
Popullsia	Banore	1,663	1,836	2,027	2,238	2,471
REMBEC		1663	1,836	2,027	2,238	2,471
Norma mesatare per person (sipas VKM Nr 722, date 19.11.1998, I azhornuar me 19.11.2008).	l/b/dite	150.0	150.0	150.0	150.0	150.0
konsumatoret e pritshem (20-30% e normes) biznese, sociale, shkolla, spitale etj	l/dite	20.0	22.0	25.0	28.0	30.0
Norma totale	l/b/dite	170.0	172.0	175.0	178.0	180.0
Prurja mesatare ditore per popullsine	m3/dite	282.7	315.8	354.8	398.4	444.8
Humbjet, rrjedhje, lidhje te paligjeshme etj.	%	8.0	12.0	16.0	20.0	20.0
Prurja mesatare ditore per popullsine duke perfshire humbjet rrjedhjet lidhjet e paligjeshme etj.	m3/dite	22.6	37.9	56.8	79.7	89.0
Kerkesa ditore totale	m3/dite	305.3	353.7	411.5	478.1	533.8

Norma mesatare per person	l/b/dite	183.60	192.64	203.00	213.60	216.00
Prurja ne rrjet (me koeficient orar K=2.6)	l/s	9.19	10.64	12.38	14.39	16.06
Prurja e kerkuar nga burimi	l/s	3.53	4.09	4.76	5.53	6.18

4.Llogaritja e Prurjes per Fshatin Orman

	Njesia	Llogaritja e prurjes				
		2017	2022	2027	2032	2037
Popullsia	Banore	1,797	1,984	2,191	2,419	2,670
ORMAN		1797	1,984	2,191	2,419	2,670
Norma mesatare per person (sipas VKM Nr 722, date 19.11.1998, I azhornuar me 19.11.2008).	l/b/dite	150.0	150.0	150.0	150.0	150.0
konsumatoret e pritshem (20-30% e normes) biznese, sociale, shkolla, spitale etj	l/dite	20.0	22.0	25.0	28.0	30.0
Norma totale	l/b/dite	170.0	172.0	175.0	178.0	180.0
Prurja mesatare ditore per popullsine	m3/dite	305.5	341.3	383.3	430.5	480.6
Humbjet, rrjedhje, lidhje te paligjeshme etj.	%	8.0	12.0	16.0	20.0	20.0
Prurja mesatare ditore per popullsine duke perfshire humbjet rrjedhjet lidhjet e paligjeshme etj.	m3/dite	24.4	41.0	61.3	86.1	96.1
Kerkesa ditore totale	m3/dite	329.9	382.2	444.7	516.6	576.8
Norma mesatare per person	l/b/dite	183.60	192.64	203.00	213.60	216.00
Prurja ne rrjet (me koeficient orar K=2.6)	l/s	9.93	11.50	13.38	15.55	17.36
Prurja e kerkuar nga burimi	l/s	3.82	4.42	5.15	5.98	6.68

5.Llogaritja e Prurjes per te Gjithe Fshatrat

	Njesia	Llogaritja e prurjes				
		2017	2022	2027	2032	2037
Popullsia	Banore	4,867	5,374	5,933	6,550	7,232
ORMAN		4867	5,374	5,933	6,550	7,232
Norma mesatare per person (sipas VKM Nr 722, date 19.11.1998, I azhornuar me 19.11.2008).	l/b/dite	150.0	150.0	150.0	150.0	150.0
konsumatoret e pritshem (20-30% e normes) biznese, sociale, shkolla, spitale etj	l/dite	20.0	22.0	25.0	28.0	30.0
Norma totale	l/b/dite	170.0	172.0	175.0	178.0	180.0

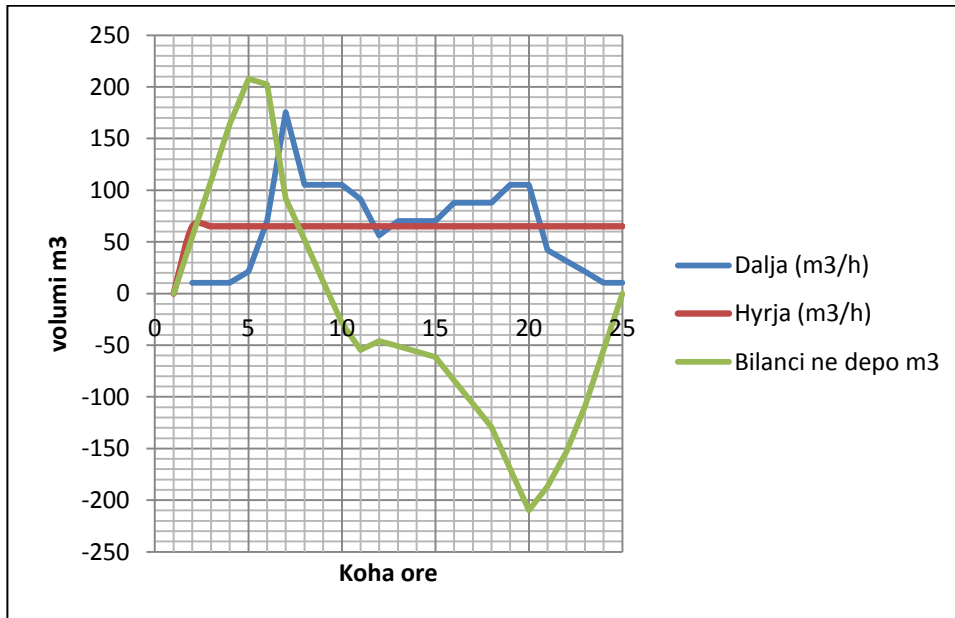
Prurja mesatare ditore per popullsine	m3/dite	827.4	924.3	1,038.2	1,166.0	1,301.8
Humbjet, rrjedhje, lidhje te paligjeshme etj.	%	8.0	12.0	16.0	20.0	20.0
Prurja mesatare ditore per popullsine duke perfshire humbjet rrjedhjet lidhjet e paligjeshme etj.	m3/dite	66.2	110.9	166.1	233.2	260.4
Kerkesa ditore totale	m3/dite	893.6	1,035.2	1,204.4	1,399.2	1,562.1
Norma mesatare per person	l/b/dite	183.60	192.64	203.00	213.60	216.00
Prurja ne rrjet (me koeficient orar K=2.6)	l/s	26.89	31.15	36.24	42.10	47.01
Prurja e kerkuar nga burimi	l/s	10.34	11.98	13.94	16.19	18.08

2.LLOGARITJET HIDRAULIKE TE DEPOS 500m3

Koha (h)		Dalja (m ³ /h)	Hyrja (m ³ /h)	Bilanci ne depo m ³
0	0.162			0
1	0.162	10.54	65.09	54.54
2	0.162	10.54	65.09	109.09
3	0.162	10.54	65.09	163.63
4	0.324	21.09	65.09	207.63
5	1.079	70.23	65.09	202.49
6	2.697	175.54	65.09	92.03
7	1.618	105.31	65.09	51.81
8	1.618	105.31	65.09	11.59
9	1.618	105.31	65.09	-28.64
10	1.402	91.25	65.09	-54.80
11	0.863	56.17	65.09	-45.89
12	1.079	70.23	65.09	-51.03
13	1.079	70.23	65.09	-56.17
14	1.079	70.23	65.09	-61.31
15	1.348	87.74	65.09	-83.96
16	1.348	87.74	65.09	-106.61
17	1.348	87.74	65.09	-129.26
18	1.618	105.31	65.09	-169.49
19	1.618	105.31	65.09	-209.71
20	0.647	42.11	65.09	-186.74
21	0.485	31.57	65.09	-153.22
22	0.324	21.09	65.09	-109.22
23	0.162	10.54	65.09	-54.67
24	0.162	10.54	65.09	-0.13
	Volumi llogarites i Depos (m3)			417.34
	Volumi prespektiv i Depos (m3)			500

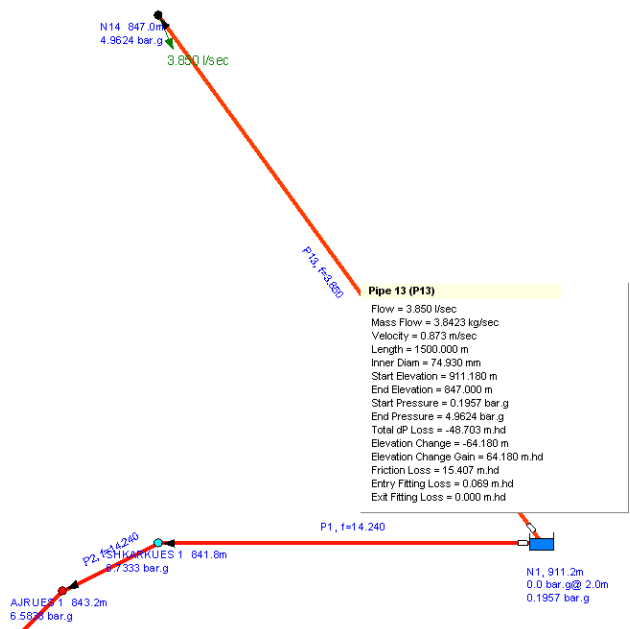
Kerkesa (l/s)	18.08
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Grafiku i volumit sipas kohes se Depos



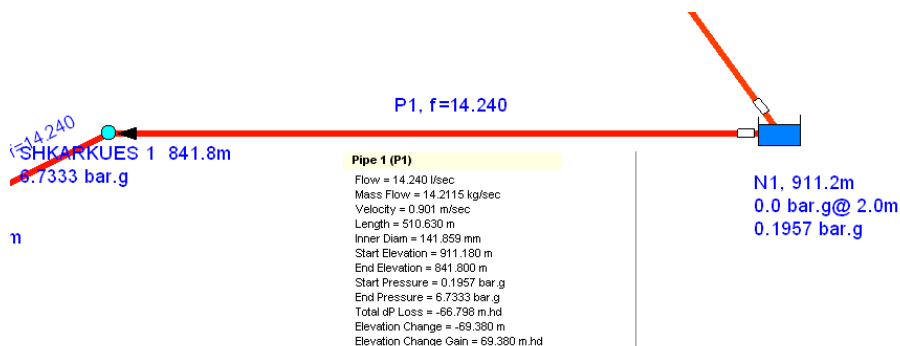
3.LLOGARITJET HIDRAULIKE TE UJESJELLESIT

1.Linja e Fshatit Burimas

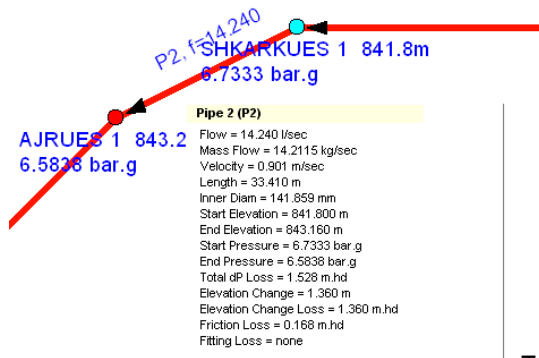


2.Linja e Fshatit Kreshpani

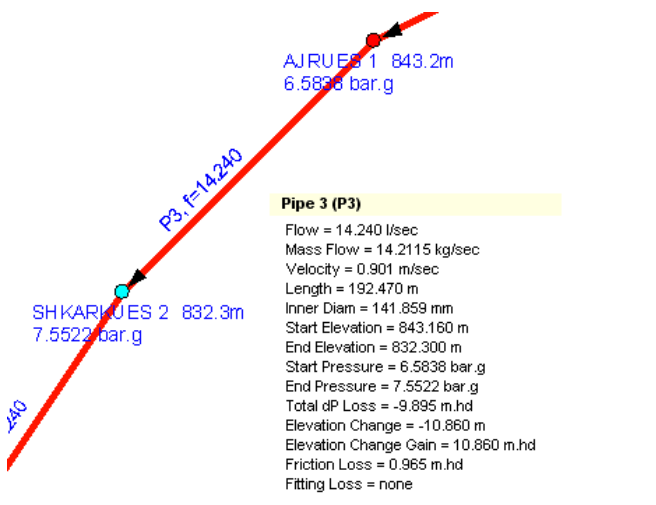
Tubi P1



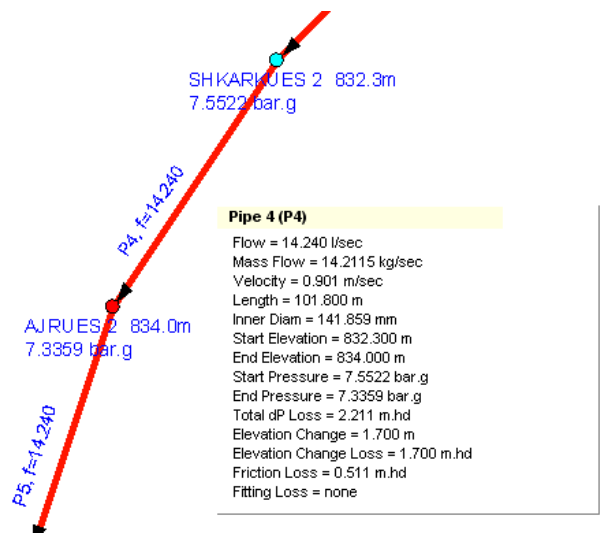
Tubi P2



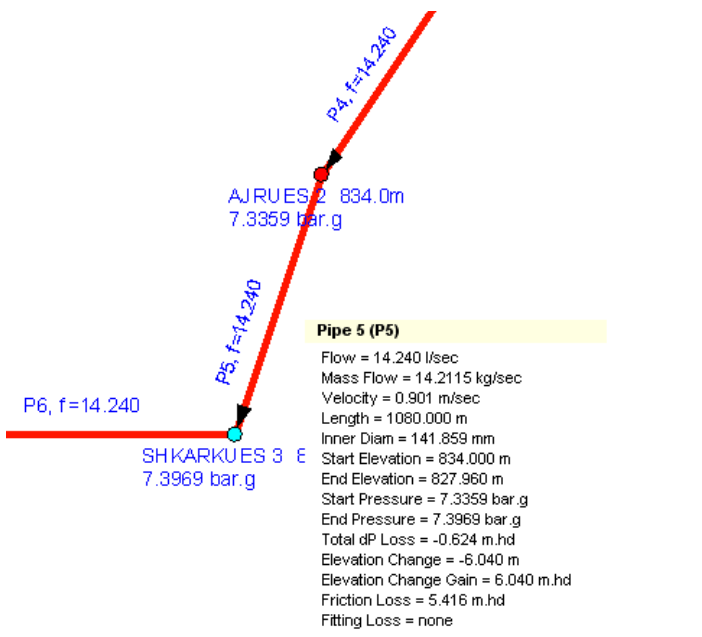
Tubi P3



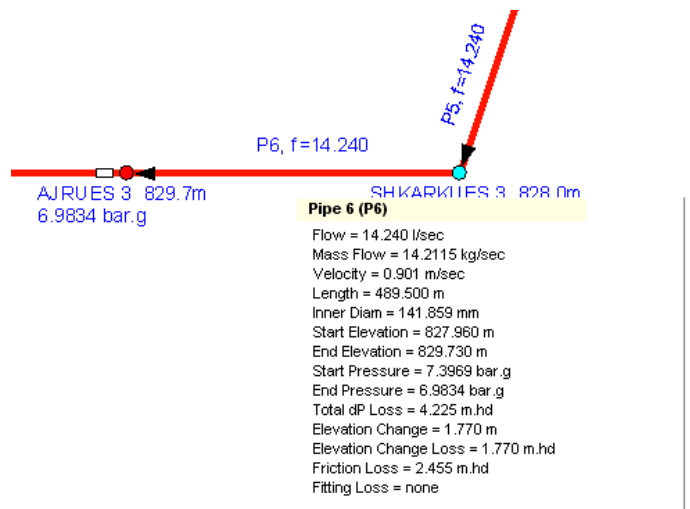
Tubi P4



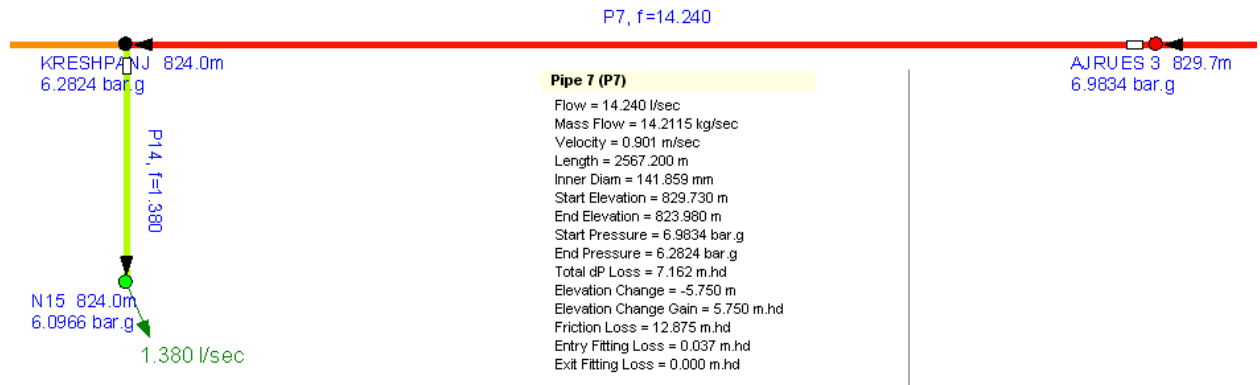
Tubi P5



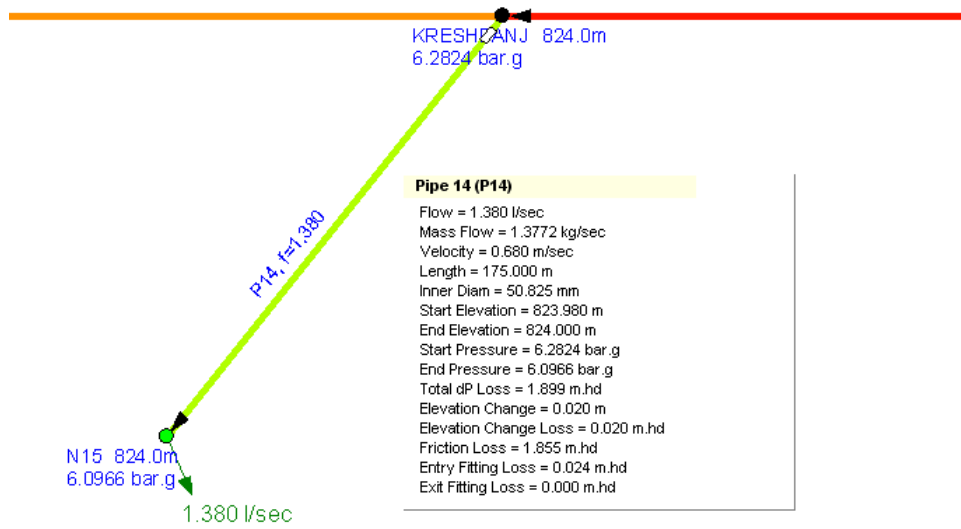
Tubi P6



Tubi P7



Tubi P14 (Tubi I dergimit te Fshatit Kreshpanj)

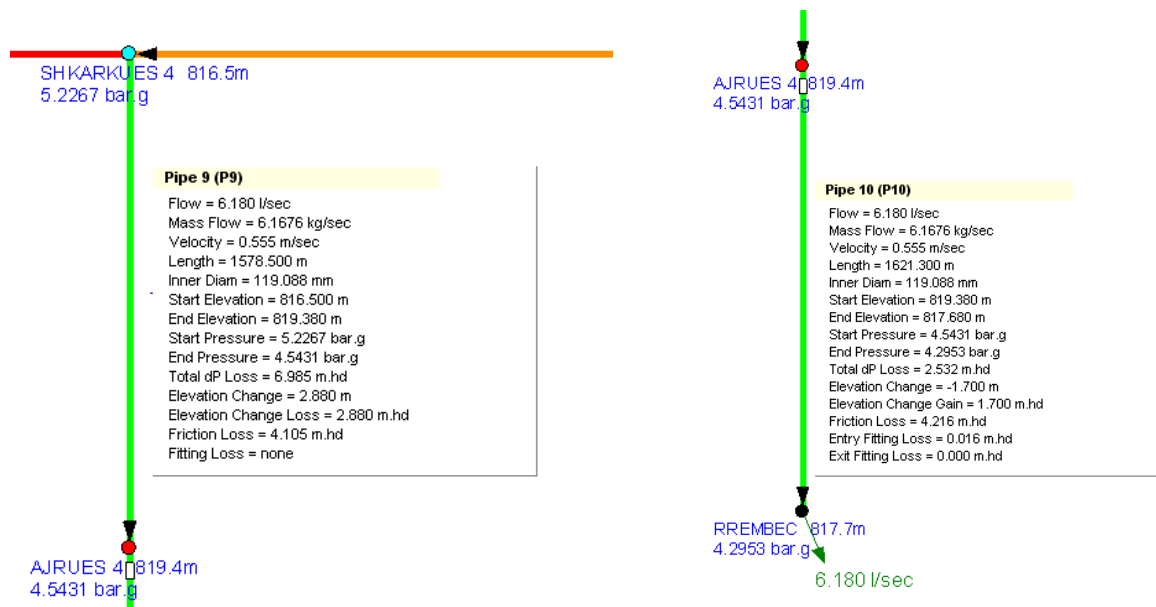


Tubi P8

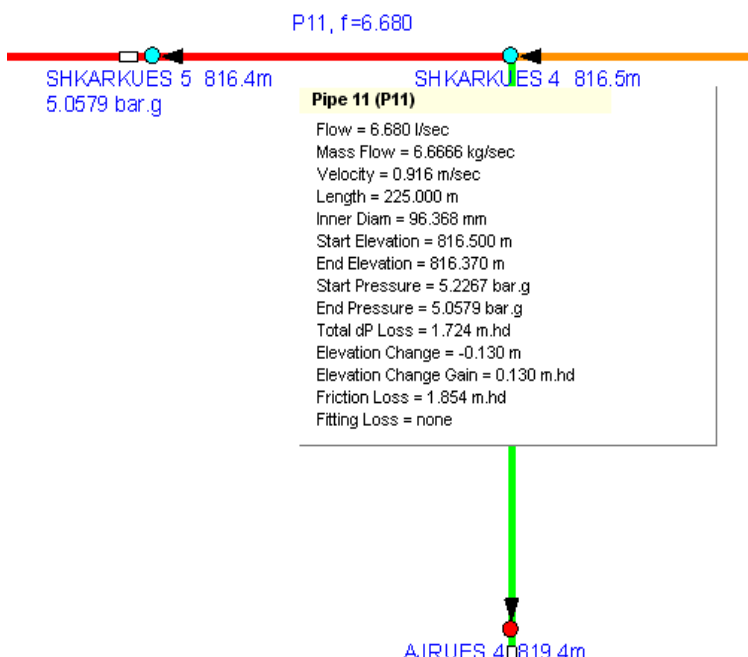


Tubi P9 (Tubi I dergimit te Fshatit Rrembec)

Tubi P10 (Tubi I dergimit te Fshatit Rrembec)



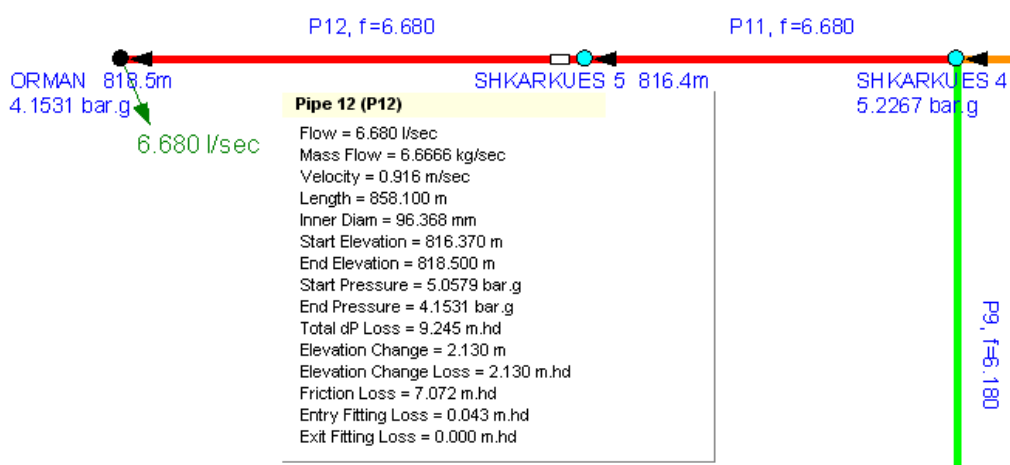
Tubi P11 (Tubi i dergimit te Fshatit Orman)



Tubi P12 (Tubi i dergimit te Fshatit Orman)

4. PRURJET LLOGARITESE

4.3. Prurjet Llogaritese



Norma e perdorimit te ujit eshte marre 150 l/dite banore.

Ne tabelen e meposhtme jepen nevojat per uje dhe prurjet llogaritese per llogaritjen e linjave te jashtme te zones.

Prurja e pergjitheshme llogaritese e te gjithe ujesjellesit, tubacionit te dergimit, pompave dhe te volumit te depos se ujit do te jete sipas tabelës llogaritese:

Lllogaritja e Prurjes per te Gjithe Fshatrat

	Njesia	Llogaritja e prurjes				
		2017	2022	2027	2032	2037
Popullsia	Banore	4,867	5,374	5,933	6,550	7,232
		4867	5,374	5,933	6,550	7,232
Norma mesatare per person (sipas VKM Nr 722, date 19.11.1998, I azhornuar me 19.11.2008).	l/b/dite	150.0	150.0	150.0	150.0	150.0

konsumatorët e pritshëm (20-30% e normës) biznese, sociale, shkolla, spitale etj	l/dite	20.0	22.0	25.0	28.0	30.0
Norma totale	l/b/dite	170.0	172.0	175.0	178.0	180.0
Prurja mesatare ditore per popullsine	m3/dite	827.4	924.3	1,038.2	1,166.0	1,301.8
Humbjet, rrjedhje, lidhje te paligjeshme etj.	%	8.0	12.0	16.0	20.0	20.0
Prurja mesatare ditore per popullsine duke perfshire humbjet rrjedhjet lidhjet e paligjeshme etj.	m3/dite	66.2	110.9	166.1	233.2	260.4
Kerkesa ditore totale	m3/dite	893.6	1,035.2	1,204.4	1,399.2	1,562.1
Norma mesatare per person	l/b/dite	183.60	192.64	203.00	213.60	216.00
Prurja ne rrjet (me koeficient orar K=2.6)	l/s	26.89	31.15	36.24	42.10	47.01
Prurja e kerkuar nga burimi	l/s	10.34	11.98	13.94	16.19	18.08

5. Llogaritja e Depos se ujesjellesit

Volumi i rezervuarit te ujit per te dy zonat e furnizimit me uje llogaritet per nje volum 5 ore per rregullimin dhe 4 ore per avarine. Per sa i perket volumit te ujit per mbrojtjen nga zjarri ai rezulton 500 m³ per kushtet e zonave. Ne tabelen e meposhtme jepet llogaritja per volumin e ujit te kerkuar te rezervuarit te ujesjellesit, bazuar ne prurjen maksimale ditore te nevojabe per uje te tyre.

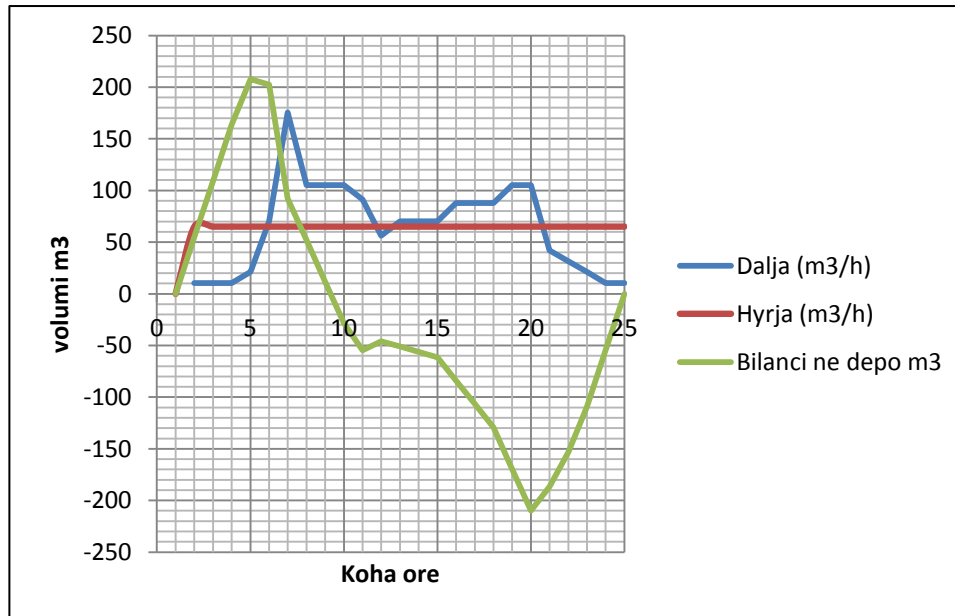
Volumi i ujit depos se ujesjellesit te furnizimit me uje per nevojat e zones eshte:

Koha (h)		Dalja (m ³ /h)	Hyrja (m ³ /h)	Bilanci ne depo m ³
0	0.162			0
1	0.162	10.54	65.09	54.54
2	0.162	10.54	65.09	109.09
3	0.162	10.54	65.09	163.63
4	0.324	21.09	65.09	207.63
5	1.079	70.23	65.09	202.49
6	2.697	175.54	65.09	92.03
7	1.618	105.31	65.09	51.81
8	1.618	105.31	65.09	11.59
9	1.618	105.31	65.09	-28.64
10	1.402	91.25	65.09	-54.80
11	0.863	56.17	65.09	-45.89
12	1.079	70.23	65.09	-51.03
13	1.079	70.23	65.09	-56.17
14	1.079	70.23	65.09	-61.31
15	1.348	87.74	65.09	-83.96
16	1.348	87.74	65.09	-106.61
17	1.348	87.74	65.09	-129.26
18	1.618	105.31	65.09	-169.49
19	1.618	105.31	65.09	-209.71
20	0.647	42.11	65.09	-186.74
21	0.485	31.57	65.09	-153.22
22	0.324	21.09	65.09	-109.22
23	0.162	10.54	65.09	-54.67

24	0.162	10.54	65.09	-0.13
Volumi llogarites i Depos (m3)				417.34
Volumi prespektiv i Depos (m3)				500

Kerkesa (l/s)	18.08
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Grafiku i volumit sipas kohes se Depos



a. Llogaritja e trashesise se tubit te celikut te dergimit te pompes:

$$\delta = \frac{P_t \times d}{2\sigma} + \beta$$

ku:

P_{tot} = presioni i pompes + grushtin hidraulik, ne kg/cm² (266 m: 10 kg/cm² = 26.6 kg/cm²);

d = diametri i brendshem i tubit, ne cm;

σ = sforcimet e lejuara te materialit ne kg/cm². (1600 kg/cm²);

β = rezerve per korozionin, ne cm.

$$\delta = \frac{26.6 \times 27}{2 \times 1600} + 0.15 = 0.28 + 0.15 = 0.69 \text{ cm.}$$

Trashesia e tubacionit meret $t = 7.11$ mm.

6. Llogaritjet e Linjave te Ujesjellesit

6.1 Llogaritjet hidraulike

Llogaritjet hidraulike per percaktimin e humbjeve te presionit ne linjat e ujesjellesit jane kryer me formulen Hazen Williams.. Humbjet e presionit sipas kesaj formule jane si me poshte.

$$H = 10.676 * \frac{Q^{1.852}}{C^{1.852} D^{4.87}} * L$$

ku;

Q prurja e ujit, m³/s

L Gjatesia e linjes, m

D Diametri i tubacionit, m

C Koeficienti Hazen Williams, = 100 - 120 per tuba celiku dhe 130-150 per tuba plastike.

13.2 Presionet e ujit ne pikat fundore

Presionet e ujit ne rrjetin shperndares duhet te jene te tilla qe te mundesohet furnizimi me uje i objekteve me vete rrjedhje. Sipas rekomandimeve duhet qe presionet e lira ne hyrje te objekteve te jene si me poshte:

Godina me nje dhe dy kate 10-12 m

Godina me tre kate 16 m

Godina me kater kate 20 m

Godina me pese kate 24 m

13.3 Llogaritjet e rrjetit shperndares

Per percaktimin e prurjeve maksimale orare te zonave te banuara zakonisht merret nje koeficient orar nga 1.5 deri ne 2, gjithmone ne funksion te numurit te banoreve.

Ky percaktim eshte i sakte kur zona e banuar eshte relativisht e madhe. Ne rastin e zonave te banuara reativisht te vogla. prurjet llogaritese maksimale percaktohen bazuar ne probabilitetin e veprimit te pajisjeve sanitare. Per percaktimin e ketyre prurjeve maksimale kemi pranuar shprehjen e meposhtme si me te thjeshte, qe merre parasysh sasine e ujit te pajisjeve ne pjesen e linjes se ujesjellesit dhe numrin e tyre ne kete pjese.

$$q_{\max} = \sum q_i \frac{1}{\sqrt{N-1}}$$

ku:

$\sum q_i$ - prurja maksimale e te gjitha paisjeve ne pjesen e linjes se ujesjellesit
 N - numuri total i paisjeve sanitare ne pjesen e linjes se ujesjellesit

Pajisjet sanitare te supozuara ne pjeset e rrjetit sipas zonave te banimit, zona e lagjeve jepen me poshte:

Pajisjet sanitare

No		

	Pajisjet sanitare	Prurja, L/s	Pajisjet sanitare	Prurja, L/s
1	Lavapjate 1 cope	0.2	Lavapjate 2 cope	0.4
2	Lavaman 1 cope	0.1	Lavaman 2 cope	0.2
3	WC 1 cope	0.1	WC 2 cope	0.2
4	Dush 1 cope	0.2	Dush 2 cope	0.4
Total	4 pajisje	0.6	8 Pajisje	1.2

Ne menyre te thjeshtuar prurja maksimale per rastin me 4 pajisje jepet me shprehjen e meposhtme:

$$q_{\max} = 0.6H \frac{\sqrt{N}}{N}$$

ku:

H - numuri i banesave.

N - numuri total i paisjeve sanitare ne pjesen e linjes se ujesjellesit.

Duke futur numrin e paisjeve te barabarte me 4 per cdo banese shprehja e prurjes maksimale jepet si me poshte vetem ne funksion te numurit te banesave:

$$q_{\max} = 0.3H^{0.5}$$

Per rastin me 8 pajisje sanitare ne apartament, sic mund te jete lagja e re 1 ne Golem, shprehja per prurja maksimale do te jete

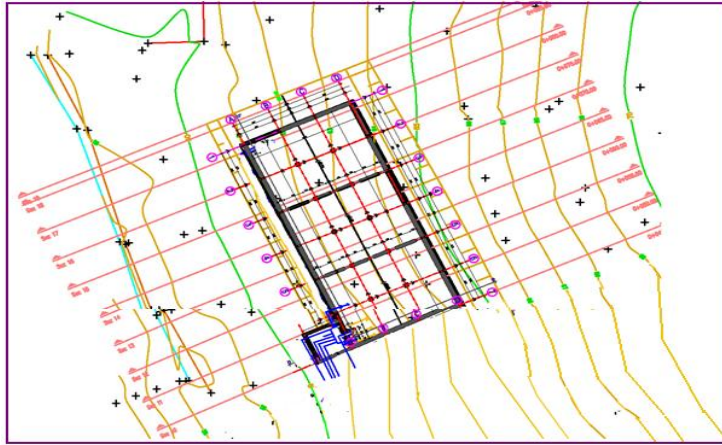
$$q_{\max} = 0.43H^{0.5}$$

7 Llogaritja konstruktive e depos se ujit 500 m3.

Llogaritja e Depos se ujit 500 m3 eshte bere ne perputhje me rekomandimet e normativave europiane Eurocode 2, 7 dhe 8.

Muret e strukture jane me trashesi 50 cm me beton C25/30.

Themeli eshte i tipit pllake me trashesi 50 cm. Soleta eshte tip monolit me trashesi 20 cm me sistem te kryqezuar traresh.



1. *Plani i depos*

7.1 Pershkrimi i struktures

7.1.1 Materialet e perdorura

Betoni

Jetegjatesia e vepres - 100 vite

Klasa e ekspozimit: XD2 (Muret, kolonat, pllaka e themelit)

Klasa e ekspozimit: XC2/XC3 (soleta)

Klasa e betonit: C25/30

Hekuri i armimit

Klasa e hekurit: "B" $f_{ys}=360MPa$

Shtresa mbrojtese

Themeli: 5cm

Muret anesor: 5cm

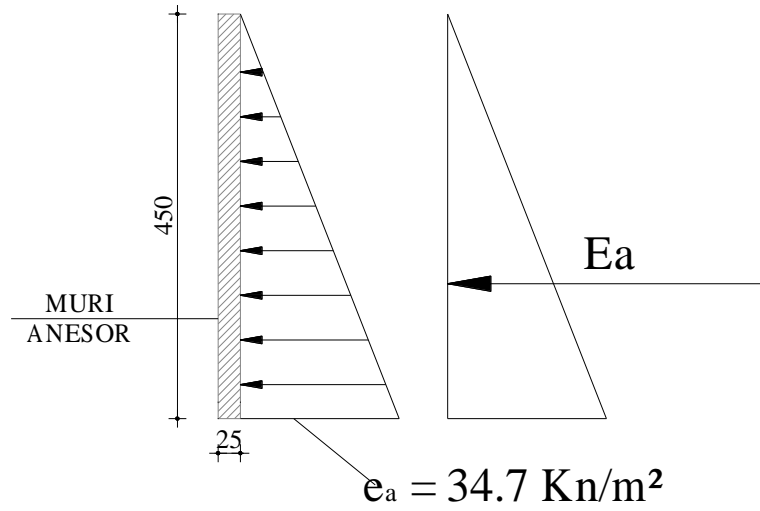
Kolonat: 5cm

Traret: 5cm

Soleta: 2.5cm

Forcat per llogaritjen statike .

Presioni aktiv I dheut ea .



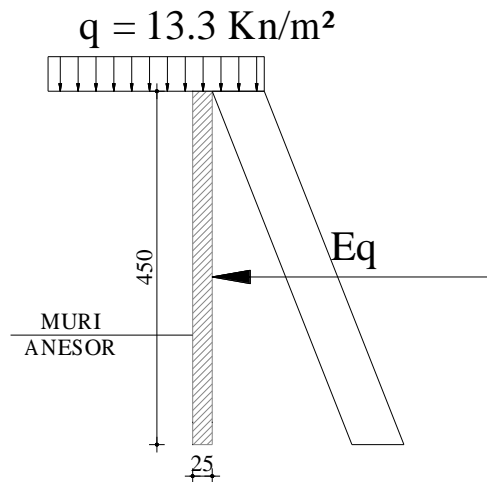
H e dheut = 4.5 m

Kendi I ferkimit $\varphi = 25^\circ$

Pasha vellimore = 19 kN/m³

$$e_a = \gamma \cdot H \cdot tg^2 \left(45 - \frac{\varphi}{2} \right) = 34.7 \frac{kN}{m^2}$$

Presioni aktiv I dheut e_a nga mbulimi me dhe.



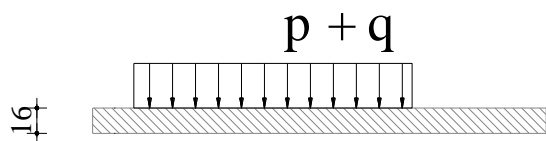
H e dheut = 0.7 m

Kendi I ferkimit $\varphi = 25^\circ$

Pasha vellimore = 19 kN/m³

$$E_q = q \cdot tg^2 \left(45 - \frac{\varphi}{2} \right) = 5.39 \frac{kN}{m^2}$$

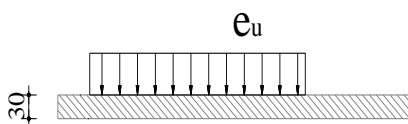
Presioni mbi solete nga betoni dhe mbulimi me dhe.



$$p = 3.84 \text{ Kn/m}^2 \text{ (pesha e betonit)}$$

$$q = 13.3 \text{ Kn/m}^2$$

Presioni mbi dysheme nga mbushja me uje.



$$e_u = 38 \text{ Kn/m}^2$$

8 Metoda e projektimit

8.1 Te pergjithshme

Strukturat qe jane te desitnuara per depozitimin e ujit pervec se duhet te permbushin kushtet normale ne qendrushmeri, solidited dhe deformim etj. duhet te perpushin dhe kushtet per mos rrjedhje nepermjet betonit.

Ne projektimin e strukturave te tilla eshte e zakonshme qe nese dimensionimi elementet jane dimensionuar dhe armuar per kushtet e mos-rrjedhjes atehere dhe soliditedi i elementeve eshte i garantuar. Strukturat uje-mbajtese eshte e rendesihme qe te dimensionohen duke patur parasysh kushte e mos-rrjedhjes se lengut, pasi nese nuk dimensionohen per keto kushte mirembajtja dhe riparimi i tyre eshte shume i kushtueshem. Nje tjeter kriter shume i rendesishem ne projektimin e strukturave uje-mbajtese eshte dhe projektimi i tyre per kushte extreme si psh termetet. Sipas Eurocode 8 keto tipe strukturash duhet te projektohen me faktor te sjelljes $q=1.0$ ose ne raste te vecanta $q=1.5$ qe dmth keto struktura duhet te jene funksionale dhe gjate termeteve te fuqishem shkaterues. Ne Eurocode kjo justifikohet me faktin se ujesjellesi furnizon me uje institucione te rendesishme si zjarr-fikeset, spitalet qendrat e emergjencave etj.

8.1.1 Standartet

Rezervuari do te llogaritet ne perputhje me metoden e gjendjeve kufitare.

Kodi ku do te bazohen llogaritjet eshte Eurocode, dhe me konkretisht:

- Eurocode 0, Bazat e projektimit.
- Eurocode 1, Forcat vepruese ne struktura
- EN 1991-1-5, Part 1-5: Forcat termike
- EN 1991-4, Part 4: Sillosat dhe rezervuaret
- Eurocode 2, Projektimi i strukturave betonarme
- EN 1992-1-1, Part 1-1: Rregulla te pergjithshme per ndertesat
- EN 1992-3, Part 3: Strukturat uje-mbajtese
- Eurocode 7, Projektimi gjeoteknik i strukturave betonarme
- EN 1997-1, Part 1: rregulla te pergjithshme.
-

9.1 Klasifikimi i forcave

9.1.1 Te perhershme

- Llogaritja e peshes vetiake te struktures

Soleta: $0.20m * 1m * 1m * 25.0 \text{ kN/m}^3 = 5.0 \text{ kN/m}^2$

Shtresat mbi solete:

Llac cemento; $0.06m * 1m * 1m * 20.0 \text{ kN/m}^3 = 1.200 \text{ kN/m}^2$

Hidroizolim: $0.02m * 1m * 1m * 18.0 \text{ kN/m}^3 = 0.36 \text{ kN/m}^2$

Muret anesore: $0.5m * 5.5m * 1m * 25.0 \text{ kN/m}^3 = 68.75 \text{ kN (per ml mur)}$

Themeli: $0.5m * 1m * 1m * 25.0 \text{ kN/m}^3 = 12.50 \text{ kN (per m}^2)$

Kolona: $0.5m * 0.5m * 25.0 \text{ kN/m}^3 = 6.25 \text{ kN (per ml kolone)}$

Pa=mbushja mbi solete: $0.5 * 1m * 1m * 18 \text{ kN/m}^3 = 9 \text{ kN (per m}^2)$

Presioni akstiv ne muret anesor: $p = \gamma * h * k_a$

ka- koeficienti i presionit aktiv (per kushte statike $\psi = 0$)

$$k_a = \frac{\left[\cos \left[(\phi - \psi - \beta) \cdot \frac{\pi}{180} \right] \right]^2}{\left[\cos \left(\frac{\pi}{180} \right) \cdot \cos \left(\beta \cdot \frac{\pi}{180} \right) \cdot \cos \left(\phi - 20^\circ \right) \cdot \cos \left[(\delta + \beta + \psi) \cdot \frac{\pi}{180} \right] \cdot \left[1 + \frac{\left[\frac{\sin \left[(\phi + \delta) \cdot \frac{\pi}{180} \right] \cdot \sin \left[(\phi - \psi - i) \cdot \frac{\pi}{180} \right]}{\cos \left[(\delta + \beta + \psi) \cdot \frac{\pi}{180} \right] \cdot \cos \left[(i - \beta) \cdot \frac{\pi}{180} \right]} \right]^2 \right]^2}$$

inklinimi i murit; $\beta = 0^\circ$

pjerresia e mbushjes: $i = 0^\circ$

kendi i ferkimit mur-terren: $\delta = 13^\circ$

duke aplikuar koeficientet e mesiperm ne formule marrim $k_a = 0.439$.

Vlera e koeficientit te presionit aktiv per kushte sizmike ($ag=0.22g$) eshte: $k_a = 0.524$.

9.1.1 Variable

Ne rastet kur ka presence uji diagram ndryshon fromen e saj. Pesha volumore e mbushjes llogaritet me formulen:

$$\square_a = \square_{sat} - \square_w$$

Ku \square_{sat} eshte pesha volumore e materialit mbushes dhe \square_w eshte pasha volumore e ujit.

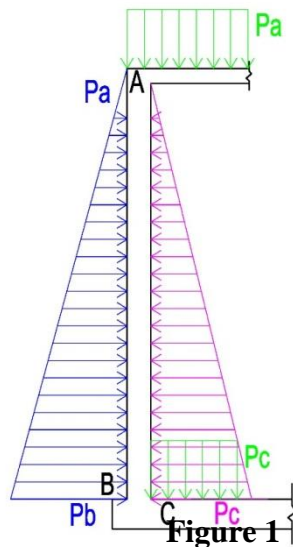
Vlera e presionit aktiv ne rastin e prezences se ujit eshte (ne fund te murit anesor):

- ne kushte statike

$$p_b = \gamma * h * k_a + \gamma_w * h = (20 - 10) \frac{kN}{m^3} * 5.5m * 0.439 + 10 \frac{kN}{m^3} * 5.5m = 79.145 \frac{kN}{m^2}$$

- ne kushte sizmike

$$p_b = \gamma * h * k_a + \gamma_w * h = (20 - 10) \frac{kN}{m^3} * 5.5m * 0.524 + 10 \frac{kN}{m^3} * 5.5m = 83.82 \frac{kN}{m^2}$$



Skema e ngarkesave te aplikuara

Presioni i ujit brenda rezervuarit:

- ne kushte statike:

$$p_c = \gamma_w * h = 10 \frac{kN}{m^3} * 3.6m = 36.0 \frac{kN}{m^2}$$

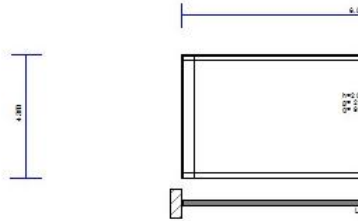
Llogaritje e soletes

1. SLAB-001

Two-way slab

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

C30/37-B500C



Concrete-Steel class: C30/37-B500C (EC2 §3)
 Environmental class : XC4 (EC2 §4.4.1)
 Concrete cover : Cnom=35 mm (EC2 §4.4.1)
 Concrete weight : 25.0 kN/m³
 $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N)
 $f_{cd}=acc \cdot f_{ck}/\gamma_c=0.85 \times 30/1.50=17.00$ MPa (EC2 §3.1.6)
 $f_{yd}=f_{yk}/\gamma_s=500/1.15=435$ MPa (EC2 §3.2.7)



1.1. Dimensions and loads

Slab thickness $h=0.200$ m, Spans $L_x=6.050$ m, $L_y=4.380$ m
 Slab loads: dead $g=(5.00+2.00)=7.00$ kN/m², live $q=9.00$ kN/m²
 Partial safety factors for actions : $\gamma_G=1.35$, $\gamma_Q=1.50$ (ECO Annex A1)
 Combination of variable actions : $\psi_0=0.70$, $\psi_1=0.60$, $\psi_2=0.30$
 Effective depth of cross section $d=h-d_l$, $d_l=C_{nom}+d/2=35+12/2=41$ mm, $d=200-41=159$ mm

Method of analysis: Czerny F., "Tafeln für vierseitig und dreiseitig gelagerte Rechteckplatten", Beton Kalender 1983, Berlin, Ernst Sohn, 1983
 $L_y/L_x=4.380/6.050=0.72$, Table 2.2.6

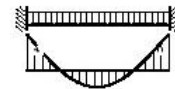
1.2. Ultimate limit state (ULS), design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Load (STR) $q_{ed}=\gamma_G \cdot g + \gamma_Q \cdot q = 1.35g + 1.50q = 1.35 \times 7.00 + 1.50 \times 9.00 = 22.95$ kN/m

1.2.1. Direction of slab analysis x-x, $L_x=6.05$ m

Moment at support $M_{edsupA}=(1.35 \times 7.00 + 1.50 \times 9.00) \times 4.380^2 / 17.50 = -25.16$ kNm/m
 Moment at support $M_{edsupB}=(1.35 \times 7.00 + 1.50 \times 9.00) \times 4.380^2 / 17.50 = -25.16$ kNm/m
 Moment at span $M_{edspan}=(1.35 \times 7.00 + 1.50 \times 9.00) \times 4.380^2 / 81.53 = 5.40$ kNm/m
 Reactions dead, $V_{gA}=7.00 \times 4.380 / 2.09 = 14.67$ V_{gB}= $7.00 \times 4.380 / 2.09 = 14.67$ kN/m
 Reactions live, $V_{qA}=9.00 \times 4.380 / 2.09 = 18.86$ V_{qB}= $9.00 \times 4.380 / 2.09 = 18.86$ kN/m



1.2.2. Direction of slab analysis y-y, $L_y=4.38$ m

Moment at support $M_{edsupA}=(1.35 \times 7.00 + 1.50 \times 9.00) \times 4.380^2 / 13.81 = -31.88$ kNm/m
 Moment at support $M_{edsupB}=(1.35 \times 7.00 + 1.50 \times 9.00) \times 4.380^2 / 13.81 = -31.88$ kNm/m
 Moment at span $M_{edspan}=(1.35 \times 7.00 + 1.50 \times 9.00) \times 4.380^2 / 32.42 = 13.58$ kNm/m
 Reactions dead, $V_{gA}=7.00 \times 4.380 / 1.93 = 15.91$ V_{gB}= $7.00 \times 4.380 / 1.93 = 15.91$ kN/m
 Reactions live, $V_{qA}=9.00 \times 4.380 / 1.93 = 20.45$ V_{qB}= $9.00 \times 4.380 / 1.93 = 20.45$ kN/m



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

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Med= 5.40kNm/m, d=147mm, Kd= 6.33 x/d=0.04 $\epsilon_c/\epsilon_{s1}=-0.9/20.0$ ks=2.33, As= 0.86cm²/m
Med= 13.58kNm/m, d=159mm, Kd= 4.31 x/d=0.06 $\epsilon_c/\epsilon_{s1}=-1.3/20.0$ ks=2.35, As= 2.01cm²/m
Med=-25.16kNm/m, d=159mm, Kd= 3.17 x/d=0.09 $\epsilon_c/\epsilon_{s1}=-2.0/20.0$ ks=2.38, As= 3.77cm²/m
Med=-31.88kNm/m, d=159mm, Kd= 2.82 x/d=0.11 $\epsilon_c/\epsilon_{s1}=-2.4/20.0$ ks=2.40, As= 4.81cm²/m

Minimum slab reinforcement, As>=0.26bd·fctm/fyk, (As= 2.40cm²/m) (EC2 §9.3.1)

minimum principal reinforcement $\emptyset 12/40.0$ (2.82cm²/m) , secondary $\emptyset 12/45.0$ (2.40cm²/m)

Span reinforcement: x-x $\emptyset 12/25.0$ (2.82cm²/m)
y-y $\emptyset 12/25.0$ (2.82cm²/m) , (bottom layer)
Reinforcement over supports: Left $\emptyset 12/20.0$ (3.77cm²/m)
Right $\emptyset 12/20.0$ (3.77cm²/m)
Bottom $\emptyset 12/20.0$ (4.81cm²/m)
Top $\emptyset 12/20.0$ (4.81cm²/m)

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

(EC2 EN1992-1-1:2004, §6.2, §9.2.2)

Maximum shear forces at distance d from support face maxV=50.33 kN/m

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)

$Vrdc=[Crdc \cdot k \cdot (100\rho_1 \cdot f_{ck})^{0.33} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$ (EC2 Eq.6.2.a)

$Vrdc > (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ (EC2 Eq.6.2.b)

Crdc=0.18/vc=0.18/1.50=0.120, fck=30.00MPa, bw=1000mm, d=159mm

k=1+√(200/d) <=2, k=2.00, k1=0.15

$\rho_1=As_1/(b_w \cdot d)=481/(1000 \times 159)=0.0030$

$v_{min}=0.035 \cdot k^{1.50} \cdot \sqrt{f_{ck}} = 0.54N/mm^2$ (EC2 Eq.6.3N)

Vrd,c(min)=0.001x(0.54)x1000x159=85.86kN/m

$Vrdc=0.001x[0.120x2.00x(0.30x30.00)^{0.33}]x1000x159=79.38$, Vrdc=Vrdc(min)=85.86kN/m

Ved=50.33 kN/m <= Vrdc=85.86 kN/m, Ved<=Vrdc shear reinforcement is not needed

1.5. Serviceability limit state (SLS)

(EC2 EN1992-1-1:2004, §7)

L=4.380m, b=1.000m, h=0.200m, d=0.159m

Load (quasi-permanent combination) qed=g+ψ2·q=7.00+0.30x9.00=9.70 kN/m

ly<Lx, Leff=4.380m, Med=(9.70/22.95)x13.58=5.74 kNm/m, Med(SLS)=5.74 kNm/m

Final creep coefficient φ(∞, to)=2.50 (EC2 §3.1.4, Annex B)

Total shrinkage strain εcs=-0.30o/o

vc=1.00, vs=1.00 (EC2 §2.4.2.4.2)

Modulus of elasticity of concrete Ecm=32GPa, Ec,eff=32/(1+2.50)=9.14GPa=9140MPa (EC2 Eq.7.20)

Modulus of elasticity of steel Es=200GPa=200000MPa

Modular ratio αe=Es/Ec=200/32.00=6.25, effective αe=Es/Ec,eff=200/9.14=21.88

Tension reinforcement: $\emptyset 12/400$

Reinforcement ratio ρ=As1/(b·d)=282/(1000x159)=0.002

1.5.1. State I (uncracked section) (SLS)

Bending stiffness of uncracked section, EI=(200/21.88)x(0.001x0.667)=6094 kNm²

S=As.zs1=(0.001)²x282x0.059=(0.001)x0.017 m³ (EC2 Eq.7.21)

Curvature due to moment 1/rM=5.739/6094=(0.001)x0.942 (1/m)

Curvature due to shrinkage 1/rcs=(0.001x0.30)x21.880x(0.017/0.667)=(0.001)x0.164 (1/m)

Total curvature 1/r=(0.001)x0.942+(0.001)x0.164=(0.001)x1.106 (1/m)

Cracking moment, Mcr=fctm·(I/y2)=2.9x(0.667/0.100)=19.33 kNm

1.5.2. State II (fully cracked section) (SLS)

ρ=As/(b·d)=0.002, n·αe=21.88, n·ρ=0.044, ξ=0.681, α=0.255, x=α·d=0.041m

Bending stiffness of fully cracked section, EI=ξ·Es·As·d²=0.681x200x282x0.159²=971 kNm²

S=As.zs1=(0.001)²x282x0.118=(0.001)x0.033 m³ (EC2 Eq.7.21)

Curvature due to moment 1/rM=5.739/971=(0.001)x5.908 (1/m)

Curvature due to shrinkage 1/rcs=(0.001x0.30)x21.880x(0.033/0.106)=(0.001)x0.329 (1/m)

Total curvature 1/r=(0.001)x5.908+(0.001)x0.329=(0.001)x6.236 (1/m)

Med=5.74 kNm, εc/εs=0.24/0.70, x=41mm, σs=140 N/mm²

1.5.3. Checking deflections without calculation (SLS)

(EC2 EN1992-1-1:2004, §7.4.2)

$$l/d = K[11 + 1.5 \sqrt{f_{ck}(\rho_o/\rho)} + 3.2 \sqrt{f_{ck}(\rho_o/\rho - 1)^{3/2}}] = 110.52 \quad (\text{EC2 Eq. 7.16a})$$

$$f_{ck} = 30.00 \text{ N/mm}^2, \rho_o = 0.001 \times \sqrt{30.00} = 0.005, \rho = 0.002, \rho' = 0.000, \rho < \rho_o, K = 1.5$$

$$l/d = (310/\sigma_s) \times (l/d), \sigma_s = 140 \text{ N/mm}^2, l/d = (310/140) \times 110.52 = 244.89 \quad (\text{EC2 Eq. 7.17})$$

$$l_{eff}/d = 4.380/0.159 = 27.55 < 244.89, \quad \text{Span/depth under limits}$$

1.5.4. Checking deflections by calculation (SLS)

(EN1992-1-1, §7.4.3)

$$M_{ed} = 5.74 < 0.70 \times M_{cr} = 0.70 \times 19.33 = 13.53 \text{ kNm}, \zeta = 0.00$$

(Eq. 7.19)

$$\text{Final curvature } (1/r) = 0.00 \times (0.001 \times 6.236) + (1 - 0.00) \times (0.001 \times 1.106) = (0.001) \times 1.106 (1/m)$$

(Eq. 7.18)

$$\beta = (M_a + M_b)/M_c = (31.88 + 31.88)/13.58 = 4.69, k = 0.104 (1 - 4.69/10) = 0.0552$$

$$f = k \cdot l_{eff}^2 \cdot (1/r) = 0.0552 \times 4.380^2 \times 1.106 = 1.2 \text{ mm}$$

$$f = 1.17 < 1000 \times 4.380/250 = 17.5 \text{ mm}, \quad \text{Deflection under limits}$$

1.5.5. Minimum reinforcement areas (SLS)

(EC2 EN1992-1-1:2004, §7.3.2)

$$\text{Minimum reinforcement areas } A_{s,min} = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} / \sigma_s \quad (\text{EC2 Eq. 7.1})$$

$$b = 1.000 \text{ m}, b_{eff} = 1.000 \text{ m}, h = 0.200 \text{ m}, d = 0.159 \text{ m}, x = 0.041 \text{ m}, \emptyset = 12 \text{ mm}$$

$$N_{ed} = 0.00 \text{ kN}, \sigma_c = (N_{ed}/bh) = 0.0 \text{ N/mm}^2, \sigma_s = 140 \text{ N/mm}^2$$

$$A_{ct} = (h - x) \cdot b = (200 - 41) \times 1000 = 159408 \text{ mm}^2$$

$$\max(h, b) = 0 \text{ mm}, f_{ctm} = 2.90 \text{ N/mm}^2, A_{c,eff} = 159408 \text{ mm}^2, k = 1.00, k_c = 0.40, k_1 = 1.50$$

$$\text{Minimum reinforcement, } A_{s,min} = 0.40 \times 1.00 \times 2.90 \times 159408 / 140 = 1322 \text{ mm}^2/\text{m}$$

1.5.6. Control of cracking without direct calculation (SLS)

(EC2 EN1992-1-1:2004, §7.3.4)

$$\text{crack width } w_k = 0.3 \text{ mm (XC4)}, \text{ steel stress } \sigma_s = 140 \text{ N/mm}^2, \Phi^* = 25 \text{ mm}, \max s = 250 \text{ mm} \quad (\text{EC2 T.7.2N T.7.3N})$$

$$\emptyset s = \emptyset^* (f_{ctm}/2.9) [k_c \cdot h_{cr} / (2(h-d))] = 13 \text{ mm} \quad (\text{EC2 Eq. 7.6N})$$

$$f_{ctm} = 2.90 \text{ N/mm}^2, k_c = 0.40, h_{cr} = 0.5 \times 200 = 100 \text{ mm}, h = 200 \text{ mm}, d = 159 \text{ mm}$$

$$\text{Maximum bar diameter } \emptyset = 13 \text{ mm}, \text{ maximum bar spacing } s = 250 \text{ mm}$$

$$\text{Bar diameter } \emptyset = 12 < 13 \text{ mm}, \quad \text{Bar diameter under maximum limit}$$

1.5.7. Calculation of crack width (SLS)

(EC2 EN1992-1-1:2004, §7.3.3)

$$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \quad (\text{EC2 Eq. 7.8})$$

$$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot (f_{ct,eff}/\rho_{eff}) (1 + \alpha_e \cdot \rho_{eff})] / E_s > 0.6 \sigma_s / E_s \quad (\text{EC2 Eq. 7.9})$$

$$\sigma_s = 140 \text{ N/mm}^2, \text{ short term loading: } \alpha_e = 6.25, k_t = 0.6, \text{ long term loading: } \alpha_e = 21.88, k_t = 0.4$$

$$A_{c,eff} = 0.333 (h - x) b = 0.333 \times (200 - 41) \times 1000 = 53083 \text{ mm}^2 \quad (\text{§7.3.2.3})$$

$$\rho_{eff} = A_s / A_{c,eff} = 282 / 53083 = 0.005$$

$$\epsilon_{sm} - \epsilon_{cm} = [140 - 0.4 \times (2.9 / 0.005) (1 + 21.88 \times 0.005)] / 200 = -0.52 \text{ o/oo} > 0.6 \times 140 / 200 = 0.42 \text{ o/oo}$$

$$s_{r,max} = k_3 \cdot C_{nom} + k_1 \cdot k_2 \cdot k_4 \cdot \emptyset / \rho_{eff} \quad (\text{EC2 Eq. 7.11})$$

$$\emptyset = 12 \text{ mm}, k_1 = 0.8, k_2 = (e_1 + e_2) / 2e_1 = 0.5, k_3 = 3.4, k_4 = 0.425$$

$$s_{r,max} = 3.4 \times 35.00 + 0.8 \times 0.5 \times 0.425 \times 12 / 0.005 = 503.00 \text{ mm}$$

$$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = 503.00 \times 0.001 \times 0.42 = 0.21 \text{ mm}$$

$$w_k = 0.21 \text{ mm} < 0.30 \text{ mm} = w_{max}, \text{ Environmental class: XC4}, \quad \text{Crack width under limit}$$

Skema statike e rezervuarit

Rezervuari eshte llogaritur me ndihmen e software-it Sap2000 duke e modeluar sipas metodës se Elementeve te Fundem.

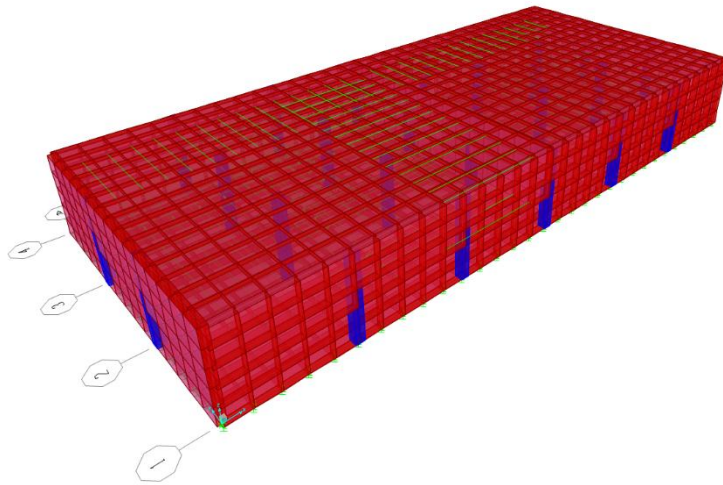
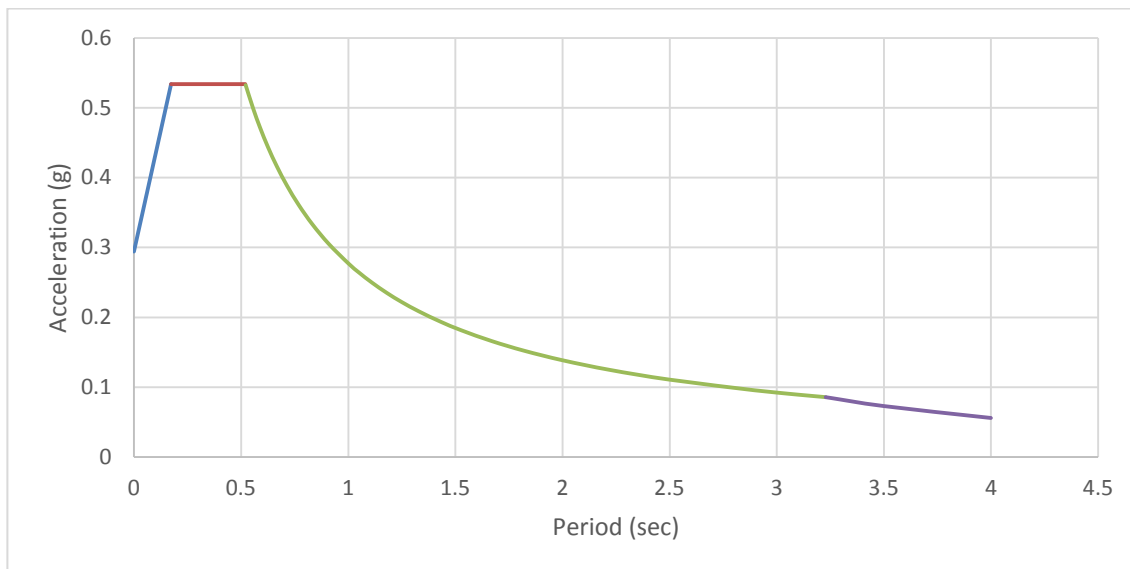


Figure Error! No text of specified style in document.-2 Pamje 3D e rezervuarit
 Skema e llogaritjes e plakes se themelit eshte si pllake mbi bazament elastik. Efekti i deformimit te dheut nen themel do te meret parasysh duke vendosur ne modelin llogarites susta. The foundation scheme is: slab of elastic soil. The soil will be modelled with springs that reflect the soil deformation characteristics.



3 spektri reagimit

Te dhena sizmike

Sipas hartes sizmike te Shqiperise zona ku do ndertohet rezervuari eshte zone me intensitet te larte sizmik, dhe i perket shkalles 8 sipas klasifikimit MSK-64. Akseleracioni sizmik eshte mare $a_g=0.22$. Kategoria e truallit sipas klasifikimit te Eurocode 7 eshte kategoria C. Faktori I sjelljes per projektimin sipas gjendjeve kufitare eshte $q=1$ per gjendjen kufitare te sherbimit dhe $q=1.5$ per gjendjen kufitare te shkaterimit.

Kontrolli i plasaritjeve

Per kontrollin e plasaritjeve eshte pranuar qe ato te behen sipas klases 2 qe jep Eurocode. Klasa 2 dhe 3 parashikon qe plasaritjet te mos jene te vazhduara ne gjeresine e seksionit.

Rekomandime per madhesine e plasaritjeve per klasen 2 jepen ne EN1992-3:

Rekomandimi per madhesine e plasaritjeve eshte funksion i koeficientit h_D/h :

$$h_D/h \leq 5 \text{ wkl is } 0,2 \text{ mm}$$

$$h_D/h \geq 35 \text{ wkl is } 0,05 \text{ mm.}$$

Rezultatet e llogaritjeve

Ne paragrafin me poshte jepet dhe llogaritja e detajuar e momenteve ne muret anesor.

Analiza sizmike ne moden impulsive dhe konvektive:

Analiza sizmike ne drejtimin y-y:

a-3 Pesha e themelit te rezervuarit

$$W_f := (L_r + 2 \cdot d_r) \cdot (B_r + 2 \cdot d_r) \cdot d_f \cdot \gamma_{conc} = 2543.625 \quad [\text{kN}]$$

Masa e themelit te rezervuarit

$$m_f := \frac{(L_r + 2 \cdot d_r) \cdot (B_r + 2 \cdot d_r) \cdot d_f \cdot \gamma_{conc} \cdot 1000}{9.81} = 259289 \quad [\text{kg}]$$

a-4 Pesha e ujit

$$W_{wat} := L_r \cdot B_r \cdot d_w \cdot \gamma_{wat} = 6326.64 \quad [\text{kN}]$$

Masa e ujit

$$m_{wat} := \frac{L_r \cdot B_r \cdot d_w \cdot \gamma_{wat} \cdot 1000}{9.81} = 644917 \quad [\text{kg}]$$

Analiza ne drejtimin gjatesor

a- Parametrat e modelit suste - mas

$$\frac{m_i}{m_{wat}} = \frac{\tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)}{0.866 \cdot \frac{L_r}{d_w}} = 0.274 \quad m_i := m_{wat} \cdot \frac{\tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)}{0.866 \cdot \frac{L_r}{d_w}} = 176719 \quad [\text{kg}]$$

$$h_i := d_w \cdot 0.375 = 1.35 \quad \text{for } d_w / L_r < 0.75$$

$$h_{i1} := 0.5 - \frac{0.09375}{\frac{d_w}{L_r}} = 0.105 \quad \text{for } d_w / L_r > 0.75$$

for $d_w / L_r < 1.33$

$$\frac{h_{sylv}}{d_w} = \frac{0.866 \cdot \frac{L_r}{d_w}}{2 \cdot \tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)} - 0.125 = 1.7$$

$$h_{\text{cyl}} := d_w \cdot \frac{0.866 \cdot \frac{L_r}{d_w}}{2 \cdot \tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)} - 0.125 = 6.4 \quad [\text{m}]$$

$$\frac{m_c}{m_{\text{wat}}} = 0.264 \cdot \frac{\tanh\left(3.16 \cdot \frac{d_w}{L_r}\right)}{\frac{d_w}{L_r}} = 0.71$$

$$m_c := m_{\text{wat}} \cdot 0.264 \cdot \frac{\tanh\left(3.16 \cdot \frac{d_w}{L_r}\right)}{\frac{d_w}{L_r}} = 455467 \quad [\text{kg}]$$

$$\frac{h_c}{d_w} = \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 1}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} = 0.52$$

$$h_c := d_w \cdot \left(1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 1}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} \right) = 1.88 \quad [\text{m}]$$

$$\frac{h_{\text{cyl}}}{d_w} = 1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 2.01}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} = 2.16$$

$$h_{\text{cyl}} := d_w \cdot \left(1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 2.01}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} \right) = 7.8 \quad [\text{m}]$$

$$K_c := 0.833 \cdot \frac{m_{wat} \cdot 9.81}{d_w} \cdot \left(\tanh \left(3.16 \cdot \frac{d_w}{L_r} \right) \right)^2 = 5.916 \cdot 10^5 \quad [\text{N/m}]$$

b- Perioda e lekundjeve

b-1 Perioda ne moden impulsive

$$m_{w.def} := B_r \cdot d_r \cdot h_r \cdot \gamma_{conc} \cdot \frac{1000}{9.81} = 70948 \quad [\text{kg}] \quad [\text{masa e murit te rez. ne drejtimin pingul me sizmen}]$$

$$h_{def} := \frac{\frac{m_i}{2} \cdot h_i + m_{w.def} \cdot \frac{d_w}{2}}{\frac{m_i}{2} + m_{w.def}} = 1.55 \quad [\text{m}]$$

$$q_{def} := \frac{\left(\frac{m_i}{2} + m_{w.def} \right) \cdot 9.81}{B_r \cdot d_w \cdot 1000} = 37 \quad [\text{kN/m}^2]$$

Deformimi i mureve nen veprimin e presionit q_{def} ne lartesine h_{def} eshte:

$$d_{def} := 0.0065 \quad [\text{mm}]$$

$$\text{Perioda ne moden impulsive} \quad T_i := 2 \cdot \pi \cdot \sqrt{\frac{d_{def}}{9.81}} = 0.162$$

b-2 Perioda ne moden konvektive

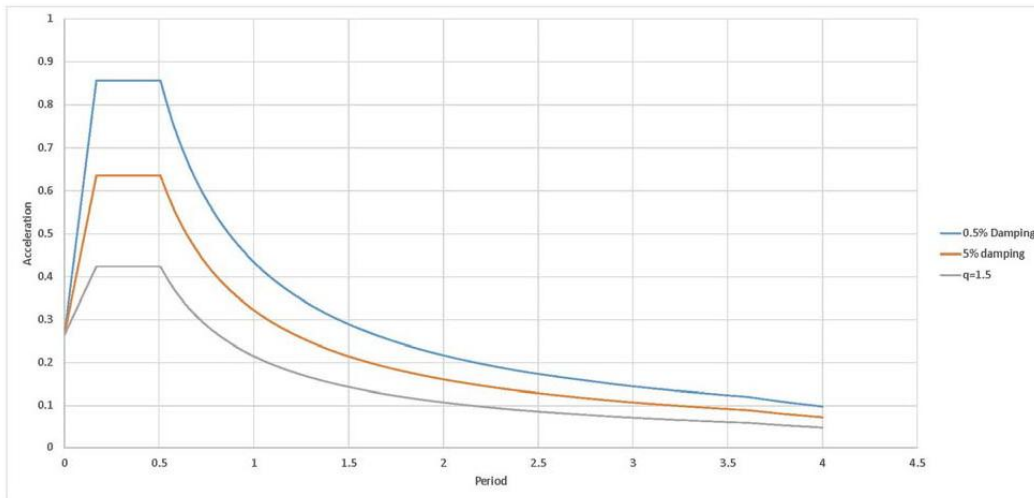
$$C_c := 4.25 \quad [\text{coefficient of convective mode time period}]$$

$$T_c := C_c \cdot \sqrt{\frac{L_r}{9.81}} = 5.282 \quad [\text{sec}]$$

c -forca prerese ne baze

Sjellja elastike do te modifikohet duke mare parasysh efektin e reduktimit si pasoje e krijimit te kunder-valeve

$$\eta := \sqrt{\frac{10}{5 + 0.5}} = 1.348$$



$$S_{eTimp} := 0.385 \quad \text{[vlera e projektit per moden impulsive]}$$

$$S_{eTcon} := 0.1 \quad \text{[vlera e projektit per moden konvektive]}$$

$$Q := \langle (m_i + m_w + m_s) \cdot S_{eTimp} + m_c \cdot S_{eTcon} \rangle \cdot \frac{9.81}{1000} = 2834 \quad \text{[kN]}$$

d - Momentet perkules

$$h_w := 2.4 \quad \text{[m]} \quad \text{qendra e gravitetit te murit}$$

$$h_s := 4.9 \quad \text{[m]} \quad \text{qendra e gravitetit e soletes}$$

d-1 Momenti perkules ne fund te murit ne moden impulsive

$$M_{imp} := \langle m_i \cdot h_i + m_w \cdot h_w + m_s \cdot h_s \rangle \cdot S_{eTimp} \cdot \frac{9.81}{1000} = 6008 \quad \text{[kNm]}$$

d-2 Momenti perkules ne fund te murit ne moden konvektive

$$M_{con} := m_c \cdot h_c \cdot S_{eTcon} \cdot \frac{9.81}{1000} = 840 \quad \text{[kNm]}$$

$$M_{total} := \sqrt{M_{imp}^2 + M_{con}^2} = 6067 \quad [\text{kNm}] \quad M_{total.1m} := \frac{M_{total}}{B_r} = 523 \quad [\text{kNm}]$$

e- Momentet

e1- Momentet perkulese ne fund te murit ne moden impulsive

$$M_{imp.over} := (m_i \cdot (h_{cyl} + d_f) + m_w \cdot (h_w + d_f) + m_s \cdot (h_s + d_f)) \cdot S_{eTimp} \cdot \frac{9.81}{1000} = 10602 \quad [\text{kNm}]$$

e2- Momentet perkulese ne fund te murit ne moden konvektive

$$M_{con.over} := m_c \cdot (h_{cyl} + d_f) \cdot S_{eTcon} \cdot \frac{9.81}{1000} = 3691 \quad [\text{kNm}]$$

$$M_{total.over} := \sqrt{M_{imp.over}^2 + M_{con.over}^2} = 11226 \quad [\text{kNm}]$$

f- Presioni hidrodinamik

f1- Presioni hidrodinamik ne moden impulsive

ne murin anesor

$$y_1 := 0 \quad Q_{iw} := 0.866 \cdot \left(1 - \left(\frac{y_1}{d_w}\right)^2\right) \cdot \tanh\left(0.866 \cdot \frac{L_r}{d_w}\right) = 0.865$$

$$P_{iw1} := Q_{iw} \cdot S_{eTimp} \cdot \gamma_{wat} \cdot d_w = 11.986 \quad [\text{kN/m}^2]$$

ne baze te themelit

$$x_1 := \frac{L_r}{2} \quad Q_{ib} := \frac{\sinh\left(1.732 \cdot \frac{x_1}{L_r}\right)}{\cosh\left(0.866 \cdot \frac{L_r}{d_w}\right)} = 0.051$$

$$P_{iw2} := Q_{ib} \cdot S_{eTimp} \cdot \gamma_{wat} \cdot d_w = 0.708 \quad [\text{kN/m}^2]$$

f2- Presioni hidrodinamik ne moden konvektive

ne muret anesor

$$y_{c1} := 0 \quad Q_{cw1} := 0.4165 \cdot \frac{\cosh\left(3.162 \cdot \frac{y_{c1}}{L_r}\right)}{\cosh\left(3.162 \cdot \frac{d_w}{L_r}\right)} = 0.321$$

$$p_{cw1} := Q_{cw1} \cdot S_{eTcon} \cdot \gamma_{wat} \cdot L_r = 4.87 \quad [\text{kN/m}^2]$$

ne baze te themelit

$$y_{c2} := d_w \quad Q_{cw2} := 0.4165 \cdot \frac{\cosh\left(3.162 \cdot \frac{y_{c2}}{L_r}\right)}{\cosh\left(3.162 \cdot \frac{d_w}{L_r}\right)} = 0.417$$

$$p_{cw2} := Q_{cw2} \cdot S_{eTcon} \cdot \gamma_{wat} \cdot L_r = 6.31 \quad [\text{kN/m}^2]$$

f3- Presioni hidrodinamik ne moden konvektive ne baze (y=0)

$$y_{cb} := 0 \quad x_2 := \frac{L_r}{2} \quad Q_{cb} := 1.25 \cdot \left(\frac{x_2}{L_r} - \frac{4}{3} \cdot \left(\frac{x_2}{L_r} \right)^3 \right) \cdot \operatorname{sech}\left(3.162 \cdot \frac{d_w}{L_r}\right) = 0.322$$

$$p_{cb} := Q_{cb} \cdot S_{eTcon} \cdot \gamma_{wat} \cdot L_r = 4.871 \quad [\text{kN/m}^2]$$

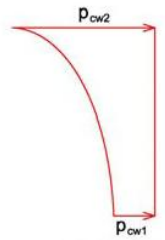
g- Presioni nga inercia e mureve anesor

$$p_{ww} := S_{eTimp} \cdot d_r \cdot \gamma_{conc} = 4.813 \quad [\text{kN/m}^2]$$

h- Presioni nga sizma vertikave

$$S_{eTv} := 0.25 \quad y_v := 0 \quad p_v := S_{eTv} \cdot \left(\gamma_{wat} \cdot d_w \cdot \left(1 - \frac{y_v}{d_w} \right) \right) = 9 \quad [\text{kN/m}^2]$$

$$b_c := \frac{q_c}{d_w^2} \cdot \langle 6 \cdot h_i - 2 \cdot d_w \rangle = 1.337 \quad [\text{kN/m}^2]$$



Real distribution



Linear idealization distribution

lartesia e vales gjate sizmes

$$R_{fix} := 2$$

$$d_{wave,max} := S_{\epsilon Tecm} \cdot R_{fix} \cdot \frac{L_r}{2} = 1.515$$

16.1.1. naliza sizmike ne drejtimin x- x

Te dhena

$L_r := 11.60$	[m]	dimensioni i depos ne drejtimin gjatesor
$B_r := 15.15$	[m]	dimensioni i dhomes se depos ne drejtimin terthor
$h_r := 4.8$	[m]	lartesia e rezervuarit
$d_r := 0.5$	[m]	trashesia e murit
$d_s := 0.20$	[m]	trashesia e soletes
$d_f := 0.5$	[m]	trashesia e themelit
$d_w := 3.6$	[m]	niveli ujit
$\gamma_{conc} := 25$	[kN/m ³]	pesha volumore e betonit
$\gamma_{wat} := 10$	[kN/m ³]	pesha volumore e ujit

a- Llogaritja e peshave

a-1 Pesha e mureve te rezervuarit

$$W_w := 2 \cdot (L_r + 2 \cdot d_r + B_r + 2 \cdot d_r) \cdot d_r \cdot h_r \cdot \gamma_{conc} = 3450 \quad [\text{kN}]$$

Masa e mureve te rezervuarit

$$m_w := \frac{2 \cdot (L_r + 2 \cdot d_r + B_r + 2 \cdot d_r) \cdot d_r \cdot h_r \cdot \gamma_{conc} \cdot 1000}{9.81} = 351682 \quad [\text{kg}]$$

a-2 Pesha e soletes se rezervuarit

$$W_s := (L_r + 2 \cdot d_r) \cdot (B_r + 2 \cdot d_r) \cdot d_s \cdot \gamma_{conc} = 1017.45 \quad [\text{kN}]$$

Masa e soletes se rezervuarit

$$m_s := \frac{(L_r + 2 \cdot d_r) \cdot (B_r + 2 \cdot d_r) \cdot d_s \cdot \gamma_{conc} \cdot 1000}{9.81} = 103716 \quad [\text{kg}]$$

a-3 Pesha e themelit te rezervuarit

$$W_f := (L_r + 2 \cdot d_r) \cdot (B_r + 2 \cdot d_r) \cdot d_f \cdot \gamma_{conc} = 2543.625 \quad [\text{kN}]$$

Masa e themelit te rezervuarit

$$m_f := \frac{(L_r + 2 \cdot d_r) \cdot (B_r + 2 \cdot d_r) \cdot d_f \cdot \gamma_{conc} \cdot 1000}{9.81} = 259289 \quad [\text{kg}]$$

a-4 Pesha e ujit

$$W_{wat} := L_r \cdot B_r \cdot d_w \cdot \gamma_{wat} = 6326.64 \quad [\text{kN}]$$

Masa e ujit

$$m_{wat} := \frac{L_r \cdot B_r \cdot d_w \cdot \gamma_{wat} \cdot 1000}{9.81} = 644917 \quad [\text{kg}]$$

Analiza ne drejtimin gjatesor

a- Parametrat e modelit suste - mas

$$\frac{m_i}{m_{wat}} = \frac{\tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)}{0.866 \cdot \frac{L_r}{d_w}} = 0.356 \quad m_i := m_{wat} \cdot \frac{\tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)}{0.866 \cdot \frac{L_r}{d_w}} = 229381 \quad [\text{kg}]$$

$$h_i := d_w \cdot 0.375 = 1.35 \quad \text{for } d_w / L_r < 0.75$$

$$h_{i1} := 0.5 - \frac{0.09375}{\frac{d_w}{L_r}} = 0.198 \quad \text{for } d_w / L_r > 0.75$$

for $d_w / L_r < 1.33$

$$\frac{h_{iyll}}{d_w} = \frac{0.866 \cdot \frac{L_r}{d_w}}{2 \cdot \tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)} - 0.125 = 1.28$$

$$h_{qyll} := d_w \cdot \frac{0.866 \cdot \frac{L_r}{d_w}}{2 \cdot \tanh\left(0.866 \cdot \frac{L_r}{d_w}\right)} - 0.125 = 4.9 \quad [\text{m}]$$

$$\frac{m_c}{m_{wat}} = 0.264 \cdot \frac{\tanh\left(3.16 \cdot \frac{d_w}{L_r}\right)}{\frac{d_w}{L_r}} = 0.64$$

$$m_c := m_{wat} \cdot 0.264 \cdot \frac{\tanh\left(3.16 \cdot \frac{d_w}{L_r}\right)}{\frac{d_w}{L_r}} = 413303 \quad [\text{kg}]$$

$$\frac{h_c}{d_w} = 1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 1}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} = 0.54$$

$$h_c := d_w \cdot \left(1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 1}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} \right) = 1.93 \quad [\text{m}]$$

$$\frac{h_{cyll}}{d_w} = 1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 2.01}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} = 1.44$$

$$h_{cyll} := d_w \cdot \left(1 - \frac{\cosh\left(3.16 \cdot \frac{d_w}{L_r}\right) - 2.01}{3.16 \cdot \frac{d_w}{L_r} \cdot \sinh\left(3.16 \cdot \frac{d_w}{L_r}\right)} \right) = 5.2 \quad [\text{m}]$$

$$K_c := 0.833 \cdot \frac{m_{wat} \cdot 9.81}{d_w} \cdot \left(\tanh \left(3.16 \cdot \frac{d_w}{L_r} \right) \right)^2 = 8.309 \cdot 10^5 \quad [\text{N/m}]$$

b- Perioda e lekundjeve

b-1 Perioda ne moden impulsive

$$m_{w,def} := B_r \cdot d_r \cdot h_r \cdot \gamma_{conc} \cdot \frac{1000}{9.81} = 92661 \quad [\text{kg}] \quad [\text{masa e murit te rez. ne drejtimin pingul me sizmen}]$$

$$h_{def} := \frac{\frac{m_i}{2} \cdot h_i + m_{w,def} \cdot \frac{d_w}{2}}{\frac{m_i}{2} + m_{w,def}} = 1.551 \quad [\text{m}]$$

$$q_{def} := \frac{\left(\frac{m_i}{2} + m_{w,def} \right) \cdot 9.81}{B_r \cdot d_w \cdot 1000} = 37 \quad [\text{kN/m}^2]$$

Deformimi i mureve nen veprimin e presionit q_{def} ne lartesine h_{def} eshte:

$$d_{def} := 0.0068 \quad [\text{mm}]$$

$$\text{Perioda ne moden impulsive} \quad T_i := 2 \cdot \pi \cdot \sqrt{\frac{d_{def}}{9.81}} = 0.165$$

b-2 Perioda ne moden konvektive

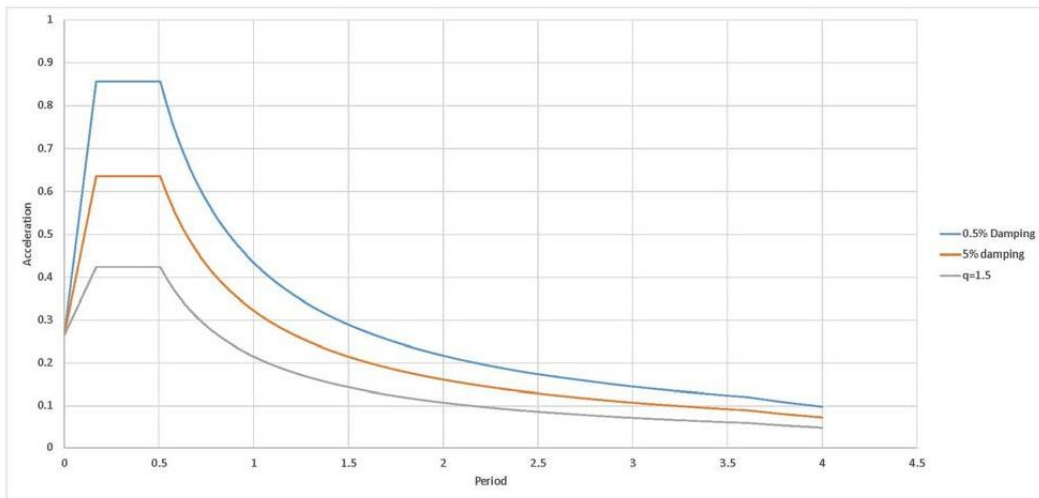
$$C_c := 4.25 \quad [\text{coefficient of convective mode time period}]$$

$$T_c := C_c \cdot \sqrt{\frac{L_r}{9.81}} = 4.622 \quad [\text{sec}]$$

c - forca prerese ne baze

Sjellja elastike do te modifikohet duke mare parasysh efektin e reduktimit si pasoje e krijimit te kunder-valeve

$$\eta := \sqrt{\frac{10}{5 + 0.5}} = 1.348$$



$$S_{eT\text{imp}} := 0.380 \quad [\text{vlera e projektit per moden impulsive}]$$

$$S_{eT\text{con}} := 0.1 \quad [\text{vlera e projektit per moden konvektive}]$$

$$Q := \langle (m_i + m_w + m_s) \cdot S_{eT\text{imp}} + m_c \cdot S_{eT\text{con}} \rangle \cdot \frac{9.81}{1000} = 2958 \quad [\text{kN}]$$

d - Momentet perkulese

$$h_w := 2.4 \quad [\text{m}] \quad \text{qendra e gravitetit te murit}$$

$$h_s := 4.9 \quad [\text{m}] \quad \text{qendra e gravitetit e soletes}$$

d-1 Momenti perkules ne fund te murit ne moden impulsive

$$M_{\text{imp}} := \langle m_i \cdot h_i + m_w \cdot h_w + m_s \cdot h_s \rangle \cdot S_{eT\text{imp}} \cdot \frac{9.81}{1000} = 6195 \quad [\text{kNm}]$$

d-2 Momenti perkules ne fund te murit ne moden konvektive

$$M_{\text{con}} := m_c \cdot h_c \cdot S_{eT\text{con}} \cdot \frac{9.81}{1000} = 783 \quad [\text{kNm}]$$

$$M_{total} := \sqrt{M_{imp}^2 + M_{con}^2} = 6245 \quad [\text{kNm}] \quad M_{total.1m} := \frac{M_{total}}{B_r} = 412 \quad [\text{kNm}]$$

e- Momentet

e1- Momentet perkulose ne fund te murit ne moden impulsive

$$M_{imp.over} := (m_i \cdot (h_{cyl} + d_f) + m_w \cdot (h_w + d_f) + m_s \cdot (h_s + d_f)) \cdot S_{eTimp} \cdot \frac{9.81}{1000} = 10538 \quad [\text{kNm}]$$

e2- Momentet perkulose ne fund te murit ne moden konvektive

$$M_{con.over} := m_c \cdot (h_{cyl} + d_f) \cdot S_{eTcon} \cdot \frac{9.81}{1000} = 2298 \quad [\text{kNm}]$$

$$M_{total.over} := \sqrt{M_{imp.over}^2 + M_{con.over}^2} = 10785 \quad [\text{kNm}]$$

f- Presioni hidrodinamik

f1- Presioni hidrodinamik ne moden impulsive

ne murin anesor

$$y_1 := 0 \quad Q_{iw} := 0.866 \cdot \left(1 - \left(\frac{y_1}{d_w}\right)^2\right) \cdot \tanh\left(0.866 \cdot \frac{L_r}{d_w}\right) = 0.859$$

$$p_{iw1} := Q_{iw} \cdot S_{eTimp} \cdot \gamma_{wat} \cdot d_w = 11.758 \quad [\text{kN/m}^2]$$

ne baze te themelit

$$x_1 := \frac{L_r}{2} \quad Q_{ib} := \frac{\sinh\left(1.732 \cdot \frac{x_1}{L_r}\right)}{\cosh\left(0.866 \cdot \frac{L_r}{d_w}\right)} = 0.12$$

$$p_{iw2} := Q_{ib} \cdot S_{eTimp} \cdot \gamma_{wat} \cdot d_w = 1.637 \quad [\text{kN/m}^2]$$

f2- Presioni hidrodinamik ne moden konvektive

ne muret anesor

$$y_{c1} := 0 \quad Q_{cw1} := 0.4165 \cdot \frac{\cosh\left(3.162 \cdot \frac{y_{c1}}{L_r}\right)}{\cosh\left(3.162 \cdot \frac{d_w}{L_r}\right)} = 0.274$$

$$p_{cw1} := Q_{cw1} \cdot S_{eTcon} \cdot \gamma_{wat} \cdot L_r = 3.176 \quad [\text{kN/m}^2]$$

ne baze te themelit

$$y_{c2} := d_w \quad Q_{cw2} := 0.4165 \cdot \frac{\cosh\left(3.162 \cdot \frac{y_{c2}}{L_r}\right)}{\cosh\left(3.162 \cdot \frac{d_w}{L_r}\right)} = 0.417$$

$$p_{cw2} := Q_{cw2} \cdot S_{eTcon} \cdot \gamma_{wat} \cdot L_r = 4.831 \quad [\text{kN/m}^2]$$

f3- Presioni hidrodinamik ne moden konvektive ne baze ($y=0$)

$$y_{cb} := 0 \quad x_2 := \frac{L_r}{2} \quad Q_{cb} := 1.25 \cdot \left(\frac{x_2}{L_r} - \frac{4}{3} \cdot \left(\frac{x_2}{L_r} \right)^3 \right) \cdot \operatorname{sech}\left(3.162 \cdot \frac{d_w}{L_r}\right) = 0.274$$

$$p_{cb} := Q_{cb} \cdot S_{eTcon} \cdot \gamma_{wat} \cdot L_r = 3.177 \quad [\text{kN/m}^2]$$

g- Presioni nga inercia e mureve anesor

$$p_{ww} := S_{eTimp} \cdot d_r \cdot \gamma_{conc} = 4.75 \quad [\text{kN/m}^2]$$

h- Presioni nga sizma vertikave

$$S_{eTv} := 0.25 \quad y_v := 0 \quad p_v := S_{eTv} \cdot \left(\gamma_{wat} \cdot d_w \cdot \left(1 - \frac{y_v}{d_w} \right) \right) = 9 \quad [\text{kN/m}^2]$$

k- Presioni hidrodinamik maksimal

$$p := \sqrt{\langle p_{iw1} + p_{ww} \rangle^2 + p_{cw1}^2 + p_v^2} = 19.068 \quad [\text{kN/m}^2]$$

m- Presioni linear ekuivalent

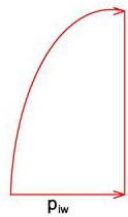
forca prerese per 1 ml ne baze per masen ne moden impulsive eshte

$$q_i := \frac{S_{eImp} \cdot m_i \cdot 9.81}{2 \cdot B_r \cdot 1000} = 28 \quad [\text{kN/m}^2]$$

vlera e presionit ekuivalent linear sipër dhe poshtë eshte

$$a_i := \frac{q_i}{d_{w1}^2} \cdot \langle 4 \cdot d_{w1} - 6 \cdot h_i \rangle = 13.718 \quad [\text{kN/m}^2]$$

$$b_i := \frac{q_i}{d_{w1}^2} \cdot \langle 6 \cdot h_i - 2 \cdot d_{w1} \rangle = 1.96 \quad [\text{kN/m}^2]$$



Real distribution



Linear idealization distribution

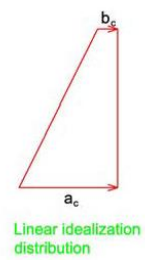
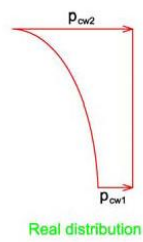
forca prerese per 1 ml ne baze per masen ne moden konvektive

$$q_c := \frac{S_{eCon} \cdot m_c \cdot 9.81}{2 \cdot B_r \cdot 1000} = 13 \quad [\text{kN/m}^2]$$

vlera e presionit ekuivalent linear sipër dhe poshtë eshte

$$a_c := \frac{q_c}{d_{w1}^2} \cdot \langle 4 \cdot d_{w1} - 6 \cdot h_i \rangle = 6.505 \quad [\text{kN/m}^2]$$

$$b_c := \frac{q_c}{d_w^2} \cdot (6 \cdot h_i - 2 \cdot d_w) = 0.929 \quad [\text{kN/m}^2]$$



lartesia e vales gjate sizmes

$$R_{fix} := 2$$

$$d_{wave,max} := S_{eTcom} \cdot R_{fix} \cdot \frac{L_r}{2} = 1.16$$

Rezultatet e analizës statike

Ne menyre qe te kapet bashkeveprimi truell-strukture, vepra eshte analizuar ne software-in Sap2000.

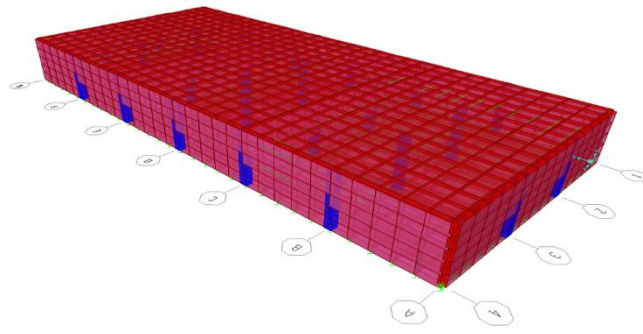


Figure -4 Pamje 3D e modelit te rezervuarit
*Momente perkulese nga veprimi i presionit te mbushjes anesore (Njesite kN*m)*

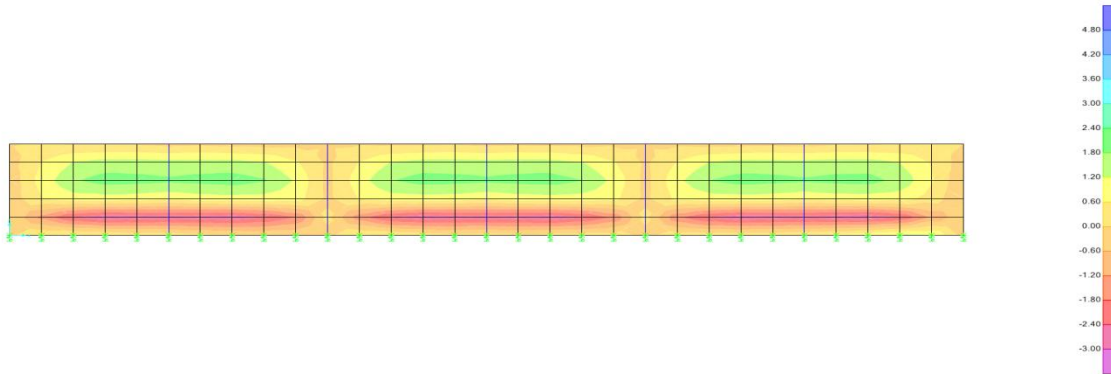


Figure-5 Momentet perkulese M1-1, ne aksin 1-1 (te modelit)

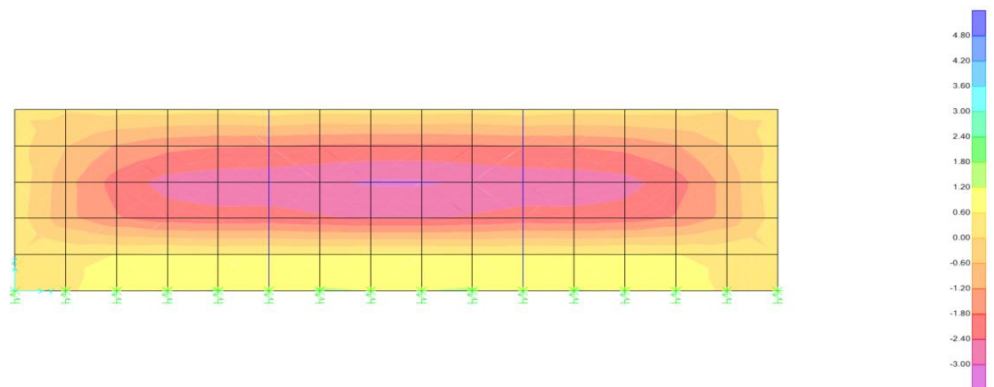


Figure -6 Momentet perkulese M1-1, ne aksin A-A (te modelit)

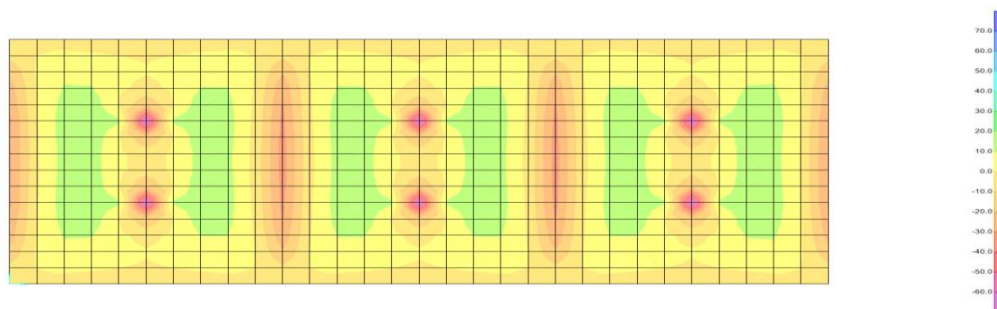


Figure -7 Momentet perkulese M1-1 ne solete

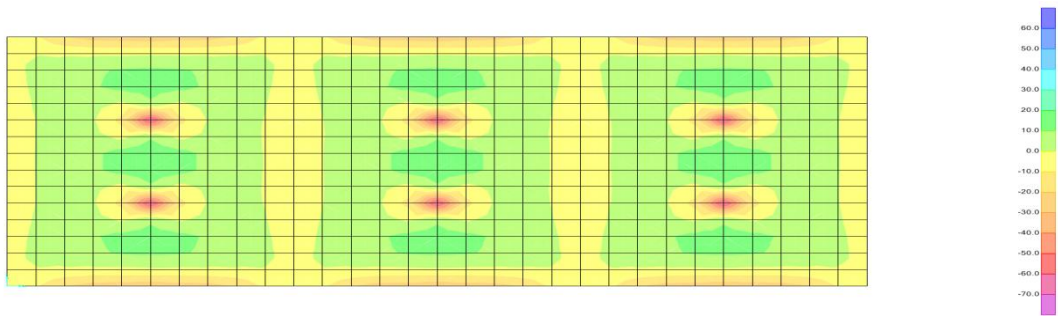


Figure 8 Momentet perkulese M2-2
*Momentet pekulese ne pllaken e themelit (njesite kN*m)*

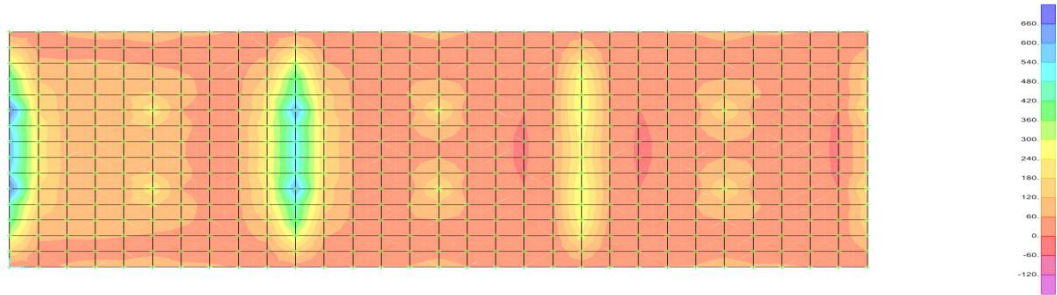


Figure -9 Momentet pekulese M 1-1 sizma xx

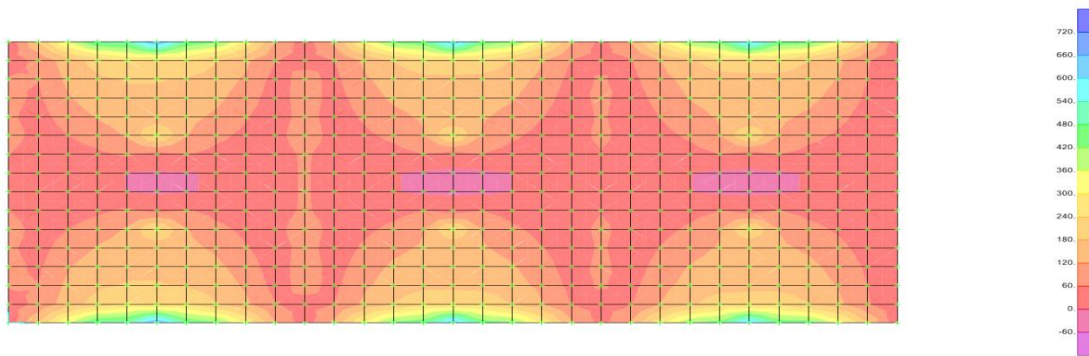


Figure -10 Momentet pekulese M 2-2 sizma y-y

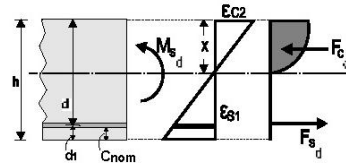
Dimensionimi i seksionit te mureve

Dimenionimi i seksionit te murit anesor per drejtimin gjatesor

1. SLAB-002

Cross section of solid slab in bending
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002,)

h=0.500 m, Med=523.00 kNm
Concrete-Steel class: C30/37-B500C (EC2 §3)
Environmental class : XD1 (EC2 §4.4.1)
Concrete cover : Cnom=40 mm (EC2 §4.4.1)
 $\gamma_c=1.50, \gamma_s=1.15$ (EC2 Table 2.1N)
 $f_{cd}=\alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \times 30 / 1.50 = 17.00$ MPa (EC2 §3.1.6)
 $f_{yd} = f_{yk} / \gamma_s = 500 / 1.15 = 435$ MPa (EC2 §3.2.7)



1.1. Dimensions and loads

Slab thickness h=0.500 m, Bending moment Med=523.00 kNm (ULS), Med=150.00kNm (SLS)
Effective depth of cross section d=h-d1, d1=Cnom+Ø/2=40+16/2=48mm, d=500-48=452mm

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

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

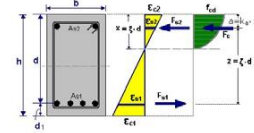
Med(ULS)=523.00 kNm/m
Dimensioning for bending: Allgower, G.-Avak, R. Bemessungstabeln nach Eurocode 2
für Rechteck und Plattenbalkenquerschnitte, In: Beton - und Stahlbetonbau 87 (1992)
 $(\alpha_{cc} = f_{cd} [1 - (1 - \alpha_{cc} / \alpha_{cc2})^2])$, $f_{cd} = 17.00$ MPa, $\alpha_{cc2} = 0.0020$, $\alpha_{cc1} = 0.0035$, $f_{yd} = 435$ MPa
Med=523.00kNm/m, d=452mm, $K_d = 1.98 \times d = 0.20 \times \alpha_{cc2} / \alpha_{cc1} = -3.5 / 13.7$ ks=2.51, **As=29.05cm²/m**
Minimum slab reinforcement, $A_{s1} \geq 0.26 b d \cdot f_{ctm} / f_{yk}$, (As= 6.82cm²/m) (EC2 §9.3.1)
minimum principal reinforcement Ø16/29.5 (6.81cm²/m), secondary Ø16/34.5 (5.83cm²/m)

Slab principal reinforcement Ø16/12.5 (20.01cm²/m) , secondary Ø16/34.5 (5.83cm²/m)

1.2.1. Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

b=1000mm, h=500mm, d=452mm, As1=3092mm²
 $\alpha_{cc2} = -3.50 / \alpha_{cc1}$, $\alpha_{cc1} = 12.67 / \alpha_{cc1}$, $A_{s1} / b \cdot d = 0.00685$ (0.685%)
 $\eta / d = \alpha_{cc2} / (\alpha_{cc2} + \alpha_{cc1}) = 3.50 / (3.50 + 12.67) = 0.216$, $\eta = 97.8$ mm
 $\alpha_r = 0.810$, $k_a = 0.416$, $F_c = \alpha_r \cdot b \cdot \eta \cdot f_{cd} = F_{s1} = 1346.40$ kN, $A_{s1} = F_{s1} / f_{yd} = 3095$ mm²/m
 $z = d - k_a \cdot \eta = [(1 - k_a \cdot \alpha_{cc2} / (\alpha_{cc2} + \alpha_{cc1}))] d$, $z / d = 1 - 0.416 \times 0.216 = 0.910$, $z = 411.3$ mm,
 $K_d^2 = 1 / (0.810 \cdot 0.216 \cdot 0.910 \cdot 17.00) = 0.369$ mm²/N, $K_d = 0.607$
Bending capacity $M_r = b \cdot d^2 \cdot K_d^2 = 0.000001 \times 1000 \times 452^2 / 0.369 = 554.00$ kNm



1.3. Serviceability limit state (SLS)

(EC2 EN1992-1-1:2004, §7)

Med(SLS)=150.00 kNm/m
Final creep coefficient $\varphi(\infty, t_0) = 2.50$ (EC2 §3.1.4, Annex B)
Total shrinkage strain $\epsilon_{cs} = -0.300 / \alpha_{cc}$
 $\gamma_c = 1.00, \gamma_s = 1.00$ (EC2 §2.4.2.4.2)
Modulus of elasticity of concrete $E_{cm} = 32$ GPa, $E_{c,eff} = 32 / (1 + 2.50) = 9.14$ GPa = 9140 MPa (EC2 Eq. 7.20)
Modulus of elasticity of steel $E_s = 200$ GPa = 200000 MPa
Modular ratio $\alpha_e = E_s / E_{c,eff} = 200 / 9.14 = 6.25$, effective $\alpha_e = E_s / E_{c,eff} = 200 / 9.14 = 21.88$
Tension reinforcement: Ø16/65
Reinforcement ratio $\rho = A_{s1} / (b \cdot d) = 3092 / (1000 \times 452) = 0.007$

1.3.1. State I (uncracked section) (SLS)

Bending stiffness of uncracked section, $EI = (200 / 21.88) \times (0.001 \times 10.417) = 95216$ kNm²
 $S = A_{s1} \cdot z_{s1} = (0.001) \times 3092 \times 0.202 = (0.001) \times 0.625$ m³ (EC2 Eq. 7.21)
Curvature due to moment $1/r_M = 150.000 / 95216 = (0.001) \times 1.575$ (1/m)
Curvature due to shrinkage $1/r_{cs} = (0.001 \times 0.30) \times 21.880 \times (0.625 / 10.417) = (0.001) \times 0.394$ (1/m)
Total curvature $1/r = (0.001) \times 1.575 + (0.001) \times 0.394 = (0.001) \times 1.969$ (1/m)
Cracking moment, $M_{cr} = f_{ctm} \cdot (I / y_2) = 2.9 \times (10.417 / 0.250) = 120.83$ kNm

1.3.2. State II (fully cracked section) (SLS)

$\rho = A_s / (b \cdot d) = 0.007$, $n \cdot \alpha_e = 21.88$, $n \cdot \rho = 0.153$, $\xi = 0.498$, $\alpha = 0.421$, $x = \alpha \cdot d = 0.190\text{m}$
Bending stiffness of fully cracked section, $EI = \xi \cdot E_s \cdot A_s \cdot d^2 = 0.498 \times 200 \times 3092 \times 0.452^2 = 62872 \text{ kNm}^2$
 $S = A_s \cdot z_{s1} = (0.001)^2 \times 3092 \times 0.262 = (0.001) \times 0.809 \text{ m}^3$ (EC2 Eq.7.21)
Curvature due to moment $1/r_M = 150.000 / 62872 = (0.001) \times 2.386 \text{ (1/m)}$
Curvature due to shrinkage $1/r_{cs} = (0.001 \times 0.30) \times 21.880 \times (0.809 / 6.879) = (0.001) \times 0.510 \text{ (1/m)}$
Total curvature $1/r = (0.001) \times 2.386 + (0.001) \times 0.510 = (0.001) \times 2.896 \text{ (1/m)}$
 $M_{ed} = 150.00 \text{ kNm}$, $\epsilon_c / \epsilon_{cs} = 0.45 / 0.62$, $x = 190\text{mm}$, $\sigma_s = 125 \text{ N/mm}^2$

1.3.3. Checking deflections by calculation (SLS)

(EN1992-1-1, §7.4.3)

$\zeta = 1 - 0.50 \cdot (M_{cr} / M_{ed})^2 = 1 - 0.50 \times (120.83 / 150.00)^2 = 0.68$ (Eq.7.19)
Final curvature $(1/r) = 0.68 \times (0.001 \times 2.896) + (1 - 0.68) \times (0.001 \times 1.969) = (0.001) \times 2.595 \text{ (1/m)}$ (Eq.7.18)

1.3.4. Minimum reinforcement areas (SLS)

(EC2 EN1992-1-1:2004, §7.3.2)

Minimum reinforcement areas $A_{s,min} = k_c \cdot k_{fct,eff} \cdot A_{ct} / \sigma_s$ (EC2 Eq.7.1)
 $b = 1.000\text{m}$, $b_{eff} = 1.000\text{m}$, $h = 0.500\text{m}$, $d = 0.452\text{m}$, $x = 0.190\text{m}$, $\phi = 16\text{mm}$
 $N_{ed} = 0.00\text{kN}$, $\sigma_c = (N_{ed} / b h) = 0.0\text{N/mm}^2$, $\sigma_s = 125\text{N/mm}^2$
 $A_{ct} = (h - x) \cdot b = (500 - 190) \times 1000 = 309661 \text{ mm}^2$
 $\max(h, b) = 1\text{m}$, $f_{ctm} = 2.90\text{N/mm}^2$, $A_{c,eff} = 309661\text{mm}^2$, $k = 0.86$, $k_c = 0.40$, $k_1 = 1.50$
Minimum reinforcement, $A_{s,min} = 0.40 \times 0.86 \times 2.90 \times 309661 / 125 = 2474\text{mm}^2/\text{m}$

1.3.5. Calculation of crack width (SLS)

(EC2 EN1992-1-1:2004, §7.3.3)

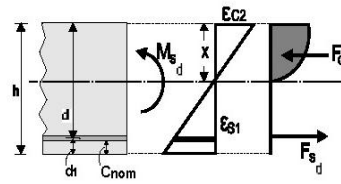
$w_k = s_r, \max \cdot (\epsilon_{sm} - \epsilon_{cm})$ (EC2 Eq.7.8)
 $\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot (f_{ct,eff} / \rho_{eff}) (1 + \alpha_e \cdot \rho_{eff})] / E_s \geq 0.6 \sigma_s / E_s$ (EC2 Eq.7.9)
 $\sigma_s = 125\text{N/mm}^2$, short term loading: $\alpha_e = 6.25$, $k_t = 0.6$, long term loading: $\alpha_e = 21.88$, $k_t = 0.4$
 $A_{c,eff} = 0.333 (h - x) b = 0.333 \times (500 - 190) \times 1000 = 103117 \text{ mm}^2$ (§7.3.2.3)
 $\rho_{eff} = A_s / A_{c,eff} = 3092 / 103117 = 0.030$
 $\epsilon_{sm} - \epsilon_{cm} = [125 - 0.4 \times (2.9 / 0.030) (1 + 21.88 \times 0.030)] / 200 = 0.30 \text{ o/o} > 0.6 \times 125 / 200 = 0.37 \text{ o/o}$
 $s_r, \max = k_3 \cdot C_{nom} + k_1 \cdot k_2 \cdot k_4 \cdot \phi / \rho_{eff}$ (EC2 Eq.7.11)
 $\phi = 16\text{mm}$, $k_1 = 0.8$, $k_2 = (e_1 + e_2) / 2e_1 = 0.5$, $k_3 = 3.4$, $k_4 = 0.425$
 $s_r, \max = 3.4 \times 40.00 + 0.8 \times 0.5 \times 0.425 \times 16 / 0.030 = 226.71 \text{ mm}$
 $w_k = s_r, \max \cdot (\epsilon_{sm} - \epsilon_{cm}) = 226.71 \times 0.001 \times 0.37 = 0.08 \text{ mm}$

Dimenionimi i seksionit te murit anesor per drejtimin gjatesor

1. SLAB-002

Cross section of solid slab in bending
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002,)

h=0.500 m, Med=412.00 kNm
Concrete-Steel class: C30/37-B500C (EC2 §3)
Environmental class : XD1 (EC2 §4.4.1)
Concrete cover : Cnom=40 mm (EC2 §4.4.1)
 $\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N)
 $f_{cd}=w_{cc} \cdot f_{ck} / \gamma_c = 0.85 \times 30 / 1.50 = 17.00$ MPa (EC2 §3.1.6)
 $f_{yd}=f_{yk} / \gamma_s = 500 / 1.15 = 435$ MPa (EC2 §3.2.7)



1.1. Dimensions and loads

Slab thickness h=0.500 m, Bending moment Med=412.00 kNm (ULS), Med=288.40kNm (SLS)
Effective depth of cross section d=h-d1, d1=Cnom+ $\phi/2=40+16/2=48$ mm, d=500-48=452mm

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

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

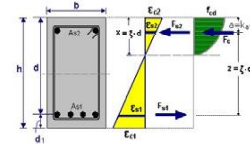
Med(ULS)=412.00 kNm/m
Dimensioning for bending: Allgower, G.-Avak, R. Bemessungstabeln nach Eurocode 2
für Rechteck und Plattenbalkenquerschnitte, In: Beton - und Stahlbetonbau 87 (1992)
($\alpha_c = f_{cd} [1 - (1 - \alpha_{ct}/\alpha_{cs})^2]$, $f_{cd} = 17.00$ MPa, $\alpha_{ct} = 0.0020$, $\alpha_{cs} = 0.0035$, $f_{yd} = 435$ MPa)
Med=412.00kNm/m, d=452mm, Kd= 2.23 $x/d=0.16$ $\alpha_{ct}/\alpha_{cs} = -3.5/18.8$ ks=2.46, As=22.42cm²/m
Minimum slab reinforcement, As>=0.26bd · fctm/fyk, (As= 6.82cm²/m) (EC2 §9.3.1)
minimum principal reinforcement $\phi 16/29.5$ (6.81cm²/m) , secondary $\phi 16/44.5$ (4.52cm²/m)

Slab principal reinforcement $\phi 16/13.0$ (22.33cm²/m) , secondary $\phi 16/44.5$ (4.52cm²/m)

1.2.1. Ultimate moment capacity of cross section

(EC2 EN1992-1-1:2004, §6.1)

b=1000mm, h=500mm, d=452mm, As1=2233mm²
 $\alpha_{ct} = -3.50/\alpha_{cs}$, $\alpha_{cs} = 18.89/\alpha_{cs}$, As1/b · d=0.00495 (0.495%)
 $x/d = \alpha_{ct} / (\alpha_{ct} + \alpha_{cs}) = 3.50 / (3.50 + 18.89) = 0.156$, x=70.7mm
ar=0.810, ka=0.416, Fc=ar · b · x · fcd=Fs1=972.37kN, As1=Fs1/fyd=2235mm²/m
z=d-ka · x=(1-ka · $\alpha_{ct} / (\alpha_{ct} + \alpha_{cs})$)d, z/d=1.0-0.416 · 0.156=0.935, z=422.6mm,
Kd²=1/(0.810 · 0.156 · 0.935 · 17.00)=0.497 mm²/N, Kd=0.705
Bending capacity Mr=b · d²/Kd²=0.00001x1000x452²/0.497=411.00kNm



1.3. Serviceability limit state (SLS)

(EC2 EN1992-1-1:2004, §7)

Med(SLS)=288.40 kNm/m
Final creep coefficient $\phi(\infty, t_0) = 2.50$ (EC2 §3.1.4, Annex B)
Total shrinkage strain $\epsilon_{cs} = -0.30/\alpha_{cs}$
 $\gamma_c = 1.00$, $\gamma_s = 1.00$ (EC2 §2.4.2.4.2)
Modulus of elasticity of concrete Ecm=32GPa, Ec,eff=32/(1+2.50)=9.14GPa=9140MPa (EC2 Eq.7.20)
Modulus of elasticity of steel Es=200GPa=200000MPa
Modular ratio $\alpha_e = Es/Ec = 200/9.14 = 21.88$, effective $\alpha_e = Es/Ec,eff = 200/9.14 = 21.88$
Tension reinforcement: $\phi 16/90$
Reinforcement ratio $\rho = As1 / (b \cdot d) = 2233 / (1000 \times 452) = 0.005$

1.3.1. State I (uncracked section) (SLS)

Bending stiffness of uncracked section, EI=(200/21.88)x(0.001x10.417)=95216 kNm²
S=As · zs1=(0.001) · 2233 · 0.202=(0.001) · 0.451 m³ (EC2 Eq.7.21)
Curvature due to moment 1/rM=288.400/95216=(0.001) · 3.029 (1/m)
Curvature due to shrinkage 1/rCS=(0.001x0.30)x21.880x(0.451/10.417)=(0.001) · 0.284 (1/m)
Total curvature 1/r=(0.001) · 3.029+(0.001) · 0.284=(0.001) · 3.313 (1/m)
Cracking moment, Mcr=fctm · (I/y2)=2.9x(10.417/0.250)=120.83 kNm

1.3.2. State II (fully cracked section) (SLS)

$\rho = A_s / (b \cdot d) = 0.005$, $n \cdot \alpha_e = 21.88$, $n \cdot \rho = 0.109$, $\xi = 0.551$, $\alpha = 0.371$, $x = \alpha \cdot d = 0.168 \text{ m}$
Bending stiffness of fully cracked section, $EI = \xi \cdot E_s \cdot A_s \cdot d^2 = 0.551 \times 200 \times 2233 \times 0.452^2 = 50296 \text{ kNm}^2$
 $S = A_s \cdot z_{s1} = (0.001)^2 \times 2233 \times 0.284 = (0.001) \times 0.635 \text{ m}^3$ (EC2 Eq.7.21)
Curvature due to moment $1/r_M = 288.400 / 50296 = (0.001) \times 5.734 \text{ (1/m)}$
Curvature due to shrinkage $1/r_{cs} = (0.001 \times 0.30) \times 21.880 \times (0.635 / 5.503) = (0.001) \times 0.400 \text{ (1/m)}$
Total curvature $1/r = (0.001) \times 5.734 + (0.001) \times 0.400 = (0.001) \times 6.134 \text{ (1/m)}$
 $M_{ed} = 288.40 \text{ kNm}$, $\epsilon_c / \epsilon_{cs} = 0.96 / 1.63$, $x = 168 \text{ mm}$, $\sigma_s = 326 \text{ N/mm}^2$

1.3.3. Checking deflections by calculation (SLS)

(EN1992-1-1, §7.4.3)

$\zeta = 1 - 0.50 \cdot (M_{cr} / M_{ed})^2 = 1 - 0.50 \times (120.83 / 288.40)^2 = 0.91$ (Eq. 7.19)
Final curvature $(1/r) = 0.91 \times (0.001 \times 6.134) + (1 - 0.91) \times (0.001 \times 3.313) = (0.001) \times 5.887 \text{ (1/m)}$ (Eq. 7.18)

1.3.4. 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)
 $b = 1.000 \text{ m}$, $b_{eff} = 1.000 \text{ m}$, $h = 0.500 \text{ m}$, $d = 0.452 \text{ m}$, $x = 0.168 \text{ m}$, $\emptyset = 16 \text{ mm}$
 $N_{ed} = 0.00 \text{ kN}$, $\sigma_c = (N_{ed} / bh) = 0.0 \text{ N/mm}^2$, $\sigma_s = 326 \text{ N/mm}^2$
 $A_{ct} = (h - x) \cdot b = (500 - 168) \times 1000 = 332315 \text{ mm}^2$
 $\max(h, b_1) = 1 \text{ m}$, $f_{ctm} = 2.90 \text{ N/mm}^2$, $A_{c,eff} = 332315 \text{ mm}^2$, $k = 0.86$, $k_c = 0.40$, $k_1 = 1.50$
Minimum reinforcement, $A_{s,min} = 0.40 \times 0.86 \times 2.90 \times 332315 / 326 = 1017 \text{ mm}^2 / \text{m}$

1.3.5. Calculation of crack width (SLS)

(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)
 $\sigma_s = 326 \text{ N/mm}^2$, short term loading: $\alpha_e = 6.25$, $k_t = 0.6$, long term loading: $\alpha_e = 21.88$, $k_t = 0.4$
 $A_{c,eff} = 0.333 (h - x) b = 0.333 \times (500 - 168) \times 1000 = 110661 \text{ mm}^2$ (§7.3.2.3)
 $\rho_{eff} = A_s / A_{c,eff} = 2233 / 110661 = 0.020$
 $\epsilon_{sm} - \epsilon_{cm} = [326 - 0.4 \times (2.9 / 0.020) (1 + 21.88 \times 0.020)] / 200 = 1.22 \text{ o/o} \geq 0.6 \times 326 / 200 = 0.98 \text{ o/o}$
 $s_r, \max = k_3 \cdot C_{nom} + k_1 \cdot k_2 \cdot k_4 \cdot \emptyset / \rho_{eff}$ (EC2 Eq.7.11)
 $\emptyset = 16 \text{ mm}$, $k_1 = 0.8$, $k_2 = (e_1 + e_2) / 2e_1 = 0.5$, $k_3 = 3.4$, $k_4 = 0.425$
 $s_r, \max = 3.4 \times 40.00 + 0.8 \times 0.5 \times 0.425 \times 16 / 0.020 = 270.80 \text{ mm}$
 $w_k = s_r, \max \cdot (\epsilon_{sm} - \epsilon_{cm}) = 270.80 \times 0.001 \times 1.22 = 0.33 \text{ mm}$

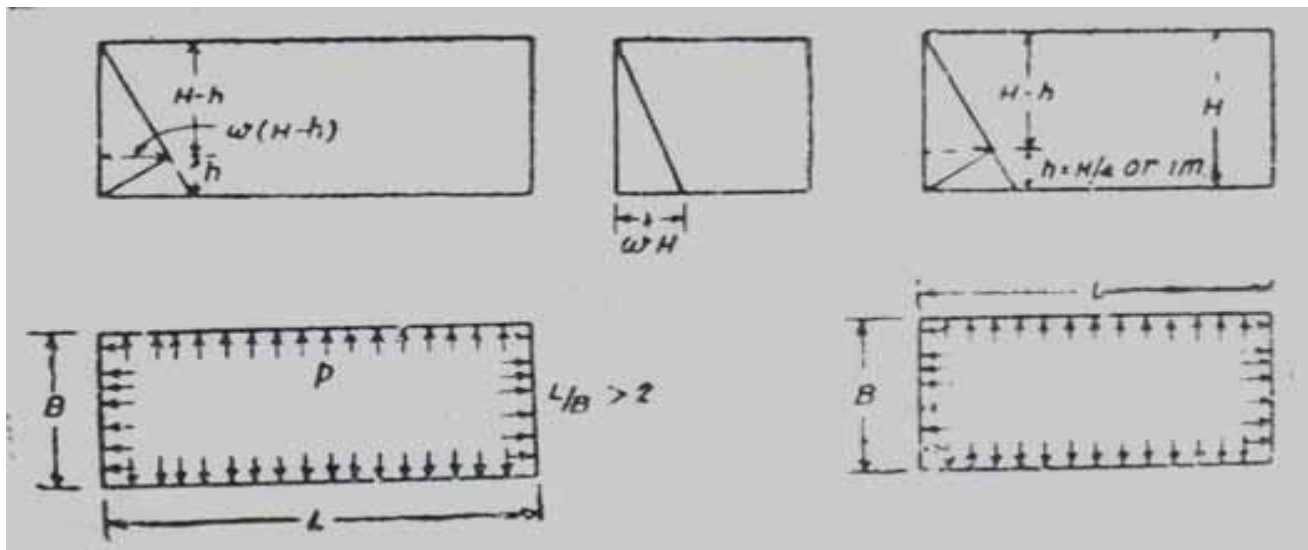
1- Llogaritjet Konstruktive te Depos se Re 200 m3

Zgjidhja e depos eshte konceptuar drejtekendore si per nga ana ndertimore , e cila eshte me praktike , ashtu dhe per nga ana ekonomike .

Struktura e depos do te konceptohet me sistem tra-kollone te arme si dhe diafragma b/a me C20/25 dhe hekur armature çelik S-500 (Kufiri i rrjedhshmerise 500kg/cm, koeficient sigurie $\gamma_s = 1.15$ dhe zgjatim relativ $\geq 12-18\%$) . Soleta eshte projektuar monolite $h = 16 \text{ cm}$. Zgjidhja e struktures se soletes eshte modeluar me trare te fshehte me (rripa) ne hapesire ne te dyja drejtimet e kollones kerpudhe e vendosur ne mes te hapesires se soletes .

Skema statike e llogaritjes se struktures parashikon te gjitha nyjet te inkastruara .

Llogaritjet e mureve te depove behen ne baze te te raportit gjatesi me gjeresi .Ne rastin konkrete (gjatesia e mureve brenda per brenda) raporti $L/B = 8/7 < 2$. Ne kete rast muret e rezervuarit do te llogariten si pllake e vazhdueshme me presion ne faqet e tyre qe varion nga 0 nga maja ne vlere maxime ne lartesine $H/4$.



Muret do te llogariten per rastet kur kemi presion te ujit ne faqet e brendshme te murit dhe presionin e dheut nga jashte rezervuarit .

8.1. Themelet

Themeli i depos eshte projektuar pllake me trashesi 30 cm me beton C 20/25 dhe armature çelik S-500.

Thellesia e zhytjes se pllakes se themelit (pa perfshire shtresen e betonit te varfer $t=10$ cm dhe ate te zhavorrit $t=15$ cm) do te jete minimalisht 2.50 m nga fundi tabanit te depos (kjo do te shikohet ne baze te sistemimit ne realitet). Pllakes se themelit eshte menduar ti realizohet nje dalje perimetrare me gjatesi $b=40$ cm per te krijuar nje shtangesi me te mire te rezervuarit . Mbi pllaken e themelit realizohen mure mbajtese b/a me trashesi $b=30$ cm te nevojshme per te perballuar presionin e ujit nga brenda objektit dhe presionin e dheut nga jashte faqeve te depos . Tabani i themelit do te perforcohet me nje shtrese betoni te varfer $t=10$ cm dhe shtrese zhavorri $t=15$ cm .

8.2. Kollonat

Kollona eshte e konceptuar si kollona me kapitel e cila eshte vendosur ne mes te hapesires se soletes se mbulimit me baze kapiteli $b=160$ cm . Kjo per arsye te nje mbshtetje me te mire te soletes . Kollona eshte e llogaritur me permasa 30x30 dhe kapitel ne pjesen e sipërme . Betoni i perdorur eshte C20/25 armature çelik S-500 .

8.3. Soletat

Zgjidhja e soletes eshte e konceptuar pa trare por eshte zgjedhur skema e armimit me rripa , rripa mbi kollonen e mesit dhe rripa mbi hapesire e cila ne thelb mund te konsiderohen si trare te fshehur ne solete . Ne periferi te saj soleta mbeshtet ne muret beton arme . Soleta eshte projektuar me trashesi $H=16$ cm me beton C 16/20 dhe armature Çeliku S-500 .

8.4.Muret b/a

Muret b/a janë konceptuar si pllaka të vazhdueshme nën efektin e presioneve të ujit nga brenda faqeve të murit dhe nga jashtë mureve nga presioni i dheut . Qoshet e mureve , takimet e mureve me pllaken dhe takimet e mureve me soleten do të realizohen me kënd .Muret do të realizohen me trashësi $b=30$ cm dhe armature çeliku S-500.

Normativat e Referimit.

KODET DHE STANDARTET

Për projektimin e këtij objekti, janë përdorur kodet dhe standartet e mëposhtëme:

Kushtet teknike Shqipëtare – KTP:

Në aplikimin e Rregullave Teknike referohen dhe respektohen Standardet dhe Rregullat Teknike të Projektimit në fuqi në vendin tonë.

Kodet europiane (Eurokodet)

- EN 1990 – Eurokodi 0: Bazat e projektimit të konstruksioneve të ndërtimit.
- EN 1991 – Eurokodi 1: Veprimet mbi strukturat
- EN 1992 – Eurokodi 2: Projektimi i konstruksioneve betonarme
- EN 1996 – Eurokodi 6: Projektimi i ndërtesave me mure mbajtëse
- EN 1997 – Eurokodi 7: Projektimi gjeoteknik
- EN 1998 – Eurokodi 8: Projektimi i strukturave rezistente ndaj tërmeteve

Shënim:

Nisur nga periudha e gjatë kohore në të cilën janë formuluar standartet shqiptare, d.m.th. ato janë akoma të pa rinovuara, struktura do të llogaritet kryesisht referuar normave europiane EC, dhe vetëm ku do të jetë e nevojshme apo e domosdoshme do të përdoren standartet KTP

Standarde dhe Rregulla Teknike të referuara

Përveç referencave të përgjithshme, në kushtet teknike citohen, në vendet përkatëse, edhe këto referenca të tjera:

- ISO 1000: Njësitë e Sistemit Nderkombëtar (S.I.) dhe rekomandimet për përdorimin e shumëfishave, nënfishave të tyre, si dhe disa njësi të tjera;
- ISO 8930: Parimet e përgjithshme mbi sigurinë (besueshmërinë–riliabilitetin) e konstruksioneve – Lista e termave ekuivalente;
- EN 1090–1: Zbatimi i konstruksioneve metalike–Rregulla të përgjithshme dhe rregulla për ndërtesat;
- EN 10025: Produkte hekuri (çeliku) të paleguar (jo aliazh), të përpunuar në të nxehtë–Kushte teknike të dorezimit (furnizimit)
- EN 1337–1: Mbështetjet strukturore – Kërkesa të përgjithshme

– EN 1998–4: Parimet dhe rregullat e aplikimit për projektimin sizmik sillosave ,rezervuareve dhe tubacioneve

Njesitë matëse

Njësitë S.I. duhet të përdoren në përputhje me Rregullat Teknike në fuqi (sipas Standarteve ISO 1000). Për llogaritjet rekomandohet të përdoren njësitë që vijojnë:

- forcat dhe ngarkesat: kN, kN/m, kN/m^2 ;
- masat njësore (densitetet)dhe masat $\text{kg/m}^3, \text{t/m}^3, \text{kg, t}$
- peshat njësore (peshat specifike): kN/m^3 ;
- nderjet dhe rezistencat: N/mm^2 (= MN/m^2 ose MPa), kN/m^2 (=kPa);
- momentet (përkulje etj): kN
- shpejtimet (akseleracionet): $\text{m/s}^2, \text{g}$ (= $9,81 \text{ m/s}^2$).

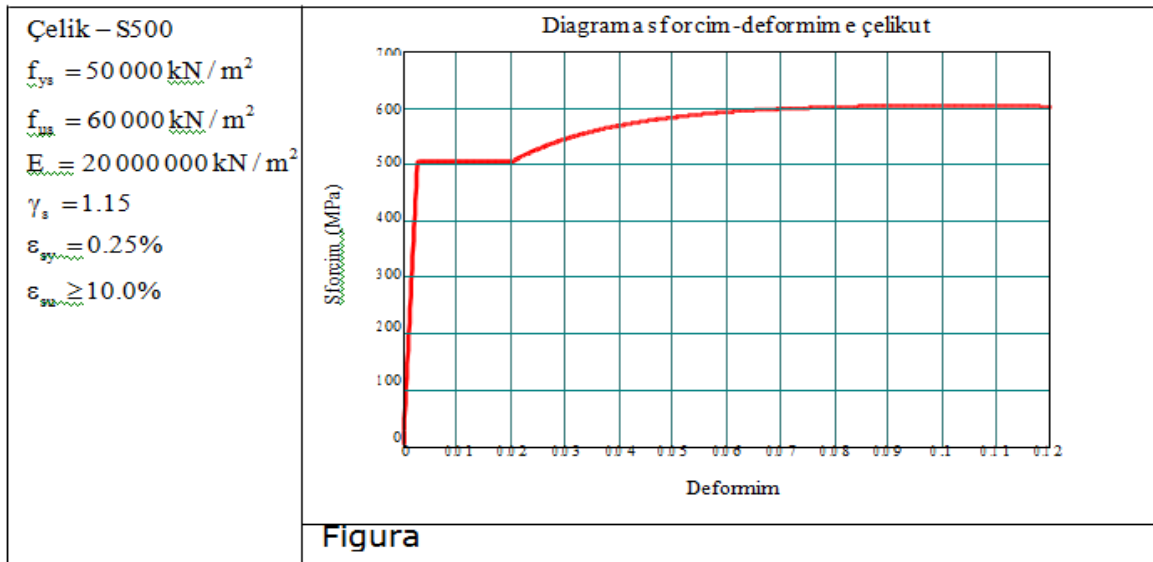
Vetitë fiziko-mekanike të materialeve

Materialet që do të përdoren për projektimin e strukturës (betoni dhe çeliku) duhet të plotësojnë të gjitha kriteret e parashikuara në Eurokodin

2 si dhe në Eurokodin 8.

Çeliku

Çeliku që do të përdoret duhet të gëzojë veti të mira si në rezistencë ashtu edhe në deformueshmëri (duktilitet). Në elementët parësorë sizmike, për armaturën e hekurit duhet të përdoret çelik i klasës B ose C, sipas tabelës C1 në Aneksin Normativ C të Eurokudit 2, EN 1992. Më poshtë jepen karakteristikat dhe diagrama e çelikut të përdorur në strukturën tonë. Referuar eurokodeve shufrat e çelikut duhet të jenë patjetër të vjaskuara (çelik periodik)



Betoni

Bazuar te EC8, në strukturat me duktilitet mesatar DCM, nuk mund të përdoret, për elementet përsore sizmike beton me klase më të vogël se C16/20.

1. Materjalet :

a. Beton	C20/25
β. Çelik	S-500
γ. Stafa	S-500

2. Koeficientet e sigurisë së materialeve:

a. Betoni	$\gamma_c = 1,50$
β. Çeliku	$\gamma_s = 1,15$

3. Ngarkesat e perhershme:

Pesha vetjake e betonit C20/25:	24,00 kN/m ³
Pesha vetjake e ujit:	10,00 kN/m ³
Pesha vetjake e dheut:	20,00 kN/m ³

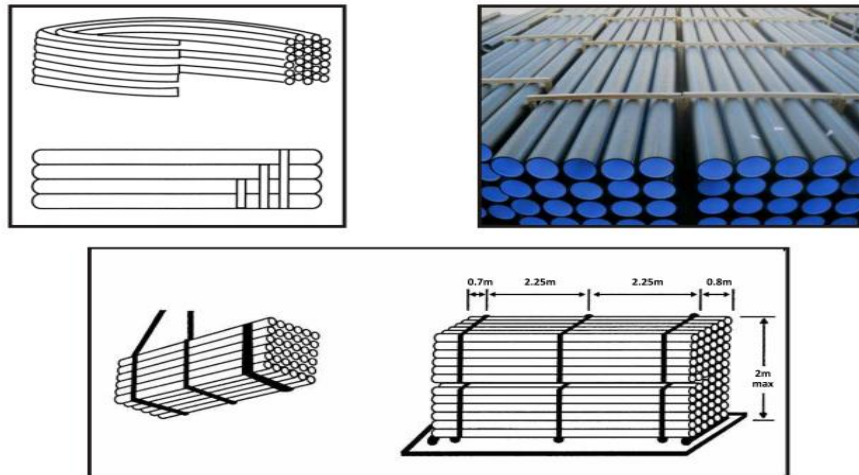
4. Ngarkesat e perkohshme:

Për hapsirat ujembjtesë:	5,80 kN/m ²
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9. KANALET E VENDOSJES SE TUBACIONEVE.

TUBACIONET

Per ndertimin e ujesjellesit per shtimin e sasise se furnizimit me uje per qytetin e Maliqit do te perdren tubo PE100 PN16 me diameter nga \varnothing 100 mm per ndertimin e linjave te jashtme deri \varnothing 50 mm per linjat e furnizimit me uje te grup shtepive.



Karakteristikat kryesore te tubove jane:

Sistem Cilësie i Certifikuar– UNI EN ISO 9001:14001.

Karakteristika fizike dhe Mekanike si në vijim:

Elasticitet/aftësi ripërtërirëse (Charpy) -30°C: 40 kJ/m²

Elasticitet/aftësi ripërtërirëse 23°C: 25 kJ/m²

Elasticitet/aftësi ripërtërirëse -30°C: 2.5 kJ/m²

Elasticitet/aftësi ripërtërirëse (Izod) -30°C: 28 kJ/m²

Elasticitet/aftësi ripërtërirëse Gërvishtje 23°C: 23 kJ/m²

Elasticitet/aftësi ripërtërirëse Gërvishtje -30°C: 2.5 kJ/m²

Testi produktivitetit: 27 N/mm²

Elasticiteti produktivitetit: 11%

Zgjatime thyerëse: >800%

Module tërheqje E: 900 N/mm²

Produktivitet elasticiteti në tension tangent: 450 N/mm²

Shtypje përthyerëse 3.5%: 24 N/mm²

Test ashpërsie Brinell: 49 N/mm²

Stabilitet nxehe Dimensionale °C: 75 °C

Rezistencë sipërfaqeje: > 1013 Ω

Densiteti i massës: > 1016 Ωcm

Konstant relative dielektrik: 2.3

Ngurtësi Dielektrike: 75 kV/mm

Konduktivitet Termal në 20°C: 0.22 W/mK

Faktor Termal ekspansioni: 0.15 mm/m°C

Ngrohje Specifike : 2.0 Kj/KgK

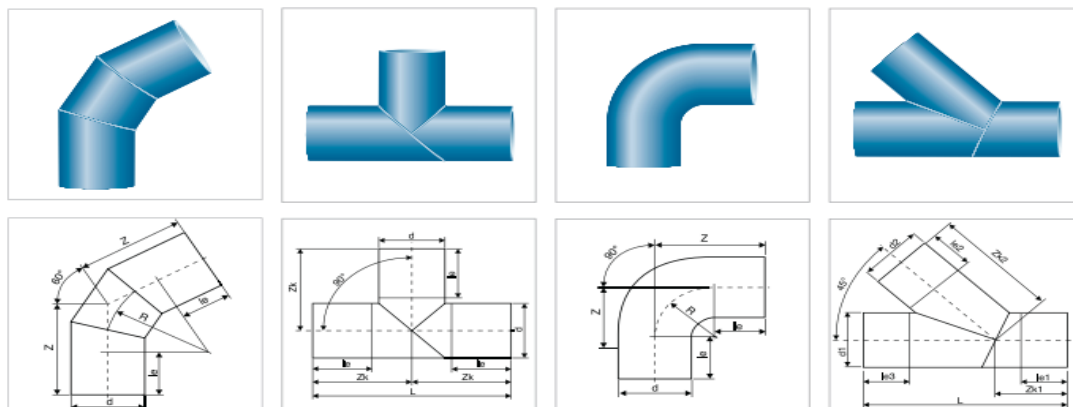
Kritere të përgjithshme për skicimin e tubacioneve :

Dizajnimi, instalimi, punëtorja, inspektimi dhe testimi i rrjeti i tubacioneve do të kryhet në përputhje me kodet e dizajnimit dhe specifikimet të miratuara .Të gjitha tubacionet nuk duhet të jenë me vrima , të pastra dhe të lëmuara kudo, nga ana tregtare të drejta dhe të kalibruara, pa korrozion dhe defekte të tjera prodhimi në sipërfaqe .

Prodhimi i tyre behet me rrota 100 ml per diametra 63 – 90 mm, 50 ml per diametra 110 – 125 mm dhe 12 ml per diametra mbi 125 mm. Bashkimet do te behen me elektrofuzion ose buttfuzion.

Tubat vendosen mbi nje shtrese rere 10 cm dhe mbulohen per te ruajtur nga goditjet me rere deri 10 cm mbi pjesen e sipërme te diametrit te tubit..

Armaturat (saracineska , valvola etj. montohen me flanxha metalike te cilat lidhen me qafa speciale me krah dhe krah te shkurter.



Te gjitha keto bashkime behen jashte kanalit dhe mbasi garantohet cilesia shtrihet me kujdes pa u mbuluar.

Mbulimi behet mbas kryerjes se proves hidraulike.

Ne vendet e kryqezimit jane parashikuar pusetat betoni (shih projektin) me kapak gize.

Pusetat jane parashikuar te kene dimensione te mjaftueshme per te manovruar gjate avarive, ose zevendesimit te pjeseve te difektuara .

Gjithashtu kujdes duhet treguar ne zonat ujembajtese. Ne pusetat e shkarkimit vendoset e tub per largimin e ujrave duke e derdhur ate ne vendkullimi te sigurt.

Para hapjes se kanalit do te verifikohen te gjitha pikat e kontaktit per te shmangur avarite e mundeshme sidomos kabllot elektrike, telefonike etj

Te respektohen distancat midis tyre kuotat e kryqezimeve etj.

Projektues
“ERALD-G” sh.p.k.

Ing. Gezim ISLAMI