

**SANACIJA KLIZIŠTA
"TOPLIČKA ULICA"
OPĆINA KRAPINSKE TOPLICE**

GEOMEHANIČKO MIŠLJENJE SA PRIJEDLOGOM TEHNIČKOG RIJEŠENJA

Naručitelj : OPĆINA KRAPINSKE TOPLICE, A.MIHANOVIĆA 3,KRAPINSKE TOPLICE
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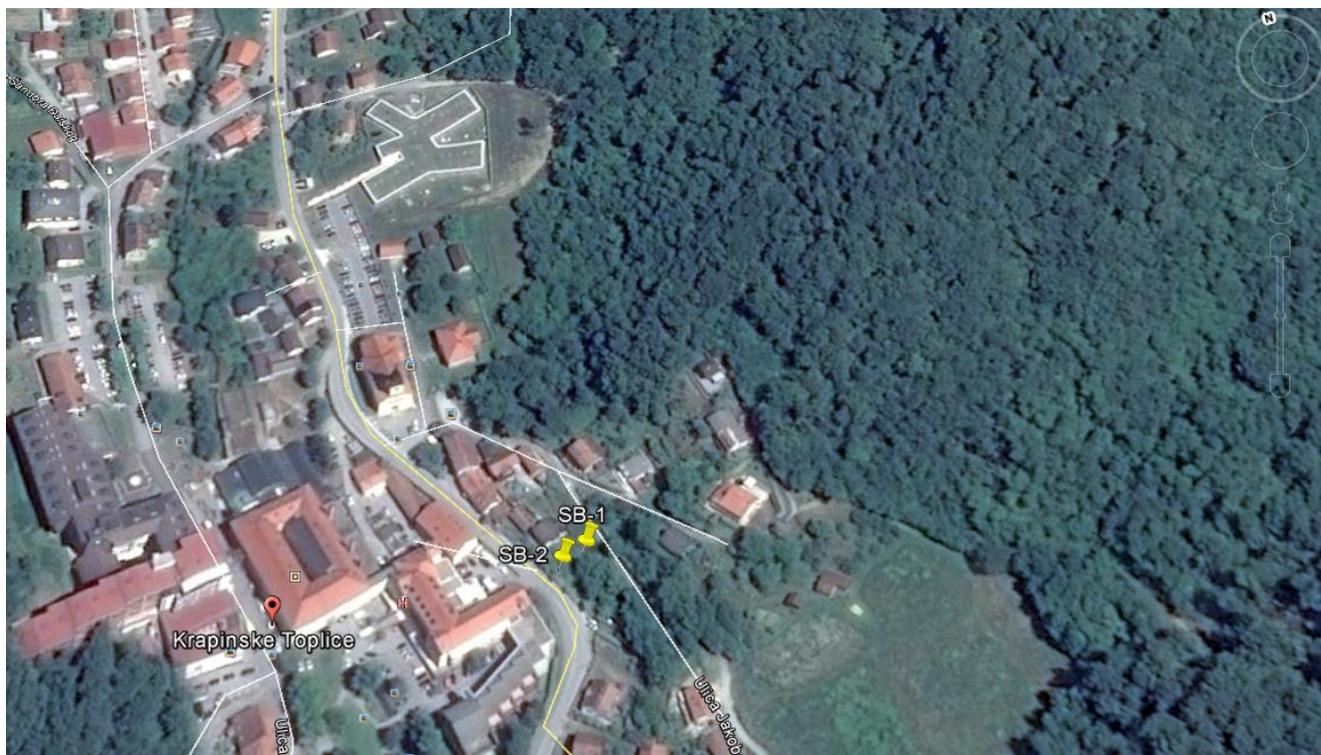
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1. UVOD

Na traženje Naručitelja, općine Krapinske Toplice, na lokaciji Topličke ulice, na dijelu ceste ranije poznate po zahvaćenju nestabilnosti terena, registrirano je klizište koje zahvaća šire područje pokosa iznad prometnice, te manji pristupni odvojak prema stambenom objektu.

Klizanjem je zahvaćena postojeći pokos u zoni zasjeka Topličke ulice, u neposrednoj blizini stambenog objekta. Rezultati klizanja vidljivi su u pomaku kliznog tijela uslijed rotacijsko translacijskog klizanja prema navedenoj ulici, sa erozijom veće količine kliznog tijela prema osi prometnice. Također, u zoni pristupnog puta stambenom objektu u neposrednoj blizini, klizanjem je onemogućen predviđeni prilaz objektu, uslijed pomicanja klizne mase koja je zapriječila i onemogućila sigurno korištenje navedenog puta.

Pregledom same lokacije vidljivo je da je područje morfološki zahtjevnog oblika, školjkaste forme koja ima tendenciju prikupljanja veće količine oborina na predmetnoj površini. Obzirom da navedeni pokos nije pridržan nikakvom potpornom konstrukcijom, a sastav tla je podložan saturaciji prikupljenom vodom isti je lako pokazao znakove popuštanja uslijed vešestrukog natapanja u hidrološki nepovoljnim mjesecima. Dio prometnice uslijed daljnjeg klizanja potpuno bi onemogućio promet ovim krajem, koji bi tada ostao izvan funkcije. Predlaže se što hitnije saniranje navedenog predjela pokosa kako bi se spriječilo napredovanje klizanja, te osiguralo korištenje pristupnog puta te nesmetano odvijanje prometa, te stabiliziralo uže područje zahvata.



1.1. prikaz lokacije klizišta na satelitskoj snimci

U listopadu, 2016. godine, obavljena je inženjerska geotehnička prospekcija s geomehaničkim istražnim radovima, te strojnim bušenjem dvije sondažne bušotine za utvrđivanje dubine čvrste laporovite podloge u predjelu deformiranog kliznog tijela, te nožice klizišta. Sondažna bušotina SB-1, bušena je na poziciji kliznog tijela u predjelu budućeg potpornog zida 2, dok je sondažna bušotina SB-2 bušena na poziciji budućeg potpornog zida 1 koji će štititi prometnicu daljnje erozije pokosa. Istražni radovi imaju zadatak da se ustanove uzroci nestabilnosti padine, te stabilizira teren na uznemirenom pokosu padine.

Inženjersko geomehničkim istražnim radovima, terenskim zapažanjima i mjerenjima, te rezultatima laboratorijskih ispitivanja izradit će se prijedlog osiguranja pokosa od daljnjeg klizanja. Prijedlog će obuhvaćati izradu potporno zaštitne konstrukcije u dvije pozicije na predjelu pokosa zahvaćenog klizanjem, te sanaciju kolničke konstrukcije pristupnom putu stambenom objektu.

2. INŽENJERSKO GEOTEHNIČKI PRIKAZ

Terenskim istražnim radovima, te vizualnim pregledom na predmetnom klizištu vidljivi su jasni tragovi koji ukazuju na klizanje terena koje se u trenutku pregleda nalazi u fazi prividne labilne ravnoteže. Obzirom da proces nije dovršen, moguće je i aktivnije klizanje već destabiliziranog terena pri čemu bi zahvaćeni klizni dio mogao zatvoriti prometovanje ove bitne prometnice, te dodatno ugroziti stambeni objekt u neposrednoj blizini.

Navedeni pokos proteže se morfološki razvedenim terenom, između ulica Topličke te odvojka ulice Jakoba Badie. Ulice su paralelne te se protežu u smjeru sjever jug, dok se sama padina strmog pokosa proteže okomito tj, u smjeru istok zapad. Klizanje strmog pokosa uvjetovano je najprije njegovom geometrijom, te sastavom tla, te ne osiguranjem nožice padine potrebnim potpornim konstrukcijama. Tokom ranijih godina na predjelu nožice padine, izvedeno je usjecanje pristupnog puta čiji zasjeci nisu osigurani adekvatnim potpornim konstrukcijama. Tokom proteklih godina, zbog povećanih oborina na predjelu kliznog tijela formirala se manja depresija koja je u periodu najvećih oborina zadržavala prikupljenu oborinsku vodu te koncentrirano vlažila navedeno područje. Pregledom okolnog terena je vidljiva nestabilnost nizbriježnog pokosa na odvodjku ulice J.Badie prema lokaciji predmetnog klizišta, iako u ovom trenutku ista lokacija nije predmet sanacije. Dimenzije zahvata ovog klizišta šireg su karaktera, dok će se za sanacijom obuhvatiti pozicija postojećeg kliznog tijela te pokosa u neposrednom kontaktu sa Topličkom ulicom.

Dužina zahvata ove sanacije je 28 m', i obuhvaća dio pokosa uz prometnicu visine oko 2.3 m koji bi daljnjim napredovanjem mogao potpuno staviti izvan funkcije ovu prometnicu te uzrokovati podsjećanje nožice većeg razmjera. Uz navedenu poziciju, osigurati će se i pozicija pristupnog puta izvođenjem potporne konstrukcije kako bi se prihvatilo opterećenje kliznog tijela te stabilizirati utjecaj strme padine u dužini 18 m'. Cilj sanacije je stabilizacija klizne zone, te lokalno rješenje prometovanja do stambenog objekta te kontrolirana odvodnja oborinskih voda u zoni zahvata.

Istražni radovi provedeni strojnim bušenjem sondažnih bušotina pokazali su uvid u vrlo procjedan površinski materijal, koji se nalazi iznad čvrstog laporovitog materijala, ne premostivog za prodiranje infiltrirane oborinske vode. Prikupljanje i koncentriranje vode u toj zoni pomaže slabljenju kohezivnih sila u materijalu, te kad one padnu ispod vršnih vrijednosti kreće proces klizanja. Upravo taj utjecaj ne kontroliranog utjecaja vode u navedenim zonama razlog je pojave nestabilnosti. Glavna zadaća u rješavanju navedene problematike biti će kvalitetan prihvat oborinskih, te podzemnih/infiltriranih voda u tlu, te njihova kontrolirana odvodnja.

Bušenjem na dvije pozicije, vidljivi je površinski sloj gline srednje plastičnosti, niskog indeksa konzistencije, povećane vlažnosti do dubine 1.7 do 2.3 m, nakon kojega je registriran sloj visoko plastične gline srednje konzistencije. Navedeni slojevi ne predstavljaju adekvatan sloj za prijenos dinamičkog opterećenja od potporno zaštitnih konstrukcija. Nakon navedenih procjednih slojeva gline, nalazi se laporoviti prah visoke plastičnosti do lapor koja predstavlja povoljan i čvrst materijal za prijenos navedenog opterećenja te temeljne navedenog dijela konstrukcije pristupne ceste. Navedena dubina pojave laporovite podloge predstavlja i dubinu temeljnja buduće potporno zaštitne konstrukcije kako je to prikazano u prilogu 04/042/2016.

3. TERENSKI ISTRAŽNI RADOVI S OPISOM TLA

3.1 TERENSKI ISTRAŽNI RADOVI

Terenski istražni radovi su provedeni u listopadu, 2016. godine.

U okviru terenskih istražnih radova su obavljene *in situ* radovi sondažnog bušenja dvije sondažne bušotine, uz izvedbu terenskih i laboratorijskih ispitivanja za ocjenu mehaničkih svojstva tla (N_{60}^1).

Bušenje je izvedeno strojno motornom bušilicom uz kontinuirano praćenje jezgre iskopa.

Tokom bušenja su uzimani reprezentativni PU^2 za potrebe laboratorijskih ispitivanja općih i mehaničkih svojstava tla.

Sva jezgra dobivena bušenjem je identificirana i klasificirana prema AC^3 klasifikaciji pri čemu su korištene *in situ* izmjerene q_{PP}^4 .

U sondažnim bušotinama je opažena PPV^5 i NPV^6 na kraju bušenja.

Tlocrtni položaj sondažnih bušotina je prikazan na prilogu br. 1/042/16.

Opisi sondažnih bušotina s pripadnim presječnim profilom i klasifikacijom slojeva dan je u prilogu br. 2/042/16.

3.2 LABORATORIJSKA ISPITIVANJA

Laboratorijskim ispitivanjima su obuhvaćeni pokusi za određivanje općih i mehaničkih karakteristika reprezentativnih poremećenih uzoraka tla:

3.2.1 IDENTIFIKACIJSKI POKUSI

- prirodni sadržaj vlage i indeks konzistencije (w_0, I_c), HRN U.B1.012-1979
- Atterbergove granice plastičnosti (w_L, w_P), HRN U.B1.020-1980
- prirodna vlažna i suha zapreminska težina (γ, γ_d), HRN U.B1.016-1968.

3.2.2 POKUSI ODREĐIVANJA MEHANIČKIH SVOJSTAVA TLA

- jednoosna čvrstoća sa slobodnim bočnim širenjem (q_{uL}), HRN U.B1.030-1968.
- posmična čvrstoća izravnim smicanjem (Φ, c), HRN U.B1.028-1969.

Rezultati laboratorijskih ispitivanja prikazani su na prilozima 7/042/16 do 10/042/16.

¹ Rezultat Standardnog Penetracionog Pokusa [broj udaraca/stopa] za $ER, 60\%$ (Rod Energy Ratio-koeficijent iskorištenja energije)

² Poremećeni Uzorak

³ Airfield Classification

⁴ Približna jednoosna čvrstoća sa slobodnim bočnim širenjem (džepni penetrometar - Pocket Penetrometer) koristi se samo za klasifikaciju

⁵ Pojava Podzemne Vode

⁶ Nivo Podzemne Vode

3.3 SASTAV I SVOJSTVA TLA

Detaljan opis sastava i karakteristika temeljnog tla je prikazan na prilogu 2/042/16, te kroz rezultate laboratorijske analize, a ovdje je samo sažetak s osvrtom na geomehničke karakteristike značajne za temeljenje. Temeljno tlo je slijedećih općih i mehaničkih svojstava :

SLOJ 1

CI Sloj gline srednje plastičnosti, srednje konzistentnog stanja. U površinskom dijelu sloj ima više udjela prahovito pijeksovite komponente te je relativno procjedan, a sloj je ukupno smeđe boje. Navedeni sloj se proteže od dubine 1.7 (SB-1) do 2.3 m (SB-2). Površinski materijal je uznemiren i izmiješan u procesu klizanja.

Laboratorijskim ispitivanjima **NU** su dobivene slijedeće vrijednosti: $I_c = 0.36$ do 0.49

SLOJ 2

CH Sloj gline visoke plastičnosti, pri vrhu sloja srednje konzistencije dok sa porastom dubine sloj prelazi u kruto konzistentno stanje, smeđe boje, registriran je do dubine 3.8 do 4.0 m od razine postojećeg terena. Pri dubini 3.7 m do 3.9 m vidljive su mjestimične naznake lapora koji je u toj zoni u tragovima. Sloj je pri površini povećane prirodne vlage.

Laboratorijskim ispitivanjima **NU** su dobivene slijedeće vrijednosti: $I_c = 0.62$ do 0.67 , $c=25$ kPa, $\phi=18^\circ$

SLOJ 3

MH/LAPOR Sloj čvrstog visokoplastičnog praha do lapora, polučvrste do čvrste konzistencije. Navedeni sloj registriran je do dna bušenja, te predstavlja dobru podlogu za prijenos opterećenja od potpuno zaštitne konstrukcije.

Laboratorijskim ispitivanjima **NU** su dobivene slijedeće vrijednosti: $I_c = 0.96$ do 1.01

Pri istražnim radovima nije registrirana pojava podzemne vode, ali je zamjećena veća vlažnost materijala u zoni slojeva 1 i 2. Obzirom na sastav tla u navedenoj zoni, pocjeđivanje oborinske vode kroz porozne slojeve tla lako se ostvaruje.

Parametri mehaničkih svojstava tla vidljivi su u donjoj tabeli:

SLOJ	Kut unutrašnjeg trenja Φ [°]	Kohezija c [kPa]	Zapreminska težina γ [kN/m ³]	Modul stižljivosti M_v [MPa]
SLOJ 1 / CI	22	13	18	5
SLOJ 2 / CH	25	18	18	6
SLOJ 3 / MH /LAPOR	30	50	19	15

3. GEOSTATIČKI PRORAČUN

Geostatičkim proračunom predviđena je zaštitna konstrukcija koja se sastoji od:

KARAKTERISTIČNI PRESJEČNI PROFIL - potporno zaštitni zid u dva nivoa sa bermom na sredini raspona.

Potporna konstrukcija sastoji se od dva zida međusobno odvojena bermom, od čega prvi (bliži cesti) savladava ukupnu denivelaciju terena u visini 4.0 m, dok drugi zid savladava denivelaciju 3.8 m. Temeljna stopa oba zida je visine je 1.5 m, dok je nadtemeljna visina prvog potpornog zida 2.5 m, a drugog 2.3 m. Materijal za izvođenje zida je 70% kamen te 30 % beton.

Za proračunski pristup koristi se projektni pristup 3, prema EUROCODE načelima (A1/A2+M2+R3). Razmatrani karakteristični presječni profili su vidljivi su u prilogu ovog projekta. Za naveden presjek provedene su analize opterećenja, geostatički proračun, provjera na klizanje i prevrtanje zaštitne konstrukcije.

GEOTEHNIČKI PARAMETRI TLA

Na temelju provedenih terenskog iskopa, usvajaju se geotehnički parametri za provedbu geostatičkih analiza prikazani u tablici 1.

SLOJ	DEBLJINA SLOJA	SIMBOL	OPIS TLA	KARAKTERISTIČNI PARAMETRI TLA	PRORAČUNSKI PARAMETRI TLA (PRISTUP 3, M2)
1	PASIVNI OTPOR	CI	Sloj gline srednje plastičnosti, srednje do krute konzistencije.	$\gamma = 18.0 \text{ kN/m}^3$ $c = 18 \text{ kPa}$ $\varphi = 25.0^\circ$	$c = 14.4 \text{ kPa}$ $\varphi = 20.0^\circ$
2	DRENAŽA	GP	Drenažni sloj šljunka iza potporne konstrukcije	$\gamma = 19.0 \text{ kN/m}^3$ $c = 0 \text{ kPa}$ $\varphi = 45.0^\circ$	$c = 0 \text{ kPa}$ $\varphi = 36.0^\circ$
3	TEMELJNO TLO	LAPOR/MH	Laporoviti prah do lapor	$\gamma = 19.0 \text{ kN/m}^3$ $c = 50 \text{ kPa}$ $\varphi = 30.0^\circ$	$c = 40 \text{ kPa}$ $\varphi = 24.0^\circ$

Tablica 1. Geotehnički parametri za geostatičke analize



3.1. KARAKTERISTIČNI PROFIL POTPORNOG ZIDA 1

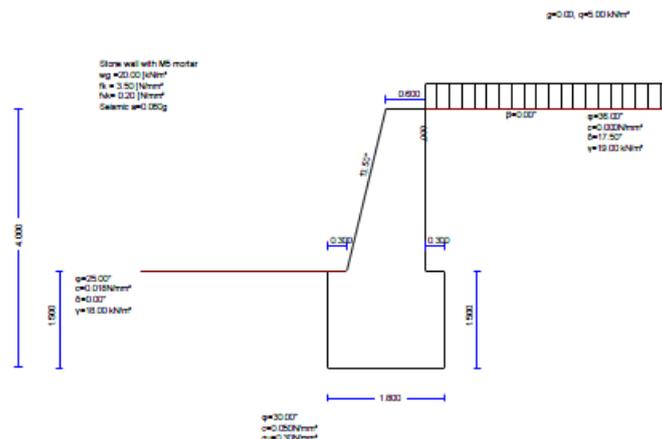
ANALIZA OPTEREĆENJA

Profil razmatra kritični presjek "potpornog zida 1" koji se izvodi na poziciji uz cestu prema pristupnoj cesti u sanaciji. Dodatna opterećenja na predmetni zid ostvaruju se preko prometnog opterećenja pristupnom cestom, te su ista uzeta u obzir. Analiza će se provesti sistemom eurocode, projektnim pristupom 3, koji podrazumijeva faktorizacija svojstva materijala ϕ i c sa faktorima 1.25. Parcijalni faktori otpora i povoljna djelovanja se ne faktoriziraju, dok će se trajna nepovoljna djelovanja faktorizirati sa 1.35.

1. ZID 1

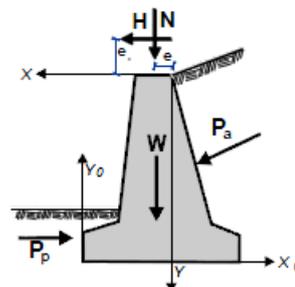
Gravity retaining wall

(EC2 EN1992-1-1:2004, EC0 EN1990:2002, EC7 EN1997-1-1:2004, EC8 EN1998-5:2004,)



1.1. Wall properties-Parameters-Code requirements

Dimensions	
Height of wall	$h = 4.000 \text{ m}$
Transverse length of wall	$L = 10.000 \text{ m}$
Stem thickness at top	$B1 = 0.600 \text{ m}$
Stem thickness at bottom	$B2 = 1.200 \text{ m}$
Width of wall base	$B = 1.800 \text{ m}$
Width of wall toe	0.300 m
Width of wall heel	0.300 m
Height of wall stem	2.500 m
Thickness of wall footing	1.500 m
Front thickness of wall toe	1.500 m
Back thickness of wall heel	1.500 m
Slope (batter) at frontface	$18.496^\circ (1:4.17)$
Slope (batter) at backface	$0.000^\circ (0:1)$



Weight of wall

Unit weight of wall material	$\gamma_g = 20.000 \text{ kN/m}^3$
Cross section area of wall	$A = 4.950 \text{ m}^2$
Self weight per meter of wall	$W = 4.950 \times 20.000 = 99.00 \text{ kN/m}$
Center of gravity of wall at	$x = 0.539 \text{ m}, y = 2.404 \text{ m} (x_0 = 0.961 \text{ m}, y_0 = 1.596 \text{ m})$



Project Beton

Pg. 2

Wall materials

Compressive strength 3.50 N/mm²
Shear strength 0.20 N/mm²

Weight of backfill

Weight of backfill per meter Ws=14.25 kN/m
Center of gravity of backfill x=-0.150 m, y=1.250 m

1.2. Partial factors for actions and soil properties

(EC7 Tab. A.1-A.4, EC8-5 §3.1)

Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)

		(EQU)	(STR)	(GEO)	(Seismic)
Actions	Permanent Unfavourable	γ_{Gdst} : 1.10	1.35	1.35	1.00
	Permanent Favourable	γ_{Gstb} : 0.90	1.00	1.00	1.00
	Variable Unfavourable	γ_{Qdst} : 1.50	1.50	1.50	1.00
	Variable Favourable	γ_{Qstb} : 0.00	0.00	0.00	0.00
Soil parameters	Angle of shearing resistance	γ_{ϕ} : 1.25	1.25	1.25	1.25
	Effective cohesion	γ_c : 1.25	1.25	1.25	1.25
	Undrained shear strength	γ_{cu} : 1.40	1.40	1.40	1.40
	Unconfined strength	γ_{qu} : 1.40	1.40	1.40	1.40
	Weight density	γ_w : 1.00	1.00	1.00	1.00

1.3. Properties of foundation soil

Bearing capacity of foundation soil $q_u=0.30$ N/mm²
Friction angle between wall footing and soil $\phi=30.00^\circ$, friction coefficient $\tan(\phi)=0.577$
Cohesion between wall footing and soil $c=0.050$ N/mm²

1.4. Seismic coefficients

(EC8 EN1998-5:2004, §7.3.2)

Design ground acceleration ratio $g_h=a_{avg}$, $a=0.06$ (EC8-5 §7.3.2)
Soil factor $S=1.00$ (EC8 §3.2.2.2)
Importance factor $\gamma_I=1.00$ (EC8 §3.2.1, T.4.3)
Reduction factor for seismic coefficient $r=1.50$ (EC8-5 Table 7.1)
Coefficient for horizontal seismic force $k_h=1.00 \times 0.06 \times 1.00 / 1.500 = 0.040$ (EC8-5 Eq.7.1)
Coefficient for vertical seismic force $k_v=0.30 \times 0.040 = 0.020$ (EC8-5 Eq.7.2)

Forces due to seismic load (except from earth pressure)

Horizontal seismic force due to self weight $F_{wx} = 99.00 \times 0.040 = 3.96$ kN/m
Vertical seismic force due to self weight $F_{wy} = 99.00 \times 0.020 = 1.98$ kN/m
Horizontal seismic force of backfill $F_{wsx} = 14.25 \times 0.040 = 0.57$ kN/m
Vertical seismic force of backfill $F_{wsy} = 14.25 \times 0.020 = 0.28$ kN/m

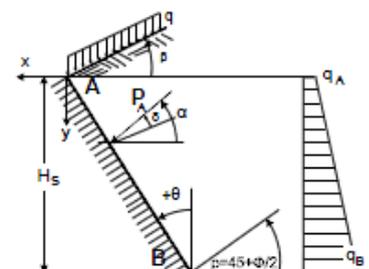
1.5. Computation of active earth pressure (Coulomb theory)

1.5.1. Wall part from $y=0.000$ m to $y=4.000$ m, $H_s=4.000$ m

Top point A $x=0.000$ m $y=0.000$ m
Bottom point B $x=0.000$ m $y=4.000$ m

Soil properties

Soil type : Dense sand
Unit weight of soil $\gamma = 19.00$ kN/m³
Unit weight of soil (saturated) $\gamma_s = 19.00$ kN/m³
Unit weight of water $\gamma_w = 10.00$ kN/m³
Angle of shearing resistance of ground $\phi = 36.00^\circ$
Cohesion of ground $c = 0.000$ N/mm²
Slope angle of ground surface $\beta = 0.00^\circ$
Inclination angle of the wall backface $\theta = 0.00^\circ$
Angle of shear resist. between ground-wall $\delta = 17.50^\circ$





Project Beton

Fig. 2

Wall materials

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Weight of backfill

Weight of backfill per meter Ws=14.25 kN/m
Center of gravity of backfill x=-0.150 m, y=1.250 m

1.2. Partial factors for actions and soil properties

(EC7 Tab. A.1-A.4, EC8-5 §3.1)

		Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)			
		(EQU)	(STR)	(GEO)	(Seismic)
Actions	Permanent Unfavourable	γ_{Gdst} : 1.10	1.35	1.35	1.00
	Permanent Favourable	γ_{Gstb} : 0.90	1.00	1.00	1.00
	Variable Unfavourable	γ_{Qdst} : 1.50	1.50	1.50	1.00
	Variable Favourable	γ_{Qstb} : 0.00	0.00	0.00	0.00
Soil parameters	Angle of shearing resistance	γ_{ϕ} : 1.25	1.25	1.25	1.25
	Effective cohesion	γ_c : 1.25	1.25	1.25	1.25
	Undrained shear strength	γ_{cu} : 1.40	1.40	1.40	1.40
	Unconfined strength	γ_{qu} : 1.40	1.40	1.40	1.40
	Weight density	γ_w : 1.00	1.00	1.00	1.00

1.3. Properties of foundation soil

Bearing capacity of foundation soil $q_u=0.30$ N/mm²
Friction angle between wall footing and soil $\phi=30.00^\circ$, friction coefficient $\tan(\phi)=0.577$
Cohesion between wall footing and soil $c=0.050$ N/mm²

1.4. Seismic coefficients

(EC8 EN1998-5:2004, §7.3.2)

Design ground acceleration ratio $g_h=a_{avg}$, $a=0.06$ (EC8-5 §7.3.2)
Soil factor $S=1.00$ (EC8 §3.2.2.2)
Importance factor $\gamma_I=1.00$ (EC8 §3.2.1, T.4.3)
Reduction factor for seismic coefficient $r=1.50$ (EC8-5 Table 7.1)
Coefficient for horizontal seismic force $k_h=1.00 \times 0.06 \times 1.00 / 1.500 = 0.040$ (EC8-5 Eq. 7.1)
Coefficient for vertical seismic force $k_v=0.50 \times 0.040 = 0.020$ (EC8-5 Eq. 7.2)

Forces due to seismic load (except from earth pressure)

Horizontal seismic force due to self weight $F_{wx} = 99.00 \times 0.040 = 3.96$ kN/m
Vertical seismic force due to self weight $F_{wy} = 99.00 \times 0.020 = 1.98$ kN/m
Horizontal seismic force of backfill $F_{wmx} = 14.25 \times 0.040 = 0.57$ kN/m
Vertical seismic force of backfill $F_{wmy} = 14.25 \times 0.020 = 0.28$ kN/m

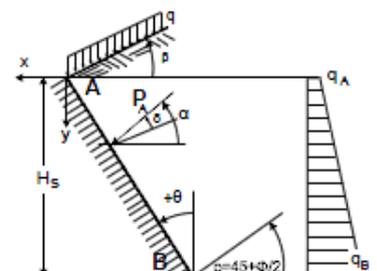
1.5. Computation of active earth pressure (Coulomb theory)

1.5.1. Wall part from $y=0.000$ m to $y=4.000$ m, $H_s=4.000$ m

Top point A $x=0.000$ m $y=0.000$ m
Bottom point B $x=0.000$ m $y=4.000$ m

Soil properties

Soil type : Dense sand
Unit weight of soil $\gamma = 19.00$ kN/m³
Unit weight of soil (saturated) $\gamma_s = 19.00$ kN/m³
Unit weight of water $\gamma_w = 10.00$ kN/m³
Angle of shearing resistance of ground $\phi = 36.00^\circ$
Cohesion of ground $c = 0.000$ N/mm²
Slope angle of ground surface $\beta = 0.00^\circ$
Inclination angle of the wall backface $\theta = 0.00^\circ$
Angle of shear resist. between ground-wall $\delta = 17.50^\circ$





Project Beton

Pg. 3

Loads on soil surface

Permanent uniform load $g = 0.00 \text{ kN/m}^2$
Variable uniform load $q = 5.00 \text{ kN/m}^2$

Earth pressure according to Coulomb theory

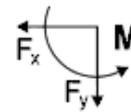
	EQU	STR	GEO
Angle of rupture plane $\rho = 45^\circ + \varphi/2$	= 59.40	59.40	59.40
Coefficient of active earth pressure K_a	0.317	0.317	0.317

$$K_A = \frac{\cos^2(\varphi - \theta)}{\cos^2\theta \cos(\theta + \delta) \left[1 + \frac{\sin(\varphi + \delta) \sin(\varphi - \beta)}{\cos(\theta + \delta) \cos(\theta - \beta)} \right]^2}$$

Earth pressure $q(y) = q_A + \gamma \cdot y \cdot K_a$

Permanent actions

	EQU	STR	GEO
Earth pressure at the top ($y=y_A$)	$q_A = 0.00$	0.00	0.00 kN/m^2
Earth pressure at the bottom ($y=y_A + 4.00\text{m}$)	$q_B = 24.09$	24.09	24.09 kN/m^2
Earth force $P_a = \frac{1}{2}(q_A + q_B)H$	$P_a = 48.18$	48.18	48.18 kN/m
Angle of earth force	$\alpha = 14.00$	14.00	14.00°
Earth force in x direction	$F_{ax} = 45.95$	45.95	45.95 kN/m
Earth force in y direction	$F_{ay} = 14.49$	14.49	14.49 kN/m
Moment of earth force at top point ($x=0, y=0$)	$M = -122.55$	-122.55	-122.55 kNm/m
Point of application of earth force $x = 0.000 \text{ m}$, $y = 2.667 \text{ m}$			



Variable actions

	EQU	STR	GEO
Earth pressure at the top ($y=y_A$)	$q_A = 1.59$	1.59	1.59 kN/m^2
Earth pressure at the bottom ($y=y_A + 4.00\text{m}$)	$q_B = 1.59$	1.59	1.59 kN/m^2
Earth force $P_a = \frac{1}{2}(q_A + q_B)H$	$P_a = 6.36$	6.36	6.36 kN/m
Angle of earth force	$\alpha = 14.00$	14.00	14.00°
Earth force in x direction	$F_{ax} = 6.07$	6.07	6.07 kN/m
Earth force in y direction	$F_{ay} = 1.91$	1.91	1.91 kN/m
Moment of earth force at top point ($x=0, y=0$)	$M = -12.14$	-12.14	-12.14 kNm/m
Point of application of earth force $x = 0.000 \text{ m}$, $y = 2.000 \text{ m}$			

Total forces and moments

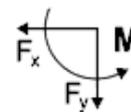
Forces and moments at bottom point B ($x=0.000 \text{ m}$, $y=4.000 \text{ m}$)

Permanent actions

	EQU	STR	GEO
Total horizontal earth force F_{sx}	45.95	45.95	45.95 kN/m
Total vertical earth force F_{sy}	14.49	14.49	14.49 kN/m
Total moment of earth force M_s	61.25	61.25	61.25 kNm/m

Variable actions

	EQU	STR	GEO
Total horizontal earth force F_{sx}	6.07	6.07	6.07 kN/m
Total vertical earth force F_{sy}	1.91	1.91	1.91 kN/m
Total moment of earth force M_s	12.14	12.14	12.14 kNm/m



Seismic loading

(EC8 EN1998-1-1:2004, §7.3.2, Annex E)

Horizontal seismic coefficient $k_h = 1.00 \times 0.06 \times 1.00 / 1.500 = 0.040$ (EC8 Eq.7.1, T.7.1)
Vertical seismic coefficient $k_v = 0.50 \times 0.040 = 0.020$ (EC8 Eq.7.2)
Soil above the water table (EC8 Annex E.5)
 $\tan(\omega) = k_h / (1 - k_v) = 0.040 / (1 - 0.020) = 0.041$, $\omega = 2.34^\circ$

Method Mononobe-Okabe (EC8 Annex E.4)

for active earth force during seismic loading
Coefficient of active earth pressure, $K_e = 0.337$
Additional earth pressure due to seismic load
over STR load case $\xi = (K_e' / K_e - 1) = (0.337 / 0.317 - 1) = 0.063$

$$K_E = \frac{\cos^2(\varphi - \theta)}{\cos\omega \cos^2\theta \cos(\theta + \omega) \left[1 + \frac{\sin(\varphi + \delta) \sin(\varphi - \omega - \beta)}{\cos(\theta + \omega + \delta) \cos(\theta - \beta)} \right]^2}$$

Earth force due to seismic load (Permanent actions) $F_x = 1.063 \times 45.95 = 48.84 \text{ kN/m}$
Earth force due to seismic load (Variable actions) $F_x = 1.063 \times 6.07 = 6.45 \text{ kN/m}$



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Pg. 4

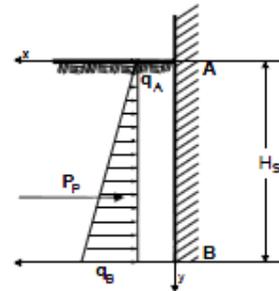
1.6. Computation of passive earth pressure (Rankine theory)

1.6.1. Wall part from y=2.500 m to y=4.000 m, Hs=1.500 m

Top point A x= 1.500 m y= 2.500 m
Bottom point B x= 1.500 m y= 4.000 m

Soil properties

Soil type : Dense sand
Unit weight of soil $\gamma = 18.00 \text{ kN/m}^3$
Unit weight of soil (saturated) $\gamma_s = 18.00 \text{ kN/m}^3$
Unit weight of water $\gamma_w = 10.00 \text{ kN/m}^3$
Angle of shearing resistance of ground $\varphi = 25.00^\circ$
Cohesion of ground $c = 0.018 \text{ N/mm}^2$
Slope angle of ground surface $\beta = 0.00^\circ$
Earth pressure on vertical surface $\theta = 0.00^\circ$
Angle of shear resist. between ground-wall $\delta = 0.00^\circ$



Earth pressure according to Coulomb theory

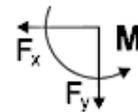
	EQU	STR	GEO
Angle of rupture plane $\rho = 45^\circ - \varphi/2 = 35.00^\circ$	35.00	35.00	35.00
Coefficient of passive earth pressure $K_p = 2.040$	2.040	2.040	2.040

$$K_p = \frac{\cos^2(\varphi + \theta)}{\cos^2 \theta \cos(\theta - \delta) \left[1 - \frac{\sin(\varphi + \delta) \sin(\varphi + \beta)}{\cos(\theta - \delta) \cos(\theta - \beta)} \right]^2}$$

Earth pressure $q(y) = q_A + \gamma \cdot y \cdot K_p$

Permanent actions

	EQU	STR	GEO
Earth pressure at the top (y=yA)	qA= 0.00	0.00	0.00 kN/m ²
Earth pressure at the bottom (y=yA+ 1.50m)	qB=-55.08	-55.08	-55.08 kN/m ²
Earth force Pa= 1/2(qA+qB)H	Pp= 41.31	41.31	41.31 kN/m
Angle of earth force	$\alpha = 0.00$	0.00	0.00 °
Earth force in x direction	Fpx=-41.31	-41.31	-41.31 kN/m
Earth force in y direction	Fpy= 0.00	0.00	0.00 kN/m
Moment of earth force at top point (x=0, y=0)	M = 144.59	144.59	144.59 kNm/m
Point of application of earth force x= 1.500 m, y= 3.500 m			

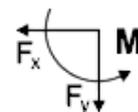


Total forces and moments

Forces and moments at bottom point B (x=1.500 m, y=4.000 m)

Permanent actions

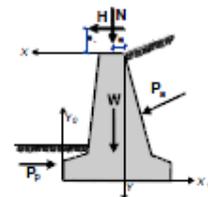
	EQU	STR	GEO
Total horizontal earth force	Fsx=-41.31	-41.31	-41.31 kN/m
Total vertical earth force	Fsy= 0.00	0.00	0.00 kN/m
Total moment of earth force	Ms = -20.66	-20.66	-20.66 kNm/m



1.7. Checks of wall stability (EQU)

1.7.1. Forces (driving and resisting) on the wall (EQU)

Action		y1 -	y2	Fx	Fy	x	y
				[kN/m]	[kN/m]	[m]	[m]
Active earth pressure	Pa	0.00-	4.00	45.95	14.49	0.000	2.667
Backfill surcharge (live)	Pq	0.00-	4.00	6.07	1.91	0.000	2.000
Passive earth pressure	Pp	2.50-	4.00	-41.31	0.00	1.500	3.500
Wall weight	W			0.00	99.00	0.539	2.404
Backfill weight	Ws			0.00	14.25	-0.150	1.250





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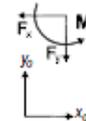
Pg. 5

1.7.2. Check of soil bearing capacity (EQU)

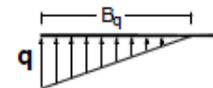
(EC7 EN1997-1-1:2004, §6.5.2)

Check for $0.90 \times (\text{self weight} + \text{top vertical dead load}) + 0.00 \times (\text{top vertical live load})$

Action	(P,γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]
Active earth pressure	Pax1.10	0.00- 4.00	50.55	15.94	1.500	1.333	43.46
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	13.91
Wall weight	W x0.90		0.00	89.10	0.961	1.596	-85.63
Backfill weight	Wsx0.90		0.00	12.82	1.650	2.750	-21.16
			Sum=	120.72			-49.42



Sum of vertical forces = 120.72 kN/m
Sum of moments at front toe = -49.42 kNm/m
Sum of moments at middle of base = 59.23 kNm/m
Eccentricity $ec = 59.23/120.72 = 0.491\text{m}$, $ec > 1.800/6 = 0.300\text{m}$
Soil pressure $q = 0.197 \text{ N/mm}^2$, $Bq = 1.228 \text{ m}$
Effective footing $L' = 1.800 - 2 \times 0.491 = 0.819 \text{ m}$
Soil bearing capacity $Rd = L' \cdot qu / \gamma M = 0.819 \times (1000 \times 0.30) / 1.40 = 175.50 \text{ kN/m}$
Bearing resistance check $Vd = 120.72 < Rd = 175.50 \text{ kN/m}$, Is verified

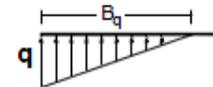


(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

Check for $1.10 \times (\text{self weight} + \text{top vertical dead load}) + 1.50 \times (\text{top vertical live load})$

Action	(P,γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]
Active earth pressure	Pax1.10	0.00- 4.00	50.55	15.94	1.500	1.333	43.46
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	13.91
Wall weight	W x1.10		0.00	108.90	0.961	1.596	-104.65
Backfill weight	Wsx1.10		0.00	15.68	1.650	2.750	-25.86
			Sum=	143.38			-73.14



(EC7 Annex D)

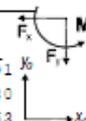
(EC7 Eq.2.2, Eq.6.1)

1.7.3. Failure check due to overturning (EQU)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe ($x_0=0, y_0=0$) ($x=1.500, y=4.000 \text{ m}$)

Action	(P,γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.10	0.00- 4.00	50.55	15.94	1.500	1.333	67.38	23.91
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	18.21	4.30
Wall weight	W x0.90		0.00	89.10	0.961	1.596	0.00	85.63
Backfill weight	Wsx0.90		0.00	12.82	1.650	2.750	0.00	21.16
			Sum=				85.59	135.00



Sum of overturning moments = 85.59 kNm/m
Sum of moments resisting overturning = 135.00 kNm/m
Overturning check $Med = 85.59 < Mrd = 135.00 \text{ kNm/m}$, Is verified



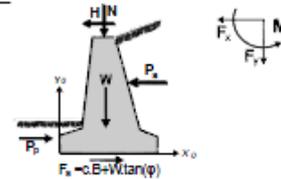
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Fig. 6

1.7.4. Failure check against sliding (EQU)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(P,y)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.10	0.00- 4.00	50.55	0.00	15.94
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	0.00	2.86
Passive earth pressure	Ppx0.90	2.50- 4.00	0.00	37.18	0.00
Wall weight	W x0.90		0.00	0.00	89.10
Backfill weight	Wsx0.90		0.00	0.00	12.82
Sum=			59.66	37.18	120.72



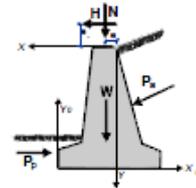
Soil friction $R_d = V_d \cdot \tan(\phi) / \gamma M = 120.72 \cdot \tan(30.00^\circ) / 1.25 = 55.76 \text{ kN/m}$
 Soil cohesion $R_d = A \cdot c_u / \gamma M = 1000 \cdot 1.228 \cdot 0.050 / 1.25 = 49.12 \text{ kN/m}$
 (resisting forces from effective cohesion are neglected)
 Sum of driving forces = 59.66 kN/m
 Sum of resisting forces (37.18+55.76) = 92.94 kN/m
 Sliding resistance check $H_d = 59.66 < R_d = 92.94 \text{ kN/m}$, Is verified

(EC7 §6.5.3. 10)

1.8. Checks of wall stability (STR)

1.8.1. Forces (driving and resisting) on the wall (STR)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 4.00	45.95	14.49	0.000	2.667
Backfill surcharge (live)	Pq	0.00- 4.00	6.07	1.91	0.000	2.000
Passive earth pressure	Pp	2.50- 4.00	-41.31	0.00	1.500	3.500
Wall weight	W		0.00	99.00	0.539	2.404
Backfill weight	Ws		0.00	14.25	-0.150	1.250

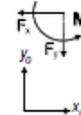


1.8.2. Check of soil bearing capacity (STR)

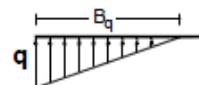
(EC7 EN1997-1-1:2004, §6.5.2)

Check for 1.00x(self weight+top vertical dead load)+0.00x(top vertical live load)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]
Active earth pressure	Pax1.35	0.00- 4.00	62.03	19.56	1.500	1.333	53.34
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	13.91
Wall weight	W x1.00		0.00	99.00	0.961	1.596	-95.14
Backfill weight	Wsx1.00		0.00	14.25	1.650	2.750	-23.51
Sum=			71.14	22.41			-51.40



Sum of vertical forces = 135.67 kN/m
 Sum of moments at front toe = -51.40 kNm/m
 Sum of moments at middle of base = 70.70 kNm/m
 Eccentricity $ec = 70.70 / 135.67 = 0.521 \text{ m}$, $ec > 1.800 / 6 = 0.300 \text{ m}$
 Soil pressure $q = 0.239 \text{ N/mm}^2$ $Bq = 1.137 \text{ m}$
 Effective footing $L' = 1.800 - 2 \cdot 0.321 = 0.758 \text{ m}$
 Soil bearing capacity $R_d = L' \cdot q_u / \gamma M = 0.758 \cdot (1000 \cdot 0.30) / 1.40 = 162.43 \text{ kN/m}$
 Bearing resistance check $V_d = 135.67 < R_d = 162.43 \text{ kN/m}$, Is verified



(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

Check for 1.35x(self weight+top vertical dead load)+1.50x(top vertical live load)

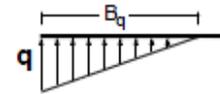
Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]
Active earth pressure	Pax1.35	0.00- 4.00	62.03	19.56	1.500	1.333	53.34
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	13.91
Wall weight	W x1.35		0.00	133.65	0.961	1.596	-128.44
Backfill weight	Wsx1.35		0.00	19.24	1.650	2.750	-31.74
Sum=			71.14	22.41			-92.93



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Pg. 7

Sum of vertical forces = 175.31 kN/m
Sum of moments at front toe = -92.93 kNm/m
Sum of moments at middle of base = 64.85 kNm/m
Eccentricity $e_c = 64.85 / 175.31 = 0.370\text{m}$, $e_c > 1.800 / 6 = 0.300\text{m}$
Soil pressure $q = 0.220\text{ N/mm}^2$ $E_q = 1.890\text{ m}$
Effective footing $L' = 1.800 - 2 \times 0.370 = 1.060\text{ m}$
Soil bearing capacity $R_d = L' \cdot q_u / \gamma M = 1.060 \times (1000 \times 0.30) / 1.40 = 227.14\text{ kN/m}$
Bearing resistance check $V_d = 175.31 < R_d = 227.14\text{ kN/m}$, Is verified



(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

1.8.3. Failure check due to overturning (STR)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe ($x_0=0, y_0=0$) ($x=1.500, y=4.000\text{ m}$)

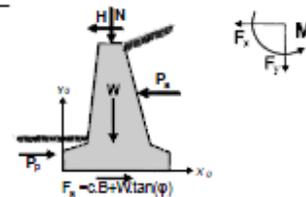
Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	Mo+ [kNm/m]	Mo- [kNm/m]	F_x F_y	M
Active earth pressure	$P_{ax} 1.35$	0.00- 4.00	62.03	19.56	1.500	1.333	82.69	29.35	F_x	F_y
Backfill surcharge (live)	$P_{qx} 1.50$	0.00- 4.00	9.11	2.86	1.500	2.000	18.21	4.30		
Wall weight	$W \times 1.00$		0.00	99.00	0.961	1.896	0.00	96.14		
Backfill weight	$W_{sx} 1.00$		0.00	14.25	1.650	2.750	0.00	23.51		
							Sum=	100.90	152.30	

Sum of overturning moments = 100.90 kNm/m
Sum of moments resisting overturning = 152.30 kNm/m
Overturning check $M_{ed} = 100.90 < M_{rd} = 152.30\text{ kNm/m}$, Is verified

1.8.4. Failure check against sliding (STR)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(P,y)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]	
Active earth pressure	$P_{ax} 1.35$	0.00- 4.00	62.03	0.00	19.56	
Backfill surcharge (live)	$P_{qx} 1.50$	0.00- 4.00	9.11	0.00	2.86	
Passive earth pressure	$P_{px} 1.00$	2.50- 4.00	0.00	41.31	0.00	
Wall weight	$W \times 1.00$		0.00	0.00	99.00	
Backfill weight	$W_{sx} 1.00$		0.00	0.00	14.25	
			Sum=	71.14	41.31	135.67



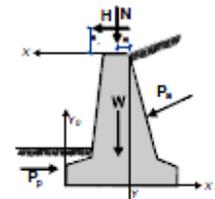
Soil friction $R_d = V_d \cdot \tan \phi / \gamma M = 135.67 \times \tan(30.00^\circ) / 1.25 = 62.66\text{ kN/m}$
Soil cohesion $R_d = A \cdot c_u / \gamma M = 1000 \times 1.137 \times 0.050 / 1.25 = 45.47\text{ kN/m}$
(resisting forces from effective cohesion are neglected)
Sum of driving forces = 71.14 kN/m
Sum of resisting forces (41.31+62.66) = 103.97 kN/m
Sliding resistance check $H_d = 71.14 < R_d = 103.97\text{ kN/m}$, Is verified

(EC7 §6.5.3. 10)

1.9. Checks of wall stability (GEO)

1.9.1. Forces (driving and resisting) on the wall (GEO)

Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	P_a	0.00- 4.00	45.95	14.49	0.000	2.667
Backfill surcharge (live)	P_q	0.00- 4.00	6.07	1.91	0.000	2.000
Passive earth pressure	P_p	2.50- 4.00	-41.31	0.00	1.500	3.500
Wall weight	W		0.00	99.00	0.539	2.404
Backfill weight	W_s		0.00	14.25	-0.150	1.250





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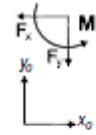
Pg. 8

1.9.2. Check of soil bearing capacity (GEO)

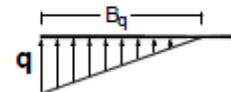
(EC7 EN1997-1-1:2004, §6.5.2)

Check for 1.00x(self weight+top vertical dead load)+0.00x(top vertical live load)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.35	0.00- 4.00	62.03	19.56	1.500	1.333	53.34
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	13.91
Wall weight	W x1.00		0.00	99.00	0.961	1.596	-95.14
Backfill weight	Wsx1.00		0.00	14.25	1.650	2.750	-23.51
			Sum=	135.67			-51.40



Sum of vertical forces = 135.67 kN/m
Sum of moments at front toe = -51.40 kNm/m
Sum of moments at middle of base = 70.70 kNm/m
Eccentricity $ec=70.70/135.67=0.521m$, $ec>1.800/6=0.300m$
Soil pressure $q=0.239 N/mm^2$ $Bq=1.137 m$
Effective footing $L'=1.800-2x0.521= 0.758 m$
Soil bearing capacity $Rd=L' \cdot qu/\gamma M=0.758x(1000x0.30)/1.40= 162.43 kN/m$
Bearing resistance check $Vd=135.67 < Rd=162.43 kN/m$, Is verified

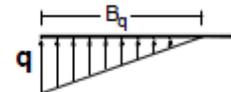


(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

Check for 1.35x(self weight+top vertical dead load)+1.50x(top vertical live load)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.35	0.00- 4.00	62.03	19.56	1.500	1.333	53.34
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	13.91
Wall weight	W x1.35		0.00	133.65	0.961	1.596	-128.44
Backfill weight	Wsx1.35		0.00	19.24	1.650	2.750	-31.74
			Sum=	175.31			-92.93



(EC7 Annex D)

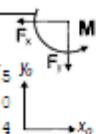
(EC7 Eq.2.2, Eq.6.1)

1.9.3. Failure check due to overturning (GEO)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe ($xo=0, yo=0$) ($x=1.500, y=4.000 m$)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.35	0.00- 4.00	62.03	19.56	1.500	1.333	82.69	29.35
Backfill surcharge (live)	Pqx1.50	0.00- 4.00	9.11	2.86	1.500	2.000	18.21	4.30
Wall weight	W x1.00		0.00	99.00	0.961	1.596	0.00	95.14
Backfill weight	Wsx1.00		0.00	14.25	1.650	2.750	0.00	23.51
			Sum=				100.90	152.30



Sum of overturning moments = 100.90 kNm/m
Sum of moments resisting overturning = 152.30 kNm/m
Overturning check $Med=100.90 < Mrd=152.30 kNm/m$, Is verified



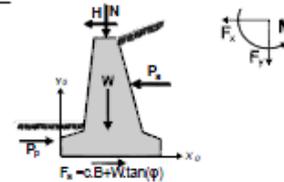
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Fig. 9

1.9.4. Failure check against sliding (GEO)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(P,y)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Fax1.38	0.00- 4.00	62.03	0.00	19.56
Backfill surcharge (live)	Fqxl.50	0.00- 4.00	9.11	0.00	2.86
Passive earth pressure	Fpx1.00	2.50- 4.00	0.00	41.31	0.00
Wall weight	W x1.00		0.00	0.00	99.00
Backfill weight	Wsx1.00		0.00	0.00	14.25
Sum=			71.14	41.31	135.67



Soil friction $Rd = Vd \cdot \tan(\phi/M) = 135.67 \times \tan(30.00^\circ) / 1.25 = 62.66$ kN/m
Soil cohesion $Rd = A \cdot cu / \gamma M = 1000 \times 1.197 \times 0.050 / 1.25 = 45.47$ kN/m
(resisting forces from effective cohesion are neglected)
Sum of driving forces = 71.14 kN/m
Sum of resisting forces (41.31+62.66) = 103.97 kN/m
Sliding resistance check $Hd = 71.14 < Rd = 103.97$ kN/m, Is verified

(EC7 §6.5.3. 10)

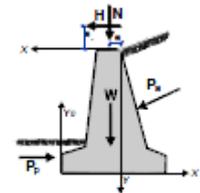
1.10. Seismic design

(EC8 EN1998-1-1:2004)

Checks of wall stability (with seismic loading)

1.10.1. Forces (driving and resisting) on the wall

Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 4.00	45.95	14.49	0.000	2.667
Backfill surcharge (live)	Fq	0.00- 4.00	6.07	1.91	0.000	2.000
Passive earth pressure	Fp	2.50- 4.00	-41.31	0.00	1.500	3.500
Wall weight	W		0.00	99.00	0.539	2.404
Backfill weight	Ws		0.00	14.25	-0.150	1.250



1.10.2. Additional forces due to seismic load

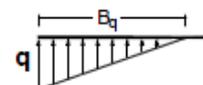
Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 4.00	2.89		0.000	2.667
Backfill surcharge (live)	Fq	0.00- 4.00	0.38		0.000	2.000
Wall weight	W		3.96	-1.98	0.539	2.404
Backfill weight	Ws		0.57	-0.28	-0.150	1.250

1.10.3. Check of soil bearing capacity (with seismic loading)

(EC7 §6.5.2)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]
Active earth pressure	Fax1.00	0.00- 4.00	48.84	14.49	1.500	1.333	43.37
Backfill surcharge (live)	Fqxl.00	0.00- 4.00	6.45	1.91	1.500	2.000	10.03
Wall weight	W x1.00		3.96	100.98	0.961	1.596	-86.92
Backfill weight	Wsx1.00		0.57	14.53	1.650	2.750	-21.48
Sum=			131.91				-55.00

Sum of vertical forces = 131.91 kN/m
Sum of moments at front toe = -55.00 kNm/m
Sum of moments at middle of base = 63.72 kNm/m
Eccentricity $ec = 63.72 / 131.91 = 0.483m$, $ec > 1.800/6 = 0.300m$
Soil pressure $q = 0.211$ N/mm² $E_q = 1.251$ m
Effective footing $L' = 1.800 - 2 \times 0.483 = 0.834$ m
Soil bearing capacity $Rd = L' \cdot qu / \gamma M = 0.834 \times (1000 \times 0.30) / 1.40 = 178.71$ kN/m
Bearing resistance check $Vd = 131.91 < Rd = 178.71$ kN/m, Is verified



(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)



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Fig. 10

1.10.4. Failure check due to overturning (with seismic loading)

(EC7 §9.7.4)

Overturning with respect to the toe ($x_0=0, y_0=0$) ($x=1.500, y=4.000$ m)

Action	(F, γ)	y1 - y2	Fx	Fy	x0	y0	Mo+	Mo-
			[kN/m]	[kN/m]	[m]	[m]	[kNm/m]	[kNm/m]
Active earth pressure	Fax1.00	0.00- 4.00	48.84	14.49	1.500	1.333	65.11	21.74
Backfill surcharge (live)	Fqxl.00	0.00- 4.00	6.45	1.91	1.500	2.000	12.90	2.87
Wall weight	W x1.00		3.96	100.98	0.961	1.896	8.22	95.14+
Backfill weight	Wsx1.00		0.87	14.53	1.650	2.750	2.03	23.51+
							Sum=	88.26 143.26

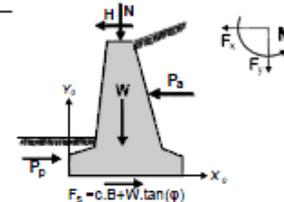
(*moments of negative seismic vertical loads, are added to the overturning moments)

Sum of overturning moments = 88.26 kNm/m
Sum of moments resisting overturning = 143.26 kNm/m
Overturning check $M_{ed}=88.26 < M_{rd}=143.26$ kNm/m, Is verified

1.10.5. Failure check against sliding (with seismic loading)

(EC7 §9.7.3, §6.5.3)

Action	(F, γ)	y1 - y2	Fx+	Fx-	Fy
			[kN/m]	[kN/m]	[kN/m]
Active earth pressure	Fax1.00	0.00- 4.00	48.84	0.00	14.49
Backfill surcharge (live)	Fqxl.00	0.00- 4.00	6.45	0.00	1.91
Passive earth pressure	Fpx1.00	2.50- 4.00	0.00	41.31	0.00
Wall weight	W x1.00		3.96	0.00	97.02
Backfill weight	Wsx1.00		0.87	0.00	13.97
		Sum=	59.82	41.31	127.39



Soil friction $R_d = V_d \cdot \tan(\phi) / \gamma_M = 127.39 \cdot \tan(30.00^\circ) / 1.25 = 58.84$ kN/m
Soil cohesion $R_d = A \cdot c_u / \gamma_M = 1000 \times 1.137 \times 0.050 / 1.25 = 45.47$ kN/m
(resisting forces from effective cohesion are neglected)
Sum of driving forces = 59.82 kN/m
Sum of resisting forces (41.31+58.84) = 100.15 kN/m
Sliding resistance check $H_d=59.82 < R_d=100.15$ kN/m, Is verified

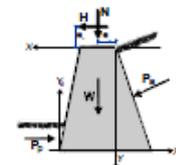
(EC7 §6.5.3, 10)

1.11. Design of wall stem

1.11.1. Loading 1.35x(permanent unfavourable)+1.00x(permanent favourable)+1.50x(variable unfav.)

Forces (at centroid of cross section) and stresses at wall stem
x, y: cross section width, b: cross section width, e: eccentricity
Fx: horizontal force, Fy: vertical force, M: moment, e/b: relative eccentricity
 σ_1, σ_2, τ : cross section normal and shear stress, B_q : effective cross section width

y	x	b	Fx	Fy	M	e/b	σ_1	σ_2	B_q/B	τ
[m]	[m]	[m]	[kN/m]	[kN/m]	[kNm/m]		[N/mm ²]	[N/mm ²]		[N/mm ²]
0.50	0.360	0.720	2.10	7.27	0.01	-0.003	-0.010	-0.010	1.000	0.003
1.00	0.420	0.840	6.15	16.38	0.80	-0.058	-0.026	-0.013	1.000	0.007
1.50	0.480	0.960	12.14	27.23	3.13	-0.120	-0.049	-0.008	1.000	0.013
2.00	0.540	1.080	20.04	39.91	7.81	-0.181	-0.077	0.000	0.956	0.019
2.50	0.600	1.200	29.90	54.43	15.61	-0.239	-0.116	0.000	0.783	0.025



1.11.2. Strength check according to EC6 EN1996-1-1:2005

Strength check in normal stresses $N_{ed} \leq N_{rd}$ (EC6 §6.1)
Vertical resistance load $N_{rd} = \xi \cdot f_k \cdot t / \gamma_M$, Vertical design load N_{ed} (EC6 §6.1.2)
 $\xi = 1 - 2e/t$, ξ capacity reduction factor for slenderness and eccentricity of loading
 e = load eccentricity, e_s = accidental eccentricity $= h/450$, h = wall height
 f_k characteristic compressive strength $f_k = 3.50$ N/mm²
 $\gamma_M = 2.50$, γ_M partial safety factor for the material



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Fig. 11

y	t	Fy	M	e/t	ξ	Ned	Nrd
[m]	[m]	[kN/m]	[kNm/m]			[kN/m]	[kN/m]
0.50	0.720	7.27	0.01	0.005	0.990	7.27	997.92 (Ned<=Nrd)
1.00	0.840	16.35	0.80	0.061	0.878	16.35	1032.53 (Ned<=Nrd)
1.50	0.960	27.23	3.13	0.123	0.754	27.23	1013.38 (Ned<=Nrd)
2.00	1.080	39.91	7.81	0.185	0.630	39.91	952.56 (Ned<=Nrd)
2.50	1.200	54.43	15.61	0.244	0.512	54.43	860.16 (Ned<=Nrd)

Design for shear strength $V_{ed} \leq V_{rd}$

Shear resistance $V_{rd} = f_{vk} \cdot t / \gamma_M$, design shear load V_{ed} (EC6 §6.2.1)

$f_{vk} = f_{vko} + 0.40 \cdot \sigma_{ed}$, σ_{ed} design compressive stress (EC6 §6.2.1)

f_{vko} shear strength under zero compressive stress $f_{vko} = 0.20 \text{ N/mm}^2$ (EC6 §3.6.2)

$\gamma_M = 2.50$, γ_M partial safety factor for the material

y	t	Fx	σ_d	Ved	Vrd
[m]	[m]	[kN/m]	[N/mm ²]	[kN/m]	[kN/m]
0.50	0.720	2.10	0.010	2.10	59.75 (Ved<=Vrd)
1.00	0.840	6.15	0.019	6.15	69.75 (Ved<=Vrd)
1.50	0.960	12.14	0.028	12.14	81.10 (Ved<=Vrd)
2.00	1.080	20.04	0.037	20.04	92.79 (Ved<=Vrd)
2.50	1.200	29.90	0.045	29.90	104.64 (Ved<=Vrd)

1.11.3. Loading $1.00x(\text{permanent unfav.}) + 1.00x(\text{permanent favour.}) + 1.00x(\text{variable}) + 1.00x(\text{seismic})$

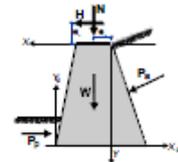
Forces (at centroid of cross section) and stresses at wall stem (with seismic loading)

x, y: cross section centroid, b: cross section width, e: eccentricity

Fx: horizontal force, Fy: vertical force, M: moment, e/b: relative eccentricity

σ_1 , σ_2 , τ : cross section normal and shear stress, Bq: effective cross section width

y	x	b	Fx	Fy	M	e/b	σ_1	σ_2	Bq/B	τ
[m]	[m]	[m]	[kN/m]	[kN/m]	[kNm/m]		[N/mm ²]	[N/mm ²]		[N/mm ²]
0.50	0.360	0.720	1.89	6.94	0.07	-0.013	-0.011	-0.009	1.000	0.003
1.00	0.420	0.840	5.30	15.50	0.78	-0.057	-0.026	-0.013	1.000	0.006
1.50	0.480	0.960	10.29	25.69	2.75	-0.105	-0.046	-0.010	1.000	0.011
2.00	0.540	1.080	16.83	37.50	6.65	-0.154	-0.071	-0.003	1.000	0.016
2.50	0.600	1.200	24.96	50.95	13.07	-0.200	-0.101	0.000	0.900	0.021



1.11.4. Strength check according to EC6 EN1996-1-1:2005 (with seismic loading)

Strength check in normal stresses $N_{ed} \leq N_{rd}$ (with seismic loading)

Vertical resistance load $N_{rd} = \xi \cdot f_k \cdot t / \gamma_M$, Vertical design load N_{ed} (EC6 §6.1)

$\xi = 1 - 2e/t$, ξ capacity reduction factor for slenderness and eccentricity of loading (EC6 §6.1.2)

e=load eccentricity-es, es=accidental eccentricity=h/450, h=wall height

f_k characteristic compressive strength $f_k = 3.50 \text{ N/mm}^2$

$\gamma_M = 2.50$, γ_M partial safety factor for the material

y	t	Fy	M	e/t	ξ	Ned	Nrd
[m]	[m]	[kN/m]	[kNm/m]			[kN/m]	[kN/m]
0.50	0.720	6.94	0.07	0.015	0.970	6.94	977.76 (Ned<=Nrd)
1.00	0.840	15.50	0.78	0.060	0.880	15.50	1034.88 (Ned<=Nrd)
1.50	0.960	25.69	2.75	0.108	0.784	25.69	1053.70 (Ned<=Nrd)
2.00	1.080	37.50	6.65	0.158	0.684	37.50	1034.21 (Ned<=Nrd)
2.50	1.200	50.95	13.07	0.205	0.590	50.95	991.20 (Ned<=Nrd)



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Pg. 12

Design for shear strength $V_{ed} \leq V_{rd}$ (with seismic loading)

(EC6 §6.2.1)

Shear resistance $V_{rd} = f_{vk} \cdot t / \gamma_M$, design shear load V_{ed}

(EC6 §6.2.1)

$f_{vk} = f_{vko} + 0.40 \cdot \sigma_{cd}$, σ_{cd} design compressive stress

(EC6 §3.6.2)

f_{vko} shear strength under zero compressive stress $f_{vko} = 0.20 \text{ N/mm}^2$

$\gamma_M = 2.50$, γ_M partial safety factor for the material

y [m]	t [m]	F_{xk} [kN/m]	σ_{cd} [N/mm ²]	V_{ed} [kN/m]	V_{rd} [kN/m]	
0.50	0.720	1.89	0.010	1.89	88.75	($V_{ed} \leq V_{rd}$)
1.00	0.840	5.30	0.018	5.30	69.62	($V_{ed} \leq V_{rd}$)
1.50	0.960	10.29	0.027	10.29	80.95	($V_{ed} \leq V_{rd}$)
2.00	1.080	16.83	0.035	16.83	92.45	($V_{ed} \leq V_{rd}$)
2.50	1.200	24.96	0.042	24.96	104.06	($V_{ed} \leq V_{rd}$)



3.2. KARAKTERISTIČNI PROFIL POTPORNOG ZIDA 2

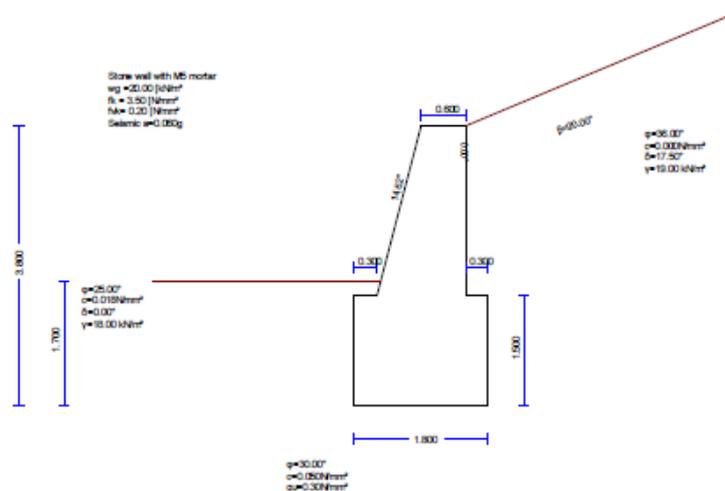
ANALIZA OPTEREĆENJA

Profil razmatra kritični presjek "potpornog zida 2" koji se izvodi na poziciji iznad pristupne cesti u sanaciji i pridržava preostalo klizno tijelo nakon odstranjivanja veće njegove mase strojnim iskopom. Dodatna opterećenja na predmetni zid ostvaruju se većim nagibom terena iza zida, te su ista uzeta u obzir. Analiza će se provesti sistemom eurocode, projektnim pristupom 3, koji podrazumijeva faktorizacija svojstva materijala ϕ i c sa faktorima 1.25. Parcijalni faktori otpora i povoljna djelovanja se ne faktoriziraju, dok će se trajna nepovoljna djelovanja faktorizirati sa 1.35.

.. ZID 2

Gravity retaining wall

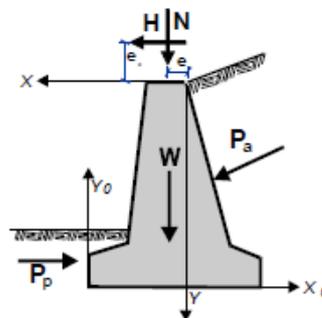
(EC2 EN1992-1-1:2004, EC0 EN1990:2002, EC7 EN1997-1-1:2004, EC8 EN1998-5:2004,)



..1. Wall properties-Parameters-Code requirements

Dimensions

Height of wall	$h = 3.800 \text{ m}$
Transverse length of wall	$L = 10.000 \text{ m}$
Stem thickness at top	$B1 = 0.600 \text{ m}$
Stem thickness at bottom	$B2 = 1.200 \text{ m}$
Width of wall base	$B = 1.800 \text{ m}$
Width of wall toe	0.300 m
Width of wall heel	0.300 m
Height of wall stem	2.300 m
Thickness of wall footing	1.500 m
Front thickness of wall toe	1.500 m
Back thickness of wall heel	1.500 m
Slope (batter) at frontface	$14.621^\circ (1:3.83)$
Slope (batter) at backface	$0.000^\circ (0:1)$



Weight of wall

Unit weight of wall material	$\gamma_g = 20.000 \text{ kN/m}^3$
Cross section area of wall	$A = 4.770 \text{ m}^2$
Self weight per meter of wall	$W = 4.770 \times 20.000 = 95.40 \text{ kN/m}$
Center of gravity of wall at	$x = 0.542 \text{ m}, y = 2.281 \text{ m} (x_0 = 0.958 \text{ m}, y_0 = 1.519 \text{ m})$



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Pg.2

Wall materials

Compressive strength 3.50 N/mm²
Shear strength 0.20 N/mm²

Weight of backfill

Weight of backfill per meter Ws=13.42 kN/m
Center of gravity of backfill x=-0.151 m, y=1.122 m

1.2. Partial factors for actions and soil properties

(EC7 Tab. A.1-A.4, EC8-5 §3.1)

Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)

Actions			(EQU)	(STR)	(GEO)	(Seismic)
Permanent	Unfavourable	γ_{Gdst}	1.10	1.25	1.35	1.00
	Favourable	γ_{Gstb}	0.90	1.00	1.00	1.00
	Variable Unfavourable	γ_{Qdst}	1.50	1.50	1.50	1.00
	Variable Favourable	γ_{Qstb}	0.00	0.00	0.00	0.00
Soil parameters	Angle of shearing resistance	γ_{ϕ}	1.25	1.25	1.25	1.25
	Effective cohesion	γ_c	1.25	1.25	1.25	1.25
	Undrained shear strength	γ_{cu}	1.40	1.40	1.40	1.40
	Unconfined strength	γ_{qu}	1.40	1.40	1.40	1.40
	Weight density	γ_w	1.00	1.00	1.00	1.00

1.3. Properties of foundation soil

Bearing capacity of foundation soil $q_u=0.30 \text{ N/mm}^2$
Friction angle between wall footing and soil $\phi=30.00^\circ$, friction coefficient $\tan(\phi)=0.577$
Cohesion between wall footing and soil $c=0.050 \text{ N/mm}^2$

1.4. Seismic coefficients

(EC8 EN1998-5:2004, §7.3.2)

Design ground acceleration ratio $g_h=a_{xg}$, $a=0.06$ (EC8-5 §7.3.2)
Soil factor $S=1.00$ (EC8 §3.2.2.2)
Importance factor $\gamma_I=1.00$ (EC8 §3.2.1, T.4.3)
Reduction factor for seismic coefficient $r=1.50$ (EC8-5 Table 7.1)
Coefficient for horizontal seismic force $k_h=1.00 \times 0.06 \times 1.00 / 1.500 = 0.040$ (EC8-5 Eq.7.1)
Coefficient for vertical seismic force $k_v=0.50 \times 0.040 = 0.020$ (EC8-5 Eq.7.2)

Forces due to seismic load (except from earth pressure)

Horizontal seismic force due to self weight $F_{wx} = 95.40 \times 0.040 = 3.82 \text{ kN/m}$
Vertical seismic force due to self weight $F_{wy} = 95.40 \times 0.020 = 1.91 \text{ kN/m}$
Horizontal seismic force of backfill $F_{wsx} = 13.42 \times 0.040 = 0.54 \text{ kN/m}$
Vertical seismic force of backfill $F_{wsy} = 13.42 \times 0.020 = 0.27 \text{ kN/m}$

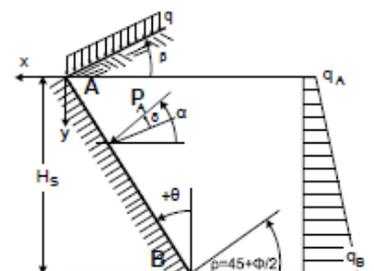
1.5. Computation of active earth pressure (Coulomb theory)

1.5.1. Wall part from $y=0.000 \text{ m}$ to $y=3.800 \text{ m}$, $H_s=3.800 \text{ m}$

Top point A $x=0.000 \text{ m}$ $y=0.000 \text{ m}$
Bottom point B $x=0.000 \text{ m}$ $y=3.800 \text{ m}$

Soil properties

Soil type : Dense sand
Unit weight of soil $\gamma = 19.00 \text{ kN/m}^3$
Unit weight of soil (saturated) $\gamma_s = 19.00 \text{ kN/m}^3$
Unit weight of water $\gamma_w = 10.00 \text{ kN/m}^3$
Angle of shearing resistance of ground $\phi = 36.00^\circ$
Cohesion of ground $c = 0.000 \text{ N/mm}^2$
Slope angle of ground surface $\beta = 20.00^\circ$
Inclination angle of the wall backface $\theta = 0.00^\circ$
Angle of shear resist. between ground-wall $\delta = 17.50^\circ$





Project Beton

Pg. 3

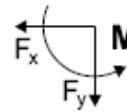
Earth pressure according to Coulomb theory

	EQU	STR	GEO
Angle of rupture plane $\rho=45^\circ+\varphi/2$	= 59.40	59.40	59.40°
Coefficient of active earth pressure K_a	= 0.442	0.442	0.442
Earth pressure $q(y)=q_A+\gamma \cdot y \cdot K_a$			

$$K_A = \frac{\cos^2(\varphi-\theta)}{\cos^2\theta \cos(\theta+\delta) \left[1 + \frac{\sin(\varphi+\delta)\sin(\varphi-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)} \right]^2}$$

Permanent actions

	EQU	STR	GEO
Earth pressure at the top ($y=y_A$)	$q_A = 0.00$	0.00	0.00 kN/m^2
Earth pressure at the bottom ($y=y_A+3.80\text{m}$)	$q_B = 31.91$	31.91	31.91 kN/m^2
Earth force $F_a = \frac{1}{2}(q_A+q_B)H$	$F_a = 60.63$	60.63	60.63 kN/m
Angle of earth force	$\alpha = 14.00$	14.00	14.00°
Earth force in x direction	$F_{ax} = 57.82$	57.82	57.82 kN/m
Earth force in y direction	$F_{ay} = 18.23$	18.23	18.23 kN/m
Moment of earth force at top point ($x=0, y=0$)	$M = -146.46$	-146.46	-146.46 kNm/m
Point of application of earth force $x = 0.000 \text{ m}$, $y = 2.533 \text{ m}$			

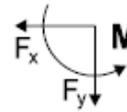


Total forces and moments

Forces and moments at bottom point B ($x=0.000 \text{ m}$, $y=3.800 \text{ m}$)

Permanent actions

	EQU	STR	GEO
Total horizontal earth force F_{ax}	57.82	57.82	57.82 kN/m
Total vertical earth force F_{ay}	18.23	18.23	18.23 kN/m
Total moment of earth force M_a	73.26	73.26	73.26 kNm/m



Seismic loading

(EC8 EN1998-1-1:2004, §7.3.2, Annex E)

Horizontal seismic coefficient $k_h=1.00 \times 0.06 \times 1.00/1.500=0.040$ (EC8 Eq.7.1, T.7.1)
Vertical seismic coefficient $k_v=0.50 \times 0.040=0.020$ (EC8 Eq.7.2)
Soil above the water table (EC8 Annex E.5)
 $\tan(\omega)=k_h/(1-k_v)=0.040/(1-0.020)=0.041$, $\omega=2.34^\circ$

Method Mononobe-Okabe (EC8 Annex E.4)

for active earth force during seismic loading

Coefficient of active earth pressure, $K_{e^*}=0.489$

Additional earth pressure due to seismic load

over STR load case $\xi=(K_{e^*}/K_e-1)=(0.489/0.442-1)=0.106$

$$K_E = \frac{\cos^2(\varphi-\omega-\theta)}{\cos\omega \cos^2\theta \cos(\theta+\omega) \left[1 + \frac{\sin(\varphi+\delta)\sin(\varphi-\beta)}{\cos(\theta+\omega+\delta)\cos(\theta-\beta)} \right]^2}$$

Earth force due to seismic load (Permanent actions) $F_x=1.106 \times 57.82=63.95 \text{ kN/m}$

1.6. Computation of passive earth pressure (Rankine theory)

1.6.1. Wall part from $y=2.100 \text{ m}$ to $y=3.800 \text{ m}$, $H_s=1.700 \text{ m}$

Top point A $x=1.500 \text{ m}$, $y=2.100 \text{ m}$

Bottom point B $x=1.500 \text{ m}$, $y=3.800 \text{ m}$

Soil properties

Soil type : Dense sand

Unit weight of soil $\gamma = 18.00 \text{ kN/m}^3$

Unit weight of soil (saturated) $\gamma_s = 18.00 \text{ kN/m}^3$

Unit weight of water $\gamma_w = 10.00 \text{ kN/m}^3$

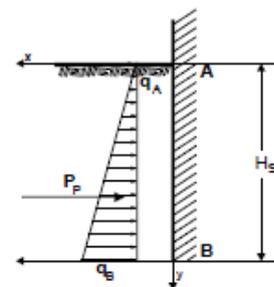
Angle of shearing resistance of ground $\varphi = 25.00^\circ$

Cohesion of ground $c = 0.018 \text{ N/mm}^2$

Slope angle of ground surface $\beta = 0.00^\circ$

Earth pressure on vertical surface $\theta = 0.00^\circ$

Angle of shear resist. between ground-wall $\delta = 0.00^\circ$





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Fg. 4

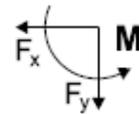
Earth pressure according to Coulomb theory

	EQU	STR	GEO
Angle of rupture plane $\rho=45^\circ-\varphi/2$	= 35.00	35.00	35.00
Coefficient of passive earth pressure K_p	2.040	2.040	2.040
Earth pressure $q(y)=q_A+y \cdot K_p$			

$$K_p = \frac{\cos^2(\varphi+\theta)}{\cos^2\theta \cos(\theta-\delta)} \left[1 - \frac{\sin(\varphi+\delta)\sin(\varphi+\beta)}{\cos(\theta-\delta)\cos(\theta-\beta)} \right]^2$$

Permanent actions

	EQU	STR	GEO
Earth pressure at the top ($y=y_A$)	$q_A= 0.00$	0.00	0.00 kN/m ²
Earth pressure at the bottom ($y=y_A+ 1.70$ m)	$q_B=-62.42$	-62.42	-62.42 kN/m ²
Earth force $P_a= \frac{1}{2}(q_A+q_B)H$	$P_p= 53.06$	53.06	53.06 kN/m
Angle of earth force	$\alpha = 0.00$	0.00	0.00
Earth force in x direction	$F_{px}=-53.06$	-53.06	-53.06 kN/m
Earth force in y direction	$F_{py}= 0.00$	0.00	0.00 kN/m
Moment of earth force at top point ($x=0, y=0$)	$M = 171.54$	171.54	171.54 kNm/m
Point of application of earth force $x= 1.500$ m, $y= 3.233$ m			

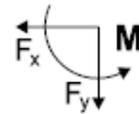


Total forces and moments

Forces and moments at bottom point B ($x=1.500$ m, $y=3.800$ m)

Permanent actions

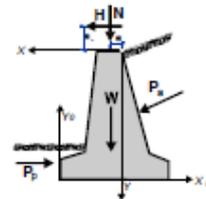
	EQU	STR	GEO
Total horizontal earth force F_{sx}	-53.06	-53.06	-53.06 kN/m
Total vertical earth force F_{sy}	0.00	0.00	0.00 kN/m
Total moment of earth force M_s	-30.09	-30.09	-30.09 kNm/m



1.7. Checks of wall stability (EQU)

1.7.1. Forces (driving and resisting) on the wall (EQU)

Action	$y_1 - y_2$	F_x [kN/m]	F_y [kN/m]	x [m]	y [m]
Active earth pressure	P_a 0.00- 3.80	57.82	18.23	0.000	2.533
Passive earth pressure	P_p 2.10- 3.80	-53.06	0.00	1.500	3.233
Wall weight	W	0.00	95.40	0.542	2.261
Backfill weight	W_s	0.00	13.42	-0.151	1.122

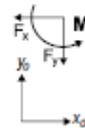


1.7.2. Check of soil bearing capacity (EQU)

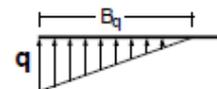
(EC7 EN1997-1-1:2004, §6.5.2)

Check for $0.90x(\text{self weight}+\text{top vertical dead load})+0.00x(\text{top vertical live load})$

Action	(P, y)	$y_1 - y_2$	F_x [kN/m]	F_y [kN/m]	x_0 [m]	y_0 [m]	M [kNm/m]
Active earth pressure	$P_{ax}1.10$	0.00- 3.80	63.60	20.05	1.500	1.267	50.51
Wall weight	$W \times 0.90$		0.00	85.86	0.958	1.519	-82.25
Backfill weight	$W_s \times 0.90$		0.00	12.08	1.651	2.678	-19.94
			Sum=	117.99			-51.68



Sum of vertical forces = 117.99 kN/m
Sum of moments at front toe = -51.68 kNm/m
Sum of moments at middle of base = 54.51 kNm/m
Eccentricity $ec=54.51/117.99=0.462$ m, $ec>1.800/6=0.300$ m
Soil pressure $q=0.180$ N/mm² $B_q=1.314$ m
Effective footing $L'=1.800-2 \times 0.462= 0.876$ m
Soil bearing capacity $R_d=L' \cdot q_u/\gamma M=0.876 \times (1000 \times 0.30) / 1.40= 187.71$ kN/m
Bearing resistance check $V_d=117.99 < R_d=187.71$ kN/m, Is verified



(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)



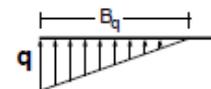
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Pg. 5

Check for $1.10 \times (\text{self weight} + \text{top vertical dead load}) + 1.50 \times (\text{top vertical live load})$

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]
Active earth pressure	Pax1.10	0.00- 3.80	63.60	20.05	1.500	1.267	50.51
Wall weight	W x1.10		0.00	104.94	0.958	1.519	-100.53
Backfill weight	Wsx1.10		0.00	14.76	1.651	2.678	-24.38
			Sum=	139.75			-74.40

Sum of vertical forces = 139.75 kN/m
Sum of moments at front toe = -74.40 kNm/m
Sum of moments at middle of base = 51.37 kNm/m
Eccentricity $ec = 51.37/139.75 = 0.368\text{m}$, $ec > 1.800/6 = 0.300\text{m}$
Soil pressure $q = 0.175\text{ N/mm}^2$ $Bq = 1.897\text{ m}$
Effective footing $L' = 1.800 - 2 \times 0.368 = 1.065\text{ m}$
Soil bearing capacity $Rd = L' \cdot qu / \gamma M = 1.065 \times (1000 \times 0.30) / 1.40 = 228.21\text{ kN/m}$
Bearing resistance check $Vd = 139.75 < Rd = 228.21\text{ kN/m}$, Is verified



(EC7 Annex D)

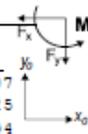
(EC7 Eq.2.2, Eq.6.1)

1.7.3. Failure check due to overturning (EQU)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe ($x_0=0, y_0=0$) ($x=1.500, y=3.800\text{ m}$)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.10	0.00- 3.80	63.60	20.05	1.500	1.267	80.59	30.07
Wall weight	W x0.90		0.00	85.86	0.958	1.519	0.00	82.25
Backfill weight	Wsx0.90		0.00	12.08	1.651	2.678	0.00	19.94
			Sum=				80.59	132.26

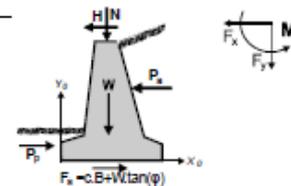


Sum of overturning moments = 80.59 kNm/m
Sum of moments resisting overturning = 132.26 kNm/m
Overturning check $Med = 80.59 < Mrd = 132.26\text{ kNm/m}$, Is verified

1.7.4. Failure check against sliding (EQU)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(P,y)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]	
Active earth pressure	Pax1.10	0.00- 3.80	63.60	0.00	20.05	
Passive earth pressure	Ppx0.90	2.10- 3.80	0.00	0.00	47.75	
Wall weight	W x0.90		0.00	0.00	85.86	
Backfill weight	Wsx0.90		0.00	0.00	12.08	
			Sum=	63.60	47.75	117.99



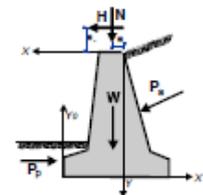
Soil friction $Rd = Vd \cdot \tan\phi / \gamma M = 117.99 \times \tan(30.00^\circ) / 1.25 = 54.50\text{ kN/m}$
Soil cohesion $Rd = A \cdot cu / \gamma M = 1000 \times 1.314 \times 0.050 / 1.25 = 52.56\text{ kN/m}$
(resisting forces from effective cohesion are neglected)
Sum of driving forces = 63.60 kN/m
Sum of resisting forces (47.75+54.50) = 102.25 kN/m
Sliding resistance check $Hd = 63.60 < Rd = 102.25\text{ kN/m}$, Is verified

(EC7 §6.5.3. 10)

1.8. Checks of wall stability (STR)

1.8.1. Forces (driving and resisting) on the wall (STR)

Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 3.80	57.82	18.23	0.000	2.523
Passive earth pressure	Pp	2.10- 3.80	-53.06	0.00	1.500	3.233
Wall weight	W		0.00	95.40	0.542	2.281
Backfill weight	Ws		0.00	13.42	-0.161	1.122





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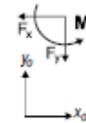
Pg. 6

1.8.2. Check of soil bearing capacity (STR)

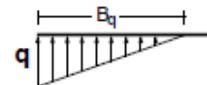
(EC7 EN1997-1-1:2004, §6.5.2)

Check for 1.00x(self weight+top vertical dead load)+0.00x(top vertical live load)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pa _x 1.35	0.00- 3.80	78.06	24.61	1.500	1.267	61.99
Wall weight	W x1.00		0.00	95.40	0.958	1.519	-91.39
Backfill weight	Ws _x 1.00		0.00	13.42	1.651	2.678	-22.16
			Sum=	133.43			-51.56



Sum of vertical forces = 133.43 kN/m
Sum of moments at front toe = -51.56 kNm/m
Sum of moments at middle of base = 68.53 kNm/m
Eccentricity $ec=68.53/133.43=0.514m$, $ec>1.800/6=0.300m$
Soil pressure $q=0.230 N/mm^2$ $Bq=1.159 m$
Effective footing $L'=1.800-2x0.514= 0.773 m$
Soil bearing capacity $Rd=L' \cdot qu/\gamma M=0.773x(1000x0.30)/1.40= 165.64 kN/m$
Bearing resistance check $Vd=133.43 < Rd=165.64 kN/m$, Is verified

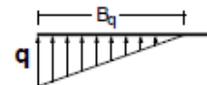


(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

Check for 1.35x(self weight+top vertical dead load)+1.50x(top vertical live load)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pa _x 1.35	0.00- 3.80	78.06	24.61	1.500	1.267	61.99
Wall weight	W x1.35		0.00	128.79	0.958	1.519	-123.38
Backfill weight	Ws _x 1.35		0.00	18.12	1.651	2.678	-29.92
			Sum=	171.52			-91.31



(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

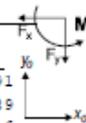
Sum of vertical forces = 171.52 kN/m
Sum of moments at front toe = -91.31 kNm/m
Sum of moments at middle of base = 63.06 kNm/m
Eccentricity $ec=63.06/171.52=0.368m$, $ec>1.800/6=0.300m$
Soil pressure $q=0.215 N/mm^2$ $Bq=1.597 m$
Effective footing $L'=1.800-2x0.368= 1.065 m$
Soil bearing capacity $Rd=L' \cdot qu/\gamma M=1.065x(1000x0.30)/1.40= 228.21 kN/m$
Bearing resistance check $Vd=171.52 < Rd=228.21 kN/m$, Is verified

1.8.3. Failure check due to overturning (STR)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe ($xo=0, yo=0$) ($x=1.500, y=3.800 m$)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pa _x 1.35	0.00- 3.80	78.06	24.61	1.500	1.267	98.90	36.91
Wall weight	W x1.00		0.00	95.40	0.958	1.519	0.00	91.39
Backfill weight	Ws _x 1.00		0.00	13.42	1.651	2.678	0.00	22.16
			Sum=				98.90	150.46

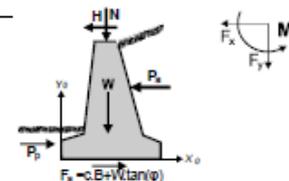


Sum of overturning moments = 98.90 kNm/m
Sum of moments resisting overturning = 150.46 kNm/m
Overturning check $Med=98.90 < Mrd=150.46 kNm/m$, Is verified

1.8.4. Failure check against sliding (STR)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(P,y)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pa _x 1.35	0.00- 3.80	78.06	0.00	24.61
Passive earth pressure	Pp _x 1.00	2.10- 3.80	0.00	53.06	0.00
Wall weight	W x1.00		0.00	0.00	95.40
Backfill weight	Ws _x 1.00		0.00	0.00	13.42
			Sum=	78.06	53.06





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Pg. 7

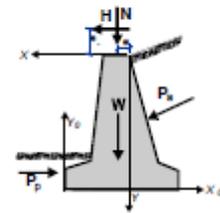
Soil friction $R_d = V_d \cdot \tan \phi / \gamma M = 133.43 \tan(30.00^\circ) / 1.25 = 61.63 \text{ kN/m}$
 Soil cohesion $R_d = A \cdot c_u / \gamma M = 1000 \times 1.159 \times 0.050 / 1.25 = 46.37 \text{ kN/m}$
 (resisting forces from effective cohesion are neglected)
 Sum of driving forces = 78.06 kN/m
 Sum of resisting forces (53.06+61.63) = 114.69 kN/m
 Sliding resistance check $H_d = 78.06 < R_d = 114.69 \text{ kN/m}$, Is verified

(EC7 §6.5.3. 10)

1.9. Checks of wall stability (GEO)

1.9.1. Forces (driving and resisting) on the wall (GEO)

Action		$y_1 - y_2$	F_x [kN/m]	F_y [kN/m]	x [m]	y [m]
Active earth pressure	P_a	0.00- 3.80	57.82	18.23	0.000	2.533
Passive earth pressure	P_p	2.10- 3.80	-53.06	0.00	1.500	3.233
Wall weight	W		0.00	95.40	0.542	2.281
Backfill weight	W_s		0.00	13.42	-0.161	1.122

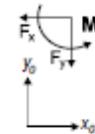


1.9.2. Check of soil bearing capacity (GEO)

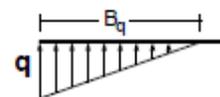
(EC7 EN1997-1-1:2004, §6.5.2)

Check for $1.00 \times (\text{self weight} + \text{top vertical dead load}) + 0.00 \times (\text{top vertical live load})$

Action	(P,y)	$y_1 - y_2$	F_x [kN/m]	F_y [kN/m]	x_0 [m]	y_0 [m]	M [kNm/m]
Active earth pressure	$P_{ax} 1.35$	0.00- 3.80	78.06	24.61	1.500	1.267	61.99
Wall weight	$W \times 1.00$		0.00	95.40	0.958	1.519	-91.39
Backfill weight	$W_{sx} 1.00$		0.00	13.42	1.651	2.678	-22.16
			Sum=	133.43			-51.56



Sum of vertical forces = 133.43 kN/m
 Sum of moments at front toe = -51.56 kNm/m
 Sum of moments at middle of base = 68.53 kNm/m
 Eccentricity $ec = 68.53 / 133.43 = 0.514 \text{ m}$, $ec > 1.800 / 6 = 0.300 \text{ m}$
 Soil pressure $q = 0.230 \text{ N/mm}^2$ $B_q = 1.159 \text{ m}$
 Effective footing $L' = 1.800 - 2 \times 0.514 = 0.773 \text{ m}$
 Soil bearing capacity $R_d = L' \cdot q_u / \gamma M = 0.773 \times (1000 \times 0.30) / 1.40 = 165.64 \text{ kN/m}$
 Bearing resistance check $V_d = 133.43 < R_d = 165.64 \text{ kN/m}$, Is verified



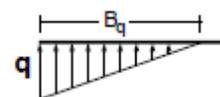
(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

Check for $1.35 \times (\text{self weight} + \text{top vertical dead load}) + 1.50 \times (\text{top vertical live load})$

Action	(P,y)	$y_1 - y_2$	F_x [kN/m]	F_y [kN/m]	x_0 [m]	y_0 [m]	M [kNm/m]
Active earth pressure	$P_{ax} 1.35$	0.00- 3.80	78.06	24.61	1.500	1.267	61.99
Wall weight	$W \times 1.35$		0.00	128.79	0.958	1.519	-123.38
Backfill weight	$W_{sx} 1.35$		0.00	18.12	1.651	2.678	-29.92
			Sum=	171.52			-91.31

Sum of vertical forces = 171.52 kN/m
 Sum of moments at front toe = -91.31 kNm/m
 Sum of moments at middle of base = 63.06 kNm/m
 Eccentricity $ec = 63.06 / 171.52 = 0.368 \text{ m}$, $ec > 1.800 / 6 = 0.300 \text{ m}$
 Soil pressure $q = 0.215 \text{ N/mm}^2$ $B_q = 1.597 \text{ m}$
 Effective footing $L' = 1.800 - 2 \times 0.368 = 1.065 \text{ m}$
 Soil bearing capacity $R_d = L' \cdot q_u / \gamma M = 1.065 \times (1000 \times 0.30) / 1.40 = 228.21 \text{ kN/m}$
 Bearing resistance check $V_d = 171.52 < R_d = 228.21 \text{ kN/m}$, Is verified



(EC7 Annex D)

(EC7 Eq.2.2, Eq.6.1)

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Fig. 8

1.9.3. Failure check due to overturning (GEO)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe ($x_0=0, y_0=0$) ($x=1.500, y=3.800$ m)

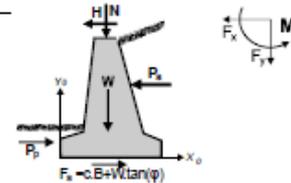
Action	(P,γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.35	0.00- 3.80	78.06	24.61	1.500	1.267	98.90	36.91
Wall weight	W x1.00		0.00	95.40	0.958	1.519	0.00	91.39
Backfill weight	Wsx1.00		0.00	13.42	1.651	2.678	0.00	22.16
Sum=							98.90	150.46

Sum of overturning moments = 98.90 kNm/m
Sum of moments resisting overturning = 150.46 kNm/m
Overturning check $M_{ed}=98.90 < M_{rd}=150.46$ kNm/m, Is verified

1.9.4. Failure check against sliding (GEO)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(P,γ)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.35	0.00- 3.80	78.06	0.00	24.61
Passive earth pressure	Ppx1.00	2.10- 3.80	0.00	53.06	0.00
Wall weight	W x1.00		0.00	0.00	95.40
Backfill weight	Wsx1.00		0.00	0.00	13.42
Sum=			78.06	53.06	133.43



Soil friction $R_d = V_d \cdot \tan(\phi/M) = 133.43 \cdot \tan(30.00^\circ) / 1.25 = 61.63$ kN/m
Soil cohesion $R_d = A \cdot c_u / \gamma M = 1000 \cdot 1.159 \cdot 0.050 / 1.25 = 46.37$ kN/m
(resisting forces from effective cohesion are neglected)
Sum of driving forces = 78.06 kN/m
Sum of resisting forces (53.06+61.63) = 114.69 kN/m
Sliding resistance check $H_d=78.06 < R_d=114.69$ kN/m, Is verified

(EC7 §6.5.3. 10)

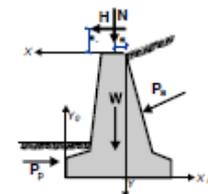
1.10. Seismic design

(EC8 EN1998-1-1:2004)

Checks of wall stability (with seismic loading)

1.10.1. Forces (driving and resisting) on the wall

Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 3.80	57.82	18.23	0.000	2.533
Passive earth pressure	Pp	2.10- 3.80	-53.06	0.00	1.500	3.233
Wall weight	W		0.00	95.40	0.542	2.281
Backfill weight	Ws		0.00	13.42	-0.151	1.122



1.10.2. Additional forces due to seismic load

Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 3.80	6.13	0.000	0.000	2.533
Wall weight	W		3.82	-1.91	0.542	2.281
Backfill weight	Ws		0.54	-0.27	-0.151	1.122

1.10.3. Check of soil bearing capacity (with seismic loading)

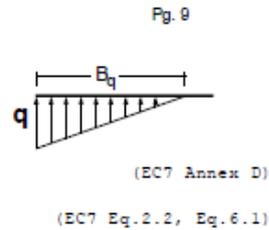
(EC7 §6.5.2)

Action	(P,γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	M [kNm/m]	
Active earth pressure	Pax1.00	0.00- 3.80	63.95	18.23	1.500	1.267	53.69	
Wall weight	W x1.00		3.82	97.31	0.958	1.519	-83.76	
Backfill weight	Wsx1.00		0.54	13.69	1.651	2.678	-20.27	
Sum=							129.23	-50.34



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Sum of vertical forces = 129.23 kN/m
Sum of moments at front toe = -50.34 kNm/m
Sum of moments at middle of base = 65.97 kNm/m
Eccentricity $ec=65.97/129.23=0.510m$, $ec>1.800/6=0.300m$
Soil pressure $q=0.221 N/mm^2$ $Bq=1.169 m$
Effective footing $L'=1.800-2x0.510= 0.779 m$
Soil bearing capacity $Rd=L' \cdot qu/\gamma M=0.779x(1000x0.30)/1.40= 166.93 kN/m$
Bearing resistance check $Vd=129.23 < Rd=166.93 kN/m$, Is verified



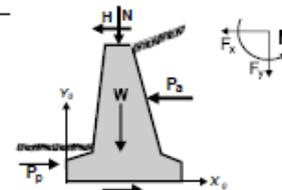
1.10.4. Failure check due to overturning (with seismic loading) (EC7 §9.7.4)
Overturning with respect to the toe ($x_0=0, y_0=0$) ($x=1.500, y=3.800 m$)

Action	(P,y)	y1 - y2	Fx [kN/m]	Fy [kN/m]	x0 [m]	y0 [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.00	0.00- 3.80	63.95	18.23	1.500	1.267	81.03	27.34
Wall weight	W x1.00		3.82	97.31	0.958	1.519	7.63	91.39*
Backfill weight	Wsx1.00		0.54	13.69	1.651	2.678	1.89	22.16*
Sum=							90.55	140.89

(*moments of negative seismic vertical loads, are added to the overturning moments)
Sum of overturning moments = 90.55 kNm/m
Sum of moments resisting overturning = 140.89 kNm/m
Overturning check $M_{ed}=90.55 < M_{rd}=140.89 kNm/m$, Is verified

1.10.5. Failure check against sliding (with seismic loading) (EC7 §9.7.3, §6.5.3)

Action	(P,y)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.00	0.00- 3.80	63.95	0.00	18.23
Passive earth pressure	Ppx1.00	2.10- 3.80	0.00	53.06	0.00
Wall weight	W x1.00		3.82	0.00	93.49
Backfill weight	Wsx1.00		0.54	0.00	13.15
Sum=			68.31	53.06	124.87



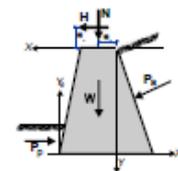
Soil friction $Rd=Vd \cdot \tan\phi/\gamma M= 124.87x\tan(30.00^\circ)/1.25= 57.67 kN/m$
Soil cohesion $Rd=A \cdot cu/\gamma M= 1000x1.159x0.050/1.25= 46.37 kN/m$
(resisting forces from effective cohesion are neglected)
Sum of driving forces = 68.31 kN/m
Sum of resisting forces ($53.06+57.67$) = 110.73 kN/m
Sliding resistance check $Hd=68.31 < Rd=110.73 kN/m$, Is verified

1.11. Design of wall stem

1.11.1. Loading 1.35x(permanent unfavourable)+1.00x(permanent favourable)+1.50x(variable unfav.)

Forces (at centroid of cross section) and stresses at wall stem
x, y: cross section centroid, b: cross section width, e: eccentricity
Fx: horizontal force, Fy: vertical force, M: moment, e/b: relative eccentricity
 σ_1, σ_2, τ : cross section normal and shear stress, Bq: effective cross section width

y	x	b	Fx	Fy	M	e/b	σ_1	σ_2	Bq/B	τ
[m]	[m]	[m]	[kN/m]	[kN/m]	[kNm/m]		[N/mm ²]	[N/mm ²]		[N/mm ²]
0.46	0.360	0.720	1.15	6.44	-0.13	0.029	-0.007	-0.010	1.000	0.002
0.92	0.420	0.840	4.58	14.69	0.05	-0.004	-0.018	-0.017	1.000	0.005
1.38	0.480	0.960	10.30	24.78	1.39	-0.058	-0.035	-0.017	1.000	0.011
1.84	0.540	1.080	18.31	36.69	4.74	-0.120	-0.058	-0.010	1.000	0.017
2.30	0.600	1.200	28.59	50.42	11.01	-0.182	-0.088	0.000	0.954	0.024





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Fig. 10

1.11.2. Strength check according to EC6 EN1996-1-1:2005

Strength check in normal stresses $N_{ed} \leq N_{rd}$

(EC6 §6.1)

Vertical resistance load $N_{rd} = \eta \cdot f_k \cdot t / \gamma_M$, Vertical design load N_{ed}

(EC6 §6.1.2)

$\eta = 1 - 2e/t$, η capacity reduction factor for slenderness and eccentricity of loading

$e = \text{load eccentricity} + e_s$, $e_s = \text{accidental eccentricity} = h/450$, $h = \text{wall height}$

f_k characteristic compressive strength $f_k = 3.50 \text{ N/mm}^2$

$\gamma_M = 2.50$, γ_M partial safety factor for the material

y	t	Fy	M	e/t	η	Ned	Nrd	
[m]	[m]	[kN/m]	[kNm/m]			[kN/m]	[kN/m]	
0.46	0.720	6.44	-0.13	0.030	0.940	6.44	947.52	(Ned ≤ Nrd)
0.92	0.840	14.69	0.05	0.006	0.988	14.69	1161.89	(Ned ≤ Nrd)
1.38	0.960	24.78	1.39	0.061	0.878	24.78	1180.03	(Ned ≤ Nrd)
1.84	1.080	36.69	4.74	0.124	0.752	36.69	1137.02	(Ned ≤ Nrd)
2.30	1.200	50.42	11.01	0.186	0.628	50.42	1055.04	(Ned ≤ Nrd)

Design for shear strength $V_{ed} \leq V_{rd}$

(EC6 §6.2.1)

Shear resistance $V_{rd} = f_{vk} \cdot t / \gamma_M$, design shear load V_{ed}

(EC6 §6.2.1)

$f_{vk} = f_{vko} + 0.40 \cdot \sigma_d$, σ_d design compressive stress

(EC6 §3.6.2)

f_{vko} shear strength under zero compressive stress $f_{vko} = 0.20 \text{ N/mm}^2$

$\gamma_M = 2.50$, γ_M partial safety factor for the material

y	t	Fx	σ_d	Ved	Vrd
[m]	[m]	[kN/m]	[N/mm ²]	[kN/m]	[kN/m]
0.46	0.720	1.15	0.009	1.15	58.64 (Ved ≤ Vrd)
0.92	0.840	4.58	0.017	4.58	69.48 (Ved ≤ Vrd)
1.38	0.960	10.30	0.026	10.30	80.79 (Ved ≤ Vrd)
1.84	1.080	18.31	0.034	18.31	92.28 (Ved ≤ Vrd)
2.30	1.200	28.59	0.042	28.59	104.06 (Ved ≤ Vrd)

1.11.3. Loading 1.00x(permanent unfav.)+1.00x(permanent favour.)+1.00x(variable)+1.00x(seismic)

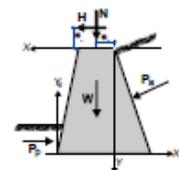
Forces (at centroid of cross section) and stresses at wall stem (with seismic loading)

x, y: cross section centroid, b: cross section width, e: eccentricity

Fx: horizontal force, Fy: vertical force, M: moment, e/b: relative eccentricity

σ_1, σ_2, τ : cross section normal and shear stress, B_q/B : effective cross section width

y	x	b	Fx	Fy	M	e/b	σ_1	σ_2	B_q/B	τ
[m]	[m]	[m]	[kN/m]	[kN/m]	[kNm/m]		[N/mm ²]	[N/mm ²]		[N/mm ²]
0.46	0.360	0.720	1.18	6.22	-0.07	0.016	-0.008	-0.010	1.000	0.002
0.92	0.420	0.840	4.28	14.05	0.20	-0.016	-0.019	-0.016	1.000	0.005
1.38	0.480	0.960	9.30	23.51	1.52	-0.064	-0.036	-0.016	1.000	0.010
1.84	0.540	1.080	16.23	34.57	4.62	-0.117	-0.058	-0.010	1.000	0.015
2.30	0.600	1.200	25.08	47.25	10.25	-0.169	-0.085	0.000	0.992	0.021



1.11.4. Strength check according to EC6 EN1996-1-1:2005 (with seismic loading)

Strength check in normal stresses $N_{ed} \leq N_{rd}$ (with seismic loading)

(EC6 §6.1)

Vertical resistance load $N_{rd} = \eta \cdot f_k \cdot t / \gamma_M$, Vertical design load N_{ed}

(EC6 §6.1.2)

$\eta = 1 - 2e/t$, η capacity reduction factor for slenderness and eccentricity of loading

$e = \text{load eccentricity} + e_s$, $e_s = \text{accidental eccentricity} = h/450$, $h = \text{wall height}$

f_k characteristic compressive strength $f_k = 3.50 \text{ N/mm}^2$

$\gamma_M = 2.50$, γ_M partial safety factor for the material



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Fig. 11

y	t	F _y	M	e/t	±	N _{ed}	N _{rd}
[m]	[m]	[kN/m]	[kNm/m]			[kN/m]	[kN/m]
0.46	0.720	6.22	-0.07	0.017	0.966	6.22	973.73 (N _{ed} ≤N _{rd})
0.92	0.840	14.05	0.20	0.018	0.964	14.05	1133.66 (N _{ed} ≤N _{rd})
1.38	0.960	23.51	1.52	0.067	0.866	23.51	1163.90 (N _{ed} ≤N _{rd})
1.84	1.080	34.57	4.62	0.121	0.758	34.57	1146.10 (N _{ed} ≤N _{rd})
2.30	1.200	47.25	10.25	0.173	0.654	47.25	1098.72 (N _{ed} ≤N _{rd})

Design for shear strength $V_{ed} \leq V_{rd}$ (with seismic loading)

(EC6 §6.2.1)

Shear resistance $V_{rd} = f_{vk} \cdot t / \gamma_M$, design shear load V_{ed}

(EC6 §6.2.1)

$f_{vk} = f_{vko} + 0.40 \sigma_{cd}$, σ_{cd} design compressive stress

(EC6 §3.6.2)

f_{vko} shear strength under zero compressive stress $f_{vko} = 0.20 \text{ N/mm}^2$

$\gamma_M = 2.50$, γ_M partial safety factor for the material

y	t	F _x	σ_{cd}	V _{ed}	V _{rd}
[m]	[m]	[kN/m]	[N/mm ²]	[kN/m]	[kN/m]
0.46	0.720	1.18	0.009	1.18	58.64 (V _{ed} ≤V _{rd})
0.92	0.840	4.28	0.017	4.28	69.48 (V _{ed} ≤V _{rd})
1.38	0.960	9.30	0.024	9.30	80.49 (V _{ed} ≤V _{rd})
1.84	1.080	16.23	0.032	16.23	91.93 (V _{ed} ≤V _{rd})
2.30	1.200	25.08	0.039	25.08	103.49 (V _{ed} ≤V _{rd})



4. PRIJEDLOG SANACIJE

Na temelju provedenih terenskih istraživanja i rezultata laboratorijskih ispitivanja daje se prijedlog sanacije klizišta na lokaciji Toplička ulica.

Prijedlog sanacije klizišta sastoji se od izvedbe potporne konstrukcije kroz dva zida kao zasebna elementa udaljenih bermom koja će služiti za formiranje pristupnog puta ranije spomenutom stambenom objektu. Potporni zid 1 izvodi se od stacionaže 0+000 do 0+028 m, u dužini 28 m' uz samu Topličku ulicu, te potporni zid 2 koji se izvodi uzbrično uz pokos od stacionaže 0+005 do 0+023, u dužini 18 m'. Na poziciji pristupnog puta bi se izvela kolnička konstrukcija, te pripadajuća drenaža iza izvedenih potpornih zidova, kako je to prikazano u prilogima 03/042/2016 te 04/042/2016.

Izrada sanacije klizišta počinje od vrha prema nožici klizišta, iskopom za izradu potpornog zida 2, prema tlocrtu u prilogu 03/042/2016, te presjeku iskopa 05/042/2016. Iskop postojećeg materijala te temeljnog tla predviđen je do dubine 3.8 m od kote postojećeg terena. Nakon izvedenog potpornog zida 2 kreće se u iskop potpornog zida 1 na poziciji uz prometnicu Na navedenoj visinskoj poziciji iskopom treba ostvariti kontakt sa čvrstom laporovitom podlogom.

Kameno betonski potporni zid 1 izvodi se u dužini 28 m', dok je dužina potpornog zida 2 18 m', širina temeljne stope oba zida jednakih je dimenzija 1.8 m, visine 1.5 m. Nadtemeljni zid 1 visine je 2.5 m dok je, nadtemeljni dio zida 2 visine 2.3 m od ruba temeljne stope. Širina nadtemelnog dijela zida je 1.2 m, dok je visina u kruni 0.6m za oba zida. Na kontaktu temeljne stope (njenog stražnjeg dijela širine 0.3 m) te kamenog zida izvodi se drenaža po cijeloj dužini obje potporne konstrukcije. Drenaža se izvodi u širini 0.3 do 0.8 (prosječno 0.5) m, te visini 1.5 m. Na postavljenu podlogu drenažnoj cijevi polaže se drenažna cijev Ø120 mm i zasipava drenažnim šljunkom Ø32-64 mm u prosječnoj širini 0.5 te visini 1.5 m. Drenažne cijevi povezuju se u dva sabirna okna SO1 i SO2, izvode se oknom pvc tipskim 800 mm, uz tipski poklopac te vodonepropusno dno. Potporna konstrukcija izvodit će se kamenom Ø10-50(60) cm u omjeru 70% kamena Ø30-50(60) cm i oko 30% kamena Ø10-30 cm. Sitnijom kamenom granulacijom popunjava se prostor između kamena krupnije granulacije radi što boljeg popunjavanja i uklještenja. Kamen u potpornoj konstrukciji povezuje se betonom C20/25. Kruna potpornog zida izvodi se u debljini 20 cm, od betona C20/25, iznad visine postojeće kote postojećeg asfaltnog sloja, bez upotrebe kamena. Vanjsko vidljivo lice nadtemelnog dijela kamenog suhozida slaže se/zida, ručno uz prethodno postavljanje profila.

Sanacija prilaznog puta stambenom objektu izvodi se prema prilogima 3/042/2016 i 4/042/2016. Kolnička konstrukcija prilaznog puta izvodi se iskopom do predviđene dubine, na koju se polaže kamen frakcije Ø 32-64 mm, debljine sloja 0.7 m, po čitavoj širini 2.0 m, nakon kojeg se polaže sloj tamponskog kamenog materijala frakcije Ø 0.1-32 mm, u punoj širini te visini sloja 0.3 m. Slojevi se strojno zbijaju. Na dobro zbijene slojeve ustroja polaže se asfaltni zastor u dva sloja. Bazni sloj predstavlja zastor AC16 base 50/70 debljine 4 cm, na koji se postavlja habajući sloj asfalta AC11 surf 50/70 u debljini 3 cm. Uz nizbrični rub asfaltiranog prilaznog puta postavljaju se beonski rubnjaci koji će pomoći usmjeravanju prikupljene oborinske vode sa asfaltnog zastora prema oborinskoj odvodnji uz prometnicu.

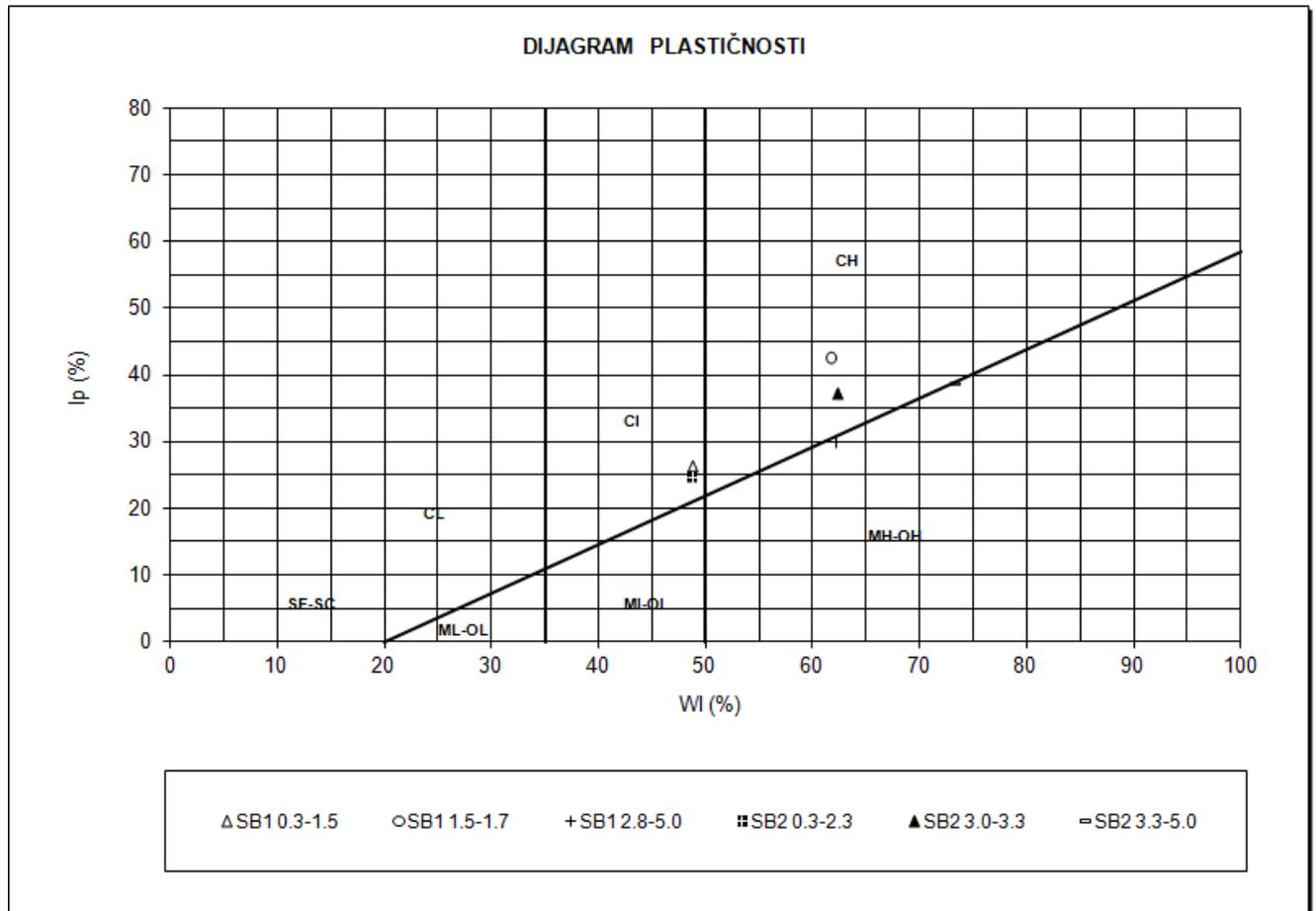
Obzirom na poziciju drenažne odvodnje, ista će se spojiti na sabirno okno SO3 uz prometnicu sa postojećim kanalizacijskom odvodnjom.

Inženjer:

Trsek Brosig, dipl.ing.građ.

5. POPIS PRILOGA

Naziv priloga	Oznaka
Tlocrt klizišta i sondažnih jama	1/042/16
Geomehanički presječni profil kroz sondažne bušotine	2/042/16
Tlocrt sanacije klizišta	3/042/16
Presjek potporno zaštitnih konstrukcija i pristupne kolničke konstrukcije	4/042/16
Poprečni presjek iskopa	5/042/16
Detalj drenaže	6/042/16
Dijagram plastičnosti	7/042/16
Dijagram smicanja	8/042/16
Dijagram jednoosne čvrstoće	9/042/16
Tabela rezultata terenskih i laboratorijskih ispitivanja	10/042/16
Fotodokumentacija istražnih radova	11/042/16 i 12/042/16
Troškovnik	troškovnik/042/16



LEGENDA: CH - Glina anorganska visoke plastičnosti
CI - Glina anorganska srednje plastičnosti
CL - Glina anorganska niske plastičnosti

MH - Tinjčasta i dijatomejska tla
MI - Prah glinovit
ML - Prah

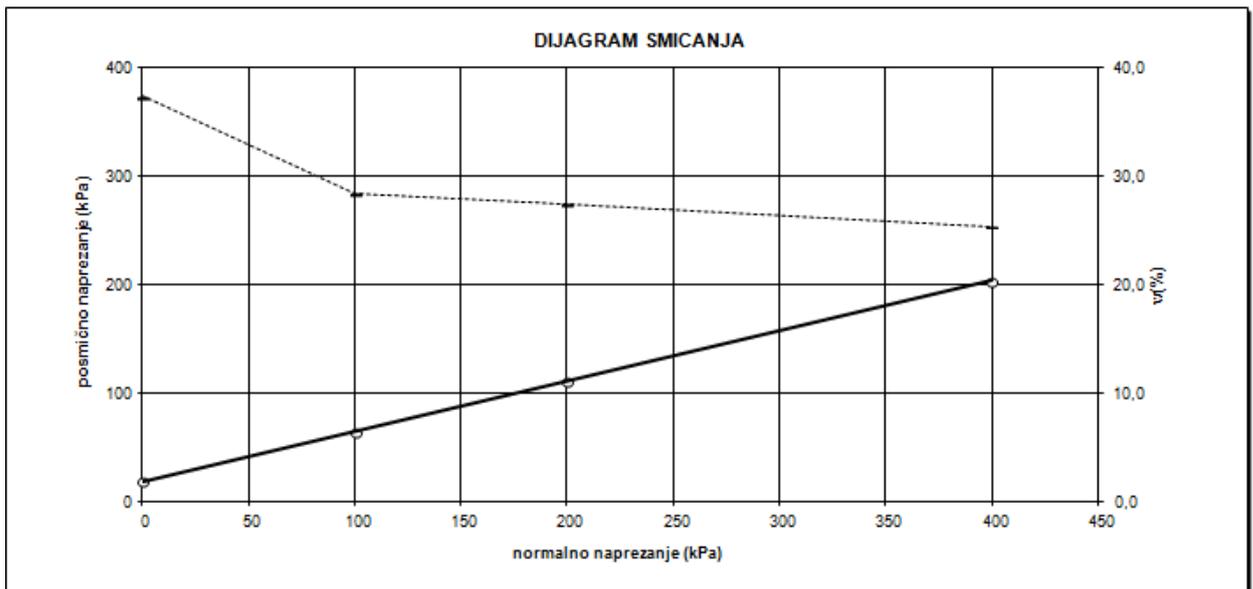
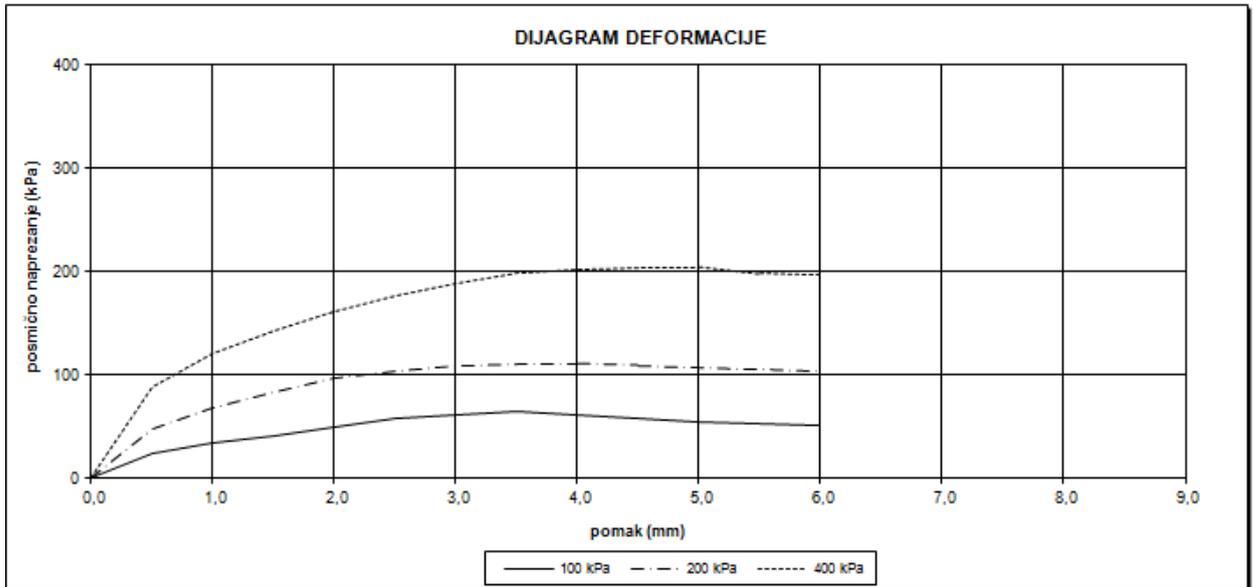
OH - Glina organska visoke plastičnosti
OI - Glina organska srednje plastičnosti
OL - Glina organska niske plastičnosti

08.11.2016.

Odgovorni inženjer: Trsek Brosig, dipl.ing.građ.

DIJAGRAM PLASTIČNOSTI
GEOMEHANIČKO MIŠLJENJE

PRILOG 7/042/2016



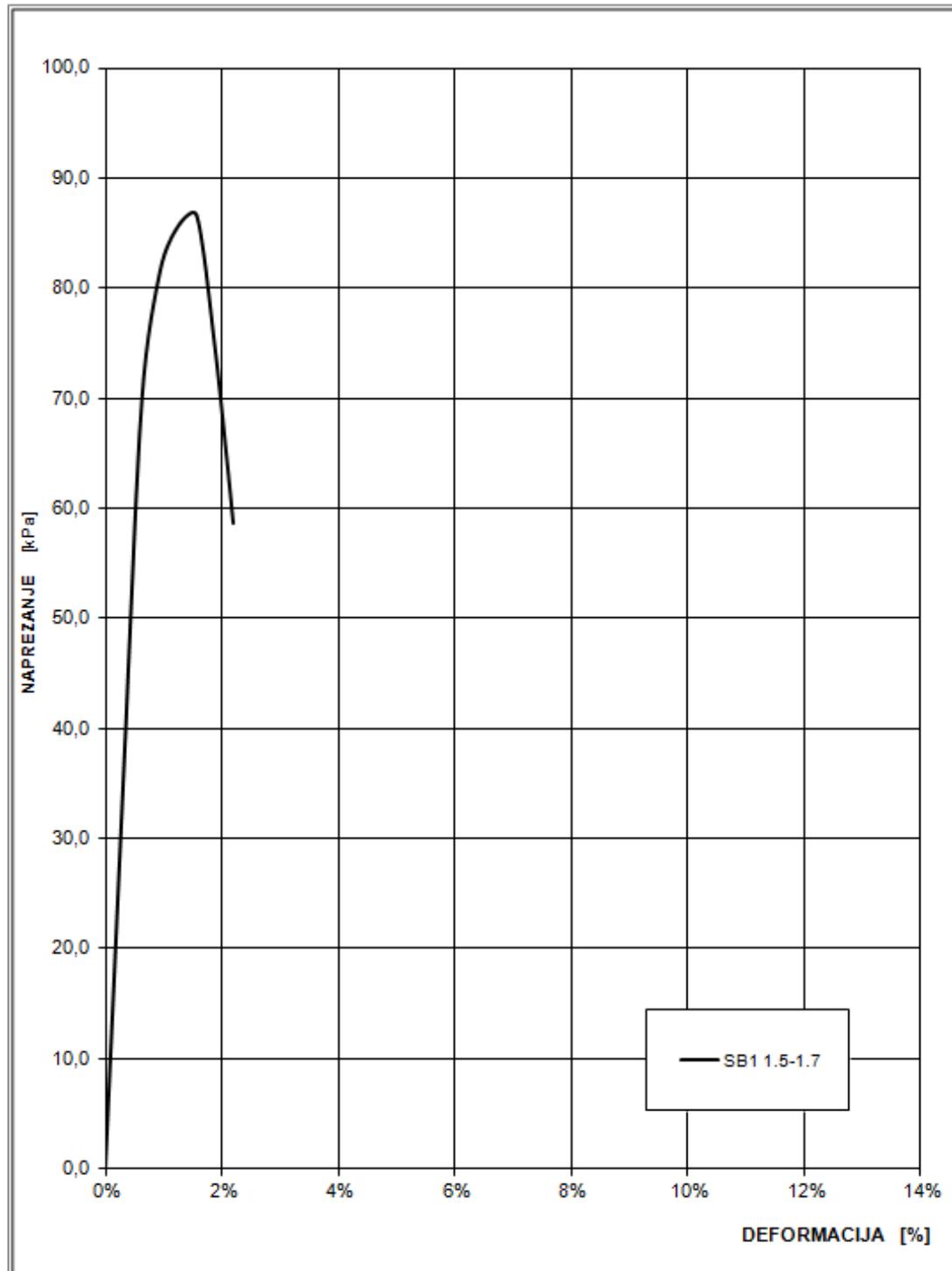
PARAMETRI ČVRSTOĆE	
ϕ (°)	c (kPa)
24,9	18,3

SONDA	DUBINA	AC
SB2	3.0-3.3	CI

LEGENDA: CH - Glina anorganska visoke plastičnosti
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08.11.2016.

Odgovorni inženjer: Trsek Brosig, dipl.ing.građ.

DIJAGRAM JEDNOOSNE ČVRSTOĆE
GEOMEHANIČKO MIŠLJENJE

PRILOG 9/042/2016



SANACIJA KLIZIŠTA "TOPLIČKA ULICA", OPĆINA KRAPINSKE TOPLICE

042/16

OBJEKT:		SAMACIJA KLIZIŠTA			LOKACIJA:			KRAPINSKE TOPLICE			DATUM: 11.2016.			MARIČIOČ:															
UZORAK		GRANULACIJA		VLAGA I KONZISTENCIJA			JEDINIČNE TEŽINE			ZBUENOST			ČVRSTOĆA			MODUL STIŠLJIVOSTI INKREMENTA VERTIKALNOG NADJEZVANJA			TERENSKI POKUSI										
DUBINA [m]	VRSTA	AC	G	S	M	C	W ₀	W _L	W _p	I _p	I ₀	γ	γ _s	γ _d	γ _{max}	W _{opt} [%]	q _u [kPa]	q _u [kPa]	φ [°]	φ [°]	c [kPa]	c [kPa]	M _v [MPa]	N [udist]	q _u [kPa]	τ [kPa]	OPASKA		
0.3-1.5	P	CI					39.2	48.7	22.3	26.4	0.36	17.7	12.7																
1.5-1.7	N	CH					35.2	61.6	19.1	42.5	0.62		86.7																
2.8-5.0	P	MH					32.0	62.0	32.2	29.9	1.01																		
0.3-2.3	P	CI					36.8	48.7	24.2	24.5	0.49																		
3.0-3.3	N	CH					37.3	62.3	25.1	37.1	0.67																		
3.3-5.0	P	MH					36.2	73.2	34.5	35.7	0.96																		



6.1. prikaz pozicije sondažne bušotine SB-1



6.2. prikaz jezgre sondažne bušotine SB-1



6.3. prikaz pozicije sondažne bušotine SB-2



6.4. prikaz jezgre sondažne bušotine SB-2