FONDI SHQIPTAR I ZHVILLIMIT
ALBANIAN DEVELOPMENT FUND

Bashkia Vlorë

RAPORT TEKNIK STRUKTURA

KONTRAT N°: SH/Sherbime/2019/GoA/SH-20

iRI & HYDRO&ENERGY

KORRIK 2019

1. Hyrje

Qellimi i punes se ketij relacioni eshte projektimi i strukturave b/a te meposhtme, ne Shetitoren e Vlores nga fushat e sportit deri tek tuneli. The scope of work of this report is the design of the following RC structures in the Vlora Promenade from sport fields to the tunnel:

- Pishina, Dhoma Teknike, Tualetet
- Rampa Rrethore
- Muri Mbajtes
- Shetitorja dhe Sheshi me Beton te Furçuar
- Shkalla ne Rruge
- Kolonada

2. Standartet dhe kodet e aplikuar

Llogaritja strukturale eshte bere bazuar ne standartet e meposhtme:

1. Ngarkesat ne strukture:
   - Ngarkese e Perhershme – (EN 1991-1-1:2002);
   - Ngarkese e Perkohshme – (EN 1991-1-1:2002);
   - Ngarkesa e debores – (EN 1991-1-3:2003);
   - Ngarkesa e Eeres– (prEN 1991-1-4:2004);
   - Sizmika – (prEN 1998-1:2005);
2. Llogaritja e elementeve b/a – (EN 1992-1-1:2005);
3. Llogaritja e themeleve b/a – (prEN 1997-1-2005);
4. Llogaritja e soletave ne toke – (ACI 224 – Joints in concrete construction);

3. Dokumentacion i projektimit

Dokumentacioni i meposhtem do te jete pjese e ketij relacioni

- Vizatimet arkitektonike dhe strukturale;
- Raporti gjeoteknik – Ing. Y. Muceku (Qershor 2014)
- Raporti i vleresimit sizmik – Prof.Dr. Rrapo Ormeni (Qershor 2018)

4. Pishina, Dhoma Teknike dhe Tualetet

4.1. Pershkrim i pergjithshem

Dimensionet e pergjithshme:

<u>Pishina</u>

Gjeresi – 25 m;

Gjatesi – 50 m;

Thellesi – 1.15 to 2.4 m;

Pllaka – 50 cm;

Muret – 50 cm;

<u>Dhoma teknike</u>

Gjeresi – 4.5 m;

Gjatesi – 44.5 m;

Lartesi dysheme-tavan – 2.6 m;

Pllaka – 50 cm;

Muret – 30 cm;

Soleta – 22 cm;

<u>Tualetet</u>

Gjeresi – 5.35 m;

Gjatesi – 49.5 m;

Lartesi dysheme-tavan – 2.9 m;

Pllaka – 50 cm;

Muret – 30-50 cm;

Kollonat – 25/25 cm;

Soleta – 22 cm;

Per me shume informacion te shihen fletet K-05, K-06, K-07, K-08.

Llogaritje eshte bere me programin me elemente te fundem SAP2000.

4.2. Materialet

Nenshtrese betoni - C12/15;

Pllaka e themelit, muret, soletat, shkallet – Beton C30/37;

Çelik armimi – B500b

4.3. Ngarkesat

Ngarkesat e meposhtme jane marre parasysh ne llogaritje:

4.3.1. Pesha vetjake (SW)

Pesha vetjake e elementeve b/a eshte marre parasysh automatikisht nga programi.

4.3.2. Ngarkesa e Perhershme (DL)

Ngarkesat e perhershme te meposhtme jane marre parasysh ne llogaritje:

Soleta e dhomes teknike (+2.420)
Shtresa. 50 cm – 12.5 kN/m^2

Soleta e tualeteve (+3.019)
Shtresa. 5 cm – 0.75 kN/m^2

Soleta e tualeteve (+6.189)
Shtresa. 40 cm – 10 kN/m^2

4.3.3. Ngarkesat e Perkohshme (LL)

Ngarkesat e perkohshme te meposhtme jane marre parasysh ne llogaritje:

Soleta e dhomes teknike (+2.420) - 5 kN/m^2 (EN 1991-1-1:2002 Category C3);
Soleta e tualeteve (+3.019) - 3 kN/m^2 (EN 1991-1-1:2002 Category A);
Soleta e tualeteve (+6.189) - 5 kN/m^2 (EN 1991-1-1:2002 Category G);

4.3.4. Presioni i terrenit (EL)

Te dhena:
$\alpha = 90^\circ$ - kendi i murit
$\beta = 0^\circ$ - kendi i faqes se murit
$\Phi = 30^\circ$ - kendi i ferkimit te brendshem te materialit mbushes (material granular)
$c = 0$ - kohezioni i materialit mbushes
$\delta = 0^\circ$ - kendi i ferkimit midis murit dhe terrenit (materialit mbushes)
$\gamma = 18.5$ kN/m^3 - densiteti i materialit mbushes
$\sigma = 1.5$ kg/cm^2 - sforcimi i lejuar ne baze(nga raporti gjeoteknik)

Presioni i terrenit mbi mur eshte llogaritur duke perdorur koefcientin aktiv te presionit te dheut te meposhtem, duke perdorur thellesite dhe lartesite respektive te mureve

$$K_a := \frac{\sin(\alpha+\phi)^2}{\sin(\alpha)^2\cdot\sin(\alpha-\delta)\left(1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\sin(\alpha-\delta)\cdot\sin(\alpha+\beta)}}\right)^2} = 0.334$$

Nje ngarkese e perkohshme prej 10 kN/m^2 i eshte shtuar presionit aktiv te murit.

4.3.5. Sizmika (EQ)

Te dhenat sizmike jane marre nga relacioni i vleresimit sizmik.

Truall Tip – E (Table 3.1);

Nxitimi sizmik - agR=0.249g

Koeficienti i rendesise se objektit - γ_I = 1 (Table 4.3)

Nxitimi sizmik llogarites ag = γ I * agR = 0.249g * 1 = 0.249g

Per Truall Tip E dhe Tip 1 te spektrit te reagimit sizmik jane perdorur parametrat e meposhtem (Table 3.2):

S = 1.4 , T_B = 0.15 s, T_C = 0.5 s, T_D = 2 s

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Table 4.3 Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

The design spectrum for the horizontal components shall be defined by the following expressions:

$$0 \le T \le T_\mathrm{B} : \; S_\mathrm{d}(T) = a_\mathrm{g} \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_\mathrm{B}} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \qquad (3.13)$$

$$T_\mathrm{B} \le T \le T_\mathrm{C} : \; S_\mathrm{d}(T) = a_\mathrm{g} \cdot S \cdot \frac{2,5}{q} \qquad (3.14)$$

$$T_\mathrm{C} \le T \le T_\mathrm{D} : \; S_\mathrm{d}(T) \begin{cases} = a_\mathrm{g} \cdot S \cdot \dfrac{2,5}{q} \cdot \left[\dfrac{T_\mathrm{C}}{T} \right] \\ \ge \beta \cdot a_\mathrm{g} \end{cases} \qquad (3.15)$$

$$T_\mathrm{D} \le T : \quad S_\mathrm{d}(T) \begin{cases} = a_\mathrm{g} \cdot S \cdot \dfrac{2,5}{q} \cdot \left[\dfrac{T_\mathrm{C} T_\mathrm{D}}{T^2} \right] \\ \ge \beta \cdot a_\mathrm{g} \end{cases} \qquad (3.16)$$

Faktoret e sjelljes $q_x = q_x = 1.5$ (Struktura nuk pritet te kete sjellje duktile).

4.4. Kombinimet e ngarkesave

Kombinime e ngarkesave jane berse konform EN 1990-1:1994.

Table A1.2(A) - Design values of actions (EQU) (Set A)

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,1} Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{Gj,sup} = 1,10$

$\gamma_{Gj,inf} = 0,90$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with the following set of recommended values. The recommended values may be altered by the National annex.

$\gamma_{Gj,sup} = 1,35$

$\gamma_{Gj,inf} = 1,15$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

provided that applying $\gamma_{Gj,inf} = 1,00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Table A1.3 - Design values of actions for use in accidental and seismic combinations of actions

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions (**)	
	Unfavourable	Favourable		Main (if any)	Others
Accidental (*) (Eq. 6.11a/b)	$G_{kj,sup}$	$G_{kj,inf}$	A_d	ψ_{11} or $\psi_{21} Q_{k1}$	$\psi_{2,i}\ Q_{k,i}$
Seismic (Eq. 6.12a/b)	$G_{kj,sup}$	$G_{kj,inf}$	$\gamma_I A_{Ek}$ or A_{Ed}		$\psi_{2,i}\ Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National annex, depending on the accidental action under consideration. See also EN 1991-1-2.

(**) Variable actions are those considered in Table A1.1.

KOMBINIMET E NGARKESAVE

LOAD COMBINATION	LOAD CASES				
	GRAVITY			SEISMIC	
	SWEIGHT (SW)	DEAD (DL)	LIVE (LL)	Q(X)	Q(Y)
DL+1.5LL	1	1	1.5		
1.35 DL + 1.5LL	1.35	1.35	1.5		
EQ(X)	1	1	0.3	1	
EQ(Y)	1	1	0.3		1

4.5. Analiza & rezultate

Llogaritja eshte bere me programin me elemente te fundem SAP2000.

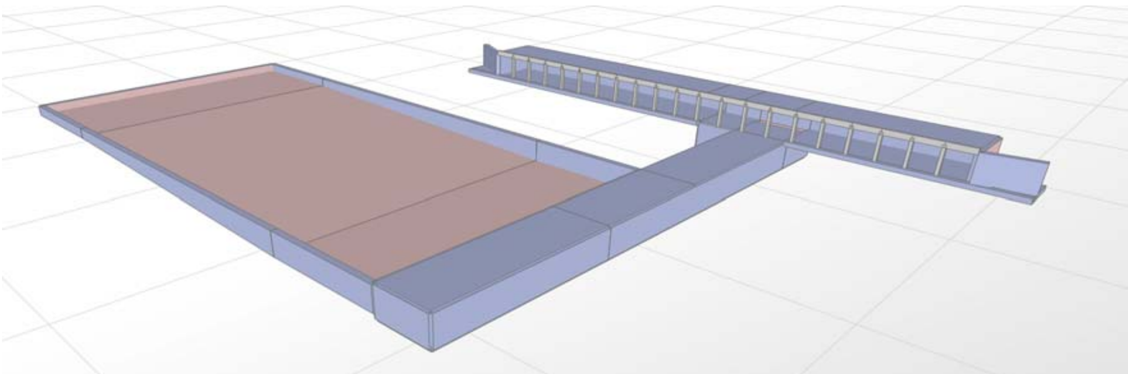
Pllakat e themelit jane modeluar si elemente “shell” me koeficient suste – 10000 kN/m^3.

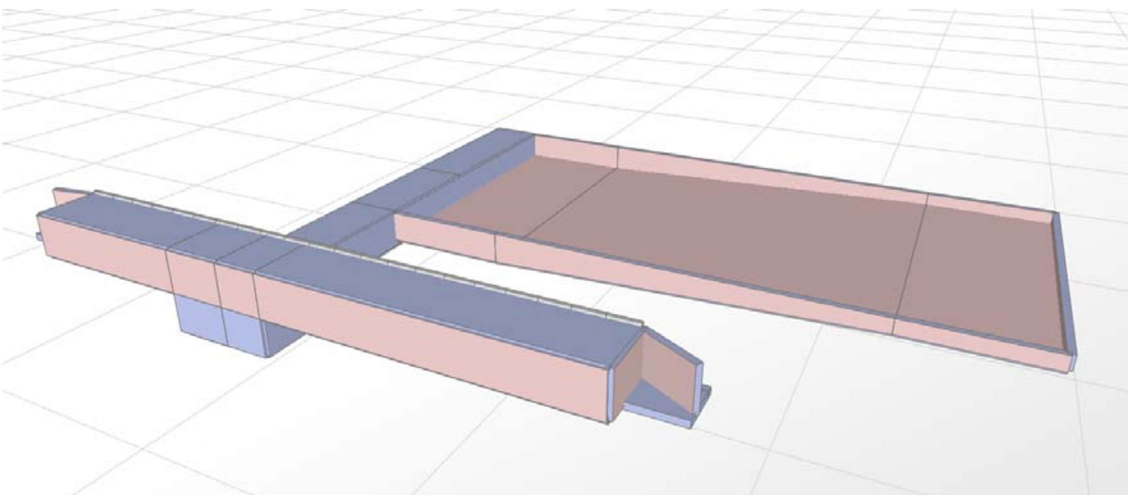
Muret dhe soletat jane modeluar si elemente “shell”

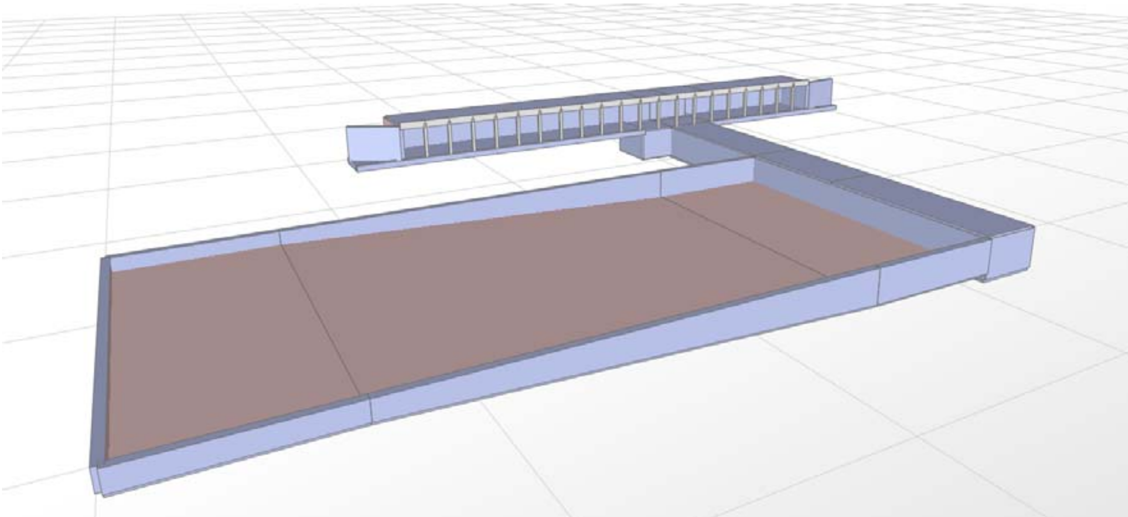
Kollonat dhe traret jane modeluar si elemente “frame”

Kollonat jane konsideruar “nyje cerniere” ne maje.

Pamje 3-dimensionale Modeli SAP

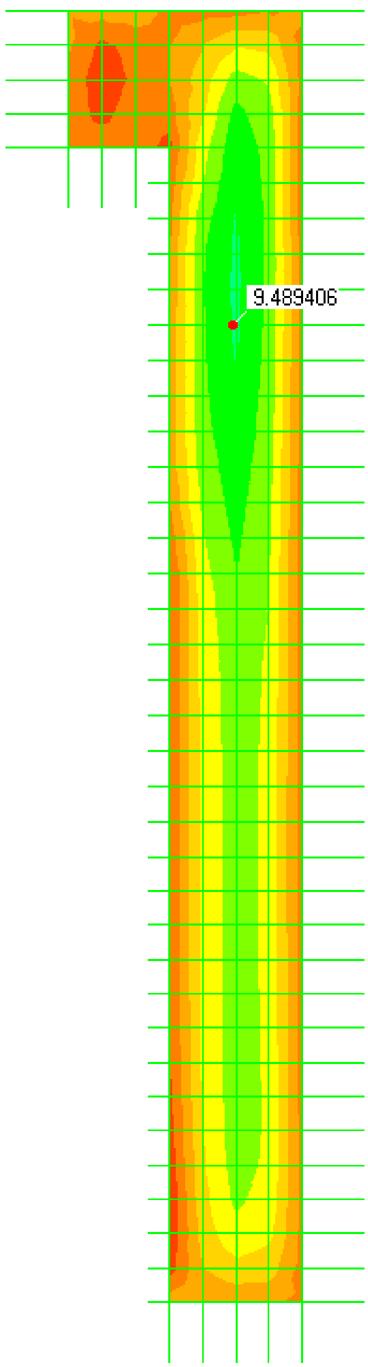


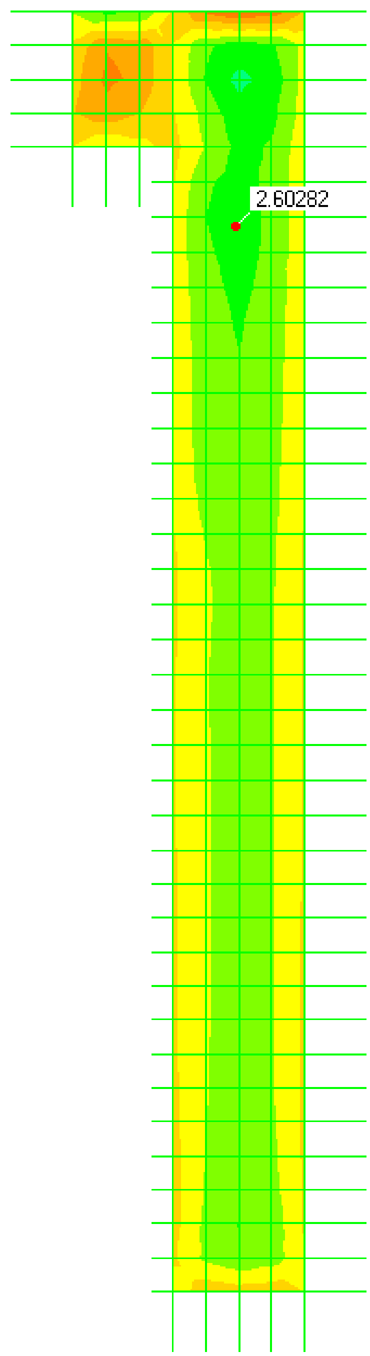




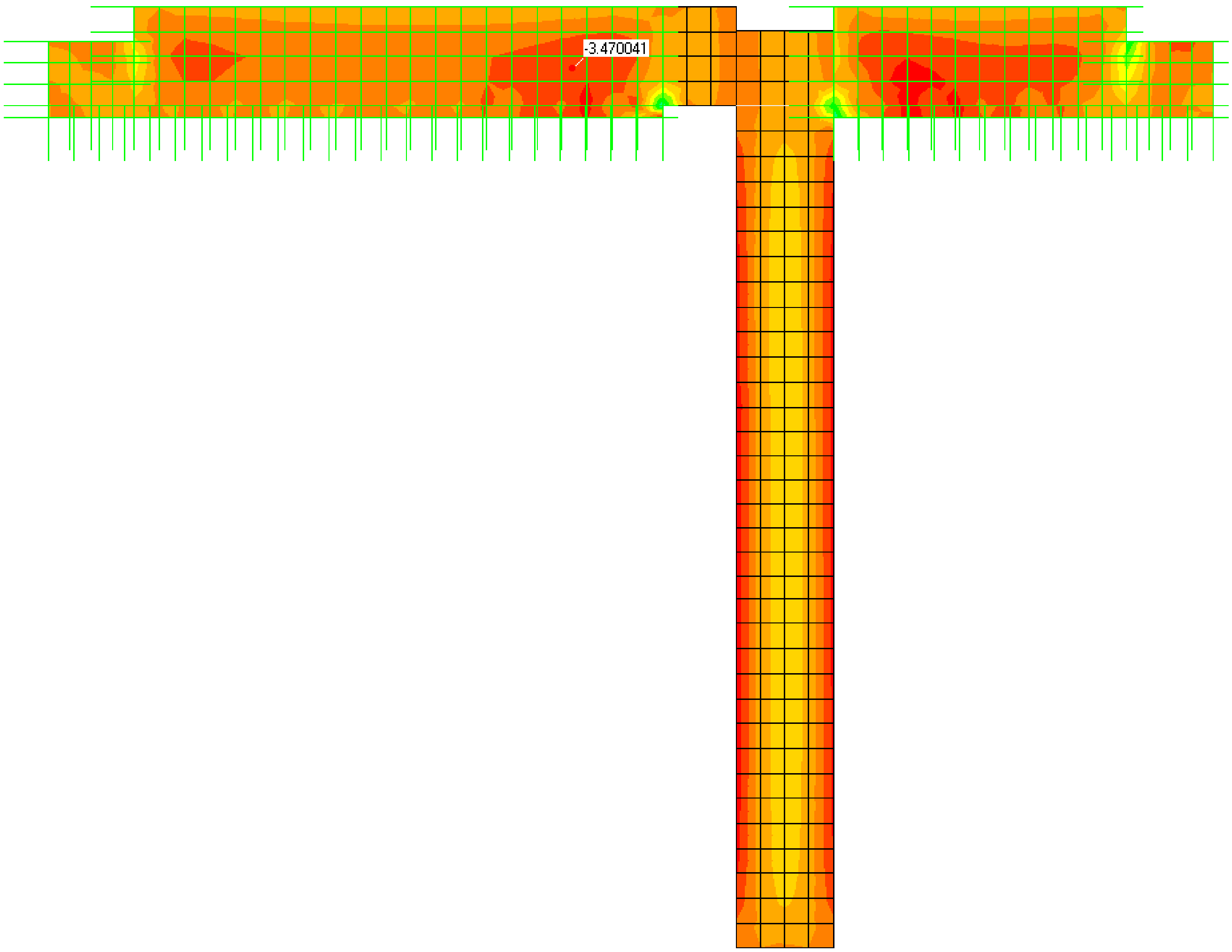
Ne faqet pasardhese jane paraqitur diagrama te momenteve per pllakat e themelit, soletat dhe llogaritje te armimit per disa elemente.
Llogaritja e trareve e kollonave eshte bere automatikisht nga programi.

<u>Pllaka e dhomes teknike ne kuoten +6.189</u>
<u>Moment M11 & M22 Diagram</u>

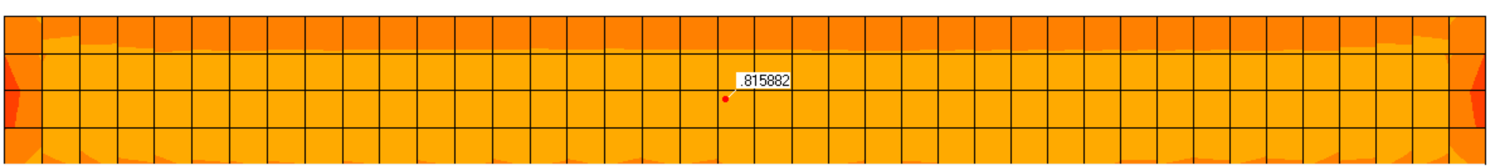




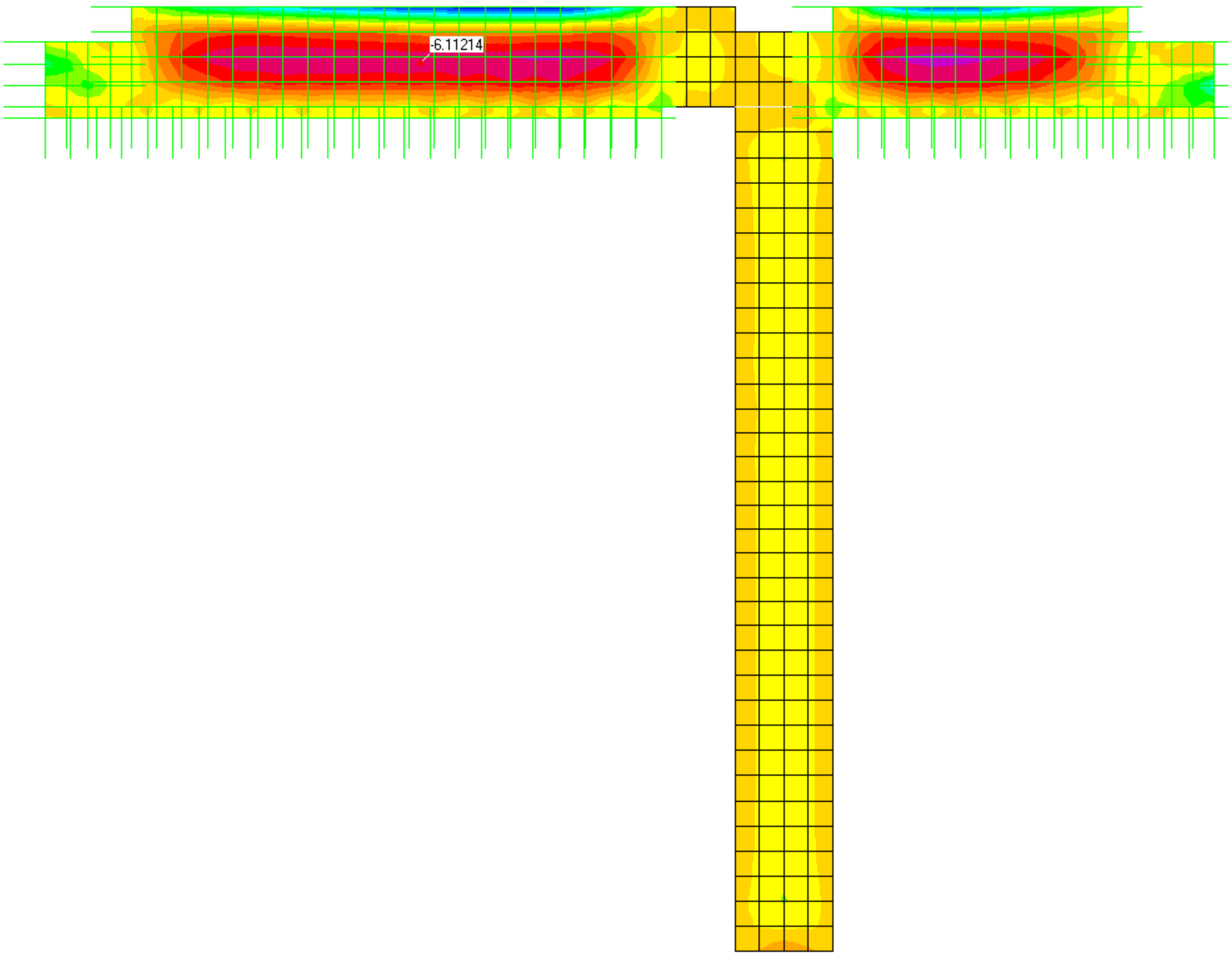
Pllaka e tualeteve ne kuoten +2.200
Soleta e dhomes teknike ne kuoten +2.420
Moment M11 Diagram



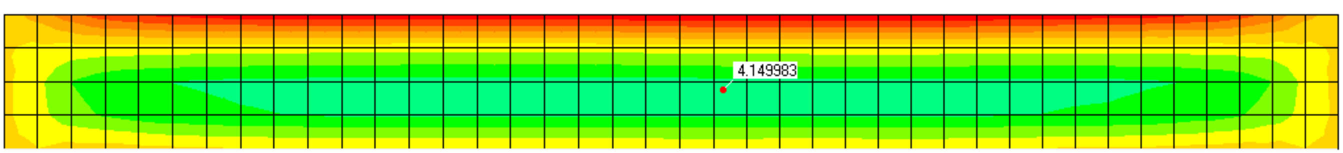
Soleta e tualeteve ne kuoten +6.189
Moment M11 Diagram



Pllaka e tualeteve ne kuoten +2.200
Soleta e dhomes teknike ne kuoten +2.420
Moment M22 Diagram



Soleta e tualeteve ne kuoten +6.189
Moment M22 Diagram



RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Technical room mat plate

Design according Eurocode 2

SECTION DATA:

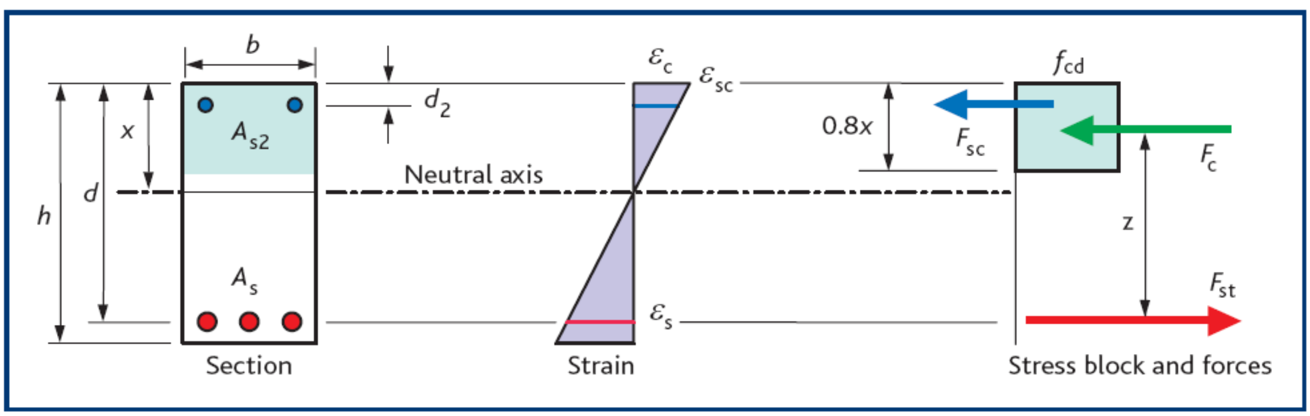
$h := 50$	cm	height of section;
$b := 100$	cm	width of section;
$a := 5$	cm	cover to reinforcement;
$d_{t.bar} := 1.4$	cm	tension bar diameter;
$d_{c.bar} := 1.4$	cm	compression bar diameter;
$d_{link} := 1.4$	cm	link diameter;
$d = 42.9$	cm	effective depth of the beam;
$d' = 7.1$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 9.5$	ton*m	Design Moment;
$\delta := 0.8$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.017 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25}\right)^2 = 0.116$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}}\right)\right] = 42.24\ cm \ < \ z_{max} := 0.95 \cdot d = 40.75\ cm$$

$$x := \frac{(d - z)}{0.4} = 1.65 \quad cm$$ depth of neutral axis;

Compression steel:

$$\frac{d'}{d} = 0.166 \ < \ 0.171$$ compression steel will have yielded;

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')}\right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}}\right)$$

$$A_s = 5.17 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.12 \ \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 6.46 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 6.5\ cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 171.6 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^2}{4}} = 3.36 \quad bars$$

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Technical Rooom slab at +2.420

Design according Eurocode 2

SECTION DATA:

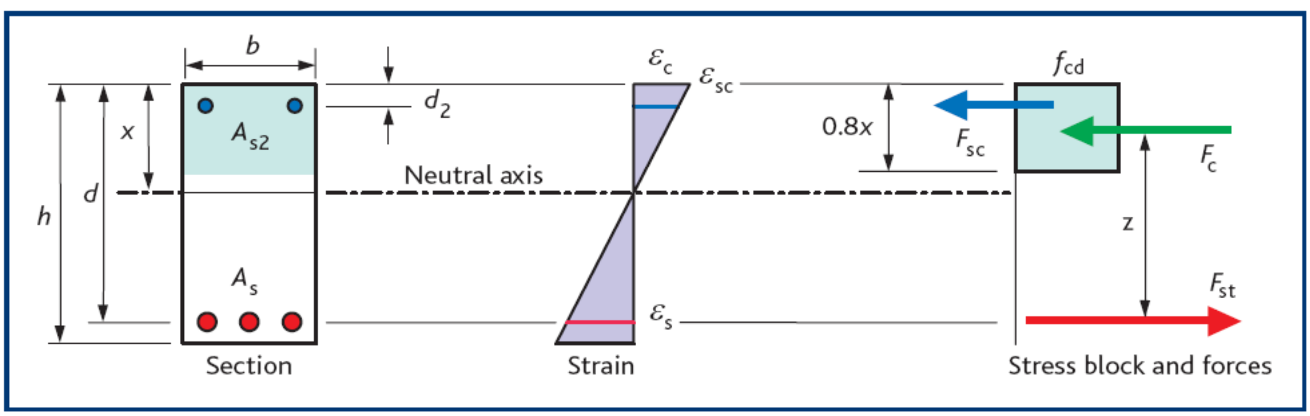
$h := 22$	cm	height of section;
$b := 100$	cm	width of section;
$a := 4$	cm	cover to reinforcement;
$d_{t.bar} := 1.2$	cm	tension bar diameter;
$d_{c.bar} := 1.2$	cm	compression bar diameter;
$d_{link} := 1$	cm	link diameter;
$d = 16.4$	cm	effective depth of the beam;
$d' = 5.6$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg/cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg/cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg/cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg/cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 2.9$	ton*m	Design Moment;
$\delta := 1$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.036 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25}\right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}}\right)\right] = 15.86\ cm \ < \ z_{max} := 0.95 \cdot d = 15.58\ cm$$

$$x := \frac{(d - z)}{0.4} = 1.34 \quad cm$$ *depth of neutral axis;*

Compression steel:

$$\frac{d'}{d} = 0.341 \quad < \quad 0.171$$ *compression steel will have yielded;*

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')}\right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}}\right)$$

$$A_s = 4.20 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.26 \ \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 2.47 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 2.9\ cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 65.6 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^2}{4}} = 3.72 \quad bars$$

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT

Cloackroom mat plate

Design according Eurocode 2

SECTION DATA:

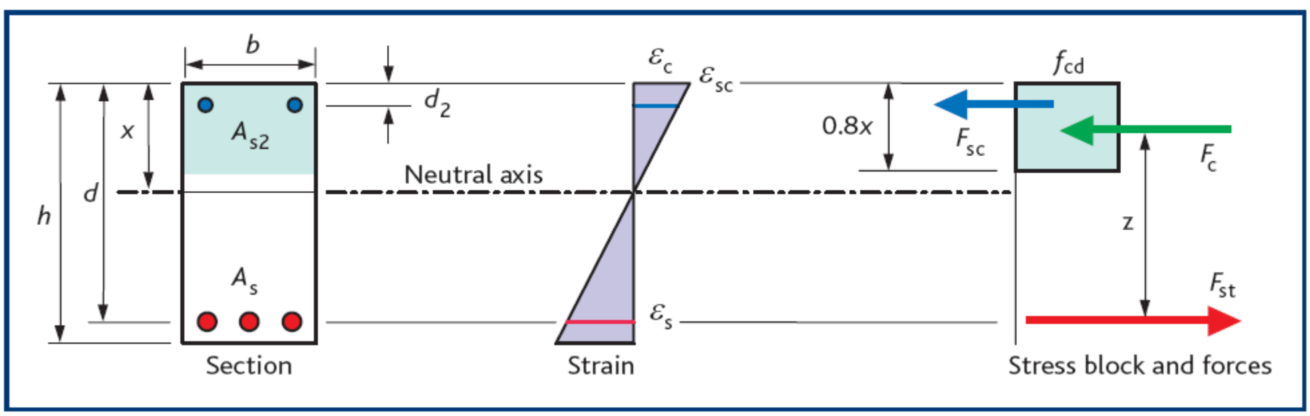
$h := 50$	cm	height of section;
$b := 100$	cm	width of section;
$a := 5$	cm	cover to reinforcement;
$d_{t.bar} := 1.4$	cm	tension bar diameter;
$d_{c.bar} := 1.4$	cm	compression bar diameter;
$d_{link} := 1.4$	cm	link diameter;
$d = 42.9$	cm	effective depth of the beam;
$d' = 7.1$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 6.1$	ton*m	Design Moment;
$\delta := 0.8$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.011 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25} \right)^2 = 0.116$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}} \right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}} \right) \right] = 42.48\,cm \; < \; z_{max} := 0.95 \cdot d = 40.75\,cm$$

$$x := \frac{(d - z)}{0.4} = 1.06 \quad cm$$ *depth of neutral axis;*

Compression steel:

$\frac{d'}{d} = 0.166 \; < \; 0.171$ *compression steel will have yielded;*

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')} \right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}} \right)$$

$$A_s = 3.30 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.08 \; \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 6.46 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 6.5\,cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 171.6 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^{\,2}}{4}} = 2.14 \quad bars$$

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Cloackroom slab at +6.189

Design according Eurocode 2

SECTION DATA:

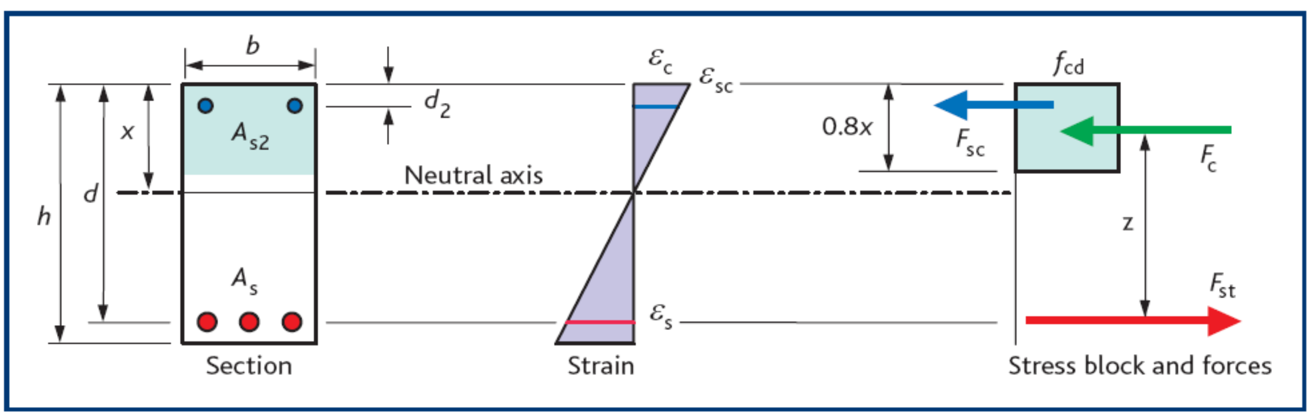
$h := 22$	cm	height of section;
$b := 100$	cm	width of section;
$a := 4$	cm	cover to reinforcement;
$d_{t.bar} := 1.2$	cm	tension bar diameter;
$d_{c.bar} := 1.2$	cm	compression bar diameter;
$d_{link} := 1$	cm	link diameter;
$d = 16.4$	cm	effective depth of the beam;
$d' = 5.6$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 4.15$	ton*m	Design Moment;
$\delta := 1$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.051 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25} \right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}} \right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}} \right) \right] = 15.62 \, cm \; < \; z_{max} := 0.95 \cdot d = 15.58 \, cm$$

$$x := \frac{(d - z)}{0.4} = 1.95 \quad cm$$ depth of neutral axis;

Compression steel:

$$\frac{d'}{d} = 0.341 \quad < \quad 0.171$$ compression steel will have yielded;

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')} \right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}} \right)$$

$$A_s = 6.11 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.37 \; \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 2.47 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 2.9 \, cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 65.6 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^{\;2}}{4}} = 5.40 \quad bars$$

5. Rampa rrethore

5.1. Pershkrim i pergjithshem

Rampa do te nise nga sheshi me beton te furçuar deri ne trotuarin ne krah te tualeteve. Ajo ka dy segmente te drejta dhe nje pjese rrethore (spirale) me pjerresi perafersisht 5%. Gjeresia e rampes eshte 2.1 m.

Per me shume informacion te shihen fletet K-09, K-10.

5.2. Materialet

Nenshtrese betoni - C12/15;
Pllaka e themelit, muret, soletat– Beton C30/37;
Çelik armimi – B500b

5.3. Ngarkesat

Ngarkesat e meposhtme jane marre parasysh ne llogaritje:

5.3.1. Pesha vetjake (SW)

Pesha vetjake e elementeve b/a eshte marre parasysh automatikisht nga programi.

5.3.2. Ngarkesa e perkohshme (LL)

Ngarkesat e perkohshme te meposhtme jane marre parasysh ne llogaritje:

Soleta - 5 kN/m2 (EN 1991-1-1:2002 Category C3);

5.3.3. Sizmika (EQ)

Referoju seksionit 4.3.5.

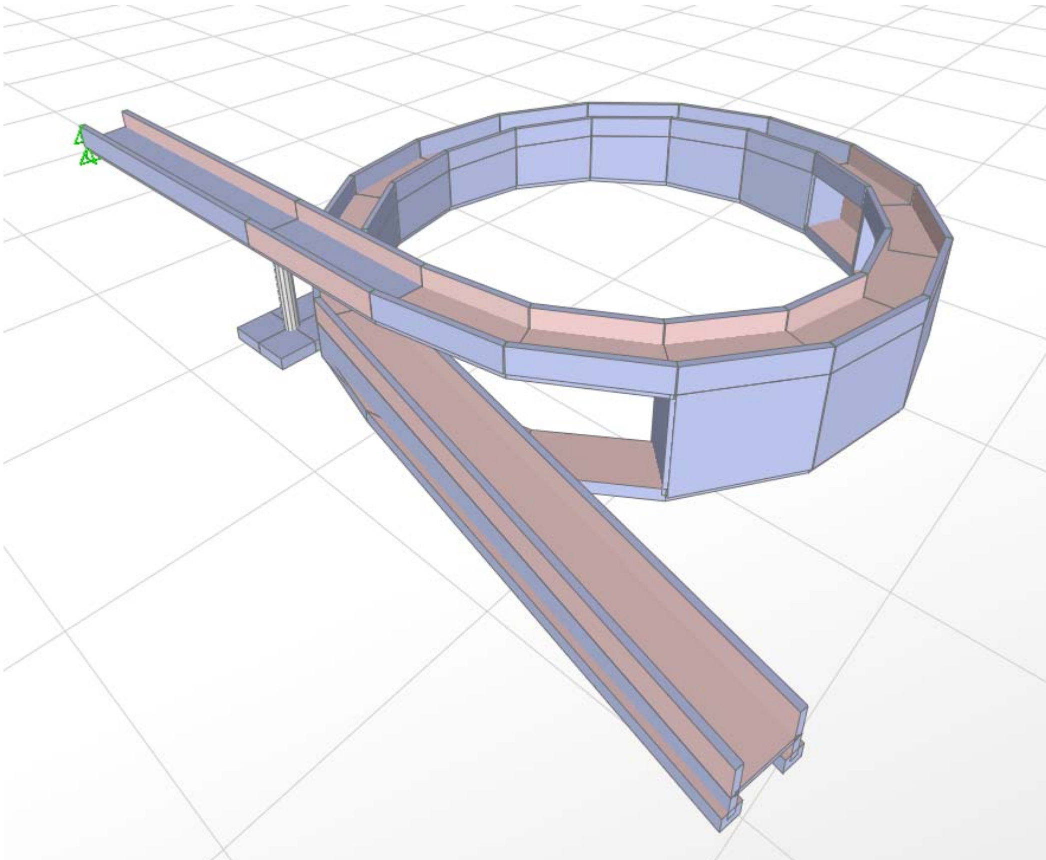
5.4. Kombinimi i ngarkesave

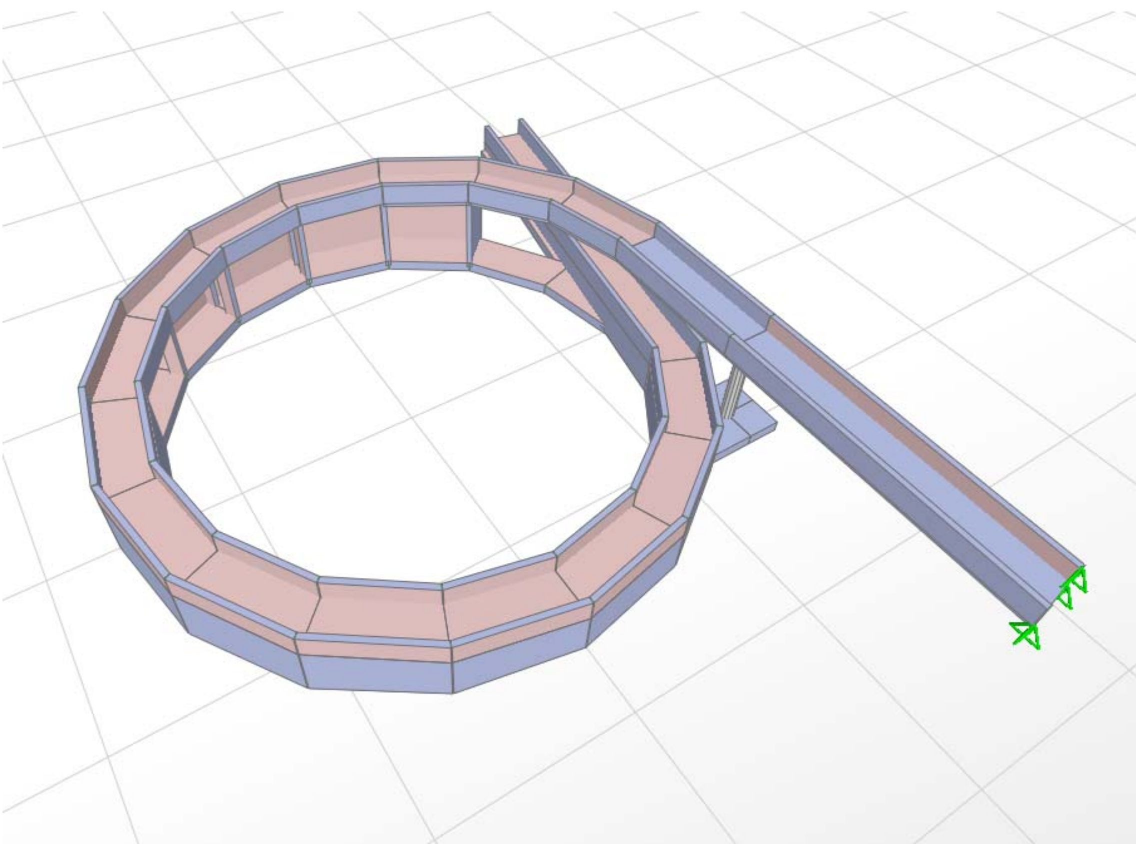
Referoju seksionit 4.4.

5.5. Analiza dhe rezultate

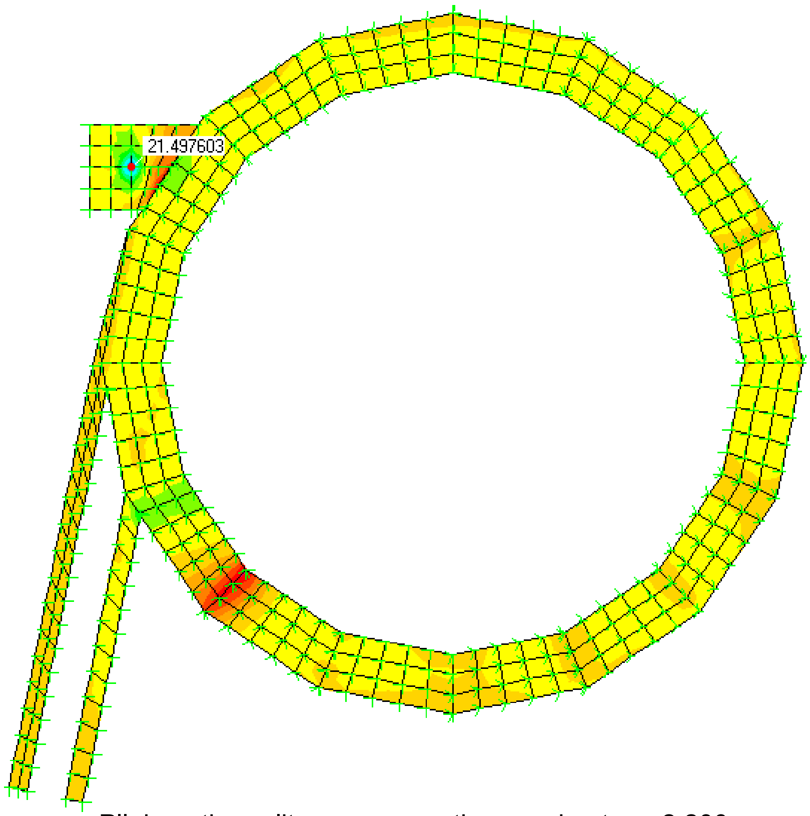
Llogaritja eshte bere me programin me elemente te fundem SAP2000.
Pllaka e themelit eshte modeluar si elemente “shell” me koeficient suste – 10000 kN/m3.
Ne faqet pasardhese jane paraqitur diagrama te momenteve per pllaken e themelit, soletat dhe llogaritje te armimit per disa elemente.

Pamje 3-dimensionale Modeli SAP

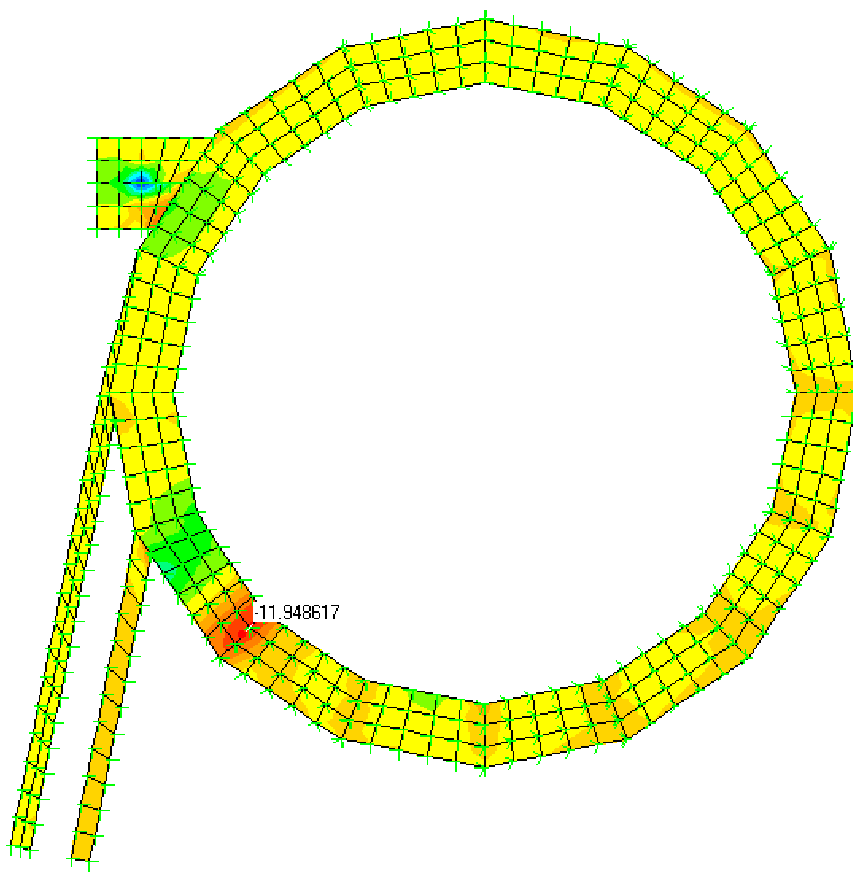




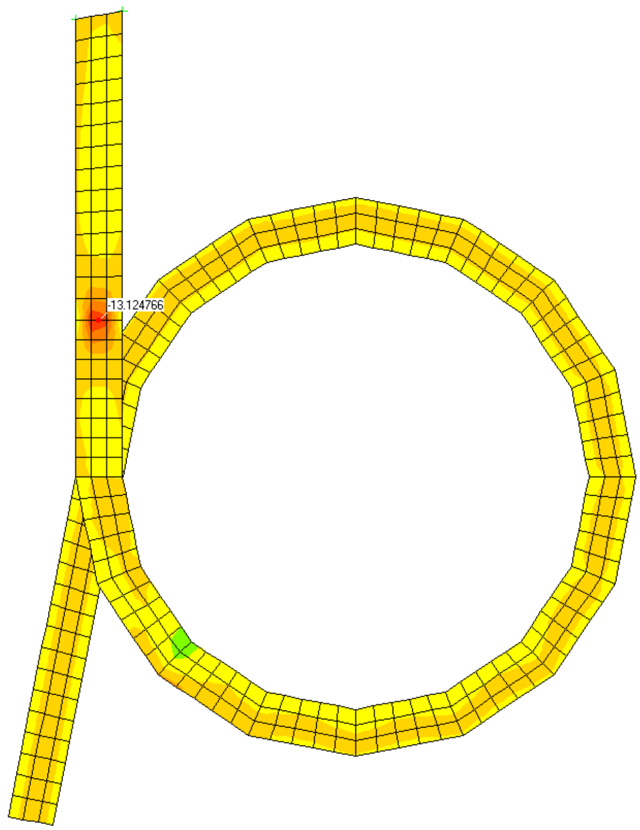
Pllaka e themelit e rampes rrethore ne kuoten +2.200
Moment M11 Diagram



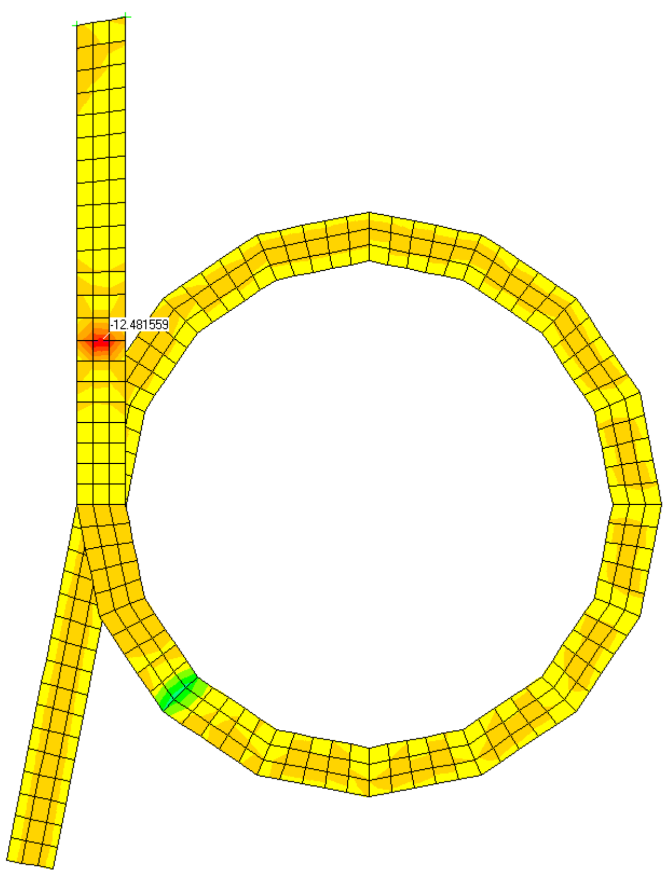
Pllaka e themelit e rampes rrethore ne kuoten +2.200
Moment M22 Diagram



Soleta e rampes rrethore
Moment M11 Diagram



Soleta e rampes rrethore
Moment M22 Diagram



RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Circular ramp mat plate

Design according Eurocode 2

SECTION DATA:

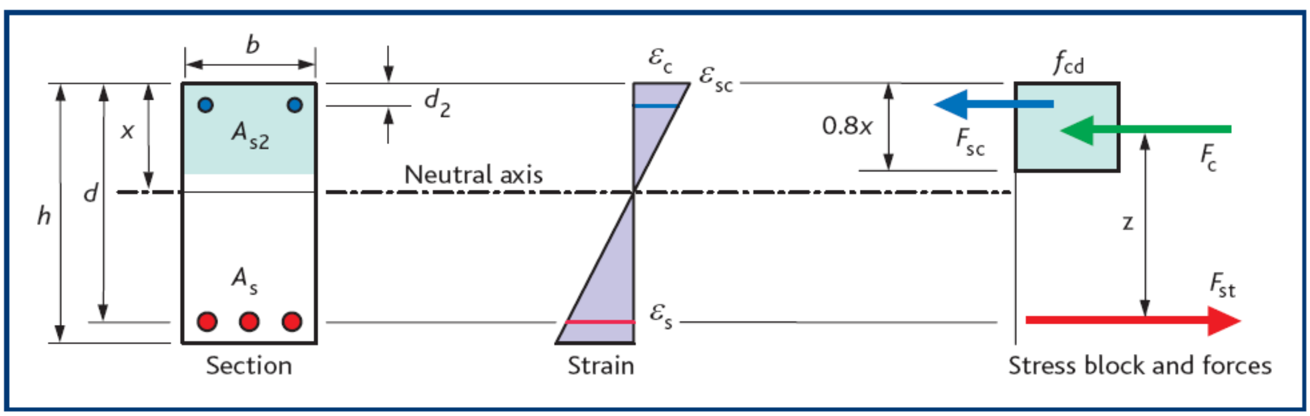
$h := 50$	cm	height of section;
$b := 100$	cm	width of section;
$a := 5$	cm	cover to reinforcement;
$d_{t.bar} := 1.4$	cm	tension bar diameter;
$d_{c.bar} := 1.4$	cm	compression bar diameter;
$d_{link} := 1$	cm	link diameter;
$d = 43.3$	cm	effective depth of the beam;
$d' = 6.7$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 12$	ton*m	Design Moment;
$\delta := 1$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.021 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25}\right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}}\right)\right] = 42.47\,cm \; < \; z_{max} := 0.95 \cdot d = 41.13\,cm$$

$$x := \frac{(d - z)}{0.4} = 2.08 \quad cm \qquad \text{depth of neutral axis;}$$

Compression steel:

$$\frac{d'}{d} = 0.155 \quad < \quad 0.171 \qquad \text{compression steel will have yielded;}$$

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')}\right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}}\right)$$

$$A_s = 6.50 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.15 \; \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 6.52 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 6.5\,cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 173.2 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^2}{4}} = 4.22 \quad bars$$

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Circular ramp slab

Design according Eurocode 2

SECTION DATA:

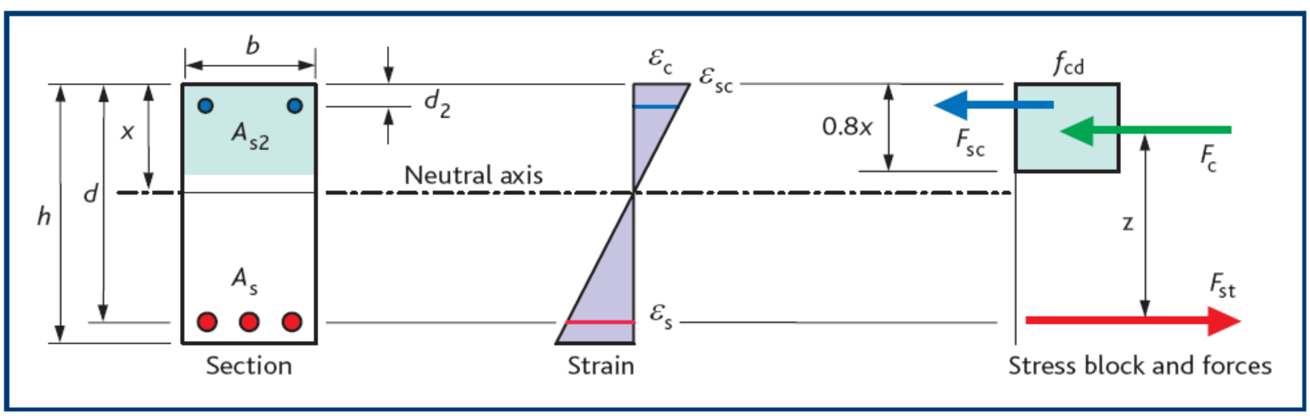
$h := 50$	cm	*height of section;*
$b := 100$	cm	*width of section;*
$a := 5$	cm	*cover to reinforcement;*
$d_{t.bar} := 2$	cm	*tension bar diameter;*
$d_{c.bar} := 2$	cm	*compression bar diameter;*
$d_{link} := 1$	cm	*link diameter;*
$d = 43$	cm	*effective depth of the beam;*
$d' = 7$	cm	*depth of centroid of compression steel;*

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	*characteristic cylinder strength of concrete;*
$f_{ctm} = 29$	kg / cm^2	*mean axial tensile strength of concrete;*
$f_{yk} := 5000$	kg / cm^2	*characteristic yield strength of steel;*
$f_{sck} = 4830$	kg / cm^2	*characteristic yield strength of compressed steel considering the displaced concrete;*

LOADING DATA:

$M := 13$	ton*m	*Design Moment;*
$\delta := 1$		*Moment distribution factor;*



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.023 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25} \right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}}\right)\right] = 42.09\ cm \ < \ z_{max} := 0.95 \cdot d = 40.85\ cm$$

$$x := \frac{(d - z)}{0.4} = 2.27 \quad cm$$ *depth of neutral axis;*

Compression steel:

$$\frac{d'}{d} = 0.163 \ < \ 0.171$$ *compression steel will have yielded;*

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')}\right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}}\right)$$

$$A_s = 7.10 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.17 \ \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 6.48 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 6.5\ cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 172 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^{\,2}}{4}} = 2.26 \quad bars$$

```
Eurocode 2-2004 COLUMN SECTION DESIGN  Type: Sway Frame  Units: Tonf, cm, C  (Summary)

L=440.000
Element     : 122                D=60.000        dc=8.703
Section ID  : D60                E=253.105       fck,cyl=0.250   Lt.Wt. Fac.=1.000
Combo ID    : EQ(Y)              fyk=5.099       fywk=5.099
Station Loc : 440.000            RLLF=1.000      SOM: Nominal Stiffness
Combo Eq.   : Eq. 6.10

Gamma(Concrete): 1.500           AlphaCC=1.000   AlphaCT=1.000
Gamma(Steel)   : 1.150           AlphaLCC=0.850  AlphaLCT=0.850


AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NEd, MEd2, MEd3
               Rebar      Design      Design      Design     Minimum     Minimum
                Area         NEd        MEd2        MEd3          M2          M3
               5.655      61.762    -333.738     506.633     123.523     123.523

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                             M0e        Madd     Minimum        Beta           L
                          Moment      Moment         Ecc      Factor      Length
   Major Bending(M3)     353.389      11.216       2.000       1.000     440.000
   Minor Bending(M2)    -133.495       4.237       2.000       1.000     440.000

SHEAR DESIGN FOR V2,V3
                           Rebar       Shear       Shear       Shear
                           Asw/s         VEd        VRdc        VRds
   Major Shear(V2)         0.000       2.148      18.952       0.000
   Minor Shear(V3)         0.000       1.640      18.952       0.000
```

6. Retaining wall

6.1. Pershkrim i pergjithshem

Muret mbajtes kane lartesi variabel pergjate shetitores.

Ata ndahen ne fuga termike cdo 20 m.

Per me shume informacion mbi gjeometrine e murit shih fletet K-13, K-14, K-15, K-16.

6.2. Materialet

Nenshtrese betoni - C12/15;

Muri– Beton C30/37;

Çelik armimi – B500b

6.3. Ngarkesat

Ngarkesat e meposhtme jane marre parasysh ne llogaritje:

6.3.1. Ngarkesa e perkohshme (LL)

Nje ngarkese e perkohshme prej 10 kN/m^2 i eshte shtuar presionit aktiv te murit.

6.3.2. Presioni i terrenit (EL)

Te dhena:

$\alpha = 90^\circ$ - kendi i murit

$\beta = 0^\circ$ - kendi i faqes se murit

$\Phi = 30^\circ$ - kendi i ferkimit te brendshem te materialit mbushes (material granular)

$c = 0$ - kohezioni i materialit mbushes

$\delta = 0^\circ$ - kendi i ferkimit midis murit dhe terrenit (materialit mbushes)

$\gamma = 18.5\ kN/m^3$ - densiteti i materialit mbushes

$\sigma = 1.5\ kg/cm^2$ - sforcimi i lejuar ne baze(nga raporti gjeoteknik)

Presioni i terrenit mbi mur eshte llogaritur duke perdorur koefcientin aktiv te presionit te dheut te meposhtem, duke perdorur thellesite dhe lartesite respektive te mureve

$$K_a := \frac{\sin(\alpha + \phi)^2}{\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\left(1 + \sqrt{\dfrac{\sin(\phi + \delta)\cdot\sin(\phi - \beta)}{\sin(\alpha - \delta)\cdot\sin(\alpha + \beta)}}\right)^2} = 0.334$$

6.3.3. Sizmika (EQ)

The seismic design is done according Eurocode 8 – Part 5 (EN1998-5-2004).

The design shown below corresponds to the seismic earth pressure of Section 1 - 1

$H := 6$ m	Lartesia e murit
$\gamma_s := 18.5$ kN/m^3	Densiteti I materialit mbushes
$\phi_d := 30$	Kendi I ferkimit te materialit mbushes
$\alpha := 0.249$	Raporti I nxitimin sizmik llogarites / g
$S := 1.4$	Parameter I terrenit Category E
$r := 1$	Faktor I tipit te murit Tabela 7.1
$k_h := \alpha \cdot \frac{S}{r} = 0.349$	Koeficienti sizmik horizontal
$k_v := 0.33 \cdot k_h = 0.115$	Koeficienti sizmik vertikal

$$\theta := \operatorname{atan}\left(\frac{k_h}{1 + k_v}\right) = 0.303$$

$\beta := 0$ $\qquad \delta := 0$

Koeficientet e presionit te terrenit sipas ekuacioneve te Monobe - Okabe

$$K_{aed} := \frac{\left(\sin\left(\frac{\pi}{2} + \phi_d - \theta\right)\right)^2}{\cos(\theta)\cdot\sin\left(\frac{\pi}{2} - \theta - \delta\right)\cdot\left(1 + \sqrt{\frac{\sin(\phi_d + \delta)\cdot\sin(\phi_d - \theta)}{\sin\left(\frac{\pi}{2} - \theta - \delta\right)}}\right)^2} = 0.583$$

$$K_{ped} := \frac{\left(\sin\left(\frac{\pi}{2} + \phi_d - \theta\right)\right)^2}{\cos(\theta)\cdot\sin\left(\frac{\pi}{2} + \theta\right)\cdot\left(1 - \sqrt{\frac{\sin(\phi_d)\cdot\sin(\phi_d - \theta)}{\sin\left(\frac{\pi}{2} + \theta\right)}}\right)^2} = 2.389$$

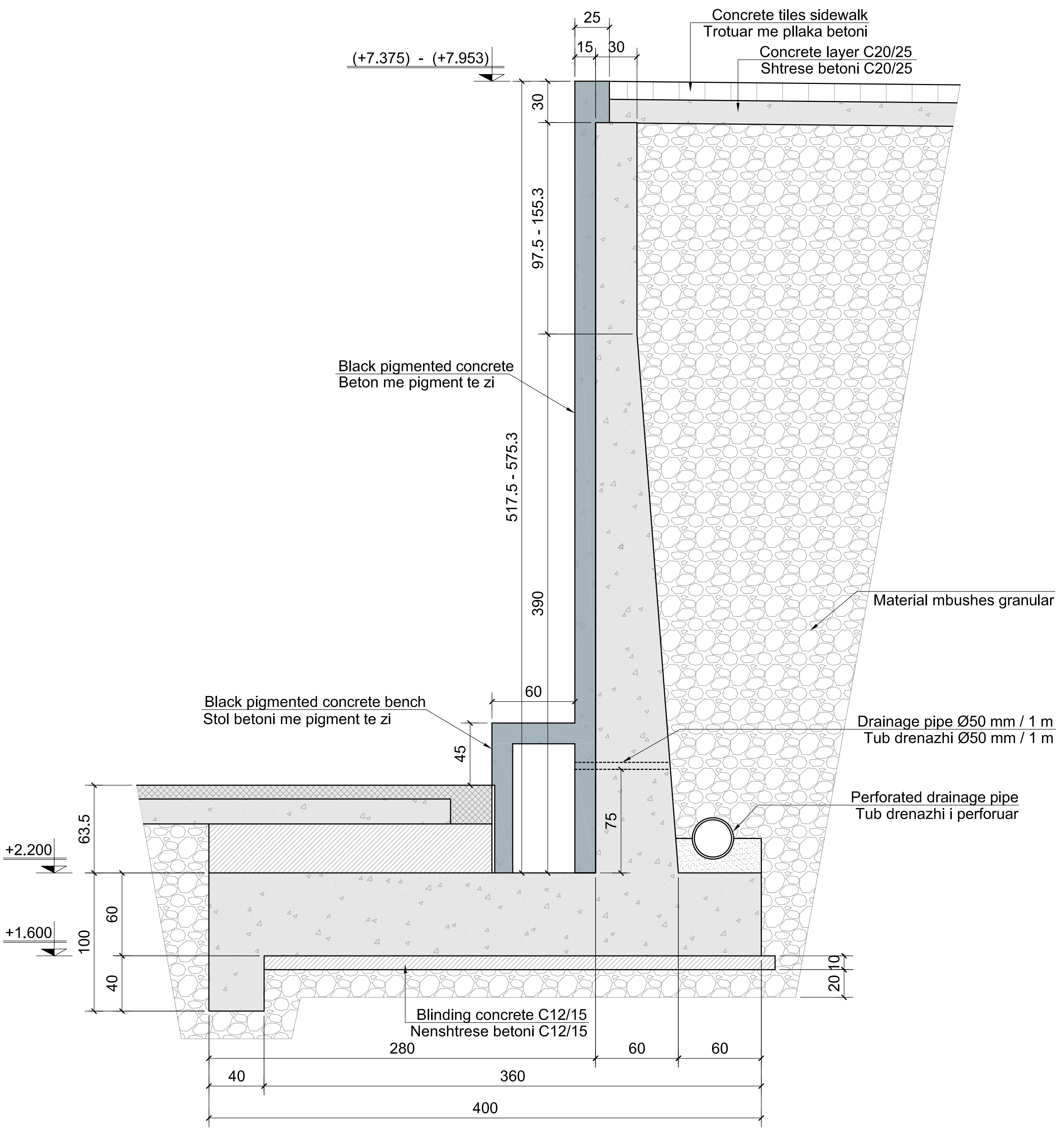
Presioni llogarites sizmik I terrenit eshte :

$$E_{aed} := \gamma_s \cdot (1 + k_v) \cdot K_{aed} \cdot \frac{H^2}{2} = 216.6 \qquad \frac{kN}{m}$$

$$E_{ped} := \gamma_s \cdot (1 + k_v) \cdot K_{ped} \cdot \frac{H^2}{2} = 887.1 \qquad \frac{kN}{m}$$

Ne faqet e meposhtme jane paraqitur llogaritje per 3 tipe muresh.

SECTION 1 - 1

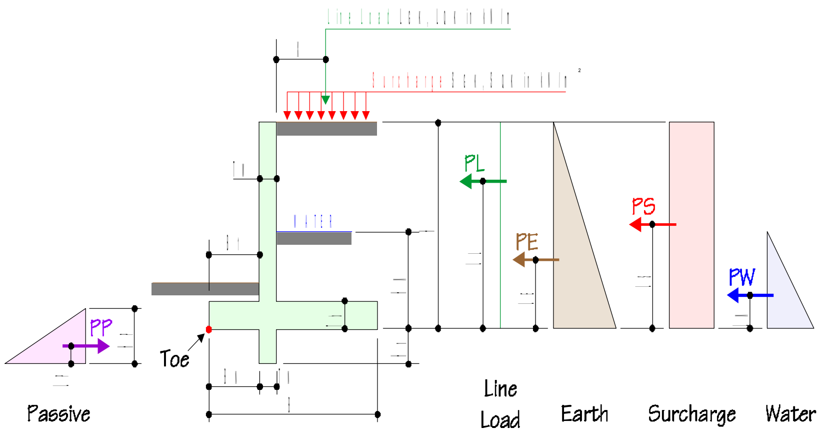

25
15
30
(+7.375) - (+7.953)
Concrete tiles sidewalk
Trotuar me pllaka betoni
Concrete layer C20/25
Shtrese betoni C20/25
30
97.5 - 155.3
517.5 - 575.3
390
Black pigmented concrete
Beton me pigment te zi
Material mbushes granular
Black pigmented concrete bench
Stol betoni me pigment te zi
60
45
Drainage pipe Ø50 mm / 1 m
Tub drenazhi Ø50 mm / 1 m
Perforated drainage pipe
Tub drenazhi i perforuar
63.5
75
+2.200
+1.600
60
100
40
20
10
Blinding concrete C12/15
Nenshtrese betoni C12/15
280
60
60
40
360
400

Project	Spreadsheets to BS 8110etc	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Section 1 - 1	GL	31-Jul-2019	101
	RETAINING WALL design to BS 8110:1997, BS 8002:1994. BS 8004:19	Checked	Revision	Job No
	Originated from 'RCC62.xls' on CD © 1999 BCA for RCC	GL	-	-

IDEALISED STRUCTURE and FORCE DIAGRAMS

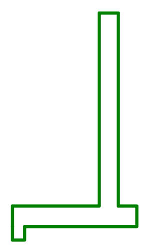
DESIGN STATUS:

Passive pressure should only be considered if it can be guaranteed that there will be no future excavation in front of the wall.



DIMENSIONS (mm)

H =	6300	B =	4000	Tw =	600
Hw =	0	Bl =	2800	Tb =	600
Hp =	1200	BN =	0	TN =	400
Hn =	400				



Wall Geometry

MATERIAL PROPERTIES

fcu =	30	N/mm²	γm =	1.5	concrete
fy =	500	N/mm²	γm =	1.05	steel

cover to tension steel = 35 mm
Max allowable design surface crack width (W) = 0.3 mm
Concrete density = 24 kN/m³

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (Ø) = 30 degree
Design cohesion of retained mat'l (C) = 0 kN/m² (Only granular backfil considered, "C" = zero)
Density of retained mat'l (q) = 18.5 kN/m³
Submerged Density of retained mat'l (qs) = 12.33 kN/m³ [default=2/3*q (only apply when 12.33
Design angle of int'l friction of base mat'l (Øb) = 20 degree
Design cohesion of base material (Cb) = 25 kN/m²
Density of base material (qb) = 10 kN/m³
Allowable gross ground bearing pressure (GBP) = 150 kN/m²

ASSUMPTIONS
a) Wall friction is zero
b) Minimum active earth pressure = 0.25qH
c) Granular backfill
d)Does **not** include check of rotational slide/slo
e)Does **not** include effect of seepage of ground water beneath the wall.
f)Does **not** include deflection check of wall due lateral earth pressures
h) Design not intended for walls over 3.0 m hig

LOADINGS

Surcharge load -- live (SQK) = 10 kN/m²
Surcharge load -- dead (SGK) = 0
Line load -- live (LQK) = 0 kN/m
Line load -- dead (LGK) = 0 kN/m
Distance of line load from wall (X) = 0 mm

LATERAL FORCES (unfactored)

Ka = 0.33 [default ka = (1-SIN Ø)/(1+SIN Ø0.33
Kp = 2.04 [default kp = (1+SIN Øb)/(1-SIN2.04
Kpc = 2.86 [default kpc = $2kp^{0.5}$] = 2.86
Kac = 1.15 [$2ka^{0.5}$]

	Force (kN)	Lever arm (m)	Moment about TOE (kNm)	γ_f	F_{ult} (kN)	M_{ult} (kNm)
PE =	122.38	LE = 2.100	256.99	1.35	165.21	346.94
PS(GK) =	0.00	LS = 3.15	0.00	1.35	0.00	0.00
PS(QK) =	10.50	LS = 3.15	33.07	1.50	15.75	49.61
PL(GK) =	0.00	LL = 6.30	0.00	1.35	0.00	0.00
PL(QK) =	0.00	LL = 6.30	0.00	1.50	0.00	0.00
PW =	0.00	LW = 0.00	0.00	1.35	0.00	0.00
Total	132.88		290.07		180.96	396.55
PP =	-100.37	(LP-HN) = 0.17	-17.14	1.00	-100.37	-17.14

EXTERNAL STABILITY

STABILITY CHECKS : OK

OVERTURNING about TOE   F.O.S = 1.50

(using overall factor of safety instead of partial safety factor)

LOADING OPTION
(select critical load combination

EARTH
☑ PS(GK)
☑ PS(QK)
☑ PL(GK)
☑ PL(QK)
☑ PW

Overturning Moments

Lateral FORCE (kN)	Lever arm (m)	Moment (kNm)
PE = 122.38	LE = 2.10	256.99
PS(GK) = 0.00	LS = 3.15	0.00
PS(QK) = 10.50	LS = 3.15	33.07
PL(GK) = 0.00	LL = 6.30	0.00
PL(QK) = 0.00	LL = 6.30	0.00
PW = 0.00	LW = 0.00	0.00
∑ **P =** 132.88		
Pp = -100.37	(LP-HN) = 0.17	-17.14
		∑ **Mo =** 272.93

Under some conditions,surcharge & line loads may have stabilising effects on the structure, and it is recommended that stability checks should also be carried out without these loads.

Restoring Moments

Vertical FORCE (kN)	Lever arm (m)	Moment (kNm)
Wall = 82.08	3.10	254.45
Base = 57.60	2.00	115.20
Nib = 3.84	0.20	0.77
Earth = 63.27	3.70	234.10
Water = 0.00	3.70	0.00
Surcharge = 3.00	3.70	11.10
Line load = 0.00	3.40	0.00
∑ **V =** 209.79		∑ **Mr =** 615.62

Factor of Safety, Mr / Mo = 2.26 > 1.50 OK

SLIDING (using overall factor of safety instead of partial safety factor) F.O.S = 1.50

Sum of LATERAL FORCES, **P** = 132.88 kN

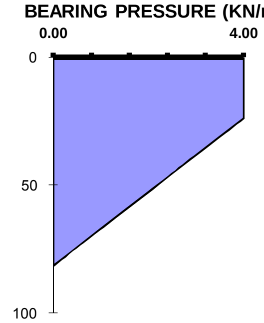
ASSIVE FORCE, Pp x Reduction factor (1) = -100.37 kN   Red'n factor for passive force = 1.00
BASE FRICTION (∑ **V** TANØb + B Cb) = -176.36 kN
Sum of FORCES RESISTING SLIDING,**Pr** = -276.73 kN

Factor of Safety, Pr / P = 2.08 > 1.50 OK

GROUND BEARING Taking moments about centre of base (anticlockwise "+") :

Vertical FORCES (kN)	Lever arm (m	Moment (kNm)
Wall = 82.08	-1.10	-90.29
Base = 57.60	0.00	0.00
Nib = 3.84	1.80	6.91
Earth = 63.27	-1.70	-107.56
Water = 0.00	-1.70	0.00
Surcharge= 3.00	-1.70	-5.10
Line load = 0.00	-1.40	0.00
∑ **V =** 209.79		∑ **Mv =** -196.04

BEARING PRESSURE (KN/m²)



Moment due to LATERAL FORCES, **Mo** = 272.93 kNm

Resultant Moment, **M** = **Mv** + **Mo** = 76.89 kNm

Eccentricity from base centre, **M / V** = 0.37 m

Therefore, MAXIMUM Gross Bearing Pressure **GRP**) = 81 kN/m² < 150 OK

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Retaining wall Section 1 - 1

Design according Eurocode 2

SECTION DATA:

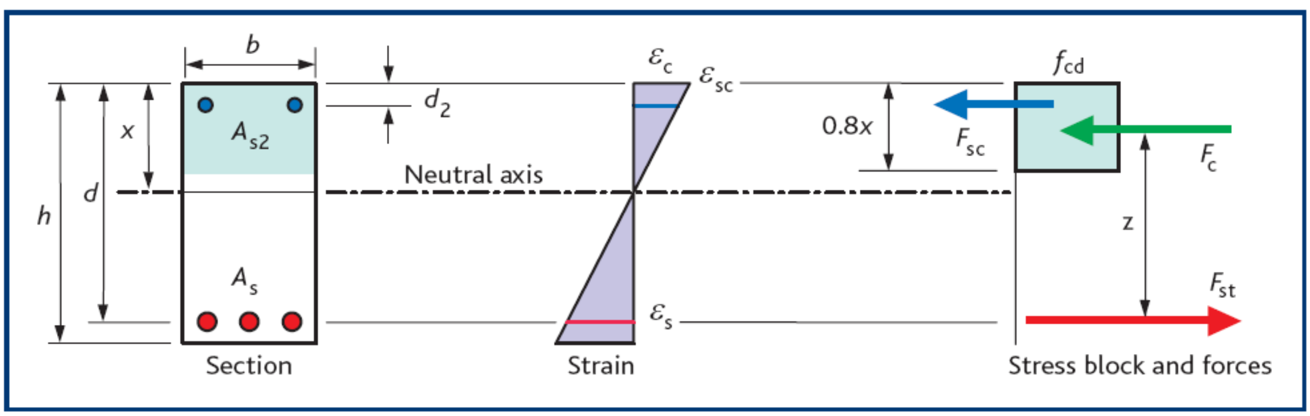
$h := 60$	cm	*height of section;*
$b := 100$	cm	*width of section;*
$a := 5$	cm	*cover to reinforcement;*
$d_{t.bar} := 2$	cm	*tension bar diameter;*
$d_{c.bar} := 1.4$	cm	*compression bar diameter;*
$d_{link} := 1.2$	cm	*link diameter;*
$d = 52.8$	cm	*effective depth of the beam;*
$d' = 6.9$	cm	*depth of centroid of compression steel;*

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	*characteristic cylinder strength of concrete;*
$f_{ctm} = 29$	kg / cm^2	*mean axial tensile strength of concrete;*
$f_{yk} := 5000$	kg / cm^2	*characteristic yield strength of steel;*
$f_{sck} = 4830$	kg / cm^2	*characteristic yield strength of compressed steel considering the displaced concrete;*

LOADING DATA:

$M := 39.6$	ton*m	*Design Moment;*
$\delta := 1$		*Moment distribution factor;*



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.047 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25} \right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}} \right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}} \right) \right] = 50.49\,cm \; < \; z_{max} := 0.95 \cdot d = 50.16\,cm$$

$$x := \frac{(d - z)}{0.4} = 5.76 \quad cm \qquad \text{depth of neutral axis;}$$

Compression steel:

$$\frac{d'}{d} = 0.131 \quad < \quad 0.171 \quad \text{compression steel will have yielded;}$$

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')} \right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}} \right)$$

$$A_s = 18.03 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.34 \;\; \%$$

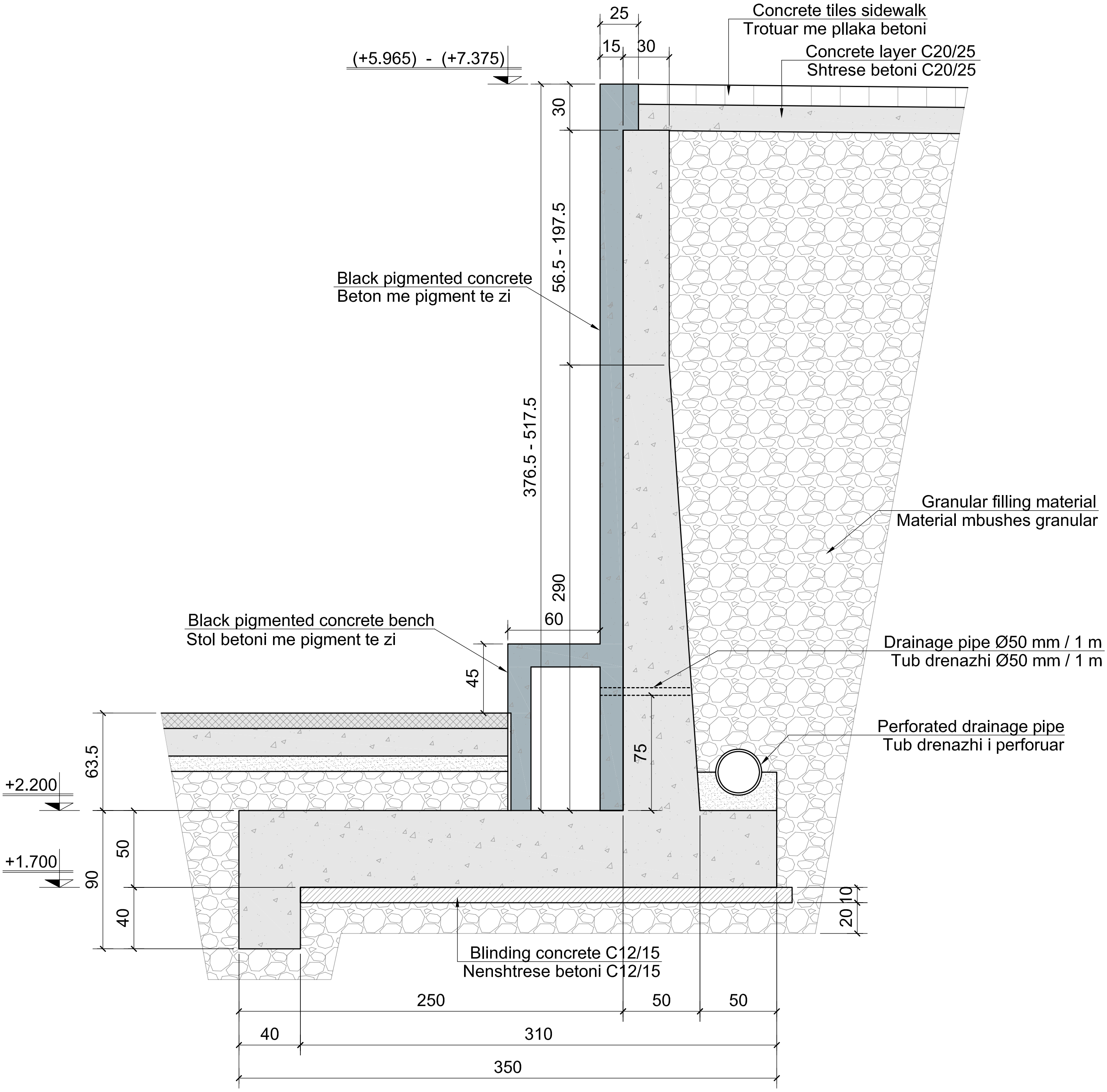
$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 7.95 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 7.8\,cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 211.2 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^{\;2}}{4}} = 5.74 \quad bars$$

SECTION 2 - 2



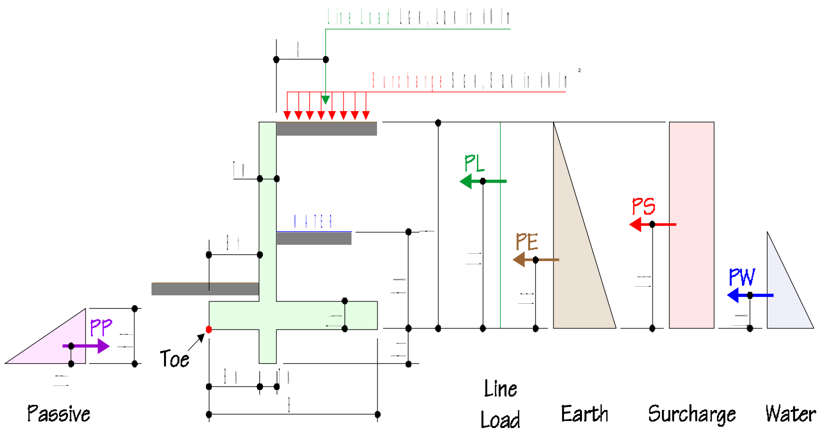
25
15
30
(+5.965) - (+7.375)
Concrete tiles sidewalk
Trotuar me pllaka betoni
Concrete layer C20/25
Shtrese betoni C20/25
30
56.5 - 197.5
376.5 - 517.5
290
Black pigmented concrete
Beton me pigment te zi
Granular filling material
Material mbushes granular
Black pigmented concrete bench
Stol betoni me pigment te zi
60
45
Drainage pipe Ø50 mm / 1 m
Tub drenazhi Ø50 mm / 1 m
Perforated drainage pipe
Tub drenazhi i perforuar
75
63.5
+2.200
+1.700
50
90
40
10
20
Blinding concrete C12/15
Nenshtrese betoni C12/15
250
50
50
40
310
350

Project	Spreadsheets to BS 8110etc	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Section 2 - 2	GL	31-Jul-2019	101
	RETAINING WALL design to BS 8110:1997, BS 8002:1994. BS 8004:19	Checked	Revision	Job No
	Originated from 'RCC62.xls' on CD © 1999 BCA for RCC	GL	-	-

IDEALISED STRUCTURE and FORCE DIAGRAMS

DESIGN STATUS:

Passive pressure should only be considered if it can be guaranteed that there will be no future excavation in front of the wall.



DIMENSIONS (mm)

H =	5800	B =	3500	Tw =	500
Hw =	0	Bl =	2500	Tb =	500
Hp =	1200	BN =	0	TN =	400
Hn =	400				

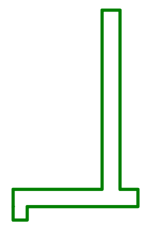
MATERIAL PROPERTIES

fcu =	30	N/mm²	γm =	1.5	concrete
fy =	500	N/mm²	γm =	1.05	steel

cover to tension steel = 35 mm

Max allowable design surface crack width (W) = 0.3 mm

Concrete density = 24 kN/m³



Wall Geometry

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (Ø) =	30	degree
Design cohesion of retained mat'l (C) =	0	kN/m²
Density of retained mat'l (q) =	18.5	kN/m³
Submerged Density of retained mat'l (qs) =	12.33	kN/m³
Design angle of int'l friction of base mat'l (Øb) =	20	degree
Design cohesion of base material (Cb) =	25	kN/m²
Density of base material (qb) =	10	kN/m³
Allowable gross ground bearing pressure (GBP) =	150	kN/m²

(Only granular backfil considered, "C" = zero)

[default=2/3*q (only apply when 12.33

ASSUMPTIONS

a) Wall friction is zero

b) Minimum active earth pressure = 0.25qH

c) Granular backfill

d)Does **not** include check of rotational slide/slc

e)Does **not** include effect of seepage of ground water beneath the wall.

f)Does **not** include deflection check of wall due lateral earth pressures

h) Design not intended for walls over 3.0 m hig

LOADINGS

Surcharge load -- live (SQK) =	10	kN/m²
Surcharge load -- dead (SGK) =	0	
Line load -- live (LQK) =	0	kN/m
Line load -- dead (LGK) =	0	kN/m
Distance of line load from wall (X) =	0	mm

LATERAL FORCES (unfactored)

Ka =	0.33	[default ka = (1-SIN Ø)/(1+SIN Ø 0.33
Kp =	2.04	[default kp = (1+SIN Øb)/(1-SIN 2.04
Kpc =	2.86	[default kpc = $2kp^{0.5}$] = 2.86
Kac =	1.15	[$2ka^{0.5}$]

	Force (kN)	Lever arm (m)	Moment about TOE (kNm)	γ_f	F_{ult} (kN)	M_{ult} (kNm)
PE =	103.72	LE = 1.933	200.53	1.35	140.03	270.72
PS(GK) =	0.00	LS = 2.90	0.00	1.35	0.00	0.00
PS(QK) =	9.67	LS = 2.90	28.03	1.50	14.50	42.05
PL(GK) =	0.00	LL = 5.80	0.00	1.35	0.00	0.00
PL(QK) =	0.00	LL = 5.80	0.00	1.50	0.00	0.00
PW =	0.00	LW = 0.00	0.00	1.35	0.00	0.00
Total	113.39		228.56		154.53	312.77
PP =	-100.37	(LP-HN) = 0.17	-17.14	1.00	-100.37	-17.14

EXTERNAL STABILITY

STABILITY CHECKS : OK

OVERTURNING about TOE

F.O.S = 1.50

(using overall factor of safety instead of partial safety factor)

LOADING OPTION

(select critical load combination

EARTH

☑ PS(GK)
☑ PS(QK)
☑ PL(GK)
☑ PL(QK)
☑ PW

Overturning Moments

Lateral FORCE (kN)	Lever arm (m)	Moment (kNm)
PE = 103.72	LE = 1.93	200.53
PS(GK) = 0.00	LS = 2.90	0.00
PS(QK) = 9.67	LS = 2.90	28.03
PL(GK) = 0.00	LL = 5.80	0.00
PL(QK) = 0.00	LL = 5.80	0.00
PW = 0.00	LW = 0.00	0.00
Σ **P** = 113.39		
Pp = -100.37	(LP-HN) = 0.17	-17.14
		Σ **Mo** = 211.43

Under some conditions,surcharge & line loads may have stabilising effects on the structure, and it is recommended that stability checks should also be carried out without these loads.

Restoring Moments

Vertical FORCE (kN)	Lever arm (m)	Moment (kNm)
Wall = 63.60	2.75	174.90
Base = 42.00	1.75	73.50
Nib = 3.84	0.20	0.77
Earth = 49.03	3.25	159.33
Water = 0.00	3.25	0.00
Surcharge = 2.50	3.25	8.13
Line load = 0.00	3.00	0.00
Σ **V** = 160.97		Σ **Mr** = 416.62

Factor of Safety, Mr / Mo = 1.97 > 1.50 OK

SLIDING

(using overall factor of safety instead of partial safety factor) F.O.S = 1.50

Sum of LATERAL FORCES, **P** = 113.39 kN

ASSIVE FORCE, Pp x Reduction factor (1) = -100.37 kN Red'n factor for passive force = 1.00

BASE FRICTION (Σ **V** TANØb + B Cb) = -146.09 kN

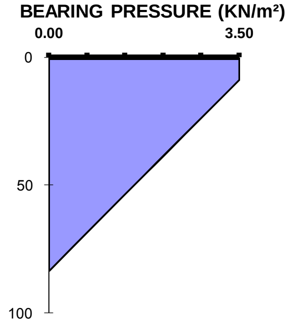
Sum of FORCES RESISTING SLIDING,**Pr** = -246.46 kN

Factor of Safety, Pr / P = 2.17 > 1.50 OK

GROUND BEARING

Taking moments about centre of base (anticlockwise "+") :

Vertical FORCES (kN)	Lever arm (m	Moment (kNm)
Wall = 63.60	-1.00	-63.60
Base = 42.00	0.00	0.00
Nib = 3.84	1.55	5.95
Earth = 49.03	-1.50	-73.54
Water = 0.00	-1.50	0.00
Surcharge= 2.50	-1.50	-3.75
Line load = 0.00	-1.25	0.00
Σ **V** = 160.97		Σ **Mv** = -134.94



Moment due to LATERAL FORCES, **Mo** = 211.43 kNm

Resultant Moment, **M** = **Mv** + **Mo** = 76.49 kNm

Eccentricity from base centre, **M** / **V** = 0.48 m

Therefore, MAXIMUM Gross Bearing Pressure **GRP**) = 83 kN/m² < 150 OK

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Retaining wall Section 2 - 2

Design according Eurocode 2

SECTION DATA:

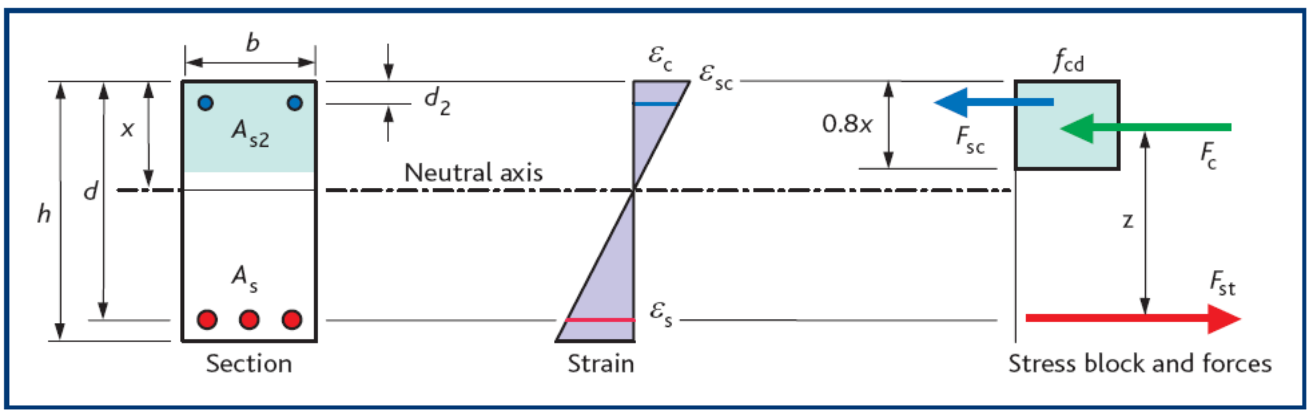
$h := 50$	cm	height of section;
$b := 100$	cm	width of section;
$a := 5$	cm	cover to reinforcement;
$d_{t.bar} := 1.8$	cm	tension bar diameter;
$d_{c.bar} := 1.4$	cm	compression bar diameter;
$d_{link} := 1.2$	cm	link diameter;
$d = 42.9$	cm	effective depth of the beam;
$d' = 6.9$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 31.3$	ton*m	Design Moment;
$\delta := 1$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.057 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182\left(\frac{\delta - 0.44}{1.25}\right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}}\right)\right] = 40.64\ cm \ < \ z_{max} := 0.95 \cdot d = 40.75\ cm$$

$$x := \frac{(d - z)}{0.4} = 5.66 \quad cm$$ depth of neutral axis;

Compression steel:

$$\frac{d'}{d} = 0.161 \quad < \quad 0.171$$ compression steel will have yielded;

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')}\right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}}\right)$$

$$A_s = 17.71 \ cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.41 \ \%$$

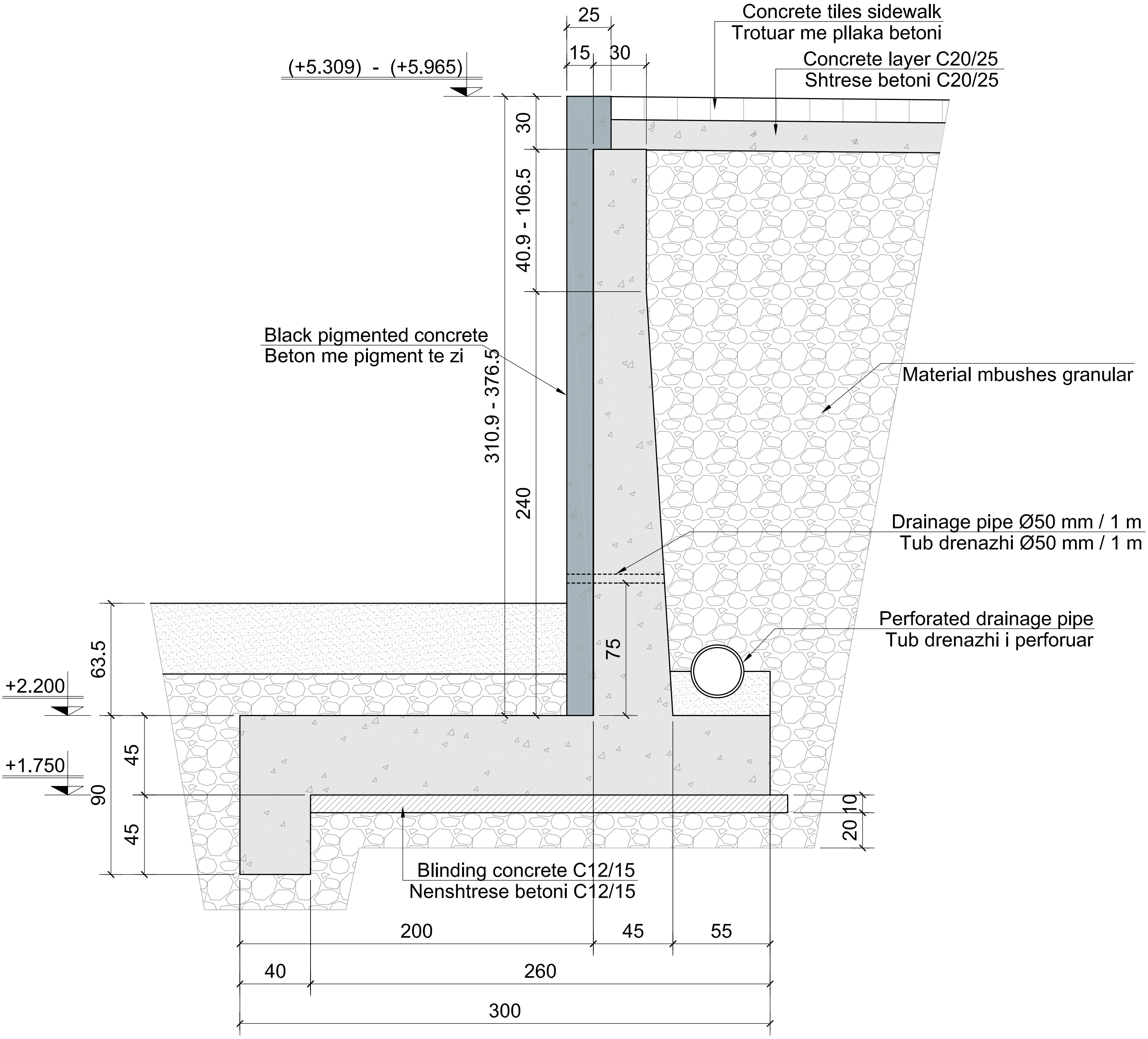
$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 6.46 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 6.5\ cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 171.6 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^2}{4}} = 6.96 \quad bars$$

SECTION 3 - 3



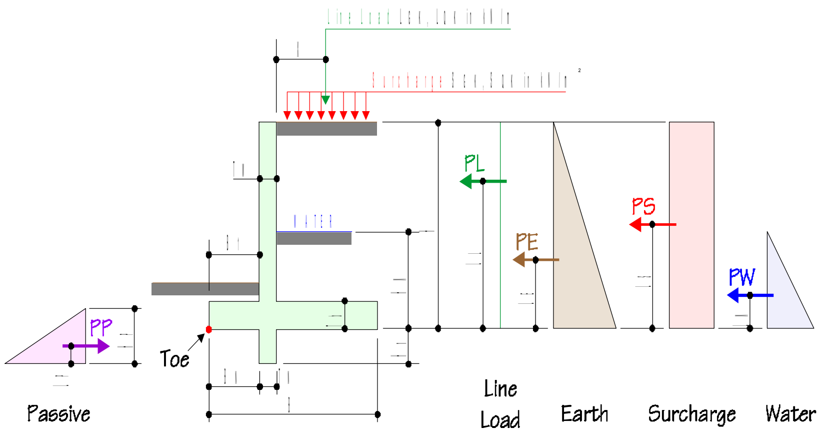
25
15
30
(+5.309) - (+5.965)
Concrete tiles sidewalk
Trotuar me pllaka betoni
Concrete layer C20/25
Shtrese betoni C20/25
30
40.9 - 106.5
310.9 - 376.5
240
Black pigmented concrete
Beton me pigment te zi
Material mbushes granular
Drainage pipe Ø50 mm / 1 m
Tub drenazhi Ø50 mm / 1 m
Perforated drainage pipe
Tub drenazhi i perforuar
75
63.5
+2.200
45
90
45
+1.750
10
20
Blinding concrete C12/15
Nenshtrese betoni C12/15
200
45
55
40
260
300

Project	Spreadsheets to BS 8110etc		REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page	
Location	Section 3 - 3	GL	31-Jul-2019	101	
	RETAINING WALL design to BS 8110:1997, BS 8002:1994. BS 8004:19	Checked	Revision	Job No	
	Originated from 'RCC62.xls' on CD © 1999 BCA for RCC	GL	-	-	

IDEALISED STRUCTURE and FORCE DIAGRAMS

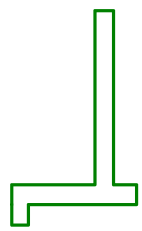
DESIGN STATUS:

Passive pressure should only be considered if it can be guaranteed that there will be no future excavation in front of the wall.



DIMENSIONS (mm)

H =	4300	B =	3000	Tw =	450
Hw =	0	Bl =	2000	Tb =	450
Hp =	1200	BN =	0	TN =	400
Hn =	450				



Wall Geometry

MATERIAL PROPERTIES

fcu =	30	N/mm²	γm =	1.5	concrete
fy =	500	N/mm²	γm =	1.05	steel

cover to tension steel = 35 mm

Max allowable design surface crack width (W) = 0.3 mm

Concrete density = 24 kN/m³

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (Ø) =	30	degree	
Design cohesion of retained mat'l (C) =	0	kN/m²	(Only granular backfil considered, "C" = zero)
Density of retained mat'l (q) =	18.5	kN/m³	
Submerged Density of retained mat'l (qs) =	12.33	kN/m³	[default=2/3*q (only apply when 12.33
Design angle of int'l friction of base mat'l (Øb) =	20	degree	ASSUMPTIONS
Design cohesion of base material (Cb) =	25	kN/m²	a) Wall friction is zero
Density of base material (qb) =	10	kN/m³	b) Minimum active earth pressure = 0.25qH
Allowable gross ground bearing pressure (GBP) =	150	kN/m²	c) Granular backfill

d)Does **not** include check of rotational slide/slc

LOADINGS Surcharge load -- live (SQK) =	10	kN/m²	e)Does **not** include effect of seepage of ground water beneath the wall.
Surcharge load -- dead (SGK) =	0		
Line load -- live (LQK) =	0	kN/m	f)Does **not** include deflection check of wall due lateral earth pressures
Line load -- dead (LGK) =	0	kN/m	
Distance of line load from wall (X) =	0	mm	h) Design not intended for walls over 3.0 m hig

LATERAL FORCES (unfactored)

Ka =	0.33	[default ka = (1-SIN Ø)/(1+SIN Ø0.33
Kp =	2.04	[default kp = (1+SIN Øb)/(1-SIN2.04
Kpc =	2.86	[default kpc = $2kp^{0.5}$] = 2.86
Kac =	1.15	[$2ka^{0.5}$]

	Force (kN)	Lever arm (m)	Moment about TOE (kNm)	γ_f	F_{ult} (kN)	M_{ult} (kNm)
PE =	57.01	LE = 1.433	81.72	1.35	76.96	110.32
PS(GK) =	0.00	LS = 2.15	0.00	1.35	0.00	0.00
PS(QK) =	7.17	LS = 2.15	15.41	1.50	10.75	23.11
PL(GK) =	0.00	LL = 4.30	0.00	1.35	0.00	0.00
PL(QK) =	0.00	LL = 4.30	0.00	1.50	0.00	0.00
PW =	0.00	LW = 0.00	0.00	1.35	0.00	0.00
Total	64.18		97.12		87.71	133.43
PP =	-100.37	(LP-HN) = 0.12	-12.12	1.00	-100.37	-12.12

EXTERNAL STABILITY

STABILITY CHECKS : OK

OVERTURNING about TOE F.O.S = 1.50

(using overall factor of safety instead of partial safety factor)

LOADING OPTION

(select critical load combination

EARTH

☑ PS(GK)
☑ PS(QK)
☑ PL(GK)
☑ PL(QK)
☑ PW

Overturning Moments

Lateral FORCE (kN)	Lever arm (m)	Moment (kNm)
PE = 57.01	LE = 1.43	81.72
PS(GK) = 0.00	LS = 2.15	0.00
PS(QK) = 7.17	LS = 2.15	15.41
PL(GK) = 0.00	LL = 4.30	0.00
PL(QK) = 0.00	LL = 4.30	0.00
PW = 0.00	LW = 0.00	0.00
∑ **P** = 64.18		
Pp = -100.37	(LP-HN) = 0.12	-12.12
		∑ **Mo** = 85.00

Under some conditions,surcharge & line loads may have stabilising effects on the structure, and it is recommended that stability checks should also be carried out without these loads.

Restoring Moments

Vertical FORCE (kN)	Lever arm (m)	Moment (kNm)
Wall = 41.58	2.23	92.52
Base = 32.40	1.50	48.60
Nib = 4.32	0.20	0.86
Earth = 39.17	2.73	106.75
Water = 0.00	2.73	0.00
Surcharge = 2.75	2.73	7.49
Line load = 0.00	2.45	0.00
∑ **V** = 120.22		∑ **Mr** = 256.22

Factor of Safety, Mr / Mo = 3.01 > 1.50 OK

SLIDING (using overall factor of safety instead of partial safety factor) F.O.S = 1.50

Sum of LATERAL FORCES, **P** = 64.18 kN

ASSIVE FORCE, Pp x Reduction factor (1) = -100.37 kN Red'n factor for passive force = 1.00

BASE FRICTION (∑ **V** TANØb + B Cb) = -118.76 kN

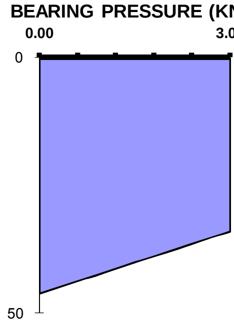
Sum of FORCES RESISTING SLIDING,**Pr** = -219.13 kN

Factor of Safety, Pr / P = 3.41 > 1.50 OK

GROUND BEARING FTaking moments about centre of base (anticlockwise "+") :

Vertical FORCES (kN)	Lever arm (m	Moment (kNm)
Wall = 41.58	-0.73	-30.15
Base = 32.40	0.00	0.00
Nib = 4.32	1.30	5.62
Earth = 39.17	-1.23	-47.99
Water = 0.00	-1.23	0.00
Surcharge= 2.75	-1.23	-3.37
Line load = 0.00	-0.95	0.00
∑ **V** = 120.22		∑ **Mv** = -75.89

BEARING PRESSURE (KN/m²)



Moment due to LATERAL FORCES, **Mo** = 85.00 kNm

Resultant Moment, **M** = **Mv** + **Mo** = 9.12 kNm

Eccentricity from base centre, **M** / **V** = 0.08 m

Therefore, MAXIMUM Gross Bearing Pressure **GRP**) = 46 kN/m² < 150 OK

RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Retaining wall Section 3 - 3

Design according Eurocode 2

SECTION DATA:

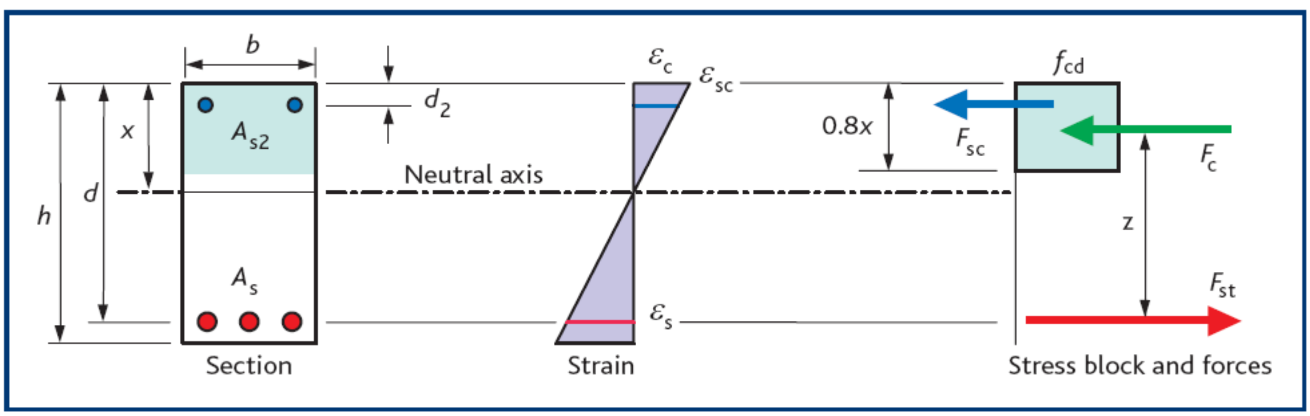
$h := 45$	cm	height of section;
$b := 100$	cm	width of section;
$a := 5$	cm	cover to reinforcement;
$d_{t.bar} := 1.6$	cm	tension bar diameter;
$d_{c.bar} := 1.4$	cm	compression bar diameter;
$d_{link} := 1.2$	cm	link diameter;
$d = 38$	cm	effective depth of the beam;
$d' = 6.9$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 13.4$	ton*m	Design Moment;
$\delta := 1$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.031 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25} \right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}}\right)\right] = 36.93\,cm \; < \; z_{max} := 0.95 \cdot d = 36.10\,cm$$

$$x := \frac{(d - z)}{0.4} = 2.67 \quad cm$$ depth of neutral axis;

Compression steel:

$$\frac{d'}{d} = 0.182 \quad < \quad 0.171$$ compression steel will have yielded;

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')}\right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}}\right)$$

$$A_s = 8.34 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.22 \; \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 5.72 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 5.9\,cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 152 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^2}{4}} = 4.15 \quad bars$$

7. Shetitorja dhe Sheshi me Beton te Furçuar

7.1. Shetitorja

Shtresat e shetitores jane si me poshte:

- Stabilizant – 15 cm;
- Solete b/a (C25/30) – 18 cm
- Beton me pllaka guri (C25/30) – 10 cm

Gjeresia e shetitores eshte 7 m. Soleta b/a do te ndahet ne nyje kontraktimi cdo 5 m.

Armimi do te jete dopio zgare Φ 8/20 cm.

Armimi do te nderpritet ne nyje dhe transferimi i forcave prerese do te behet nepermjet shufrave (Φ16/25 cm). Shtresa e betonit 10 cm me pllaka guri do te jete e vazhduar (pa nyje kontraktimi) duke lejuar krijimin e plasaritjeve ne menyre te cfaredoshme.

Per me shume informacion te shihen fletet K-21, K-22.

7.2. Sheshi me beton te furçuar

Shtresat e sheshit jane si me poshte:

- Stabilizant – 15 cm;
- Solete b/a (C25/30) – 18 cm
- Solete me beton te furçuar (C25/30) – 10 cm

Soleta b/a 18 cm do te ndahet ne nyje kontraktimi cdo 5x5 m.

Armimi do te jete dopio zgare Φ 8/20 cm. Armimi do te nderpritet ne nyje dhe transferimi i forcave prerese do te behet nepermjet shufrave (Φ16/25 cm).

Shtresa e betonit te furçuar 10 cm do te ndahet ne nyje kontraktimi cdo 5x5 m, ne nje linje me nyjet e soletes se meposhmte. Armimi do te jete tek zgare Φ 8/20 cm.

Per me shume informacion te shihen fletet K-23, K-24.

Ne faqet pasardhese jane paraqitur llogaritje te aftesise mbajtese te soletes se dyshemese
Llogaritja eshte bere per mjet (pirun) me ngarkese aksiale 65 kN.

SLAB ON GRADE DESIGN

Design according TR34 - Concrete Industrial Ground Floors

GENERAL SLAB DATA:

$h := 180$	mm	slab height;
$f_{ck} := 25$	Mpa	characteristic cilinder compressive strength;
$f_{cu} := 30$	Mpa	characteristic cube compressive strength;
$f_{cm} := f_{ck} + 8 = 33$	Mpa	mean compressive strength;
$f_{ctm} := 0.3 \cdot f_{ck}^{0.666} = 2.6$	Mpa	mean axial tensile strength;
$f_{ctk.005} := 0.7 \cdot f_{ctm} = 1.8$	Mpa	characteristic axial tensile strength;
$E_{cm} := 22 \cdot \left(\frac{f_{cm}}{10}\right)^{0.3} = 31.5$	Mpa	Secant Modulus of Elasticity;
$\nu := 0.2$		Poisson's ratio;
$k := 0.15$	N/mm3	modulus of subgrade reaction;
$\gamma_c := 1.5$		concrete partial safety factor;

RACK POST DATA:

$a_1 := 150$	mm	length of contact area;
$a_2 := 150$	mm	width of contact area;
$a_s := \left[\frac{(a_1 \cdot a_2)}{\pi}\right]^{0.5} = 84.6$	mm	equivalent contact radius for 1 rack post;
$d_r := 250$	mm	distance between two adjacent rack posts;
$a_d := \left[\frac{[a_1 \cdot (a_2 + d_r)]}{\pi}\right]^{0.5} = 138.2$	mm	equivalent contact radius for 2 rack posts;

MATERIAL HANDLING EQUIPMENT DATA:

$Q_k := 6.5$	ton	axle load;
$\phi := 1.4$		dynamic magnification factor for pneumatic tyres;
$x := 1800$	mm	axle Width;
$a_{t1} := 200$	mm	length of tyre contact;
$a_{t2} := 200$	mm	width of tyre contact;
$a_t := \left(\frac{a_{t1} \cdot a_{t2}}{\pi}\right)^{0.5} = 112.8$	mm	equivalent tyre contact radius;

REINFORCEMENT DATA:

$A_{s.t} := 251$ mm2/ml slab top reinforcement;

$A_{s.b} := 251$ mm2/ml slab bottom reinforcement;

$f_y := 500$ N/mm2 reinforcement yield strength;

$\gamma_s := 1.15$ reinforcement partial safety factor;

$a_r := 40$ mm cover to reinforcement;

$d := h - a_r = 140$ mm slab effective depth;

PARTIAL LOAD SAFETY FACTORS:

Ultimate limit state

$\gamma_v := 1.5$ Variable Actions (rack post loads);

$\gamma_d := 1.6$ Dynamic Actions (MHE tyres);

Serviceability limit state

$\gamma_{sv} := 1$

Concrete shear strength

$$k_1 := \min\left[1 + \left(\frac{200}{d}\right)^{0.5}, 2\right] = 2$$

$$\rho_x := \frac{\left(A_{s.t} + A_{s.b}\right)}{d \cdot 1000} = 0.0036$$ pecentage of reinforcement in x direction;

$$\rho_y := \frac{\left(A_{s.t} + A_{s.b}\right)}{d \cdot 1000} = 0.0036$$ pecentage of reinforcement in y direction;

$$\rho_1 := \sqrt{\rho_x \cdot \rho_y} = 0.0036$$

$$v_{Rd.c.1} := \frac{0.18}{\gamma_c} \cdot k_1 \cdot \left(100 \cdot \rho_1 \cdot f_{ck}\right)^{\frac{1}{3}} = 0.499 \text{ N/mm2}$$ shear strength taking into account the reinforcement;

$$v_{Rd.c.2} := 0.035 \cdot k_1^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} = 0.49 \text{ N/mm2}$$ minimum concrete shear strength;

$$v_{Rd.c} := \max\left(v_{Rd.c.1}, v_{Rd.c.2}\right) = 0.5 \text{ N/mm2}$$ design shear strength;

Plain concrete flexural tensile strength

$$f_{ctk.fl} := \min\left[\left[1 + \left(\frac{200}{h}\right)^{0.5}\right] \cdot f_{ctk.005}, 2 \cdot f_{ctk.005}\right] = 3.6 \text{ N/mm2}$$

Moment per unit length at which the flexural tensile strength of the concrete is reached

Positive Moment Capacity

$$M_p := 0.95 \cdot A_{s.b} \cdot f_y \cdot \frac{d}{\gamma_s \cdot 10^7} = 1.451 \quad ton*m/m$$

Negative Moment Capacity

$$M_n := \frac{f_{ctk.fl}}{\gamma_c \cdot 10^4} \cdot \left(\frac{h^2}{6}\right) = 1.29 \quad ton*m/m$$

Radius of relative stiffness

$$l := \left[\frac{E_{cm} \cdot 1000 \cdot h^3}{12 \cdot \left(1 - \nu^2\right) \cdot k}\right]^{0.25} = 570.9 \quad mm$$

1. SINGLE RACK POST LOAD

a/l condition

$$\frac{a_s}{l} = 0.148$$

1.1. Internal Load Condition

For a/l = 0

$$P_{u.i.s1} := 2 \cdot \pi \left(M_p + M_n\right) = 17.2 \quad ton$$

For a/l > 0.2

$$P_{u.i.s2} := \frac{4 \cdot \pi \left(M_p + M_n\right)}{\left(1 - \frac{a_s}{3 \cdot l}\right)} = 36.2 \quad ton$$

$$P_{u.i.s} := min\left[P_{u.i.s2}, P_{u.i.s1} + \left(P_{u.i.s2} - P_{u.i.s1}\right) \cdot \frac{a_s}{l \cdot 0.2}\right] = 31.3 \quad ton$$

The maximum unfactored single rack post load acting in internal condition is:

$$\boxed{\frac{P_{u.i.s}}{\gamma_v} = 20.88} \quad ton$$

1.2. Free Edge Load Condition

For a/l = 0

$$P_{u.e.s1} := \frac{\pi}{2}\left(M_p + M_n\right) + 2 \cdot M_n = 6.9 \quad ton$$

For a/l > 0.2

$$P_{u.e.s2} := \frac{\pi\left(M_p + M_n\right) + 4 \cdot M_n}{\left(1 - \frac{2 \cdot a_s}{3 \cdot l}\right)} = 15.3 \quad ton$$

$$P_{u.e.s} := min\left[P_{u.e.s2}, P_{u.e.s1} + \left(P_{u.e.s2} - P_{u.e.s1}\right) \cdot \frac{a_s}{l \cdot 0.2}\right] = 13.1 \quad ton$$

The maximum unfactored single rack post load acting in free edge condition is:

$$\boxed{\frac{P_{u.e.s}}{\gamma_v} = 8.74} \quad ton$$

1.3. Free Corner Load Condition

For a/l = 0

$$P_{u.c.s1} := 2 \cdot M_n = 2.6 \quad ton$$

For a/l > 0.2

$$P_{u.c.s2} := \frac{4 \cdot M_n}{\left(1 - \frac{a_s}{l}\right)} = 6.1 \quad ton$$

$$P_{u.c.s} := min\left[P_{u.c.s2}, P_{u.c.s1} + \left(P_{u.c.s2} - P_{u.c.s1}\right) \cdot \frac{a_s}{l \cdot 0.2}\right] = 5.2 \quad ton$$

The maximum unfactored single rack post load acting in free corner condition is:

$$\boxed{\frac{P_{u.c.s}}{\gamma_v} = 3.44} \quad ton$$

2. DOUBLE RACK POST LOAD

a/l condition

$$\frac{a_d}{l} = 0.242$$

2.1. Internal Load Condition

For a/l = 0

$$P_{u.i.d1} := 2\cdot\pi\left(M_p + M_n\right) = 17.2 \quad ton$$

For a/l > 0.2

$$P_{u.i.d2} := \frac{4\cdot\pi\left(M_p + M_n\right)}{\left(1 - \frac{a_d}{3\cdot l}\right)} = 37.5 \quad ton$$

$$P_{u.i.d} := min\left[P_{u.i.d2}, P_{u.i.d1} + \left(P_{u.i.d2} - P_{u.i.d1}\right)\cdot\frac{a_d}{l\cdot 0.2}\right] = 37.5 \quad ton$$

The maximum unfactored rack post load for double post acting in internal condition is:

$$\boxed{\frac{P_{u.i.d}}{2\cdot\gamma_v} = 12.49} \quad ton$$

2.2. Free Edge Load Condition

For a/l = 0

$$P_{u.e.d1} := \frac{\pi}{2}\left(M_p + M_n\right) + 2\cdot M_n = 6.9 \quad ton$$

For a/l > 0.2

$$P_{u.e.d2} := \frac{\pi\left(M_p + M_n\right) + 4\cdot M_n}{\left(1 - \frac{2\cdot a_d}{3\cdot l}\right)} = 16.4 \quad ton$$

$$P_{u.e.d} := min\left[P_{u.e.d2}, P_{u.e.d1} + \left(P_{u.e.d2} - P_{u.e.d1}\right)\cdot\frac{a_d}{l\cdot 0.2}\right] = 16.4 \quad ton$$

The maximum unfactored rack post load for double post acting in free edge condition is:

$$\boxed{\frac{P_{u.e.d}}{2\cdot\gamma_v} = 5.47} \quad ton$$

2.3. Free Corner Load Condition

For a/l = 0

$$P_{u.c.d1} := 2 \cdot M_n = \blacksquare \quad ton$$

For a/l > 0.2

$$P_{u.c.d2} := \frac{4 \cdot M_n}{\left(1 - \frac{a_d}{l}\right)} = 6.8 \quad ton$$

$$P_{u.c.d} := min\left[P_{u.c.d2}, P_{u.c.d1} + \left(P_{u.c.d2} - P_{u.c.d1}\right) \cdot \frac{a_d}{l \cdot 0.2}\right] = 6.8 \quad ton$$

The maximum unfactored single rack post load acting in free corner condition is:

$$\boxed{\frac{P_{u.c.d}}{2 \cdot \gamma_v} = 2.27} \quad ton$$

3. PUNCHING CHECK FOR SINGLE RACK POST

3.1. Internal Load Condition

Punching Shear Perimeter: $u_{i.s} := 2 \cdot \left(a_1 + a_2 + 8 \cdot d\right) = 2840$ mm

Punching Capacity: $P_{p.i.s} := u_{i.s} \cdot d \cdot v_{Rd.c} \cdot 10^{-4} = 19.8$ ton

Unfactored rack post load: $\boxed{\frac{P_{p.i.s}}{\gamma_v} = 13.2}$ ton

3.2. Edge Load Condition

Punching Shear Perimeter: $u_{e.s} := a_1 + 4 \cdot d + 2a_2 + 4 \cdot d = 1570$ mm

Punching Capacity: $P_{p.e.s} := u_{e.s} \cdot d \cdot v_{Rd.c} \cdot 10^{-4} = 11$ ton

Unfactored rack post load: $\boxed{\frac{P_{p.e.s}}{\gamma_v} = 7.3}$ ton

3.3. Corner Load Condition

Punching Shear Perimeter: $u_{c.s} := a_1 + 2 \cdot d + a_2 + 2 \cdot d = 430 \cdot 2$ mm

Punching Capacity: $P_{p.c.s} := u_{c.s} \cdot d \cdot v_{Rd.c} \cdot 10^{-4} = 6$ ton

Unfactored rack post load: $\boxed{\frac{P_{p.c.s}}{\gamma_v} = 4}$ ton

4. PUNCHING CHECK FOR DOUBLE RACK POST

4.1. Internal Load Condition

Punching Shear Perimeter: $u_{i.d} := 2 \cdot \left(a_1 + a_2 + d_r + 8 \cdot d\right) = 3340$ mm

Punching Capacity: $P_{p.i.d} := u_{i.d} \cdot d \cdot v_{Rd.c} \cdot 10^{-4} = 23.3$ ton

Unfactored rack post load: $\boxed{\dfrac{P_{p.i.d}}{2 \cdot \gamma_v} = 7.8}$ ton

4.2. Edge Load Condition

Punching Shear Perimeter: $u_{e.d} := a_1 + 4 \cdot d + 2a_2 + 4 \cdot d + d_r = 1820$ mm

Punching Capacity: $P_{p.e.d} := u_{e.d} \cdot d \cdot v_{Rd.c} \cdot 10^{-4} = 12.7$ ton

Unfactored rack post load: $\boxed{\dfrac{P_{p.e.d}}{2 \cdot \gamma_v} = 4.2}$ ton

4.3. Corner Load Condition

Punching Shear Perimeter: $u_{c.d} := a_1 + 2 \cdot d + a_2 + 2 \cdot d + d_r = 555 \cdot 2$ mm

Punching Capacity: $P_{p.c.d} := u_{c.d} \cdot d \cdot v_{Rd.c} \cdot 10^{-4} = 7.7$ ton

Unfactored rack post load: $\boxed{\dfrac{P_{p.c.d}}{2 \cdot \gamma_v} = 2.6}$ ton

5. LINE LOADS

$$\lambda := \left(\frac{3k \cdot 10^9}{E_{cm} \cdot h^3}\right)^{0.25} = 1.251 \quad m^{-1}$$

The line load capacity of the slab per unit length is the minimum of:

$P_{lin.p} := 4 \cdot \lambda \cdot M_p = 7.3$ ton/ml $\qquad P_{lin.n} := 4 \cdot \dfrac{\lambda}{0.12} \cdot M_n = 53.8$ ton/ml

$P_{lin} := min\left(P_{lin.p}, P_{lin.n}\right) = 7.3$ ton/ml

The maximum unfactored line load slab capacity is:

$\boxed{\dfrac{P_{lin}}{\gamma_v} = 4.84}$ ton/ml

6. UNIFORMLY DISTRIBUTED LOADS

Critical aisle width:

$$b_{cr} := \frac{\pi}{2 \cdot \lambda} = 1.255 \quad m$$

The uniformly distributed load capacity of the slab per unit area is the minimum of:

$$w_p := \frac{1}{0.161} \cdot \lambda^2 \cdot M_p = 14.1 \quad ton/m2 \qquad\qquad w_n := \frac{1}{0.168} \cdot \lambda^2 \cdot M_n = 12 \quad ton/m2$$

$$w := min\left(w_p , w_n\right) = 12 \quad ton/m2$$

The maximum unfactored distributed load slab capacity is:

$$\frac{w}{\gamma_v} = 8.01 \quad ton/m2$$

Note: The above-calculated load is critical for two adjacent loaded aisles with width $\frac{\pi}{\lambda}$ *spacing* $\frac{\pi}{2 \cdot \lambda}$.

The capacity of the slab to support uniformly distributed loads is higher in other arrangement types.

7. MATERIALS HANDLING EQUIPMENT (FORKLIFT)

Data for the forklift are given from the Client.

Two load conditions are taken into account (internal and edge) since corner load condition is not relevant provided that adequate shear transfer is provided between the slab panels.

The influence of the tyre loaded zones on each other is neglected since the tyre distance is larger than the radius of relative stiffness.

$Q_k = 6.5$	ton	axial load;
$Q_t := \frac{Q_k}{2} = 3.3$	ton	tyre load;
$Q_{Rd.t} := Q_t \cdot \phi \cdot \gamma_d = 7.28$	ton	design tyre load;

a/l condition

$$\frac{a_t}{l} = 0.198$$

7.1. Internal Load Condition

For $a_t/l = 0$

$$P_{f.i1} := 2\cdot\pi\left(M_p + M_n\right) = 17.2 \qquad ton$$

For $a_t/l > 0.2$

$$P_{f.i2} := \frac{4\cdot\pi\left(M_p + M_n\right)}{\left(1 - \frac{a_t}{3\cdot l}\right)} = 36.9 \qquad ton$$

$$P_{f.i} := \min\left[P_{f.i2}, P_{f.i1} + \left(P_{f.i2} - P_{f.i1}\right)\cdot\frac{a_t}{l\cdot 0.2}\right] = 36.6 \qquad ton$$

$P_{f.i} = 36.6$ ton > $Q_{Rd.t} = 7.28$ ton

7.2. Edge Load Condition

For $a_t/l = 0$

$$P_{f.e1} := \frac{\pi}{2}\left(M_p + M_n\right) + 2\cdot M_n = 6.9 \qquad ton$$

For $a_t/l > 0.2$

$$P_{f.e2} := \frac{\pi\left(M_p + M_n\right) + 4\cdot M_n}{\left(1 - \frac{2\cdot a_t}{3\cdot l}\right)} = 15.9 \qquad ton$$

$$P_{f.e} := \min\left[P_{f.e2}, P_{f.e1} + \left(P_{f.e2} - P_{f.e1}\right)\cdot\frac{a_t}{l\cdot 0.2}\right] = 15.8 \qquad ton$$

$P_{f.e} = 15.8$ ton > $Q_{Rd.t} = 7.28$ ton

8. DEFLECTIONS

Deflections are calculated according Westergaard's equation for the three load conditions for 10 ton point load:

$c_i := 0.125$ internal deflection coefficient;

$c_e := 0.442$ edge deflection coefficient;

$c_c := 1$ corner deflection coefficient;

$P := 10$ ton

internal load condition	edge load condition	corner load condition
$\delta_i := c_i\cdot 10^4\cdot\left(\frac{P}{k\cdot l^2}\right) = 0.26$ mm	$\delta_e := c_e\cdot 10^4\cdot\left(\frac{P}{k\cdot l^2}\right) = 0.9$ mm	$\delta_c := c_c\cdot 10^4\cdot\left(\frac{P}{k\cdot l^2}\right) = 2.05$ mm

9. DOWEL SHEAR TRANSFER CAPACITY

$D_d := 16$	mm	*dowel diameter;*
$f_{y.d} := 0.275$	kN/mm^2	*dowel yield strength;*
$f_{y.s} := 0.6 \cdot f_{y.d} = 0.165$	kN/mm^2	*dowel shear strength;*
$x_j := 10$	mm	*maximum expected joint opening;*
$a_{d.sp} := 290$	mm	*dowel spacing;*
$R_{max} := 40$	%	*maximum allowable rate of edge shear trasferred by dowels;*

The connection shear capacity will be limited by the slab bearing capacity and the dowel bending capacity:

$$Z_b := \frac{\pi \cdot D_d^{\,3}}{20} = 643.4 \qquad mm^3$$

dowel plastic section modulus;

$$M_{cb} := Z_b \cdot f_{y.d} = 176.9 \qquad kN \cdot mm^2$$

dowel bending moment capacity;

Solving the equation $\left(\frac{l_{eff}}{2} + x_j\right) \cdot \frac{f_{cu}}{\gamma_c \cdot 10^3} \cdot l_{eff} \cdot D_d - M_{cb} = 0$

$l_{eff} = 26.5$ mm — *effective length of dowel transmiting compression to the slab;*

The shear force afforded by the dowel will be the minimum of:

- *Dowel shear capacity:*

$$P_{s.d} := 0.9 \cdot 3.14 \cdot \frac{D_d^{\,2}}{4} \cdot f_{y.s} = 29.8 \qquad kN$$

- *Dowel shear capacity limited by bearing capacity of the foundation and bending capacity of bolts:*

$$P_{b.d} := \frac{f_{cu}}{\gamma_c \cdot 10^3} \cdot l_{eff} \cdot D_d = 8.46 \qquad kN$$

<u>The shear capacity for 1 dowel is:</u>

$$\boxed{P_d := \frac{\min\left(P_{s.d}, P_{b.d}\right)}{10} = 0.85} \qquad ton$$

10. DOWEL SHEAR TRANSFER RATE ET EDGES AND CORNERS

The loads acting at the continous edges and corners will be transmitted by the action of the dowels in shear to the adjacent slab panels.

*For edge load condition dowels in distance 0.9*l on each side of the load will contribute to the load transmission between two panels. Therefore the load carried by the dowels is:*

$$\Sigma P_{d.s} := P_d \cdot \frac{(2 \cdot 0.9 \cdot l + a_1)}{a_{d.sp}} = 3$$ ton — *For single rack post edge loading;*

$$\Sigma P_{d.d} := P_d \cdot \frac{(2 \cdot 0.9 \cdot l + a_1 + d_r)}{a_{d.sp}} = 3.73$$ ton — *For double rack post edge loading;*

$$\Sigma P_{d.t} := P_d \cdot \frac{(2 \cdot 0.9 \cdot l + a_t)}{a_{d.sp}} = 3.33$$ ton — *For forklit tyre edge loading;*

10.1 Shear transfer rate for single rack post edge load

$$\frac{P_{u.e.s}}{\gamma_v} = 8.7$$ ton — *unfactored single rack post edge capacity;*

$$R_s := \frac{\Sigma P_{d.s} \cdot 100}{\frac{P_{u.e.s}}{\gamma_v} + \Sigma P_{d.s}} = 25.6$$ % — *rate of shear transfer by dowels;*

$$\boxed{R_{d.s} := \min(R_{max}, R_s) = 25.6}$$ % — *accepted rate of shear transfer by dowels;*

The load capacity at continous edges can be increased :

$$\boxed{P_{u.e.s.m} := \frac{P_{u.e.s} \cdot 100}{\gamma_v \cdot (100 - R_{d.s})} = 11.7}$$ ton — *modified unfactored single rack post edge load capacity;*

The load capacity at continous corners can be increased :

$$\boxed{P_{u.c.s.m} := \frac{P_{u.c.s} \cdot 100}{\gamma_v \cdot (100 - R_{d.s}) \cdot 0.5} = 9.2}$$ ton — *modified unfactored single rack post corner load capacity;*

10.2 Shear transfer rate for double rack post edge load

$\frac{P_{u.e.d}}{\gamma_v} = 10.9$ ton unfactored double rack post edge capacity;

$R_d := \frac{\Sigma P_{d.d} \cdot 100}{\frac{P_{u.e.d}}{2 \cdot \gamma_v} + \Sigma P_{d.d}} = 40.5$ % rate of shear transfer by dowels;

$\boxed{R_{d.d} := min\left(R_{max}, R_d\right) = 40}$ % accepted rate of shear transfer by dowels;

The load capacity at continous edges can be increased :

$\boxed{P_{u.e.d.m} := \frac{P_{u.e.d} \cdot 100}{2 \cdot \gamma_v \cdot \left(100 - R_{d.d}\right)} = 9.1}$ ton modified unfactored double rack post edge load capacity;

The load capacity at continous corners can be increased :

$\boxed{P_{u.c.d.m} := \frac{P_{u.c.d} \cdot 100}{2 \cdot \gamma_v \cdot \left(100 - R_{d.d}\right) \cdot 0.5} = 7.6}$ ton modified unfactored double rack post corner load capacity;

10.3 Shear transfer rate for forklift tyre edge action

$\frac{P_{f.e}}{\gamma_d} = 9.8$ ton unfactored tyre action capacity;

$R_t := \frac{\Sigma P_{d.t} \cdot 100}{\frac{P_{f.e}}{\gamma_d} + \Sigma P_{d.t}} = 25.3$ % rate of shear transfer by dowels;

$\boxed{R_{d.t} := min\left(R_{max}, R_t\right) = 25.3}$ % accepted rate of shear transfer by dowels;

The load capacity at continous edges can be increased :

$\boxed{P_{u.e.t.m} := \frac{\frac{P_{f.e}}{\gamma_d} \cdot 100}{\left(100 - R_{d.t}\right)} = 13.2}$ ton modified unfactored tyre edge action capacity;

To the same extent as the flexural capacities will be modified the punching shear capacities

11. SUMMARY TABLE

Single rack post unfactored load capacities

Flexural capacities at continous edge & corner conditions

- Internal Load Condition $P_{i.s.f} = 20.88$ *ton*
- Edge Load Condition $P_{e.s.f} = 11.74$ *ton*
- Corner Load Condition $P_{c.s.f} = 9.24$ *ton*

Flexural capacities at free edge & corner conditions

- Edge Load Condition $P_{e.s.f} = 8.74$ *ton*
- Corner Load Condition $P_{c.s.f} = 3.44$ *ton*

Punching capacities at continous edge & corner conditions

- Internal Load Condition $P_{i.s.p} = 13.22$ *ton*
- Edge Load Condition $P_{e.s.p} = 9.81$ *ton*
- Corner Load Condition $P_{c.s.p} = 9.81$ *ton*

Punching capacities at free edge & corner conditions

- Edge Load Condition $P_{e.s.p} = 7.31$ *ton*
- Corner Load Condition $P_{c.s.p} = 4$ *ton*

Double rack post unfactored load capacities

Flexural capacities at continous edge & corner conditions

- Internal Load Condition $P_{i.d.f} = 12.49$ *ton*
- Edge Load Condition $P_{e.d.f} = 9.12$ *ton*
- Corner Load Condition $P_{c.d.f} = 7.56$ *ton*

Flexural capacities at free edge & corner conditions

- Edge Load Condition $P_{e.d.f} = 5.47$ *ton*
- Corner Load Condition $P_{c.d.f} = 2.27$ *ton*

Punching capacities at continous edge & corner conditions

- Internal Load Condition $P_{i.d.p} = 7.77$ *ton*
- Edge Load Condition $P_{e.d.p} = 7.06$ *ton*
- Corner Load Condition $P_{c.d.p} = 7.06$ *ton*

Punching capacities at free edge & corner conditions

- Edge Load Condition $P_{e.d.p} = 4.23$ *ton*
- Corner Load Condition $P_{c.d.p} = 2.58$ *ton*

Forklift tyre unfactored load capacities

- Internal Load Condition $P_{i.t} = 22.91$ *ton*
- Edge Load Condition $P_{e.t} = 13.18$ *ton*
- Corner Load Condition $P_{c.t} = 13.18$ *ton*

Line load capacity $p_{lin} = 4.84$ *ton/ml*

Uniformly ditributed load capacity $p_{unif} = 8.01$ *ton/m²*

8. Shkalla ne rruge

8.1 Pershkrim i pergjithshem

Shkalla ne rruge lidh trotuarin me shetitoren. Shkalla eshte 11 m e gjate dhe ka gjeresi 4 m.

Soleta do te jete 40 m.

Per me shume informacion te shihet fleta K-17.

8.2 Materialet

Nenshtrese betoni - C12/15;

Soleta dhe parapetet – Beton C30/37;

Çelik armimi – B500b

8.3 Ngarkesat

Ngarkesat e meposhtme jane marre parasysh ne llogaritje:

8.3.1 Pesha vetjake (SW)

Pesha vetjake e elementeve b/a eshte marre parasysh automatikisht nga programi.

8.3.2 Ngarkesa e perkohshme (LL)

Ngarkesat e perkohshme te meposhtme jane marre parasysh ne llogaritje:

LL= 5 kN/m2 (EN 1991-1-1:2002 Category C3);

8.3.3 Sizmika (EQ)

Referoju seksionit 4.3.5.

8.4 Kombinimi i ngarkesave

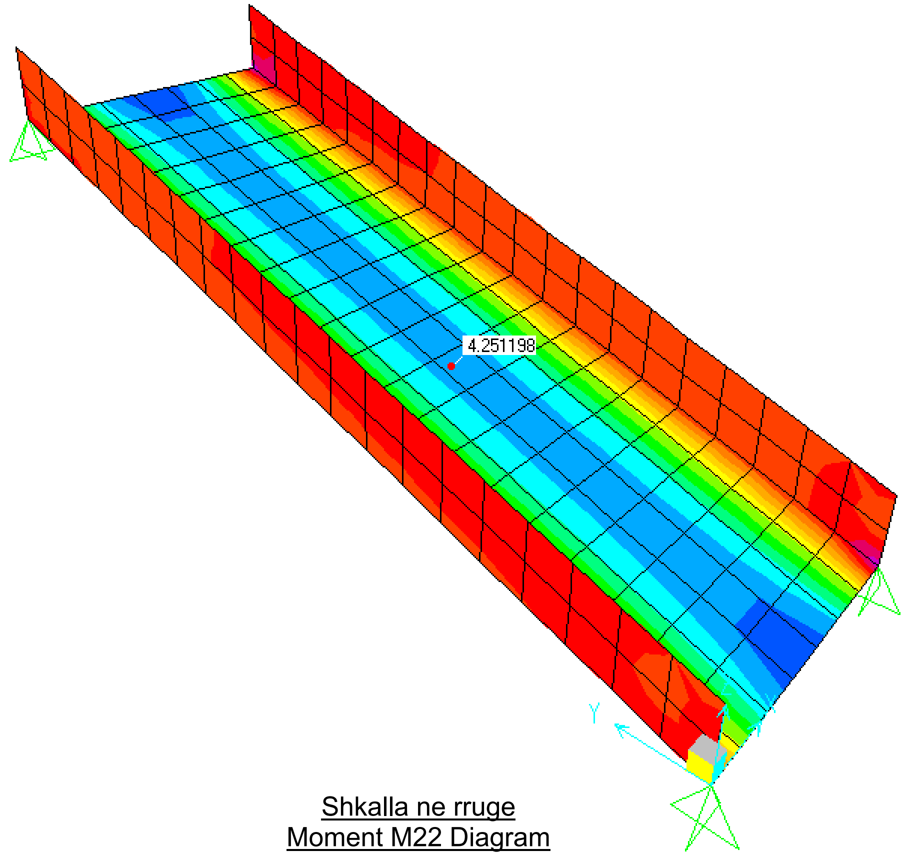
Referoju seksionit 4.4.

8.5 Analiza & rezultate

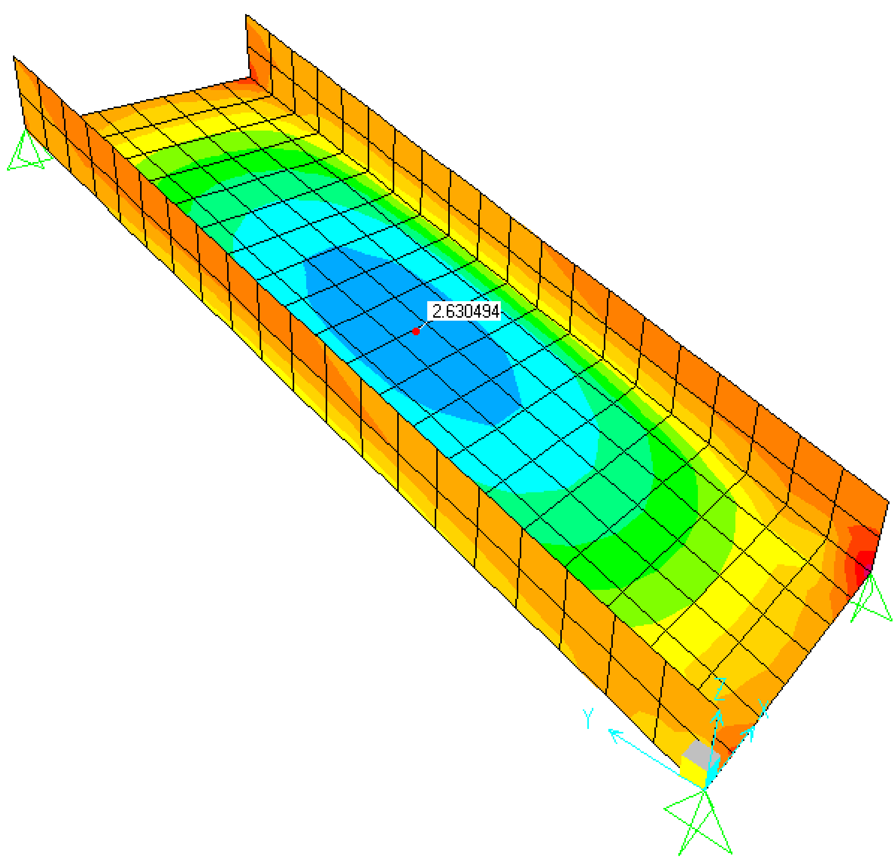
Llogaritja eshte bere me programin me elemente te fundem SAP2000.

Ne faqet pasardhese jane paraqitur diagrama te momenteve per soleten dhe llogaritje te armimit. Armimi eshte bere per forcat maksimale ne solete, duke mos marre parasysh efektin pozitiv ne rritje te aftesise qe kane parapetet.

Shkalla ne rruge
Moment M11 Diagram



Shkalla ne rruge
Moment M22 Diagram



RECTANGULAR SECTION IN BENDING AT ULS - TOP REINFORCEMENT
Stair to road

Design according Eurocode 2

SECTION DATA:

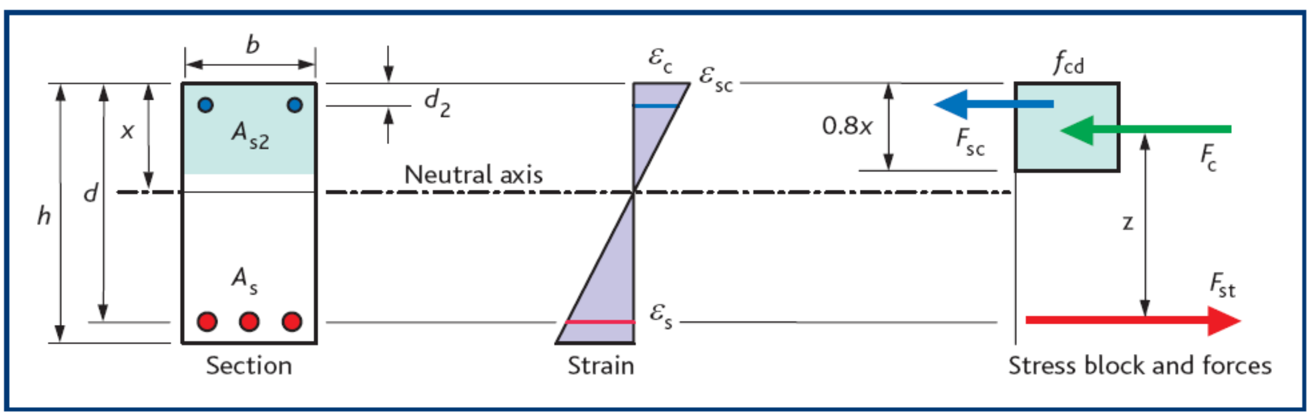
$h := 40$	cm	height of section;
$b := 100$	cm	width of section;
$a := 5$	cm	cover to reinforcement;
$d_{t.bar} := 2$	cm	tension bar diameter;
$d_{c.bar} := 1.4$	cm	compression bar diameter;
$d_{link} := 1.2$	cm	link diameter;
$d = 32.8$	cm	effective depth of the beam;
$d' = 6.9$	cm	depth of centroid of compression steel;

MATERIAL DATA:

$f_{ck} := 300$	kg / cm^2	characteristic cylinder strength of concrete;
$f_{ctm} = 29$	kg / cm^2	mean axial tensile strength of concrete;
$f_{yk} := 5000$	kg / cm^2	characteristic yield strength of steel;
$f_{sck} = 4830$	kg / cm^2	characteristic yield strength of compressed steel considering the displaced concrete;

LOADING DATA:

$M := 29$	ton*m	Design Moment;
$\delta := 1$		Moment distribution factor;



FLEXURAL REINFORCEMENT DESIGN:

$$K := \frac{M \cdot 10^5}{b \cdot d^2 \cdot f_{ck}} = 0.09 \qquad K' := 0.454 \cdot \frac{\delta - 0.44}{1.25} - 0.182 \left(\frac{\delta - 0.44}{1.25} \right)^2 = 0.167$$

Condition = "K < K' - Compression steel is not required"

Lever arm of section:

$$z := if\left[K < K', d \cdot \left(0.5 + \sqrt{0.25 - \frac{K}{1.134}} \right), d \cdot \left(0.5 + \sqrt{0.25 - \frac{K'}{1.134}} \right) \right] = 29.95\, cm \; < \; z_{max} := 0.95 \cdot d = 31.16\, cm$$

$$x := \frac{(d - z)}{0.4} = 7.11 \quad cm$$ *depth of neutral axis;*

Compression steel:

$$\frac{d'}{d} = 0.210 \quad < \quad 0.171$$ *compression steel will have yielded;*

$$A'_s := max\left[0, \frac{(K - K') \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{sck} \cdot (d - d')} \right] = 0.00 \quad cm^2$$

Tension steel:

$$A_s := if\left(K < K', \frac{M \cdot 10^5}{0.87 \cdot f_{yk} \cdot z}, \frac{K' \cdot f_{ck} \cdot b \cdot d^2}{0.87 \cdot f_{yk} \cdot z} + A'_s \cdot \frac{f_{sck}}{f_{yk}} \right)$$

$$A_s = 22.26 \quad cm^2 \qquad \rho := \frac{A_s}{b \cdot d \cdot 10^{-2}} = 0.68 \quad \%$$

$$A_{s.min} := \frac{0.26 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}} = 4.94 \quad cm^2 \qquad A_{s.min2} := 0.0013 \cdot h \cdot b = 5.2\, cm^2$$

$$A_{s.max} := 4 \cdot \frac{b \cdot d}{100} = 131.2 \quad cm^2$$

Required number of tension bars:

$$n_b := \frac{A_s}{\pi \cdot \frac{d_{t.bar}^2}{4}} = 7.08 \quad bars$$

9. Kolonada

9.1. Pershkrim i pergjithshem

Elementet e kolonades do te jene si me poshte:

Trare themeli – 60/50 cm;
Kollonat – 40/40 cm;
Traret – 40/40 cm.

Per me shume informacion te shihen fletet K-18, K-19, K-20.

9.2. Materialet

Nenshtrese betoni - C12/15;
Trare e kollona – Concrete C25/30;
Çelik armimi – B500b

9.3. Ngarkesat

Ngarkesat e meposhtme jane marre parasysh ne llogaritje:

9.3.1 Pesha vetjake (SW)

Pesha vetjake e elementeve b/a eshte marre parasysh automatikisht nga programi.

9.3.2 Ngarkesa e perkohshme (LL)

The following live loads are considered in design:
LL= 5 kN/m2 (EN 1991-1-1:2002 Category C3);

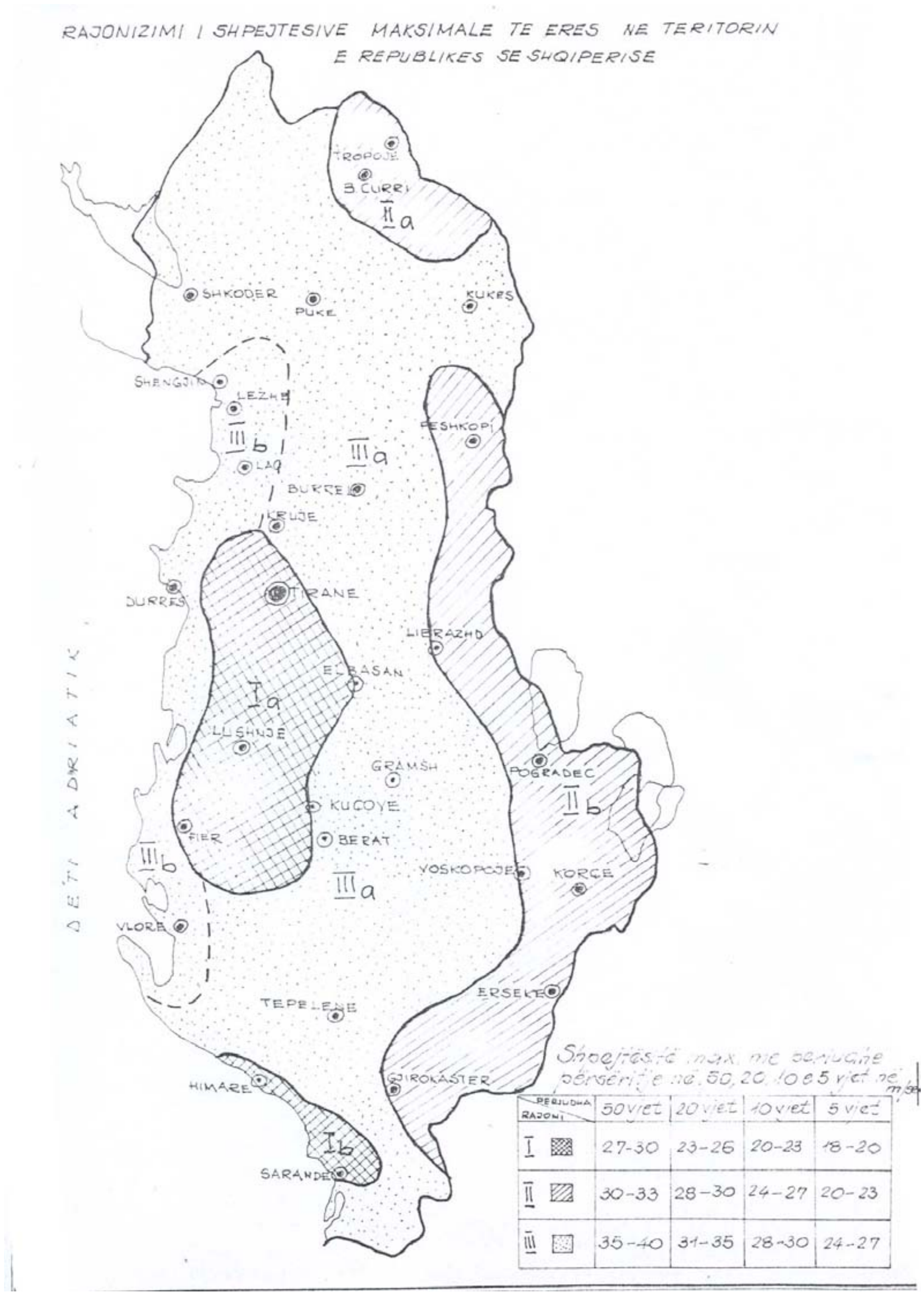
9.3.3 Sizmika (EQ)

Referoju seksionit 4.3.5.

9.3.4 Ngarkesa e eres (WL)

Struktura eshte menduar te mbulohet perkohesisht.
Ngarkesay e eres qe veprojne ne mbulese jane aplikuar sipas EN1991-1-4 Wind Loads.
Vlera karakteristike e shpejtesise se eres eshte marre nga Harta e rajonizimit te shpejtesive maksimale te eres te Institutit Hidrometereologjik.

Vlore – Zona IIIb



RAJONI \ PERIUDHA	50 viet	20 viet	10 viet	5 viet
I	27-30	23-26	20-23	18-20
II	30-33	28-30	24-27	20-23
III	35-40	31-35	28-30	24-27

BASIC WIND VELOCITY

$v_{b.0} := 36$ m/s Fundamental value of the basic wind velocity

$c_{season} := 1$ Seasonal factor

$c_{dir} := 1$ Directional factor

$v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 36$ m/s

TERRAIN ROUGHNESS FACTOR

Terrain Category III

$z_0 := 0.003$ m Roughness length

$z_{min} := 1$ m Minimum height

$z := 3$ m Building effective height

$z_{0.II} := 0.05$ Roughness length for Terrain Category II

$$k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.156$$ Terrain factor

$$c_{r.z} := k_r \cdot \ln\left(\frac{z}{z_0}\right) = 1.078$$ Roughness factor

OROGRAPHY FACTOR

The orography is not significant

$c_{o.z} := 1$

MEAN WIND VELOCITY

$v_{m.z} := c_{r.z} \cdot c_{o.z} \cdot v_b = 38.8$ m /s

WIND TURBULENCE

$k_I := 1$ Turbulence factor

$$I_{v.z} := \frac{k_I}{c_{o.z} \cdot \ln\left(\frac{z}{z_0}\right)} = 0.145$$

PEAK VELOCITY PRESSURE

$\rho := 1.25$ kg / m3 Air density

$$q_{p.z} := \left(1 + 7 \cdot I_{v.z}\right) \cdot \frac{1}{2} \cdot \rho \cdot \frac{{v_{m.z}}^2}{9.81} = 193$$ kg / m2

$$q_b := \frac{1}{2 \cdot 9.81} \cdot \rho \cdot {v_b}^2 = 82.569$$ kg / m2 basic velocity pressure

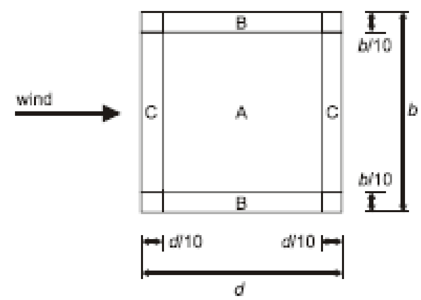
$$c_{e.z} := \frac{q_{p.z}}{q_b} = 2.339$$ exposure factor

The Wind Load has been applied in the roof using the pressure coefficients given in the following table:

Table 7.6 — $c_{p,net}$ and c_f values for monopitch canopies

Net Pressure coefficients $c_{p,net}$

Key plan



Roof angle α	Blockage φ	Overall Force Coefficients c_f	Zone A	Zone B	Zone C
0°	Maximum all φ	+ 0,2	+ 0,5	+ 1,8	+ 1,1
	Minimum φ = 0	- 0,5	- 0,6	- 1,3	- 1,4
	Minimum φ = 1	- 1,3	- 1,5	- 1,8	- 2,2
5°	Maximum all φ	+ 0,4	+ 0,8	+ 2,1	+ 1,3
	Minimum φ = 0	- 0,7	- 1,1	- 1,7	- 1,8
	Minimum φ = 1	- 1,4	- 1,6	- 2,2	- 2,5
10°	Maximum all φ	+ 0,5	+ 1,2	+ 2,4	+ 1,6
	Minimum φ = 0	- 0,9	- 1,5	- 2,0	- 2,1
	Minimum φ = 1	- 1,4	- 2,1	- 2,6	- 2,7
15°	Maximum all φ	+ 0,7	+ 1,4	+ 2,7	+ 1,8
	Minimum φ = 0	- 1,1	- 1,8	- 2,4	- 2,5
	Minimum φ = 1	- 1,4	- 1,6	- 2,9	- 3,0
20°	Maximum all φ	+ 0,8	+ 1,7	+ 2,9	+ 2,1
	Minimum φ = 0	- 1,3	- 2,2	- 2,8	- 2,9
	Minimum φ = 1	- 1,4	- 1,6	- 2,9	- 3,0
25°	Maximum all φ	+ 1,0	+ 2,0	+ 3,1	+ 2,3
	Minimum φ = 0	- 1,6	- 2,6	- 3,2	- 3,2
	Minimum φ = 1	- 1,4	- 1,5	- 2,5	- 2,8
30°	Maximum all φ	+ 1,2	+ 2,2	+ 3,2	+ 2,4
	Minimum φ = 0	- 1,8	- 3,0	- 3,8	- 3,6
	Minimum φ = 1	- 1,4	- 1,5	- 2,2	- 2,7

NOTE + values indicate a net downward acting wind action

- values represent a net upward acting wind action

Per kend te mbuleses α=0° dy raste te koeficienteve te presionit te eres jane konsideruar:

cf = 0.2 (bllokim minimal)

cf = -1.3 (tende e bllokuar totalisht poshte)

9.4. Kombinimet e ngarkesave

Referoju seksionit 4.4.

KOMBINIMET E NGARKESAVE

LOAD COMBINATION	LOAD CASES						
	GRAVITY			WIND LOADS		SEISMIC	
	SWEIGHT (SW)	DEAD (DL)	LIVE (LL)	W(up)	W(down)	Q(X)	Q(Y)
DL+1.5LL	1	1	1.5				
1.35 DL + 1.5LL	1.35	1.35	1.5				
DL+1.5W(up)	1	1		1.5			
1.35DL+1.5W(up)	1.35	1.35		1.5			
DL+1.5W(down)	1	1			1.5		
1.35DL+1.5W(down)	1.35	1.35			1.5		
EQ(X)	1	1				1	
EQ(Y)	1	1					1

9.5. Analiza & rezultate

Llogaritja eshte bere me programin me elemente te fundem SAP2000.

Traret e kollonat jane armuar me sasite minimale te lejueshme te armatures pasi ngarkesat jane minimale.