ENGINEERING DESIGN OF AN ELEVEN STOREY BUILDING WITH UNDERGROUND CAR PARKING TO EUROCODES

THESIS PRESENTED TO OBTAIN

MASTER CERTIFICATE

OPTION: CIVIL ENGINEERING

Presented and defend in January 2016

By:

Edem Koami AGBEHONOU

Directed by: Dr Adamah MESSAN

Supervised by: Ing Beverly Naa Ameley OKAI

<u>JURY:</u>

President: Mr David L. BELLO

Members:

Mr Moussa LO

Mlle Nafissatou SAWADOGO

Mr Seick Omar SORE

Academic year :2014/2015

"WHEN E ARE NO LONGER ABLE TO CHANGE A SITUATION, WE ARE CHALLENGED TO CHANGE OURSELVES"

VIKTOR E. FRANKL

ACKNOWLEDGEMENT

I thank the almighty God for all he has done in my life, His provision of health, protection and means to pursue this programme. I also want to thank my parents and all my family for their continuous support all these years.

I wish to express my profound gratitude to:

- Ing. ABRAHAM FISCIAN the founder of CSENG CONSULT Company for the opportunity he gave me by accepting me in his company;

- Ing. Beverly Naa Ameley OKAI, Director of operation for accepting the task of supervising this work and for advices she gave me during my internship;

- Dr. Adamah MESSAN, Director of Laboratoire Eco Matériaux de Construction (LEMC) of 2iE, our academic supervisor for his availability and his advices despite his schedule;

- All the staff of CSENG CONSULT for their availability and the good atmosphere reigning in the office;

- My Uncles Jesus and Edoh AGBEHONOU for all the supports they gave me while I as in Ouagadougou

- My Uncle Kossi KOMLA ADECHIAN for all he has done for me during my stay in Accra in order to fulfil this;

- All my classmates for their supports and their good collaboration.

I remain grateful to you all. May the almighty God bless all of you for all you have done in my life.

ABSTRACT

In Ghana as well as in the other African countries, policies are made to enhance construction sector. Thus POLO VIEWS, a company that bear several upstanding residential building is building apartments like the ones proposed in this document.

The objective of this work is to do a structural analysis, design the bearing structure of the building, draw the reinforcement details and estimate the global cost of the structure. In order to achieve that purpose, this document is treating different tasks divided in seven (7) chapters. The main part of this work that concern the structural design of the building have been done according to European Standards (EN) and the design of the elements with the software Robot Structural Analysis professional 2012 except the design of slabs where we used Excel spreadsheets labelled by the International community.

After calculation, we obtained for the beams located under the ground floor slab a section of $700x1000$ mm (main beam) and $500x800$ (secondary beam). As for the most stressed columns, we got a size of $700x1000$ mm and the less stressed ones are about $400x700$ mm. the preliminary sizing of the slabs brought us to 175mm thickness for all the floor except the ninth floor which is 200 mm because supporting water tanks. The geotechnical report suggested that we can use 0.25 MPa for the ground bearing pressure.

The design calculations came out with a certain amount of reinforcement ratios for the different elements. We got a ratio within the range of 3.17% to 3.32% for the most stressed beams located on the ground floor slab. Concerning the columns reinforcements, we had about 1.33% for the less stressed and about 2.5 % (presence of high moment) for the most stressed. For footings, we got averagely about 0.5%.

The quantity surveying of this particular building is about **504732406 FCFA**, say about **630915 FCFA** per metre square.

Key words:

Structural design, Preliminary sizing, Ground bearing pressure, reinforcement ratio.

RESUME

Au Ghana comme partout ailleurs aujourd'hui en Afrique, des politiques sont mises en place pour favoriser le secteur de la construction. Ainsi POLO VIEWS, une structure porteuse de projets d'habitation de haut standing construit des appartements à l'instar de ceux proposés dans le présent rapport.

L'objectif de ce travail est de faire une analyse structural, dimensionner la structure portante du bâtiment, d'élaborer les dessins de détails des armatures et de calculer le coût de réalisation du gros œuvre. Pour atteindre ce but, ce document traite plusieurs tâches subdivisées en sept (7) chapitres. La partie la plus importante de ce travail qui concerne le dimensionnement a été effectuée selon les spécifications de la Norme Européenne (EN) et les calculs des éléments structuraux avec le logiciel Robot Structural Analysis Professional 2012 sauf les calculs de dalle où nous avons utilisé des feuilles de calcul Excel programmées et certifiées par la communauté internationale.

Après calcul, nous avons obtenus pour les poutres du plancher haut du rez de chaussé, une section de $700x1000$ mm pour les poutres primaires et $500x800$ mm pour les poutres secondaires. Concernant les poteaux, nous avons obtenus une section de $700x1000$ mm pour ceux qui sont les plus chargés et $400x700$ mm pour les moins chargés. Le pré-dimensionnement des dalles nous a permis d'obtenir des épaisseurs de 175 mm pour les dalles de chaque niveau excepté celui du neuvième niveau qui est de 200 mm parce qu'il supporte des réservoirs de stockage d'eau. Il en est ressortit après les essais géotechniques que la capacité portante du sol est de 0,25 MPa.

Les calculs de dimensionnement nous ont permis d'obtenir pour les poutres les plus sollicités un taux variant de 3,17% à 3,32%. Ces taux sont particulièrement élevés en raison de la présence des moments de torsion. En ce qui concerne les poteaux, on obtient un taux de 1,33% pour le moins chargé et 2,5% pour le poteau le plus chargé (présence de moment assez conséquent). Enfin pour les semelles nous avons un taux moyen de 0,5%.

Le calcul du devis estimatif de ce bâtiment nous donne un montant de **504732406 FCFA** soit **630915 FCFA** au mètre carré.

Mots clés :

Dimensionnement, Pré-dimensionnement, Capacité portante du sol, taux de ferraillage.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

GLOSSARY OF ABBREVIATION & SYMBOL

LEMC: Laboratoire Eco-Materiaux de Construction

IMF: International Monetary Found

CIA: Central Intelligence Agency

PW: Polo Views

EC: Eurocodes

EN: European Standards

BS: British Standards

ULS: Ultimate Limit State

SLS: Serviceability Limit State RSA: Robot Structural Analysis

CBS: Concrete Building Structures

SI: Semelle Isolée (Pad footing)

POU: Poutre (Beam)

POT: Poteau (Column)

Cnom: Nominal Cover

Cmin: Minimum Cover

f_{ck}: Characteristic compressive cylinder strength of concrete at 28 days

f_{yd}: Design yield strength of reinforcement

f_{yk}: Characteristic yield strength of reinforcement

f_{ctm}: Mean value of axial tensile strength of concrete

γ_f: Partial factor for actions, F

γ_c: Partial factor for concrete

γ_s: Partial factor for reinforcing or prestressing

ΔC_{dev}: Allowance in design for Deviation

E_s: Modulus of Elasticity of Steel

Mf: Modification Factor

A_c: Section of Concrete

A_s: Cross sectional area of reinforcement

$A_{s,req}$: Required Area of bars

$A_{s,prov}$: Provided Area of bars

$A_{s,min}$: Minimum cross sectional area of reinforcement

$A_{s,max}$: Maximum cross sectional area of reinforcement

N_{Ed}: Design value of the applied axial force

INTRODUCTION

Ghana is one of the most prosperous countries in West Africa. According to IMF (2014), the population is estimated to 26 millions of people and the population growth is about 2.19 per cent (CIA, 2014) [10]. That human potential offers a good and interesting market for investors. By the economical means, the growth of the country during the past ten years is mostly greater than 5 per cent every year. According to the World Bank it was 7.6 per cent in 2013 (quite similar to Germany economic growth rate in 2015) and 4.5 per cent (IMF, 2014) the last year [10]. Regarding that impressive economic growth, we can obviously guess the high needs of infrastructures (Buildings, routes, highways, bridges, etc.). In other to satisfy that demand, a lot of investors such as Polo Views and some others are acting on site to provide enough facilities for the population.

As a finishing student in civil engineering looking for the best experience in civil engineering domain, we chose to do our final year internship in that environment full of lessons and challenges. Thus we were hosted by CSENG CONSULT, one of the best consultant cabinet acting on the site. Among the projects of building designing that we were doing in the office, our attention was brought to a particular one which gather most of element design known in structural design activities. The theme of this report which is: **“ENGINEERING DESIGN OF AN ELEVEN STOREY BUILDING PLUS CAR PARK UNDERGROUND TO EUROCODES”**, is set to put us in a real condition of a structural design process.

The specific objectives of this work is to meet the client satisfaction, to comply with the architect’s desires and provide the different structural drawings for implementation. Finally a certain survey will be done to give to the client a fair idea about the cost of the first major part which does not include finishing.

To achieve these specific goals (objectives), the work process went through several specific tasks which are:

- Presentation of the project;
- Conceptual design of the building;
- Preliminary sizing;
- Structural design;
- Quantity surveying.

CHAPTER 1: ENVIRONMENT AND PRESENTATION OF THE PROJECT

1.1.About CSENG CONSULT

CSENG CONSULT (Civil and Structural Engineers Consultant), a consultant cabinet created in 2006 is actually located in Accra (Westlands, LEGON) which provide several services. They provide services in Civil Engineering domain (structural design services, structural survey of buildings) and hydraulic services (construction of canalisations and drainage system). They are one of the well-known consultant cabinet in Ghana regarding the number of project they have undertaken in the whole country. They are actually providing consultancy on several project throughout the country.

1.2. About POLO VIEWS

Polo Views (PW) brings its rich experience into the Residential Development market in Ghana. For over forty years they have proved their reliability and credibility in the West African sub-region, where their list of projects have included the construction of Roads, Bridges, Airports, Seaports, and various building works. PW has become a respected building and civil engineering contractor in West Africa, with highly skilled management and technical staff.

In Ghana, their appointed Residential development in Adjiringano is a good to behold. Their recently completed POLO COURT, and POLO HEIGHTS projects have been outstanding successes, changing the landscape of Accra, and at the same time pleasing both home owners and residents. Repeating this success, PW is now undertaken POLO VIEWS.

1.3.Objectives of the report

This report is undertaken as an intern in a consultant cabinet (CSENG) and is likely to take the student throughout the process of building design. Therefore it is dealing with:

- Preliminary sizing of the structural members;
- Design of the structure using Robot RSA professional 2012;
- Provide drawing for implementation;
- Design of non-structural elements;
- Quantity survey of the building.

1.4.Presentation of the project

1.4.1. Site location

The site of the project is located in the airport zone in Accra. The map below is showing the position of the site corresponding to different points such as Accra Mall (Ashampong junction), Polo height and Kotoka international airport.

Figure 1: Site Location

1.4.2. Features & specification

The Polo Views project is located on a 2 acre plot of land overlooking the Polo Height. The project comprises four (4) apartment blocks. Each block comprises a basement parking, ground floor, eight (8) upper floors, one (1) upper floor supporting water storage tanks and two (2) levels of roof floor.

The facility has been divided into blocks (Block A, B, C and D) by the introduction of expansion joints with blocks B and C being separated by entrance and exit at the ground floor level. Each block is an eleven (11) storey frame structure with slabs transferring loads to down stand beams and onto columns have been tied by beams at all levels: basement to roof level 2.

The lift and staircases are centrally located.

Isolated pad footings have been used for foundation based on soil. Geotechnical parameters are identified in the Geotechnical Engineering Survey Report.

Figure 2:Perspective View

1.5.Building geometry

The building geometry is presented in the table 1.1. It comprises length, breadth and height of each floor. Except the basement floor where the parking is, the ninth floor for water tanks and the roof levels, the each of other floors are hosting two (2) apartments separated in two (2) wings. From ground floor to sixth floor, all the west wing is made by 2 bedrooms apartment (190 m²) and the east wing by 3 bedrooms apartment (220 m²). We have two (2) penthouses at seventh and eighth floors. The west penthouse level1 occupy 190 m² and level 2 spreads over 140 m² and the east one 220 m² at the first level and 150 m² at the second.

Table 1.1:Building Geometry

Level	Length (L), m	Breadth (B), m	Height (H), m
Basement	35.40	24.45	3.6
Ground	35.40	24.45	3.3
First to Sixth	35.40	20.50	3.3
Seventh	35.40	18.90	3.3
Eighth to ninth	32.50	18.90	3.3
Roof level 1	17.60	14.15	3.3
Roof level 2	5.8	8.125	---

The target of this first chapter was to describe the environment in which was held our internship and to present briefly the project undertaken. As we could noticed it above, the work was held in a consultant cabinet known as CSENG CONSULT principally. Concerning the project itself, it is composed of 4 identical buildings and we chose to pursue the work with bloc B. The next chapter will then treat all about the preliminary sizing of the building.

CHAPTER 2: CONCEPTUAL AND PRELIMINARY DESIGN

The European standards commonly known as "Eurocodes" EN 1992 (EC2) deals with the design of reinforced concrete structures. EC2 allows the calculation of action effects and of resistances of concrete structures submitted to specific actions and contains all the prescriptions and good practices for properly detailing the reinforcements.

EC2 consist of three (3) parts [2]:

- EN 1992-1 Design of concrete structures – Part 1-1 General rules and rules for buildings; Part 1-2 Structural fire design (CEN,2002)
- EN 1992-2 Design of concrete structures – Part 2: Concrete Bridges – Design and detailing rules (CEN, 2007)
- EN 1992-3 Design of concrete structures – Part 3: Liquid retaining and containment structures (CEN, 2006).

In this document, the principles of Eurocode 2, part 1-1 are applied to the design of a ten-storey building with one underground parking area.

2.1. Actions on the building

Actions have been obtained from EC1 [1]:

- EN 1991-1.1 Densities, self-weight and imposed loads
- EN 1991-1.2 Fire actions
- Ghana building code, August 1977 Wind loads

The convenient safety factors are taken as in EC2 (Table 2.1)

Table 2.1: Factors of safety

Design situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.5	1.15	1.15
Accidental	1.2	1.0	1.0

2.1.1. Loading assessment

The table 2.2 below is grouping the dead loads acting on the building depend on the materials that are going to be used. Since our principal material is concrete (density took from EC2), we need some plastering (mortar) of 50 mm to make all the surfaces neat.

Table 2.2:Dead Load calculations

Dead loads	Values
R.C self-weight:	25 (kN/m^3)
Screed (plastering, 50 mm): 1.89 × 50	0.945 (kN/m^2)
Finishing, partitions	3 (kN/m^2)
Ceiling and service loads	0.5 (kN/m^2)

Concerning the imposed loads (table 2.3), they are established according to the use of the different areas. All the values have been taken from the codes as we can notice it here. For the water tanks we used fresh water density combine with the highest tank we have on market which is about 2.5 m.

Table 2.3: Imposed load

Imposed loads	Values (kN/m^2)	References
Dwelling	2	EC1 (Part 1-1) table 6.2
Balconies, Parking	2.5	EC1 (Part 1-1) table 6.2
Stairs	3	EC1 (Part 1-1) table 6.2
Water storage	25	EC1 (Part 1-1) table A.7
Open terrace	5	EC1 (Part 1-1) table 6.2
Lift machine	5	BS 6399:1 table 1- B

2.1.2. Wind load [9]

According to the EC2, the combination factor is $\psi_1 = 0.5$

The design wind speed Vs should be calculated from:

$$Vs = VS_1S_2S_3 \tag{1}$$

Where V is the basic wind speed, $S_1$is topography factor (table 2.4), S_2 is a factor for ground roughness (table 2.5), building size and height above ground, S_3 is a factor for building life (fig 3.1). [9]

$$Vs = 29 \times 1 \times 1.08 \times 1.05$$

$$\boxed{Vs = 32.9\, m/s}$$

The dynamic pressure of the wind "q" above atmospheric pressure may be calculated from:

$$\boxed{q = kV_s^2} \tag{2}$$

$$\boxed{q = 0.613 \times 33^2 = 0.67\, kN/m^2}$$

Table 2.4: Topography factor S1

TOPOGRAPHY	VALUE OF
1. All cases except in 2 and 3 below	1.0
2. Very exposed hill slopes and crests where acceleration of wind is known to occur. Valleys shaped so that funnelling of wind may occur Sites that are known to be abnormally windy due to some local influence	1.1
3. Steep sided enclosed valleys, sheltered from all winds	0.9

Table 2.5: Ground Roughness, building size and height above ground factor S2

H (m)	1. Open Country with no obstructions			2. Open country with few trees & houses			3. Towns, Suburbs, Forest areas		
	CLASS			CLASS			CLASS		
	A	B	C	A	B	C	A	B	C
3 or less	0.83	0.78	0.73	0.72	0.67	0.63	0.64	0.60	0.55
5	0.88	0.83	0.78	0.79	0.74	0.70	0.70	0.65	0.60
10	1.00	0.95	0.90	0.93	0.88	0.83	0.78	0.74	0.69
15	1.03	0.99	0.94	1.00	0.95	0.91	0.88	0.83	0.78
20	1.06	1.01	0.96	1.03	0.98	0.94	0.95	0.90	0.85
30	1.09	1.05	1.00	1.07	1.03	0.98	1.01	0.97	0.92
40	1.12	1.08	1.03	1.10	1.06	1.01	1.05	1.01	0.96
50	1.14	1.10	1.06	1.12	1.08	1.04	1.08	1.04	1.00
60	1.15	1.12	1.08	1.14	1.10	1.06	1.10	1.06	1.02
80	1.18	1.15	1.11	1.17	1.13	1.09	1.13	1.10	1.06
100	1.20	1.17	1.13	1.19	1.16	1.12	1.16	1.12	1.09
120	1.22	1.19	1.15	1.21	1.18	1.14	1.18	1.15	1.11
140	1.24	1.20	1.17	1.22	1.19	1.16	1.20	1.17	1.13
160	1.25	1.22	1.19	1.24	1.21	1.18	1.21	1.18	1.15
180	1.26	1.23	1.20	1.25	1.22	1.19	1.23	1.20	1.17
200	1.27	1.24	1.21	1.26	1.24	1.21	1.24	1.21	1.18

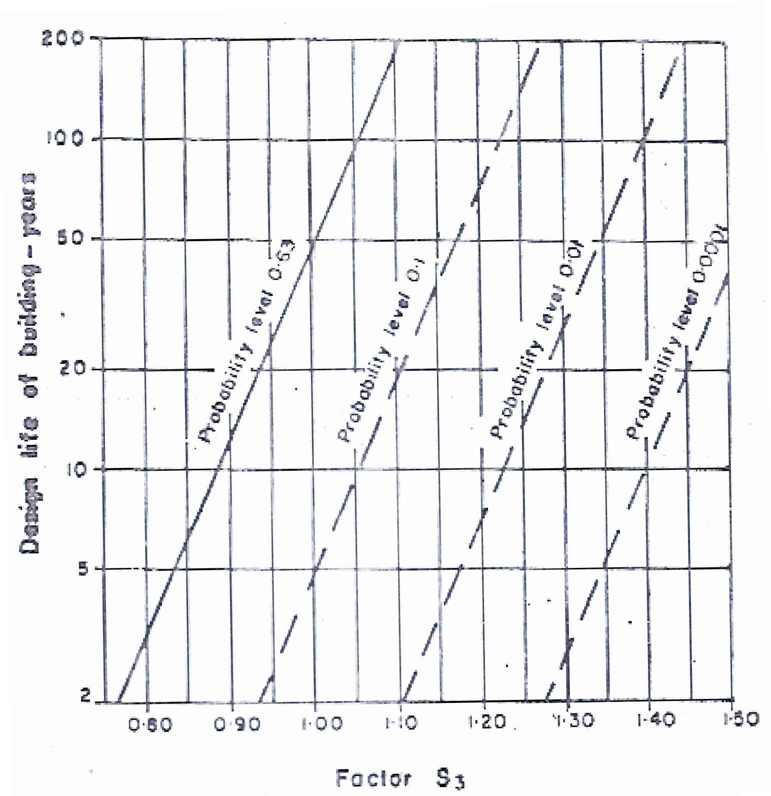


Figure 3:Factor for building life

2.2.Materials

2.2.1. Concrete

2.2.1.1. Exposure classes and concrete strength class

EC2 requires that the structure has to be design such that:

- 50 years design working life,
- “Normal” supervision during execution,
- “Normal” inspection and maintenance during use. Quality management procedure to be adopted during execution are described in EN13670. [4]
- Concrete grade C30/37 for columns and foundations, C25/30 for the other elements.[4]

The nominal cover to reinforcement, C_{nom}, is obtained from:

$$\boxed{C_{nom} = max\{C_{min} + \Delta C_{dev}; 20mm\}} \tag{3}$$

Since we do not have the assurance that the implementation will be well-done, we are going to take $\Delta C_{dev} = 10mm$.

$$C_{min} = max\{C_{min,dur}; C_{min,b}; 10mm\} \tag{4}$$

Where $C_{min,dur}$ (table 2.6) is the minimum cover for durability and $C_{min,b}$ is the minimum cover for bond.

Table 2.6:Minimum cover for durability [2]

Exposure class	[1]*Minimum cover (mm)*							
X0	*Not recommended for reinforced concrete*							
XC1	15	15	15	15	15	15	15	15
XC2	–	25	25	25	25	25	25	25
XC3/4	–	35	30	25	25	20	20	20
XD1	–	–	[2]35	[2]30	30	[2]25	25	25
XD2	–	–	[3]40	[3]35	[2]35	[3]30	30	30
XD3	–	–	–	–	[3]50	[3]45	[2]40	40
XS1	–	–	–	–	[2]40	[2]35	35	30
XS2	–	–	[3]40	[3]35	[2]35	[3]30	30	30
XS3	–	–	–	–	–	[3]50	[2]45	45
Maximum free water/cement ratio	0.70	0.65	0.60	0.55	0.5	0.45	0.40	0.35
Minimum cement content (kgm^{-3})	240	260	280	300	320	340	360	380
[4]Lowest concrete class	C20/25	C25/30	C28/35	C32/40	C35/45	C40/50	C45/55	C50/60

The following classes have been use for our design:

- XC1 for internal slabs and beams

$C_{min}(slabs) = max\{15mm; 12\ mm; 10mm\}$; assuming $\emptyset_{max} = 12\ mm$ for slabs, $C_{min,b} = 12\ mm$

$C_{nom}(slabs) = \max\{\ 15 + 10; 20\}$

$\boxed{C_{nom}(slabs) = 25\ mm}$

$C_{min}(beams) = max\{15mm; 25\ mm; 10mm\}$; assuming $\emptyset_{max} = 25\ mm$ for slabs, $C_{min,b} = 20\ mm$

$C_{nom}(beams) = \max\{\ 25 + 10; 20\}$

$\boxed{C_{nom}(beams) = 35mm}$

- XC1 for columns.

$C_{min}(Columns) = max\{25mm; 25\ mm; 10mm\}$; assuming $\emptyset_{max} = 25\ mm$ for slabs, $C_{min,b} = 25\ mm$

$C_{nom}(Columns) = \max\{\ 25 + 10; 20\}$

$\boxed{C_{nom}(Columns) = 35mm}$

- XC2 for foundation.

For earth retaining walls and foundations, $C_{nom} = 50\ mm$ is common, due to the difficulty of any visual inspection to detect deterioration.

2.2.1.2. *Structural classification*

According to table 2.7 (from EC2, table 4.3N) the recommended structural design class (design working life of 50 years) is S4. But some corrections can be made according to the structural members.

Table 2.7: Recommended structural classification [2]

Structural Class							
Criterion	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class 1) 2)	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Hence we have:

- Slabs: S3 (reduction due to the geometry)

- Beams: S4 (no reduction)
- Columns: S4 (no reduction)
- Footings S4 (no reduction)

2.2.2. Reinforcing steel

For this design, the grade S500B (medium) of steel reinforcements have been used.

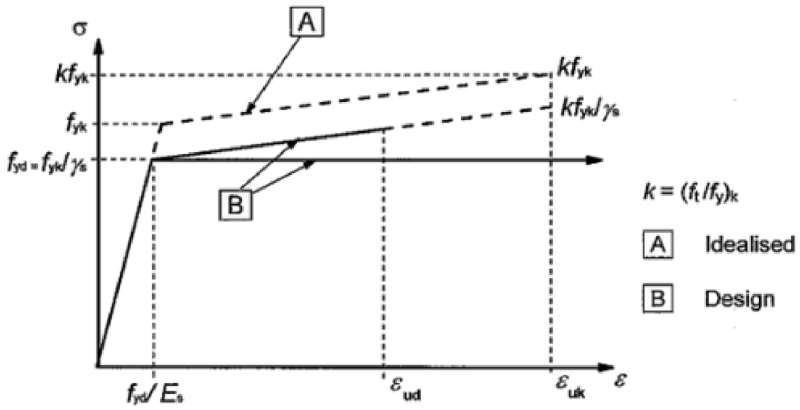


Figure 4: Idealised and design stress-strain diagrams for prestressing steel

The partial safety factors that are going to be use are: $\gamma_s = 1{,}15$ for ultimate limit state (ULS) and $\gamma_s = 1{,}0$ for service-ability limit state (SLS)

$f_{yk} \geq 500\, N/mm^2$; $Es = 200000\, N/mm^2$;

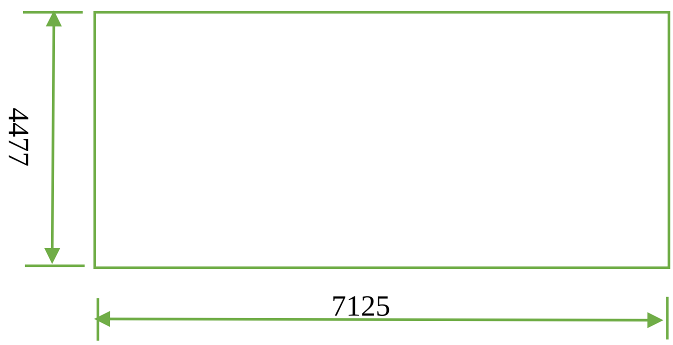
$$f_{yd} = \frac{500}{1.15} = 435\ N/mm^2$$

2.3. Conceptual design of the elements

2.3.1. Conceptual design of slabs

2.3.1.1. Slab height

Considering one of the most critical panels on the ground floor slab, here are the calculations leading to the height of the concerning slab. For the rest of the building, we are going to consider the same thickness of slab.



Check of the spanning of the panel:

$$\frac{L_y}{L_x} = \frac{7125}{4477} = 1.59 < 2 \begin{cases} L_y: Long\ span \\ L_x: short\ span \end{cases} \tag{5}$$

Hence this panel is spanning two (2) ways.

2.3.1.2. Slab thickness:

Assumptions:

- The fire resistance of the building is 1 h 30. Therefore the slab thickness should be greater than 120 mm, table 5.8 [3];
- The slab will be design as a continuous slab;
- Considering a lightly stressed concrete C25/30, the basic span/effective depth ratios are shown in table 2.8 [2];
- Class of exposure XC1, table 4.4 N (EN 1992-1-1); $\Delta C_{dev} = 10\ mm$.

Table 2.8: Basic span/effective depth ratios for reinforced concrete members

Structural system	**K**	**Highly stressed concrete**	**Lightly stressed concrete**
Simply supported beams and slabs	1.0	14	20
End span of continuous beams and slabs	1.3	18	26
Interior span of continuous beams and slabs	1.5	20	30
Cantilevered beams and slabs	0.4	6	8

The panel that we are dealing with is at the edge. Then according to the table 3.7 above (lightly stressed concrete):

$$d = \frac{L_x}{Effective\ depth\ ratio} = \frac{4477}{26} = 172.2\ mm \tag{6}$$

The overall depth is given by:

$$h = d + C_{nom} + t \begin{cases} d: effective\ depth \\ C_{nom}: nominal\ cover \\ t: half\ diameter\ of\ main\ bars \end{cases} \tag{7}$$

$$\boxed{h = 172.2 + 25 + \frac{12}{2} = 186{,}7\ mm}$$

For the design, we will consider the overall depth ***h= 175 mm (find more information in section §4.1).***

2.3.2. Conceptual design of beams

- ***Beam height:***

Assumptions:

- The fire resistance of the building is 1 h 30. Therefore the beam breadth should be greater than 150 mm, table 5.6 (EN 1992-1-2);
- The beam will be design as a continuous beam;
- Considering a lightly stressed concrete, the basic span/effective depth ratios are shown in table 3.7;
- Class of exposure XC1, table 4.4 N (EN 1992-1-1); $\Delta C_{dev} = 10\ mm$.

With an interior span of 7.625 m. By using the formula 6 in relation with the table 3.8 and considering lightly stressed concrete:

$$d = \frac{L_x}{Effective\ depth\ ratio} = \frac{7625}{30} = 254{,}17\ mm$$

The overall depth is given by:

$$h = d + C_{nom} + t + t0 \begin{cases} d: effective\ depth \\ C_{nom}: nominal\ cover \\ t: half\ diameter\ of\ main\ bars \\ t0: diameter\ of\ links \end{cases}$$

$$\boxed{h = 254{,}17 + 35 + \frac{25}{2} + 10 = 311{,}67\ mm}$$

For the preliminary sizing of our beams, we are taking as input of the software a section of 400×600 for the beams.

2.3.3. Conceptual design of columns

This part is dealing with the columns design. The column in a structure carry the loads from the beams and slabs down to the foundation, and therefore they are primarily compression members, although they may also have to resist bending forces due to the continuity of the structure. The analysis of a section subjected to an axial load plus bending which is treated in chapter 4 of reinforcement concrete design to EC2 [5], where it is noted that a direct solution of the equations that determine the area of reinforcement can be very laborious and impractical. Therefore design chart or computers are often use to facilitate the routine design of column section. The code distinguish two (2) types of columns:

- Braced, where lateral loads are resisted by shear walls or other forms of bracing capable of transmitting all horizontal loading to the foundations;

- Unbraced, where horizontal loads are resisted by the frame action of rigidly connected columns, beams and slabs.

With a braced structure, the axial forces and moments in the column are caused by the vertical permanent and variable actions only, whereas with an unbraced the loading arrangements which include the effect of the lateral loads must also be considered.
The most loaded columns are located at the basement.

As for the preliminary sizing of the columns, we took into consideration:

- The fire resistance of the building is 1 h 30. Therefore the column size should be greater than 300 mm, and the axis distance (a) is 25 mm from table 5.2 [2];
- Considering a short- unbraced column;
- Considering a lightly stressed concrete C30/37 and high yield steel reinforcement of 500 MPa;
- Class of exposure XC1, table 4.4 N (EN 1992-1-1); $\Delta C_{dev} = 10\ mm$.

Procedure: [5]

A short-unbraced column satisfy the condition: ${l_{ex}}/{h} < 15$ and ${l_{ey}}/{b} < 15$.

The effective length l_{ex} and l_{ey} are relative to the XX and YY axis, h is the overall depth of the section in the plane of bending about the XX axis, and the dimension perpendicular to the XX axis. The effective lengths are specified as

$$l_e = \beta l_0 \tag{8}$$

l_0 is the clear distance between the column end restraints; β is a coefficient which depends on the degree of end restraints as specified in table 2.9 [5]

Table 2.9: Preliminary size of columns

β for braced columns				β for unbraced columns			
End condition at top	End condition at bottom			End condition at top	End condition at bottom		
	1	2	3		1	2	3
1	0.75	0.80	0.90	1	1.2	1.3	1.6
2	0.80	0.85	0.95	2	1.3	1.5	1.8
3	0.90	0.95	1.00	3	1.6	1.8	–
				4	2.2	–	–

In our case, $\beta = 1.5$ and $l_0 = 3.6m$. Therefore $h \geq 360\ mm$ and $b \geq 360\ mm$. The input sizes are: $400\ x\ 700\ mm$; $700\ x\ 1000\ mm$; $550\ x\ 700\ mm$.

This chapter had to define the different sizes of the different members that are going to be input in the different software that we intend to use for our design. Thus we obtained for slabs a thickness of 175 mm and several dimensions for both columns and beams. The footings will be generated by the software itself. Now we can move to the analysis of the structural calculations.

CHAPTER 3: STRUCTURAL ANALYSIS

In terms of structural analysis of a building, the following steps can be follow:

1) Structural framing scheme design
   a) Request for geotechnical and hydrological survey report from supervisor
   b) Print architectural drawings ensuring complete visibility of all components of drawing.
   c) Carefully review architectural drawings, ensuring that
      i) Floor to floor heights are consistent with staircase rise heights;
      ii) A feasible structural scheme can be obtained from architectural concept;
      iii) Gridlines are correctly numbered and correctly add up on both sides of transverse and longitudinal directions;
      iv) Sections correctly match with plans and elevations;
      v) All relevant sections and details necessary for structural design and detailing have been provided and in sufficient detail;
      vi) Finished floor levels have been provided in plans or/and in sections. Take note of step-downs in scheme design;
      vii) You note any other items of importance for structural analysis, design and detailing. Exercise engineering discretion here!
      viii) Make notes of all information which are not consistent with i) to vii) above to be discussed with supervisor and subsequently, project architect if necessary.
   d) Identify all possible columns, beams and shear wall positions: ensure that
      i) No walls or columns are within openings and open spaces unless proposed by the architectural drawings
      ii) Beams are concealed by drop ceilings and/or block walls;
   e) Perform preliminary sizing of all structural elements ensuring that
      i) Exposure conditions and fire resistance requirements have been accounted for using relevant codes. This will facilitate the choice of cover to rebar for all structural elements.
      ii) Slab thickness and beam depths meet deflection requirements. Provide hand calculations in addition to any available computer print out to show this for critical slab panels (largest external panels, largest internal panels and cantilevers above 1.5m) and long beam spans (spans above 6.5m and cantilevers above 2m).
      iii) Beam depths do not reduce floor-to-floor heights. Where this cannot be avoided after careful consideration of alternatives, discuss with supervisor.
      iv) Beams are concealed by block-walls and/or drop ceilings. Where this cannot be avoided after careful consideration of alternatives, discuss with supervisor.

v) Column sizes are adequate for vertical load consideration taking into account column minimum rebar requirements. Add 30% of dead load to design load for lateral load consideration. Ensure that a minimum of 40% residual capacity is maintained for all foundation and columns.

f) Give structural schematic layout with preliminary sizing to draftsman to generate general arrangement of all floors and roof. Generate transverse and longituginal sections through the entire building as part of general arrangement drawings.

g) Design structural elements using any verifiable software of choice. Validate results for design of key elements with hand calculations making reference to relevant codes where necessary.

The finite element modeling of the hole structure with Robot structural analysis (RSA pro 2012) is chown in the figure3-3 below. Here are the different elements parameters we have used for the modeling.

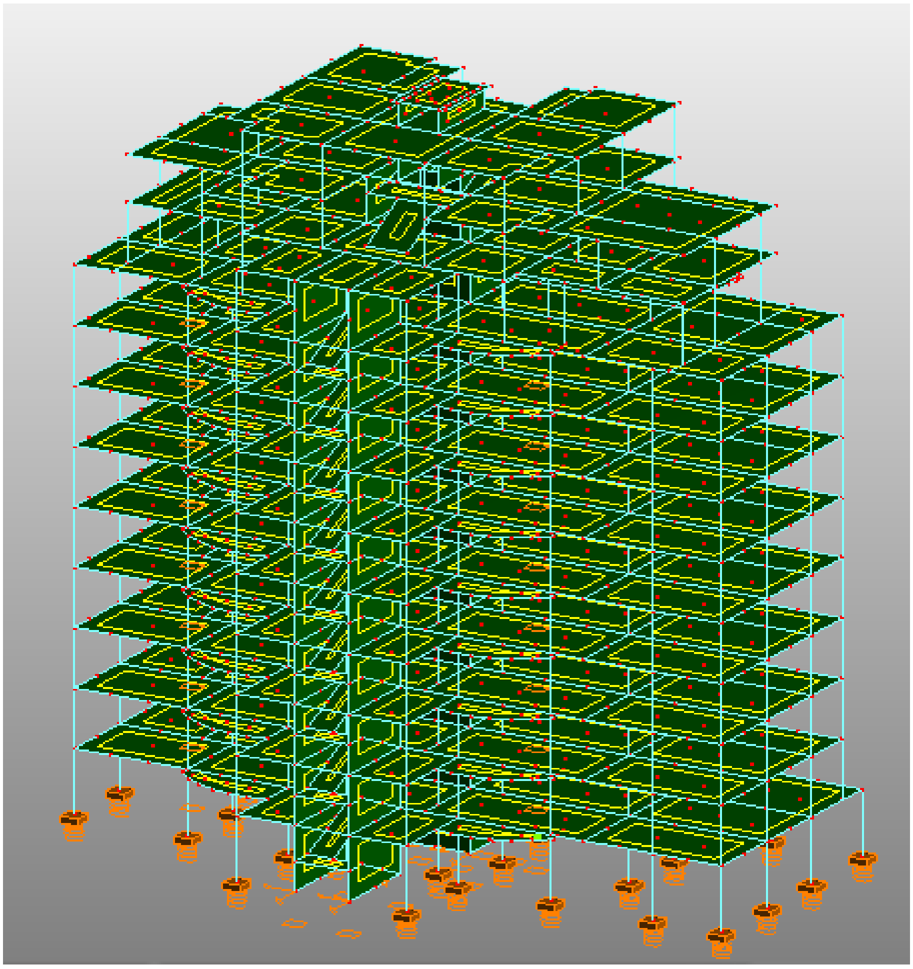


Figure 5 Modeling of the structure:

Verifications of the model of the calculation contained no errors.

3.1.Loads, load cases and their combinations

3.1.1. Loads

As we have shown the loading assessment in the *section 1.1* above, refer to that section to know more about the loads. It is states in Manuel de formation Autodesk robot structural analysis professional 2011 (pge 60) that for the advanced calculations of the structural design, the software takes the self-weight into consideration and combine the different load cases according to the designing code (EC2) at the different limit states (ULS, SLS).

3.1.2. Rules for the combination of load cases

For the ultimate limit state (ULS) only one combination is calculated (general combination):

$\gamma_G G\oplus \gamma_{Q,1} Q_1 \oplus \gamma_{Q,i} \Sigma(\psi_{o,i} Q_i)$ with $\gamma_G = 1{,}35$ and $\gamma_{Q,1} = \gamma_{Q,i} = 1{,}5$ (9)

For the serviceability limit states (SLS) the two following combinations are calculated:

- Characteristic combination:

  $G\oplus Q_1 \oplus \Sigma(\psi_{o,i} Q_i)$ (10)
- Quasi permanent combination:

  $G\oplus\Sigma(\psi_{2,i} Q_i)$ (11)

3.1.3. Different cases of combination

The following load combination successively at ultimate limit state and serviceability limit state, table 3.1 were generated by robot RSA.

N/B: the numbers from 1 to 11 are the designation of the different loads, respectively, Self weight, Dead load, Dwelling areas, Balconies areas, Terraces, Staircase areas, Water storage, East Wind, West Wind, North Wind and South Wind.

3.1.3.1. ULS

Table 3.1: Load combination at the Ultimate Limit State

1	1*1.35 + 2*1.35
2	1*1.00 + 2*1.00
3	1*1.35 + 2*1.35 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50
4	1*1.00 + 2*1.00 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50
5	1*1.35 + 2*1.35 + 8*1.50
6	1*1.00 + 2*1.00 + 8*1.50
7	1*1.35 + 2*1.35 + 9*1.50
8	1*1.00 + 2*1.00 + 9*1.50
9	1*1.35 + 2*1.35 + 10*1.50
10	1*1.00 + 2*1.00 + 10*1.50

11	1*1.35 + 2*1.35 + 11*1.50
12	1*1.00 + 2*1.00 + 11*1.50
13	1*1.35 + 2*1.35 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 8*1.00
14	1*1.00 + 2*1.00 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 8*1.00
15	1*1.35 + 2*1.35 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 9*1.00
16	1*1.00 + 2*1.00 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 9*1.00
17	1*1.35 + 2*1.35 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 10*1.00
18	1*1.00 + 2*1.00 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 10*1.00
19	1*1.35 + 2*1.35 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 11*1.00
20	1*1.00 + 2*1.00 + 3*1.50 + 4*1.50 + 5*1.50 + 6*1.50 + 7*1.50 + 11*1.00
21	1*1.35 + 2*1.35 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 8*1.50
22	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 8*1.50
23	1*1.35 + 2*1.35 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 9*1.50
24	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 9*1.50
25	1*1.35 + 2*1.35 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 10*1.50
26	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 10*1.50
27	1*1.35 + 2*1.35 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 11*1.50
28	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 11*1.50

3.1.3.2. SLS

Table 3.2:Load combination at the Service-ability Limit State

1	1*1.00 + 2*1.00
2	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 8*0.77
3	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00
4	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 9*0.77
5	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 10*0.77
6	1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 11*0.77
7	1*1.00 + 2*1.00 + 3*0.77 + 4*0.77 + 5*0.77 + 6*0.77 + 7*0.77 + 8*1.00
8	1*1.00 + 2*1.00 + 3*0.77 + 4*0.77 + 5*0.77 + 6*0.77 + 7*0.77 + 9*1.00
9	1*1.00 + 2*1.00 + 3*0.77 + 4*0.77 + 5*0.77 + 6*0.77 + 7*0.77 + 10*1.00
10	1*1.00 + 2*1.00 + 3*0.77 + 4*0.77 + 5*0.77 + 6*0.77 + 7*0.77 + 11*1.00

3.2.Results analysis

3.2.1. Internal forces and moments

In the case of this report, we are just going to focus on the results of the basement floor. The table 3.3 below is giving the maximum forces apply to the columns located at the concerning floor. These values are going to be use by the software to design the columns.

- F_X: the axial force (X-X);
- F_y: the internals forces that acting in the direction of (Y-Y);

- F_Z: the internals forces that acting in the direction of (Z-Z);
- M_X: the rotation around (X-X)
- M_y: the rotation around (Y-Y)
- M_Z: the rotation around (Z-Z)

Table 3.3: Forces and moments acting on the basement columns

	FX [kN]	FY [kN]	FZ [kN]	MX [kN.m]	MY [kN.m]	MZ [kN.m]
Pot 0_01	3734.50	56.89	251.34	5.04	1625.32	172.23
Pot 0_02	2978.85	24.65	0.22	0.98	162.41	312.17
Pot 0_03	2062.61	52.20	4.57	1.03	167.72	423.09
Pot 0_04	2474.30	82.96	3.90	0.42	620.83	494.71
Pot 0_05	2466.43	-296.16	-23.37	4.74	512.71	401.03
Pot 0_06	2018.69	-214.42	-3.50	1.91	140.23	313.91
Pot 0_07	3640.34	-29.32	-25.03	1.77	387.18	103.80
Pot 0_08	3729.80	-78.64	-66.50	1.85	449.62	141.56
Pot 0_09	1742.18	2.88	-9.71	1.36	571.15	74.39
Pot 0_10	1714.35	-47.77	-26.38	2.45	514.55	77.98
Pot 0_11	3213.92	-68.87	-74.64	0.95	325.09	102.05
Pot 0_12	3888.67	-59.85	-130.90	8.65	1729.66	180.48
Pot 0_13	3844.69	-56.13	-522.52	6.83	1865.49	178.17
Pot 0_14	3974.04	-62.24	32.42	11.99	1887.69	200.03
Pot 0_15	2854.98	-324.59	79.15	1.31	310.50	416.93
Pot 0_16	2546.55	318.01	149.51	1.38	791.23	1563.48
Pot 0_17	1704.51	124.84	80.25	0.75	284.72	802.71
Pot 0_18	2921.77	-4.48	-6.55	0.50	402.76	34.37
Pot 0_19	2612.55	1.35	-35.77	1.05	355.54	61.40
Pot 0_20	1919.34	-40.96	97.32	3.60	660.07	95.91
Pot 0_21	893.53	-0.12	7.57	0.27	27.86	8.94
Pot 0_22	930.55	-3.86	130.91	-0.11	274.07	-8.32
Pot 0_23	1257.05	-4.08	4.03	0.06	86.86	-7.59
Pot 0_24	2356.87	-19.89	-12.83	0.72	99.84	182.14
Pot 0_25	2445.35	-131.09	-11.39	1.46	113.43	223.19
Pot 0_26	1292.98	-43.66	1.64	1.41	152.74	126.25
Pot 0_27	1868.95	-105.97	-7.79	2.24	408.66	149.13
Pot 0_28	1901.84	27.59	8.71	1.35	476.13	196.01

Pot 0_29	2514.17	-30.55	2.82	0.93	161.00	125.28
Pot 0_30	1266.09	-64.88	-3.32	1.80	132.27	155.17
Pot 0_31	2238.46	-23.45	29.44	-0.04	479.89	48.14

3.2.2. Displacements

The table 3.4 below is giving the envelope of the maximum displacement of the elements of the whole structure. We can see that the major displacement is about 7 cm along the axis YY which is lateral. This value has been obtained with the combination 25 at ultimate limit state. According to the code, the maximum lateral displacement of the storey building due to lateral loads should be less or equal to H/500. The total height of the building is roughly 40 m. therefore the limiting displacement is 8 cm which is greater than our maximum displacement.

Table 3.4: Envelop of the total displacement of the nodes

	UX [cm]	UY [cm]	UZ [cm]	RX [Rad]	RY [Rad]	RZ [Rad]
Max	0.8	7	0.2	0.009	0.004	0.001
Node	287	1147	67	4830	3143	3262
Case	ULS/5	ULS/25	11	ULS/13	ULS/19	ULS/19
Min	-1.1	-1.2	-6.0	-0.006	-0.006	0
Node	543	1165	1122	2189	4830	4384
Case	ULS/23	11	ULS/17	ULS/13	ULS/17	ULS/13

3.2.3. Deflections

The following table 3.5 is giving the maximum and the minimum deflection of the building different elements. We can notice that the deflections shown in the table is currently occurring on the columns. And the deflection on the beams are even less. The major deflection check is going to be handle in the detailing calculations in the annex which will show more details about it.

Table 3.5: Envelop of total deflection of the nodes

	UX [cm]	UY [cm]	UZ [cm]
Max	0.0	0.2	0.2
Bar	703	1159	659
Case	ULS/13	ULS /17	ULS /19
Min	0.0	-0.1	-1
Bar	707	1066	568
Case	ULS /15	ULS /13	ULS /17

The main concern of this particular chapter is to ensure that the building is stable. After the analysis, we have to do the detailing of all the element of the building. But in the following lines our major concern is about the element that are subjected to the highest loads. Thus the chapter 5 is going to show the process of the different detailing and provide the results.

CHAPTER 4: DESIGN OF MEMBERS

After showing the results of the analysis the software have made, this chapter is going to deal with the structural design of some members (slab, beam, column, pad footing, spread footing, shear wall, retaining wall and stairs). In the following lines, we are going to identify one element of each member and design it according to the code (EC2).

4.1.Slab design

For the structural design calculations, we are going to perform the design of the ground floor slab which will be used for dwellings. As we set it in the section *§2.3* the conceptual characteristics of our slabs are:

- *Slab height = 175 mm; maximum diameter of rebar =14mm, high yield tensile (fy=500 MPa); concrete grade = C25/30; Structure class: S3; cover = 25mm*
- *Loads: dead load = 4.5 kN/m²; imposed load = 2 kN/m² (Dwelling); imposed load = 2.5 kN/m² (Balconies). imposed load = 3 kN/m² (Stairs & lobby);*

As usual the analysis of a slab is conducted panel by panel. A spread sheet certified by Reinforced concrete council is available to perform good design of 1 way or 2 way slabs (See annex 3 for the complete calculations).

4.1.1. Design calculation procedure

a) Loading calculations at ULS:

$$W = 1.35\ (dead\ loads) + 1.5\ (Imposed\ load) \qquad (12)$$

b) Bearing moment calculations:

As a two ways slab, we have to calculate the bearing moment on both directions. The formulas are as follows:

- ✓ Direction (Ox): $M_{x^+} = \beta_{sx^+} W L_{x^2}\ (\,sagging\ moment)$ (13)

  $M_{x^-} = \beta_{sx^-} W L_{x^2}\ (hogging\ moment)$ (14)
- ✓ Direction (Oy): $M_{y^+} = \beta_{sy^+} W L_{x^2}\ (\,sagging\ moment)$ (15)

  $M_{y^-} = \beta_{sy^-} W L_{x^2}\ (hogging\ moment)$ (16)

The coefficients $\beta_{sx^+}, \beta_{sx^-}, \beta_{sy^+}$ & β_{sy^-} are shown in table 4.1

Table 4.1 Bending moment coefficients for two-way rectangular spanning slab supported by beams:

Type of panel and moments considered	Short span coefficients for values of l_y/l_x 1.0	1.25	1.5	1.75	2.0	Long-span coefficients for all values of l_y/l_x
Interior panels						
Negative moment at continuous edge	0.031	0.044	0.053	0.059	0.063	0.032
Positive moment at midspan	0.024	0.034	0.040	0.044	0.048	0.024
One short edge discontinuous						
Negative moment at continuous edge	0.039	0.050	0.058	0.063	0.067	0.037
Positive moment at midspan	0.029	0.038	0.043	0.047	0.050	0.028
One long edge discontinuous						
Negative moment at continuous edge	0.039	0.059	0.073	0.083	0.089	0.037
Positive moment at midspan	0.030	0.045	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous						
Negative moment at continuous edge	0.047	0.066	0.078	0.087	0.093	0.045
Positive moment at midspan	0.036	0.049	0.059	0.065	0.070	0.034

c) Design for steel reinforcement on both directions (x-x and y-y)

- *Coefficient* $K = \frac{M}{bd^2 f_{ck}} \leq k_{bal} = 0.167$ $(limit\ of\ no\ compression\ steel\ reinforcement)$ (17)

- *Lever arm calculation* on both directions (x-x and y-y)

$$Z = d\left[0.5 + \sqrt{\left(0.25 - \frac{3K}{3.4}\right)}\right] \quad (18)$$

- *Area of bars* on both directions (x-x and y-y); top and bottom reinforcements.

The formula that is used to calculate the area of bars is

$$A_s = \frac{M}{0.87\, f_{yk}\, z} \quad (19)$$

- *Minimum and maximum reinforcement required*

The formula we need to assess the minimum reinforcement required is set in Eurocode 2 [5].

$$A_{s,min} = 0.13\%\ bh;\ A_{s,max} = \frac{100A_s}{A_c}$$

- Deflection check [2]

$$Actual\ deflection = \frac{Span}{Effective\ depth}$$

$Limiting\ deflection = M_f \times \frac{l}{d}$; Where M_f is the modification factor.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho}\right) + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right]\ if\ \rho \leq \rho_0 \quad (20)$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho-\rho'}\right) + 3.2\sqrt{f_{ck}}\left(\frac{\rho'}{\rho}\right)^{3/2}\right]\ if\ \rho > \rho_0 \qquad (21)$$

$$\rho = \frac{A_{req}}{bd};\ \ \rho_0 = \sqrt{f_{ck}} \times 10^{-3} = 5 \times 10^{-3} \qquad (22)$$

$$M_f = \frac{310}{\sigma_s};\ \sigma_s = \frac{5 \times f_{yk} \times A_{s,req}}{8 \times A_{s,prov}} \qquad (23);(24)$$

When the limiting deflection is too high we can reduce the slab thickness by correcting the span/effective depth ratio by the modification factor.

N/B: See annex for cracking and lapping.

4.1.2. Results

Recapitulative table 4.2 of the ground floor slab design calculations

Table 4.2: Dwelling floors reinforcement details

	X-X		Y-Y	
	Provided	Area	Provided	Area
Top layer	T10@150 mm	523 mm²	T10@150 mm	523 mm²
Bottom layer	T10@150 mm	523 mm²	T10@150 mm	523 mm²

Recapitulative table 4.3 of service floor slab design calculations

Table 4.3: Service floor slab reinforcement details

water storage areas	X-X			Y-Y		
	Provided	Area	Additional	Provided	Area	Additional
Top layer	T12@75 mm	1507 mm²	6T10	T12@100 mm	1130 mm²	6T10
Bottom layer	T12@75 mm	1507 mm²		T12@100 mm	1130 mm²	
Terrace	T10 @225 mm	349 mm²		T10 @225 mm	349 mm²	

4.2. Design for bending of a rectangular beam

Generally, a beam is a structural element that is capable of withstanding load primarily by resisting bending. The bending forces induced into the material of the beam as a result of the external loads. We can identify different categories of beams depend on the support condition and the number of support. Thus the calculations are different. In the framework of this document, we are going to consider our beams to be continuous with no moment redistribution. Our preliminary sizing and considerations of the ground floor beams are:

- *Beam size = 400 x 600 mm; maximum diameter of rebar =20mm, high yield tensile (fy=500 MPa); concrete grade = C25/30; Structure class: S4; cover = 35mm*
- *Loads: dead load = 4.5 kN/m²; imposed load = 2 kN/m² (Dwelling); imposed load = 2.5 kN/m² (Balconies).*

The global design procedure of a continuous beam with no moment redistribution once you get the internal forces is as follows:

a) Check whether compression steel is needed or not by determining (formula 17):

$$K = \frac{M}{bd^2 f_{ck}} < K_{bal} = 0.167 \text{ then no steel reinforcement is required}$$

b) Determine the lever-arm, z, from the equation (18)

$$z = d[0.5 + \sqrt{(0.25 - K/1.134)}]$$

c) Calculate the area of tension steel reinforcement by the formula (19):

$$A_s = \frac{M}{0.87\, f_{yk} z}$$

d) Select suitable bar sizes according to the area of reinforcement chart Annex5

e) Check that the area of steel actually provided is within the limits required by the code, that is

$$100\frac{A_{s,max}}{bh} \leq 4.0\% \text{ and } 100\frac{A_{s,min}}{bd} \geq 26\frac{f_{ctm}}{f_{yk}}\%;$$

$$\text{where } f_{ctm} = 0.3 \times f_{ck}^{\frac{2}{3}} \text{ for } f_{ck} \leq C50 \qquad (25)$$

f) If compression steel is required, ie $K > K_{bal} = 0.167$, $x = x_{bal} = 0.45d$

- Calculate the area of compression steel from:

$$A'_s = \frac{M - K_{bal} f_{ck} bd^2}{f_{sc}(d - d')} \qquad (26)$$

Where f_{sc} is the compressive stress in the steel.

If $d'/x \leq 0.38$ the compression steel has yielded and $f_{sc} = 0.87 f_{yk}$

If $d'/x > 0.38$ then the strain ε_{sc} in the compressive steel must be calculated from the proportions of the strain diagram and $f_{sc} = E_s \varepsilon_{sc} = 200 \times 10^3 \varepsilon_{sc}$.

- Calculate the area of tension steel required from

$$A_s = \frac{K_{bal} f_{ck} bd^2}{0.87\, f_{yk} z} + A'_s \frac{f_{sc}}{0.87\, f_{yk}} \text{ with } z = 0.82d \qquad (27)$$

- Check for the areas of steel required and the areas provided that

$$(A'_{s\,prov} - A'_{s\,req}) \geq (A_{s\,prov} - A_{s\,req}) \quad (28)$$

This is to ensure that the depth of the neutral axis has not exceeded the maximum value of 0.45d by providing an over-excess of tensile reinforcement.

- Finally check that the area of steel actually provided is within the limits required by the code practice.

N/B: See the results in the Annex 5.

4.3.Design of columns

According to the conceptual design of our columns, we distinguished three (3) types of columns at the basement level: $400\ x\ 700\ mm$; $700\ x\ 1000\ mm$; $550\ x\ 700\ mm$. According to the results in chapter 2, we are going to show the design calculations of the columns POT0_02, POT0_14, POT0_04 and POT0_16.

The design of a braced column involves consideration of the following steps:

a) Determination of the slenderness ratio, λ

$$\lambda = \frac{l_0}{i} = \frac{l_0}{\sqrt{(I/A)}}; \quad (29)$$

Where: ***lo*** is the effective length of the column; ***i*** is the radius of gyration of the uncracked concrete section; ***I*** is the second moment of area of section about the axis; ***A*** is the cross-sectional area of the column.

b) Limiting slenderness ratio – short or slender columns

EC2 places an upper limit on the slenderness ratio of a single member below which second order effects may be ignored. This limits is given by:

$$\lambda_{lim} = 20 \times A \times B \times C/\sqrt{n}; \quad (30)$$

Where:

$$A = 1/(1 + 0.2\emptyset_{ef}); B = \sqrt{1 + 2w}; C = 1.7 - r_m; \quad (31); (32); (33)$$

These factors can be calculated but if their respective parameters are not known, A, B and C can respectively be taken as 0.7, 1.1 and 0.7.

c) Failure modes

Short columns usually fail by crushing but a slender column is liable to fail by buckling. Euler derived the critical load for a pin-ended strut as

$$N_{crit} = \frac{\pi^2 EI}{l^2}; \quad (34)$$

The crushing load N_{ud} of a truly axially loaded column may be taken as:

$N_{ud} = 0.567 f_{ck} A_c + 0.87\, A_s f_{yk}$ (35)

Where Ac is the area of the concrete and As is the longitudinal steel.

The mode of failure of a column can be one of the following:

- Material failure with negligible lateral deflection, which usually occurs with short columns but can also occur when there are large end moments on a column with an intermediate slenderness ratio.
- Material failure intensified by the lateral deflection and the additional moment. This type of failure is typical of intermediate columns.
- Instability failure which occurs with slender columns and is liable to be preceded by excessive deflections.

d) Reinforcement details

The rules governing the minimum and maximum amounts of reinforcement in load bearing column are as follows.

Longitudinal steel

- A minimum of four bars is required in a rectangular column (one bar in each corner) and six bars in a circular column. Bar diameter not less than 12 mm;
- The minimum area of steel is given by

$A_s = \frac{0.10\, N_{Ed}}{0.87 f_{yk}} \geq 0.002 A_c;$ (36)

- The maximum area of steel, at laps is given by

$\frac{A_{s,max}}{A_c} < 0.08$ where As is the total area of longitudinal steel and Ac the cross-sectional area of the column.

Otherwise, in regions away from laps: $\frac{A_{s,max}}{A_c} < 0.04.$

e) Links

- Minimum $size\ =\ 0.25 \times size\ of\ the\ compression\ bar\ but\ not\ less\ than\ 6\ mm.$
- Maximum spacing should not exceed the lesser of $20 \times size$ of the smallest compression bar or the least lateral dimension of the column or 400 mm. this spacing should be reduced by a factor of 0.60. (for a distance equal to the larger lateral dimension of the column above and below a beam or slab, and at lapped joints of longitudinal bars > 14 mm diameter).

- Where the direction of the longitudinal reinforcement changes, the spacing of the links should be calculated, while taking account of the lateral forces involved. If the change in direction is less than or equal to 1 in 12 no calculation is necessary.
- Every longitudinal bar placed in a corner should be held by transverse reinforcement.
- No compression bar should be further than 150 mm from a restrained bar.

N/B: Since we are not using shear walls to resist the lateral loads, in this case the wind loads, our column is unbraced and the design changes because of the moment acting in the section. More information can be find in the Reinforced Concrete Design to Eurocode [5].

The complete results of the columns are shown in Annex6 and here is the recapitulative, table 4.4

Table 4.4: Recapitulative of the calculated columns

Designation	Size	Longitudinal reinforcement	Transversal reinforcement	Stirrups
POT0_14	$70\ x\ 100$	14H20	13H10	26H10
POT0_02	$70\ x\ 40$	6H20	12H10	12H10
POT0_04	$70\ x\ 55$	8H20	12H10	24H10
POT0_16	$70\ x\ 100$	26H20	13H10	65H10

The results in this chapter have been provided by the software. Full calculation results can be find in Annex 4, 5 & 6. Design of elements to Eurocodes passes throughout a lot of steps that we tried to simplify in the current chapter. After the superstructure we are now about to design the elements that are going to transfer the loads from the superstructure to the ground.

CHAPTER 5: GEOTECHNICAL ASPECTS OF THE BUILDING DESIGN

This part is dealing with the design of the elements that are going to transfer the loads to the soil. Here our concern is about footings and retaining wall since the basement floor is underground.

5.1.Design of pad footings

EC7 presents three alternative design approaches. In this design approach, two sets of load combinations must be considered at the ultimate limit state. These two combinations will be used for consideration of both structural failure (excessive deformation, cracking or failure of the structure), and geotechnical failure (excessive deformation or complete failure of the supporting mass of earth). A third combination must be taken when considering possible loss of equilibrium of the structure such as overturning. The partial safety factors to be used for these three combinations are given in table 10.1 [5].

For simple spread foundations such as strip and pad footings EC7 gives three alternative methods of design.

- The 'direct method' where calculations are required for each limit state using the partial factors of safety as appropriate from tables 10.1 and 10.2 [5].
- The 'indirect method' which allows for a simultaneous blending of ultimate limit state and serviceability limit state procedures
- The 'prescriptive method' where an assumed safe bearing pressure is used to size the foundations based on the serviceability limit state followed by detailed structural design based on the ultimate limit state.

The first method is going to be use in this document. The design of the footings in the software is given the results showing in table 5.1:

Table 5.1: Typical pad footings reinforcement details

	AXIS (XX)			AXIS (Y-Y)			Pier (transveral)	Dowel bars
	Bottom reinf.	Top reinf.	Pier (longitudinal)	Bottom reinf.	Top reinf.	Pier (longitudinal)		
SI0_07	25H16	25H16	2H12	25H16	-----	2H12	8H10	14H20
SI0_13	11H12	-----	2H12	14H12	-----	2H12	6H10	6H20
SI0_19	18H16	-----	2H12	15H16	-----	2H12	7H10	8H20

5.2.Design of retaining walls

There is three (3) types of retaining walls. But our concern in this document is the cantilever retaining wall. The walls are going to support a soil material of density 1800 kg/m². The design process may be split into these 3 fundamental stages:

h) Check the stability of the wall
i) Assess the bearing pressures at the ULS
j) Design the bending reinforcement using high yield steel, fyk = 500 MPa and concrete class C30/37.

Here are the recapitulative table 5.2 of the retaining reinforcements. See the Annex 8 for the details

Table 5.2: Retaining walls reinforcement details

Member	Main reinforcement	Area [mm²]	Compression	Area [mm²]	Distribution	Area [mm²]
Wall	H20 @ 175 mm	1795	H10 @ 200 mm	392	H10 @ 200 mm	392
Heel (Base)	H12 @ 150 mm	753	H12 @ 150 mm	753	H10 @ 200 mm	392
Toe (Base)	H12 @ 150 mm	753	H12 @ 150 mm	753	H10 @ 200 mm	392

Here, based on the characteristics of the soil given by the geotechnical report, we designed our pad footings and retaining walls. Complete calculation result is available in Annex 7 & 8. We obtained a size of $\mathbf{250\ x\ 500mm}$ for the walls. For footings, we concentrated, within the scope of this report, our attention on those that are likely to be the most loaded. Thus we obtained $\mathbf{2000\ x\ 2500mm}$; $\mathbf{2000\ x\ 1750mm}$ and $\mathbf{2750\ x\ 2750mm}$

CHAPTER 6: STAIRS AND SHEAR WALLS

6.1.Stairs

The stairs are spanning longitudinally and set into pockets in the two supporting beams. The effective span is 2.5 m and the rise of the stairs is 1.5 m with 250 mm treads and 150 mm risers. The variable load is 3.0 kN/m² and the characteristic material strengths are fck = 25 MPa and fyk = 500 MPa. The stairs slab thickness is 150 mm.

The full design calculations are in the Annex 9. Here are the results of the calculations, table 6.1:

Table 6.1: Stairs reinforcements details

STAIRS& LANDING	X-X		Y-Y	
	Provided	Area	Provided	Area
Top layer	T10@ 200 mm	392 mm²	T10@ 200 mm	392 mm²
Bottom layer	T12@ 200mm	565 mm²	T12@ 200 mm	565 mm²

6.2.Shear walls

The shear walls in this report are just supporting the lift and the stairs. Basically, we didn't specify to the software to take the columns to be braced. Therefore the columns are taking care of the moment induce by the wind loads. Thus we are going to design our shear walls to be supporting only the slabs and the same results will be use for the shear wall around the lift.

The shear walls are mostly designed as a column with one meter width. Thus the design process is same as we introduced it for the columns (see section §4.3). Here are the results of the design performed with Robot RSA pro.

Tableau 6.2: Shear walls main reinforcement

MAIN REINFORCEMENTS	Provided	Area	Spacing (mm)
Vertical reinforcement	36HA12	---	250
Horizontal reinforcement	30 HA8	---	250
U-reinforcement	30 HA8	---	250
Transversal reinforcement	144HA8	---	250

Tableau 6.3: Bondary elements reinforcement

BONDARY ELEMENTS	Provided	Area (mm^2)	Spacing (mm)
Longitudinal reinforcement left side	4HA12	452.4	---
Longitudinal reinforcement right side	4HA12	452.4	---
Pins	30 HA 8	---	250

CHAPTER 7: QUANTITY SURVEY

This part is dealing with the quantities calculations. Indeed after completing all the calculations above, we need to evaluate a preliminary cost of what we did, thus we can find full calculation in annex 11. But here is the summary of the preliminary quantity survey. We emphasis here that this quantities evaluations is only for the framework of the building plus partitions (Blockwork 15 mm thick). To convert this money from GHC to FCFA, we mostly use 6 GHC corresponds to 1000 FCFA.

Table 6.2: Quantity survey

QUANTITIES					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**ESCAVATION**				
I.1	Setting out of the building	Total	1	500	500
I.2	Escavation of soil for basement	m^3	3600	25	90000
	Subtotal 1				90500
II	**CONCRETE & REINFORCED CONCRETE**				
II.1	Blinding C16/20 and 5 cm thickness	m^3	13,8915	420	5834
II.2	Reinforced concrete C30/37	m^3	798,67	680	543097
II.3	Reinforced concrete C25/30	m^3	2069,45	550	1138195
	Subtotal 2				1687126
III	**PLASTERING**				
III.1	Total Screed apply on bottom slab	m^2	5748	35	201180
III.2	Total Screed apply on top slab	m^2	5900	30	177000
	Subtotal 3				378180
IV	**WATER PROOFING**				
IV.1	Total Protection against moisture	m^2	1390	95	132050
V	**MASONRY**				
II.1	***Blockwork 15 cm thick***	m^2	11393	65	740538
	Subtotal 4				740538
	TOTAL				3028394
TOTAL OF 504732406 CFA					

Hence we got an amount of **504732406 FCFA** for the realisation of the structural framework (gather in Slab, Beams, Columns, Footings, Retaining walls, Shear walls and stairs) of the building. This amount is does not consider the finishing work which includes painting, tiling, doors and windows, etc.

CONCLUSION

The current project had the target to design an eleven storey building plus car park underground and give a fair idea of the total cost of implementation. The building located in Accra around the airport have been designed using the software Robot Structural Analysis to analyse the superstructure and check the different displacements and deflections to meet the code provisions. Thus we had lateral displacement about 7 centimetres which is less than H/500, the limit provided by the code. The slabs have been calculated using Excel spreadsheets. The inputs of the software were the results of preliminary sizing of the different elements. The analysis with the software issued the reinforcement detailing of beams, columns and footings. The other details have been drawn manually using AutoCAD 2014.

The design calculations came out with a certain amount of reinforcement ratios for the different elements. We got a ratio within the range of 3.17% to 3.32% for the most stressed beams located on the ground floor slab. Concerning the columns reinforcements, we had about 1.33% for the less stressed and about 2.5 % (presence of high moment) for the most stressed. For footings, we got averagely about 0.5%.

Finally, no project can be done without having a fair idea about what is going on by economical means. Therefore, a quantity survey has been done for the building. The total cost of the implementation of this particular building is about **504732406 FCA**.

REFERENCES

[1] Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings;

[2] Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings, December 2004;

[3] Eurocode 2: Design of concrete structures – Part 1-2: General rules and rules – Structural fire design, December 2004;

[4] Eurocode 2: Background & Applications – Design of Concrete Buildings (JRC Scientific and Policy Report), 2014;

[5] Reinforced Concrete Design to Eurocode 2, Bill Mosley, John Bungley and Ray Hulse, Sixth Edtion;

[6] Design of Structural Elements, CHANAKYA ARYA, Third Edition;

[7] Manuel de formation Autodesk robot structural analysis professional 2011;

[8] Reinforced Concrete Designer's Handbook, Charles E. Reynolds and James C. Steedman, Tenth Edition;

[9] Ghana building code, August 1977;

[10] www.diplomatie.gouv.fr/fr/dossiers-pays/ghana/presentation

ANNEXS

ANNEX 1: Building architectural drawings

ANNEX 2 General arrangements

ANNEX 3 Slab calculations

- Spreadsheet calculations of panel P1: 2 ways slab

Project	RESIDENTIAL BUILDINGS	REINFORCED CONCRETE COUNCIL	
Client	POLO VIEWS	Made by: E.A	Date: 13-oct-2015 · Page: 130
Location	32nd floor - corner panel · H to D: 1 to 2	Checked: E.A	Revision: - · Job No
	2-WAY SPANNING INSITU CONCRETE SLABS to BS 8110:1997 (Table 3.14)		
	Originated from RCC84.xls on CD · © 1999 BCA for RCC		

DIMENSIONS

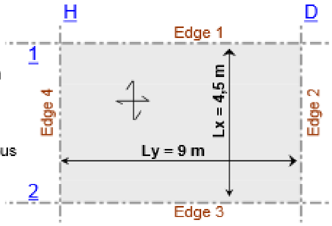
short span, lx	m	4,50	
long span, ly	m	9,00	
h	mm	175	
Top cover	mm	25	
Btm cover	mm	25	

MATERIALS

fcu	N/mm²	25	γ_c = 1,50
fy	N/mm²	500	γ_s = 1,15
Density	kN/m³	25	
(Normal weight concrete)			

STATUS VALID DESIGN

Plan



LOADING *characteristic*

Self weight	kN/m²	4,38		
Extra dead	kN/m²	4,50		
Total Dead, gk	kN/m²	8,88	γ_f=	1,35
Imposed, qk	kN/m²	2,00	γ_f=	1,50
Design load, n	kN/m²	14,98		

EDGE CONDITIONS

Edge 1	C	C = Continuous
Edge 2	C	D = Discontinuous
Edge 3	D	
Edge 4	D	

See Figure 3.8 and clauses 3.5.3.5-6

MAIN STEEL

		SHORT SPAN	LONG SPAN	EDGE 1 Continuous	EDGE 2 Continuous	EDGE 3 Free	EDGE 4 Free	BS8110 Reference
ßs		0,070	0,034	0,093	0,045	0,000	0,000	Table 3.14
M	kNm/m	21,2	10,3	28,3	13,8	0,0	0,0	
d	mm	146,0	138,0	146,0	138,0	146,0	138,0	
k'		0,156	0,156	0,156	0,156	0,156	0,156	
k		0,040	0,022	0,053	0,029	0,000	0,000	
Z	mm	138,7	131,1	136,8	131,1	138,7	131,1	3.4.4.4
As req	mm²/m	352	181	476	241	0	0	
As min	mm²/m	228	228	228	228	228	228	Table 3.25
As deflection	mm²/m	353	181	~	~	~	~	
Ø	mm	8	8	8	8	8	8	
Layer		B 1	B 2	T 1	T 2	T 1	T 2	
@	mm	125	275	100	200	200	200	
As prov	mm²/m	402	183	503	251	251	251	
=	%	0,275	0,132	0,344	0,182	0,172	0,182	%
S max	mm	446	422	446	422	446	422	Clause
Subclause		(a)	(a)	(a)	(a)	(a)	(a)	3.12.11.2.7
DEFLECTION								
fs		292	330	316	320	0	0	Eqn 8
Mod factor		1,364						Eqn 7
Perm L/d		35,47	Actual L/d	30,82	As enhanced 0,3% for deflection control			Table 3.10

TORSION STEEL

			BOTH EDGES DISCONTINUOUS X		BOTH EDGES DISCONTINUOUS Y	ONE EDGE DISCONTINUOUS X		ONE EDGE DISCONTINUOUS Y	
Ø	mm	10							
As req	mm²/m		251	264			228		3.5.3.5
As prov T	mm²/m				251	251		251	
Additional As T req	mm²/m		14		14	0		0	
As prov B	mm²/m		402		183	402		183	

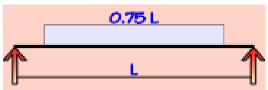
Bottom steel not curtailed in edge strips at free edges

SUPPORT REACTIONS (kN/m char uno) *(See Figure 3.10)*

Sum ßvx = 1,003
Sum ßvy = 0,660

		EDGE 1 1, H-D	EDGE 2 D, 2-1	EDGE 3 2, H-D	EDGE 4 H, 2-1
ßv		0,602	0,396	0,401	0,264
Dead	kN/m	24,04	15,82	16,03	10,54
Imposed	kN/m	5,42	3,56	3,61	2,38
Vs	kN/m	40,6	26,7	27,1	17,8

Table 3.15, equations 19 & 20



OUTPUT/SUMMARY

	SHORT SPAN	LONG SPAN	EDGE 1 1, H-D	EDGE 2 D, 2-1	EDGE 3 2, H-D	EDGE 4 H, 2-1
PROVIDE MAIN STEEL	T8 @ 125 B1	T8 @ 275 B2	T8 @ 100 T1	T8 @ 200 T2	T8 @ 200 T1	T8 @ 200 T2

ADDITIONAL TORSION STEEL	0	CORNER 2	CORNER 3	CORNER 4	
	0	D1	D2	H2	
X direction	0			1 T10 T	*placed in edge strips*
Y direction	0			1 T10 T	

CHECKS Lx > Ly	BAR Ø < COVER	SINGLY REINFORCED	MIN SPACING	MAX SPACING	DEFLECTION	GLOBAL STATUS
OK	OK	OK	OK	OK	OK	VALID DESIGN

- Spreadsheet calculations of panel P2: 2 ways slab

Project	RESIDENTIAL BUILDINGS	REINFORCED CONCRETE COUNCIL	
Client	POLO VIEWS	Made by: E.A	Date: 13-oct-2015 / Page: 130
Location	32nd floor - corner panel 5' to 10: G to E	Checked: E.A	Revision: - / Job No
	2-WAY SPANNING INSITU CONCRETE SLABS to BS 8110:1997 (Table 3.14)		
	Originated from RCC84.xls on CD © 1999 BCA for RCC		

DIMENSIONS

short span, lx	m	4,76
long span, ly	m	8,28
h	mm	175
Top cover	mm	25
Btm cover	mm	25

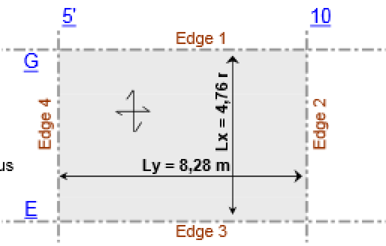
MATERIALS

fcu	N/mm²	25	γc =	1,50
fy	N/mm²	500	γs =	1,15
Density	kN/m³	25		

(Normal weight concrete)

STATUS VALID DESIGN

Plan



LOADING *characteristic*

Self weight	kN/m²	4,38		
Extra dead	kN/m²	4,50		
Total Dead, gk	kN/m²	8,88	γf=	1,35
Imposed, qk	kN/m²	2,00	γf=	1,50
Design load, n	kN/m²	14,98		

EDGE CONDITIONS

Edge 1	D	C = Continuous
Edge 2	C	D = Discontinuous
Edge 3	C	
Edge 4	C	

See Figure 3.8 and clauses 3.5.3.5-6

MAIN STEEL		SHORT SPAN	LONG SPAN	EDGE 1 Free	EDGE 2 Continuous	EDGE 3 Continuous	EDGE 4 Continuous	BS8110 Reference
ßs		0,061	0,028	0,000	0,037	0,082	0,037	Table 3.14
M	kNm/m	20,8	9,3	0,0	12,4	27,8	12,4	
d	mm	146,0	138,0	146,0	138,0	146,0	138,0	
k'		0,156	0,156	0,156	0,156	0,156	0,156	
k		0,039	0,020	0,000	0,026	0,052	0,026	
Z	mm	138,7	131,1	138,7	131,1	137,0	131,1	3.4.4.4
As req	mm²/m	345	164	0	218	466	218	
As min	mm²/m	228	228	228	228	228	228	Table 3.25
As deflection	mm²/m	362	171	~	~	~	~	
Ø	mm	8	8	8	8	8	8	
Layer		B 1	B 2	T 1	T 2	T 1	T 2	
@	mm	125	275	200	200	100	200	
As prov	mm²/m	402	183	251	251	503	251	
=	%	0,275	0,132	0,172	0,182	0,344	0,182	%
S max	mm	446	422	446	422	446	422	Clause
Subclause		(a)	(a)	(a)	(a)	(a)	(a)	3.12.11.2.7
DEFLECTION								
fs		286	299	0	290	309	290	Eqn 8
Mod factor		1,396						Eqn 7
Perm L/d		36,30	Actual L/d	32,60	As enhanced 4,7% for deflection control			Table 3.10

TORSION STEEL

			BOTH EDGES DISCONTINUOUS X		BOTH EDGES DISCONTINUOUS Y	ONE EDGE DISCONTINUOUS X		ONE EDGE DISCONTINUOUS Y	
Ø	mm	10							
As req	mm²/m		5000	259			228		3.5.3.5
As prov T	mm²/m				5000	251		251	
Additional As T req	mm²/m		0		0	0		0	
As prov B	mm²/m		402		183	402		183	

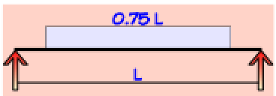
Bottom steel not curtailed in edge strips at free edges

SUPPORT REACTIONS (kN/m char uno) *(See Figure 3.10)*

Sum ßvx = 0,919 Table 3.15

Sum ßvy = 0,720

		EDGE 1 G, 5'-10	EDGE 2 10, E-G	EDGE 3 E, 5'-10	EDGE 4 5', E-G
ßv		0,368	0,360	0,552	0,360
Dead	kN/m	15,54	15,21	23,30	15,21
Imposed	kN/m	3,50	3,43	5,25	3,43
Vs	kN/m	26,2	25,7	39,3	25,7



equations 19 & 20

OUTPUT/SUMMARY

	SHORT SPAN	LONG SPAN	EDGE 1 G, 5'-10	EDGE 2 10, E-G	EDGE 3 E, 5'-10	EDGE 4 5', E-G
PROVIDE MAIN STEEL	T8 @ 125 B1	T8 @ 275 B2	T8 @ 200 T1	T8 @ 200 T2	T8 @ 100 T1	T8 @ 200 T2

ADDITIONAL TORSION STEEL		CORNER 2	CORNER 3	CORNER 4
	0			
	0	10G	10E	5'E
X direction	0			
Y direction	0			

placed in edge strips

CHECKS	BAR Ø	SINGLY	MIN	MAX		GLOBAL
Lx > Ly	< COVER	REINFORCED	SPACING	SPACING	DEFLECTION	STATUS
OK	OK	OK	OK	OK	OK	VALID DESIGN

- Spreadsheet calculations of panel (P3): one way slab

Project	Residential building	REINFORCED CONCRETE COUNCIL			
Client	POLO VIEWS	Made by	PYY	Date	12-oct-2015
Location	ACCRA SPINTEX	Page	116		
	1-WAY SOLID CONCRETE SLAB DESIGN to BS 8110:1997 Table	Checked	AKBF	Revision	-
	Originated from RCC91.xls on CD © 1999 BCA for RCC	Job No	R68		

LOCATION Supports from grid 9 to grid 14
End support condition is C (C)ontinuous or (S)imple

STATUS
VALID DESIGN

DIMENSIONS

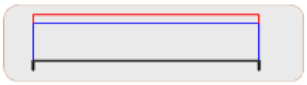
Nº of spans	Nº	1
Max Span	m	3,900
Thickness, h	mm	175
cover	mm	25

MATERIALS

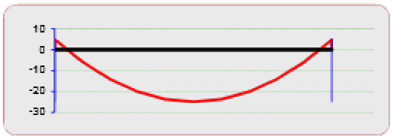
fy	N/mm²	500	γs = 1,15
fcu	N/mm²	25	γc = 1,50
Density	kN/m³	25	
(Normal weight concrete)			

LOADING

Self Weight	kN/m²	4,38
Additional Dead	kN/m²	4,50
Total Dead, gk	kN/m²	8,88
Imposed Load, qk	kN/m²	2,00
Design load, n =	kN/m²	15,63



Geometry and Loading



Indicative Bending Moment Diagram

MAIN STEEL

		END SUPPORTS (9 & 14)	END SPANS	FIRST INT SUPPORTS	INTERIOR SPANS	INTERNAL SUPPORTS	BS 8110 Reference
Factor		0,040	0,105	0,000	0,000	0,000	Table 3.12
M	kNm/m	9,5	25,0	0,0	0,0	0,0	
d	mm	146	145				
K		0,018	0,047				M/bd²fcu
z		138,7	136,9				Clause 3.4.4.4
As	mm²/m	158	419				
Rebar		T	T				
Ø	mm	8	10	10	10	10	
@	mm c/c	200	150				
As prov	mm²/m	251	524				
=	%	0,172	0,361				
Max S		446	445				Clause
subclause		(a)	(a)				3.12.11.2.7

DEFLECTION

		END SUPPORTS	END SPANS	FIRST INT SUPPORTS	INTERIOR SPANS	INTERNAL SUPPORTS	
fs	N/mm²	209	267				Eqn 8
Top steel provided	% bd		0		0		
Comp Mod factor			1,000				Table 3.11
Tens Mod factor			1,389				Eqn 7
Perm L/d			27,8	As auto-increased			Table 3.9
Actual L/d			26,8	by 18 %			

DISTRIBUTION STEEL As = 0,13% = 227,5 mm²/m Table 3.25
Provide T 8 at 200 = 251 mm²/m

SHEAR

		END SUPPORT	FIRST INT SUPT	INTERNAL SUPTS	
V	kN/m	30,5			Table 3.12
As prov	%	0,181			
v	N/mm²	0,210			equation 3
vc	N/mm²	0,460			Table 3.8

OUTPUT/SUMMARY

	END SUPPORTS	END SPANS	FIRST INT SUPPORTS	INTERIOR SPANS	INTERNAL SUPPORTS	DISTRIBUTION
PROVIDE	T8 @ 200 T1	T10 @ 150 B1				T8 @ 200

CHECKS

BAR Ø < COVER	SINGLY REINFORCED	BAR SPACING	DEFLECTION	NO SHEAR LINKS	GLOBAL STATUS
OK	OK	OK	OK	OK	VALID DESIGN

- Spreadsheet calculations of panel (P4): CANTILEVER

ELEMENT DESIGN to BS 8110:1997
SOLID SLABS
Originated from RCC11.xls on CD © 1999 BCA for RCC

OPERATING INSTRUCTIONS
ENTER DATA IN BLUE CELLS ONLY
COPY OUTPUT SECTION & PASTE WHERE YOU WANT.

INPUT Location D&D: interior span solid slab
Design moment, M 7,7 kNm/m fcu 25 N/mm² γc = 1,50
ßb 1,00 fy 500 N/mm² γs = 1,15
span 2 mm
Height, h 175 mm Section location CANTILEVER
Bar Ø 10 mm
cover 25 mm to this reinforcement

Choose SECTION LOCATION here
CANTILEVER

COMPRESSION STEEL
Nominal
(deflection control only)

OUTPUT D&D: interior span solid slab Compression steel = Nominal
d = 175 - 25 - 10/2 = 145,0 mm
(3.4.4.4) K' = 0,156 > K = 0,015 ok
(3.4.4.4) z = 145,0 [0.5 + (0.25 - 0,015 /0.9)^½ = 142,6 > 0.95d = 137,8 mm
(3.4.4.1) As = 7,72E6 /500 /137,8 x 1,15 = 129 < min As = 228 mm²/m
PROVIDE Y10 @ 350 = 224 mm²/m .
(Eqn 8) fs = 2/3 x 500 x 129 /224 /1,00 = 191,6 N/mm²
(Eqn 7) Tens mod factor = 0.55 + (477 - 191,6) /120 /(0.9 + 0,367) = 2,000
(Equation 9) Comp mod factor = 1 + 0.13/(3 + 0.13) = 1,042
(3.4.6.3) Permissible L/d = 7,0 x 2,000 x 1,042 = 14,5
Actual L/d = 2 /145,0 = 0,014 ok

CHECKS
Deflection is ok
Maximum spacing is ok
Minimum spacing is ok

- Maximum spacing, cracking and lapping

Maximum spacing: $\boxed{s \geq 3h = 525\ mm}$

Crack control: [2]

According to the eurocode2, there is no need to control cracks when these conditions are verified:

- $h \leq 200\ mm$
- Minimum and maximum steel percentages in the main direction are verified;
- The spacing of bars should not exceed S_{max}.

Lapping calculation: Conditions mentioned in EN 1992.1.1-2004 (p138) will be applied. The length calculations are as follows:

$l_0 = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5\alpha_6\ l_{b,rqd} \geq l_{0,min}$

Values of $\alpha_1,\ \alpha_2,\ \alpha_3,\ \alpha_4,\ \alpha_5$ are to be picked from table 8.2 EN 1992.1.1-2004 p124

$l_0 = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5\alpha_6\ l_{b,rqd} \geq l_{0,min}$

Further details are given in the code. For our calculation we have:

$\alpha_1 = 1;\ \alpha_2 = 0.78;\ \alpha_3 = 0.7;\ \alpha_4 = 0.7;\ \alpha_5 = 0.7;\ \alpha_6 = 1.4$

$l_{b,rqd} = \left(\emptyset/_4\right)\left(\sigma_{sd}/_{f_{bd}}\right)$; where $l_{b,rqd}$ is the basic required anchorage length, f_{bd} is bond stress, σ_{sd} is the design stress of the bars and $\emptyset$ is the diameter of the bars.

Hence $\underline{l_0 = 245\ mm}$

$l_{0,min} \geq max\{0.3\alpha_6 l_{b,rqd}; 15\emptyset; 200mm\}$

$\underline{l_{0,min} \geq 525\ mm}$

The lapping length is then $\boxed{l_0 = 525\ mm}$

ANNEX 4 : Images of the software calculations

❖ Lateral loads

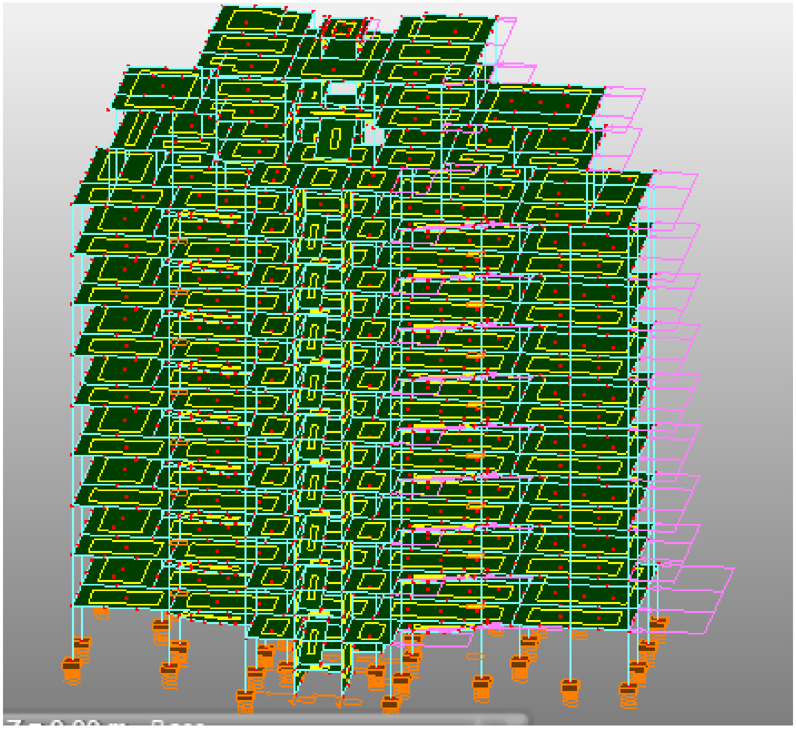


Figure 6: Wind load 1+

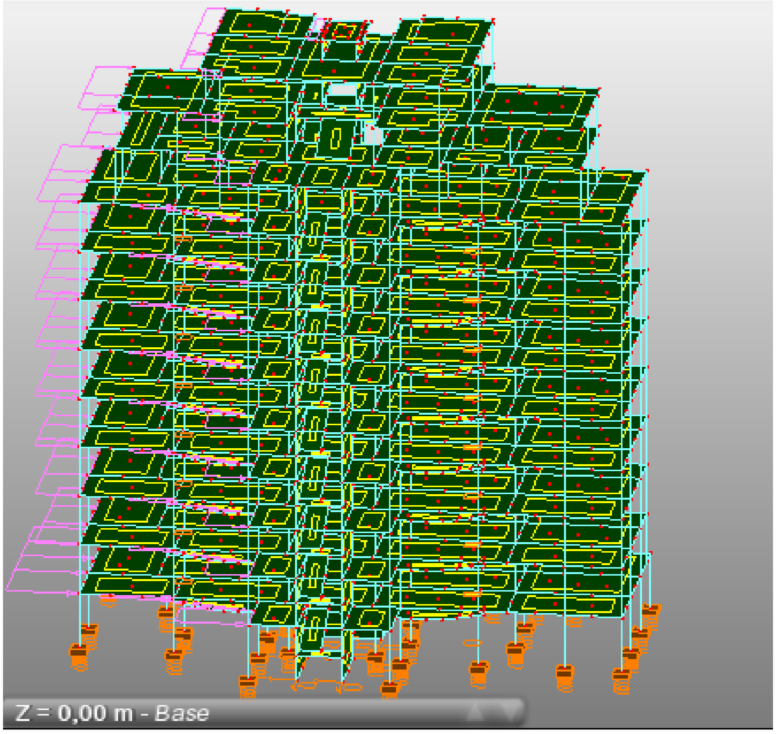


Figure 7: Wind load 1-

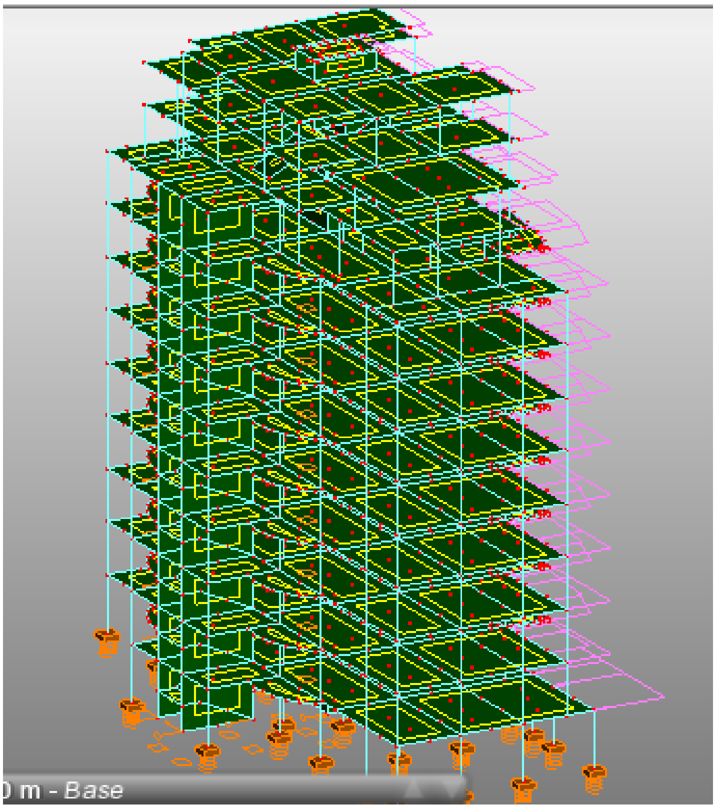


Figure 8: Wind load 2+

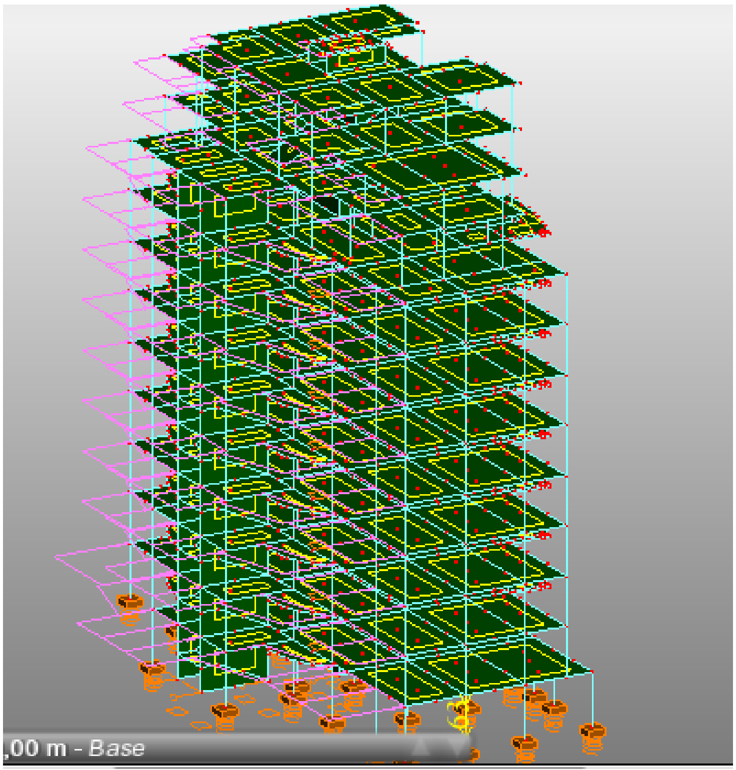


Figure 9: Wind load 2-

❖ Displacement Results

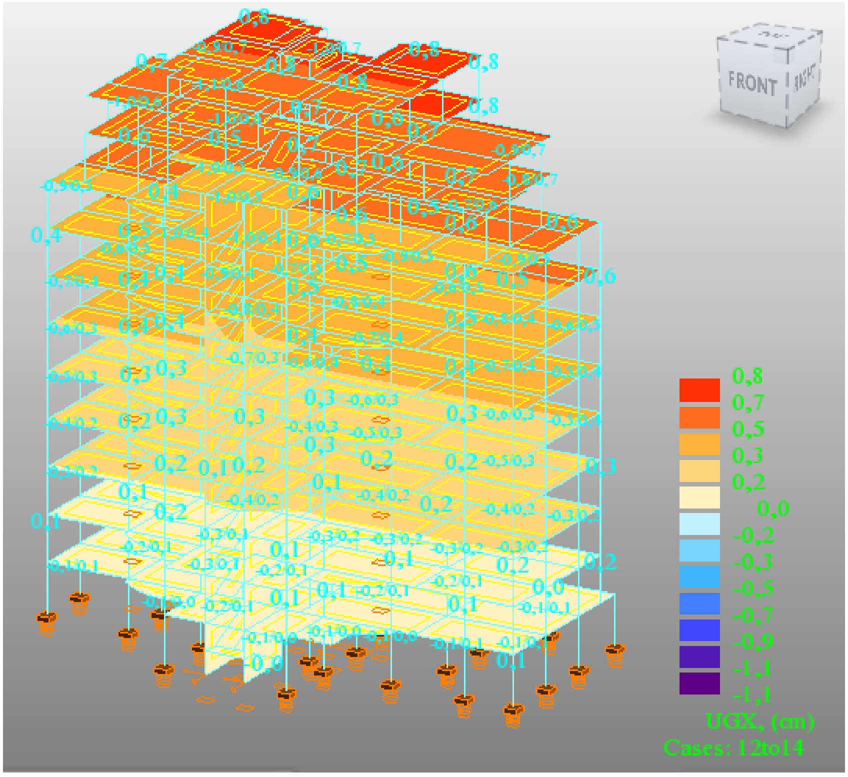


Figure 10: Displacement [UX]

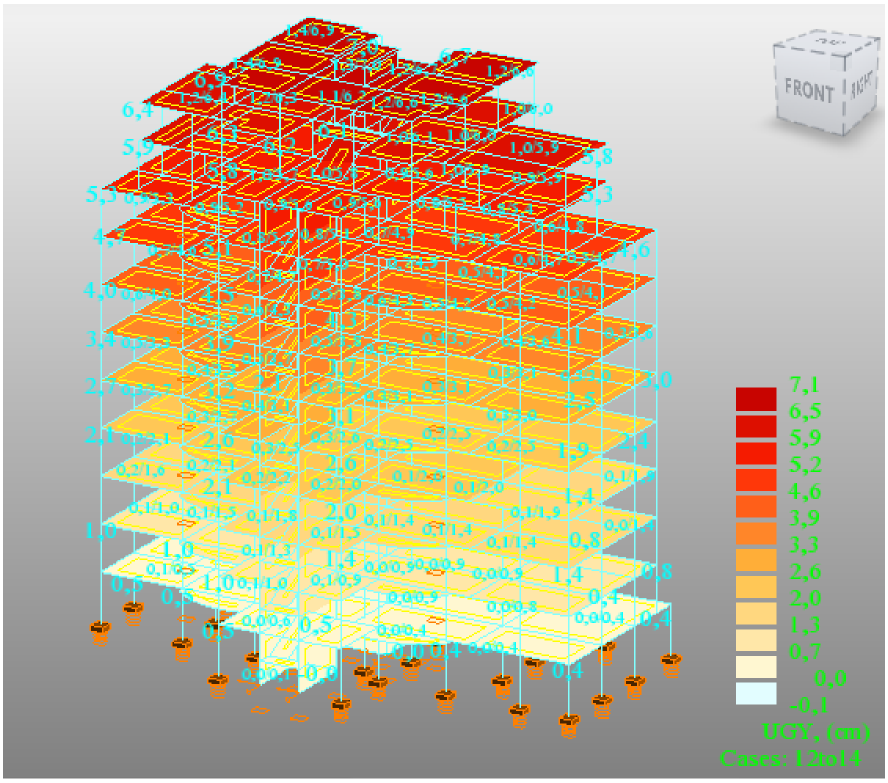


Figure 11: Displacement [UY]

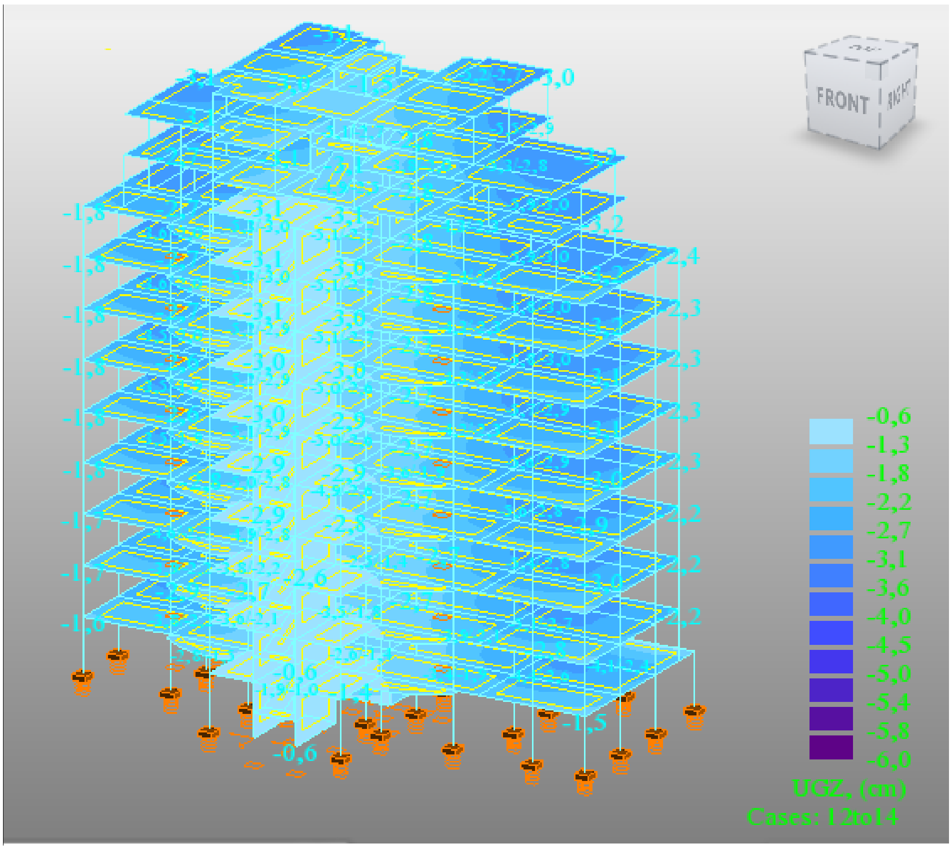


Figure 12: Displacement [UZ]

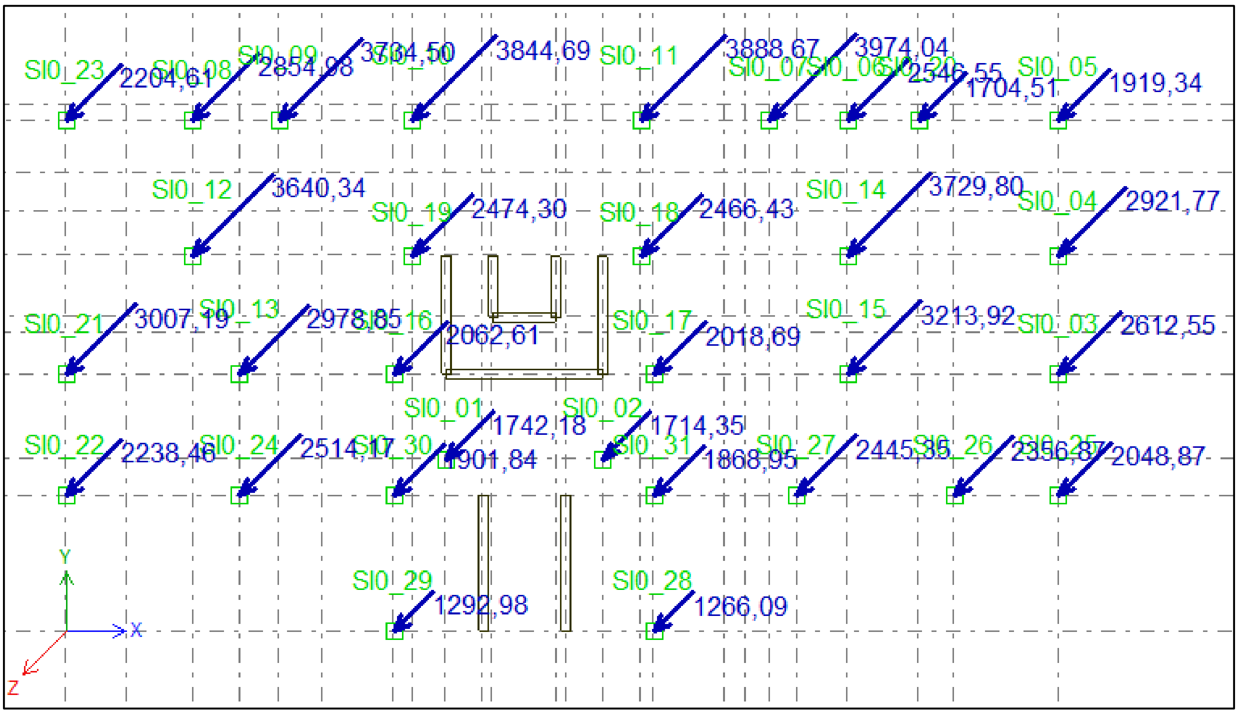


Figure 13: Axial forces (Fx) on footings

ANNEX 5 : Beams calculation details and reinforcement detailing

This annex is going to show the calculation report of the beam calculation. Depend on the arrangement of the beams on the ground floor slab, here are the calculations related to the beams (POU0_22 & POU0_08) followed by their detailing which have been done with Robot RSA.

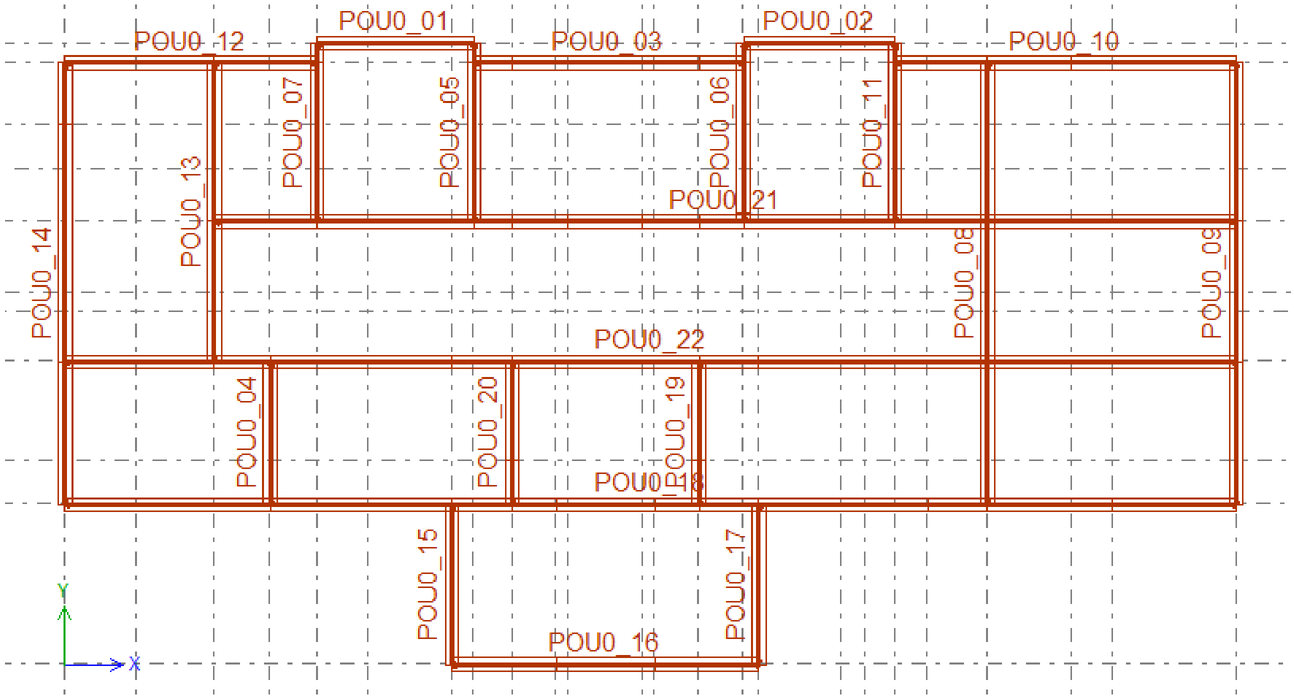


Figure 14: Beams Arrangement

a) Report of beam POU0_22 calculations

Autodesk Robot Structural Analysis Professional 2012	
Author:	File: **POLO VIEWS DESIGN.rtd**
Address:	Project: POLO VIEWS DESIGN

1 Level:

- Name : 0
- Reference level : 3,60 (m)
- Maximum cracking : 0,40 (mm)
- Exposure : XC1
- Concrete creep coefficient : φ_p = No results
- cement class : N
- Concrete age (loading moment) : 28 (days)
- Concrete age : 50 (years)
- Structure class : S4
- Fire resistance class : R 60(EN 1992-1-2:2004)

2 Beam: 1_POU0_22 Number: 1

2.1 Material properties:

- Concrete : C25/30 f_{ck} = 25,00 (MPa)
  Rectangular stress distribution [3.1.7 (3)]
  Density : 2501,36 (kG/m3)
  Aggregate size : 20,0 (mm)
- Longitudinal reinforcement: : B500B f_{yk} = 500,00 (MPa)
  Horizontal branch of the stress-strain diagram
  Ductility class : B
- Transversal reinforcement: : B500B f_{yk} = 500,00 (MPa)

2.2 Geometry:

2.2.1

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P1	**Span**	**0,40**	**5,60**	**0,70**

Span length: L_o = 6,15 (m)
Section from 0,00 to 5,60 (m)
50,0 x 80,0 (cm)
without left slab
without right slab

2.2.2

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P2	**Span**	**0,70**	**4,75**	**0,70**

Span length: L_o = 5,45 (m)
Section from 0,00 to 4,75 (m)
50,0 x 80,0 (cm)
without left slab
without right slab

2.2.3

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P3	**Span**	**0,70**	**1,34**	**0,25**

Span length: L_o = 1,81 (m)
Section from 0,00 to 1,34 (m)

Date : 05/11/15 Page : **1**

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:
File: **POLO VIEWS DESIGN.rtd**
Project: POLO VIEWS DESIGN

50,0 x 80,0 (cm)
without left slab
without right slab

2.2.4 Span Position L.supp. L R.supp.

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P4	**Span 0,25**	**5,34**	**0,25**	

Span length: L_o = 5,59 (m)
Section from 0,00 to 5,34 (m)
50,0 x 80,0 (cm)
without left slab
without right slab

2.2.5

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P5	**Span 0,25**	**1,33**	**0,70**	

Span length: L_o = 1,80 (m)
Section from 0,00 to 1,33 (m)
50,0 x 80,0 (cm)
without left slab
without right slab

2.2.6

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P6	**Span 0,70**	**6,30**	**0,40**	

Span length: L_o = 6,85 (m)
Section from 0,00 to 6,30 (m)
50,0 x 80,0 (cm)
without left slab
without right slab

2.2.7

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P7	**Span 0,40**	**7,05**	**0,40**	

Span length: L_o = 7,45 (m)
Section from 0,00 to 7,05 (m)
50,0 x 80,0 (cm)
without left slab
without right slab

2.3 Calculation options:

- Regulation of combinations : CBS_Pro_EN_1992_2004
- Calculations according to : EN 1992-1-1:2004
- Seismic dispositions : No requirements
- Precast beam : no
- Cover : bottom c = 2,9 (cm)
  : side c1= 2,9 (cm)
  : top c2= 2,9 (cm)
- Cover deviations : Cdev = 1,0(cm), Cdur = 0,0(cm)
- Coefficient β_2 =0.50 : long-term or cyclic load
- Method of shear calculations : strut inclination

2.4 Loads:

Date : 05/11/15 Page : **2**

γf - load factor

2.5 Calculation results:

2.5.1 Reactions

NO INFORMATION PROVIDED!

2.5.2 Internal forces in ULS

Span	Mt max. (kN*m)	Mt min. (kN*m)	Ml (kN*m)	Mr (kN*m)	Ql (kN)	Qr (kN)
P1	201,82	-0,00	49,98	-110,17	96,84	-191,61
P2	533,72	-326,74	533,72	-705,13	-142,01	-399,17
P3	33,91	-441,60	-100,23	-477,83	-372,58	-385,84
P4	62,46	-21,79	-217,92	-212,34	186,26	-184,60
P5	32,48	-437,75	-472,62	-102,78	383,55	370,39
P6	376,55	-261,81	-755,68	376,55	365,67	23,21
P7	322,80	-0,00	122,08	52,90	129,54	-102,67

2.5.3 Internal forces in SLS

Span	Mt max. (kN*m)	Mt min. (kN*m)	Ml (kN*m)	Mr (kN*m)	Ql (kN)	Qr (kN)
P1	146,79	0,00	-22,02	-79,20	70,65	-140,16
P2	390,24	-147,17	390,24	-515,37	-103,62	-291,49
P3	23,73	-231,54	23,73	-348,73	-272,01	-281,83
P4	45,61	-12,11	-159,08	-154,98	136,06	-134,82
P5	22,68	-229,19	-344,80	22,68	280,06	270,31
P6	275,08	-108,92	-551,96	275,08	266,83	16,28
P7	234,79	0,00	62,69	-35,22	94,44	-74,95

2.5.4 Required reinforcement area

Span	Span (cm2)		Left support (cm2)		Right support (cm2)	
	bottom	top	bottom	top	bottom	top
P1	6,41	0,00	1,50	0,91	1,09	3,40
P2	17,66	0,00	17,66	0,00	0,00	23,90
P3	1,05	0,00	1,04	3,09	0,00	15,70
P4	1,91	0,00	0,00	6,93	0,00	6,75
P5	1,01	0,00	0,00	15,52	1,00	3,17
P6	12,21	0,00	0,00	25,80	12,21	0,00
P7	10,40	0,00	3,84	0,00	1,56	1,43

2.5.5 Fire resistance

Fire resistance :R 90(EN 1992-1-2:2004)
Calculations according to :EN 1992-1-2:2004
Estimation in accordance with section 5. Tabulated data.
Number of sides exposed to fire :3
Web type :WA
Beam type :continuous
b_min = 0,12(m)
a_min = 0,01(m)

2.5.6 Deflection and cracking

wt(QP) Total due to quasi-permanent combination
wt(QP)dop Allowable due to quasi-permanent combination
Dwt(QP) Deflection increment from the quasi-permanent load combination after erecting a structure.
Dwt(QP)dop Admissible deflection increment from the quasi-permanent load combination after erecting a structure.

wk - width of perpendicular cracks

Autodesk Robot Structural Analysis Professional 2012
Author: File: **POLO VIEWS DESIGN.rtd**
Address: Project: POLO VIEWS DESIGN

Span	wt(QP) (cm)	wt(QP)dop (cm)	Dwt(QP) (cm)	Dwt(QP)dop (cm)	wk (mm)
P1	0,0	2,5	0,0	0,0	0,0
P2	0,0	2,2	0,0	0,0	0,0
P3	0,0	0,7	0,0	0,0	0,0
P4	0,0	2,2	0,0	0,0	0,0
P5	0,0	0,7	0,0	0,0	0,0
P6	0,0	2,7	0,0	0,0	0,0
P7	0,0	3,0	0,0	0,0	0,0

2.5.7 Compressive stress in strut

		h (m)	h gen (m)	σbc A (MPa)	Atheor (cm2)	Ar (cm2)
Span P1	Left support ΘA = 52,3 (Deg) a = 0,24 (m) Vu = 96,84(kN) Lower diagonal	0,26	-	0,93	0,71	15,71
Span P1	Right support ΘA = 45,7 (Deg) a = 0,70 (m) Vu = 191,61(kN) Lower diagonal	0,50	-	1,07	2,63	18,85
Span P2	Left support ΘA = 45,7 (Deg) a = 0,70 (m) Vu = 0,00(kN) Lower diagonal	0,50	-	0,79	3,18	18,85
Span P2	Right support ΘA = 45,8 (Deg) a = 0,70 (m) Vu = 399,17(kN) Lower diagonal	0,50	-	2,22	0,00	15,71
Span P3	Left support ΘA = 45,8 (Deg) a = 0,70 (m) Vu = 0,00(kN) Lower diagonal	0,50	-	2,07	8,33	15,71
Span P3	Right support ΘA = 56,2 (Deg) a = 0,25 (m) Vu = 385,84(kN) Lower diagonal	0,21	-	4,47	0,00	15,71
Span P4	Left support ΘA = 56,2 (Deg) a = 0,25 (m) Vu = 186,26(kN) Lower diagonal	0,21	-	2,16	0,00	15,71
Span P4	Right support ΘA = 56,2 (Deg)					

Date : 05/11/15 Page : **4**

Autodesk Robot Structural Analysis Professional 2012	
Author:	File: **POLO VIEWS DESIGN.rtd**
Address:	Project: POLO VIEWS DESIGN

	a = 0,25 (m) Vu = 184,60(kN) Lower diagonal	0,21	-	2,14	0,00	15,71
Span P5	Left support ΘA = 56,2 (Deg) a = 0,25 (m) Vu = 383,55(kN) Lower diagonal	0,21	-	4,44	0,00	15,71
Span P5	Right support ΘA = 45,8 (Deg) a = 0,70 (m) Vu = 0,00(kN) Lower diagonal	0,50	-	2,06	8,28	15,71
Span P6	Left support ΘA = 45,8 (Deg) a = 0,70 (m) Vu = 365,67(kN) Lower diagonal	0,50	-	2,04	0,00	15,71
Span P6	Right support ΘA = 52,4 (Deg) a = 0,40 (m) Vu = 0,00(kN) Lower diagonal	0,32	-	0,18	0,41	15,71
Span P7	Left support ΘA = 52,4 (Deg) a = 0,40 (m) Vu = 129,54(kN) Lower diagonal	0,32	-	1,03	2,29	15,71
Span P7	Right support ΘA = 52,3 (Deg) a = 0,24 (m) Vu = 102,67(kN) Lower diagonal	0,26	-	0,98	0,27	15,71

2.6 Reinforcement:

2.6.1 P1 : Span from 0,40 to 6,00 (m)

Longitudinal reinforcement:

- bottom (B500B)
  5 ϕ20 l = 5,54 from 0,14 to 5,69
- support (B500B)
  5 ϕ20 l = 4,44 from 0,04 to 4,35
  5 ϕ20 l = 8,18 from 2,05 to 10,23

Transversal reinforcement:

- main (B500B)
  stirrups 40 ϕ10 l = 2,50
  e = 1*0,14 + 19*0,28 (m)

  pins 40 ϕ10 l = 2,50
  e = 1*0,14 + 19*0,28 (m)

2.6.2 P2 : Span from 6,70 to 11,45 (m)

Longitudinal reinforcement:

Date : 05/11/15	Page : **5**

- bottom (B500B)
  5 ϕ20 l = 7,23 from 4,07 to 11,31
  1 ϕ20 l = 2,02 from 5,82 to 7,84
- support (B500B)
  5 ϕ20 l = 6,04 from 7,92 to 13,97

Transversal reinforcement:

- main (B500B)
  stirrups 36 ϕ10 l = 2,50
  e = 11*0,28 + 7*0,22 (m)

  pins 36 ϕ10 l = 2,50
  e = 11*0,28 + 7*0,22 (m)

2.6.3 P3 : Span from 12,15 to 13,49 (m)

Longitudinal reinforcement:

- support (B500B)
  4 ϕ20 l = 4,01 from 9,95 to 13,97

Transversal reinforcement:

- main (B500B)
  stirrups 14 ϕ10 l = 2,50
  e = 1*0,01 + 6*0,22 (m)

  pins 14 ϕ10 l = 2,50
  e = 1*0,01 + 6*0,22 (m)

2.6.4 P4 : Span from 13,74 to 19,08 (m)

Longitudinal reinforcement:

- bottom (B500B)
  5 ϕ20 l = 9,12 from 9,69 to 18,81
- support (B500B)
  9 ϕ20 l = 5,89 from 11,67 to 17,56
  9 ϕ20 l = 5,88 from 15,25 to 21,14

Transversal reinforcement:

- main (B500B)
  stirrups 40 ϕ10 l = 2,50
  e = 1*0,01 + 19*0,28 (m)

  pins 40 ϕ10 l = 2,50
  e = 1*0,01 + 19*0,28 (m)

2.6.5 P5 : Span from 19,33 to 20,65 (m)

Longitudinal reinforcement:

- bottom (B500B)
  5 ϕ20 l = 6,21 from 17,20 to 23,42
- support (B500B)
  4 ϕ20 l = 4,11 from 18,84 to 22,95

Transversal reinforcement:

- main (B500B)
  stirrups 14 ϕ10 l = 2,50
  e = 1*0,00 + 6*0,22 (m)

  pins 14 ϕ10 l = 2,50
  e = 1*0,00 + 6*0,22 (m)

2.6.6 P6 : Span from 21,35 to 27,65 (m)

Longitudinal reinforcement:

- bottom (B500B)
  5 ϕ20 l = 8,46 from 21,80 to 30,27
- support (B500B)
  5 ϕ20 l = 6,81 from 18,84 to 25,65

Transversal reinforcement:

- main (B500B)

stirrups 46 ϕ10 l = 2,50
e = 1*0,18 + 8*0,24 + 14*0,28 (m)

pins 46 ϕ10 l = 2,50
e = 1*0,18 + 8*0,24 + 14*0,28 (m)

2.6.7 P7 : Span from 28,05 to 35,10 (m)

Longitudinal reinforcement:

- bottom (B500B)
  5 ϕ20 l = 6,70 from 28,65 to 35,36
- support (B500B)
  5 ϕ20 l = 9,38 from 23,35 to 32,73
  5 ϕ20 l = 5,16 from 30,42 to 35,46

Transversal reinforcement:

- main (B500B)

stirrups 52 ϕ10 l = 2,50
e = 1*0,03 + 25*0,28 (m)

pins 52 ϕ10 l = 2,50
e = 1*0,03 + 25*0,28 (m)

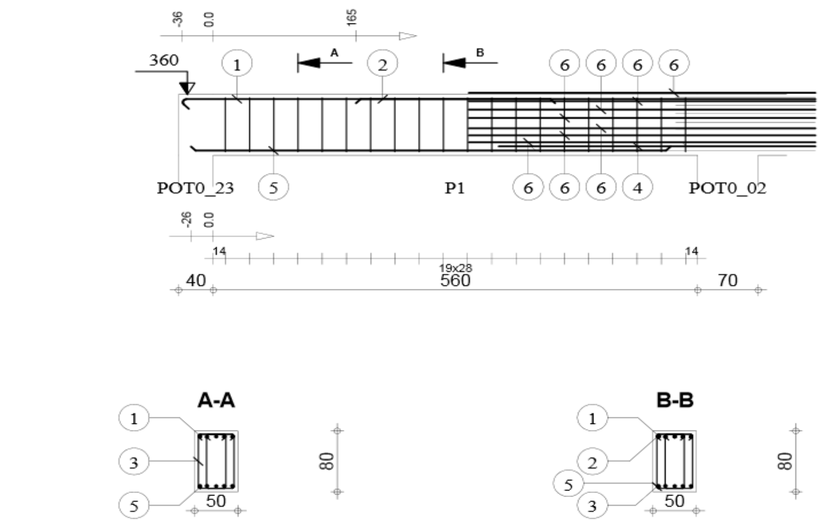
3 Material survey:

- Concrete volume = 14,20 (m3)
- Formwork = 73,45 (m2)

- Steel B500B
  - Total weight = 3533,35 (kG)
  - Density = 248,83 (kG/m3)
  - Average diameter = 16,8 (mm)
  - Survey according to diameters:

Diameter	Length (m)	Weight (kG)
10	604,61	372,89
20	1281,10	3160,46

b) Drawings of beam POU0_22 details



Pos.	Reinforcement		Code	Shape	Steel
1	5Ø20	l=444	00	431	B500B
2	5Ø20	l=818	00	818	B500B
3	40Ø10	l=250	41	33	B500B
4	6Ø20	l=381	00	381	B500B
5	5Ø20	l=554	00	554	B500B
6	6*3Ø20	l=450	00	450	B500B

Tel. Fax

Exposure class : XC1 | Fire resistance : R60 | Max. aggregate diameter : 20mm | Structure class : S4

Concrete : C25/30 = 2.54 m3

Steel B500B = 480 kg

Steel B500B = 61.6 kg

0

POLO VIEWS DESIGN

1_POU0_22 : P1

Section 50x80

Number 1

Formwork = 13.4 m2

Bottom cover 3.9 cm | Top cover 3.9 cm

Side cover 3.9 cm

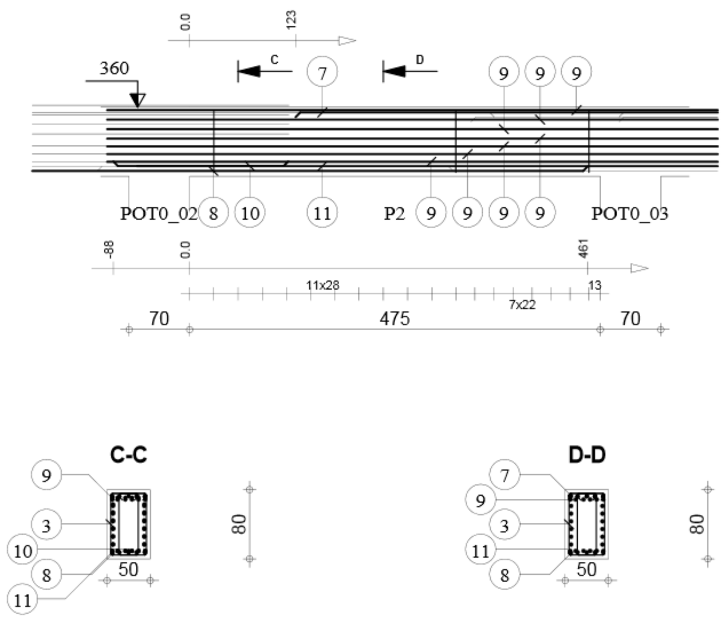
Density (unit weight) = 213 kg/ m3

Average diameter = 16.6mm

View scale 1/50

Section scale 1/50

Page 1/7



Pos.	Reinforcement		Code	Shape	Steel
3	36Ø10	l=250	41	33	B500B
7	5Ø20	l=604	00	604	B500B
8	5Ø20	l=723	00	723	B500B
9	6*3Ø20	l=1125	00	1125	B500B
10	1Ø20	l=203	00	202	B500B
11	6Ø20	l=1056	00	1056	B500B

Tel. Fax

Exposure class : XC1 | Fire resistance : R60 | Max. aggregate diameter : 20mm | Structure class : S4

Concrete : C25/30 = 2.18 m3

Steel B500B = 824 kg

Steel B500B = 55.5 kg

0

POLO VIEWS DESIGN

1_POU0_22 : P2

Section 50x80

Number 1

Formwork = 11.1 m2

Bottom cover 3.9 cm | Top cover 3.9 cm

Side cover 3.9 cm

Density (unit weight) = 403.7 kg/ m3

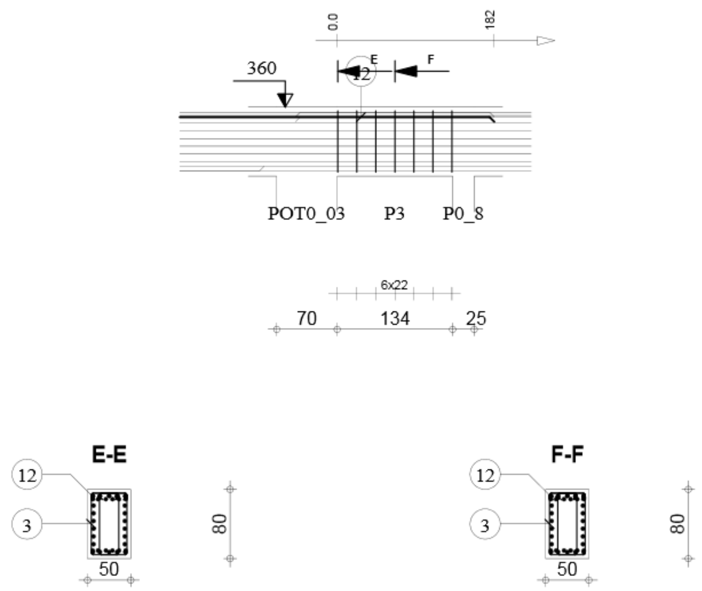
Average diameter = 17.9mm

View scale 1/50

Section scale 1/50

Page 2/7

Pos.	Reinforcement		Code	Shape	Steel
3	14Ø10	l=250	41		B500B
12	4Ø20	l=401	00	401	B500B



		Tel.	Fax
Exposure class : XC1	Fire resistance : R60	Max. aggregate diameter : 20mm	Structure class : S4

0

POLO VIEWS DESIGN

1_POU0_22 : P3 Number 1

Section 50x80

Concrete : C25/30 = 0.724 m3

Formwork = 3.56 m2

Density (unit weight) = 84.39 kg/m3

Average diameter = 13.1mm

Steel B500B = 39.6 kg

Steel B500B = 21.6 kg

Bottom cover 3.9 cm

Top cover 3.9 cm

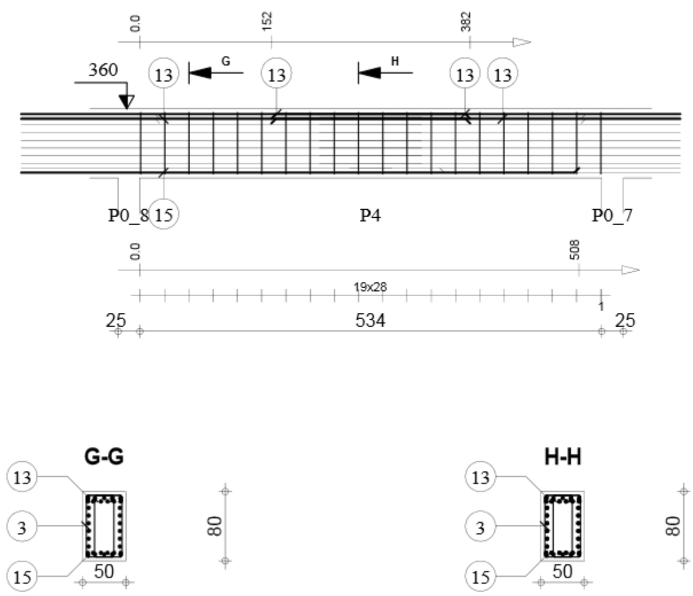
Side cover 3.9 cm

View scale 1/50

Section scale 1/50

Page 3/7

Pos.	Reinforcement		Code	Shape	Steel
3	40Ø10	l=250	41		B500B
13	18Ø20	l=589*	00	589	B500B
15	5Ø20	l=912	00	912	B500B



		Tel.	Fax
Exposure class : XC1	Fire resistance : R60	Max. aggregate diameter : 20mm	Structure class : S4

0

POLO VIEWS DESIGN

1_POU0_22 : P4 Number 1

Section 50x80

Concrete : C25/30 = 2.24 m3

Formwork = 11.6 m2

Density (unit weight) = 194.2 kg/m3

Average diameter = 16mm

Steel B500B = 374 kg

Steel B500B = 61.6 kg

Bottom cover 3.9 cm

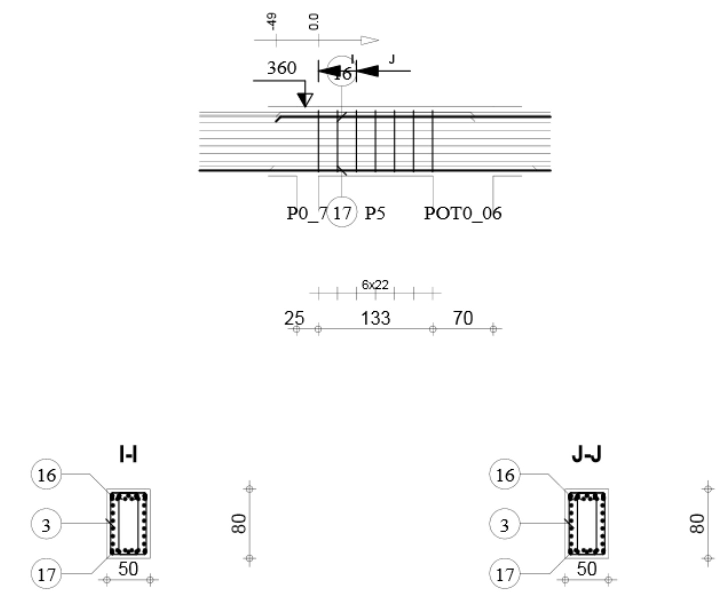
Top cover 3.9 cm

Side cover 3.9 cm

View scale 1/50

Section scale 1/50

Page 4/7



Pos.	Reinforcement		Code	Shape	Steel
3	14Ø10	l=250	41	[illegible]	B500B
16	4Ø20	l=411	00	411	B500B
17	5Ø20	l=621	00	621	B500B

Tel. Fax

Exposure class : XC1 Fire resistance : R60 Max. aggregate diameter : 20mm Structure class : S4

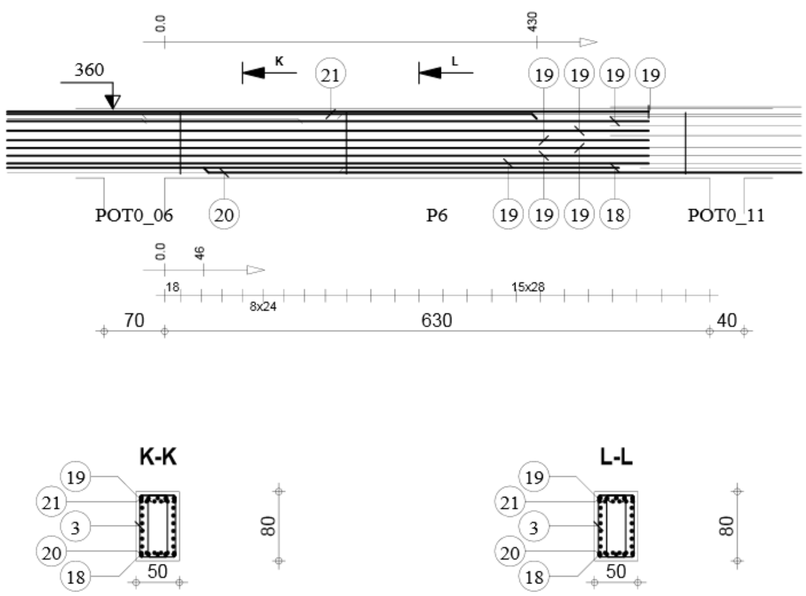
Concrete : C25/30 = 0.72 m3
Formwork = 3.54 m2
Density (unit weight) = 193.1 kg/m3
Average diameter = 15.8mm

Steel B500B = 117 kg
Steel B500B = 21.6 kg
Bottom cover 3.9 cm Top cover 3.9 cm
Side cover 3.9 cm
View scale 1/50
Section scale 1/50

0
POLO VIEWS DESIGN

1_POU0_22 : P5
Section 50x80
Number 1

Page 5/7



Pos.	Reinforcement		Code	Shape	Steel
3	46Ø10	l=250	41	[illegible]	B500B
18	6Ø20	l=1044	00	1044	B500B
19	6*3Ø20	l=1113	00	1113	B500B
20	5Ø20	l=846	00	846	B500B
21	5Ø20	l=681	00	681	B500B

Tel. Fax

Exposure class : XC1 Fire resistance : R60 Max. aggregate diameter : 20mm Structure class : S4

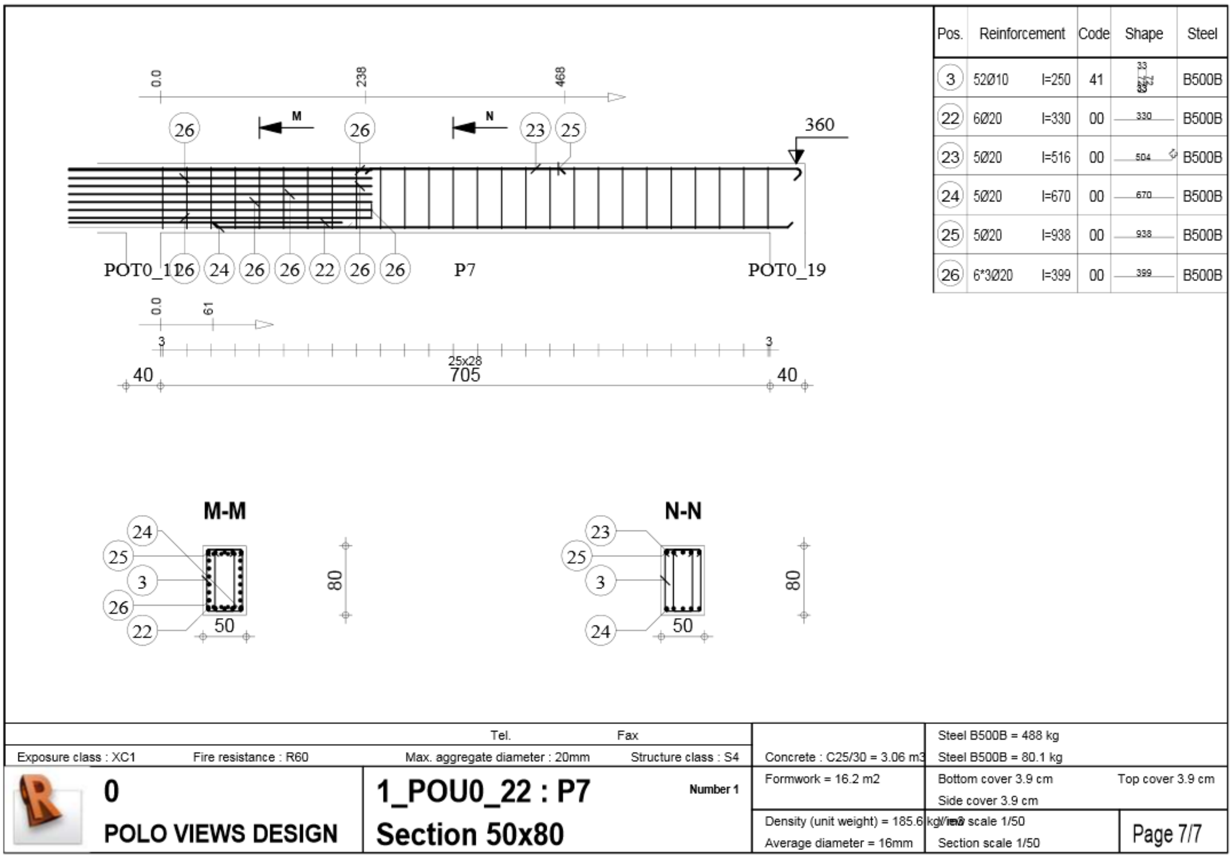
Concrete : C25/30 = 2.74 m3
Formwork = 14.1 m2
Density (unit weight) = 331.4 kg/m3
Average diameter = 17.5mm

Steel B500B = 837 kg
Steel B500B = 70.9 kg
Bottom cover 3.9 cm Top cover 3.9 cm
Side cover 3.9 cm
View scale 1/50
Section scale 1/50

0
POLO VIEWS DESIGN

1_POU0_22 : P6
Section 50x80
Number 1

Page 6/7



Pos.	Reinforcement		Code	Shape	Steel
3	52Ø10	l=250	41	33 33	B500B
22	6Ø20	l=330	00	330	B500B
23	5Ø20	l=516	00	504	B500B
24	5Ø20	l=670	00	670	B500B
25	5Ø20	l=938	00	938	B500B
26	6*3Ø20	l=399	00	399	B500B

c) Report of beam POU0_08 calculations

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:
File: **POLO VIEWS DESIGN.rtd**
Project: POLO VIEWS DESIGN

1 Level:

- Name : 0
- Reference level : 3,60 (m)
- Maximum cracking : 0,40 (mm)
- Exposure : XC1
- Concrete creep coefficient : φ_p = No results
- cement class : N
- Concrete age (loading moment) : 28 (days)
- Concrete age : 50 (years)
- Structure class : S4
- Fire resistance class : R 90(EN 1992-1-2:2004)

2 Beam: 1_POU0_08 Number: 1

2.1 Material properties:

- Concrete : C25/30 f_{ck} = 25,00 (MPa)
  Rectangular stress distribution [3.1.7 (3)]
  Density : 2501,36 (kG/m3)
  Aggregate size : 20,0 (mm)
- Longitudinal reinforcement: : B500B f_{yk} = 500,00 (MPa)
  Horizontal branch of the stress-strain diagram
  Ductility class : B
- Transversal reinforcement: : B500B f_{yk} = 500,00 (MPa)

2.2 Geometry:

2.2.1

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P1	**Span**	**0,70**	**4,06**	**0,70**

Span length: L_o = 4,76 (m)
Section from 0,00 to 4,06 (m)
70,0 x 100,0 (cm)
without left slab
without right slab

2.2.2

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P2	**Span**	**0,70**	**3,51**	**0,70**

Span length: L_o = 4,21 (m)
Section from 0,00 to 3,51 (m)
70,0 x 100,0 (cm)
without left slab
without right slab

2.2.3

Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P3	**Span**	**0,70**	**3,73**	**0,40**

Span length: L_o = 4,28 (m)
Section from 0,00 to 3,73 (m)

Date : 08/11/15 Page : **1**

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:
File: **POLO VIEWS DESIGN.rtd**
Project: POLO VIEWS DESIGN

70,0 x 100,0 (cm)
without left slab
without right slab

2.3 Calculation options:

- Regulation of combinations : CBS_PRO_ACI318_2002
- Calculations according to : EN 1992-1-1:2004
- Seismic dispositions : No requirements
- Precast beam : no
- Cover : bottom c = 2,9 (cm)
  : side c1= 2,9 (cm)
  : top c2= 2,9 (cm)
- Cover deviations : Cdev = 1,0(cm), Cdur = 0,0(cm)
- Coefficient β_2 =0.50 : long-term or cyclic load
- Method of shear calculations : strut inclination

2.4 Loads:

γ_f - load factor

2.5 Calculation results:

2.5.1 Reactions

INFORMATION UNAVAILABLE!

2.5.2 Internal forces in ULS

Span	Mt max. (kN*m)	Mt min. (kN*m)	Ml (kN*m)	Mr (kN*m)	Ql (kN)	Qr (kN)
P1	3250,58	-0,00	1721,02	355,74	2280,17	-986,85
P2	102,28	-83,10	102,28	-147,14	99,08	-132,80
P3	323,70	-0,00	323,70	62,68	-49,23	-105,08

2.5.3 Internal forces in SLS

Span	Mt max. (kN*m)	Mt min. (kN*m)	Ml (kN*m)	Mr (kN*m)	Ql (kN)	Qr (kN)
P1	2367,46	0,00	581,57	-89,88	1660,74	-717,71
P2	68,93	-23,23	68,93	-104,08	70,36	-95,22
P3	235,75	0,00	235,75	-35,36	-34,83	-76,77

2.5.4 Required reinforcement area

Span	Span (cm2)		Left support (cm2)		Right support (cm2)	
	bottom	top	bottom	top	bottom	top
P1	86,63	56,80	46,31	12,21	8,82	3,29
P2	2,41	0,00	2,41	2,15	0,84	3,58
P3	8,04	0,00	8,04	0,00	1,47	1,14

2.5.5 Fire resistance

Fire resistance :R 90(EN 1992-1-2:2004)
Calculations according to :EN 1992-1-2:2004
Estimation in accordance with section 5. Tabulated data.
Number of sides exposed to fire :3

Date : 08/11/15 Page : **2**

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:
File: **POLO VIEWS DESIGN.rtd**
Project: POLO VIEWS DESIGN

Web type :WA
Beam type :continuous
b_min = 0,15(m)
a_min = 0,03(m)
Required top reinforcement area in section 0.3*leff has been increased in accordance with formula (5.11)

2.5.6 Deflection and cracking

wt(QP) Total due to quasi-permanent combination
wt(QP)dop Allowable due to quasi-permanent combination
Dwt(QP) Deflection increment from the quasi-permanent load combination after erecting a structure.
Dwt(QP)dop Admissible deflection increment from the quasi-permanent load combination after erecting a structure.

wk - width of perpendicular cracks

Span	wt(QP) (cm)	wt(QP)dop (cm)	Dwt(QP) (cm)	Dwt(QP)dop (cm)	wk (mm)
P1	0,0	1,9	0,0	1,0	0,0
P2	0,0	1,7	0,0	0,8	0,0
P3	0,0	1,7	0,0	0,9	0,0

2.5.7 Compressive stress in strut

		h (m)	h gen (m)	σbc A (MPa)	Atheor (cm2)	Ar (cm2)
Span P1	Left support ΘA = 47,8 (Deg) a = 0,47 (m) Vu = 2280,17(kN) Lower diagonal	0,48	-	9,21	34,12	103,08
Span P1	Right support ΘA = 48,6 (Deg) a = 0,70 (m) Vu = 986,85(kN) Lower diagonal	0,52	-	3,58	18,31	137,44
Span P2	Left support ΘA = 48,5 (Deg) a = 0,70 (m) Vu = 99,08(kN) Lower diagonal	0,52	-	0,36	1,16	103,08
Span P2	Right support ΘA = 48,6 (Deg) a = 0,70 (m) Vu = 132,80(kN) Lower diagonal	0,52	-	0,48	0,85	83,45
Span P3	Left support ΘA = 48,7 (Deg) a = 0,70 (m) Vu = 0,00(kN) Lower diagonal	0,53	-	0,18	0,99	112,90
Span P3	Right support ΘA = 54,2 (Deg) a = 0,25 (m) Vu = 105,08(kN) Lower diagonal	0,28	-	0,67	0,64	34,36

Date : 08/11/15 Page : **3**

2.6 Reinforcement:

2.6.1 P1 : Span from 0,70 to 4,76 (m)

Longitudinal reinforcement:

- bottom (B500B)

7	ϕ25	l = 4,92	from 0,04	to	4,95
1	ϕ25	l = 12,17	from 0,07	to	10,49
1	ϕ25	l = 12,57	from 0,07	to	10,89
1	ϕ25	l = 12,97	from 0,07	to	11,29
1	ϕ25	l = 13,37	from 0,07	to	11,69
1	ϕ25	l = 13,77	from 0,07	to	12,09
1	ϕ25	l = 14,13	from 0,07	to	12,49
1	ϕ25	l = 14,13	from 0,07	to	12,89
1	ϕ25	l = 9,14	from 0,27	to	7,72
1	ϕ25	l = 9,54	from 0,27	to	8,12
1	ϕ25	l = 9,94	from 0,27	to	8,52
1	ϕ25	l = 10,34	from 0,27	to	8,92
1	ϕ25	l = 10,74	from 0,27	to	9,32
1	ϕ25	l = 11,14	from 0,27	to	9,72
1	ϕ25	l = 11,54	from 0,27	to	10,12

- support (B500B)

7	ϕ25	l = 4,13	from 0,04	to	4,17
7	ϕ25	l = 7,36	from 1,29	to	8,65
5	ϕ25	l = 4,77	from 0,19	to	3,31

Surface reinforcement (B500B):

4 ϕ16 l = 4,68 from 0,39 to 5,07
pins 28 ⌀6 l = 0,76
e = 1*0,08 + 13*0,30 (m)

Transversal reinforcement:

- main (B500B)

stirrups
42 ϕ8 l = 2,82
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
42 ϕ8 l = 2,42
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
42 ϕ8 l = 3,85
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
42 ϕ8 l = 2,02
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
4 ϕ16 l = 4,68
e = 1*-0,31 (m)

pins
42 ϕ8 l = 2,82
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
42 ϕ8 l = 2,42
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
42 ϕ8 l = 3,85
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
42 ϕ8 l = 2,02
e = 1*0,02 + 23*0,06 + 18*0,14 (m)
4 ϕ16 l = 4,68
e = 1*-0,31 (m)

2.6.2 P2 : Span from 5,46 to 8,97 (m)

Longitudinal reinforcement:

- bottom (B500B)

7 ϕ25 l = 8,48 from 2,94 to 11,42

Surface reinforcement (B500B):

4 ϕ16 l = 4,13 from 5,15 to 9,28
pins 24 ⌀6 l = 0,76
e = 1*0,11 + 11*0,30 (m)

Transversal reinforcement:

- main (B500B)

stirrups
13 ϕ8 l = 2,82
e = 1*0,08 + 12*0,28 (m)
13 ϕ8 l = 2,42
e = 1*0,08 + 12*0,28 (m)
13 ϕ8 l = 3,85
e = 1*0,08 + 12*0,28 (m)
13 ϕ8 l = 2,02
e = 1*0,08 + 12*0,28 (m)
4 ϕ16 l = 4,13
e = 1*-0,31 (m)

pins
13 ϕ8 l = 2,82
e = 1*0,08 + 12*0,28 (m)
13 ϕ8 l = 2,42
e = 1*0,08 + 12*0,28 (m)
13 ϕ8 l = 3,85
e = 1*0,08 + 12*0,28 (m)
13 ϕ8 l = 2,02
e = 1*0,08 + 12*0,28 (m)
4 ϕ16 l = 4,13
e = 1*-0,31 (m)

2.6.3 P3 : Span from 9,67 to 13,40 (m)

Longitudinal reinforcement:

- bottom (B500B)

7 ϕ25 l = 4,29 from 9,41 to 13,70

- support (B500B)

7 ϕ25 l = 7,20 from 5,78 to 12,97
7 ϕ25 l = 3,79 from 10,10 to 13,76

Surface reinforcement (B500B):

4 ϕ16 l = 4,20 from 9,36 to 13,56
pins 24 ø6 l = 0,76
e = 1*0,14 + 11*0,30 (m)

Transversal reinforcement:

- main (B500B)

stirrups
14 ϕ8 l = 2,82
e = 1*0,05 + 13*0,28 (m)
14 ϕ8 l = 2,42
e = 1*0,05 + 13*0,28 (m)
14 ϕ8 l = 3,85
e = 1*0,05 + 13*0,28 (m)
14 ϕ8 l = 2,02
e = 1*0,05 + 13*0,28 (m)
4 ϕ16 l = 4,20
e = 1*-0,31 (m)

pins
14 ϕ8 l = 2,82
e = 1*0,05 + 13*0,28 (m)
14 ϕ8 l = 2,42
e = 1*0,05 + 13*0,28 (m)
14 ϕ8 l = 3,85
e = 1*0,05 + 13*0,28 (m)
14 ϕ8 l = 2,02
e = 1*0,05 + 13*0,28 (m)
4 ϕ16 l = 4,20
e = 1*-0,31 (m)

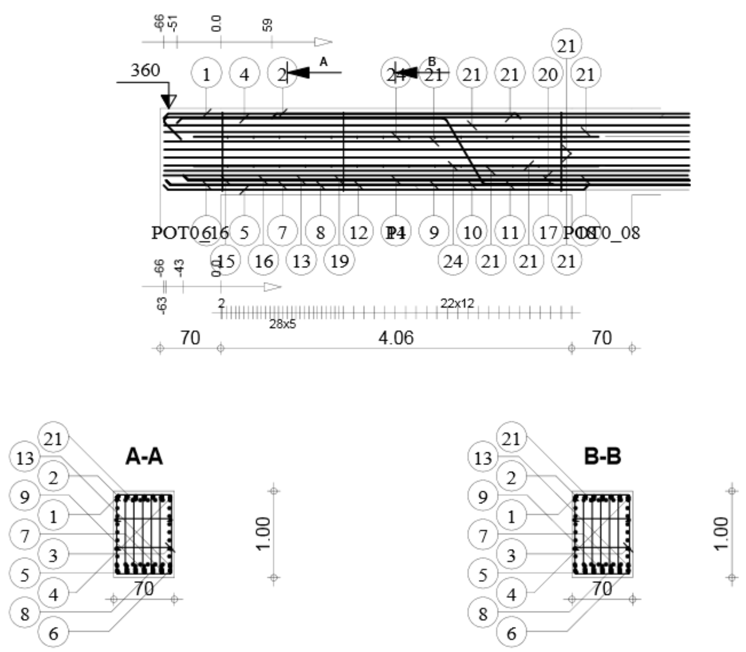
3 Material survey:

- Concrete volume = 9,66 (m3)
- Formwork = 36,91 (m2)

- Steel B500B
  - Total weight = 2518,63 (kG)
  - Density = 260,73 (kG/m3)
  - Average diameter = 14,4 (mm)
  - Survey according to diameters:

Diameter	Length (m)	Weight (kG)
6	57,47	12,76
8	766,82	302,68
16	246,81	389,68
25	470,47	1813,50

d) Drawings of beam POU0_08 details



Pos.	Reinforcement		Code	Shape	Steel
1	7Ø25	l=413	00	413	B500B
2	7Ø25	l=736	00	736	B500B
3	2*14Ø6	l=76	00	5 64	B500B
4	5Ø25	l=477		311 69 81	B500B
5	7Ø25	l=492	00	492	B500B
6	1Ø25	l=1217		1042 81	B500B
7	1Ø25	l=1257		1082 81	B500B
8	1Ø25	l=1297		1122 81	B500B
9	1Ø25	l=1337		1162 81	B500B
10	1Ø25	l=1377		1202 81	B500B
11	1Ø25	l=1413		1242 78	B500B
12	1Ø25	l=1413		1282 38	B500B
13	1Ø25	l=914		745 81	B500B
14	1Ø25	l=954		785 81	B500B
15	1Ø25	l=994		825 81	B500B
16	1Ø25	l=1034		865 81	B500B
17	1Ø25	l=1074		905 81	B500B

Tel. Fax

Exposure class : XC1 Fire resistance : R90 Max. aggregate diameter : 20mm Structure class : S4

Concrete : C25/30 = 3.58 m3

Steel B500B = 1480 kg

Steel B500B = 217 kg

Formwork = 13.8 m2

Bottom cover 3.9 cm Top cover 3.9 cm

Side cover 3.9 cm

Density (unit weight) = 474.9 kg/ m3

Average diameter = 14.8mm

View scale 1/50

Section scale 1/50

0

POLO VIEWS DESIGN

1_POU0_08 : P1

Number 1

Section 70x100

Page 1/4

Pos.	Reinforcement		Code	Shape	Steel
18	1Ø25	l=1114		945 81	B500B
19	1Ø25	l=1154		985 81	B500B
20	8Ø16	l=607	00	607	B500B
21	22Ø16	l=665	00	634	B500B
22	250Ø8	l=108	00	6 92	B500B
23	50Ø8	l=385	41	64 64	B500B
24	2*2Ø16	l=468	00	468	B500B

Tel. Fax

Exposure class : XC1 Max. aggregate diameter : 20mm Structure class : S4

Concrete : C25/30 = 3.58 m3

Steel B500B = 1480 kg

Steel B500B = 217 kg

Formwork = 13.8 m2

Bottom cover 3.9 cm Top cover 3.9 cm

Side cover 3.9 cm

Density (unit weight) = 474.9 kg/ m3

Average diameter = 14.8mm

0

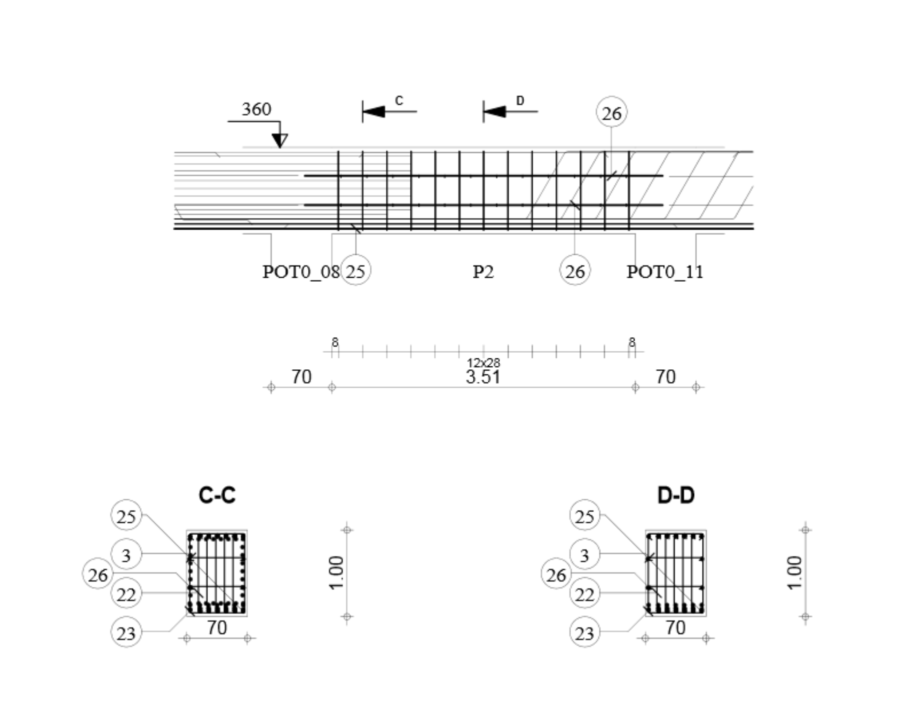
POLO VIEWS DESIGN

1_POU0_08

Number 1

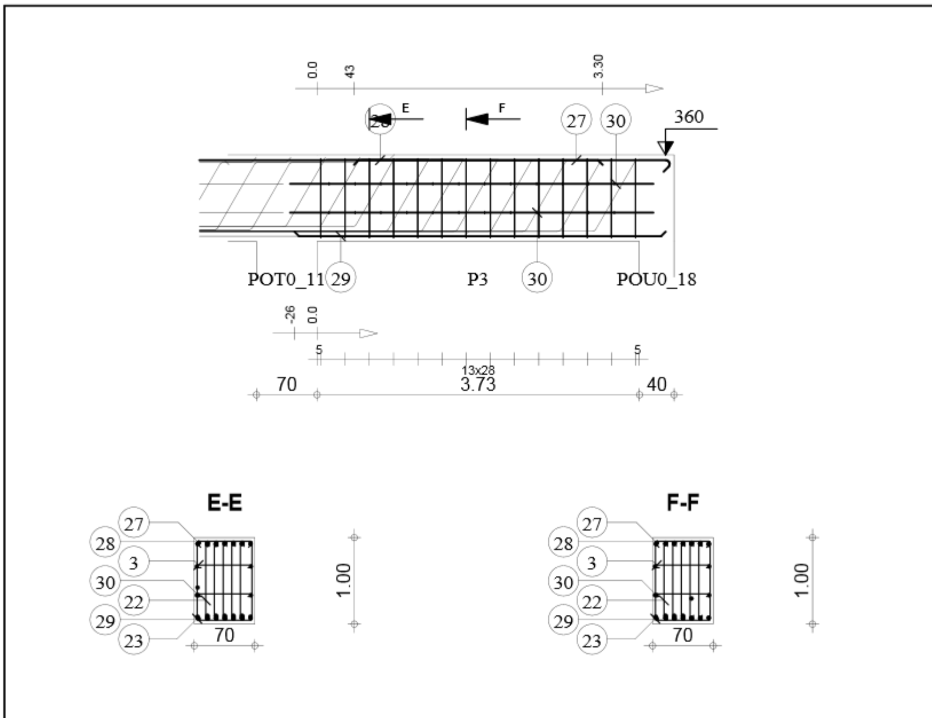
Section 70x100

Page 2/4



Pos.	Reinforcement	Code	Shape	Steel
3	2*12Ø6 l=76	00	5 64	B500B
22	65Ø8 l=108	00	6 92	B500B
23	13Ø8 l=385	41	64	B500B
25	7Ø25 l=848	00	848	B500B
26	2*2Ø16 l=413	00	413	B500B

	Tel.	Fax		Steel B500B = 229 kg
Exposure class : XC1	Fire resistance : R90	Max. aggregate diameter : 20mm	Structure class : S4	Concrete : C25/30 = 2.95 m3	Steel B500B = 77.6 kg
0 POLO VIEWS DESIGN	1_POU0_08 : P2 Section 70x100	Number 1	Formwork = 10.9 m2	Bottom cover 3.9 cm	Top cover 3.9 cm
				Side cover 3.9 cm	
			Density (unit weight) = 103.7 kg/m3	View scale 1/50	Page 3/4
			Average diameter = 13.2mm	Section scale 1/50	



Pos.	Reinforcement	Code	Shape	Steel
3	2*12Ø6 l=76	00	5 64	B500B
22	70Ø8 l=108	00	6 92	B500B
23	14Ø8 l=385	41	64	B500B
27	7Ø25 l=720	00	720	B500B
28	7Ø25 l=379	00	366	B500B
29	7Ø25 l=429	00	429	B500B
30	2*2Ø16 l=420	00	420	B500B

	Tel.	Fax		Steel B500B = 412 kg	
Exposure class : XC1	Fire resistance : R90	Max. aggregate diameter : 20mm	Structure class : S4	Concrete : C25/30 = 3.14 m3	Steel B500B = 81.6 kg
0 POLO VIEWS DESIGN	1_POU0_08 : P3 Section 70x100	Number 1	Formwork = 12.3 m2	Bottom cover 3.9 cm	Top cover 3.9 cm
				Side cover 3.9 cm	
			Density (unit weight) = 157.3 kg/m3	View scale 1/50	Page 4/4
			Average diameter = 15.1mm	Section scale 1/50	

ANNEX 6: Columns calculation details and reinforcement detailing

This part is dealing with the columns calculations. The notes below have been taken from the software to show the consideration and the results issued by this one. Our focus is on four (4) different columns which are: POT0_14, POT0_02, POT0_04, POT0_16. The choice of these columns is based on their size and the internals forces that they are going to resist. Indeed in the table 4.3 we can notice that the column POT0_16 is subjected to a huge moment. Thus the detailing will certainly be different from the others. Here are the

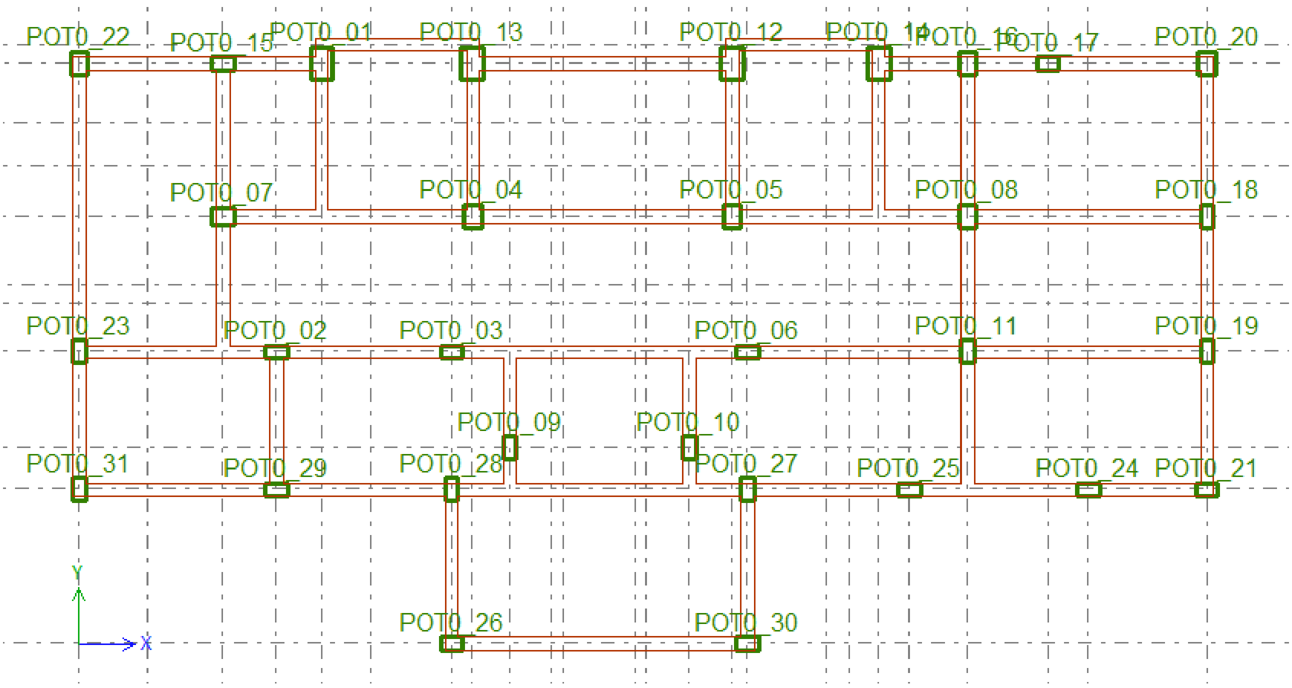


Figure 15: Columns position

1 Level:

- Name : Chain_68
- Reference level : 0,00 (m)
- Concrete creep coefficient : φ_p = 2,22
- cement class : N
- Environment class : XC1
- Structure class : S4

2 Column: 1_POT0_14 Number: 1

2.1 Material properties:

- Concrete : C30/37 f_{ck} = 30,00 (MPa)
  Unit weight : 2501,36 (kG/m3)
  Aggregate size : 20,0 (mm)
- Longitudinal reinforcement: : B500B f_{yk} = 500,00 (MPa)
  Ductility class : B
- Transversal reinforcement: : B500B f_{yk} = 500,00 (MPa)

2.2 Geometry:

2.2.1 Rectangular 70,0 x 100,0 (cm)
2.2.2 Height: L = 3,69 (m)
2.2.3 Slab thickness = 0,18 (m)
2.2.4 Beam height = 0,80 (m)
2.2.5 Cover = 3,5 (cm)

2.3 Calculation options:

- Calculations according to : EN 1992-1-1:2004
- Seismic dispositions : No requirements
- Precast column : no
- Pre-design : no
- Slenderness taken into account : yes
- Compression : with bending
- Ties : to slab
- More than 50 % loads applied: after 90 day
- Fire resistance class : R 90

2.4 Loads:

Case	Nature	Group	γ_f	N (kN)	My(s) (kN*m)	My(i) (kN*m)	Mz(s) (kN*m)	Mz(i) (kN*m)
DL1	dead load	1	1,35	1522,17	99,42	-500,57	1,81	-22,93
DL2	dead load	1	1,35	716,06	63,63	-248,92	3,81	-10,63
LL3	live load	1	1,50	205,96	29,78	-36,36	5,65	-8,43
LL4	live load	1	1,50	84,05	-5,21	-33,28	0,19	-0,01
LL5	live load	1	1,50	47,69	4,50	-45,83	-5,36	3,21
LL6	live load	1	1,50	21,44	0,45	-17,38	-2,24	2,15
LL7	live load	1	1,50	144,13	-3,42	-77,40	-5,73	0,07
WIND8	wind	1	1,50	24,27	15,80	-11,37	-12,42	99,70
WIND9	wind	1	1,50	-24,22	-15,77	11,22	12,98	-101,15
WIND10	wind	1	1,50	114,08	73,24	-443,75	-7,68	8,05
WIND11	wind	1	1,50	-115,89	-76,43	454,11	7,81	-8,26

γ_f - load factor

2.5 Calculation results:

Safety factors Rd/Ed = 1,11 > 1.0

2.5.1 Fire resistance

- Calculations according to: : EN 1992-1-2:2004
- Estimation in accordance with section 5. Tabulated data.
- Number of sides exposed to fire: : >1
- Reduction factor in fire situation : 0.7
- Calculation method : A
- Effective column length in fire conditions : $l_{0y,fi}$ = 3,69 (m)
- Effective column length in fire conditions : $l_{0z,fi}$ = 3,69 (m)
- Load level reduction factor : μfi = 0,70
- Combination for load level reduction factor : 0.7 * [1.00DL1+1.00DL2+1.00LL3+ 1.50WIND10 (A)]
- Ratio : ϖ = 0.137
- Number of main bars : 14
- Ra = 1,6 * (a * 1000 - 30) = 36,34
- Rl = 9,6 * (5 - $l_{0,fi}$) = 12,60
- Rb = 90 * b' = 40,50
- Rn = 12,00
- Rηfi = 83 * (1 - μ * (1 + ϖ)) / (0.85 / acc + ϖ)) = 16,07
- R = 115, R 115 >= R 90

2.5.2 ULS Analysis

Design combination: 1.35DL1+1.35DL2+1.00LL3+1.00LL4+1.00LL5+1.00LL6+1.00LL7+ 1.50WIND10 (B)
Internal forces:
Nsd = 3695,99 (kN) Msdy = -1887,69 (kN*m) Msdz = -36,23 (kN*m)
Design forces:
Lower node
N = 3695,99 (kN) N*etotz = -1921,76 (kN*m)N*etoty= -123,20 (kN*m)

Eccentricity:		ez (My/N)	ey (Mz/N)
Static	eEd:	-51,1 (cm)	-1,0 (cm)
Imperfection	ei:	0,9 (cm)	0,0 (cm)
Initial	e0:	-50,2 (cm)	-1,0 (cm)
Minimal	emin:	3,3 (cm)	3,3 (cm)
Total	etot:	-52,0 (cm)	-3,3 (cm)

2.5.2.1. Detailed analysis-Direction Y:

2.5.2.1.1 Slenderness analysis

Non-sway structure

L (m)	Lo (m)	λ	λlim	
3,69	3,69	12,77	61,45	Short column

2.5.2.1.2 Buckling analysis

M2 = 356,08 (kN*m) M1 = -1887,69 (kN*m)
Case: Cross-section at the column end (Lower node), Slenderness not taken into account
M0 = -1887,69 (kN*m)
ea = θ1*lo/2 = 0,9 (cm)
θ1 = θo * αη * αm = 0,01
θo = 0,01
αh = 1,00
αm = (0,5(1+1/m))^0.5 = 1,00
m = 1,00
Ma = N*ea = 34,07 (kN*m)
MEdmin = 123,20 (kN*m)
M0Ed = max(MEdmin,M0 + Ma) = -1921,76 (kN*m)

2.5.2.2. Detailed analysis-Direction Z:

2.5.2.2.1 Slenderness analysis

Non-sway structure

L (m)	Lo (m)	λ	λlim	
3,69	3,69	18,25	45,05	Short column

2.5.2.2.2 Buckling analysis

M2 = -11,42 (kN*m) M1 = -36,23 (kN*m)
Case: Cross-section at the column end (Lower node), Slenderness not taken into account
M0 = -36,23 (kN*m)
ea = 0,0 (cm)
Ma = N*ea = 0,00 (kN*m)
MEdmin = 123,20 (kN*m)
M0Ed = max(MEdmin,M0 + Ma) = -123,20 (kN*m)

2.5.3 Reinforcement:

Real (provided) area Asr = 43,98 (cm2)
Ratio: ρ = 0,63 %

2.6 Reinforcement:

Main bars (B500B):
- 14 ϕ20 l = 4,40 (m)

Transversal reinforcement: (B500B):

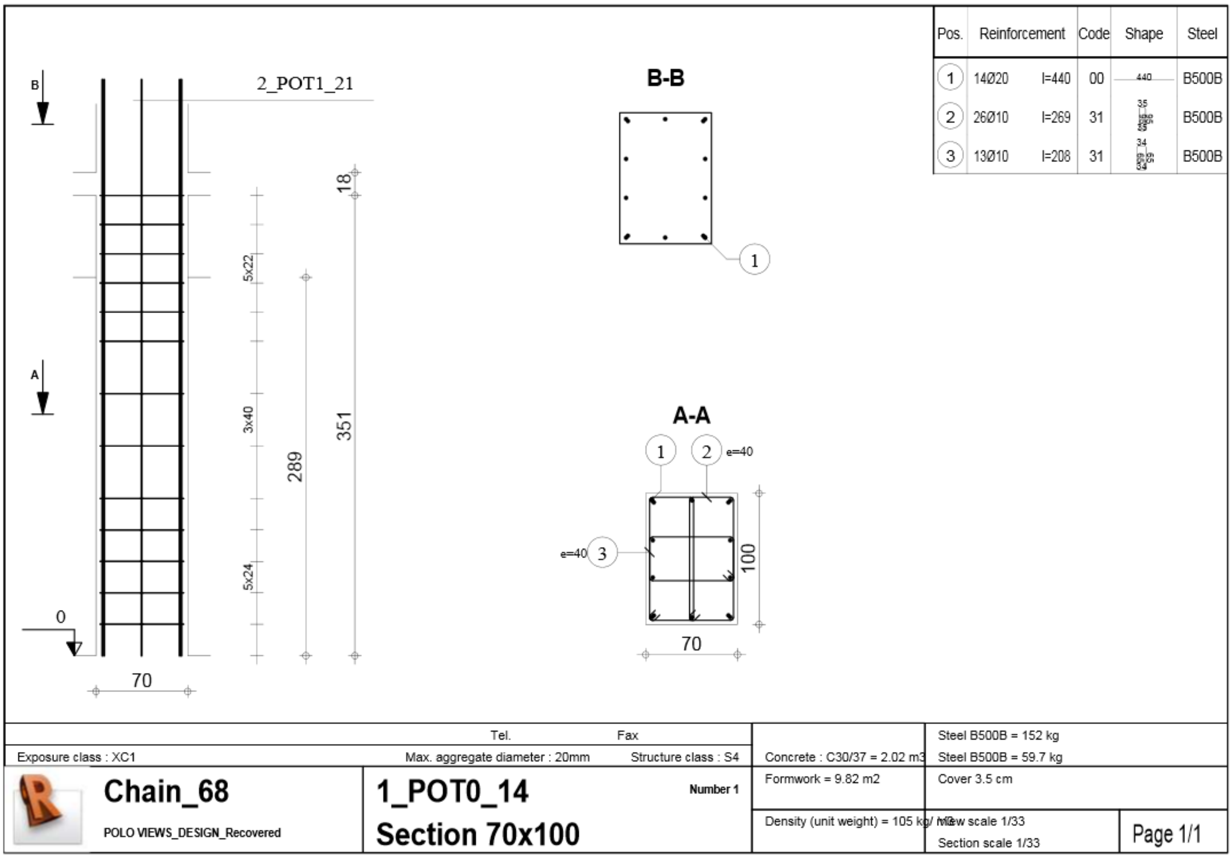
stirrups:	26 ϕ10	l = 2,69 (m)
	13 ϕ10	l = 2,08 (m)
pins	26 ϕ10	l = 2,69 (m)
	13 ϕ10	l = 2,08 (m)

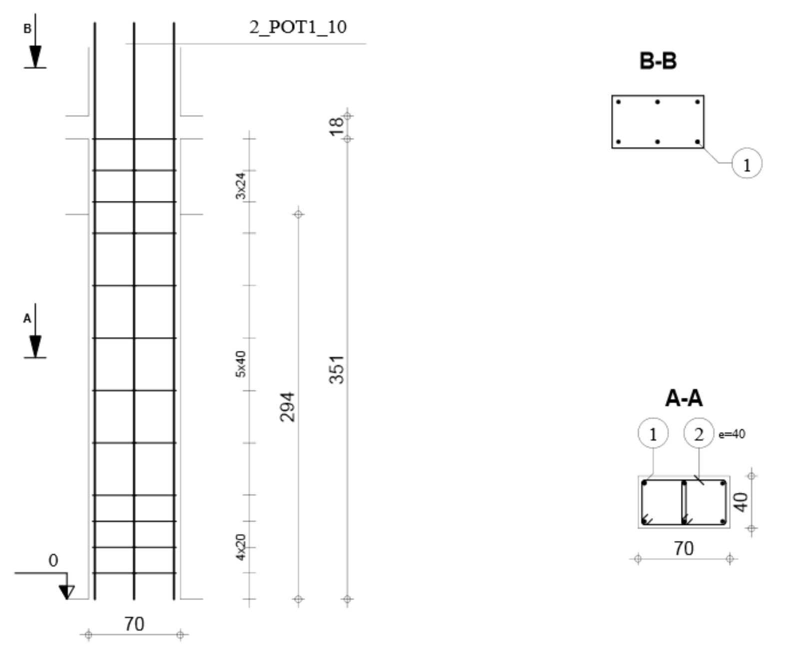
3 Material survey:

- Concrete volume = 2,02 (m3)
- Formwork = 9,82 (m2)
- Steel B500B
  - Total weight = 211,64 (kG)
  - Density = 104,71 (kG/m3)

- Average diameter = 13,9 (mm)
- Reinforcement survey:

Diameter	Length (m)	Weight (kG)
10	96,90	59,76
20	61,57	151,88

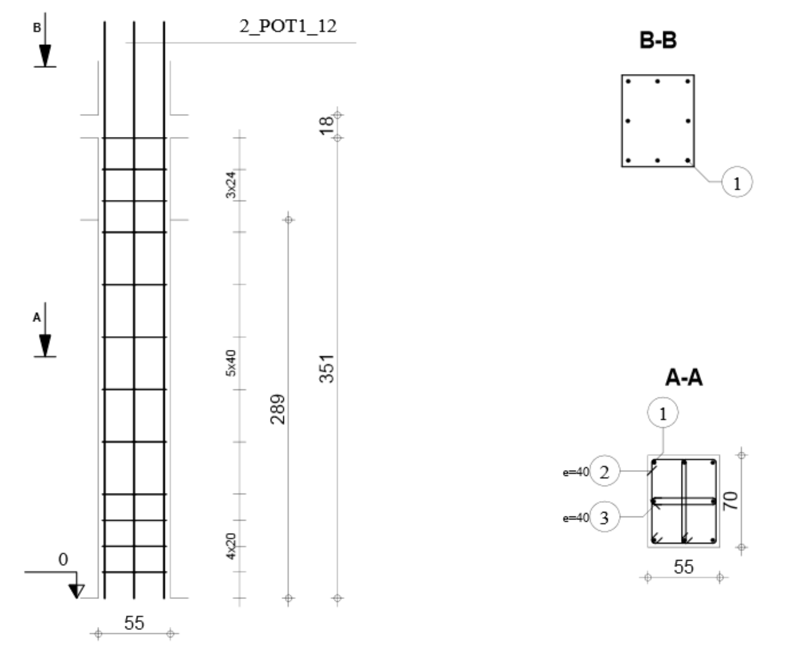

2_POT1_21
B-B
A-A
Pos. Reinforcement Code Shape Steel
1 14Ø20 l=440 00 B500B
2 26Ø10 l=269 31 B500B
3 13Ø10 l=208 31 B500B
5x22
3x40
5x24
289
351
18
70
100
e=40
Tel.
Fax
Exposure class : XC1
Max. aggregate diameter : 20mm
Structure class : S4
Concrete : C30/37 = 2.02 m3
Steel B500B = 152 kg
Steel B500B = 59.7 kg
Formwork = 9.82 m2
Cover 3.5 cm
Chain_68
POLO VIEWS_DESIGN_Recovered
1_POT0_14
Number 1
Section 70x100
Density (unit weight) = 105 kg/ m3
View scale 1/33
Section scale 1/33
Page 1/1



Pos.	Reinforcement		Code	Shape	Steel
1	6Ø20	l=440	00	440	B500B
2	24Ø10	l=149	31	35 35	B500B

Exposure class : XC1	Tel. Max. aggregate diameter : 20mm	Fax Structure class : S4	Concrete : C30/37 = 0.822 m3
Chain_55 POLO VIEWS_DESIGN_Recovered	1_POT0_02 Section 70x40	Number 1	Formwork = 6.46 m2
			Density (unit weight) = 106 kg/m3

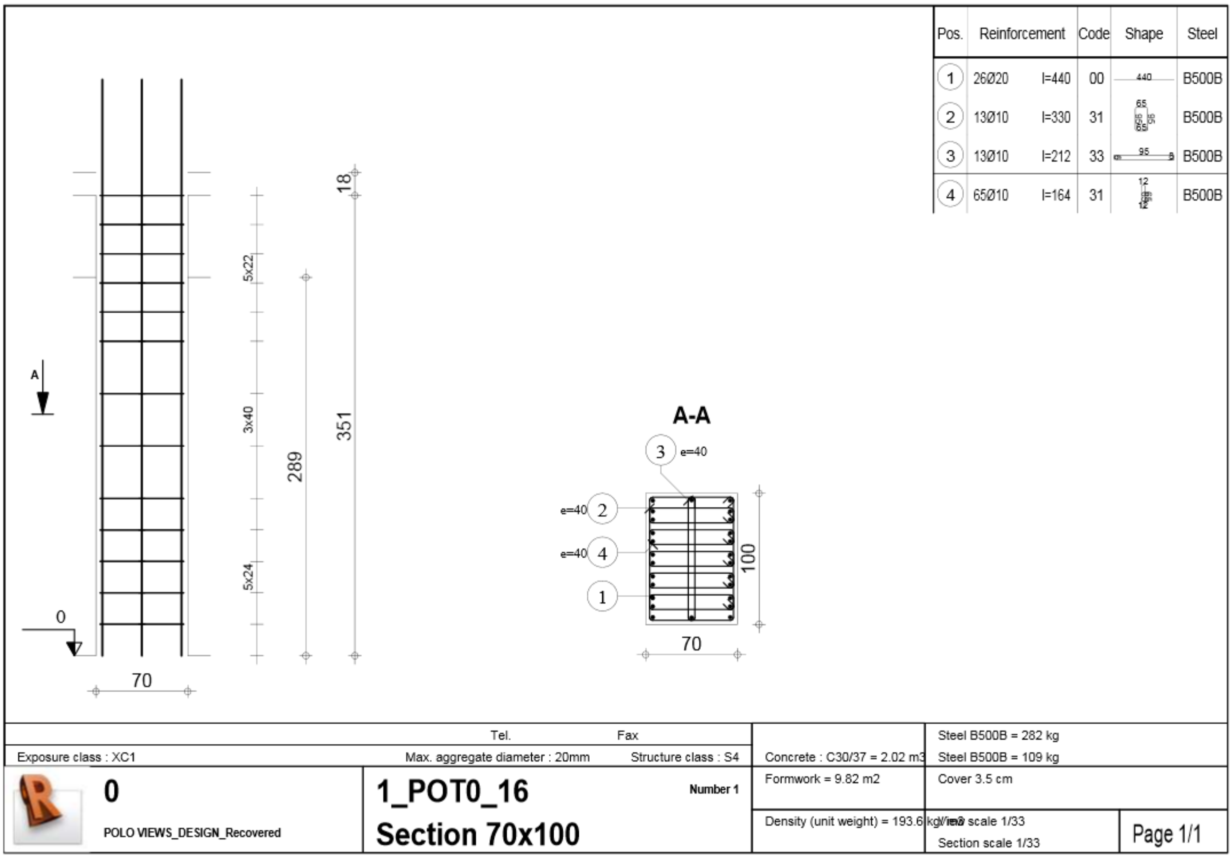
Steel B500B = 65.1 kg
Steel B500B = 22 kg
Cover 3.5 cm
View scale 1/33
Section scale 1/33
Page 1/1



Pos.	Reinforcement		Code	Shape	Steel
1	8Ø20	l=440	00	440	B500B
2	24Ø10	l=194	31	27	B500B
3	12Ø10	l=122	33	50	B500B

Exposure class : XC1	Tel. Max. aggregate diameter : 20mm	Fax Structure class : S4	Concrete : C30/37 = 1.11 m3
Chain_58 POLO VIEWS_DESIGN_Recovered	1_POT0_04 Section 55x70	Number 1	Formwork = 7.22 m2
			Density (unit weight) = 111.7 kg/m3

Steel B500B = 86.8 kg
Steel B500B = 37.6 kg
Cover 3.5 cm
View scale 1/33
Section scale 1/33
Page 1/1


Pos. Reinforcement Code Shape Steel
1 26Ø20 l=440 00 440 B500B
2 13Ø10 l=330 31 65 95 B500B
3 13Ø10 l=212 33 95 B500B
4 65Ø10 l=164 31 12 B500B
A
0
70
5x22
3x40
5x24
18
289
351
A-A
3 e=40
e=40 2
e=40 4
1
100
70
Tel.
Fax
Exposure class : XC1
Max. aggregate diameter : 20mm
Structure class : S4
Concrete : C30/37 = 2.02 m3
Steel B500B = 282 kg
Steel B500B = 109 kg
Formwork = 9.82 m2
Cover 3.5 cm
Density (unit weight) = 193.6 kg/m3
View scale 1/33
Section scale 1/33
0
POLO VIEWS_DESIGN_Recovered
1_POT0_16
Number 1
Section 70x100
Page 1/1

ANNEX 7: Pad footings

The following footing have been chosen according to columns above. Thus the pad footing SI0_07 is supporting the column POT0_14 and same for the others. As for the columns, we are going to show the output report of the software and the detailing have also been done with it.

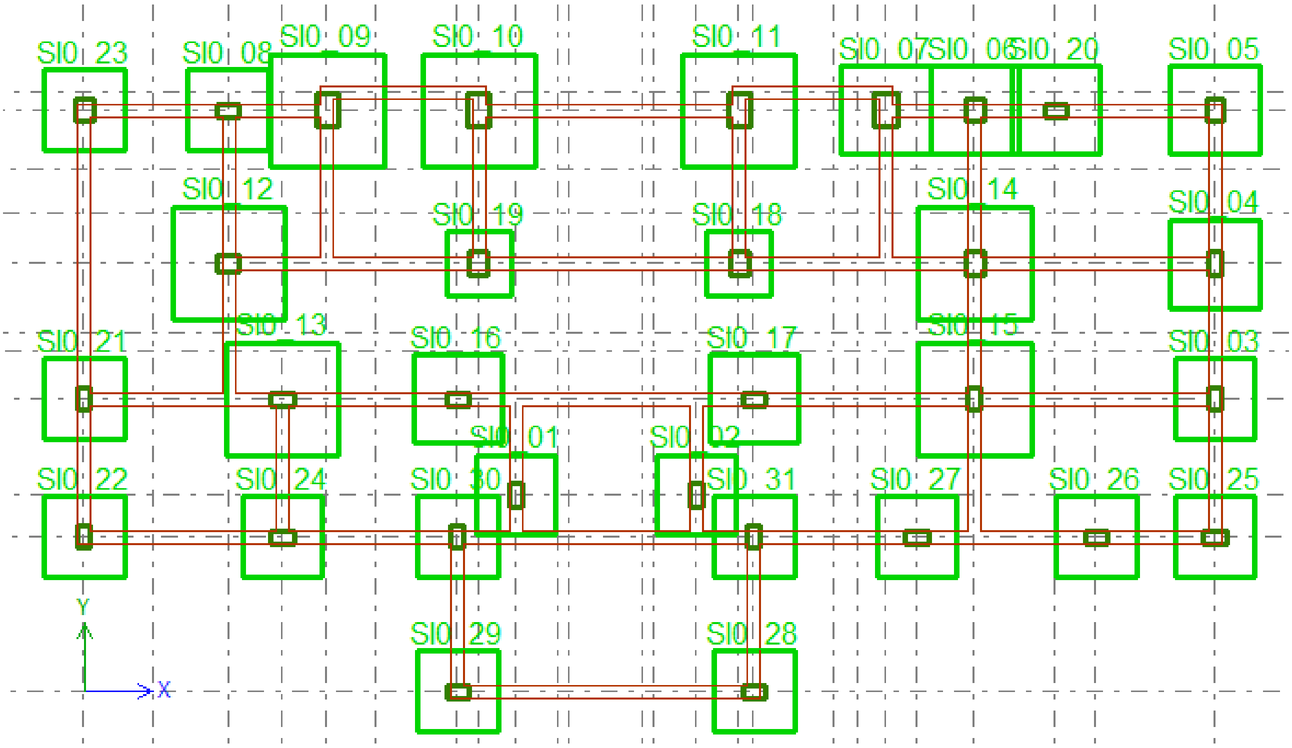


Figure 16: Footings arrangement

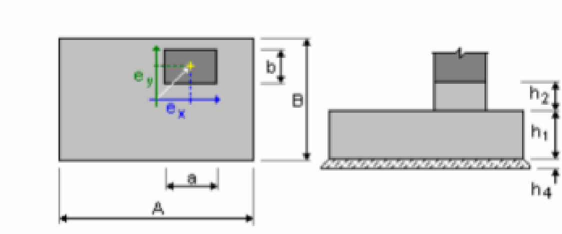
1 Spread footing: sel_1_Sl0_07 Number: 1

1.1 Basic data

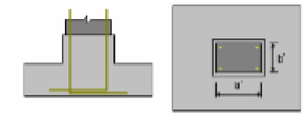
1.1.1 Assumptions

- Geotechnic calculations according to : EN 1997-1:2008
- Concrete calculations according to : EN 1992-1-1:2004
- Shape selection : without limits

1.1.2 Geometry:



A	= 1,50 (m)	a	= 1,00 (m)
B	= 1,90 (m)	b	= 0,70 (m)
h1	= 1,50 (m)	e_x	= 0,00 (m)
h2	= 0,00 (m)	e_y	= 0,00 (m)
h4	= 0,05 (m)		



a' = 100,0 (cm)
b' = 70,0 (cm)
cnom1 = 5,0 (cm)
cnom2 = 5,0 (cm)
Cover deviations: Cdev = 1,0(cm), Cdur = 0,0(cm)

1.1.3 Materials

- Concrete : C30/37; Characteristic strength = 30,00 MPa
  Unit weight = 2501,36 (kG/m3)
  Rectangular stress distribution [3.1.7(3)]
- Longitudinal reinforcement : type B500B Characteristic strength = 500,00 MPa
  Ductility class: B
  Horizontal branch of the stress-strain diagram
- Transversal reinforcement : type B500B Characteristic strength = 500,00 MPa

1.1.4 Loads:

Foundation loads:

Case	Nature	Group	N (kN)	Fx (kN)	Fy (kN)	Mx (kN*m)	My (kN*m)

DL1	dead load(self-weight)	1	1583,98	-6,87	166,66	-500,57	-22,93
DL2	dead load	1	716,06	-4,01	86,82	-248,92	-10,63
LL1	live load	1	205,96	-3,91	18,37	-36,36	-8,43
LL2	live load(Balcons)	1	84,05	-0,06	7,80	-33,28	-0,01
LL3	live load(stairs)	1	47,69	2,38	13,98	-45,83	3,21
LL4	live load(terrace)	1	21,44	1,22	4,95	-17,38	2,15
LL5	live load(water storage)	1	144,13	1,61	20,55	-77,40	0,07
WIND1	wind(Vent 1+)	1	24,27	31,14	7,55	-11,37	99,70
WIND2	wind(Vent 1-)	1	-24,22	-31,70	-7,50	11,22	-101,15
WIND3	wind(Vent 2+)	1	114,08	4,37	143,61	-443,75	8,05
WIND4	wind(Vent 2-)	1	-115,89	-4,46	-147,37	454,11	-8,26

Backfill loads:

Case	Nature	Q1 (kN/m2)

1.2 Geotechnical design

1.2.1 Assumptions

- Cohesion reduction coefficient: 0,00
- Design approach: 1

A1 + M1 + R1

$\gamma_{\phi'}$ = 1,00

$\gamma_{c'}$ = 1,00

γ_{cu} = 1,00

γ_{qu} = 1,00

γ_{γ} = 1,00

$\gamma_{R,v}$ = 1,00

$\gamma_{R,h}$ = 1,00

A2 + M2 + R1

$\gamma_{\phi'}$ = 1,25

$\gamma_{c'}$ = 1,25

γ_{cu} = 1,40

γ_{qu} = 1,40

γ_{γ} = 1,00

$\gamma_{R,v}$ = 1,00

$\gamma_{R,h}$ = 1,00

1.2.2 Soil:

Soil level: N_1 = 0,20 (m)
Column pier level: N_a = 0,00 (m)
Minimum reference level: N_f = 0,00 (m)

Medium Sand

- Soil level: 0.20 (m)
- Unit weight: 1886.47 (kG/m3)
- Unit weight of solid: 2702.25 (kG/m3)
- Internal friction angle: 30.0 (Deg)
- Cohesion: 0.00 (MPa)

1.2.3 Limit states

Average settlement

Soil type under foundation: not layered
Design combination **SLS : 1.00DL1+1.00DL2+0.77LL1+0.77LL2+0.77LL3+0.77LL4+0.77LL5+1.00WIND3**
Load factors: **1.00** * Foundation weight
1.00 * Soil weight
Weight of foundation and soil over it: Gr = 303,65 (kN)
Average stress caused by design load: q = 0,53 (MPa)
Thickness of the actively settling soil: z = 8,25 (m)
Stress on the level z:
- Additional: σzd = 0,03 (MPa)
- Caused by soil weight: σzγ = 0,18 (MPa)

Settlement:
- Original s' = 1,0 (cm)
- Secondary s'' = 0,0 (cm)
- TOTAL S = 1,0 (cm) < Sadm = 5,0 (cm)

Safety factor: 5.032 > 1

Settlement difference

Design combination **SLS : 1.00DL1+1.00DL2+0.77LL1+0.77LL2+1.00WIND2**
Load factors: **1.00** * Foundation weight
1.00 * Soil weight
Settlement difference: S = 0,3 (cm) < Sadm = 5,0 (cm)
Safety factor: 17.68 > 1

1.3 RC design

1.3.1 Assumptions

- Exposure : XC2
- Structure class : S4

1.3.2 Analysis of punching and shear

Punching

Design combination **ULS : 1.35DL1+1.35DL2+1.00LL1+1.00LL2+1.00LL3+1.00LL4+1.00LL5+1.50WIND3**
Load factors: **1.00** * Foundation weight
1.00 * Soil weight
Design load:
Nr = 4083,09 (kN) Mx = -2822,60 (kN*m) My = -46,57 (kN*m)
Length of critical circumference: 6,99 (m)
Punching force: 2175,34 (kN)
Section effective height heff = 1,43 (m)
Reinforcement ratio: ρ = 0.12 %
Shear stress: 1,39 (MPa)
Admissible shear stress: 1,54 (MPa)
Safety factor: 1.112 > 1

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:
File: **POLO VIEWS_DESIGN_Recovered.rtd**
Project: POLO VIEWS_DESIGN_Recovered

1.3.4 Provided reinforcement

Spread footing:
Bottom:
Along X axis:
25 B500B 16 l = 2,63 (m) e = 1*-1,19 + 24*0,10
Along Y axis:
25 B500B 16 l = 2,63 (m) e = 1*-1,19 + 24*0,10

Pier
Longitudinal reinforcement
Along X axis:
2 B500B 12 l = 3,88 (m) e = 1*-0,40 + 1*0,80
Along Y axis:
2 B500B 12 l = 4,53 (m) e = 1*-0,25 + 1*0,50
Transversal reinforcement
8 B500B 10 l = 3,02 (m) e = 1*0,21 + 5*0,20 + 2*0,09

Dowels
Longitudinal reinforcement
14 B500B 20 l = 2,24 (m) e = 1*-0,31 + 1*0,01 + 1*0,28 + 1*0,31 + 1*0,01

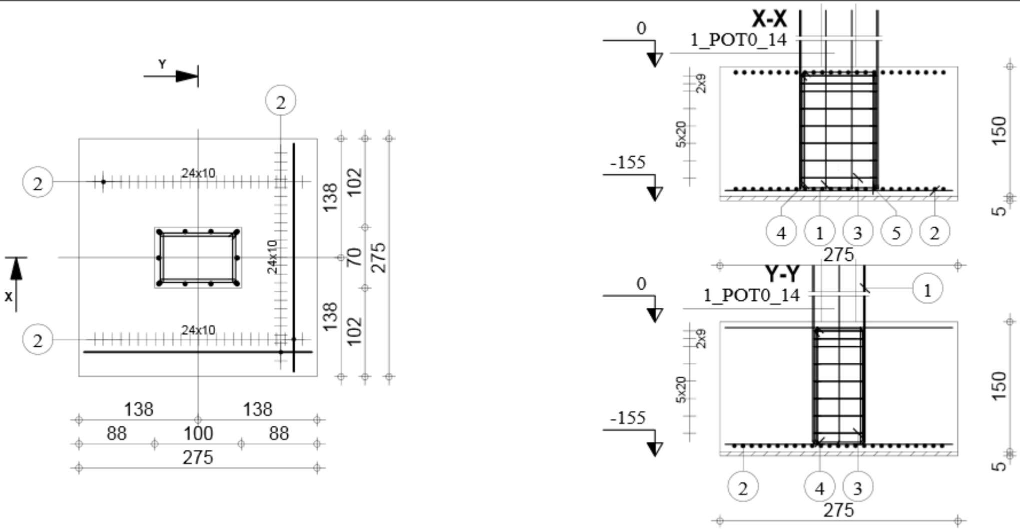
2 Material survey:

- Concrete volume = 11,34 (m3)
- Formwork = 16,50 (m2)

- Steel B500B
  - Total weight = 418,76 (kG)
  - Density = 36,92 (kG/m3)
  - Average diameter = 15,7 (mm)
  - Survey according to diameters:

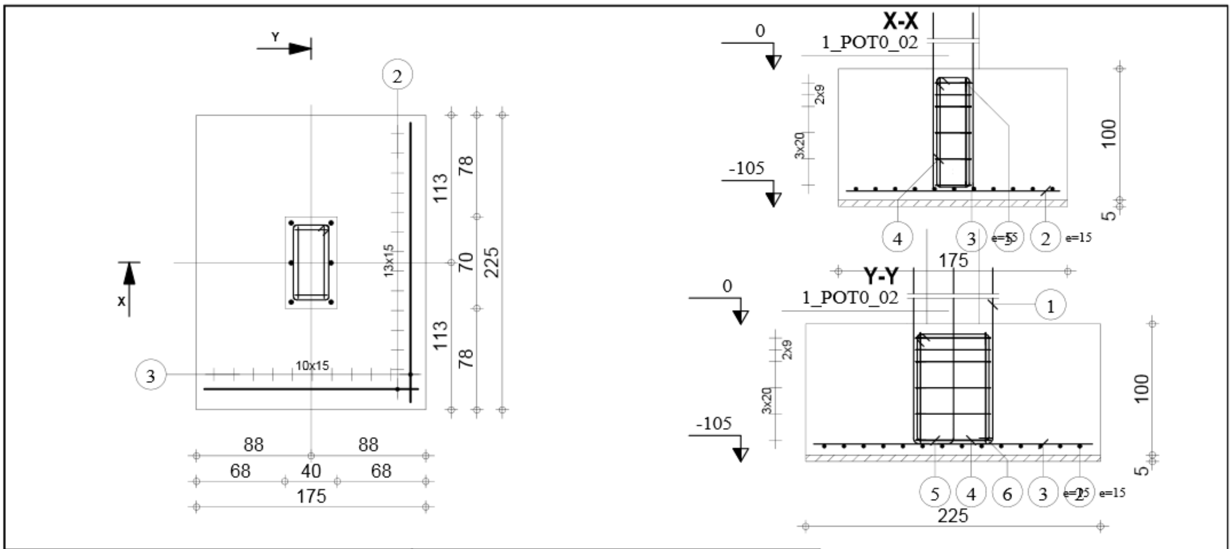
Diameter	Length (m)	Weight (kG)
10	24,12	14,88
12	16,83	14,94
16	197,25	311,43
20	31,42	77,50

Date : 16/11/15 Page : **4**



Pos.	Reinforcement	Code	Shape	Steel
1	14Ø20 l=224	00	13 215	B500B
2	75Ø16 l=263	00	263	B500B
3	8Ø10 l=302	31	58 88 88 58	B500B
4	2Ø12 l=388	31	56 132 132 56	B500B
5	2Ø12 l=453	31	86 135 135 86	B500B

Tel. Fax			Steel B500B = 389 kg
Cracking permissible		Concrete : C30/37 = 11.3 m3	Steel B500B = 29.8 kg
Standard Level POLO VIEWS_DESIGN_Recovered	**sel_1_SI0_07** Number 1	Formwork = 16.5 m2 Density (unit weight) = 37.08 kg/ m3	Cover c1 = 6 cm, c2 = 6 cm
			View scale 1/50 Section scale 1/50 — Page 1/1



Pos.	Reinforcement		Code	Shape	Steel	Pos.	Reinforcement		Code	Shape	Steel
1	6Ø20	l=174	00	13 165	B500B	4	6Ø10	l=182	31	58 28 58 28	B500B
2	14Ø16	l=163	00	163	B500B	5	2Ø12	l=288	31	56 82 82 56	B500B
3	11Ø16	l=213	00	213	B500B	6	2Ø12	l=233	31	26 85	B500B

Tel. Fax

Cracking permissible

Concrete : C30/37 = 3.94 m3 | Steel B500B = 98.8 kg | Steel B500B = 16 kg

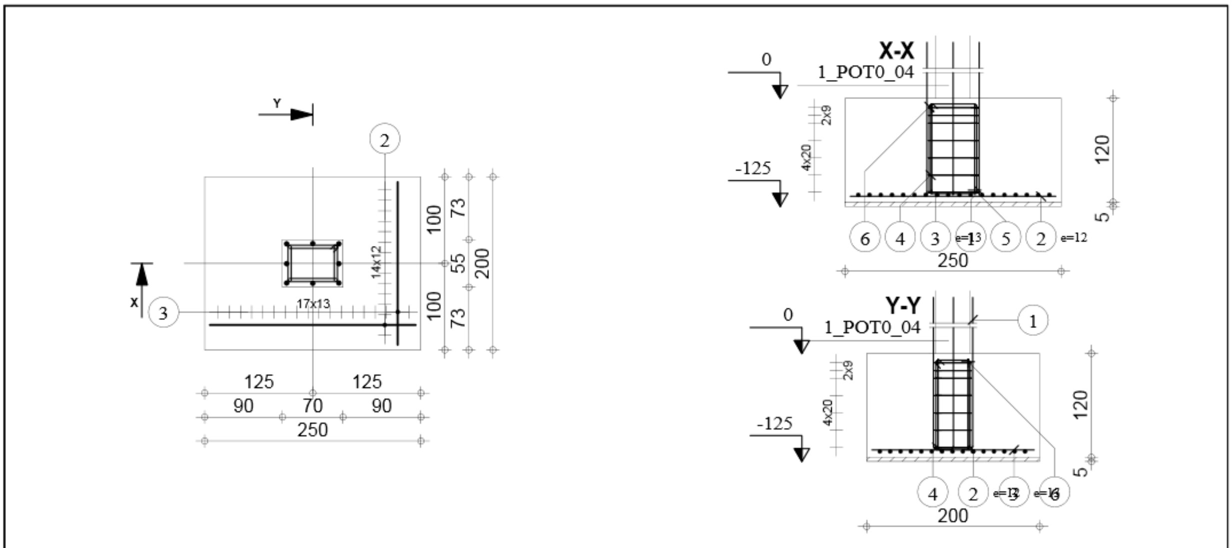
Standard Level — POLO VIEWS_DESIGN_Recovered

sel_1_SI0_13 — Number 1

Formwork = 8 m2 | Cover c1 = 6 cm, c2 = 6 cm

Density (unit weight) = 29.19 kg/ m3

View scale 1/33 | Section scale 1/33 | Page 1/1



Pos.	Reinforcement		Code	Shape	Steel	Pos.	Reinforcement		Code	Shape	Steel
1	8Ø20	l=194	00	13 185	B500B	4	7Ø10	l=212	31	43 58 58 43	B500B
2	15Ø16	l=238	00	238	B500B	5	2Ø12	l=298	31	41 102	B500B
3	18Ø16	l=188	00	188	B500B	6	2Ø12	l=333	31	56 105	B500B

Tel. Fax

Cracking permissible

Concrete : C30/37 = 6 m3 | Steel B500B = 148 kg | Steel B500B = 20.3 kg

Standard Level — POLO VIEWS_DESIGN_Recovered

sel_1_SI0_19 — Number 1

Formwork = 10.8 m2 | Cover c1 = 6 cm, c2 = 6 cm

Density (unit weight) = 28 kg/ m3

View scale 1/50 | Section scale 1/50 | Page 1/1

ANNEX 8: Retaining walls detailing

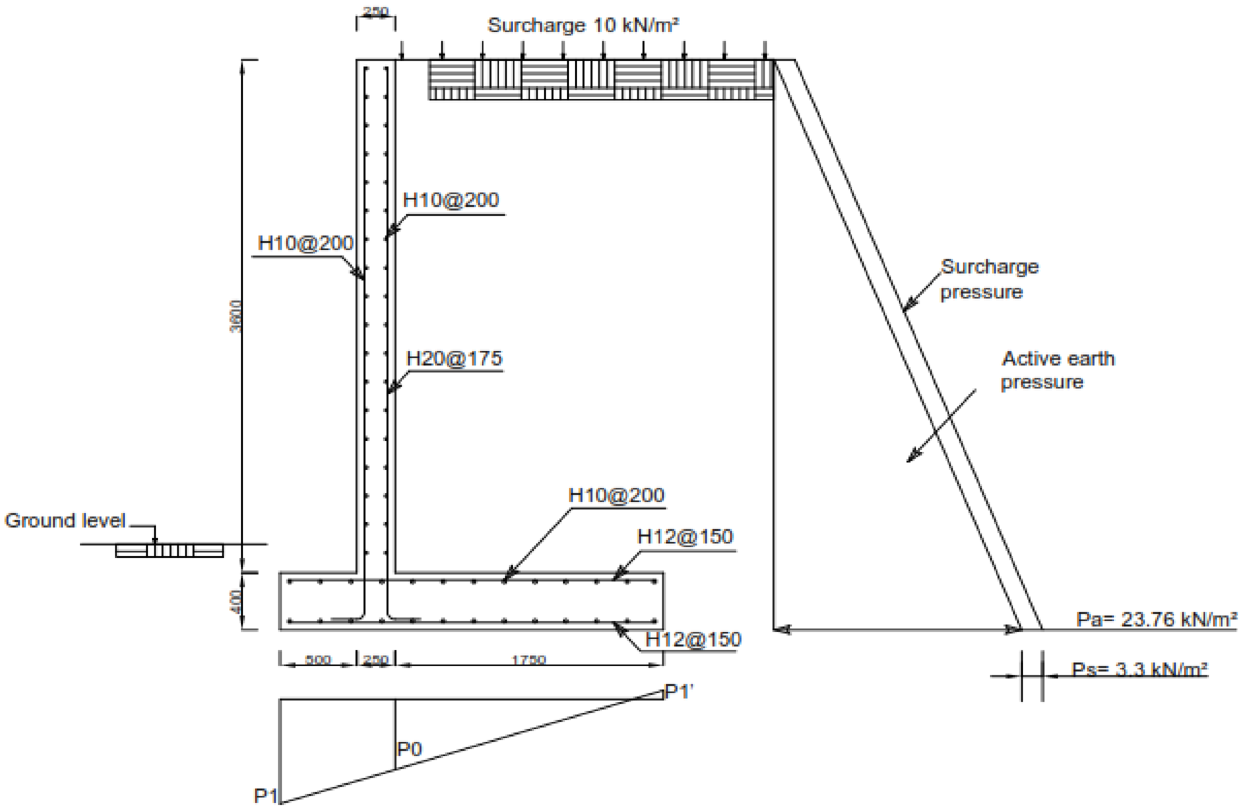


Figure 17: Retaining wall

There is three (3) types of retaining walls. But our concern in this document is the cantilever retaining wall. The walls are going to support a soil material of density 1800 kg/m². The design process may be split into these 3 fundamental stages:

a) Check the stability of the wall
b) Assess the bearing pressures at the ULS
c) Design the bending reinforcement using high yield steel, f_{yk} = 500 MPa and concrete class C30/37.

Stability

Horizontal force

It is assumed that the coefficient of active pressure Ka= 0.33, which is a typical value for a granular material. So the earth pressure is given by

$$P_a = K_a \rho g h \quad (37)$$

Where ρ is the density of the backfill and h is the depth considered. Thus at the base

$P_a = 0.33 \times 18 \times 4.0 = 23.76\ kN/m^2$

Allowing for the minimum required surcharge of 10 kN/m² an additional pressure of

$P_s = K_a \times 10 = 3.3\ kN/m^2$

Acts uniformly over the whole depth h.

Therefore the horizontal force on 1m length of wall is given by:

$H_{k(earth)} = 0.5 P_a h = 0.5 \times 23.76 \times 4.0 = 47.52\ kN$ from the active earth pressure

$H_{k(sur)} = P_s h = 3.3 \times 4.0 = 13.2\ kN$ from the surcharge pressure

Vertical loads

a) Permanent loads

$Wall\ =\ 0.25 \times 3.6 \times 25\quad = 22.5\ kN$

$Base\ =\ 0.5 \times 2.5 \times 25\quad = 25\ kN$

$Earth\ =\ 1.75 \times 3.6 \times 18 = 113.4\ kN$

TOTAL = 160.9 kN

b) Variable loads

$Surcharge = 1.75\ \times 10 = 17.5\ kN$

The partial factors of safety as given in table7.2 (table10.1 [5]) will be used

Table 6.3:Partial safety factors at ULS

Persistent or transient design situation	Permanent actions (G_k)		Leading variable action ($Q_{k,1}$)		Accompanying variable action ($Q_{k,i}$)	
	Unfavourable	Favourable	Unfavourable	Favourable	Unfavourable	Favourable
(a) for consideration of structural or geotechnical failure: combination 1 (STR) & (GEO)	1.35	1.00*	1.50	0	1.50	0
(b) for consideration of structural or geotechnical failure: combination 2 (STR) & (GEO)	1.00	1.00*	1.30	0	1.30	0
(c) for checking static equilibrium (EQU)	1.1	0.9	1.50	0	1.50	0

* To be applied to bearing, sliding and earth resistance forces.

i) Overturning: taking moments about point A at the edge of the toe, at ULS

For the overturning (unfavourable) moment a factor of 1.1 is applied to the earth pressure and a factor of 1.5 to the surcharge pressure

$$Overturning\ moment = \frac{\gamma_f H_{k(earth)} h}{3} + \frac{\gamma_f H_{k(sur)} h}{3} \tag{38}$$

$$Overturning\ moment = 1.1 \times 47.52 \times \frac{4.0}{3} + 1.5 \times 13.2 \times \frac{4.0}{3}$$

$$Overturning\ moment = 96.1\ kN\ m$$

For the restraining (favourable) moment a factor of 0.9 is applied to the permanent loads and 0 to the variable load

$$Restraining\ moment = \gamma_f (22.5 \times 0.625 + 25 \times 1.25 + 113.4 \times 1.5)$$

$$Restraining\ moment = 0.9 \times 215.413$$

$$Restraining\ moment = 193.87\ kN$$

$Overturning\ moment < Restraining\ moment$ thus the criterion for overturning is satisfied.

ii) Sliding: from equation (39) it is necessary that

$$\mu(1.0G_k + 1.0V_k) \geq \gamma_f H_k \text{ for no heel beam} \tag{39}$$

For the sliding (unfavourable) effect a factor of 1.35 is applied to the earth pressure and factor of 1.5 to the surcharge pressure

$$Sliding\ force = 1.35 \times 47.52 + 1.5 \times 13.2$$

$$Sliding\ force = 83.95\ kN$$

For the restraining (favourable) effect a factor of 1.0 is applied to the permanent loads and 0 to the variable surcharge load. With an internal angle of friction of 30°the coefficient of friction μ=0.58

$$Fictional\ resisting\ force\ =\ 0.58 \times 1.0 \times 160.9 = 93.32\ kN$$

$Sliding\ force < Frictional\ resisting\ force$ thus the criterion for sliding is satisfied and no heel beam is required.

- Bearing pressures at ultimate limit state

The weight of earth and the surcharge loading exerts a moment about the base centreline that will reduce the maximum pressure at the toe of the wall. Hence the effect of the weight of the earth is taken as a favourable

effect ($\gamma_f = 1$) and the weight of the surcharge load is also taken as a favourable effect ($\gamma_f = 0$) within the calculations below. The unfavourable effects of the lateral earth pressure and the lateral surcharge pressure are multiplied by factors of $\gamma_f = 1.35$ and $\gamma_f = 1.50$, respectively. The bearing pressures are given by

$$P = \frac{N}{D} \pm \frac{6M}{D^2} \qquad (41)$$

Where M is the moment about the base centreline. Therefore

$$M = \gamma_f\left(47.52 \times \frac{4.0}{3}\right) + \gamma_f\left(13.2 \times \frac{4.0}{3}\right) + \gamma_f \times 22.5(1.25 - 0.625) - \gamma_f \times 113.4 \times (1.625 - 1.25)$$

$$M = 1.35 \times 63.36 + 1.5 \times 17.6 + 1.35 \times 14.06 - 1.0 \times 42.525$$

$$M = 88.39 kN\ m$$

The bearing pressures at toe and heel of wall are:

Combination 1

$$P_1 = \frac{(1.35 \times (22.5 + 25) + 1.0 \times 113.4)}{2.5} \pm \frac{6 \times 88.39}{2.5^2} = 71.01 \pm 84.85$$

$$P_1 = 156\, kN/m^2\, ; P_1' = -13.84\, kN/m^2$$

Combination 2

$$P_2 = \frac{22.5 + 25 + 113.4}{2.5} \pm \frac{6 \times 52.5}{2.5^2} = 64.36 \pm 50.4$$

$$P_2 = 114.76\, kN/m^2\, ; P_2' = 13.96\, kN/m^2$$

- Bending reinforcement

i) Wall

Horizontal force

$$H_f = \gamma_f 0.5 K_a \rho g h^2 + \gamma_f P_S h \quad (42)$$

$$H_f = 1.35 \times 0.5 \times 0.33 \times 18 \times 4.0^2 + 1.5 \times 3.3 \times 4.0$$

$$H_f = 64.15 + 19.8 = 83.95\ kN$$

Considering the effective span, the maximum moment is

$$M_{Ed} = 64.15 \times \left(\frac{0.4}{2} + \frac{3.6}{3}\right) + 19.8 \times (\frac{0.4}{2} + \frac{3.6}{3})$$

$$M_{Ed} = 117.53\ kN\ m$$

$$\frac{M_{Ed}}{bd^2 f_{ck}} = \frac{117.53 \times 10^6}{1000 \times 190^2 \times 30} = 0.109 < 0.167$$

$$A_s = \frac{117.53 \times 10^6}{0.87 \times 169.52 \times 500} = 1593.82\ mm^2$$

Provide H20 @ 175 mm centres (As= 1795 mm^2)

ii) Base

The bearing pressures at ULS are obtained from part (2) of these calculations. Using the figures from (2):

Pressure: $P_1 = 156\ kN/m^2$; $P_1' = -13.84\ kN/m^2$ $P_0 = 109.2\ kN/m^2$

Heel: taking moments about the stem centreline for the vertical loads and the bearing pressures

$$M_{Ed} = \gamma_f \times 25 \times \left(\frac{2.5}{2} - 0.625\right) + \gamma_f 113.4 \times 1.0 - 109.2 \times \left(\frac{1.75}{3} + 0.125\right)$$

$$M_{Ed} = 1.35 \times 15.625 + 1.0 \times 113.4 - 77.35$$

$$M_{Ed} = 57.14\ kN.m$$

Therefore

$$\frac{M_{Ed}}{bd^2 f_{ck}} = \frac{57.14 \times 10^6}{1000 \times 344^2 \times 30} = 0.016 < 0.167$$

$$A_s = \frac{57.14 \times 10^6}{0.87 \times 339.07 \times 500} = 387.4\ mm^2$$

Provide H10 @ 200 mm centers (As=392 mm²), top steel.

Toe: taking moments about the stem centerline

$$M_{Ed} = \gamma_f \times 25 \times 0.375 \times \frac{0.5}{2.5} - 156 \times 0.375 \times 0.5$$

$$M_{Ed} = 1.35 \times 1.875 - 29.25\ kN.m$$

$$M_{Ed} = -26.72\ kN.m$$

Therefore

$$\frac{M_{Ed}}{bd^2 f_{ck}} = \frac{26.72 \times 10^6}{1000 \times 344^2 \times 30} = 0.0075 < 0.167$$

$$A_s = \frac{26.72 \times 10^6}{0.87 \times 341.71 \times 500} = 179.76\ mm^2$$

The minimum area for this, and for longitudinal distribution steel which is also required in the wall and the base, is given from table7.3 (table 6.8 [5])

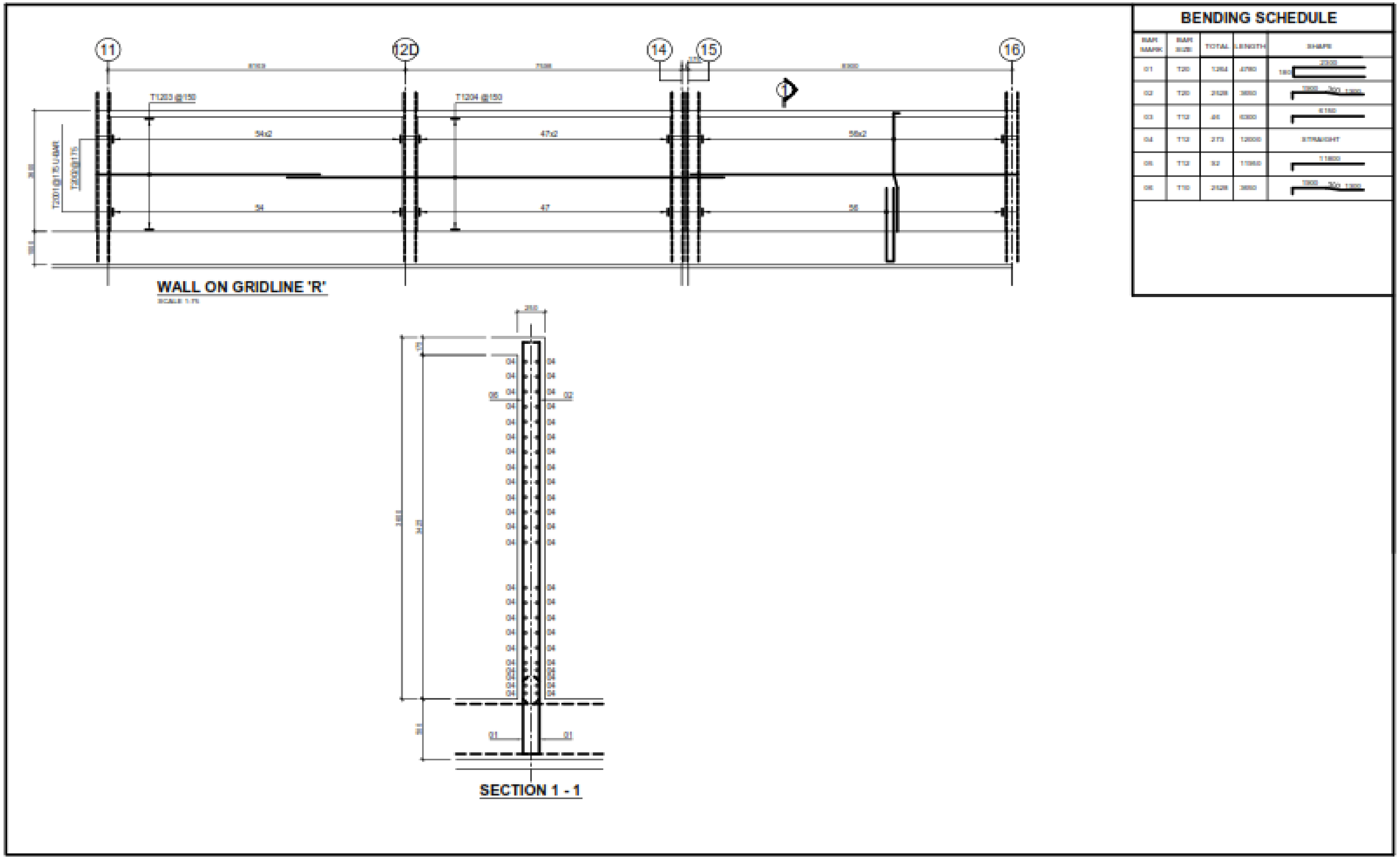
Table 6.4: Minimum area of reinforcement

Tension reinforcement in beams and slabs	Concrete class ($f_{yk} = 500\ N/mm^2$)			
	C25/30	C30/35	C40/50	C50/60
$\frac{A_{s,min}}{b_t d} > 0.26 \frac{f_{ctm}}{f_{yk}}$ (> 0.0013)	0.0013	0.0015	0.0018	0.0021
Secondary reinforcement $>$ 20% main reinforcement				
Longitudinal reinforcement in columns $A_{s,min} > 0.10N_{sd}/0.87f_{yk} > 0.002A_c$ where N_{sd} is the axial compression force				
Vertical reinforcement in walls $A_{s,min} > 0.002A_c$				

Note: b_t is the mean width of the tension zone.

$$A_{s,min} = 0.0015bd = 0.0015 \times 1000 \times 444 = 666\ mm^2$$

Thus, provide H12 bars at 150 mm centres (As = 753 mm²), bottom and distribution steel. Also steel should be provided in the compression face of the wall in order to prevent cracking – say, H10 bars and 200 mm centres each way.


BENDING SCHEDULE
11
12D
14
15
16
1
WALL ON GRIDLINE 'R'
SECTION 1 - 1

ANNEX 9: Stairs

The stairs are spanning longitudinally and set into pockets in the two supporting beams. The effective span is 2.5 m and the rise of the stairs is 1.5 m with 250 mm treads and 150 mm risers. The variable load is 3.0 kN/m² and the characteristic material strengths are fck = 25 MPa and fyk = 500 MPa. The stairs slab thickness is 150 mm.

Slope length pf stairs $=\sqrt{2.5^2 + 1.5^2} = 2.92\ m\ ; say\ 3m$

Consider a 1.5 m width of stairs:

- ***Load calculation***

$$\text{Weight of waist plus steps} = (0.15 \times 3 + 0.25 \times 0.15 \times 10/2) \times 25 \times 1.5$$

$$\text{Weight of waist plus steps} \ = 23.91\ kN$$

$$Imposed\ load\ =\ 3 \times 0.25 \times 10 \times 1.5 = 11.25\ kN$$

Ultimate load, $F\ = 1.35 \times 23.91 + 1.5 \times 11.25 = 49.15\ kN$

- ***Reinforcement calculation***

With no effective end restraint, the bending moment is:

$$M = \frac{Fl}{8} = \frac{49.15 \times 2.5}{8} = 15.36\ kN.m$$

Bending reinforcement:

$$Effective\ depth = 150 - 25 - 10 - 6 = 109\ mm$$

$$\frac{M}{bd^2 f_{ck}} = \frac{15.36 \times 10^6}{1500 \times 109^2 \times 25} = 0.034 < 0.156$$

$$z = 109[0.5 + \left(0.25 - \frac{0.034}{1.134}\right) = 78.48\ mm$$

$$A_s = \frac{M}{0.87 f_{yk} z} = \frac{15.36 \times 10^6}{0.87 \times 500 \times 78.48} = 450\ mm^2$$

The maximum allowable spacing is $3h\ = 3 \times 150 = 450\ mm$

Therefore we provide: T12 @ 200 mm C/C; Area provided = 565 mm²

Span-effective depth ratio

At the centre of the span: $\frac{100\,A_{s,prov}}{bd} = \frac{100\times565}{1500\times109} = 0.35\%$

Which is greater than the minimum requirement of 0.13% for class C25 concrete (table 6.8 reinforced concrete design to EC2).

Deflection check

$$Actual\ deflection\ = \frac{2500}{109} = 22.94$$

$$Limiting\ deflection\ = 20 \times \frac{A_{s,prov}}{A_{s,req}} = 20 \times \frac{565}{450} = 25.11$$

Secondary reinforcement

$$Transverse\ distribution\ steel\ \geq 0.2\,A_{s,prov} = 0.2 \times 565 = 113\,mm^2$$

This is very small and adequately covered by H10 bars at the maximum allowable spacing of 200 mm centers, area = 392 mm².

Continuity bars at the top and bottom of the span should be provided and, whereas about 50% of the main bars would be reasonable, the maximum spacing is limited to 400 mm. hence provide, say T12 @ 200 mm C/C as continuity steel.

ANNEX 10: Shear Walls

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:
File: **POLO VIEWS_DESIGN.RTD**
Project: POLO VIEWS_DESIGN

1 Level:

- Name : 0
- Storey level : Lower 0,00 (m)
- Position of the story : First
- Exposure : severe

2 Wall: 1_P0_5

2.1 Material properties:

- Concrete : fc28 = 25,00 (MPa) Density = 2501,36 (kG/m3)
- Longitudinal reinforcement : type HA 500 fe = 500,00 (MPa)
- Transversal reinforcement : type HA 500 fe = 500,00 (MPa)
- Concrete age (loading moment) : 28
- Behavior factor: q = 2,50

2.2 Geometry:

Name: P1

Length:	4,80 (m)
Thickness:	0,25 (m)
Height:	3,60 (m)
Ring beam height:	0,00 (m)
Vertical support:	---------
Support conditions :	Floor adjoining on two sides

2.3 Calculation options:

Calculations according to : BAEL 91 mod. 99
Cover : 3,0 (cm)

2.4 Loads:

2.4.1 Reduced:

Nature	N (kN)	M (kN*m)	H (kN)
Dead	2732,28	-321,31	-308,98
Dead	1002,67	-168,29	-172,83
Live	412,51	-28,88	-46,39
Live	39,21	-23,41	-16,03
Live	103,58	-19,12	-14,35
Live	40,60	-9,34	-3,98
Live	111,93	-54,51	-45,11
Wind	42,28	2,79	2,81
Wind	-42,41	-2,83	-3,33
Wind	-375,75	-176,46	-19,16
Wind	381,94	179,59	21,15

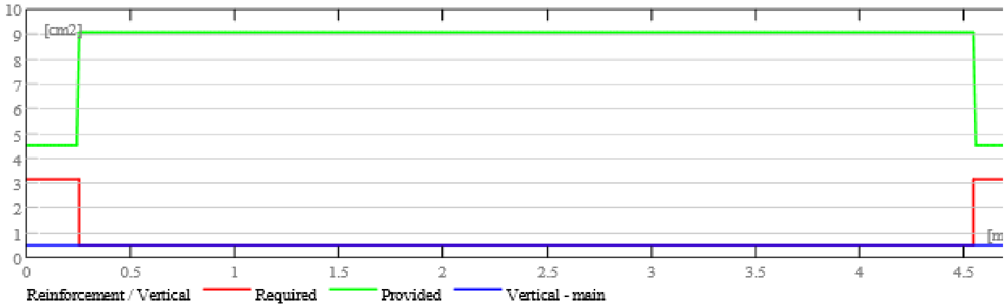
2.5 Calculation results:

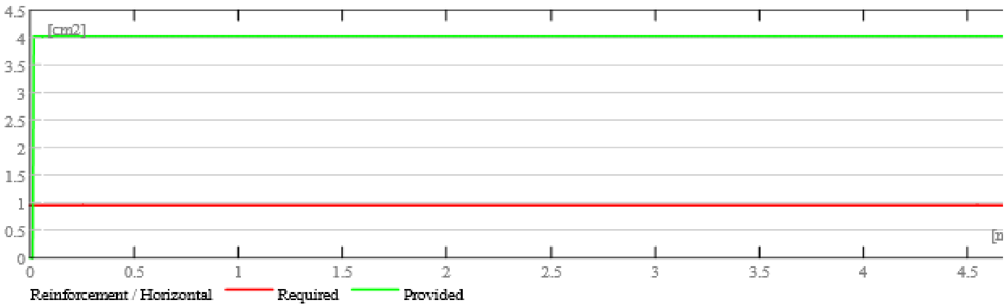
2.5.1 Diagrams

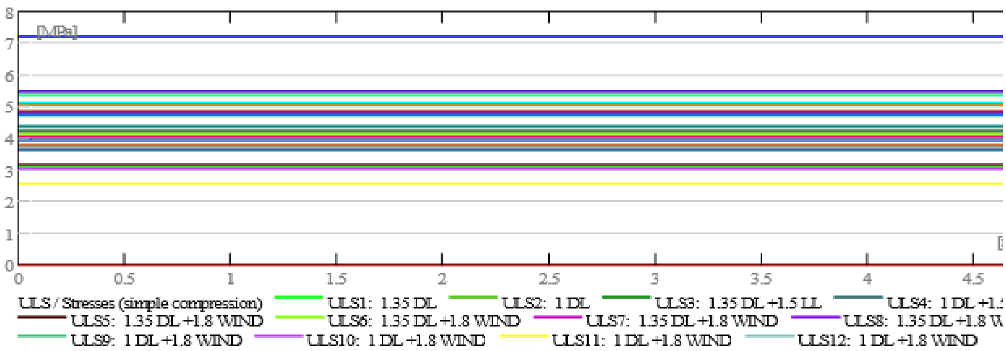
Date : 10/11/15 Page : **1**

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:

File: **POLO VIEWS_DESIGN.RTD**
Project: POLO VIEWS_DESIGN



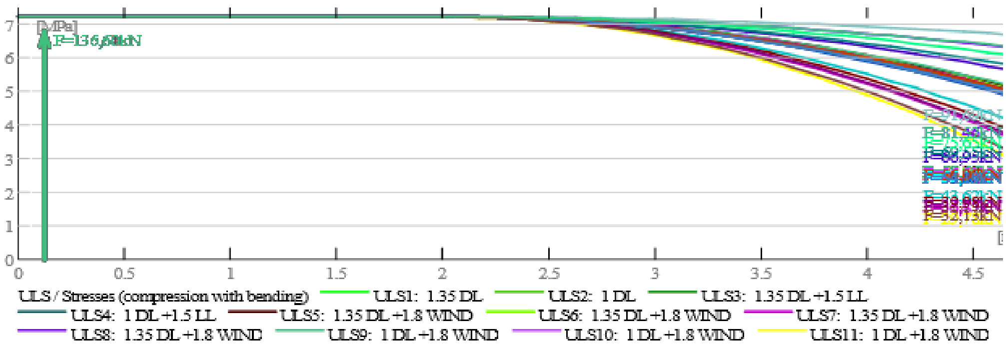


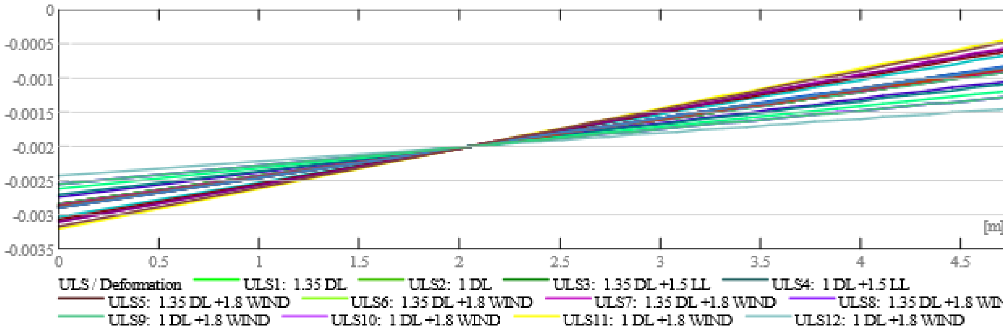


Date : 10/11/15 Page : **2**

Autodesk Robot Structural Analysis Professional 2012
Author:
Address:

File: **POLO VIEWS_DESIGN.RTD**
Project: POLO VIEWS_DESIGN





2.5.2 Theoretical results - detailed results:

2.5.2.1 Combinations

2.5.2.1.1 Internal forces in ULS

ULS.1	-	1.35 DL
ULS.2	-	1 DL
ULS.3	-	1.35 DL +1.5 LL
ULS.4	-	1 DL +1.5 LL
ULS.5	-	1.35 DL +1.8 WIND
ULS.6	-	1.35 DL +1.8 WIND
ULS.7	-	1.35 DL +1.8 WIND
ULS.8	-	1.35 DL +1.8 WIND
ULS.9	-	1 DL +1.8 WIND
ULS.10	-	1 DL +1.8 WIND
ULS.11	-	1 DL +1.8 WIND
ULS.12	-	1 DL +1.8 WIND
ULS.13	-	1.35 DL +1.5 LL +1.2 WIND
ULS.14	-	1.35 DL +1.5 LL +1.2 WIND
ULS.15	-	1.35 DL +1.5 LL +1.2 WIND
ULS.16	-	1.35 DL +1.5 LL +1.2 WIND
ULS.17	-	1 DL +1.5 LL +1.2 WIND
ULS.18	-	1 DL +1.5 LL +1.2 WIND
ULS.19	-	1 DL +1.5 LL +1.2 WIND
ULS.20	-	1 DL +1.5 LL +1.2 WIND
ULS.21	-	1.35 DL +1 LL +1.8 WIND
ULS.22	-	1.35 DL +1 LL +1.8 WIND
ULS.23	-	1.35 DL +1 LL +1.8 WIND
ULS.24	-	1.35 DL +1 LL +1.8 WIND
ULS.25	-	1 DL +1 LL +1.8 WIND
ULS.26	-	1 DL +1 LL +1.8 WIND

Date : 10/11/15 Page : **3**

ULS.27 - 1 DL +1 LL +1.8 WIND
ULS.28 - 1 DL +1 LL +1.8 WIND

2.5.2.2 Buckling length

Lf' = 3,06 (m)
Lf'_mf = 2,88 (m)
Lf = 3,06 (m)
Lf_mf = 2,88 (m)

2.5.2.3 Slenderness

λ = 42,40
λ mf = 39,91

2.5.2.4 Coefficient α

α/α1 = 1,1 (Concrete age (loading moment) :28)
α = 0,42
α mf = 0,61

2.5.2.5 Capacity of an unreinforced wall

σ ulim = 7,19 (MPa)

2.5.2.6 Distributed reinforcement

Design combination: ULS 16
N umax= 1367,14 (kN/m)
σ umax = 5,47 (MPa)
Nulim = 1798,36 (kN/m)
σ ulim = 7,19 (MPa)

Numax<Nulim => Unreinforced wall
1367,14 (kN/m) < 1798,36 (kN/m)

2.5.2.7 Edge reinforcement

2.5.2.7.1 Left edge

2.5.2.7.1.1 Stiffeners against bending with compression
Af L = 3,14 (cm2)
Design combination: ULS 1

2.5.2.7.1.2 Minimal posts
Width: d':
d' = 0,25 (m)

2.5.2.7.2 Right edge

2.5.2.7.2.1 Stiffeners against bending with compression
Af R= 3,14 (cm2)
Design combination: ULS 1

2.5.2.7.2.3 Minimal posts
Width: d':
d' = 0,25 (m)

2.5.2.8 Shear (BAEL91 A5.1,23)

Horizontal reinforcement
Design combination-ULS: ULS 15

Vu = 862,21 (kN)
τ = 0,80 (MPa)
Ah = 0,00 (cm2/m)

2.6 Reinforcement:

Vertical reinforcement:

Zone X0 (m)	X1 (m)	Number:	Steel	Diameter (mm)	Length (m)	Spacing (m)
0,25	4,55	36	HA 500	12,0	4,10	0,25

X0 - Zone beginning
X1 - Zone end

Horizontal reinforcement:

Type	Number:	Steel	Diameter (mm)	A (m)	B (m)	C (m)	Spacing (m)	Shape
Straight bars	30	HA 500	8,0	4,74	0,00	0,00	0,25	00
U loops	15	HA 500	8,0	0,38	0,18	0,38	-	21
U loops	15	HA 500	8,0	0,38	0,18	0,38	-	21

Pins:

Number:	Steel	Diameter (mm)	A (m)	B (m)	C (m)	Shape
144	HA 500	8,0	0,20	0,00	0,00	00

Edge reinforcement (Af):

	Number:	Steel	Diameter (mm)	A (m)	B (m)	C (m)	Shape
Longitudinal reinforcement - left side	4	HA 500	12,0	4,10	0,00	0,00	00
Longitudinal reinforcement - right side	4	HA 500	12,0	4,10	0,00	0,00	00
Transversal reinforcement - left side	15	HA 500	8,0	0,19	0,19	0,19	31
Transversal reinforcement - right side	15	HA 500	8,0	0,19	0,19	0,19	31

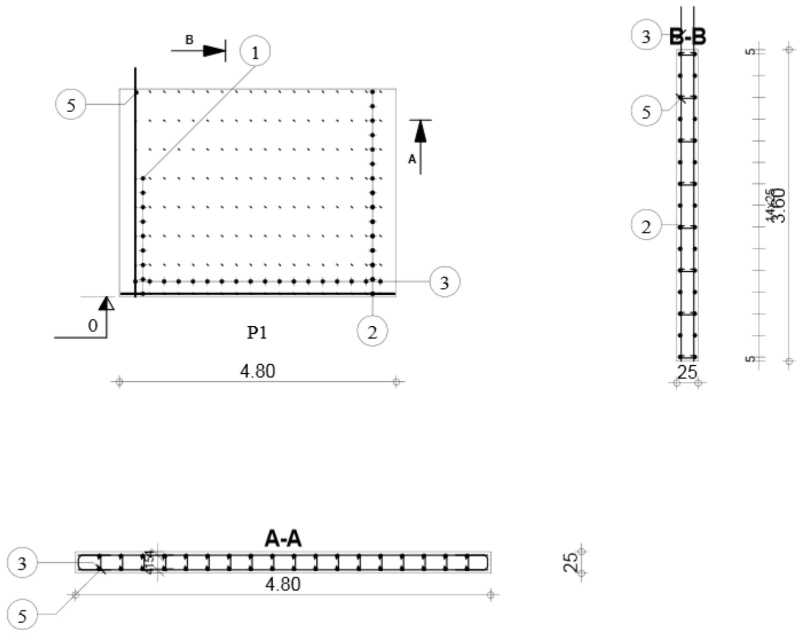
3 Material survey:

- Concrete volume = 4,32 (m3)
- Formwork = 36,36 (m2)

- Steel HA 500
  - Total weight = 257,22 (kG)
  - Density = 59,54 (kG/m3)
  - Average diameter = 9,7 (mm)

- **Survey according to diameters:**

Diameter	Length (m)	Weight (kG)
8	245,84	97,04
12	180,36	160,18



Pos.	Reinforcement		Code	Shape
1	30HA 8	l=87	21	18
2	30HA 8	l=4.74	00	4.74
3	36HA 12	l=4.10	00	4.10
5	144HA 8	l=35	00	6 20

Tel. Fax

Fire resistance 0h — Cracking permissible

Concrete : BETON = 4.32 m3 — Steel HA 500 = 227 kg — Steel HA 500 = 30.6 kg

Formwork = 34.6 m2 — Bottom cover 3 cm — Side cover 3 cm

Density (unit weight) = 59.49 kg/m3 — View scale 1/75

Average diameter = 9.69mm — Section scale 1/50

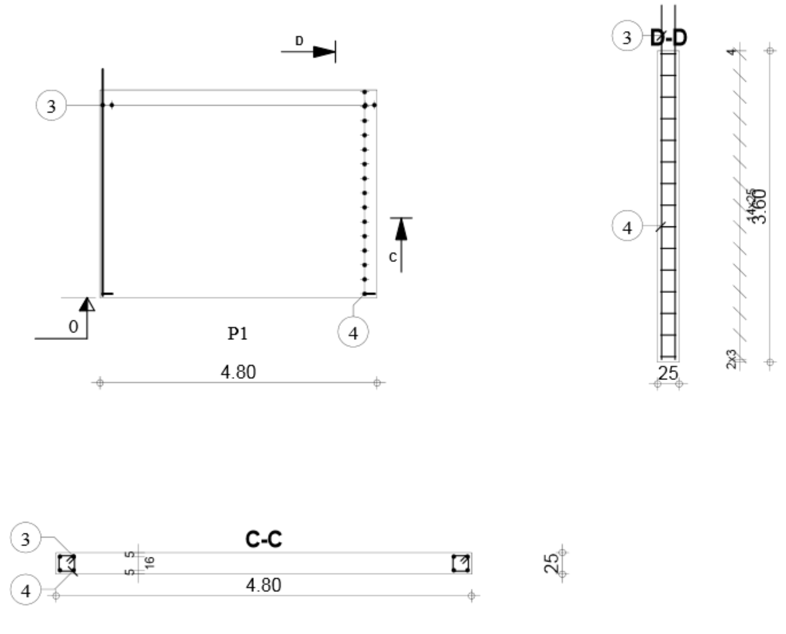
0

POLO VIEWS_DESIGN

1_P0_5 : P1 Number 1

Section 25x360

Page 1/2



Pos.	Reinforcement		Code	Shape
3	8HA 12	l=4.10	00	4.10
4	30HA 8	l=90	31	19 19

Tel. Fax

Fire resistance 0h — Cracking permissible

Concrete : BETON = 4.32 m3 — Steel HA 500 = 227 kg — Steel HA 500 = 30.6 kg

Formwork = 34.6 m2 — Bottom cover 3 cm — Side cover 3 cm

Density (unit weight) = 59.49 kg/m3 — View scale 1/75

Average diameter = 9.69mm — Section scale 1/50

0

POLO VIEWS_DESIGN

1_P0_5 : P1 Number 1

Section 25x360

Page 2/2

ANNEX 11: Quantity survey

Table 11.1: Basement floor survey

BASEMENT					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**ESCAVATION**				
I.1	Setting out of the building	Total	1	500	500
I.2	Escavation of soil for basement	m^3	3600	25	90000
	Subtotal 1				90500
II	**FOUNDATION-REINFORCED CONCRETE**				
II.1	Blinding C20/25 and 5 cm thickness	m^3	13,8915	420	5834
II.2	Reinforced concrete C30/37 for pad footings	m^3	244,2	680	166056
II.3	Reinforced concrete C30/37 for shear walls footings	m^3	60,36	680	41045
II.4	Reinforced concrete C25/30 for underground beams	m^3	28,9	550	15895
II.5	Reinforced concrete C25/30 for underground slab	m^3	90	550	49500
II.6	Reinforced concrete C30/37 for underground columns	m^3	13,13	680	8928
II.7	Reinforced concrete C30/37 for Retaining walls	m^3	185,7815	680	126331
II.8	Reinforced concrete C25/30 for Shear walls (stairs)	m^3	7,6	550	4180
II.9	Reinforced concrete C25/30 for Shear walls (lift)	m^3	14,02	550	7711
II.10	Reinforced concrete C25/30 for Stairs	m^3	5,36	550	2948
	Subtotal 2				428429
III	**PLASTERING**				
III.1	Screed on bottom slab	m^2	545	35	19075
III.2	Screed on top slab	m^2	545	30	16350
III.3	Screed on interior face of retaining walls	m^2	312	30	9360
	Subtotal 3				44785
IV	**WATER PROOFING**				
IV.1	Protection around retaining walls	m^2	312	95	29640
IV.2	Protection of pad footings	m^2	277,83	95	26394
IV.3	Protection of the basement floor slab	m^2	800	95	76000
	Subtotal 4				132034
	TOTAL BASEMENT				695748

Table 11.2:Basement floor survey

GROUND FLOOR TO 6th FLOOR					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**REINFORCED CONCRETE**				
I.1	Reinforced concreteC25/30 for Floor beams	m^3	74,70	550	41086
I.2	Reinforced concrete C25/30 for Ground Floor slab	m^3	95,38	550	52456
I.3	Reinforced concrete C30/37 for Ground Floor columns	m^3	30,48	680	20726
I.4	Reinforced concrete C25/30 for Shear walls (stairs)	m^3	7,6	550	4180
I.5	Reinforced concrete C25/30 for Shear walls (lift)	m^3	14,02	550	7711
I.6	Reinforced concrete C25/30 for Stairs	m^3	5,36	550	2948
	Subtotal 1				129107
II	***MASONRY***				
II.1	***Blockwork 15 cm thick***	m^2	1123,74	65	73043
	Subtotal 2				73043
III	**PLASTERING**				
III.1	Screed on bottom slab	m^2	545	35	19075
III.2	Screed on top slab	m^2	545	30	16350
	Subtotal 3				35425
	TOTAL FLOOR				237576

Table 11.3: 7th Floor survey

7th FLOOR					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**REINFORCED CONCRETE**				
I.1	Reinforced concrete C25/30 for 7th Floor beams	m^3	78,55	550	43203
I.2	Reinforced concrete C25/30 for 7th Floor slab	m^3	87,50	550	48125
I.3	Reinforced concrete C30/37 for 7th Floor columns	m^3	30,48	680	20726
I.4	Reinforced concrete C25/30 for Shear walls (stairs)	m^3	7,6	550	4180
I.5	Reinforced concrete C25/30 for Shear walls (lift)	m^3	14,02	550	7711
I.6	Reinforced concrete C25/30 for Stairs	m^3	11	550	6050
	Subtotal 1				129995
II	***MASONRY***				
II.1	***Blockwork 15 cm thick***	m^2	389,31	65	25305
	Subtotal 2				25305
III	**PLASTERING**				
III.1	Screed on bottom slab	m^2	500	35	17500
III.2	Screed on top slab	m^2	500	30	15000
	Subtotal 3				32500
	TOTAL FLOOR				187800

Table 11.4: 8th Floor survey

8th FLOOR					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**REINFORCED CONCRETE**				
I.1	Reinforced concrete C25/30 for 8th Floor beams	m^3	60,03	550	33017
I.2	Reinforced concrete C25/30 for 8th Floor slab	m^3	70,00	550	38500
I.3	Reinforced concrete C30/37 for 8th Floor columns	m^3	29	680	19720
I.4	Reinforced concrete C25/30 for Shear walls (stairs)	m^3	7,6	550	4180
I.5	Reinforced concrete C25/30 for Shear walls (lift)	m^3	14,02	550	7711
I.6	Reinforced concrete C25/30 for Stairs	m^3	5,36	550	2948
	Subtotal 1				106076
II	***MASONRY***				
II.1	***Blockwork 15 cm thick***	m^2	504	65	32760
	Subtotal 2				32760
III	**PLASTERING**				
III.1	Screed on bottom slab	m^2	400	35	14000
III.2	Screed on top slab	m^2	400	30	12000
	Subtotal 3				26000
	TOTAL FLOOR				164836

Table 11.5:9th Floor survey

9th FLOOR					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**REINFORCED CONCRETE**				
I.1	Reinforced concrete C25/30 for 9th Floor beams	m^3	55,39	550	30465
I.2	Reinforced concrete C25/30 for 9th Floor slab	m^3	48,00	550	26400
I.3	Reinforced concrete C30/37 for 9th Floor columns	m^3	14,2	680	9656
I.4	Reinforced concrete C25/30 for Shear walls (stairs)	m^3	7,6	550	4180
I.5	Reinforced concrete C25/30 for Shear walls (lift)	m^3	14,02	550	7711
I.6	Reinforced concrete C25/30 for Stairs	m^3	3,22	550	1771
	Subtotal 1				80183
II	***MASONRY***				
II.1	***Blockwork 15 cm thick***	m^2	291,3	65	18935
	Subtotal 2				18935
III	**PLASTERING**				
III.1	Screed on bottom slab	m^2	400	35	14000
III.2	Screed on top slab	m^2	240	30	7200
	Subtotal 3				21200
	TOTAL FLOOR				120317

Table 11.6:Roof level 1 & 2 floor survey

ROOF LV1 & LV2					
N°	**Designation**	**Unit**	**Quantity**	**Unit Price [GHC]**	**Total Price [GHC]**
I	**REINFORCED CONCRETE**				
I.1	Reinforced concrete C25/30 for 7th Floor beams	m^3	18,53	550	10192
I.2	Reinforced concrete C25/30 for 7th Floor slab	m^3	18,55	550	10203
I.3	Reinforced concrete C30/37 for Ground Floor columns	m^3	8,16	680	5549
I.4	Reinforced concrete C25/30 for Shear walls (stairs)	m^3	7,6	550	4180
I.5	Reinforced concrete C25/30 for Shear walls (lift)	m^3	14,02	550	7711
I.6	Reinforced concrete C25/30 for Stairs	m^3	1,56	550	858
	Subtotal 1				38692
II	***MASONRY***				
II.1	***Blockwork 15 cm thick***	m^2	6,25	65	406
	Subtotal 2				406
III	**PLASTERING**				
III.1	Screed on bottom slab	m^2	88	35	3080
III.2	Screed on top slab	m^2	88	30	2640
	Subtotal 3				5720
	TOTAL FLOOR				44818

After applying the corresponding rate (1000 FCFA corresponds to 6 GHC), we got an amount of **504732406 FCFA** for the realisation of the structural framework (Slab, Beams, Columns, Footings, Retaining walls, Shear walls and stairs) of the building.