Faculty Of Engineering & Built Environment

Department of Civil Engineering & Geomatics

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Subject: Design of Multiple Storey Building

Subject Code: DMB581S

Course facilitator: Dr M Pourbehi

Project : Two-Way Slab System with Lateral Loads (Wind & Seismic)

EVODIE MWANGE KIBAMBO
220074852

Team Members	Student Numbers
MIHLALI NKOMO	222047178
LINDOKUHLE MBUYAZI	216071402
EVODIE MWANGE KIBAMBO	220074852

Assignment Submittal & Identification

Student Number	220074852
Surname & Initials	KIBAMBO EM
Program	Bachelor of Engineering Technology Honours Civil Engineering
Subject Name	Design of Multi-Storey Buildings
Subject Code	D M B 5 8 1 S

Assignment Number : 0 1

Due date: 1 2 / 1 1 / 2 0 2 5

Lecturer: Dr. Mohammed Pourbehi

DECLARATION OF OWN WORK:

I, student name ___Evodie Mwange Kibambo___, student number __220074852________

declare that this report compiled is my own work, that it has not been submitted for any other degree at CPUT or any other University or any higher education institution, and that all resources that I have used or quoted are indicated in the text and acknowledged in the list of references.

Signed: EM KIBAMBO　　　　　　　　Date: 01/11/2025

Abstract

This report represents the structural analysis and design of a two-way concrete slab system of a multiple storey building subjected to both gravity and lateral loads, in accordance with the South African National Standards (SANS 10160 and SANS 10100-1). The analysis was conducted for a structure located in Durban, KwaZulu-Natal, where wind and seismic actions were determined based on site-specific parameters, including basic wind speed, terrain category, and peak ground acceleration.

The structure was modelled as a reinforced concrete frame and slab system using PROKON finite element software. The analysis considered ultimate and serviceability limit states, with verification of wind and seismic responses against simplified hand methods. The two-way slab was designed using the Direct Design Method, and beams and columns were detailed to meet the requirements of strength and serviceability.

The results demonstrate that the proposed structural system meets safety and serviceability criteria, with all member forces within acceptable limits.

In addition to technical outcomes, the report reflects professional practice through ethical decision-making, self-management, and quality assurance. The project highlights the importance of integrating engineering judgement in modern structural design under combined gravity and lateral actions.

Table of Content

List of figures

List of tables

1. INTRODUCTION

1.1 Project Background

This report presented the structural analysis and design of a reinforced concrete two-way slab system subjected to both gravity and lateral loads, including wind and seismic actions. The structure represented a typical reinforced concrete classroom block consisting of spans in each direction, supported by beams and columns (as shown in Figure 1). The project was done to demonstrate competence in integrated structural analysis, design, and professional engineering practice in accordance with South African standards.

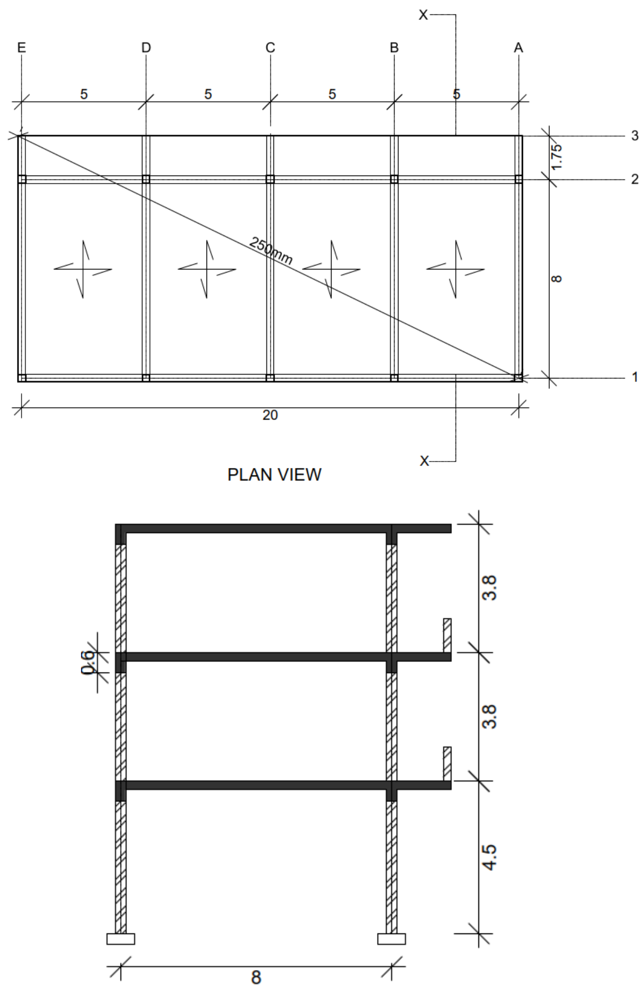


Figure 1: Building plan view and elevation

1.2 Objectives

The primary objective of the project was to analyse and design the structural floor system to safely resist gravity, wind, and seismic actions while maintaining an adequate serviceability state. The objectives included:

- The analysis of the two-way slab using the procedures outlined in SANS 10100-1.

- The determination of wind and seismic actions in accordance with SANS 10160-3 and SANS 10160-4, using site-specific parameters based on the selected project location.
- To develop a 3D finite element (FE) model of the structure in PROKON,
- Verification of results through simplified hand calculations and ensuring consistency within provided limits.
- The design of slab, beams, and columns for ultimate and serviceability limit states, including reinforcement detailing, deflection checks, and crack control.

The study adopted a systematic approach beginning with the identification of the project site and corresponding design inputs, followed by gravity load estimation, lateral load derivation, finite element modelling, and component design. The structure was modelled as a two-way slab system supported on beams located along gridlines 1 and 2, forming a fully supported floor panel. Wind and seismic actions were incorporated alongside gravity loads to evaluate the building's overall behaviour under lateral actions.

All analysis and design work adhered to the relevant provisions of the South African National Standards (SANS), ensuring that the final solution met the fundamental requirements of strength, stability, and serviceability. The outcomes of this project provided an integrated understanding of how wind and seismic effects influence slab, beam, and column systems, reinforcing the importance of combining compliance to code and professional responsibility in structural engineering practice.

2. Site Selection & Justification

The project site was located in Geduld, within the City of Ekurhuleni in the Gauteng Province of South Africa. This location was selected due to its strategic location and the availability of geotechnical information that is crucial for wind load and seismic analysis. The surrounding area of Geduld is predominantly urban with a mixture of residential and industrial developments. Based on the built environment and the level of surface roughness, the site was classified as Terrain Category C, Based on the site characteristics, Terrain Category C was selected as it represents areas with regular vegetation cover, buildings, or isolated obstacles, which aligns with the suburban context of the project location.

Figure 2 : Aerial view of site

3. Phase 1 – Analysis (Two-Way Slab + Lateral Actions)

3.1 Actions and Load Combinations (SANS 10160)

This section presents the gravity (G) and variable (Q) actions applied to the slab, beams, and columns. All load values were determined in accordance with SANS 10160-1 and SANS 10160-2.

The following steps were carried out:

- Permanent actions (self-weight, finishes, partitions) were calculated based on material densities and element dimensions.
- Variable actions were selected according to the occupancy category of the building.
- Ultimate Limit State (ULS) and Serviceability Limit State (SLS) combinations were established by referencing SANS 10160-1
- Lateral actions were included by adding wind (W) and seismic (E) to combinations where relevant.
- Load-case diagrams and load-combination diagrams were prepared to illustrate the applied actions.

3.1.1 Tabulated load combinations

Load Combination	Permanent Load (Gk)	Live Load(Qk)	Wind Load (Wk)
STR-P (ULS)			
Unfavorable			
C1	Permanent load with live load only		
	1.35	1	
C2	Permanent load with wind load only		
	1.35		1
C3	Permanent load with live load leading and wind load accompanying		
	1.35	1	0x1
C4	Permanent load with wind load leading and live load accompanying		
	1.35	0.3 x 1.0	1

Table 1 STR-P ULS load combinations

Load Combination	Permanent Load (Gk)	Live Load(Qk)	Wind Load (Wk)
STR (ULS)			
Favorable			
C1	Permanent load only		
	0.9		
Unfavorable			

C2	Permanent load and live load only		
	1.2	1.6	
C3	Permanent load and live load leading and wind load accompanying		
	1.2	1.6	0.0 x 1.6
C4	Permanent load and wind load only		
	1.2		1.6
C5	Permanent load and wind load leading and live load accompanying		
	1.2	0.3 x 1.6	1.6

Table 2: STR ULS Load combinations

Load Combination	Permanent Load (Gk)	Live Load(Qk)	Wind Load (Wk)	Seismic Load (E)
ACC (ULS)				
Unfavorable				
C1	Permanent load and seismic load only			
	1			1
C2	Permanent load and seismic load leading and live load accompanying			
	1	0.3 x 1		1
C3	Permanent load and live load leading and wind load accompanying			
	1	1	0.0 x 1	
C4	Permanent load and wind load leading and live load accompanying			
	1	0.3 x 1	1	

Table 3: Accidental ULS Load Combinations

Load Combination	Permanent Load (Gk)	Live Load(Qk)	Wind Load (Wk)
Irreversible serviceability (SLS)			
Un-favorable			
C1	Permanent load only		
	1.1		
Favorable			
C2	Permanent load and live load only		
	1	1	
C3	Permanent load and live load leading and wind load accompanying		
	1	1	0.3 x 0.6
C4	Permanent load and wind load only		
	1		0.6
C5	Permanent load and wind load leading and live load accompanying		
	1	0.3 x 1	0.6

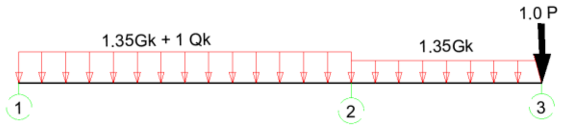
Table 4 : Irreversible serviceability SLS combinations

3.1.2 Load Cases and Combinations Diagram

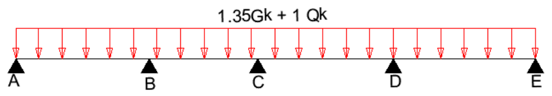
The diagrams below were developed using the different combinations

a) STR-P Combination diagrams

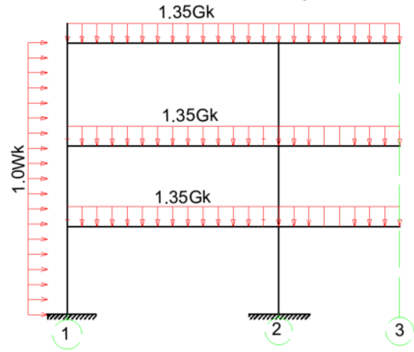
C.1 Permanent load with live load only (same as C.3)



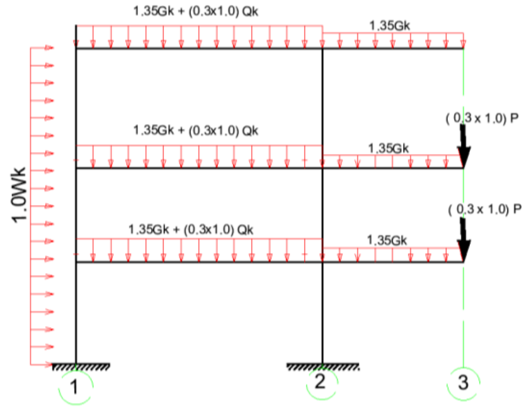
for the floor Slab



C.2 Permanent load with wind load only

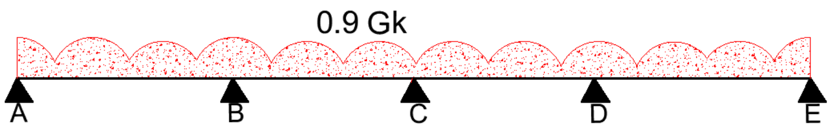


C.4 Permanent load with Wind load leading and live load accompanying



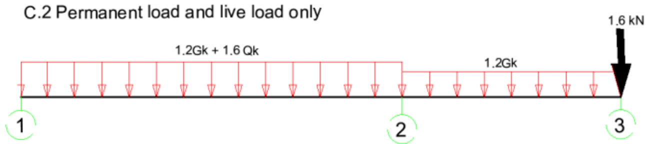
b) STR combination diagrams

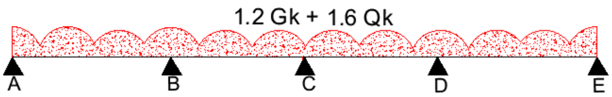
C.1 (Favorable) Permanent load only



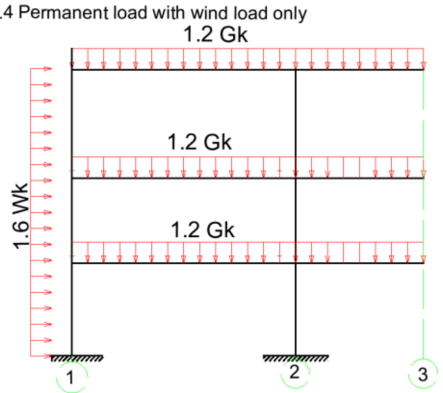
Unfavorable

C.2 Permanent load and live load only

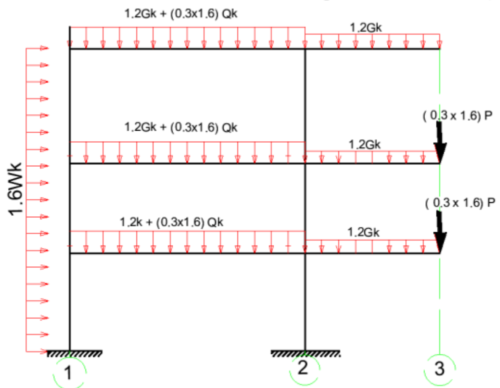




C.4 Permanent load with wind load only



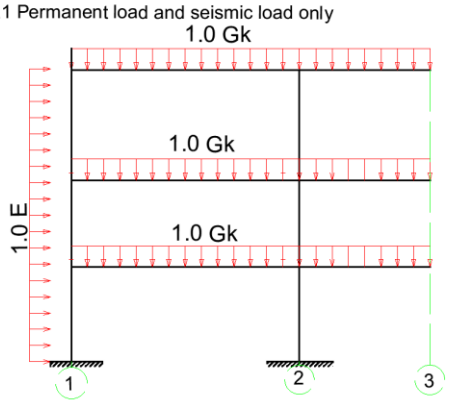
C.5 Permanent load with Wind load leading and live load accompanying



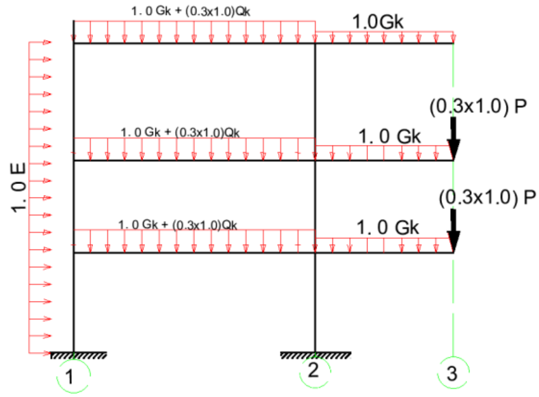
c) Accidental Combination diagrams

ACC (ULS)

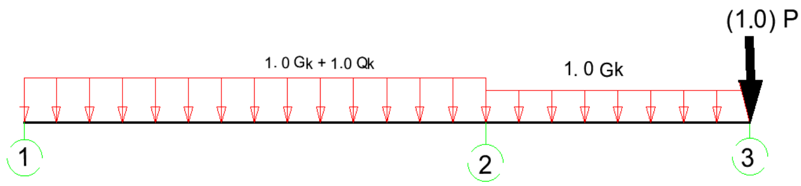
C.1 Permanent load and seismic load only

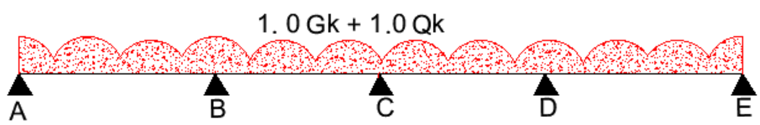


C.2 Permanent load with seismic load leading and live load accompanying

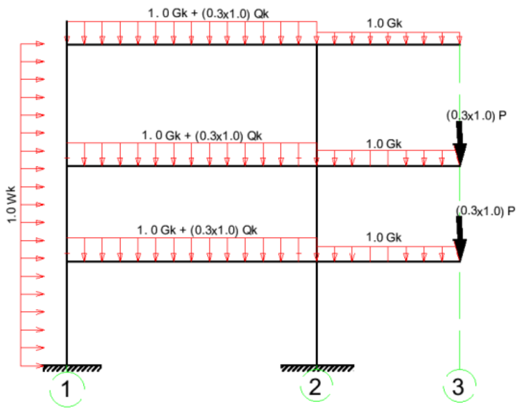


C.4 Permanent load with live load leading



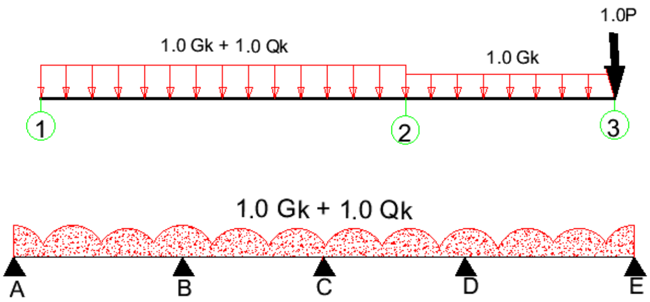


C.4 Permanent load with wind load leading and live load accompanying

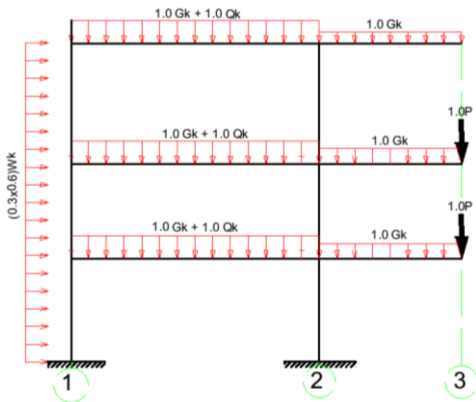


d) Irreversible Combination Diagrams

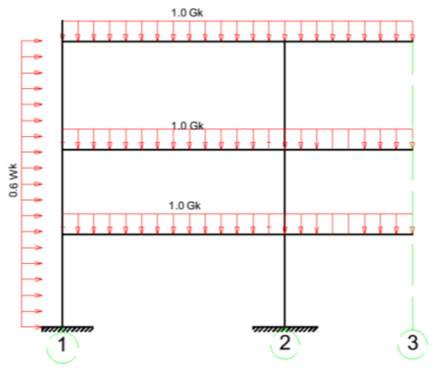
C.2 Permanent load and live load



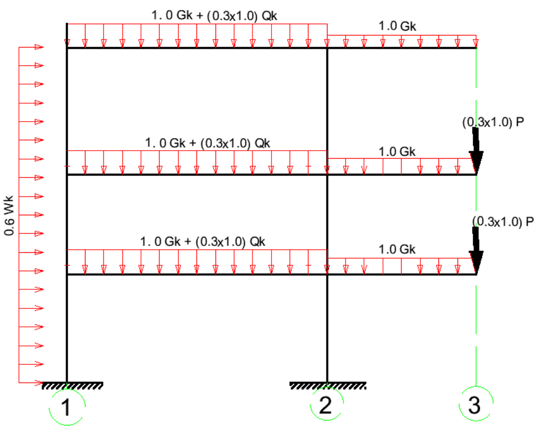
C.3 Permanent load with live load leading and wind load accompanying



C.4 Permanent load with wind only



C.5 Permanent load with wind load leading and live load accompanying



3.2 Two-Way Slab Analysis (Gravity)

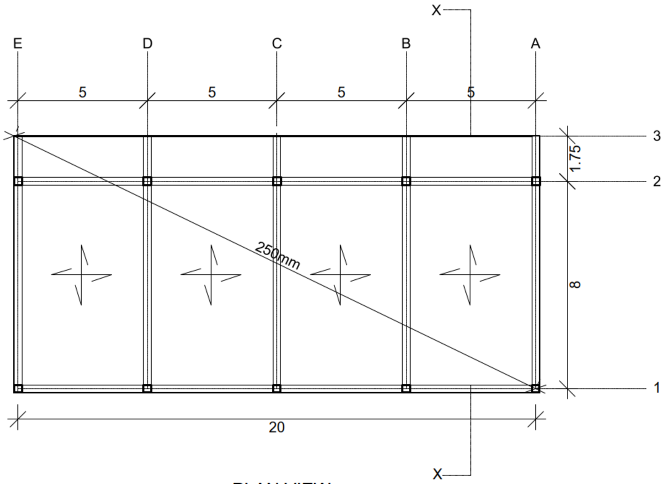
- The two-way slab system was analysed under gravity loads using the Direct Design Method (DDM) in accordance with SANS 10100-1. The slab was supported on all four edges by beams on gridlines 1 and 2, and span from grid A to E, forming a continuous panel. Below is displayed gravity load analysis and distribution on the slab, which will serve as basis for the design.

References	**Calculations**

SANS 10160-1

- **Slab Load Calculations**

Plan Layout



PLAN VIEW

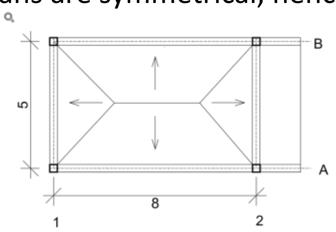
Spans:
A–B–C–D–E
Each span = 5 m
Width of slab = 8 m

- Material Properties

Element	Property	Value
Slab	Density	24 kN/m^3
	Thickness	0.250 m
Wall	Weight	5 kN/m^2
	Thickness	0.230 m
Screed	Density	23 kN/m^3
	Thickness	0.050 m
Surface	Density	0.15 kN/m^2

- **Load distribution on Span 1 and 4**

Both spans are symmetrical, hence they will have the same load distribution.



1. Classroom Load

(a) Self-weight: Classroom

Element	Density (kN/m^3)	Thickness (m)	Distributed Load (kN/m^2)
Slab	24	0.25	6
Screed	23	0.05	1.15
Service	-	-	0.15
Wall	-	-	5
Total			12.3 kN/m^2

(B) Imposed Load: Classroom

Occupancy	Distributed Load (kN/m²)
Classroom	3

- Factored Load
  n=1.2Gk+1.6Qk
  n=19.56 kN/m²
- Span check $\frac{ly}{lx} = \frac{8}{5} = 1.6 < 2$

This is a two-way slab, hence the load will be distributed as follows:

a.1 Trapezoidal load

$$w = \frac{nlx}{6}\left[3 - \left(\frac{lx}{ly}\right)^2\right]$$

w= 42.53 kN/m

a.2 Triangular load

$$w = \frac{nlx}{3}$$

w= 32.6 kN/m

2. Balcony Load

(a) Self-weight: Balcony

Element	Density (kN/m³)	Thickness (m)	Distributed Load (kN/m²)
Slab	24	0.25	6
Screed	23	0.05	1.15
Services	-	-	0.15
Total			**7.3 kN/m²**

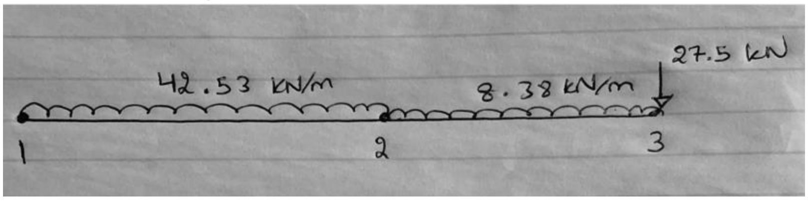
(b) Impose Load: Balcony

Occupancy	Distributed Load (kN/m²)
corridors	**5**

- Factored Load
  n=1.2Gk+1.6Qk
  n=16.76 kN/m²

-Balustrade Load = 5x1.1x5 = 27.5 kN

- Beam 1-2-3 on grid line A and E



- **Load distribution on Span 2 and 3**
  Factored Load
  n=1.2Gk+1.6Qk
  n=19.56 kN/m²

Span check $\frac{ly}{lx} = \frac{8}{5} = 1.6 < 2$

This is a two-way slab, hence the load will be distributed as follows:

a.1 Trapezoidal load

$$w = \frac{nlx}{6}\left[3 - \left(\frac{lx}{ly}\right)^2\right]$$

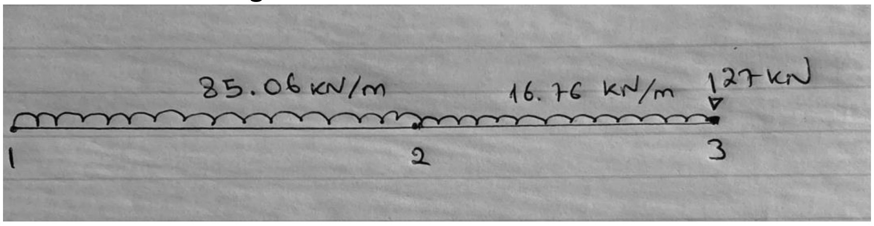
w= 42.53 kN/m

a.2 Triangular load

$$w = \frac{nlx}{3}$$

w= 32.6 kN/m

- Beam 1-2-3 on grid line A and E



- Beam on grid 1 and 2

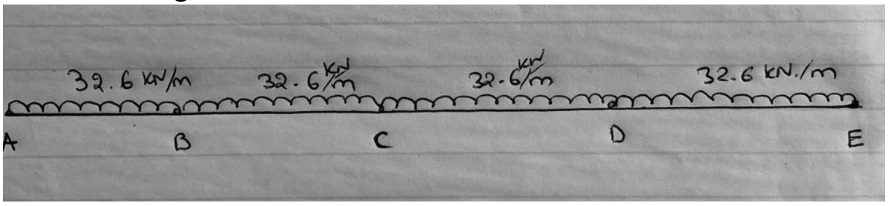
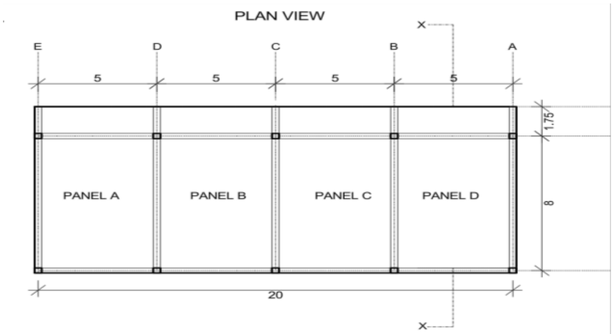


Table 15 (SABS 0100-1)	**Bending moment coefficients for rectangular panels**



The nominal self-weight and live load are the same on adjacent panels, and the spans of adjacent panels are the same.

$\frac{L_y}{L_x} = \frac{8\ m}{5\ m} = 1.6$

In Table 15, $\frac{L_y}{L_x} = 1.6$ is not available, so interpolation will be applied between 1.5 and 1.75 to calculate the bending moment coefficients.

Table 15
(SABS 0100-1)

- **Panel A and D**

The two panels will have the same configuration as they are symmetrical.
Panel A has three discontinuous edges with one long edge continuous hence, Case 7 will be used
Interpolation:

1.5	**1.6**	1.7
0.084	**0.0872**	0.092
0.063	**0.065**	0.069

❖ Short span lx direction

Clause 4.4.4.2
(SABS 0100-1)

Sagging (+) moment in mid span
β_{sx} = 0.065
$m_{sx.a1} = \beta_{sx}.n.l_x^{\ 2}$ = 0.065 × 19.53x (8)²
= 81.24 kN·m

Hogging (-) moment at support
β_{sx} = - 0.0872
$m_{sx.a.2} = \beta_{sx}.n.l_x^{\ 2}$ = -0.0872 × 19.53 × 5 × 8²
= -108.99 ≈ 147 kN·m

❖ Long span ly direction

Sagging (+) moment in mid span
β_{sy} = 0.0654
$m_{sy.a1} = \beta_{sy}.n.l_y^{\ 2}$ = 0.065 × 19.53x (5)²
= 31.73 kN·m

Hogging (-) moment at support
β_{sy} = - 0.0872
$m_{sy.a.2} = \beta_{sy}.n.l_y^{\ 2}$ = -0.0872 × 19.53 × 5 × 5²
= -42.47 kN·m

- **Panel B and C**

The two panels will have the same configuration as they are symmetrical.
Panel B has two short edges that are discontinuous, so case 5 will be used.
Interpolation:

1.5	**1.6**	1.7
0.062	**0.064**	0.067
0.045	**0.045**	0.047

❖ Short span lx direction

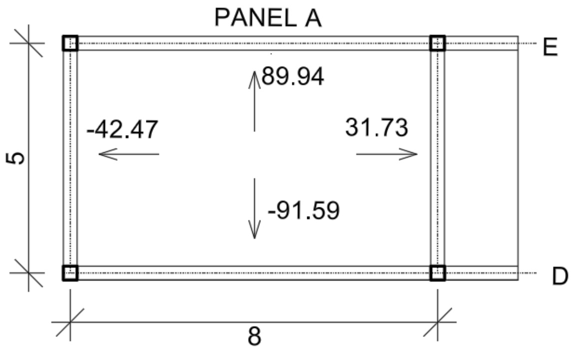
Sagging (+) moment in mid span
β_{sx} = 0.045
$m_{sx.b1} = \beta_{sx}.n.l_x^{\ 2}$ = 0.045 × 19.53x (8)²
= 56.25kN·m

SANS 10100-1 Clause 4.4.4.2.3	Hogging (-) moment at support β_{sx} = - 0.0872 $m_{sx.b2} = \beta_{sx}.n.l_x^2$ = -0.064 × 19.53 × 8² = -77.99 kN·m ❖ Long span ly direction Sagging (+) moment in mid span β_{sy} = 0.045 $m_{sy.b1} = \beta_{sy}.n.l_y^2$ = 0.045 × 19.53x (5)² = 21.97 kN·m Hogging (-) moment at support β_{sy} = - 0.064 $m_{sy.b2} = \beta_{sy}.n.l_y^2$ = -0.0664 × 19.53 × 5² = -31.248 kN·m There is a large difference at the joint support moment of panels A and B, hence some adjustments are required: a) $\Sigma\|M\| = m_{xa} + \frac{m_{xa}a_2}{2} = 31.24 + \frac{102.93}{2} = 135.735$ kN\m b) Fixed end moment Panel A: $m_{I1a} = -102.33kN/m$ Panel B: $m_{I1b} = -49.32kN/m$ c) Moment Redistribution at Support $D_a = \frac{3}{2+3} = 0.6, D_b = 0.4$ d) Unbalanced Moment: $M_o = -m_{Ia} - m_{Ib} = -108.99 - (-79.99) = -29kN/m$ e) Moment Distribution: $m_{xa,a2} = -102.99 - (0.6)(-53) = -91.59$ kN\m $m_{xb,A} = -49.33 + (0.4)(-53) = -91.59$ kN\m

f) Adjusted Midspan Moment for Panel A

$$m_{sx,a2} = \left(135.735 - \left|\frac{(-91.59)}{2}\right|\right) = 135.735 - \frac{91.59}{2}$$

$$= 89.94\mathrm{kN/m}$$



- Going from the loading obtained , a moment distribution was done for the columns, that helped in determining the worst case scenario to use for the design. Below is displayed the summarised moment distribution on the column, the details calculations are attached under the appendix D

Joint	K – per Member	$\sum k$	Distribution factor
A (Fixed)	AB: 0.8EI	---	$D_{AB} = 0$
B	AB: 0.8EI, BC:0.8EI	1.6EI	$D_{AB} = 0{,}5$ $D_{BC} = 0.5$
C	BC:0.8EI, CD: 0.8EI	1.6EI	$D_{CB} = 0{,}5$ $D_{CD} = 0.5$
D	CD: 0.8EI, DE:0.8EI	1.6EI	$D_{DC} = 0{,}5$ $D_{DE} = 0.5$
E (Fixed)	DE: 0.8EI	--	$D_{ED} = 0$

Joint	A	B		C		D		E
DF	0	0,5	0,5	0,5	0,5	0,5	0,5	0
FEM	-67,92	-67,92	-67,92	-67,92	-67,92	-67,92	-67,92	-67,92
Balance		67,92	67,92	67,92	67,92	67,92	67,92	
C,O	33,96		33,96	33,96	33,96	33,96		33,96
Bal.		-16,98	-16,98	-33,96	-33,96	-16,98	-16,98	
C.O	-8,49		-16,98	-8,49	-8,49	-16,98		-8,49
Bal.		8,49	8,49	8,49	8,49	8,49	8,49	
C.O	4,245		4,245	4,245	4,245	4,245		4,245
Bal.		-2,122	-2,122	-4,245	-4,245	-2,122	-2,122	
C.O	-1,061		-2,122	-1,061	-1,061	-2,122		-1,061
Bal.		1,061	1,061	1,061	1,061	1,061	1,061	
C.O	0,531		0,531	0,531	0,531	0,531		0,531
Bal.		-0,266	-0,266	-0,531	-0,531	-0,266	-0,266	
C.O	-0,133		-0,266	-0,138	-0,138	-0,266		-0,133
Bal		0,133	0,133	0,133	0,133	0,133	0,133	
	-38,868	-9,684	9,684	0	0	9,684	-9,684	-38,868

	A			B				C				D				E	
	Col.∑M	AB		BA	Col.∑M	BC		CB	Col.∑M	CD		DC	Col.∑M	DE		ED	Col.∑M
DF	0,233	0,767		0,434	0,132	0,434		0,434	0,132	0,434		0,434	0,132	0,434		0,767	0,233
Load (kN)			163				163				163				163		
FEM		-67,92		-67,92		-67,92		-67,92		-67,92		-67,92		-67,92		-67,92	
Balance	15,825	52,095		58,953	17,931	58,953		58,955	17,931	58,955		58,953	17,931	58,953		52,095	15,825
C,O		26,048		26,048		26,048		26,078		26,078		26,048		26,048		26,048	
Bal.	-6,069	-19,979		-22,609	-6,877	-22,609		-22,609	-6,877	-22,609		-22,609	-6,877	-22,609		-19,979	-6,069
C.O		-11,305		-9,989		-11,305		-11,305		-11,305		-11,305		-9,989		-11,305	
Bal.	2,634	8,671		9,242	2,811	9,242		9,813	2,985	9,813		9,242	2,811	9,242		8,671	2,634
C.O		4,621		4,336		4,907		4,621		4,621		4,907		4,336		4,621	
Bal.	-1,077	-3,544		-4,011	-1,22	-4,011		-4,09	-1,244	-4,09		-4,011	-1,22	-4,011		-3,544	-1,077
C.O		-2,006		-1,772		-2,045		-2,006		-2,006		-2,045		-1,772		-2,006	
Bal.	0,467	1,539		1,657	0,504	1,657		1,741	0,529	1,741		1,657	0,504	1,657		1,539	0,467
M (kNm)	**11,78**	**-11,78**		**-6,065**	**13,149**	**-7,083**		**-6,754**	**13,324**	**-6,754**		**-7,083**	**13,149**	**-6,065**		**-11,78**	**11,78**

Table 5: Column moment distribution

- Moment in each column is given by:

$$M_{col} = \sum M_{col} \times \frac{k_{col}}{\sum k_{col}}$$

$$M_{AK} = M_{EO} = 11.78 \times \frac{1.776 \times 10^{-4}}{3.276 \times 10^{-4}} = -6.386 kNm$$

$$M_{AF} = M_{EJ} = 11.78 \times \frac{1.5 \times 10^{-4}}{3.276 \times 10^{-4}} = 5.39 kNm$$

$$M_{BL} = M_{DN} = 13.149 \times \frac{1.776 \times 10^{-4}}{3.276 \times 10^{-4}} = 7.128 kNm$$

$$M_{BG} = M_{DI} = 13.149 \times \frac{1.776 \times 10^{-4}}{3.276 \times 10^{-4}} = 6.02kNm$$

3.3 Wind Actions (SANS 10160-3)

Wind loads were evaluated using the following site-specific parameters:

- **Location:** Geduld, Ekurhuleni, Gauteng
- **Terrain Category:** C
- **Elevation:** 1600 m
- **Building Height:** 12.1 m
- **Basic Wind Speed:** vb,0 = 36 m/s
- **Design Working Life:** 50 years

Reference	Calculations	Outcomes
SANS 10160-3 Sans 10160-1 (Annexure A) Sans 10160-1 (Table 1)	**<u>Load Calculations</u>** **<u>Wind loading</u>** **Given:** • Location : Geduld in the City of Ekurhuleni, Gauteng Province. • Terrain Category C • Elevation : 1600 meters • Height: 12.1 m • wind speed : 36 m/s • Design working life: 50 years	
7 7.22 7.2.3	• **Wind speed and wind pressure** $V_b = C_{prob} \times V_{b,o}$ $c_{prob} = \left[\frac{1 - K \times \ln\{-\ln(1-p)\}}{1 - K \times \ln\{-\ln 0,98\}}\right]^n$ K= 0.2 $P=\frac{1}{50}$ n= 0.5 Therefore, C_{prob} = 1 V_b = 1x 36 m/s = 36 m/s	V_b =36 m/s
7.3 7.3.1.1	• **Peak wind speed** Vp(Z) = Cr(Z) x Co(Z) x Vb,peak Vb,peak = 1.0 x Vb = 1.0 x 36 m/s = 36 m/s	

<table>
<tr>
<td>7.3.2.1
Table 1</td>
<td>$c_r(z) = 1,36\left(\frac{z - z_o}{z_g - z_o}\right)^{\alpha}$
Cr(z) = 1.0

At terrain category C:
Zg =350 Z_o =3   Zc =5   a = 0.120
At z = 12.1 m
Cr =1.36 ((12.1-3)/(350-3))0.120 = 0.88
Vp(z) = 0.88 x 1.0 x 36 = 31.68 m/s</td>
<td>Cr(z) = 1.0

Vp(z)= 31.68 m/s</td>
</tr>
<tr>
<td>7.4

8
8.10
8.10.1

8.10.1.1

8.3.9.6</td>
<td>

- **Peak wind speed pressure**

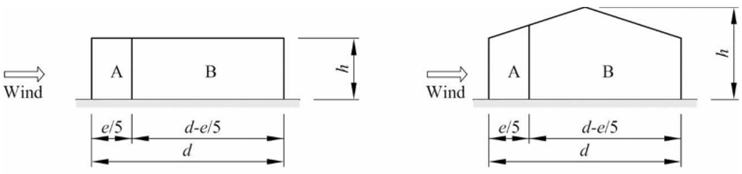
$q_p(z) = \frac{1}{2}$ x p x $v_p^2(z)$

At Geduld in the City of Ekurhuleni, Gauteng Province, the site altitude above sea level is 1600m.

Using interpolation, the air density (p) = 0.988 kg/m³

At z = 12.1 m
$V_p(z)$ = 31.68 m/s
$q_p(z) = \frac{1}{2}\ x\ 0.988\ x\ 31.68^2 = 495.789$ N/m²

External wind pressure
b=20.3m
h=12.1m
e=b or 2h the lesser
e=20.3 m or e= 24.2m
e=20.3 m
d=9.9m
e>d



c) Elevation for e ≥ d

Area = 12.1x20.3=245.63m²
Bxh >10 m² use Cpe

Area of openings = 30% Area of wall
= 30% of 245.63 m²
=73.69 m²
$h/d = \frac{12.1}{9.9} = 1.22$

<table>
<tr><td>Zone</td><td>Cpe_{10}</td></tr>
<tr><td>A</td><td>-1.2</td></tr>
<tr><td>B</td><td>-0.8</td></tr>
</table>

- Internal pressure

</td>
<td>$q_p(z) =$
495.789 N/m²</td>
</tr>
</table>

7.5.2.3 7.5.2.4	With reference to clause 8.3.9.6, the building has no dominant wall, hence Cpi is determined using Figure 16. Area of openings = 30% Area of wall = 30% of 245.63 m^2 =73.69 m^2 $\mu = \frac{sum\ of\ openings\ where\ cpe\ is\ negative\ or\ 0}{sum\ of\ openings}$ $\mu = 1$ h/d = $\frac{12.1}{9.9} = 1.22$ from the graph, Cpi = -0.5 • Internal Wind pressure Wi = qp(z) x Cpi = 495.789 x (-0.5) = -247.89 N/m^2 • External Wind pressure We = qp(z) x Cpe For Zone A We= 495.789 x (-1.2) = -594.95 N/m^2 For Zone B We= 495.789 x (-0.8) = -396.62 N/m^2	Wi = -247.89 N/m^2 We= -594.95 N/m^2 We = -396.62 N/m^2
7.5.3 7.5.3.5 **Figure 25** **Table 22**	• **Calculation of wind force** The wind force Fw is given by : $F_w = c_s \times c_d \times c_f \times q_p(z_e) \times A_{ref}$ ○ Force Coefficient Calculation $C_f = C_{f0} \times \psi_m \times \psi_r$ $\frac{L}{b} = \frac{20.3}{9.9} = 2.05$ $\frac{d}{b} = \frac{10.3}{9.9} = 1.04$ $C_{f0} = 1.65$ $\psi_r = 1$(Reduction factor for sharp corners – Figure 26) $\frac{r}{b} = 0$ **Solidity Ratio:** $\frac{A}{A_c} = 1$(Solid rectangular wall, Cl. 8.13.3) **Effective Slenderness** $L \geq 50m, \quad \boldsymbol{\lambda} = \frac{1.4x20.3}{9.9} = 2.87 < 70$	

Figure 38	$L = 15$ m, $\lambda = \dfrac{2x20.3}{9.9} = 4.10 < 70$ **Interpolation:** **L (m)** **λ (Slenderness)** 50 2.87 20.3 3.914 15 4.10 Thus, $\lambda = 3.914$ $\psi_m = 0.651$	
	Final Calculation $C_f = C_{f0} \times \psi_r \times \psi_m$ $C_f = (1.65)(1)(0.65) = 1.074$	
8.7.2 Equation 22	**Reference Area Calculation** $A_{ref} = L \times b$ $A_{ref} = (20.3 \times 9.9) = 200.97\ m^2$ The wind force is calculated as follows: Fw=1x1x495.789 N/m^2 x200.97 Fw= 99638.715N =99.638 kN	

- A 3D model was developed to analyse the effect of the combined actions of wind and gravity load on the building. When modelling on prokon, the required input use for hand calculations was applied on the model to derive storey shear forces as shown under figure 3,4, and 5. The complete prokon analysis of the building is attached under the appendix C.

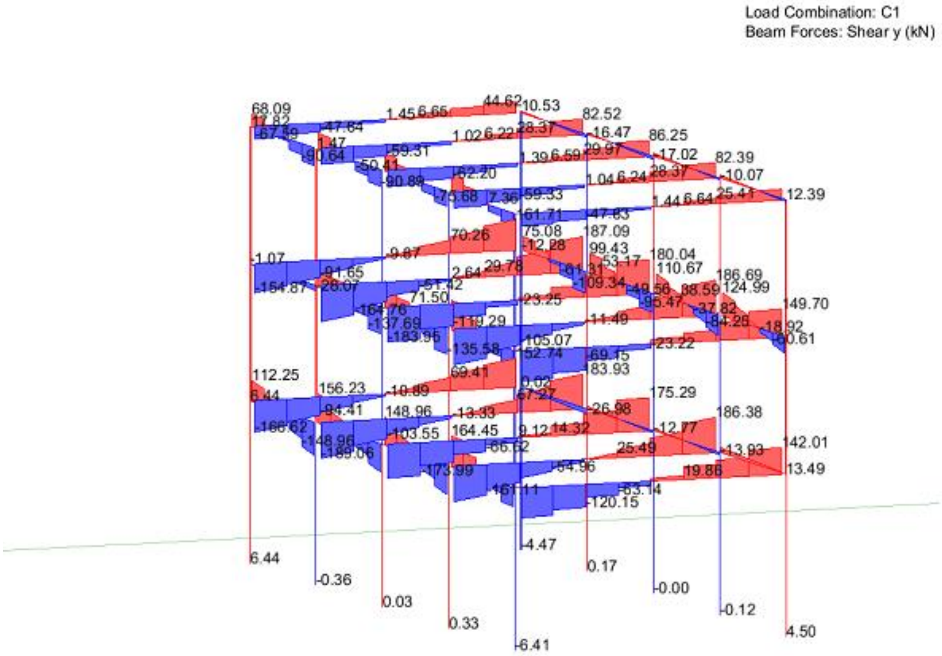


Figure 3: Storey shear force from wind loading

X-Moments: Load Case Load Combination C1

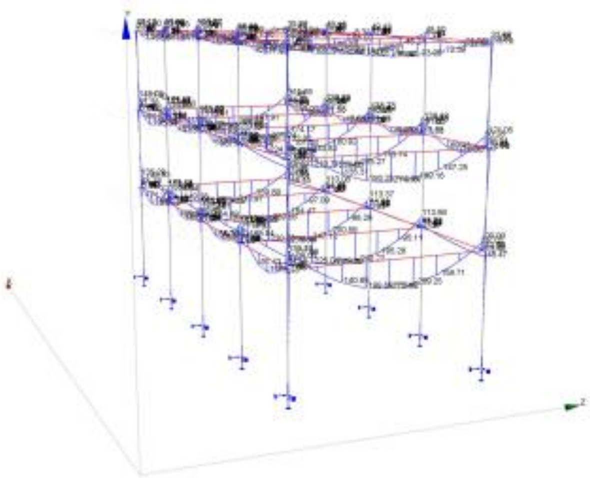


Figure 4: Moments diagrams under wind loading

Axial Forces: Load Case Load Combination C1

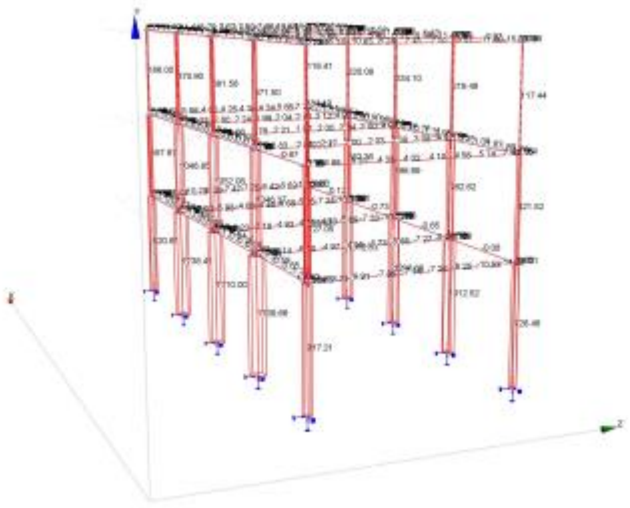


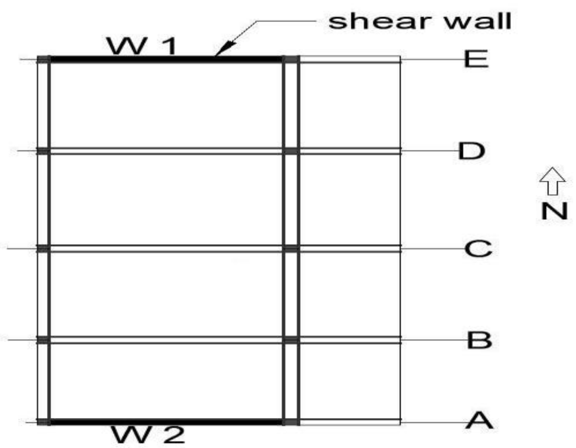
Figure 5: Axial forces diagram under wind loading

3.4 Seismic Actions (SANS 10160-4)

Although Gauteng is a region of low seismic activity, It was required to evaluate the potential effect on the structure. The analysis was based on the Equivalent Lateral Force (ELF) method because the building Satisfy the conditions stipulated under clause 8.4.3 . The system chosen was a moment resisting frame with two shear walls added on the perimeter to improve the building's lateral strength and stability. They help the structure resist earthquake forces and reduce movement or drift in the frame.

Reference	Calculations	Answer
Tutorial -2	**<u>SELF-WEIGHTS</u>** **Slab:** $W = \gamma_{concrete.} t. A_{slab}$, L = 8m W = 20m $= 24 \times 0.25 \times (8 \times 20)$, $\gamma_{concrete.} = 24kN/m^3$ $= 960kN$ **Screed:** $W = \gamma_{screed.} t. A_{slab}$ $= 23 \times (0.05) \times (8 \times 20)$ $= 184\ kN$ **Services:** 0.15kPa $W = 0.15 \times (8 \times 20)$ $= 24kN$ **<u>Masonry Walls:</u>**	

	(1) Gridline A-E $W = 5kN/m^2\ (7.7 \times 3.2)(5no.)$ $= 616\ kN$ **(2) Gridline 1 & 2:** $W = 5kN/m^2(4.7 \times 3.2)(8no.)$ $= 601.6kN$ **Balustrade:** $W = 5kN/m^2(1.1)(0.23)$ $= 1.265kN$ **Balcony:** $W_{slab} = \gamma_{concrete.}t.A_{slab}$ $= 210kN$ **TOTAL (G_n):** *Slab + Balcony + Masonry Walls+ Screed +Services* $G_n = \mathbf{2595.6\ kN}$	$\mathbf{2595.6\ kN}$
SANS 10160-1, Table 2	**IMPOSED LOADS** **Classroom** , C1 - $q_k = 3kN/m^2$ $Q_1 = 3(8 \times 20)$ $= 480kN$ **Balcony** $Q_2 = 5\ (1.75 \times 20)$ $= 175\ kN$ **TOTAL (Q):** *Classroom + Balcony* $Q = \mathbf{655}\ kN$	$\mathbf{655\ kN}$
SANS10160-1, Table 2 SANS10160-4,CL 8.3	$Combination\ Factor,\ \psi_i = 0.3$ $\boldsymbol{W_{n1} = G_n + \psi_i Q_n}$ $= 2595.6 + 0.3(655)$ $= 2792.1kN\ \times 2no$ $= \mathbf{5584.2kN}$	$\mathbf{5584.2kN}$
	ROOF **Self-weight** $G_n = (24)(9.75 \times 20)$ $= 1170kN$ Screed-roof: 184 kN	

	Services: 29.25 kN Screed-Balcony: 40.25 kN **Sub-total = 1423.5 kN**	
SANS10160-2, Table 5	**Imposed load** Category J, $q_k = 2\ kN/m^2$ $Q = 2(9.75 \times 20)$ $= 390\ kN$ $\mathbf{W_{n2} = G_n + \psi_i Q_n}$ $= 1423.5 + 0.3(390)$ $= 1540.5\ \text{kN}$ $\boldsymbol{W_{n-TOTAL} = 5584.2\ \text{kN} + 1540.5\ \text{kN}}$ $\mathbf{= 7124.7\ kN}$	
	$a_g = 0.1$	W_n = **7124.7 kN**
SANS10160-4, Figure 1	q = 5 (With reinforced concrete shear walls)	
SANS10160-4, Table 4	Ground type = 4	
SANS10160-4, Table 1 SANS10160-4, Table 2	S = 1.35 $T_B = 0.2$ $T_c = 0.8$ $T_D = 2$	
	 *Thickness of the RC shear wall = 270mm*	

SANS10160-4, Cl 8.5.2.2 8.5.2.1 SANS10160-4, Cl 5.3.4 SANS10160-4, Cl 8.5.1 SANS10160-4, Cl 8.5.3	(see below)	**287.125kN**

Wall	L_{w_i} (m)	b_w (m)	A_i (m^2)	h_t (m)	L_{wi}/h_t
W_1	7.7	0.27	2.079	12.1	0.636
W_2	7.7	0.27	2.079	12.1	0.636
			4.158		**1.272**

$$A_c \sum A_i \left[0.2 + \left(\frac{L_{wi}}{h_t}\right)^2\right] = 7.559m^2$$

$C_T = 0075\sqrt{A_c} = 0.20620$
$T = 1.34\ sec$

$T_c \leq T \leq T_D \qquad S_d(T)\left\{= a_g \times S_q^{2.5}\left[\frac{T_c}{T}\right]\right\}\ but\ \geq \beta \times a_g$

$$S_d(T) = 0.1 \times 1.35 \times \frac{2.5}{5}\left[\frac{0.8}{1.34}\right]$$
$= 0.0403$

$V_n = S_d(T) \times W$
$= (0.0403 \times 7124.7)$
$= \mathbf{287.125\ kN}$

$$F_{xn} = \frac{W_x \times h_x}{\sum_{i=1}^{n} W_i \times h_i} \times V_n$$

Floor	h (m)	W_e (kN)	W_h (kNm)	F (kN)	Cum. F (kN)
3	12.1	1540.5	18640.05	98.421	98.421
2	8.3	2792.1	23174.43	122.363	220.784
1	4.5	2792.1	12564.45	66.341	287.125
			54378.93		

3.5 Finite Element Modelling & Verification (PROKON)

For the purpose of this project, the software used was Prokon, where the entire 3D building was modelled. The model was analysed as a whole, and using the analysis results, the most critical results were further developed for design. The slabs were modelled as two-way plate elements supported by continuous beams.

The model geometry and element properties were based on the structural drawings and section sizes used in the hand calculations. This work represented Foundation supports were modelled as pinned bases, providing vertical restraint while allowing horizontal reactions to be taken directly from the Prokon output. For the Load Cases and Combinations Load cases modelled in Prokon correspond to gravity, wind (+/–) and seismic (ELF). The results displayed under Appendix C are from the worse combination case.

The following results were extracted:

- Bending moments and shear forces for critical beams
- Slab moment contours (Mx and My)
- Storey shear
- Deflections under service loads

4. Phase 2 – Design & Detailing

4.1Two-Way Slab Design

Using moments obtained from the analysis, the Top and bottom reinforcement were designed for column and middle strips, then the minimum reinforcement and spacing limits of SANS 10100-1 were checked, and Torsion reinforcement was provided at discontinuous corners.

- Input

Materials properties	Value
f_{cu}	30 MPa
f_y	450 MPa
Cover	20mm
L_x	5m
L_y	8m
n	19.53 kN/m^2
h	250mm
bs	300mm

REFERENCES	CALCULATIONS
	I. **PANEL A DESIGN**
(SABS 0100-1) Clause 4.2.4	**Assumptions:** $b = 1$ m $\emptyset x$= 16 mm (main reinforcement) $\emptyset y = 12$ mm (transverse reinforcement) **Check for Reinforcement:** $d_x = h - \text{cover} - \frac{\emptyset x}{2} = 250 - 20 - \frac{16}{2} = 222$ mm $d_y = h - \text{cover} - \emptyset x - \frac{S_x}{2} = 250 - 20 - 12 - \frac{12}{2} = 208$ mm **Assumption:** Redistribution = 10% $K' = 0.156$

Clause 4.3.3.4.1	**A. Short Span Reinforcement** **(i) At Midspan:** $M = 89.94$ kNm $K = \frac{89.94 \times 10^6}{1000 \times 222^2 \times 30} = 0.061$ Since $K < K'$, only tension reinforcement is required. $Z = 222[0.5 + \sqrt{0.25 - \frac{0.061}{0.9}}] = 205.76 \text{ mm} < 0.95d$ ✓ Area of Steel Required $A_s = \frac{89.94 \times 10^6}{0.87 \times 450 \times 205.76} = 1116.50 \text{ mm}^2$ Provide: Y16 @ spacing $Nbars = \frac{1116.50}{pi \times (16)^2 /4} = 6bars$ $\text{Spacing} = \frac{1000}{6} = 166 \text{ mm c/c}$ Provide: Y16 @ 150 mm c/c $A_{s,prov} = \frac{1000}{150} \times \frac{\pi}{4} \times 16^2 = 1340 \text{ mm}^2$ **(ii) At Support:** $M = 91.59$ kN\m $K = \frac{91.59 \times 10^6}{1000 \times 222^2 \times 30} = 0.062$ $Z = 222[0.5 + \sqrt{0.25 - \frac{0.062}{0.5}}] = 205.48 \text{ mm} < 0.95d$ $A_s = \frac{91.59 \times 10^6}{0.87 \times 450 \times 205.48} = 1138.54 \text{ mm}^2$ Provide: Y16 @ 150 mm c/c $A_{s,prov} = 1340 \text{ mm}^2$ **B. Long Span Reinforcement** **(i) At Midspan** $M = 31.73 \text{ kN m}$ $d_g = 208 \text{ mm}$ $K = \frac{31.73 \times 10^6}{1000 \times 208^2 \times 30} = 0.024$

	$$z = 208\left(0.5 + \sqrt{0.25 - \frac{0.024}{0.9}}\right) = 202.30 > 0.95d$$ Hence, use $z = 0.95d = 197.6$ mm $$A_{s,req} = \frac{31.73 \times 10^6}{0.87 \times 450 \times 197.6} = 410.16\ \text{mm}^2$$ **Provide:** Y12 @ 250 mm c/c $$A_{s,prov} = 452\ \text{mm}^2$$ **(ii) At Support** $M = 42.47$ kN m $$K = \frac{42.47 \times 10^6}{1000 \times 208^2 \times 30} = 0.033$$ $$z = 208(0.5 + \sqrt{0.25 - \frac{0.033}{0.9}}) = 200.07\ \text{mm} > 0.95d$$ Hence, use $z = 0.95d = 197.6$ mm $$A_{s,req} = \frac{42.47 \times 10^6}{0.87 \times 450 \times 197.6} = 410.16\ \text{mm}^2$$ **Provide:** Y12 @ 250 mm c/c $$A_{s,prov} = 452\ \text{mm}^2$$
Table 23, Clause 4.11	• **Minimum Reinforcement** For high yield steel, the minimum reinforcement is: $$A_{s,min} = \frac{0.13\ b\ h}{100}$$ $$A_{s,min} = \frac{0.13 \times 1000 \times 250}{100} = 325\ \text{mm}^2/\text{m width}$$ • OK for all reinforcement Provide: Y12 @ 250 mm c/c $A_{s,prov} = 452\ \text{mm}^2/\text{m width}$
Clause 4.11.8.2.2(b)	• **Maximum Spacing of Reinforcement** Short span: $f_y = 450$ MPa, $h = 250$ mm Slab depth < 200 mm ⇒additional check required $$\frac{100\ A_{s,prov}}{b\ d_x} = \frac{100 \times 1340}{1000 \times 222} = 0.6 < 3\ ✓$$ the maximum spacing should not exceed the lesser of

Claude 4.11.8.2.2	$3x(222) = 196$ mm or 180 mm max spacing $< 3x222$ **C. Torsion Reinforcement** (i) At the corner $$\frac{3}{4}A_{sx} = \frac{3}{4} \times 1340 = 1005 \text{ mm}^2$$ Provide Y16 @ 200 mm c/c **(iii)** Along edge: $$\frac{2}{8}A_{sx} = 502.5 \text{ mm}^2$$ Provide Y12 @ 225 mm c/c *(Placed in corners of 0.2Lx = 1 m)* **D. Reinforcement at Discontinuous Ends:** (i) Short span: $$\frac{1}{2} \times 1340 = 670 \text{ mm}^2 \Rightarrow \text{Provide Y16 @ 300 mm c/c}$$ (ii) Long span: $$\frac{1}{2} \times 452 = 325 \text{ mm}^2 < A_{s,min}$$ Use $A_s = 325$ mm^2 ⇒ Provide Y12 @ 325 mm c/c E. **Span effective depth ratio , short span deflection** $$\frac{M_{s_x} \cdot a}{b\, dx^2} = \frac{89.94}{1000x222^2} = 1.85 \times 10^2 \text{ kN/mm}^2 \approx 1.82 \text{ MPa}$$ • Modification factor $$f_s = 0.87fy \times \frac{\gamma_1 + \gamma_2}{\gamma_3 + \gamma_4} \times \frac{A_{sreq}}{A_{s,prov}} \times \frac{1}{B_b}$$ $\gamma_1 = 1.0$ (SLS), $\gamma_2 = 1.0$(SLS) $\gamma_3 = 1.2$ (ULS) $\gamma_4 = 1.6$(ULS) $A_{s,req} = 1116.45, A_{s,prov} = 1340$ $$f_s = 0.87(450) + \frac{(1+1)}{(1.2+1.6)} \times \frac{1116.45}{1340} = 232 \text{ MPa}$$
Clause 4.3.6.3	$$F_1 = 0.55 + \frac{(477-232)}{120(0.97+0.9)} = 1.3 < 2.0 \text{ OK ✓}$$ $$\frac{lx}{dx} = \frac{5000}{222} = 22.52 < 24F1 \text{ OK}$$

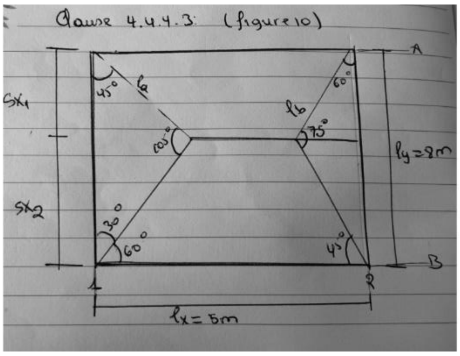
E. Load Distribution and Shear

- **Shear Check**

$$v = \frac{19.53 \times 5}{2} = 48.825 \text{ kN}$$
$$v = \frac{48.825 \times 10^3}{1000 \times 222} = 0.219 \text{ MPa}$$
$$V_c = 0.75(\frac{30}{25})^{1/3}(\frac{100 \times 1340}{1000 \times 222})^{1/3}(\frac{400}{222})^{1/4}\frac{1}{1.4}$$
$$V_c = 0.55 \text{ MPa}$$

$$\therefore \;\; \boldsymbol{v < V_c} \Rightarrow \textbf{OK}$$

Load Distribution on Beam:



Short Span:

$$l_a = \frac{sin30x8}{sin105} = 4.141 \text{ m}$$
$$S_{x1} = 4.141x\ sin45 = 2.928 \text{ m}, S_{x2} = 5.072\text{m}$$

Long Span:

$$l_b = \frac{sin45x8}{sin75} = 5.856 \text{ m}$$
$$S_{y2} = 5.071 \text{ m}, S_{y2} = 2.928 \text{ m},$$

Shear on beam along Grid line A on short span

$$V_a = 19.53 \times 2.928 = 57.18 \text{ kN/m}$$
$$\frac{V_a}{d_x} = \frac{57.18}{222} = 0.258 \text{ MPa} < V_c \text{ OK} \checkmark$$

Shear on beam along Grid line B on short span

	$$\frac{V_b}{d_x} = \frac{99.056}{222} = 0.446 \text{ OK} ✓$$ Shear at Beam along Long Span Grid 1: $$V_1 = 19.53 \times 2.928 = 57.18 kN/m$$ $$\frac{V_1}{d_x} = \frac{57.18}{208} = 0.275 < V_c \text{ OK} ✓$$ Grid 2: $$V_2 = \frac{19.53 \times 5.072}{208} = 0.476 < 0.55 \text{ OK} ✓$$

- Following above design, a bar bending schedule was generated and is as displayed under the appendix B

4.2 Beam Design (Gridlines A–E and 1–2)

The structural design process of the beam covered several key aspects to ensure both safety and efficiency. Flexural design at mid-span and supports was carried out under ultimate limit state (ULS) conditions, while shear design followed the provisions outlined in SANS 10100-1. Compression reinforcement was added where necessary to enhance structural performance. Moment redistribution was reviewed and verified in cases where it was applied. In addition, anchorage lengths, lap lengths, and bar curtailments were carefully checked to meet code requirements. Finally, detailed reinforcement layouts and bar schedules were developed to provide clear guidance for construction and compliance with design standards.

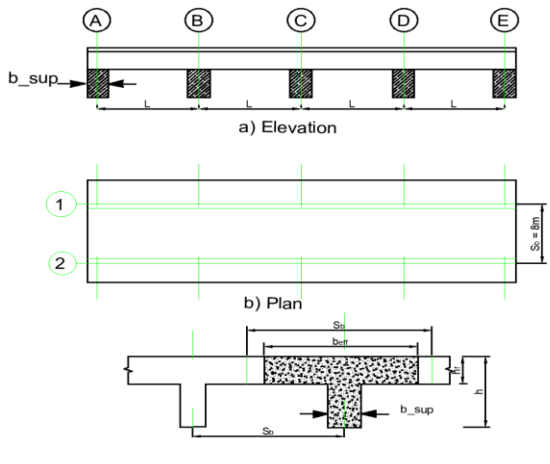


Figure 6 : Beam plan and elevation

- **Input:**

Effective span length	L	5.0 m
Spacing of beams	s_b	8.0 m
Width of supports	b_{sup}	300 mm

Cross section dimensions:		
Width of web	b_w	300 mm
Total height	h	600 mm
Height of slab	h_f	250 mm
Loading:		
Nominal live load	q_n	3.0 kN/m^2 (class rooms)
Superimposed dead load	g_{sdl}	0.15 kN/m^2 + 5 kN/m^2 = 5.15 kN/m^2 (services + walls)
The use of the structure requires a fire resistance of 1.5 hours (typically used for buildings between 5m to 18m high)		
Assuming moderate exposure conditions		
Concrete:		
Characteristic cube strength	f_{cu}	25 MPa
Ultimate strain in concrete	ε_{cu}	0.0035
Unit weight of concrete	γ_c	24 kN/m^3
Unit weight of screed	γ_{screed}	23 kN/m^3
Maximum aggregate size	h_{agg}	26.5 mm (For deep suspended slabs)
Reinforcing steel:		
Yield strength in tension	f_y	450 MPa (hot-rolled high-yield steel from SABS 0100-1 Table 3)
Yield strength in shear	f_{yv}	450 MPa
Modulus of elasticity	E_s	200 GPa (SABS 0100-1 clause 3.4.2.3)
Material safety factor for shear	γ_{mv}	1.4 (SABS 0100-1 clause 3.3.3.2.c)

References	Calculations
Robberts and Marshall Textbook	**<u>Concrete cover</u>**
	Fire resistance For fire resistance, for siliceous aggregates:
Table 10.1	Minimum with of section = 140 mm , therefore b_w = 300 mm **OK** Minimum required cover to main reinforcing bars is 35 mm For a minimum possible link size of 8mm, cover should be greater than, 35 mm – 8 mm = 27 mm.

Table 9.1	**Exposure conditions** For moderate exposure conditions: Cover = 30 mm > h_{agg} = 26.5 mm **OK**
	<u>Design loads: ULS</u> Live load on beam: $Q_n = q_n \times s_b$ = 3.0 kN/m^2 x 8.0 m = 24 kN/m Self-weight of beam and slab : G_{self} = [0.35 m x 0.3 m + 0.25m x 8.0 m] 24 kN/m^3 = 50.52 kN/m Screed : G_{screed} = 0.05 m x 23 kN/m^3 x 8.0 m = 9.2 kN/m Superimposed DL: $G_{sdl} = g_{sdl} \times s_b$ = 5.15 kN/m^2 x 8.0 m = 41.2 kN/m Total self-weight: $G_n = G_{self} + G_{screed} + G_{sdl}$ = 100.92 kN/m Design load: w_u = 1.2 G_n + 1.6 Q_n = 159.50 kN/m
	<u>Bending moments and shear forces</u> Checking if simplified analysis method may be used: (a) $\frac{Q_n}{G_n} = \frac{24}{100.92}$ = 0.24 < 1.25 (b) The loads are mainly uniformly distributed. (c) There are 4 spans. (d) The spans are equal. Therefore, the simplified method in Table 4 may be used: Total load on span: F = w_u x L = 159.50 kN/m x 5.0 m = 792.52 kN
SABS 01001-1 Table 4	Bending moments: Near mid-end span @F, $M_F = \frac{FL}{11} = \frac{792.52\ kN\ \times 5\ m}{11}$ = 360.2 kNm First interior support @B, $M_B = \frac{-FL}{9} = \frac{-\ 792.52\ kN\ \times 5\ m}{9}$ = - 440.3 kNm Interior support @C, $M_c = \frac{-FL}{12} = \frac{-\ 792.52\ kN\ \times 5\ m}{12}$ = - 330.2 kNm Shear forces: V_A = 0.45 F = 0.45 x 792.52 kN = 356.6 kN V_B = 0.6 F = 0.6 x 792.52 kN = 475.5 kN V_C = 0.55 F = 0.55 x 792.52 kN = 435.9 kN
	<u>Design for Flexure</u> **Midspan (at F)** Tension bar size: Ø = 25 mm

	Link size: $\emptyset_v$ = 10 mm Effective depth: $d_F = h - cover - \emptyset_v - \frac{\emptyset}{2}$ $= 600 - 30 - 10 - \frac{25}{2} = 547.5$ mm
4.3.1.5	Effective flange width: $b_{eff} = b_w + \frac{0.7L}{5} = 0.3 + \frac{0.7\times5}{5} = 1$m $b_{eff} = 1\text{m} < s_b = 8.0$ m **OK** Finding reinforcement, assuming $s < h_f$: At midspan, assume $\beta_b = 1.0$
4.3.3.4.1	$K' = 0.402\,(\beta_b - 0.4) - 0.18\,(\beta_b - 0.4)^2$ $= 0.402\,(1- 0.4) - 0.18\,(1- 0.4)^2 = 0.176$ K' = 0.176 > 0.156, therefore use K' = 0.156 $K = \frac{M_F}{b_{eff}\times d_F{}^2 \times f_{cu}} = \frac{360.2}{1\times(0.5475)^2 x\,(25\,x10^3)} = 0.048$ K = 0.048 < K' = 0.156, compression reinforcement is not required. $z = d_F\left(0.5+\sqrt{0.25-\frac{K}{0.9}}\right) = d_F\left(0.5+\sqrt{0.25-\frac{0.048}{0.9}}\right) = 0.943\,d_F$ $z = 0.943\,d_F <$ z= 0.95d, therefore use $z = 0.943\,d_F$. $z = 0.943\,d_F = 0.943$ x 547.5 mm = 516.3 mm $s = 2(d_F - z)$ = 2(547.5 mm - 516.3 mm) = 62.4 mm < h_f = 250 mm **OK** $A_{s.req.F} = \frac{M_F}{0.87\times f_y \times z} = \frac{360.2\times10^6}{0.87\times450\times516.3} = 1782\ mm^2$ **Therefore, Provide 3Y25 and 1Y20** $A_{s.prov.F} = 1473\ mm^2 + 314.2\ mm^2 = 1787\ mm^2$ The actual depth calculated to the centroid of the reinforcement will be slightly greater than assumed, OK.
Table 23	Minimum reinforcement: $\frac{b_w}{b_{eff}} = \frac{300}{1000} = 0.3 < 0.4$ Therefore, $\frac{100\times A_{s.prov.F}}{b_w h} = \frac{100\times 1787}{300\times600} = 0.99 > 0.18$ **OK** Maximum area of reinforcement = 4%, **OK**

	Maximum horizontal spacing of bars: $\frac{1}{2}[b_w - 2cover - 2\emptyset_v - (3 \times 25\ mm) - 20\ mm\ ]$= 62.5 mm 62.5 mm < h_{agg} + 5 mm = 31.5 mm **OK** Maximum spacing between bars (assuming 0% redistribution): s_{max} = 170 mm **OK**
Table 25	
	Support B Tension bar size: Ø = 25 mm Link size: $\emptyset_v$ = 10 mm Assuming that the tension bars will probably be placed in two layers to fit into the web. Effective depth: d_B = h – cover - $\emptyset_v$ - Ø = 600 – 30 – 10 – 25 = 535 mm
4.3.1.5	Finding reinforcement, noting that b = b_w : At the support, assume β_b = 0.9
4.3.3.4.1	K'= 0.402 (β_b - 0.4) – 0.18 (β_b – 0.4)^2 = 0.402 (0.9- 0.4) – 0.18 (0.9– 0.4)^2 = 0.156 K = $\frac{M_B}{b_{w\times} d_B{}^2 \times f_{cu}}$ = $\frac{440.3}{0.3 \times (0.535)^2 \text{x}\ (25\ \text{x}10^3)}$ = 0.205 K = 0.205 > K' = 0.156, compression reinforcement is required. d' = cover + $\emptyset_v + \frac{\emptyset}{2}$ = 30 + 10 + $\frac{25}{2}$ = 52.5 mm
Figure 2	Yield stress for compression reinforcement: $f_{yc} = \frac{f_y}{1.15+\frac{f_y}{2000\ MPa}} = \frac{450\ MPa}{1.15+\frac{450\ MPa}{2000\ MPa}}$ = 327.3 MPa $\frac{d'}{d_B} = \frac{52.5}{535}$ = 0.098 < $(\beta_b - 0.4)(1 - \frac{f_{yc}}{E_{s\times}\ \varepsilon_{cu}})$ = 0.266, the compression will yield at ultimate. Internal lever arm:
4.3.3.4.1	z = $d_B\left(0.5 + \sqrt{0.25 - \frac{K\prime}{0.9}}\right)$= $d_B\left(0.5 + \sqrt{0.25 - \frac{0.156}{0.9}}\right)$ = 0.777 d_B z = 0.777 x 535 mm = 415.7 mm

4.3.3.4.1	Required area of compression reinforcement: $A'_{s.req.B} = \frac{(K-K')f_{cu}b_w {d_B}^2}{f_{yc} \times (d_B - d')} = \frac{(0.205-0.156)\times 25 \times 300 \times (535)^2}{327.3 \times (535-52.5)}$ = 666 mm^2 Required area of tension reinforcement: $A_{s.req.B} = \frac{K' f_{cu} b_w d_B^2}{0.87\, f_y\, z} + \frac{f_{yc}}{0.87 f_y}\ A'_{s.req.B}$ $= \frac{0.156 \times 25 \times 300 \times (535)^2}{0.87 \times 450 \times 415.7} + \frac{327.3}{0.87 \times 450} \times 666$ = 2614 mm^2 For compression reinforcement at the support the tension reinforcement at midspan is extended into the support. **Therefore, Provide 3Y25 and 1Y20** $A'_{s.prov.B}$ = 1787 mm^2 Tension reinforcement: **Provide 4Y25 and 3Y20** $A_{s.prov.B}$ = 1963 mm^2 + 942 mm^2 = 2905 mm^2 The actual depth calculated to the centroid of the reinforcement will be slightly greater than assumed, OK. Minimum reinforcement:
Table 23	Tension: $\frac{100\times A_{s.prov.B}}{b_w h} = \frac{100\times 2905}{300\times 600}$ = 1.61 > 0.26 **OK** Compression: $\frac{100\times A'_{s.prov.B}}{b_w h} = \frac{100\times 1787}{300\times 600}$ = 0.99 > 0.20 **OK** Maximum area of reinforcement = 4%, **OK** Maximum horizontal spacing of bars in top beam: $\frac{1}{2}(b_w - 2cover - 2\emptyset_v - (3 \times 25\ mm) - 20\ mm$ = 62.5 mm 62.5 mm < h_{agg} + 5 mm = 31.5 mm **OK** Care should be taken here to compact the concrete during placing. There is not enough space in the top beam to allow for the standard 75 mm diameter vibrator to pass between reinforcements.
Table 25	Maximum spacing between bars (assuming 10% redistribution): s_{max} = 155 mm **OK** The reinforcement at support B can be curtailed: Anchorage curtailment length the greatest of 0.25 L = 1250 mm and 45 Ø = 45 x 25 mm = 1125 mm

4.11.7.2.3 (a)	Say $L_{ac.B}$ = 1350 mm 20% of the reinforcement at the support must extend into the span: 20% $A_{s.req.B}$ = 0.2 x 2614 mm^2 = 523 mm^2 Extend 2Y20 (628 mm^2) of the top reinforcement into the span **Support C** Tension bar size: Ø = 25 mm Link size: $\emptyset_v$ = 10 mm Finding reinforcement, assuming s < h_f : Assuming that the tension bars will probably be placed in two layers to fit into the web. Effective depth: d_c = h – cover - $\emptyset_v$ - Ø = 600 – 30 – 10 – 25 = 535 mm Finding reinforcement, noting that b = b_w :
4.3.1.5	At the support, assume β_b = 0.9 K'= 0.156
4.3.3.4.1	K = $\frac{M_C}{b_{w\times} {d_C}^2 \times f_{cu}}$ = $\frac{330.2}{0.3 \times (0.535)^2 \text{x} \ (25 \text{ x} 10^3)}$ = 0.154 K = 0.154 < K' = 0.156, compression reinforcement is not required. z = $d_C \left(0.5 + \sqrt{0.25 - \frac{K}{0.9}}\right)$= $d_C \left(0.5 + \sqrt{0.25 - \frac{0.154}{0.9}}\right)$ = 0.781 d_C z = 0.781 d_C < z= 0.95d, therefore use z = 0.781 d_C. z = 0.781 d_C = 0.781 x 535 mm = 417.8 mm s = 2(d_C - z) = 2(535 mm - 417.8 mm) = 234.4 mm < h_f = 250 mm **OK** $A_{s.req.c}$ = $\frac{M_C}{0.87 \times f_y \times z}$ = $\frac{330.2 \times 10^6}{0.87 \times 450 \times 417.8}$ = 2019 mm^2 **Therefore, Provide 3Y25 and 2Y20** $A_{s.prov.c}$ = 1473 mm^2 + 628 mm^2 = 2101 mm^2 The actual depth calculated to the centroid of the reinforcement will be slightly greater than assumed, OK.

Table 23 Table 25	Minimum reinforcement: $\frac{b_w}{b_{eff}}$ = $\frac{300}{1000}$ = 0.3 < 0.4 Therefore, $\frac{100\times A_{s.prov.c}}{b_w h}$ = $\frac{100\times 2101}{300\times600}$ = 1.17 > 0.18 **OK** Maximum area of reinforcement = 4%, **OK** Maximum horizontal spacing of bars: $\frac{1}{2}[b_w - 2cover - 2\emptyset_v - (3 \times 25\ mm) - (2\ \times 20\ mm)\ ]$= 52.5 mm 52.5 mm < h_{agg} + 5 mm = 31.5 mm **OK** Maximum spacing between bars (assuming 0% redistribution): s_{max} = 170 mm **OK** ***Minimum transverse reinforcement in flange*** To transfer shear in flange: $A_{s.trans}$ = $\frac{0.15}{100}$ h_f (1m) = 375 mm^2 <u>Provide Y10@175 mm c/c</u> $A_{s.trans.prov}$ = 449 mm^2 over a length of b_{eff} = 1000 mm
      4.3.4.1.1 4.11.4.5.3	<u>**Design for Shear**</u> **Maximum shear** Maximum shear force will occur at the face of support B. $V_{sup.B} = V_B - w_u.\frac{b_{sup}}{2}$ = 475.5 – 159.50 x $\frac{0.3}{2}$ = 451.6 kN $v_{max} = \frac{V_{sup.B}}{b_w d_B} = \frac{451.6\times10^3}{300\times535}$ = 2.81 MPa v_{max} = 2.81 Mpa < $0.75\sqrt{f_{cu}}$ = 3.75 MPa and 4.75 MPa, **OK** **Nominal shear reinforcement** Nominal shear reinforcement to be provided for f_{yv} = 450 MPa $\left(\frac{A_{sv}}{s_v}\right)_{min}$ = $0.0012b_w$ = 0.36 $\frac{mm^2}{mm}$ For two legs of a Y10 link $A_{sv.min}$ = 224 mm^2 Maximum spacing is: $0.75d_F$ = 0.75 x 547.5 mm = 410.6 mm Selecting a spacing of $s_{v.min}$ = 350 mm

	$\left(\frac{A_{sv}}{s_v}\right)_{min}$ = $\frac{143\ mm^2}{350\ m}$ = 0.41$\frac{mm^2}{mm}$,**OK** **Provide Y10 links @350 mm c/c.** Assuming that nominal links will always be placed in areas of sagging bending, reinforcement at midspan is considered.
4.3.4.1.2	ρ_F= $\frac{100\times A_{s.prov.F}}{b_w d_F}$ = $\frac{100\times 1787}{300\times 547.5}$ = 1.09 < 3, OK f_{cu} = 25 MPa < 40 MPa, OK Shear resistance of concrete: $v_{c.F} = \frac{0.75\ MPa}{\gamma_{mv}} . \left[\rho_F\ \left(\frac{f_{cu}}{25\ MPa}\right)\right]^{\frac{1}{3}} . \left(\frac{400\ mm}{d_F}\right)^{\frac{1}{4}}$ $= \frac{0.75\ MPa}{1.4} . \left[1.09\left(\frac{25\ MPa}{25\ MPa}\right)\right]^{\frac{1}{3}} . \left(\frac{400\ mm}{547.5\ mm}\right)^{\frac{1}{4}}$ = 0.51 MPa The resistance given by the nominal shear reinforcement is: $v_{s.min} = \frac{A_{sv.min}}{s_{v.min}} . \frac{0.87\ f_{yv}}{b_w} = \frac{224}{350} \times \frac{0.87\times 450}{300}$ = 0.84 MPa $v_n = v_{s.min} + v_{c.F}$ = 1.35 MPa $V_n = v_n\ b_w\ d_F$ = 1.35 MPa x 300 mm x 547.5 mm x 10^{-3} = 221.7 kN **Shear reinforcement at support A** Considering a critical section at a distance d_F = 547.5 mm from the face of the support.
4.3.4.1.2	$V_{sup.A} = V_A - w_u.\frac{b_{sup}}{2}$ = 356.6 – 159.5 x $\frac{0.3}{2}$ = 332.7 kN $V = V_{sup.A} - w_u.\ d_F$ = 332.7 – 159.5 x 0.5475 = 246.4 kN $v = \frac{V}{b_w d_F} = \frac{246.4\times 10^3}{300\times 547.5}$ = 1.5 MPa Tension reinforcement is not curtailed at support A, the full area will be used: $v_{c.F}$ = 0.51 MPa
4.3.4.1.3	Required $\frac{A_{sv}}{s_v} = \frac{(v-v_{c.F}).b_w}{0.87 f_{yv}} = \frac{(1.5-0.51)\times 300}{0.87\times 450}$ = 0.759 $\frac{mm^2}{mm}$ For two legs of a Y10 link A_{sv} = 224 mm^2 Required spacing $s_v = \frac{0.87 f_{yv}.A_{sv}}{(v-v_{c.F}).b_w} = \frac{0.87\times 450\times 224}{(1.5-0.51)\times 300}$ = 295 mm

	Say s_v = 250 mm. <u>Provide Y10 links @250 mm c/c</u> $\frac{A_{sv}}{s_v} = \frac{224\ mm^2}{250\ m}$ = 0.896 $\frac{mm^2}{mm}$,**OK** Maximum spacing is: 0.75d_F = 0.75 x 547.5 mm = 410.6 mm OK Extend this reinforcement the following distance: s = $\frac{V_{sup.A} - V_n}{w_u}$ = $\frac{332.7 - 221.7}{159.5}$ = 696 mm from the face of the support **<u>Shear reinforcement at support B</u>** Considering a critical section at a distance d_B = 535 mm from the face of the support.
4.3.4.1.2	$V_{sup.B} = V_B - w_u.\frac{b_{sup}}{2}$ = 475.5 – 159.5 x $\frac{0.3}{2}$ = 451.6 kN V = $V_{sup.B}$ - w_u. d_B = 451.6 - 159.5 x 0.535 = 366.3 kN v = $\frac{V}{b_w d_B}$ = $\frac{366.3 \times 10^3}{300 \times 535}$ = 2.282 MPa
4.3.4.1.3	Shear resistance of the concrete: $\rho_B = \frac{100\times A_{s.prov.B}}{b_w d_B} = \frac{100\times 2905}{300\times 535}$ = 1.81 < 3, OK All reinforcement in this critical section extend at a distance *d* beyond the critical section. $v_{c.B} = \frac{0.75\ MPa}{\gamma_{mv}} . \left[\rho_B \left(\frac{f_{cu}}{25\ MPa}\right)\right]^{\frac{1}{3}} . \left(\frac{400\ mm}{d_B}\right)^{\frac{1}{4}}$ $= \frac{0.75\ MPa}{1.4} . \left[1.81 \left(\frac{25\ MPa}{25\ MPa}\right)\right]^{\frac{1}{3}} . \left(\frac{400\ mm}{535\ mm}\right)^{\frac{1}{4}}$ = 0.607 MPa Required $\frac{A_{sv}}{s_v} = \frac{(v - v_{c.B}).b_w}{0.87 f_{yv}} = \frac{(2.282-0.607)\times 300}{0.87\times 450}$ = 1.284 $\frac{mm^2}{mm}$ For Y8 link in pairs (4 legs) A_{sv} = 449 mm^2 Required spacing $s_v = \frac{0.87 f_{yv}.A_{sv}}{(v - v_{c.B}).b_w} = \frac{0.87\times 450 \times 449}{(2.282-0.607)\times 300}$ = 350 mm Say s_v = 300 mm.

Provide Y10 links @300 mm c/c $\frac{A_{sv}}{s_v} = \frac{449\ mm^2}{300\ m}$ = 1.497 $\frac{mm^2}{mm}$,**OK**

Maximum spacing is: $0.75d_B$ = 0.75 x 535 mm = 401 mm **OK**

Extend this reinforcement the following distance:
s = $\frac{V_{sup.B} - V_n}{w_u}$ = $\frac{451.6 - 221.7}{159.5}$ = 1441 mm from the face of the support

A more economical alternative would be to consider a further critical section (G) at a distance $L_{ac.B}$ - d_B = 1350 mm – 535 mm = 815 mm.

V = $V_{sup.B}$ - w_u. ($L_{ac.B}$ - d_B) = 451.6 - 159.5 x 0.815 = 321.6 kN
v = $\frac{V}{b_w d_B}$ = $\frac{321.6\ \times 10^3}{300\ \times 535}$ = 2.004 MPa

Shear resistance of the concrete:

$A_{s.G}$ = 628 mm^2 (2Y20 bars)

$$v_{c.G} = \frac{0.75\ MPa}{\gamma_{mv}} \cdot \left[\left(\frac{100.A_{S.G}}{b_w d_B}\right)\left(\frac{f_{cu}}{25\ MPa}\right)\right]^{\frac{1}{3}} \cdot \left(\frac{400\ mm}{d_B}\right)^{\frac{1}{4}}$$

$$= \frac{0.75\ MPa}{1.4} \cdot \left[\left(\frac{100\times 628}{300\times 535}\right)\left(\frac{25\ MPa}{25\ MPa}\right)\right]^{\frac{1}{3}} \cdot \left(\frac{400\ mm}{535\ mm}\right)^{\frac{1}{4}} = 0.364\ \text{MPa}$$

Required $\frac{A_{sv}}{s_v} = \frac{(v - v_{c.G}).b_w}{0.87 f_{yv}} = \frac{(2.004-0.364)\times 300}{0.87\times 450}$ = 1.257 $\frac{mm^2}{mm}$

For 2 legs of a Y10 link in pairs A_{sv} = 224 mm^2

Required spacing $s_v = \frac{0.87 f_{yv}.A_{sv}}{(v - v_{c.G}).b_w} = \frac{0.87\times 450\ \times 224}{(2.004-0.364)\times 300}$ = 178 mm

Say s_v = 175 mm.

Provide Y10 links @100 mm c/c $\frac{A_{sv}}{s_v} = \frac{224\ mm^2}{175\ m}$ = 1.28 $\frac{mm^2}{mm}$,**OK**

Between 1.441 m and ($L_{ac.B}$ - d_B) = 0.815 m from the face of the support.

Shear reinforcement at support C

Considering a critical section at a distance d_C = 535 mm from the face of the support.

$V_{sup.C} = V_C - w_u.\frac{b_{sup}}{2}$ = 435.9 – 159.5 x $\frac{0.3}{2}$ = 412 kN

	$V = V_{sup.C} - w_u . d_C = 412 - 159.5 \times 0.535 = 326.7$ kN $v = \frac{V}{b_w d_C} = \frac{326.7 \times 10^3}{300 \times 535} = 2.036$ MPa Tension reinforcement is not curtailed at support C, the full area will be used: $v_{c.F} = 0.51$ MPa Required $\frac{A_{sv}}{s_v} = \frac{(v - v_{c.F}).b_w}{0.87 f_{yv}} = \frac{(2.036 - 0.51) \times 300}{0.87 \times 450} = 1.17 \frac{mm^2}{mm}$ For two legs of a Y10 link $A_{sv} = 224\ mm^2$ Required spacing $s_v = \frac{0.87 f_{yv}.A_{sv}}{(v - v_{c.F}).b_w} = \frac{0.87 \times 450 \times 224}{(2.036 - 0.51) \times 300} = 192$ mm Say $s_v = 150$ mm. <u>Provide Y10 links @150 mm c/c</u> $\frac{A_{sv}}{s_v} = \frac{224\ mm^2}{150\ m} = 1.49 \frac{mm^2}{mm}$,**OK** Maximum spacing is: $0.75 d_C = 0.75 \times 535$ mm = 401.2 mm OK Extend this reinforcement the following distance: $s = \frac{V_{sup.C} - V_n}{w_u} = \frac{412 - 221.7}{159.5} = 1193$ mm from the face of the support ***Confinement of compression reinforcement*** At support B: Smallest compression bar is $\emptyset_{min} = 20$ mm Maximum spacing of links $s_{v.max} = 12 \times \emptyset_{min} = 240$ mm, OK all links spacings are less Largest compression bar is $\emptyset_{max} = 25$ mm Minimum link size is $\frac{\emptyset_{max}}{4} = 6.3$ mm , OK all links diameter are greater than this
Table 10	<u>**Check Span-Effective Depth Ratio**</u> **Basic L/d ratio** For a beam with one end continuous: $(\frac{L}{d})_{basic} = 24$

	Since the span is less than 10m, the L/d ratio will not be adjusted.
	Modification factor for tension reinforcement
	Assuming β_b =1.0
4.3.6.3.1	$f_s = 0.87\, f_y \,.\left(\frac{1.1+1.0}{1.2+1.6}\right).\left(\frac{A_{s.req.F}}{A_{s.prov.F}}\right).\frac{1}{\beta_b}$
	$= 0.87 \times 450\,.\left(\frac{1.1+1.0}{1.2+1.6}\right).\left(\frac{1782}{1787}\right).\frac{1}{1}$ = 292.8 MPa
Table 11	Modification factor for tension reinforcement: $MF_{AS} = 0.55 + \frac{477\ MPa - f_s}{120.\left(0.9\ MPa + \frac{M_F}{b_{eff}.d_F^2}\right)}$
	$= 0.55 + \frac{477\ MPa - 292.8\ \text{MPa}}{120.\left(0.9\ MPa + \frac{360.2\ \times 10^6}{1000.\times(547.5)^2}\right)}$ = 1.28
4.3.6.3.2	***Modification factor for compression reinforcement***
	$A'_{prov.F}$ = 628 mm^2 (2Y20 bars)
Table 12	For a section at midspan $\rho' = \frac{100\times A'_{s.prov.F}}{b_{eff} d_F} = \frac{100\times 628}{1000\times 547.5}$ = 0. 115
	Modification factor for compression reinforcement:
	$MF_{A'S} = 1 + \frac{\rho'}{3+\rho'}$ = 1.037
	Check L/d ratio:
	Maximum allowable L/d ratio: *L_over_d_allow* = MF_{AS} x $MF_{A'S}$ x $(\frac{L}{d})_{basic}$ = 31.86
	Provided L/d *L_over_d_prov* = $\frac{L}{d_F} = \frac{5000}{547.5}$ = 9.13
	L_over_d_prov = 9.13 < *L_over_d_allow* = 31.86 **OK**

- Following above design, a bar bending schedule was generated and is as displayed under the appendix B

4.3 Column Design

Using the updated loads derived from both gravity and lateral load combinations, the column design was carried out comprehensively. Slenderness limits were checked in accordance with code standards. Both longitudinal and transverse reinforcement were appropriately sized and detailed to satisfy strength and serviceability requirements.

For the gridline 2 columns analysed in Phase 1, the structure produced the following result for the column to be the most critical.

$$\boldsymbol{Maximum\,Verical\,Force\;N = 1683.083kN}$$

$$\boldsymbol{Moments: \; M_1 = 3.01kNm\,;\; M_2 = 3.561kNm}$$

<u>Slenderness Classification</u>

Given :
Column Size: 300 x 300 mm
Beam Size: 600 x 300 mm
Slab Thickness: 250mm

Refference	Calculations
	Major axis (X-X) Braced
	$L_{ox} = 4500 - 600 = 3900mm$
SABS 0100-1 Table 18	Condition at top = 1 Condition at bottom = 3 $\left\{\begin{matrix}\boldsymbol{Assumed\ that\ the\ pad\ footing\ is\ not}\\ \boldsymbol{designed\ to\ resist\ any\ moment}\end{matrix}\right\}$
	$\beta = 0.9 \qquad \therefore\ L_{ex} = 0.9(3900) = 3510mm$
4.7.1.4	Limit $= 17 - {}^{7M_1}/_{M_2}$ $= 17 - \dfrac{7(3.01)}{3.561} = 11.08$
	$\dfrac{L_{eX}}{h} = \dfrac{3510}{300} = 11.7 > 11.08 \quad \therefore Slender$
	Minor Axis (Y-Y) -Unbraced
	$L_{oy} = 4500 - 600 = 3900mm$
	Condition at top = 1 Condition Bottom = 3 (Previously assumed)
Table 19	$\beta = 1.6 \qquad \therefore\ L_{ey} = 1.6(3900) = 6240mm$

4.7.1.4	Limit = 10 $\frac{L_{eY}}{h} = \frac{6240}{300} = 20.8 > 10 \quad \therefore Slender$ **Check slenderness ultimate Limit**: $l_o < 60b = 60 \times 0.3 = 18m$ $b \geq 0.25h = 75mm \qquad \therefore OK!!$

Design Moment

Refference	Calculations
	X-AXIS (Braced)
4.7.3.1, Eqn(12)	$$\beta_a = \frac{(slenderness\ ratio)^2}{2000}$$
	$h = 300mm$
	$K = 1.0\ (assumed\ for\ initial\ calculations)$
	$a_u = \beta_a Kh = (0.068)(1.0)(300) = 20.4mm$
	$M_{add} = N_{a_u} = (1699.5)(0.0204) = 34.669\ kNm$
4.7.3.2.1	$M_i = 0.4M_1 + 0.6M_2$ $= 0.4(0) + 0.6(9.551) = 5.731\ kNm$ $0.4M_2 = 0.4(9.551) = 3.820\ kNm$
	a) $M_2 = 9.551\ \boldsymbol{kNm}$ b) $M_i + M_{add} = 5.731 + 34.669 = \mathbf{40.4}\ \boldsymbol{kNm}$
	$e_{min} = 0.05h = 0.05(300) = 15mm\ (< 20)$ c) $e_{min}N = (0.015)(1699.5) = 25.493\ \boldsymbol{kNm}$
	$$\boxed{\mathbf{M_{design\ xx} = 40.4\ kNm}}$$
	Y-AXIS (Unbraced)
4.7.3.1 Eqn(12)	$$\beta_a = \frac{(slenderness\ ratio)^2}{2000}$$
	$h = 300mm$

4.7.2.3	$K = 1.0\ (assumed\ for\ initial\ calculations)$ $a_u = \beta_a Kh = (0.21632)(1.0)(300) = 64.896\ mm$ $M_{add} = N_{a_u} = (1699.5)(0.06489) = 110.281kNm$ $M_i = 0.4M_1 + 0.6M_2$ $M_i = 0.4(0) + 0.6(237.468) = 142.481kNm$ a) $M_2 = \mathbf{237.468}\boldsymbol{kNm}$ b) $M_i + M_{add} = 142.481 + 110.281 = \mathbf{252.762}\ \boldsymbol{kNm}$ $e_{min} = 0.05h = 0.05(300) = 15mm\ (< 20)$ c) $e_{min}N = 0.015(1699.5) = \mathbf{25.49}\ \boldsymbol{kNm}$ $\boxed{\mathbf{M_{design\ yy} = 252.762\ kNm}}$

Reinforcement Design

References	**Calculations**
                    Table 21 SANS0100	$N = 1699.5kN$ $M_{x\ top} = 9.551kNm$ $M_{x\ bottom} = 0\ kNm$ $M_{y\ top} = 237.468\ kNm$ $M_{y\ bottom} = 0\ kNm$ Bi-Axial Bending with: $Design\ as\ for\ the\ short\ column \begin{cases} N = 1699.5kN \\ M_{xx} = 40.4kNm \\ M_{yy} = 252.762kNm \end{cases}$ DESIGN FOR UNI-AXIAL BENDING $\frac{M_x}{h} = \frac{40.4}{300} = 0.13$ $\frac{M_y}{b} = \frac{252.762}{300} = 0.84 > 0.13$ $\therefore Design\ for\ y - axis\ bending$ Enhance moment for uniaxial design $\frac{N}{bhf_{cu}} = \frac{1699.5 \times 10^3}{(300)(300)(30)} = 0.63$ $\therefore \beta_b = 0.3$

4.7.4.4

$$M'_y = M_y + \beta_b \frac{b}{h} M_x$$

$$= 252.762 + (0.3)\left(\frac{300}{300}\right)(40.4) = 264.882kNm$$

$$\frac{N}{bhf_{cu}} = \frac{1699.5 \times 10^3}{(300)(300)(30)} = 0.63$$

$$\frac{M_y{}'}{bhf_{cu}} = \frac{264.882 \times 10^6}{(300)(300)(30)} = 0.327$$

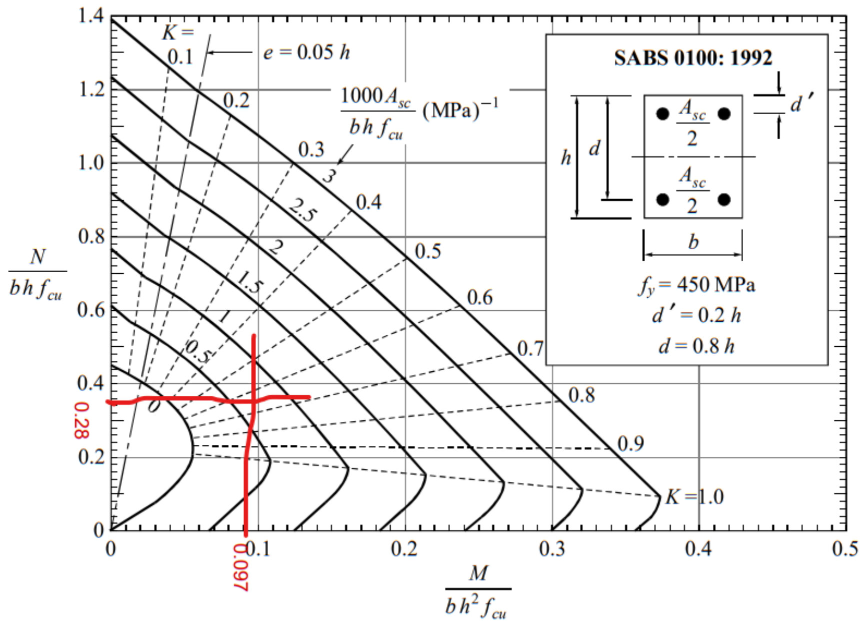


Figure 7: Chat from Robert and Marshal

$$For\ 300 \times 300\ column\ is\ not\ adequate \begin{cases} \dfrac{N}{bhf_{cu}} = 0.63 \\ \dfrac{M_y{}'}{bhf_{cu}} = 0.327 \end{cases}$$

$$Try\ (\mathbf{450 \times 450}) \begin{cases} \dfrac{N}{bhf_{cu}} = 0.28 \\ \dfrac{M_y{}'}{bhf_{cu}} = 0.097 \end{cases} \qquad \frac{1000A_{sc}}{bhf_{cu}} = 0.65$$

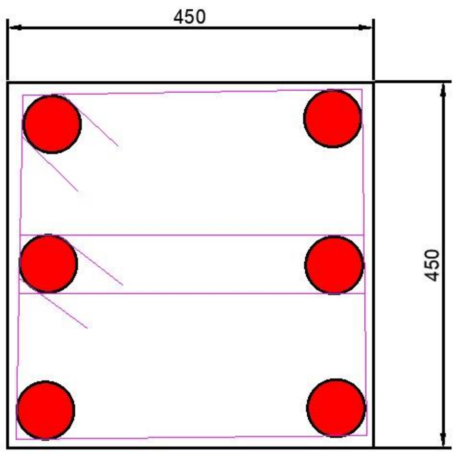
Min. $A_{sc} = 0.4\%\ \therefore OK$

Max. $A_{sc} = 6\%\ \ \therefore OK$

$$A_{sc} = \frac{0.65(450)(450)(30)}{1000} = 3290.625mm^2$$

$$Provide\ 6Y32\ bars = 4825mm^2\ > 3290.625mm^2 \therefore Ok$$

Links as per SANS

	$$\left.\begin{array}{c}\emptyset > \dfrac{32}{4} = 8mm \\ \text{-----} \\ Spacing < 12 \times 32 = 384mm\end{array}\right\} Provide\ R10@300\ stirrups$$ $$K = 0.65 < 1\ (assumed)\ \therefore Ok$$  Dist. To unrestrained bar $= \frac{450-90-90-32}{1}$ $= 236\ mm\ (> 150 \therefore additional\ restraint\ req)$

Following above design, a bar bending schedule was generated and is as displayed under the appendix

5. Discussion

The structural analysis and design process successfully demonstrated the integration of theoretical and practical design principles in accordance with SANS 10100 and SANS 10160. Both hand calculations and finite element (FE) analysis in Prokon produced consistent results, confirming the reliability of the modelling approach. However, in throughout comparison between hand calculations and prokon, it could be observed that the results obtained from hand calculations were over designed, which although it is safe and reliable, it is not cost effective. The inclusion of two perimeter shear walls proved effective in improving the building’s lateral stiffness, reducing storey drift, and enhancing overall stability under wind and seismic actions. Material selection, load combinations, and reinforcement detailing were all aligned with code requirements to ensure safety and serviceability. Minor differences observed between FE and hand results were attributed to simplifications in manual assumptions and differences in load application methods. Overall, the design achieved the intended objectives of strength, stability, and constructability.

6. Conclusion and recommendations

The project achieved its goal of designing a safe and efficient reinforced concrete structure capable of withstanding combined gravity and lateral loads. The slab, beams, and columns all satisfied the ultimate and serviceability limit state requirements, and the reinforcement layouts were optimized to ensure practicality in construction.

For future improvement, it is recommended to perform a more detailed foundation design to ensure full load transfer to the ground and to include long-term checks for deflection and cracking. It would also be beneficial to run additional simulations using different software or load scenarios to compare results and improve accuracy. Lastly, maintaining close coordination between analysis, detailing, and construction stages will help reduce errors and improve overall project quality.

7. Reflection

This project provided a meaningful learning experience by combining theoretical understanding with practical application. Throughout the process, I developed a deeper appreciation of how structural elements such as slabs, beams, and columns connect to form a stable system capable of resisting gravity and lateral forces. Using Prokon for the finite element modelling was a challenging, because the concept of 3D modelling was new, but it allowed me to visualize how loads are distributed and how stiffness assumptions influence the overall behaviour of the building.

The process also taught me the importance of verifying computer-based results with manual calculations. At times, discrepancies between the two methods required revisiting assumptions and adjusting load cases, which helped strengthen my problem-solving and critical thinking skills. I learned that accuracy in structural design does not only depend on software outputs, but also on understanding the reasoning behind each step and applying sound engineering judgement.

Additionally, working with South African design codes (SANS 10100 and SANS 10160) helped me gain confidence in applying local standards correctly to a real-life situation and interpreting code. It also showed me how design codes need to be consistently used to justify every assumption made.

Working with the group members have been an good experience, as the was a good communication an arrangement among us, making the learning experience not stressful.

Overall, this project has improved my technical knowledge, time management, and analytical abilities. More importantly, it reinforced the responsibility that comes with being a civil engineer to design safe, practical, and sustainable structures while upholding integrity and ethical decision. Making in every stage of the design process.

8. Reference

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9. Appendix