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Faculty of Engineering

Department of Civil and Environmental Engineering

Concrete Building Detailed Design Project Calculation Report

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Submitted in fulfilment of the requirements for the MSc and the Diploma of Imperial College London

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Department of Civil and Environmental Engineering

Cluster: _____MSc Structure Cluster_____

Module:_____Research / Design Project - Structures 2021-2022_____

Assignment:_____Concrete Building Detailed Design Project_____

Assignment Setter: _____Robert Vollum_____

Submission Deadline:_____23 August 2022_____

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Content Page

Calculation Report

In this part, sufficient calculations by hand and Finite Element Analysis show that the proposed design complies with Eurocode 2 criteria at the serviceability and ultimate limit states.
The detailed hand calculations include wind forces, overturning moment deflection and drifts for shear core under notional horizontal forces, axial and bi-axial stress for critical columns, sagging and hogging moments in the middle, and column strips in flat slabs, punching shear stress around columns, etc.
FEA model results will validate most of the design parameters(moment, shear). The detailed values will be presented here, and discussion and diagrams for comparison will be provided in the report.
After the design parameters are validated, the critical sections will be used to design for design and construction simplicity, and the elements with similar load conditions will be grouped to design.
For calculation sheet simplicity and conciseness, components will the same design procedures will only provide one detailed calculation, and the following repetitive calculations will be delivered as a compacted version of the calculation.
Basic illustration figures and assumptions will be presented here. The design report will discuss detailed comparisons and research methods to ensure consistency between the calculations and designs.

1 Wind force calculation

The design report defines the wind pressure zones as 3 divisions. The overall net wind pressure can be calculated for divided zones. Those values can be input into FEA models to consider wind pressure forces on the structure. For the 2D equivalent RHS cantilever beam(proposed in the design report), 5 defined uniformly distributed loads(q1, q2, q3, q4, q5) need to be calculated.
Considering the geometry changes along the height of the building, the wind surface will be defined as five equivalent rectangular surfaces, as shown in the design report. Therefore, five different uniformly distributed loads on the RHS cantilever beam can be calculated.

1.1 Wind pressure calculation (W-E)

1.1.1 Wind pressure calculation for Zone 1

Basic Values

Wind Direction (recommend value)	C_{dir}	=	1	
Season(recommend value)	C_{season}	=	1	
Altitude factor, not specified in brief, take 1	C_{alt}	=	1	
Level height	z	=	32.7	m
Assumed altitude	A	=	0	m
According to the brief, fundamental basic wind velocity	$V_{b,map}$	=	23.5	m/s
	$V_{b,0}$	=	23.5	m/s
$v_b = C_{dir} \cdot C_{season} \cdot v_{b,0}$	V_b	=	23.5	m/s

BS EN 1991-1-4 (4.1)

mean wind velocity $v_{m(z)}$

As Terrain category is category 0-Sea or coastal area exposed to the open sea				
roughness length	Z_0	=	0.003	
Fixed value	$Z_{0,2}$	=	0.05	
Altitude factor	Z_{min}	=	1	
	Z_{min}	=	200	

EC1-1-4:
4.2(1) Note 2 & NA 2.4, 2.5
4.2(2) Note 1

7.2.2(1), Note & NA 2.26

4.2(2) Note 3 & NA 2.7: Fig. NA.2
4.2(1) Note 2 & NA 2.4: Fig. NA.1

As $z_{min}<z<z_{max}$				
$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07}$	K_r	=	0.156	
$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right)$	$C_{r(z)}$	=	1.447	
As recommended take	$C_{0(z)}$	=	1	
$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b$	$V_{m(z)}$		34.09	m/s

Determine Wind turbulence

As Terrain category is category 0-Sea or coastal area exposed to the open sea				
turbulence factor	K_I	=	1	
Fixed value $\sigma_v = k_r \cdot v_b \cdot k_I$	σ_v	=	3.67	
The turbulence intensity $I_v(z) = \frac{\sigma_v}{v_m(z)}$ for $z_{min} \le z \le z_{max}$	$I_v(z)$	=	1	

Determine Peak velocity pressure qp(z)

In NA, take $\rho = 1.25$ kg/m3	ρ_{air}	=	1.25	kg/m^3
$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2$	q_b	=	345.2	
$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z)$	$q_p(z)$	=	1273	N/m^2
		=	1.273	KN/m^2
exposure factor $c_e(z) = \frac{q_p(z)}{q_b}$	$C_e(Z)$	=	3.689	

Overall net wind pressure

Overall Net wind Pressure = $q_p(z)$ ×Net Pressure Coefficient (clause NA.2.27)

Width of building	d	=	16	m
	h/d	=	2.044	
Net Pressure Coefficient		=	1.18	
Overall Net wind Pressure		=	1.502	KN/m^2

Net Pressure Coefficient—NA 2.27

1.1.2 Wind pressure calculation for Zone 2

Basic Values

C_{dir}	C_{season}	C_{alt}	z m	A m	$V_{b,map}$ m/s	$V_{b,0}$ m/s	V_b m/s
1	1	1	36.7	0	23.5	23.5	23.5

Mean wind $V_m(Z)$								Wind turbulence		
Z_0	$Z_{0,2}$	Z_{min} m	Z_{max} m	K_r	$C_r(z)$	$C_o(z)$	$V_m(Z)$ m/s	K_I	σ_v	$I_v(z)$
0.003	0.05	1	200	0.156	1.46	1	34.5	1	3.66	0.11

Peak velocity pressure $q_p(z)$				Overall Net wind pressure			
ρ_{air}	q_b	$q_p(z)$	$C_e(Z)$	d	h/d	$Factor_{net}$	Pressure
kg/m3	KN/m2	KN/m2		m			KN/m^2

1.25	0.34	1.29	3.76	16	2.29	1.21	1.57

1.1.3 Wind pressure calculation for Zone 3

C_{dir}	C_{season}	C_{alt}	z	A	$V_{b,map}$	$V_{b,0}$	V_b
			m	m	m/s	m/s	m/s
1	1	1	68.7	0	23.5	23.5	23.5

Mean wind Vm(Z)								Wind turbulence		
Z_0	$Z_{0,2}$	Z_{min}	Z_{max}	K_r	$C_r(z)$	$C_o(z)$	$V_m(Z)$	K_I	σ_v	$I_v(z)$
		m	m				m/s			
0.003	0.05	1	200	0.156	1.566	1	36.81	1	3.66	0.10

Peak velocity pressure $q_p(z)$				Overall Net wind pressure			
ρ_{air}	q_b	$q_p(z)$	$C_e(Z)$	d	h/d	$Factor_{net}$	Pressure
kg/m3	KN/m2	KN/m2		m			KN/m^2
1.25	0.345	1.437	4.165	16	4.294	1.28	1.84

1.1.4 Convert to UDL along the height

For more accurate calculation, geometry changes are also take into consideration, therefore, as shown in figure and the table below, the udl are divided in 5 parts.

Pressure Zone	Items	Range m	Corresponding Length m	No UDL Kn/m
Zone 3	q1	L18-L22	19.5	35.88
	q2	36.7-L18	22.10	40.67
Zone 2	q3	L10-36.7	25.75	40.44
Zone 1	q4	L2-L10	28.875	43.38
	q5	G-L2	40	60.09

1.2 Wind pressure calculation (N-S)

1.2.1 Wind pressure calculation for Zone 1

Basic Values

C_{dir}	C_{season}	C_{alt}	z	A	$V_{b,map}$	$V_{b,0}$	V_b
			m	m	m/s	m/s	m/s
1	1	1	16	0	23.5	23.5	23.5

Peak velocity pressure qp(z)				***Overall Net wind pressure***			
ρ_{air}	q_b	$q_p(z)$	$C_e(Z)$	d	h/d	$Factor_{net}$	Pressure
kg/m^3	KN/m^2	KN/m^2		m			KN/m^2
1.25	0.345	1.124	3.256	32	0.5	0.94	1.056

Mean wind Vm(Z)								Wind turbulence		
Z_0	$Z_{0,2}$	Z_{min} m	Z_{max} m	K_r	$C_r(z)$	$C_o(z)$	$V_m(Z)$ m/s	K_I	σ_v	$I_v(z)$
0.003	0.05	1	200	0.156	1.339	1	31.47	1	3.66	0.12

1.2.2 Wind pressure calculation for Zone 2

Basic Values

C_{dir}	C_{season}	C_{alt}	z m	A m	$V_{b,map}$ m/s	$V_{b,0}$ m/s	V_b m/s
1	1	1	52.7	0	23.5	23.5	23.5

Mean wind Vm(Z)								Wind turbulence		
Z_0	$Z_{0,2}$	Z_{min} m	Z_{max} m	K_r	$C_r(z)$	$C_o(z)$	$V_m(Z)$ m/s	K_I	σ_v	$I_v(z)$
0.003	0.05	1	200	0.156	1.525	1	35.84	1	3.66	0.10

Peak velocity pressure qp(z)				Overall Net wind pressure			
ρ_{air} kg/m^3	q_b KN/m^2	$q_p(z)$ KN/m^2	$C_e(Z)$	d m	h/d	$Factor_{net}$	Pressure KN/m^2
1.25	0.345	1.378	3.992	32	1.647	1.125	1.55

1.2.3 Wind pressure calculation for Zone 3

Basic Values

C_{dir}	C_{season}	C_{alt}	z m	A m	$V_{b,map}$ m/s	$V_{b,0}$ m/s	V_b m/s
1	1	1	68.7	0	23.5	23.5	23.5

Peak velocity pressure $q_p(z)$				Overall Net wind pressure			
ρ_{air} kg/m^3	q_b KN/m^2	$q_p(z)$ KN/m^2	$C_e(Z)$	d m	h/d	$Factor_{net}$	Pressure KN/m^2
1.25	0.345	1.437	4.165	32	2.147	1.19	1.71

Mean wind Vm(Z)								Wind turbulence		
Z_0	$Z_{0,2}$	Z_{min} m	Z_{max} m	K_r	$C_r(z)$	$C_o(z)$	$V_m(Z)$ m/s	K_I	σ_v	$I_v(z)$
0.003	0.05	1	200	0.156	1.566	1	36.81	1	3.66	0.10

1.2.4 Convert to UDL along the height

Pressure Zone	Num.	Range m	Corresponding Length m	No UDL Kn/m
Zone 3	q1	L18-L22	9	15.4
	q2	36.7-L18	16	27.37
Zone 2	q3	L10-36.7	16	24.8
Zone 1	q4	L2-L10	16	16.9
	q5	G-L2	32	33.8

2 Shear core design

After the wind load has been calculated, the horizontal load can be used to design shear core.

2.1 General introduction and analysis

The detailed calculation and deflection check are based on a simplified 2D frame RHS cantilever beam model, assuming the core takes all the lateral forces.
In the design report, deflection, drift, and extreme fibre compressive stress results in 2D and 3D FEA core models will be compared and discussed based on the calculation results here. Moreover, core and core-frame structures' behaviors will be investigated. Therefore, deflection, drift, and axial stress results will be presented here..

2.2 Section layout and Dimensions

2.2.1 Section layout for lower and upper cores

Figure 2- 1and Figure 2- 2 show the simplified RHS for lower and upper cores, the axis of direction as well as the labels of walls.

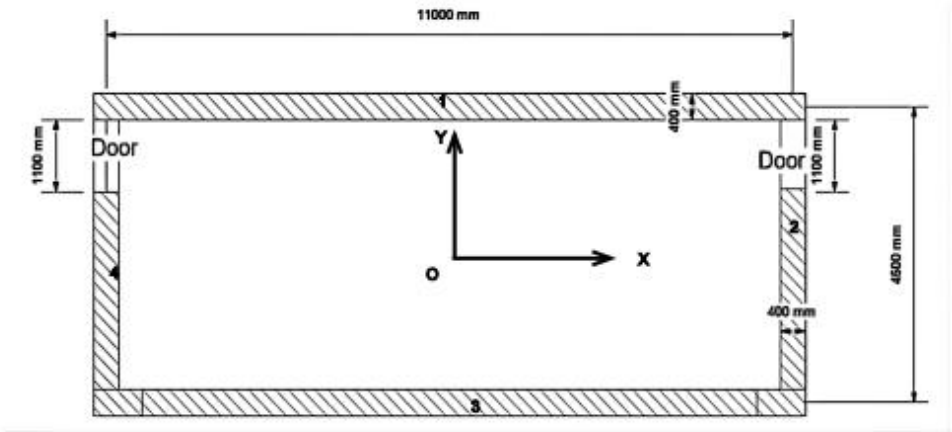


Figure 2- 1 Lower core layout and dimensions

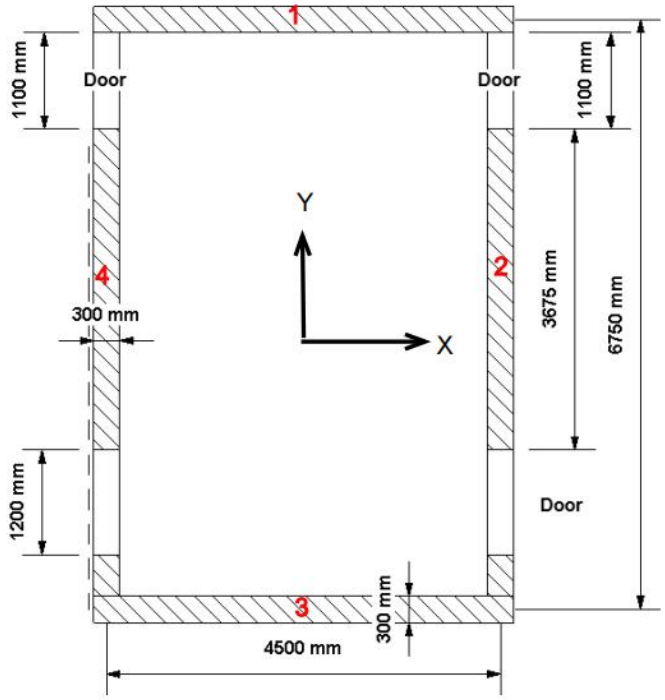


Figure 2- 2 Upper core layout and dimensions

2.2.2 Section dimensions and areas

Dimensions and area for shear walls

For lower walls	Length X-Dire mm	Y-Dire mm	Area mm^2	For upper walls	Length X-Dire mm	Y-Dire mm	Area mm^2
Wall 1	11000	400	4E+06	Wall 1	4500	300	1E+06
Wall 2	400	3400	1E+06	Wall 2	300	3675	1E+06
Wall 3	11000	400	4E+06	Wall 3	4500	300	1E+06
Wall 4	400	3400	1E+06	Wall 4	300	3675	1E+06
	Total area		1E+07		Total area		5E+06

2.3 Stiffness calculation

Assume that I_x is the second moment of area in W-E direction and I_y is that in N-S direction.

For core from Basement to Level 18

Region	For I_x b(mm)	H(mm)	For I_y b(mm)	h(mm)	Area (mm^2)
1	11000	400	400	11000	4E+06
2	400	3400	3400	400	1E+06
3	11000	400	400	11000	4E+06
4	400	3400	3400	400	1E+06
				Total Area=	1E+07

Centre of the region Y(For I_x)	X(For I_y)	For I_x a	For I_y a	I_x mm^4	I_y mm^4
2250	0	2250	0	2.23E+13	4.44E+13
-550	5500	-550	5500	1.72E+12	4.12E+13

-2250	0	-2250	0	2.23E+13	4.44E+13
-550	-5500	-550	-5500	1.72E+12	4.12E+13
			Total Ix&Iy=	4.81E+13	1.71E+14

The calculated second moment of areas can be applied to Figure 3

For core from Level 18 to Level 22

Assume the centre of the core is O in the lower core, therefore, the corresponding centre coordinates in the upper core is x=-750 mm and y= 1125 mm.

Region	For I_x b(mm)	H(mm)	For I_y b(mm)	h(mm)	Area (mm^2)
1	4500	300	300	4500	1E+06
2	300	3675	3675	300	1E+06
3	4500	300	300	4500	1E+06
4	300	3675	3675	300	1E+06
				Total Area=	5E+06

Centre of the region Y(For I_x)	X(For I_y)	For I_x a	For I_y a	I_x mm^4	I_y mm^4
2250	0	2250	0	2.23E+13	4.44E+13
-550	5500	-550	5500	1.72E+12	4.12E+13
-2250	0	-2250	0	2.23E+13	4.44E+13
-550	-5500	-550	-5500	1.72E+12	4.12E+13
			Total Ix&Iy=	5.06E+13	1.66E+13

2.4 Material properties

Characteristic uniaxial compressive strength	f_{ck}	=	50	MPa
Design compressive strength	f_{cd}	=	28.33	MPa
Young's Modulus E_{cm}=22((f_{ck}+8)/10)0.3	E_{cm}	=	37.28	MPa
Density	ρ_c	=	25	(Kn/m3)
use UK Grade	f_{yk}	=	500	MPa
Design yield strength	f_{yd}		434.8	MPa

2.5 Load condition and actions

Surface Load

Surface load		Kn/m^2
Dead		
Finishes	=	1
Perimeter Load	=	ignore
Live		
For Retail	=	5
Podium Deck	=	10
Residential	=	2.5
Garden	=	10
Circulation	=	5

Load combinations defined in EC0

Unfactored wind load

	q1 Kn/m	q2 Kn/m	q3 Kn/m	q4 Kn/m	q5 Kn/m
W-E	35.88	40.67	40.44	43.38	60.09
N-S	15.4	27.37	24.8	16.9	33.8

The calculated UDLs and second moment of areas can be applied into the 2D RHS cantilever beam(Figure 2-3)

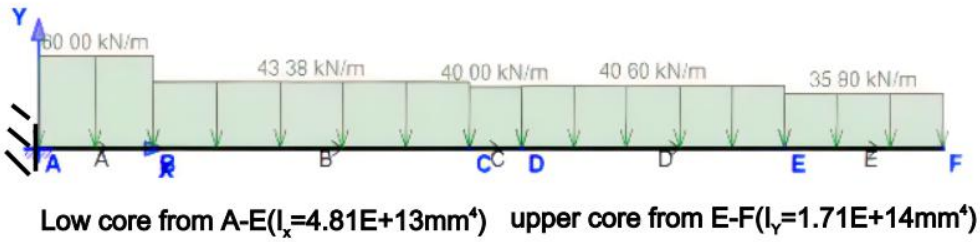


Figure 2-3 2D Equivalent cantilever beam (Wind blow from W-E)

2.6 Axial load calculation

The axial loads per floor should be calculated to obtain the notional horizontal loads of the structure(consider the imperfection),

	Level Number	Height (m)	Loading width X(m)	Loading width Y(m)	Loading Area(m^2)	Slab Thickness(mm)
Load take	Level 22	3	14	7.875	110.25	300
Upper	Level 21	3	14	7.875	110.25	300
Wall	Level 20	3	14	7.875	110.25	300
	Level 19	3	14	7.875	110.25	300
*Conservatively, assume all the loads from upper walls are transferred to the lower walls For simplicity, all the loads are evenly shared in lower walls						
Load taken	Level 18	3	15.75	9.25	145.69	300
Lower	Level 17	3	15.75	9.25	145.69	300
Wall	Level 16	3	15.75	9.25	145.69	300
Load area	Level 15	3	15.75	9.25	145.69	300
Average	Level 14	3	15.75	9.25	145.69	300
	Level 13	3	15.75	9.25	145.69	300
	Level 12	3	15.75	9.25	145.69	300
	Level 11	3	15.75	9.25	145.69	300
	Level 10	3	15.75	9.25	145.69	300
	Level 9	3	15.75	9.25	145.69	300
	Level 8	3	15.75	9.25	145.69	300
	Level 7	3	15.75	9.25	145.69	300
	Level 6	3	15.75	9.25	145.69	300
	Level 5	3	15.75	9.25	145.69	300
	Level 4	3	15.75	9.25	145.69	300
	Level 3	3	15.75	9.25	145.69	300
	Level 2	4.35	15.75	9.25	145.69	350
	Level 1	4.35	15.75	9.25	145.69	350
	Level G	3	15.75	9.25	145.69	350

	Surface Load		Wall SW	Total load per floor	
Level	Gk	Qk	Load	Gk	Qk
Number	Kn/m2	Kn/m2	Kn	Kn	Kn

Level 22	7.5	5.00	367.875	826.9	551.3
Level 21	7.5	5.00	367.875	826.9	551.3
Level 20	7.5	5.00	367.875	826.9	551.3
Level 19	7.5	5.00	367.875	826.9	551.3
Level 18	7.5	2.50	864	1093	364.2
Level 17	7.5	2.50	864	1093	364.2
Level 16	7.5	2.50	864	1093	364.2
Level 15	7.5	2.50	864	1093	364.2
Level 14	7.5	2.50	864	1093	364.2
Level 13	7.5	2.50	864	1093	364.2
Level 12	7.5	2.50	864	1093	364.2
Level 11	7.5	2.50	864	1093	364.2
Level 10	7.5	2.50	864	1093	364.2
Level 9	7.5	2.50	864	1093	364.2
Level 8	7.5	2.50	864	1093	364.2
Level 7	7.5	2.50	864	1093	364.2
Level 6	7.5	2.50	864	1093	364.2
Level 5	7.5	2.50	864	1093	364.2
Level 4	7.5	2.50	864	1093	364.2
Level 3	7.5	2.50	864	1093	364.2
Level 2	8.75	3.75	1252.8	1275	546.3
Level 1	8.75	3.75	1252.8	1275	546.3
Level G	8.75	3.75	864	1275	546.3
Accumulated SW,Gk and Qk=			18665.1	24614	9671

2.7 Notional horizontal load(per floor)

(EC2 Figure 5.1)

For total columns>20&Building height >10 Notional inclination=1/410=0.002 (EC2 Figure 5.1)

For L22-L19

H_{idead}	=	2.017	Kn
Hi_{live}	=	1.345	Kn

For L18-L3

H_{idead}	=	2.665	Kn
H_{ilive}	=	0.888	Kn

For L2-LG

H_{idead}	=	3.109	Kn
H_{ilive}	=	1.333	Kn

Dead and live imposed load

2.8 Design for West-East direction (X-direction)

2.8.1 Critical load combination for calculating overturning moments

1) Maximum compressive stress in shear wall under wind loading:

Critical load combinations for calculating overturning moments for:

1.5Wk+1.35Gk+1.5 ×0.7Qk	M_{max}	=	1E+05	Kn.m
At bottom of base level	N_{max}	=	43384	Kn

2)Minimum compressive stress in shear wall:

1.5Wk+1.0Gk	M_{max}	=	1E+05	Kn.m
At bottom of base level	N_{max}	=	24614	Kn

Because it's a core system, therefore,in W-E Direction Assume that:
The equivalent flanges will take all moment (Walls along the N-S direction)
For this reason, the Wall No.1.3 will take all the moment

2.8.2 Reinforcement design

Second moment area for W-E direction	I_x	=	5E+13	mm^4
longest length from center to section fibre	y_{max}	=	2250	mm
Z=I/y	Z	=	2E+10	mm^3

The extreme fibre flexural compressive stress f= N/A±M/Z

For 1)maximum compressive stress case

longest length from center to section fibre	f_{bigger}	=	10.22	MPa
Z=I/y	$f_{smaller}$	=	-2.69	MPa

For 2)minimum compressive stress case

longest length from center to section fibre	f_{bigger}	=	8.52	MPa
Z=I/y	$f_{smaller}$	=	-4.25	MPa

Design Stress

Take the maximum and minimum stress

longest length from center to section fibre	f_{max}	=	10.22	MPa
Z=I/y	f_{min}	=	-4.25	MPa

*Note:"+" in compression, "-" in tension

Provide Vertical reinforcement in each face

Reinforcements should be applied to resist tension in shear walls.

Design reinforcement for full tensile force

Design Stress				
the maximum tensile force per unit length	T	=	1699	Kn/m
Required tension reinforcement for resist tension	As,tension	=	3907	mm^2/m
	ρ_{req}	=	0.98%	
	ρ_{pro}	=	1.13%	
	Check		ok	

Provide	H24@200	
bar diameter	24	mm
Spacing	200	mm
Number of bars/m	5	
As,pro (one face)	2261.95	mm^2/m
As,pro(two faces)	4523.89	mm^2/m
ρpro	1.13%	

Provide Horizontal reinforcement in each face

As,horizontal =	$0.001A_c$	=	400	Kn/m
(in two face) or =	$25\%A_{vertical}$	=	1131	mm^2/m
	$\rho_{req,hor}$	=	0.28%	
	ρ_{pro}	=	0.40%	
	Check		ok	

Provide	H16@250	
bar diameter	16	mm
Spacing	250	mm
Number of bars/m	4	
As,pro (one face)	804.25	mm2/m
As,pro(two faces)	1608.49	mm2/m
ρpro	0.40%	

Distribution of vertical reinforcement

The design reinforcement can either be distributed uniformly along the length of the wall or be concentrated at the ends of the wall

See 4.10.1 &Detailing

Manual

Figure 2- 4 distribution of vertical reinforcement in shear walls

uniformly bar spacing and diameter
Check for the compressive stress provided by concrete

For 1)maximum compressive stress case

the compressive stress provided by concrete $\text{stress} = 0.44f_{ck} + \frac{\rho}{100}(0.67f_{yk} - 0.44f_{ck})$	Stress	=	23.26	MPa
Check with f_{yd}	Check		ok	

As the wind condition is symmetry
Therefore, the reinforcement arrangement is also symmetric about X direction

2.9 Design for North-South direction (Y-direction)

2.9.1 Critical load combination for calculating overturning moments

Maximum compressive stress in shear wall under wind loading:

1.5Wk+1.35Gk+1.5 ×0.7Qk	M_{max}	=	78743	Kn.m
At bottom of base level	N_{max}	=	43384	Kn

2)Minimum compressive stress in shear wall:

1.5Wk+1.0Gk	M_{max}	=	1E+05	Kn.m
At bottom of base level	N_{max}	=	24614	Kn

Because it's a core system, therefore,in W-E Direction Assume that:
And the equivalent web will take all shear forces (Walls along the E-W direction)For this reason, the Wall No.1.3 will take all the moment

2.9.2 Reinforcement design

Second moment area for W-E direction	Ix	=	5E+13	mm^4
longest length from center to section fibre	y_{max}	=	5500	mm
Z=I/y	Z	=	2E+10	mm^3

The extreme fibre flexural compressive stress

$$f = \frac{N}{A} \pm \frac{M}{Z}$$

For 1)maximum compressive stress case

longest length from center to section fibre	f_{bigger}	=	6.298	MPa
Z=I/y	$f_{smaller}$	=	-1.234	MPa

For 2)minimum compressive stress case

longest length from center to section fibre	f_{bigger}	=	6.755	MPa
Z=I/y	$f_{smaller}$	=	-2.48	MPa

Design Stress

longest length from center to section fibre	f_{max}	=	6.755	MPa
Z=I/y	f_{min}	=	-2.48	MPa

*Note:"+" in compression, "-" in tension

Provide Vertical reinforcement in each face

Design Stress				
the maximum tensile force per unit length	T	=	744.5	Kn/m
Required tension reinforcement for resist tension	As,tension	=	1712	mm^2/m
	ρ_{req}	=	0.43%	
	ρ_{pro}	=	0.50%	
	Check		ok	

Provide	H16@200	
bar diameter	16	mm
Spacing	200	mm
Number of bars/m	5	
As,pro (one face)	1005.31	mm2/m
As,pro(two faces)	2010.62	mm2/m
ρpro	0.50%	

Provide Horizontal reinforcement in each face

As,horizontal =	0.001Ac	=	400	Kn/m
(in two face) or =	$25\%A_{vertical}$	=	503	mm2/m
	$\rho_{req,hor}$	=	0.10%	
	ρ_{pro}	=	0.20%	
	Check		ok	

Provide	H10@200	
bar diameter	10	mm
Spacing	200	mm
Number of bars/m	5	
As,pro (one face)	392.70	mm2/m
As,pro(two faces)	785.4	mm2/m
ρ_{pro}	0.20%	

2.9.3 Check for the compressive stress provided by concrete

For 1)maximum compressive stress case

the compressive stress provided by concrete $\text{stress} = 0.44f_{ck} + \frac{\rho}{100}(0.67f_{yk} - 0.44f_{ck})$	Stress	=	22.61	MPa
Check with fyd	Check		ok	

As the wind condition is symmetry
Therefore, the reinforcement arrangement is also symmetric about Y direction

2.10 Deflection check

The deflection for each level in 2D RHS cantilever beam can be calculated and the deflection for 3D and 3d FEA Core-frame can be manually recorded as shown below
For SLS check, live load only.

$$\text{Allowable deflection or drift} = \frac{h}{500}$$

(h for total building height in deflection and storey height for drift)

For W-E direction

	2D-simplified model		3D FEA Core only		3D FEA Core-frame		Allowable	
	deflection mm	Drift mm	deflection mm	Drift mm	deflection mm	Drift mm	deflection mm	Drift mm
Lg	0.00	0.00	0.00	0.00	0	0.00		
L1	1.24	1.24	1.63	1.63	1.09	1.09	8.7	8.7
L2	2.47	1.24	3.42	1.79	2.19	1.10	17.4	8.7

L3	5.56	3.09	5.33	1.91	3.26	1.07	23.4	6
L4	8.65	3.09	7.59	2.26	4.52	1.27	29.4	6
L5	11.73	3.09	10.19	2.60	5.95	1.42	35.4	6
L6	14.82	3.09	13.09	2.90	7.47	1.53	41.4	6
L7	17.91	3.09	16.25	3.16	9.08	1.60	47.4	6
L8	21.00	3.09	19.63	3.38	10.88	1.80	53.4	6
L9	24.08	3.09	23.20	3.57	12.63	1.75	59.4	6
L10	27.17	3.09	26.92	3.73	14.30	1.67	65.4	6
L11	32.08	4.91	30.78	3.86	16.11	1.81	71.4	6
L12	36.57	4.49	34.74	3.96	17.88	1.77	77.4	6
L13	41.06	4.49	38.79	4.04	19.72	1.84	83.4	6
L14	45.55	4.49	42.90	4.11	21.52	1.81	89.4	6
L15	50.03	4.49	47.05	4.15	23.30	1.77	95.4	6
L16	54.52	4.49	51.22	4.18	25.00	1.70	101.4	6
L17	59.01	4.49	55.41	4.19	26.85	1.85	107.4	6
L18	63.50	4.49	59.60	4.19	28.60	1.74	113.4	6
L19	68.28	4.78	66.99	7.39	30.31	1.72	119.4	6
L20	73.06	4.78	71.28	4.29	32.02	1.71	125.4	6
L21	77.83	4.78	75.58	4.30	33.75	1.73	131.4	6
L22	82.61	4.78	79.86	4.28	35.48	1.72	137.4	6

For N-S direction

	2D-simplified model		3D FEA Core only		Allowable	
	deflection mm	Drift mm	deflection mm	Drift mm	deflection mm	Drift mm
Lg	0.00	0.00	0	0.00		
L1	0.19	0.19	0.40	0.40	8.7	8.7
L2	0.37	0.19	0.80	0.40	17.4	8.7
L3	0.83	0.46	1.20	0.40	23.4	6
L4	1.29	0.46	1.75	0.55	29.4	6
L5	1.75	0.46	2.30	0.55	35.4	6
L6	2.22	0.46	2.90	0.60	41.4	6
L7	2.68	0.46	3.50	0.60	47.4	6
L8	3.14	0.46	4.20	0.70	53.4	6
L9	3.60	0.46	4.90	0.70	59.4	6
L10	4.06	0.46	5.60	0.70	65.4	6
L11	4.90	0.84	6.50	0.90	71.4	6
L12	5.56	0.66	7.30	0.80	77.4	6
L13	6.21	0.66	8.10	0.80	83.4	6
L14	6.87	0.66	8.90	0.80	89.4	6
L15	7.52	0.66	9.70	0.80	95.4	6
L16	8.18	0.66	10.50	0.80	101.4	6
L17	8.83	0.66	11.30	0.80	107.4	6
L18	9.49	0.66	12.12	0.82	113.4	6
L19	10.22	0.73	14.30	2.18	119.4	6
L20	10.96	0.73	16.50	2.20	125.4	6
L21	11.69	0.73	18.90	2.40	131.4	6

L22	12.42	0.73	21.00	2.10	137.4	6

In the SLS check for deflection, all deflection and drifts for the 2D RHS cantilever beam can pass the requirement, while the drift between 18 and 19 in the W-E direction exceeds the limit. This may be because of the change in the core layout and the weak point at level 18. This may be strengthened by particular construction methods to make the connection stiffness enough.

3 Column design

3.1 General introduction and analysis

As assumed that the central core takes all horizontal loads. Therefore, in the column design, except for the axial load that needs to be considered, moments induced by imperfection and the load transferred by slabs also need to be considered.
Only typical detailed column calculations will be provided here as the limit of pages. All dimensions and reinforcement arrangements are provided in the drawings and have met design requirements.

3.1.1 Column grid specification and dimension

For simplicity, columns are labeled with specific numbers.As the arrangement of columns are symmetrical, only half of the columns will be designed.
Meanwhile, considering some columns have a similar loads and loading cases, simplicity for construction and design. The columns can be divided into different groups to design in Figure 3- 1.

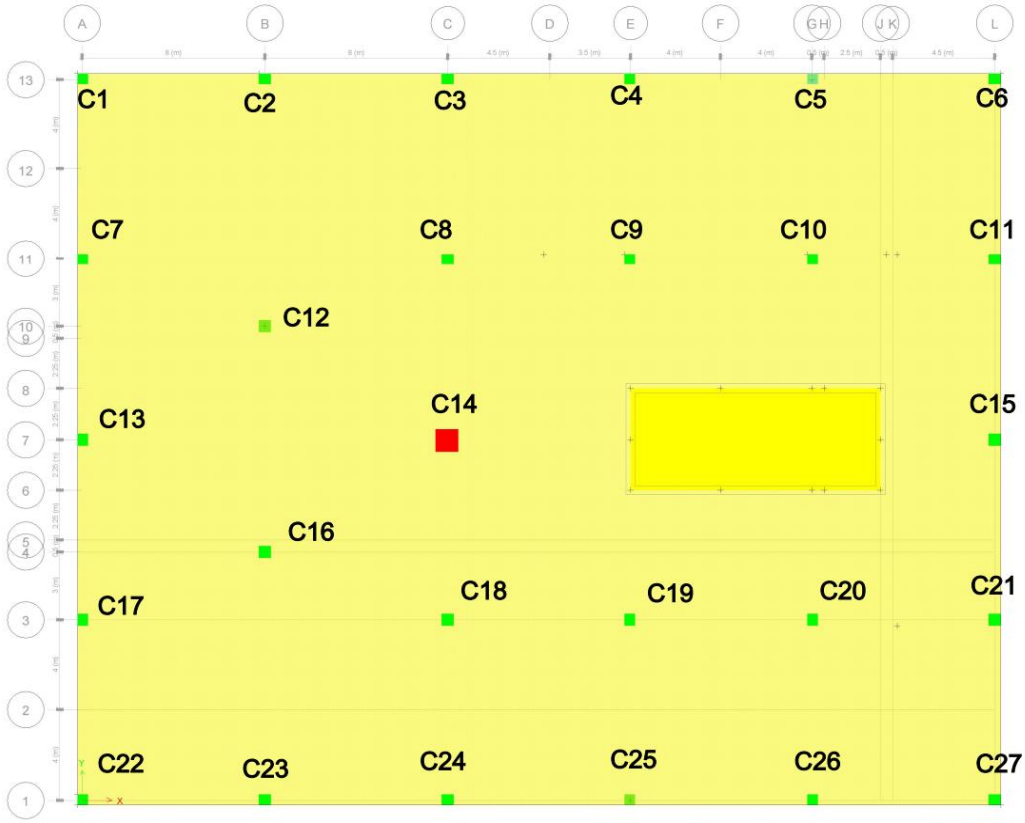


Figure 3- 1 Columns layout and label specification

3.1.2 Design groups

In level 2 column layout

The critical column use to design			
C2 (loading area max) Edge Column in Grid 13	Group 1	=	C2,C3,C4,C5
Corner Column Separately design	Group 2	=	C1,C6
C13 is critical Edge Column in Grid A	Group 3	=	C7,C13
C12	Group 4	=	C8,C9,C10,C12
C15	Group 5	=	C11, 15
C14	Group 6	=	C14

In level 18 column layout

3.2 Design for G1 column (detailed)

Design C2 for G1 columns, for the edge column, the maximum axial force is at the bottom in ground level and the maximum bending moment is at top of level 2.

3.2.1 Material Properties

f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m^3)	f_{yk} MPa	f_{yd} MPa
50	28.33	37.28	25	500	434.8

3.2.2 Relative section dimensions

For the squared column

Column width in two direction	b,h	=	500	mm
Area of the column	A_c	=	3E+05	mm^2
Slab thickness		=	350	mm
Rebar diameter		=	32	mm

3.2.3 Cover

Assuming 8mm for link	d_{link}	=	8	mm	
assuming 32 mm diameter reinforcement	$c_{min,b}$	=	24	mm	
Minimum cover due to environmental conditions.For S4 and XS1	$c_{min,dur}$	=	45	mm	Exp. (4.1)
C_{min}=max($C_{min,b}$;$C_{min,dur}$;10mm)	fixed value	=	10	mm	Cl. 4.4.1.2(3)
	c_{min}	=	45	mm	BS 8500-1: Table A4
C_{nom} =C_{min}+ ΔC_{dev}	ΔC_{dev}	=	10	mm	
	C_{nom}		55	mm	
For 2 hours resistance	a_{min}		51	mm	Cl. 4.4.1.3 & NA

3.2.4 Load Condition and Actions (Only for C2)

		Surface load		Line load(along Grid 13)
Surface load		Kn/m^2		Kn/m
Dead				
SW(Slab)	=	8.75		
Finishes	=	1		
Perimeter Load			=	2
Live				
For Retail	=	5		
Podium Deck	=	10		

Load combinations defined in EC0

3.2.5 Load taken down

Corresponding load area, surface load on floor and line load from X and Y direction

Level	Level Height	Loading width		Load Area	Surface Load Gk	Surface Load Qk	Line Load(Gk) Ydirect	Line Load(Gk) Xdirect	Line Load(Qk) Ydirect	Line Load(Qk) Xdirect
Number	(m)	X(m)	Y(m)	(m2)	Kn/m^2	Kn/m^2	Kn/m	Kn/m	Kn/m	Kn/m
L2	4.35	8	5.5	44	9.75	10	78	55.6	80	55

L1	4.35	8	5.5	44	9.75	5	78	55.6	40	27.5
LG	3.35	8	5.5	44	9.75	5	78	55.6	40	27.5

	From Floor	From Column	From Floor
Items	Gk(Kn)	Gk(Kn)	Qk(Kn)
L2	429		440
		27.19	
L1	429		220
		27.19	
G	429		220
		20.94	
Base			
Sum	1362.31250		880

Axial load transfer from above (unfactored)

3.2.6 Design Axial Load Ned (at bottom in ground level)

Most Critical Load Combination is

Ned,base = 1.35 × Gk + 1.5 × Qk = 3150 Kn

Exp. (6.10)

3.2.7 First order design moment, M

In this part, not only hand calculation and computational model are calculated, but also different span arrangement are considered.

<u>Alternative-Span-Loaded</u>

<u>Hand calculations</u>

C2 in Grid Line 13 **X-direction**

C2 in Grid Line 13 X-direction

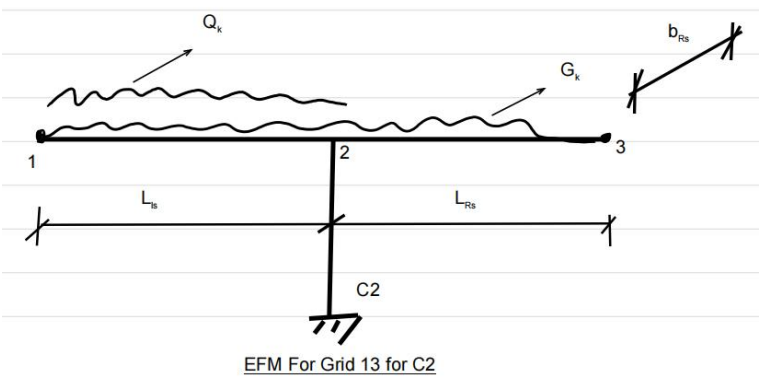


Figure 3- 2 EFM of alternative-Span-Load case for C2

Equivalent frame model is shown above

$K_{relative}=b_{lc}d_{lc}3/L_{lc}/(b_{lc}dl_c^3/L_{lc}+b_{uc}d_{uc}^3/L_{uc}+0.75b_{ls}d_{ls}^3/L_{ls}+0.75b_{rs}d_{rs}^3/L_{rs}$

blc	=	500	hlc	=	500	Llc	=	4	bh^3/L	=	1.56E+07
buc	=	500	huc	=	500	Luc	=	0	bh^3/L	=	0
bls	=	5500	hls	=	350	Lls	=	8	bh^3/L	=	2.95E+07
brs	=	5500	hrs	=	350	Lrs	=	8	bh^3/L	=	2.95E+07

$k_{relative}$	=	0.261									

For calculating relative stiffness of column

1st order moment using Exp. (6.10)

		Surface load			Line load(Kn/m)
Fully Loaded span	=	28	Kn/m^2	=	155
Gk load only span	=	10	Kn/m^2	=	54
FEM21	=	826.1	Kn.m		Fully loaded in span 12
FEM23	=	286	Kn.m		Partially loaded in span 23
Mlower,xx	=	141	Kn.m		

C2 in Grid Line 13 X-direction
Hand calculations for edge frame in Grid B do not be provided here, as the critical load arrangement is fully-loaded-span clearly.
Computational Results (Equivalent Frame Mode Method)
C2 in Grid Line 13 X-direction

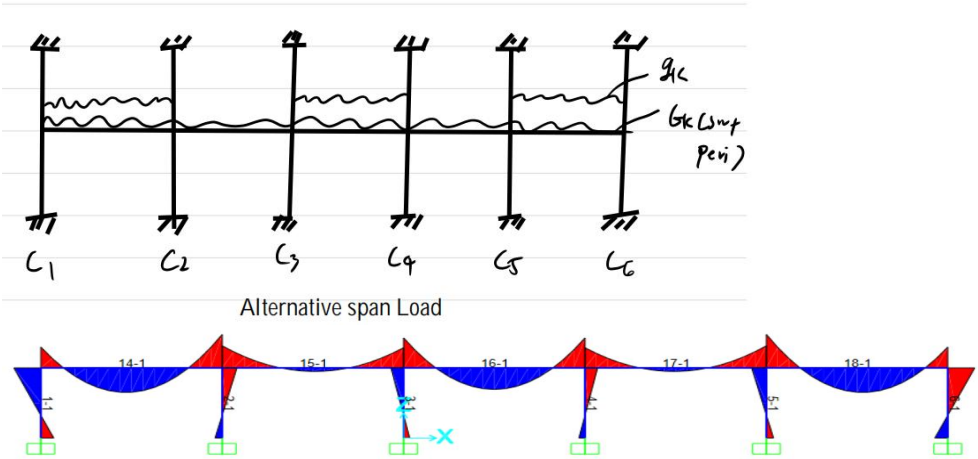


Figure 3- 3 EFM and computational moment diagram

MC2,Lower = 170 Kn.m

C2 in Grid Line B Y-direction

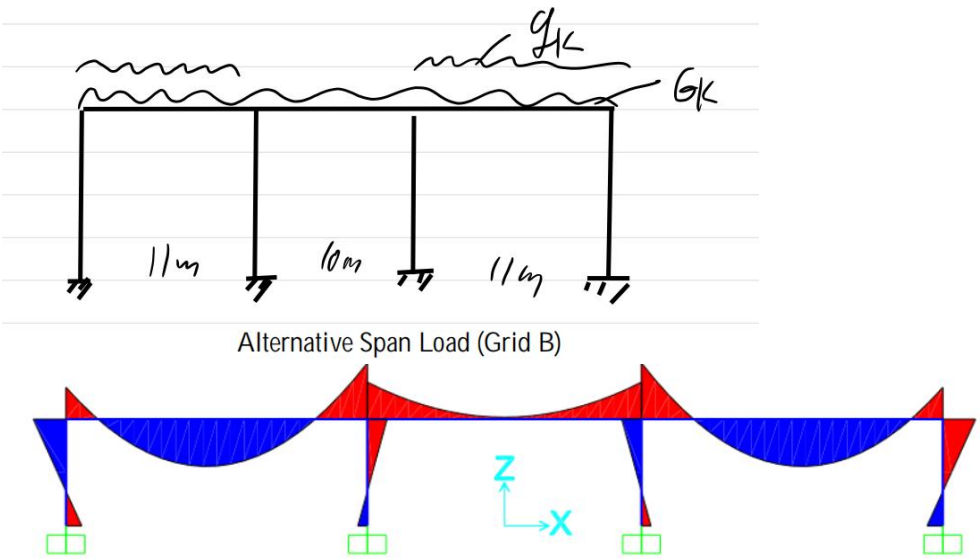


Figure 3- 4 EFM and computational moment diagram

MC2,Lower = 871 Kn.m

All Load Span Case

Hand calculations

C2 in Grid Line 13 X-direction

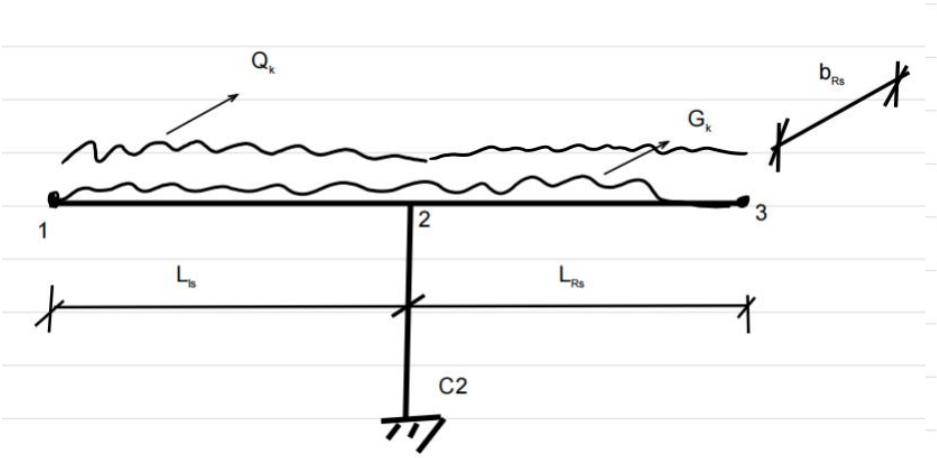


Figure 3- 5 EFM of All-Span-Loaded case for C2

Equivalent frame model is shown above

For calculating relative stiffness of column

$k_{relative}$=	$b_{lc}d_{lc}3/L_{lc}/(b_{lc}dl_c^3/L_{lc}+b_{uc}d_{uc}^3/L_{uc}+0.75b_{ls}d_{ls}^3/L_{ls}+0.75b_{rs}d_{rs}^3/L_{rs}$										
blc	=	500	hlc	=	500	Llc	=	4	bh^3/L	=	1.56E+07
buc	=	500	huc	=	500	Luc	=	0	bh^3/L	=	0
bls	=	5500	hls	=	350	Lls	=	8	bh^3/L	=	2.95E+07
brs	=	5500	hrs	=	350	Lrs	=	8	bh^3/L	=	2.95E+07
$k_{relative}$	=	0.261									

1st order moment using Exp. (6.10)

		Surface load			Line load(Kn/m)
Fully Loaded span	=	28	Kn/m^2	=	155
FEM21	=	826.1	Kn.m		Fully loaded in span 12
FEM23	=	826	Kn.m		Fully loaded in span 23
Mlower,xx	=	0	Kn.m		

Grid Line B Y-direction (the maximum moment for edge column is fully loaded)

C2 in Grid Line B Y-direction

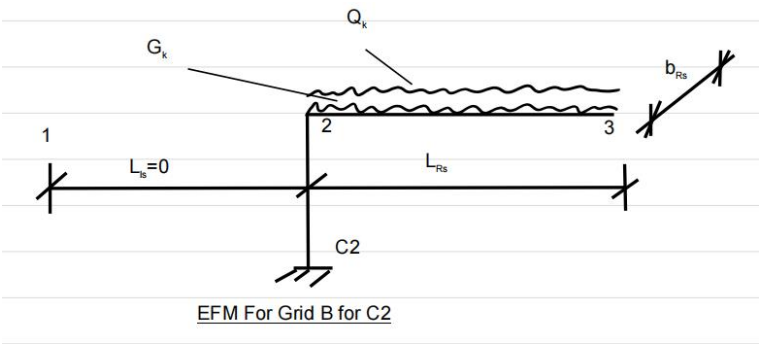


Figure 3- 6 EFM of All-Span-Loaded case for C2 (one side only)

For calculating relative stiffness of column

blc	=	500	hlc	=	500	Llc	=	4	bh3/L	=	1.56E+07
buc	=	500	huc	=	500	Luc	=	0	bh3/L	=	0
bls	=	8000	hls	=	350	Lls	=	0	bh3/L	=	0.00E+00
brs	=	8000	hrs	=	350	Lrs	=	11	bh3/L	=	3.12E+07
$k_{relative}$	=	0.401									

1st order moment using Exp. (6.10)

		Surface load			Line load(Kn/m)
Fully Loaded span	=	28	Kn/m^2	=	225
Gk load only span	=	10	Kn/m^2	=	78
FEM21	=	0	Kn.m		edge span length=0
FEM23	=	2272	Kn.m		Fully loaded in span 23
$M_{lower,xx}$	=	909.9	Kn.m		

Computational Results (Equivalent Frame Mode Method)

C2 in Grid Line 13 X-direction

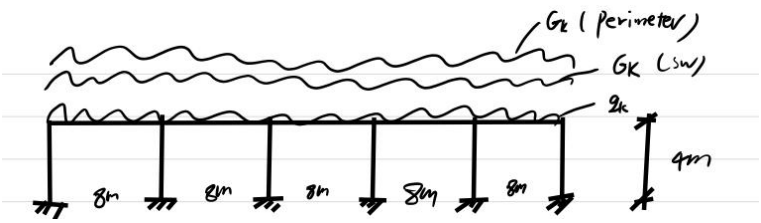


Figure 3- 7 Computational EFM and Moment diagram

$M_{C2,Lower}$ = 50 Kn.m

C2 in Grid Line B Y-direction

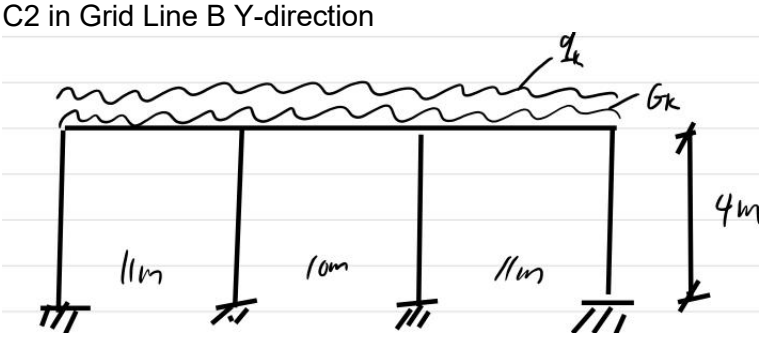


Figure 3- 8 Computational EFM and Moment diagram

$M_{C2,Lower}$ = 792 Kn.m

3.2.8 Design Moment Summary and comparison

	Alternative-span		Full-Span	
	Hand	FEA	Hand	FEA
	Med	Med	Med	Med
	Kn.m	Kn.m	Kn.m	Kn.m
Grid line13(xx)	141.03	170	0	50
Grid line B(yy)		871	909.90	792

As comparison, it can be found that Maximum Sagging Moment@Alternative Span Case and Maximum Hogging Moment@All load Span Case.

3.2.9 Summary of design forces in Column

Ned	3159	Kn	
Mxx	170	Kn.m/m	all span case
Myy	909.9	Kn.m/m	alternative span case

Note:Maximum out-of-balance moments have been calculated utilising variable actions to one side of the column exclusively in order to obtain the maximum 1st order moments in the column.
Conservatively, the impact on axial load has been disregarded.

3.2.10 Design: fire resistance

check validity of using Method A and Table 5.2a

EC2-1-2: 5.3.2, Table 5.2a

a) Check L0,fire

Column width in two direction	Clear height	=	4000	mm
For column at level 2 Assume fixed at bottom and roller at top	Effective length factor	=	0.7	
effective length of column in fire	$L_{0,fire}$	=	2800	mm
$L_{0,fire} \leq$ 3.0m	Check	=	ok	

EC2-1-2: Table 5.2a

b)Check amount of reinforcement ≤4%

For edge column@ Exposed on 3 sides(more than 1 side)	μ_{fi}	=	0.7	
2hrs fire Risistance-R120	b_{min}	=	450	
Minimum cover under fire	a_{min}	=	51	mm

Exp. (5.15)

c)Structural slenderness check--Check level with longest height--Retail level level

For edge column@ Exposed on 3 sides(more than 1 side)	k_1	=	0.08	
2hrs fire Risistance-R120	k_2	=	0.156	
Minimum cover under fire	k_{max}	=	0.156	
	L_0	=	3320	mm
radius of gyration i=(l/A)0.5=h/120.5	i	=	144.3	mm
Slenderness ratio	λ	=	23	
For limiting slenderness ratio λlim				
A=1/(1+0.2Φ_{ef}),Assume 0.7 as per default	A	=	0.7	
B=(1+200)0.5	B	=	1.1	
r_m= M_{O1}/M2,for conservative consideration,rm=0	r_m	=	0	
C=1.7-r_m	C	=	1.70	
n=Ned/Acfc	n	=	0.45	
λ_{lim}= 20ABC/$n^{0.5}$	λ_{lim}	=	39.20	
Check $\lambda<\lambda_{lim}$	Check	=	not slender	
As λ<λlim, 2nd order moments are not required				

Cl. 5.8.3.2(1)

C1.5.83.1(1)& NA.Exp.(5.13N)

3.2.11 Design using charts

	e_i	=	0.006		
e_o=h/30≥20mm	e_o	=	0.02		
Med = M + $e_iN_{ed} \geq e_oN_{Ed}$	$M_{ed,xx}$	=	189.7		
	$M_{ed,yy}$	=	929.6	Kn.m	
Critical $M_{ed,yy}$					
	M_{cri}	=	929.6	Kn.m	
	M_{cri}/bh^2fck	=	0.149		<0.25
	N_{Ed}/bhfck	=	0.253		
	number of bars	=	12		
	ϕbar	=	32	mm	
Vertical bars' spacing	Spacing	=	68.4	mm	<150mm Detailing

Cl. 5.8.9(2)

Cl.5.2.7
Cl.6.1(4)

Cl. 5.8.8.2(1)

Cl. 5.8.8.2(1), 6.1(4)

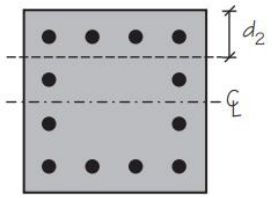
Figs C5a) to C5e) in (Worked Examples to

No need to restrain bars in face but good practice suggests alternate bars should be restrained

depth to centorid of reinforcement in half section	d2	=	101.8	mm	Eurocode 2: Volume 1, 2009)
Assuming 12 bar arrangement with H32s	d2/h	=	0.204		

Choice of chart based on d2/h

According to chart	$A_s f_{yk}/bhf_{ck}$	=	0.4	mm
	$A_{s,req}$	=	10000	mm^2



Provide	20 H32	
bar diameter	32	mm
Number of bars/m	20	
No.of spacing per face	5	
As,pro	16085	mm^2
Check	ok	

Figure 3- 9 detailing for column(as illustration)

3.2.12 Check bi-axial bending

For squared column,Slenderness: λy≈λz，Eccentricities，as h=b check ex/ey MEdy critical. Imperfection act in x direction

According to chart	e_x	=	53.81	mm	Cl.5.8.9 Cl.5.8.9(3)
	e_y	=	288	mm	
	e_y/e_z	=	0.187		Cl. 5.9.3(3), Exp. (5.38b)
in the range(5,0.2)	Check	=	yes		
Hence, design for bi-axial bending is necessary					

3.2.13 Design for bi-axial bending

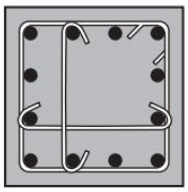
Check $(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$ — Cl. 5.9.3(4), Exp. (5.39)

	M_{Edx}	=	170	Kn.m
	M_{Edy}	=	909.9	Kn.m
Moment resistance. Using chart:	M_{Rdy}	=	170	Kn.m
	d2/h	=	0.20	
	$A_{s,pro}f_{yk}/bhf_{ck}$		0.64	
	N_{Ed}/bhf_{ck}		0.253	
	M_{Rd}/bh^2f_{ck}		0.23	
	M_{Rd}		1438	Kn.m
	N_{Rd}		14077	
	N_{Ed}/N_{Rd}		0.22	Notes to Exp. (5.39) Cl. 5.8.3(4)
	a		0.70	
$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$	Check		ok	
Therefore, use20H32 is ok				

3.2.14 Links

Minimum diameter of links	ϕ/4	=	8	mm	Cl. 9.5.3 & NA
Spacing either	0.6×20×ϕ	=	384	mm	
	0.6×h	=	300	mm	Cl. 9.5.3(3), 9.5.3(4)
	0.6×400	=	240	mm	
Spacing	Say	=	225	mm	
Provide links			H8@225		
use single leg on face bars both ways@225mm					

3.2.15 Design summary



Vertical reinforcement	20 H32	
Links	H8 @225	mm
Cover Cnom	55	
Column dimension	sq 500	
Concrete grade f_{ck}	50	mm^2

Figure 3- 10 Detailing for column

3.3 Design for G2-G6 columns (compacted)

Same procedures as calculate G1 column.
A compacted version will be provided.

3.3.1 Material properties

Items	f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m^3)	f_{yk} MPa	f_{yd} MPa
G2	50	28.33	37.28	25	500	434.8
G3	50	28.33	37.28	25	500	434.8
G4	50	28.33	37.28	25	500	434.8
G5	50	28.33	37.28	25	500	434.8
G6	50	28.33	37.28	25	500	434.8

3.3.2 Relative section dimensions

For the squared column

			G2	G3	G4	G5	G6	
Column width in two direction	b,h	=	500	500	500	500	1000	mm
Area of the column	A_c	=	3E+05	3E+05	3E+05	3E+05	1E+06	mm^2
Slab thickness		=	350	350	350	350	350	mm
Rebar diameter		=	32	26	20	20	32	mm

3.3.3 Cover

Items	d_{link} mm	$c_{min,b}$ mm	$c_{min,dur}$ mm	c_{min} mm	ΔC_{dev} mm	C_{nom} mm	a_{min} mm
G2	8	24	45	45	10	55	51
G3	8	18	45	45	10	55	51
G4	8	12	45	45	10	55	51
G5	8	12	45	45	10	55	51
G6	8	24	45	45	10	55	51

3.3.4 Summary of design forces in Columns

Items	N_{ed} Kn	M_{xx} Kn.m/m	M_{yy} Kn.m/m
G2	1117	247	255
G3	2182	190	5.1
G4	8473	27.8	44.9
G5	4243	33	16
G6	15818	60	69.5

3.3.5 Design: fire resistance

check validity of using Method A and Table 5.2a

	A) Check L0,fire				B) **Check amount of reinforcement ≤4%**		
Items	Clear Height mm	Effective L factor	$L_{0,fire}$ mm	$L_{0,fire}$≤ 3.0m	μ_{fi}	b_{min} mm	a_{min} mm
G2	4000	0.7	2800	√	0.7	450	51
G3	4000	0.7	2800	√	0.7	450	51
G4	4000	0.7	2800	√	0.7	450	51
G5	4000	0.5	2000	√	0.7	450	51
G6	4000	0.5	2000	√	0.7	450	51

C) Structural slenderness check--Check level with longest height--Retail level level

Items	k_1	k_2	k_{max}	L_0 Mm	i mm	λ	A	B	r_m	C	n	λ_{lim}	$\lambda<\lambda_{lim}$
G2	0.067	0.168	0.168	3320	144	23	0.7	1.1	0	1.7	0.16	65.93	√
G3	0.039	0.134	0.134	3320	144	23	0.7	1.1	0	1.7	0.31	47.17	√
G4	0.054	0.159	0.159	3320	144	23	0.7	1.1	0	1.7	1.2	23.94	√
G5	0.026	0.095	0.095	3320	144	23	0.7	1.1	0	1.7	0.6	33.83	√
G6	1.072	2.681	2.681	3320	288.7	11.5	0.7	1.1	0	1.7	0.56	35.04	√

Asλ<λlim, 2nd order moments are not required

3.3.6 Design using charts

Items	e_i m	e_o m	$M_{ed,x}$ x Kn.m	$M_{ed,y}$ y Kn.m	M_{cri} Kn.m	M_{cri}/bh^2f_{ck} <0.25	N_{Ed}/bhf_{ck}	No.bars	ϕ_{bar} mm	Spa mm	≤ 150
G2	0.008	0.02	256	264	264	0.042	0.089	12	32	68	√
G3	0.008	0.02	208	43.6	208	0.033	0.175	12	26	87	√
G4	0.008	0.02	170	170	170	0.027	0.678	12	20	88.5	√
G5	0.008	0.02	85	85	85	0.014	0.339	12	20	118	√
G6	0.008	0.02	316	316	316	0.006	0.316	12	32	211	√

3.3.7 Check bi-axial bending

For squared column,Slenderness: λy≈λz，Eccentricities，as h=b check ex/ey
MEdy critical. Imperfection act in x direction

Items	d_2 m	d_2/h	A_sf_{yk}/bhf_{ck}	$A_{s,req}$ mm^2	Provide	ϕ_{bar} mm	No.bars(per face)	As_{pro} mm^2	Check

G2	101.8	0.204	0.1	2500	20H32	32	5	16085	√
G3	105	0.225	0.1	2500	16H26	26	4	8495	√
G4	102.5	0.205	0.2	5000	16H20	20	4	5027	√
G5	112.3	0.225	0.1	2500	12H20	20	3	3770	√
G6	149.2	0.149	0.1	10000	16H32	32	4	12868	√

Items	e_x	e_y	e_y/e_z	∈(5,0.2)
	mm	mm		
G2	221.1	228.3	0.969	√
G3	87.08	2.337	37.25	√
G4	3.281	5.23	0.619	√
G5	7.778	3.771	2.063	√
G6	3.793	4.394	0.863	√

3.3.8 Design for bi-axial bending

Check $(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$

Items		G2	G3	G4	G5	G6	
M_{Edx}	=	247	190	27.8	33	60	Kn.m
M_{Edy}	=	255	5.1	44.9	16	69.5	Kn.m
M_{Rdy}	=	7247	75.1	27.8	16	60	Kn.m
d2/h	=	0.2	0.21	0.21	0.22	0.15	
$A_{s,pro}f_{yk}/bhf_{ck}$	=	0.64	0.34	0.20	1.15	0.13	
N_{Ed}/bhf_{ck}	=	0.089	0.175	0.678	0.339	0.316	
M_{Rd}/bh^2f_{ck}	=	0.2	0.1	0.1	0.08	0.11	
M_{Rd}	=	1250	625	625	500	5500	Kn.m
N_{Rd}	=	14077	10777	9269	8722	33928	Kn
N_{Ed}/N_{Rd}	=	0.08	0.2	0.91	0.49	0.47	
a	=	0.46	0.67	1.86	1.14	1.11	
Check $(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$		ok	ok	ok	ok	ok	

3.3.9 Links

Items	ϕ/4	0.6×20×ϕ	0.6×h	0.6×400	Spacing
	mm	mm	mm	mm	mm
G2	8	384	300	240	225
G3	7	312	300	240	225
G4	5	240	300	240	225
G5	5	240	300	240	225
G6	8	384	300	240	225

3.3.10 Design summary

Items	G2	G3	G4	G5	G6	
Vertical reinforcement	20 H32	16 H26	16 H20	12 H20	16H32	
Links	H8@225	H8@225	H8@225	H8@225	H8@225	mm
Cover Cnom	55	55	55	55	55	
Column dimension	sq 500	sq 500	sq 500	500	1000	
Concrete grade fck	50	50	50	50	50	mm^2

4 Inclined column design

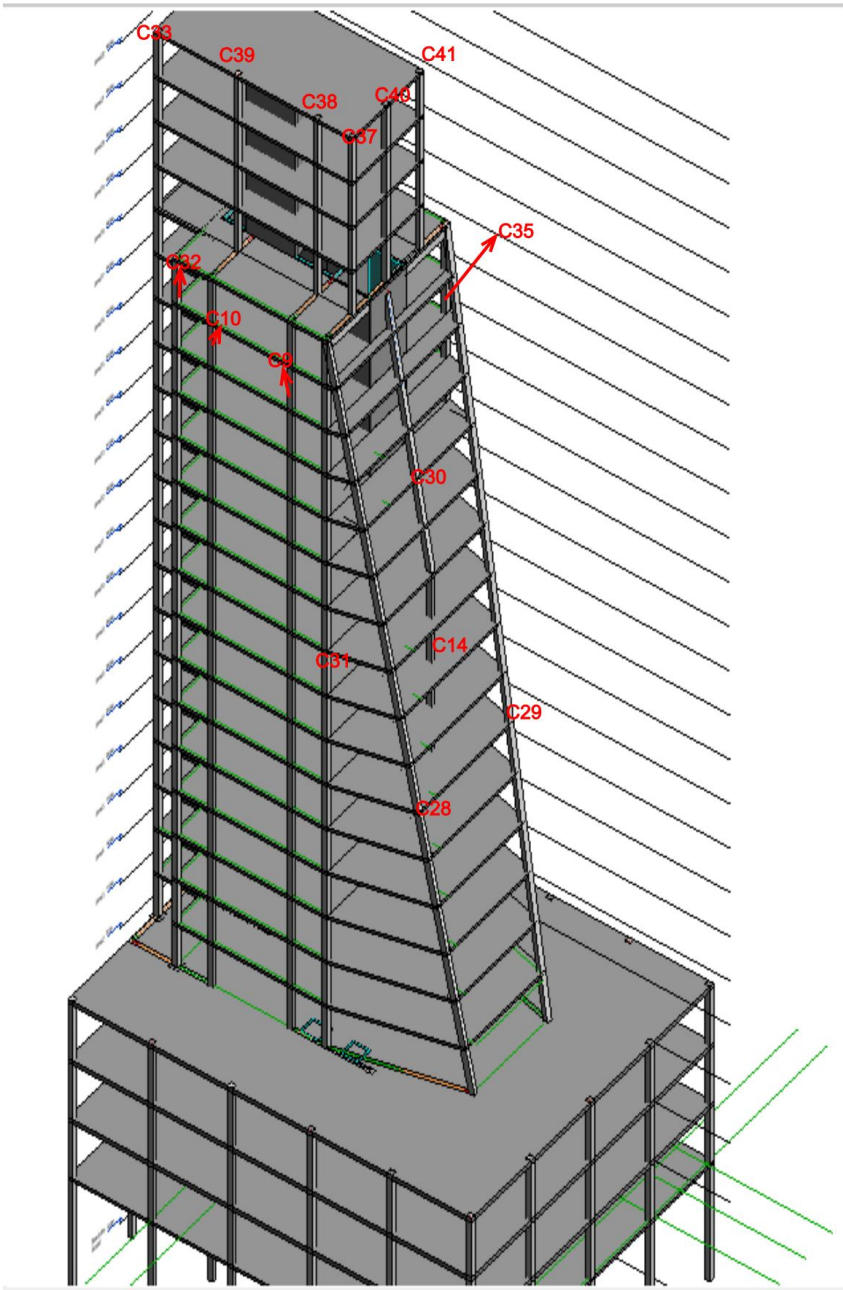


Figure 4- 1 Labels for inclined columns

Figure 4- 1 shows the inclined columns are C28, C29 and C30.

4.1 Design for inclined columns

Design calculations for inclined columns are the same with the normal design.

4.1.1 Material properties

Items	f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m^3)	f_{yk} MPa	f_{yd} MPa
C29	50	28.33	37.28	25	500	434.8
C28	50	28.33	37.28	25	500	434.8

C30	50	28.33	37.28	25	500	434.8

4.1.2 Relative section dimensions

For the squared column

Column width in two direction	b,h	=	500	mm
Area of the column	A_c	=	3E+05	mm^2
Slab thickness		=	300	mm
Rebar diameter		=	32	mm

4.1.3 Cover

Items	d_{link} mm	$c_{min,b}$ mm	$c_{min,dur}$ mm	c_{min} mm	ΔC_{dev} mm	C_{nom} mm	a_{min} mm
C29	8	24	45	45	10	55	51
C28	8	24	45	45	10	55	51
C30	8	24	45	45	10	55	51

4.1.4 Summary of design forces in Columns

Items	N_{ed} Kn	M_{xx} Kn.m/m	M_{yy} Kn.m/m
C29	3287	83.17	100.7
C28	3218	95.08	76.6
C30	2458	182	47.9

4.1.5 Design: fire resistance

check validity of using Method A and Table 5.2a

	B) Check L0,fire				D) **Check amount of reinforcement ≤4%**		
Items	Clear Height	Effective L factor	$L_{0,fire}$	$L_{0,fire}$≤ 3.0m	μ_{fi}	b_{min}	a_{min}
	mm		mm			mm	mm
C29	3106	0.5	1553	√	0.7	450	51
C28	3106	0.5	1553	√	0.7	450	51
C30	3090	0.5	1545	√	0.7	450	51

	E) **check e≤e_{max} =0.15h**		
Items	e	e_{max}	e≤e_{max}
	mm	mm	
C29	27.17	75	√
C28	29.55	75	√
C30	74.04	75	√

F) Structural slenderness check--Check level with longest height--Retail level level

Items	k_1	k_2	k_{max}	L_0 Mm	i mm	λ	A	B	r_m	C	n	λ_{lim}	λ<λ_{lim}
C29	0.127	0.248	0.248	2578	144	17.9	0.7	1.1	0	1.7	0.49	38.44	√
C28	0.127	0.248	0.248	2578	144	17.9	0.7	1.1	0	1.7	0.49	38.44	√
C30	0.127	0.248	0.248	2565	144	17.7	0.7	1.1	0	1.7	0.35	44.44	√

Asλ<λlim, 2nd order moments are not required

4.1.6 Design using charts

Items	e_i m	e_o m	$M_{ed,x}$ x Kn.m	$M_{ed,y}$ y Kn.m	M_{cri} Kn.m	M_{cri}/bh^2f_{ck}	N_{Ed}/bhf_{ck}	No.bars	ϕ_{bar} mm	Spa mm	≤150
C29	0.006	0.02	117	99.2	117.2	0.019	0.28	12	20	118	√
C28	0.006	0.02	116	97.4	115.8	0.019	0.257	12	20	118	√
C30	0.006	0.02	197	63.7	197.8	0.032	0.197	12	20	118	√

4.1.7 Check bi-axial bending

For squared column,Slenderness: λy≈λz，Eccentricities，as h=b check ex/ey
MEdy critical. Imperfection act in x direction

Items	d_2 m	d_2/h	A_sf_{yk}/bhf_{ck}	$A_{s,req}$ mm^2	Provide	ϕ_{bar} mm	No.bars(per face)	As_{pro} mm^2	Check
C29	112.3	0.225	0.1	2500	12H20	20	3	3770	√
C28	112.3	0.225	0.1	2500	12H20	20	3	3770	√
C30	112.3	0.225	0.1	2500	12H20	20	3	3770	√

Items	e_x mm	e_y mm	e_y/e_z	∈(5,0.2)
C29	27.17	21.89	1.241	√
C28	29.55	23.8	1.241	√
C30	74.04	19.49	3.8	√

4.1.8 Design for bi-axial bending

Check $(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$

Items		C29	C28	C30	
M_{Edx}	=	95.08	95.08	182	Kn.m
M_{Edy}	=	76.6	76.6	47.9	Kn.m
M_{Rdy}	=	76.6	76.6	47.9	Kn.m
d2/h	=	0.22	0.22	0.22	
$A_{s,pro}f_{yk}/bhf_{ck}$	=	0.15	0.15	0.15	
N_{Ed}/bhf_{ck}	=	0.28	0.28	0.197	
M_{Rd}/bh^2f_{ck}	=	0.1	0.1	0.1	
M_{Rd}	=	625	625	625	Kn.m
N_{Rd}	=	8722	8722	8722	Kn
N_{Ed}/N_{Rd}	=	0.4	0.4	0.28	
a	=	1	1	1	
Check$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$		ok	ok	ok	

4.1.9 Links

Items	ϕ/4	0.6×20×ϕ	0.6×h	0.6×400	Spacing
	mm	mm	mm	mm	mm
C29	5	240	300	240	240
C28	5	240	300	240	240
C30	5	240	300	240	240

4.1.10 Design summary

Items	C29	C28	C30	
Vertical reinforcement	12 H20	12 H20	12 H20	
Links	H5 @240	H5 @240	H5 @240	mm
Cover Cnom	55	55	55	
Column dimension	sq 500	sq 500	sq 500	
Concrete grade fck	50	50	50	mm^2

4.1.11 Additional reinforcement design for slab to resist pull and push forces

FEA Method

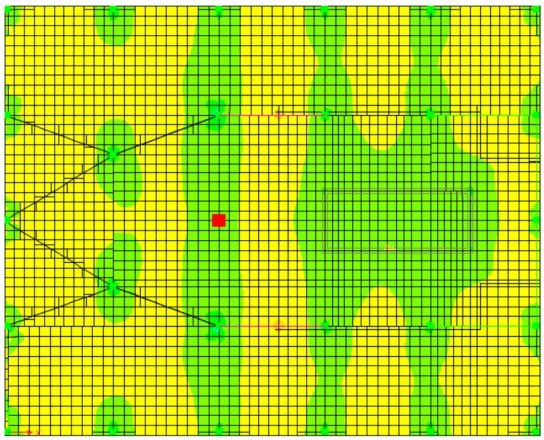


Figure 4- 2 S11 Top stress

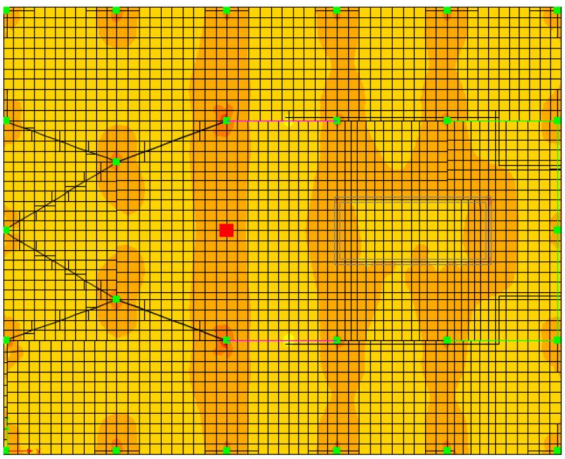


Figure 4- 3 S11 bottom stress

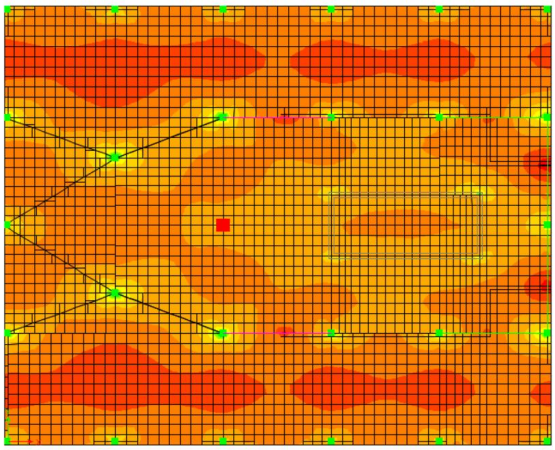


Figure 4- 4 S22 Top stress

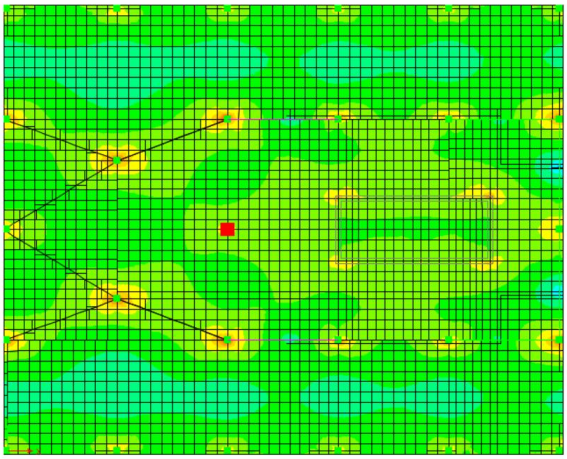


Figure 4- 5 S22 bottom stress

σ_S^T- Stress as top(ETABS), σ_S^B- Stress at bottom(ETABS) and σ_{normal}- the normal stress used to design

For inclined column C29(critical one)

	σ_S^T MPa	σ_S^B MPa	σ_{normal} MPa	depth mm	fyd MPa	As_{req} mm_2	provide	As_{pro} mm^2	Check
S11(Tension)	24	-22.51	1.49	350	435	1199	H32@200 M	1280	√

	σ_S^T MPa	σ_S^B MPa	σ_{normal} MPa	f_{ck} MPa	ρ Grid B	f_{yk} mm_2	stress	Check
S22 (Compression)	24	-25	-1	350	2.23%	500	28.9	√

$$\text{stress} = 0.44f_{ck} + \frac{\rho}{100}\left(0.67f_{yk} - 0.44f_{ck}\right)$$

Therefore, H32@200 bars will be placed in the middle of the slab in x direction and the applied compression stress is far more smaller than compression resistance in y direction.

Hand calculation-force equilibrium

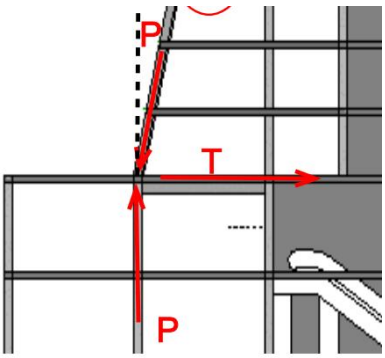


Figure 4- 6 W-E direction(Y) α≈ 75°

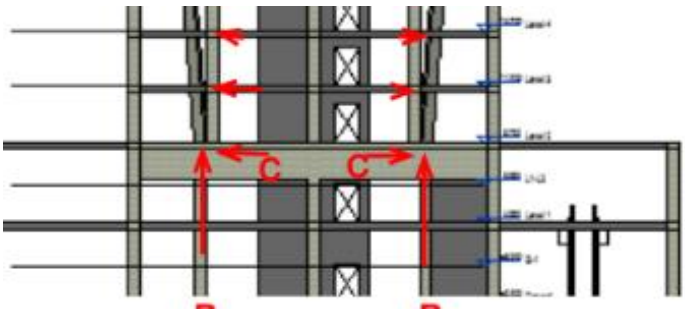


Figure 4- 7 N-S direction（X） α≈ 86.33°

The applied axial force obtained from FEA model, the angle is fixed in two direction, according to force equilibrium, tension can be calculated:

At level 2

	P_{upper} Kn	cosα MPa	T Kn	As_{req} mm^2	provide	As_{pro} mm_2	Check
W-E(Tension)	3218	0.26	833	1915	H32@125 M	2048	√

	Kn	MPa	Kn	MPa	In Grid B	MPa	
N-S(Compression)	3218	0.064	-205	-0.588	2.23%	29.98	√

5 Transfer components design

5.1 General introduction and analysis

As the spacing requirements in retail level and car parking and also the layout change is garden level, column grid will change between level 2 and level 3 and level 18 and level 19.
The high-rise building which means the loads transfer to level 2 will be quite high, for resisting bending and deflection, the transfer components' depth tend to high. However, too high depth of transfer beam may looks not pleased. Hence, for transfer components which requires high depth, them can be designed as Beam wall by using Strut and Tie Model.
Meanwhile, because the layout of the building is symmetry in level 2, therefore, for transfer components with same or similar load conditions and be designed as a group.

Layout and dimension Group division for transfer components

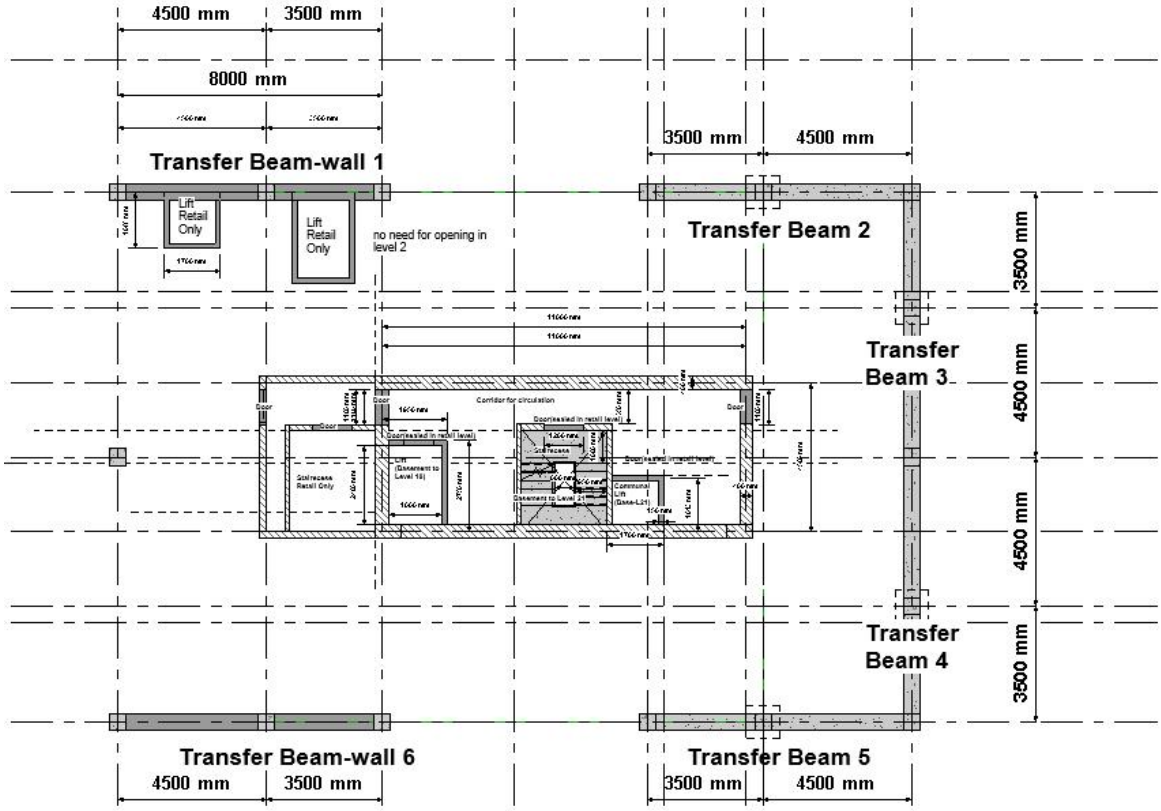


Figure 5- 1 Labeled transfer components in level 2

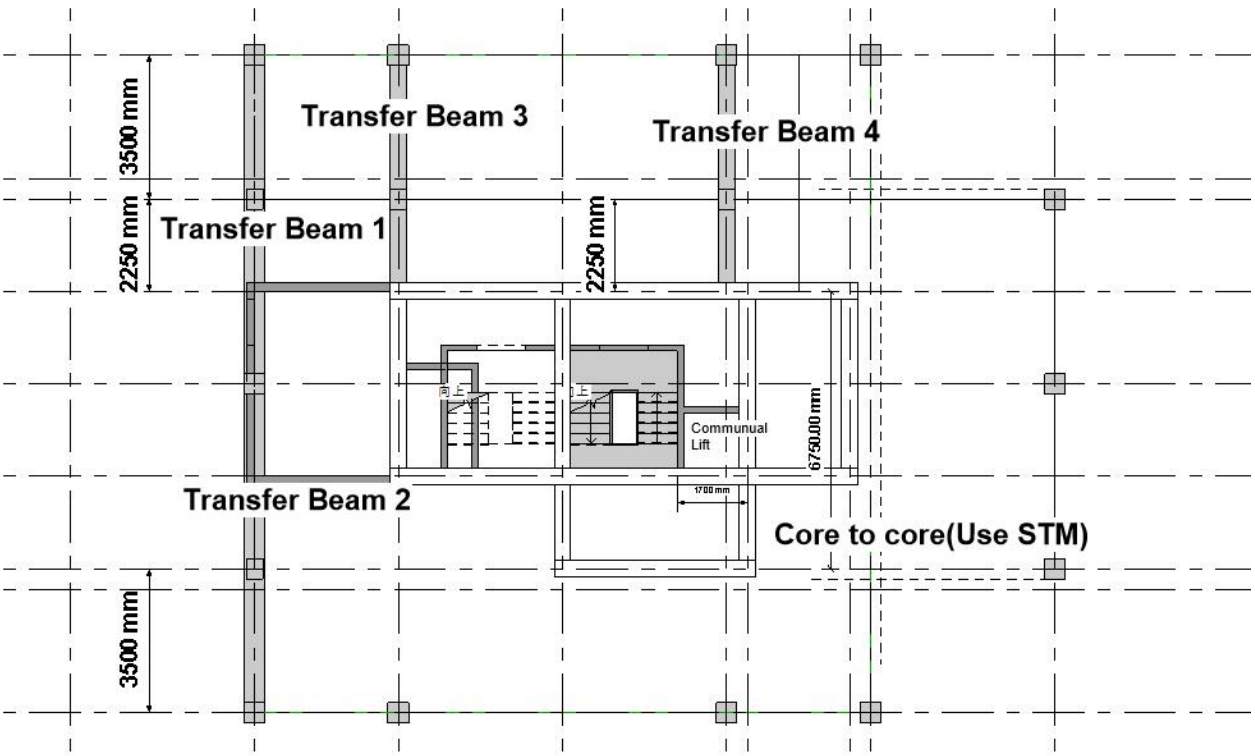


Figure 5- 2 Labeled transfer components in level 18

For simplicity and convenience of construction, the design for columns can be devided in groups

For Level 2			Length		Width	Height
		Design	Span1	Span2		
Group	Type	Number	(m)	(m)	(m)	(m)
G1	STM	Trans 1	4.5	3.5	0.5	4
		Trans 6	4.5	3.5		
G2	Beam	Trans 2	3.5	4.5	0.5	1.9
		Trans 5	3.5	4.5		
G3	Beam	Trans 3	4.5	3.5	0.5	2
		Trans 4	3.5	4.5		

*Transfer 3 and 4 are about N-S axis symmetry

For Level 18		Design	Length		Width	Height
			Span1	Span2		
Group	Type	Number	(m)	(m)	(m)	(m)
G1	Beam	Trans 1	3.5	2.25	0.5	1.2
		Trans 2	2.25	3.5		
G2	Beam	Trans 3	3.5	2.25	0.4	1.2
G3	Beam	Trans 4	3.5	2.25	0.4	1

5.1.1 General calculation parameters

Fixed Values

The recommended for design working life of 50 years	Structural Class	=	S4	mm
As for transfer beam is not exposed in environment,Therefore, Exposure class should be classified as X0 for W_{max}	Exposure Class	=	X0	
	$C_{min,dur}$	=	45	mm

	W_{max}	=	0.4	mm

5.1.2 Load Condition and Actions

		Surface load (Kn/m²)
Dead		
Finishes	=	1
Perimeter Load	=	ignore
Live		
For Retail	=	5
Podium Deck	=	10
Residential	=	2.5
Garden	=	10
Circulation	=	5

EC2 6.5 Part1-1

5.2 Group 1 design at level 2 (STM type detailed design)

5.2.1 Material Properties

f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m³)	f_{yk} MPa	f_{yd} MPa	$u'f_{cd}$ MPa	$0.85u'f_{cd}$ MPa	$0.6u'f_{cd}$ MPa
50	28.33	37.28	25	500	434.8	26.67	22.67	16

5.2.2 Section dimensions

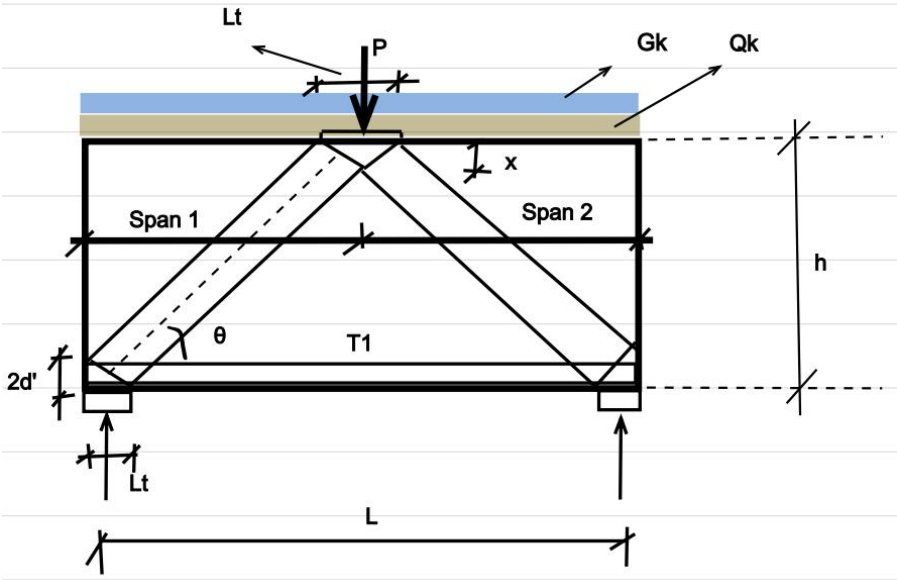


Figure 5- 3 STM for G1 component (L2)

For the beam wall								Column	Width			Area
Span 1	=	4.5	m	d'	=	100	mm	Number	X		Y	
Span 2	=	3.5	m	x	=	200	mm		mm		mm	m²
Total length L	=	8	m	twall	=	500	mm	C37	400	500	500	0.2
Wall Height h	=	4	m					C31	500	500	500	0.25

5.2.3 For calculation of Point load from floor above

Level Number	Height (m)	Loading width X(m)	Y(m)	Loading Area(m²)	Slab Thickness(mm)

Load take	Level 22	3	1.75	2.25	3.94	300
Upper	Level 21	3	1.75	2.25	3.94	300
Wall	Level 20	3	1.75	2.25	3.94	300
	Level 19	3	1.75	2.25	3.94	300
Load taken	Level 18	3	3.125	2.9	9.06	300
Lower	Level 17	3	3.125	2.9	9.06	300
Wall	Level 16	3	3.125	2.9	9.06	300
Load area	Level 15	3	3.125	2.9	9.06	300
Average	Level 14	3	3.125	2.9	9.06	300
	Level 13	3	3.125	2.9	9.06	300
	Level 12	3	3.125	2.9	9.06	300
	Level 11	3	3.125	2.9	9.06	300
	Level 10	3	3.125	2.9	9.06	300
	Level 9	3	3.125	2.9	9.06	300
	Level 8	3	3.125	2.9	9.06	300
	Level 7	3	3.125	2.9	9.06	300
	Level 6	3	3.125	2.9	9.06	300
	Level 5	3	3.125	2.9	9.06	300
	Level 4	3	3.125	2.9	9.06	300
	Level 3	3	3.125	2.9	9.06	300
	Level 2	4.35	8	6.875	55	350
	Level 1	4.35	8	6.875	55	350

	Surface Load		Column SW	axial load	
Level	Gk	Qk	Load	Per floor	Accumulated
Number	Kn/m^2	Kn/m^2	Kn	Kn	Kn
Level 22	7.5	2.5	15	74.88	74.88
Level 21	7.5	2.5	15	74.88	149.77
Level 20	7.5	2.5	15	74.88	224.65
Level 19	7.5	10	15	119.2	343.83
Level 18	7.5	2.5	18.75	151.1	494.88
Level 17	7.5	2.5	18.75	151.1	645.94
Level 16	7.5	2.5	18.75	151.1	796.99
Level 15	7.5	2.5	18.75	151.1	948.05
Level 14	7.5	2.5	18.75	151.1	1099.10
Level 13	7.5	2.5	18.75	151.1	1250.16
Level 12	7.5	2.5	18.75	151.1	1401.21
Level 11	7.5	2.5	18.75	151.1	1552.27
Level 10	7.5	2.5	18.75	151.1	1703.32
Level 9	7.5	2.5	18.75	151.1	1854.38
Level 8	7.5	2.5	18.75	151.1	2005.43
Level 7	7.5	2.5	18.75	151.1	2156.48
Level 6	7.5	2.5	18.75	151.1	2307.54
Level 5	7.5	2.5	18.75	151.1	2458.59
Level 4	7.5	2.5	18.75	151.1	2609.65
Level 3	7.5	2.5	18.75	151.1	2760.70
Level 2	8.75	6.25	18.75	151.1	
Level 1	8.75	6.25	18.75	151.1	

5.2.4 Design Loads

Factored

Hand calculated point load from above has been compared with the computational model. Considering ULS and an all-span-loaded arrangement, the factored UDL along the span can be calculated and the reaction from the right and left supports can be simply calculated. The results

are shown below:

	Axial Load P (Kn)	Line load G_{ed} (Kn/m)	Q_{ed} (Kn/m)	UDL_{ed} (Kn/m)	Reaction R_{left} (Kn)	R_{right} (Kn)
Hand calculation	2761	81.21	64.45	145.7	1790	2136
Computational	2356					
Erro	14.6%					

The error is 14.6%, which is acceptable.

5.2.5 Choose bearing plate width

Minimum required widths of the bearing plates are

Length of bearing plate@ Top	L_t	>	345.08	Say	=	400	mm
Length of bearing plate@ Column(left)	L_{bl}	>	223.80	Say	=	300	mm
Length of bearing plate@ Column(right)	L_{br}	>	266.94	Say	=	300	mm

5.2.6 Stress check and flexural tension bar design

		cotθ	=	0.49	Kn.m	
		θ	=	1.115=	63.9	degree
Moment resistance. Using chart:		W_{bl}	=	357.4	mm	
		W_t	=	357.4	mm	
Strut Force		C	=	1537	Kn	
Tie force		T_1	=	676.1	Kn	
Flextural compression stress		σ_{c0}	=	6.761	MPa	<υ'fcd
Strut stress at top node		σ_{st}	=	13.8	MPa	<0.6υ'fcd
Strut stress at bottom nodes	Left side	$\sigma_{sb,left}$	=	9.202	MPa	<0.6υ'fcd
	Right side	$\sigma_{sb,right}$	=	9.202	MPa	<0.6υ'fcd
The required area of flextural tension bar		$A_{s,t,req}$	=	1555	mm^2	
Provide		U-bars 3 H28				
Therefore, use20H32 is ok		Bar diameter	=	20	mm	
legs(each of area)			=	6		
		As,pro	=	1885	mm^2	
		Check		Satisfy		

UK National Annex to EC2

5.2.7 Minimum horizontal and vertical reinforcement and reinforcement design

Additionally, the uk National Annex to EC2 requires a minimum area of horizontal and vertical web reinforcement of area (0.002Ac) in each face both horizontally and vertically.

Ac	=	4000000	mm^2	Provide	H16@200	
$A_{s,min}$	=	8000	mm^2	bar diameter	16	mm
$A_{s,min,req}$	=	1000	mm^2/m	Spacing	200	mm
				Number of bars/m	5	
				$A_{s,pro}$	1005.31	mm^2/m
				Check		Satisfy

5.3 Group 2 design at level 2 (Transfer beam type detailed design)

5.3.1 Material Properties

f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m^3)	f_{yk} MPa	f_{yd} MPa
50	28.33	37.28	25	500	434.8

5.3.2 Section dimensions and simplified model

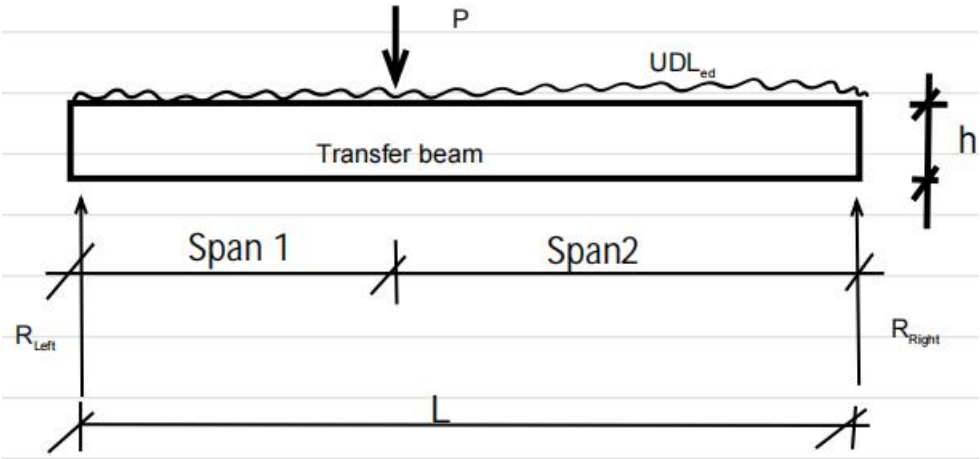


Figure 5- 4 Transfer beam design for G2 components (L2)

For the beam wall								Column	Width		Area
Span 1	=	3.5	m	h	=	1900	mm	Number	X	Y	
Span 2	=	4.5	m	b_{beam}	=	500	mm		mm	mm	m^2
Total length L	=	8	m					C10(left)	500	500	0.25
								C11(right)	500	500	0.25

5.3.3 Design axial Load

The calculation process for axial load is similar with G1 calculation which shown above. Therefore, the detailed axial load calculation do not provided in here.

Factored

	Axial Load P(Kn)	Line load G_{ed} (Kn/m)	Q_{ed} (Kn/m)	UDL_{ed} (Kn/m)	Reaction R_{left} (Kn)	R_{right} (Kn)
Hand calculation	1668	38.39	30.47	68.86	1213	1005
Computational	1321					
Erro	20.8%					

Cl. 4.4.1.2(3)

Cl. 4.4.1.2(3) Exp. (4.1)

5.3.4 Cover

assuming 32 mm diameter reinforcement	$c_{min,b}$	=	32	mm	
Minimum cover due to environmental conditions.For S4 and XS1	$c_{min,dur}$	=	45	mm	
C_{min}=max($C_{min,b}$;$C_{min,dur}$;10mm)	fixed value	=	10	mm	
	c_{min}	=	45	mm	
C_{nom} =C_{min}+ ΔC_{dev}	ΔC_{dev}	=	10	mm	
	C_{nom}	=	55	mm	
For 2 hours resistance Minimum thickness for cover	a_{min}	=	51	mm	Satisfy
Minimum width for beam	b_{min}	=	500	mm	Satisfy

EC2-1-2: 5.6.3(1), Table 5.6

EC0: Exp.(6.10a)

Cl. 5.1.3(1) & NA Table NA.1 (option b)

5.3.5 Load combination and arrangement

Most Critical Load Combination is

N = 1.35 × Gk + 1.5 × Qk

For Simply-supported beam, the critical load arrangement is all-span-loaded load cases

5.3.6 Designed moment and shear forces

M_{max} Kn.m	$V_{ed,max}$ Kn
4523	1213

5.3.7 Flextural design

Assumed links diameter	d_{links}	=	10	mm	
Effective depth	d_{eff}	=	1819	mm	
Flexture in span	k	=	0.055		
For simply supported(no redistribution)	K'	=	0.208		
false→Compression reinforcement requirement True→no compression reinforcement required	k>k'?	=	True		
Lever arm	Z	=	1727	mm	
	$A_{s,t,req}$	=	51	mm^2	Satisfy
Provide 8H32 B					
	bar diameter	=	32	mm	
	Number of bars	=	8		
	$A_{s,pro}$	=	6434	mm^2	

Spacing check

$$\sigma_s = (f_{yk}/\gamma_s)(A_{s,req}/A_{s,prov})(\text{ SLS loads/ULS loads})\ (1/\delta)$$
$$= f_{yd} \times (A_{s,req}/A_{s,prov}) \times (g_k + \psi_2 q_k)/(\gamma_G g_k + \gamma_Q q_k)(1/\delta)$$

C1.7.3.3(2)
C1.7.3.1(5)& NA
Table 7.2N& NA

Spacing of outer bars		=	338	mm	
	Spacing	=	48.29	mm	
Stress under quasi-permanent loading:					
	σ_s	=	204.2	MPa	
Maxiumu carck width	W_{max}	=	0.4		
Maximum bar sizing	d_{max}	=	32	mm	
	$Spacing_{max}$	=	300	mm^2	Satisfy
Therefore, provide 8H32 B is ok					
	bar diameter	=	32	mm	
	Number of bars	=	8		
	As,pro	=	6434	mm^2	

5.3.8 Deflection check

Check the span-to-depth ratio.

Appendix B, (Worked Examples to Eurocode 2: Volume 1, 2009)

Table 7.4N & NA

	ρ_0	=	0.71%
Basic span: effective depth ratio for	ρ	=	0.71%
Factor for simply-supported	K	=	1
As L>7m	reduction factor	=	0.875
	$(l/d)_{premissible}$		19.42
	$d>d_{permis}$		Satisfy

5.3.9 Shear design

Consevative consideration	Ved=Vmax	=	1213	KN
ved=Ved/bh	ved	=	1.277	MPa
maximum shear capacity:	As cotθ	=	2.5	
For simply supported(no redistribution)	vRd,max	=	5.52	
	vRd,max>ved	=	Satisfy	
Shear reinforcement				
Assuming z=0.9d	z	=	1637	mm

Asw/s≥Ved/(z×fywd×cotθ)	Asw/s	≥	0.68	
Minimum shear links,Asw,min/s=0.08bwfck0.5/fyk	Asw,min/s	=	0.566	
Provide H10@250				
	bar diameter	=	10	mm
	As,2legs	=	157.08	
	Spacing	=	250	mm
	Asw/s,pro	=	0.628	
Asw,pro/s≥Asw,min/s		=	Satisfy	
Spacing<$Spacing_{max}$, ok	$Spacing_{max}$	=	1364	mm

Cl. 6.2.1(8)

Table C7, (Worked Examples to Eurocode 2: Volume 1, 2009)

5.3.10 Summary of design

Cl.6.2.3(1)
Cl.6.2.3(3)
Exp.(6.8)

CL.9.2.2(5)

Cl. 9.2.2(6)

Figure 5- 5 Summary for transfer beam design

Flextural reinforcements	
Top	Bottom
Do not need	8H32B
Shear reinforcements	
H10@250	

5.3.11 Detailing check

a)Minimum areas

Cl. 9.2.1.1
Table 3.1

$f_{ctm} = 0.30 \times f_{ck} 0.666$	f_{ctm}	=	4.061	MPa
Width of tension zone	bt	=	500	mm
$A_{s,min} = 0.26(f_{ctm}/f_{yk})b_t d \geq 0.0013 b_t d$	$A_{s,min}$	=	1921	mm^2
For simply supported(no redistribution)	$A_{s,pro}$	=	6434	mm^2
	Check	=	Satisfy	

b)Curtailment of main bars

Bottom: Curtail

75% main bars 0.08L from end support	0.08L	=	640	Say	650	mm
70% main bars 0.3L- al From left support (al=1.125d)		=	353.6	Say	1000	mm

At support

25% of As to be anchored at supports	25% As	=	1506	mm^2
Provide 4H22				
	bar diameter	=	22	mm
For simply supported(no redistribution)	Number of legs/m	=	4	
	As,pro	=	1521	mm^2
	Check		Satisfy	

b)Summary of reinforcement details

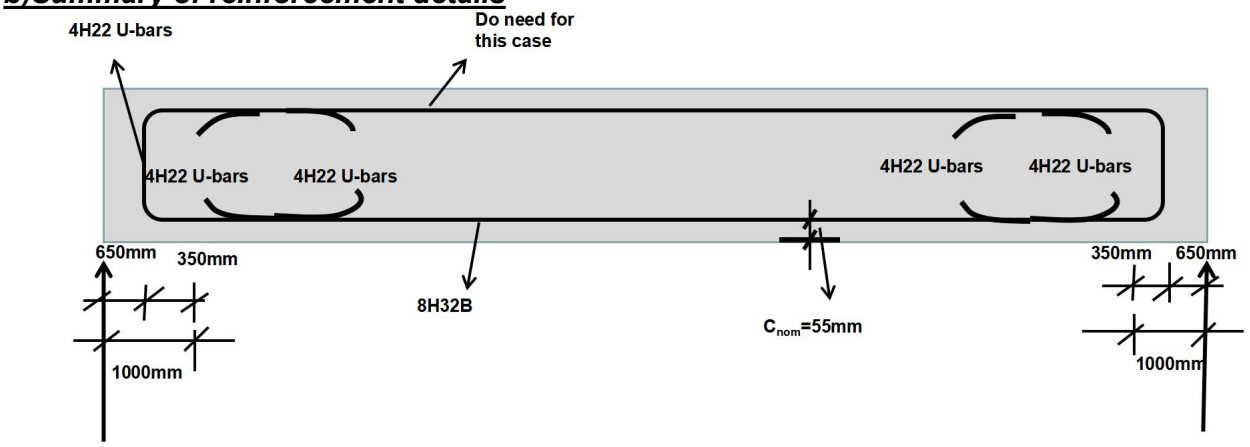


Figure 5- 6 Transfer beam detailing design (as illustration)

How to: Detailing

CL. 9.2.1.2.(1), 9.2.1.4(1)& NA CL.9.2.1.5(1)

5.4 Design transfer beams at level 18 and G3 for Level 2

G2,3 transfer beams are loaded on the core one side , as the interaction face depends on the thickness of wall and width of the beams and the interaction face in x and y direction is the same(400×400mm) . Therefore, idealize the core side as a 400×400 a column.
The calculation for axial load above and maximum bending and shear force are the same as before.

5.4.1 Material properties

	f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m^3)	f_{yk} MPa	f_{yd} MPa
G1(L18)	50	28.33	37.28	25	500	434.8
G2(L18)	50	28.33	37.28	25	500	434.8
G3(L18)	50	28.33	37.28	25	500	434.8
G3(L2)	50	28.33	37.28	25	500	434.8

5.4.2 Cover

Items	$c_{min,b}$ mm	$c_{min,dur}$ mm	c_{min} mm	ΔC_{dev} mm	C_{nom} mm	a_{min} mm	b_{min} mm
G1(L18)	32	45	45	10	55	51	500
G2(L18)	32	45	45	10	55	51	500
G3(L18)	32	45	45	10	55	51	500
G3(L2)	32	45	45	10	55	51	500

5.4.3 Section dimensions and simplified model

For beam wall						For column					
						C_{left}		C_{right}		A_{left}	A_{right}
Items	Span 1 m	Span 2 m	L_{tot} m	h mm	b_{beam} mm	x mm	y mm	x mm	y mm	m^2	m^2
G1(L18)	3.5	4.5	8	1200	500	400	500	500	500	0.2	0.25
G2(L18)	3.5	2.25	5.75	1200	400	400	500	500	500	0.2	0.25
G3(L18)	3.5	2.25	5.75	1000	400	400	500	500	500	0.2	0.25
G3(L2)	4.5	3.5	8	2000	500	500	500	500	500	0.25	0.25

5.4.4 Designed moment and shear forces

	M_{max} Kn.m	$V_{ed,max}$ Kn
G1(L18)	2675	637.4
G2(L18)	1600	711.2
G3(L18)	1475	655.8
G3(L2)	4799	1371

5.4.5 Flexural design

Items	d_{links} mm	d_{eff} mm	k mm	K' mm	k>k'? mm	z mm	$A_{s,t,req}$ mm^2	Provide	$A_{s,pro}$ mm^2
G1(L18)	10	1119	0.085	0.208	true	1027	5990	10H32 B	8042.48
G2(L18)	10	1123	0.064	0.208	true	1056	3487	8H25 B	3926.99
G3(L18)	10	921	0.087	0.208	true	843.8	4022	8H28 B	4926.02
G3(L2)	10	1919	0.052	0.208	true	1826	6043	8H32 B	6434

5.4.6 Spacing check

Items	S_{outer} mm	S mm	σ_s MPa	W_{max} mm	d_{max} mm	S_{max} mm	Check
G1(L18)	338	37.56	192.5	0.4	32	300	√
G2(L18)	245	35	220	0.4	30	275	√
G3(L18)	242	34.57	211.1	0.4	32	300	√
G3(L2)	338	48.29	204.2	0.4	32	300	√

5.4.7 Deflection check

Items	ρ_0	ρ	K	factor	$(l/d)_{premissible}$	$d>d_{permis}$
G1(L18)	0.71%	1.44%	1	0.875	14.71	√
G2(L18)	0.71%	0.87%	1	No reduction	20.16	√
G3(L18)	0.71%	1.34%	1	No reduction	17.2	√
G3(L2)	0.71%	0.67%	1	0.875	19.66	√

5.4.8 Shear design

Items	$V_{ed}=V_{max}$ Kn	v_{ed} MPa	cotθ	$v_{Rd,max}$ MPa	$v_{Rd,max}>v_{ed}$	z mm	A_{sw}/s	$A_{sw,min}/s$
G1(L18)	637.4	1.062	2.5	5.52	satisfy	1007	0.582	0.566
G2(L18)	711.2	1.482	2.5	5.52	satisfy	1010	0.648	0.453
G3(L18)	655.8	1.64	2.5	5.52	satisfy	828.9	0.728	0.453
G3(L2)	1371	1.371	2.5	5.52	satisfy	1727	0.73	0.566

Items	provide	S_{pro} mm	A_{sw}/S_{pro}	$Spacing_{max}$ mm	$Spacing<Spacing_{max}$
G1(L18)	H10@250	250	0.628	839.3	√
G2(L18)	H10@250	250	0.628	841.9	√
G3(L18)	H10@250	250	0.628	690.8	√
G3(L2)	H10@250	250	0.628	1364	√

5.4.9 Summary of design and detailing check

Items	Flexural reinforcements(Bottom)	Shear reinforcements
		mm
G1(L18)	10H32 B	H10@250
G2(L18)	8H25 B	H10@250
G3(L18)	8H18 B	H10@250
G3(L2)	8H32B	H10@250

a)Minimum areas

Items	f_{ctm} MPa	b_t mm	$A_{s,min}$ mm^2	$A_{s,pro}$ mm^2	check
G1(L18)	4.061	500	1182	8042	√
G2(L18)	4.061	400	948.2	3927	√
G3(L18)	4.061	400	778	4926	√
G3(L2)	4.061	500	2026	6434	√

b)Curtailment of main bars

Items	Bottom 0.08Lt End supprot mm	Curtail say mm	Al Left support mm	Curtail say mm	At support 25% As mm^2	d_{bar} mm	No.legs	$A_{s,pro}$ mm^2	Check
G1(L18)	640	650	1141	1000	1498	22	4	1520	√
G2(L18)	460	500	462	500	971.7	18	4	1017	√
G3(L18)	460	500	689	700	1005	22	3	1140.4	√
G3(L2)	640	650	241.1	300	1511	22	4	1521	√

6 Flat Slab Design

6.1 General introduction and analysis

In this part, a flat slab on each level will be divided into different strips to design. The slabs with similar loads and loading conditions will be considered a group to design, simplifying the repetitive design and construction process.
The regular column range's flat slabs will be designed per EC's requirements. In terms of flat slabs near the core area, the design will refer to codes, research essays, and Finite Element Analysis results from ETABS.
The design reports discuss and compare the research area in flat slabs for detailed assumptions. However, basic calculations and information which follows codes' requirements will be provided in this sheet.
Meanwhile, to the limit of pages, only critical level calculations will be provided(Level 2), and the repetitive calculation will try to avoid.

6.1.1 Grid dimension and area division

Grid dimension and area division in Retail level

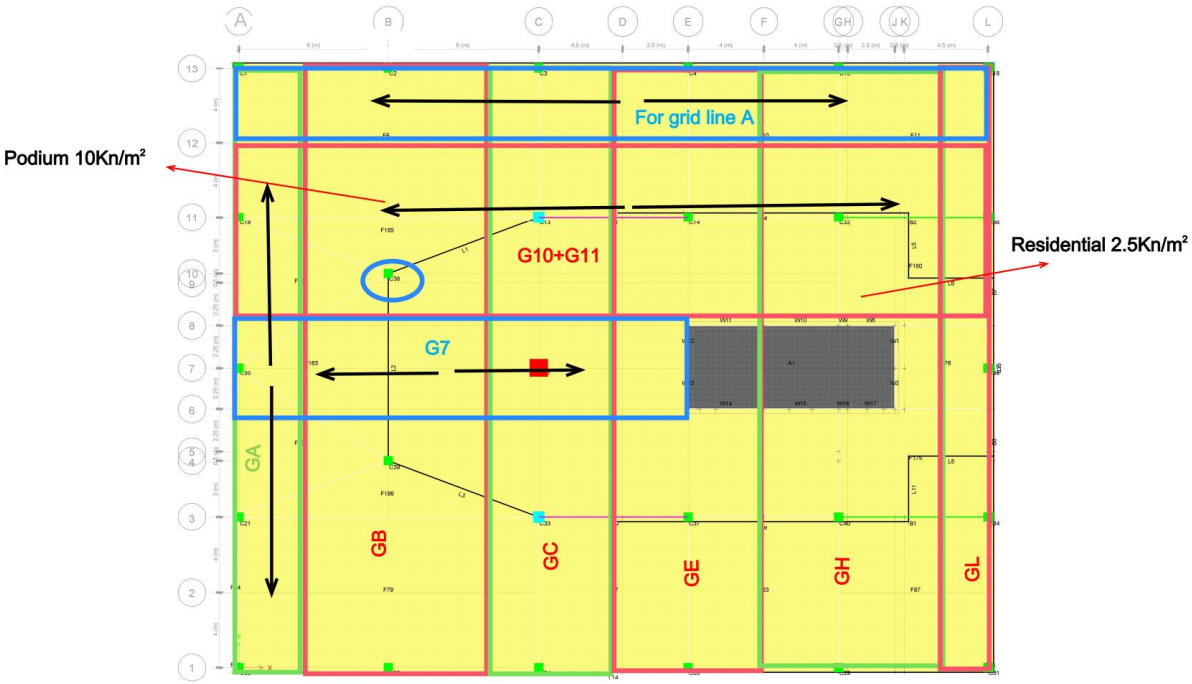


Figure 6- 1 Grid dimension and area divisions in Retail level

6.2 Flat slab design for level 2

6.2.1 General information for the whole level

Material properties

f_{ck} MPa	f_{cd} MPa	E_{cm} MPa	ρ_c (Kn/m^3)	f_{yk} MPa	f_{yd} MPa
50	28.33	37.28	25	500	434.8

Section dimensions

Slab thickness	t_{slab}	=	350	mm
Rebar Diameter	d_{bar}	=	20	mm

Cover design

assuming 20 mm diameter reinforcement	$c_{min,b}$	=	20	mm	
Minimum cover due to environmental conditions.For S4 and XS1	$c_{min,dur}$	=	45	mm	
Cmin=max(Cmin,b;Cmin,dur;10mm)	fixed value	=	10	mm	
	c_{min}	=	45	mm	
Cnom =Cmin+ ΔCdev	ΔC_{dev}	=	10	mm	
	C_{nom}	=	55	mm	
For 2 hours resistance Minimum thickness for cover	a_{min}	=	35	mm	Satisfy

Exp. (4.1)
Cl.4.4.1.2(3)
Table 4.1.
BS 8500-1:
Table A4.

EC2-12:
Table 5.9

6.2.2 Flat slab design for Grid line A(Edge)

Equivalent Frame Model(EFM) with dimensions

EFM for Grid line A level 2

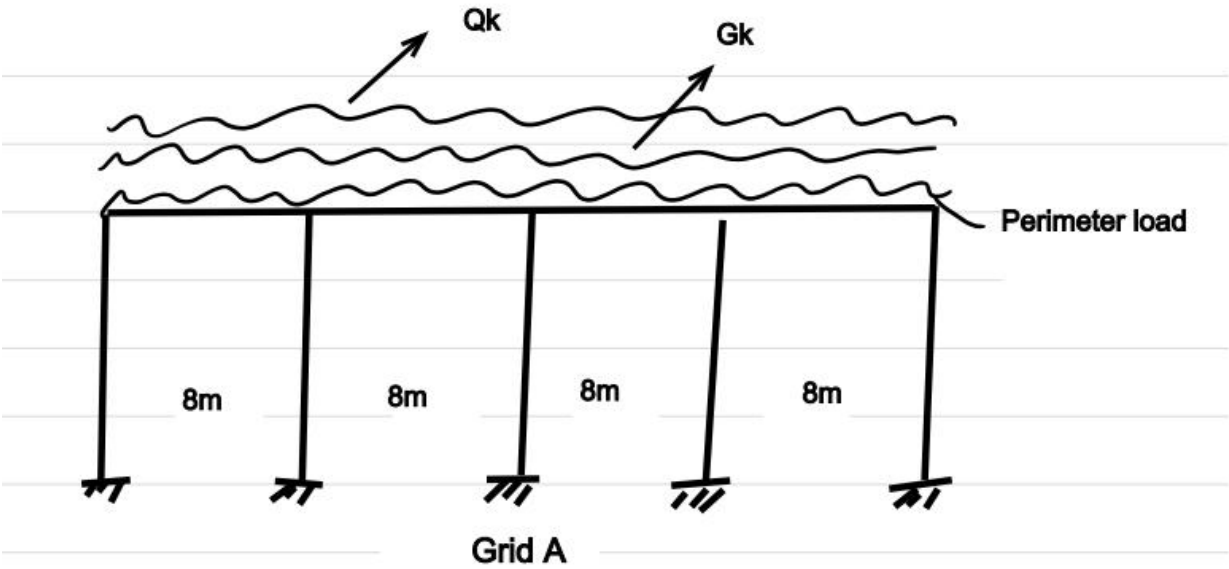


Figure 6- 2 EFM with all-span-load case

Span and dimensions

Span Number	Original Span	C_{left}	C_{right}	al_{eft}	a_{right}	$Span_{effective}$
	mm	mm	mm	mm	mm	mm
1	8000	500	500	175	175	7850
2	8000	500	500	175	175	7850
3	8000	500	500	175	175	7850
4	8000	500	500	175	175	7850
The typical effective span					=	7850

Consider Grid line= 4.25 m wide (Edge Perimeter)

Bay width=(Slab width+column width)	Ly	=	4.25	m
Column Strip Width =(Ly/2)	Ly/2	=	2.125	m
Middle Strip Width		=	2.125	m

Actions

Given in brief, at level 2

		Surface load (Kn/m2)	× Bay width		Line Load (Kn/m)
Dead					
SW(Slab)	=	8.75		=	37.188
Finishes	=	1		=	4.25
Perimeter Load	=			=	2
Total dead load	=	9.75		=	43.438
Live					
Podium Deck	=	10		=	42.5
Total live load	=	10		=	42.5

Load combination and arrangement

(considered all-span load only, maybe consider alternate span load as well later)

Consider the most critical load case

N = 1.35 × Gk + 1.5 × Qk

Critical moment and shear forces calculation

In this continuous span, the design moment and shear force is the critical one in the all actions.

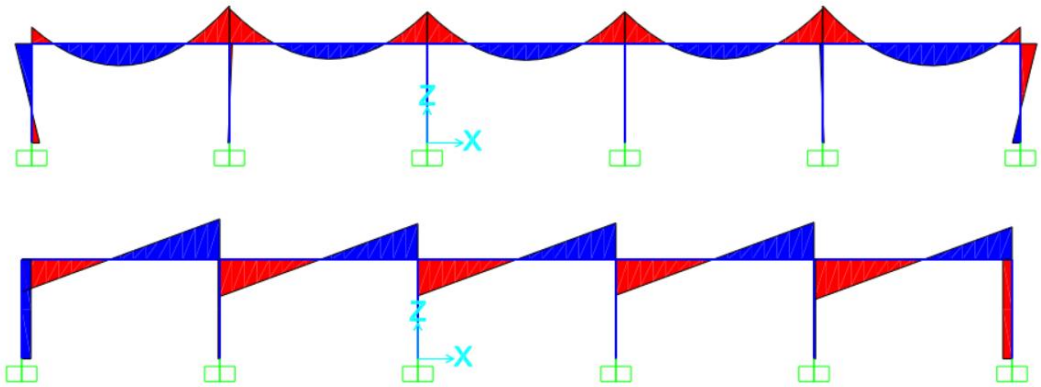


Figure 6- 3 Moment and shear forces diagrams

To investigate the bending moment with different methods, equivalent frame model, coefficient and FEA 3D model methods are introduced to calculate the sagging and hogging moments.

For EFM

Span Number	Mmid (KN.M)	Msupport (kN.M)	Ved (KN)
1	442.3	-334	436
		-761	543
2	309		
		-645	489
3	332		
		-646	489
4	309		
		-760	548
5	442.3		
		-334	436
Critical Value	442.3	761	543

the critical shear force is at C2 (penutimate column) and the critical moment is at span 1 and

5(symmetry)

For coefficient method

Bending moment coefficient	Middle Span	=	0.086
	Support	=	0.086

For the end span

Main load induced	Med-Span	=	634.3	KN.M/m
Perimeter load induced	Med-Perimeter	=	14.3	KN.M/m
Total	Med-Support	=	648.61	KN.M/m

Table C2 (Worked Examples to Eurocode 2: Volume 1, 2009)

Analysis Grid line A

Apportionment of moments between column strips and middle strips:
*Note: Identify West-East direction as long span and North-South as short span.

		Apportionment(as%)	
		Column Strip	Middle Strip
-ve(hogging)	Long Span	0.7	0.3
	Short Span	0.75	0.25
+ve (sagging)		0.5	0.5

Identify Grid Line A as Long span---Long Span moment

Coefficient Method

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	213.66	91.57
+ve (sagging)	149.25	149.25
SUM	362.91	240.82

EFM-SPA 2000

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	250.68	107.44
+ve (sagging)	104.07	104.07
SUM	354.75	211.51

FEA Model--ETABS

In FEA model, the moments of slabs can be obtained by setting reinforcement strips in X and Y direction as shown in Figure 6-4, the maximum hogging and sagging moment values in Grid line A are recorded as below:

	Med (Kn.m/m) Column Strip, /m	Med (Kn.m/m) Middle strip, /m
-ve(hogging)	262	150
+ve (sagging)	94	163
SUM	356	313

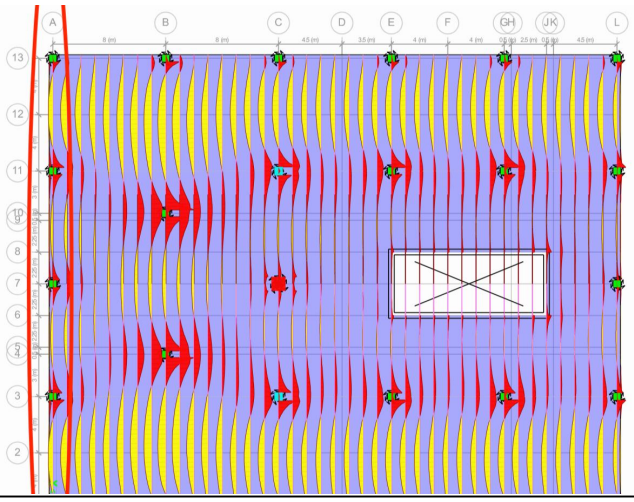


Figure 6- 4

Moments contour in Y direction (Level 2)

The detailed comparisons and discussion are shown in report.

Design value for flexural design

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	262	150
+ve (sagging)	149.25	163
SUM	411.25	313

For conservative consideration, the maximum moments will be used to design

Design Grid line A

1.Flexture: Column strip Strip, Sagging

Effective depth	deff	=	285	mm
	Med	=	149.25	KN.M/m
		=	149248154	N.mm/m
Unit width	b	=	1000	mm
	k	=	0.0367	
	z/d	=	0.9664	
	z	=	275.44	mm
	As,req/m	=	1246.28	mm2/m
Provide H20@250 B1				
Bar diameter		=	20	mm
Spacing		=	250	mm
Number of bars/m		=	4	
As,pro		=	1256.6	mm2/m
ρ,req		=	0.44%	
ρ,pro		=	0.44%	
Check			ok	

2.Flexture: Middle Strip, Sagging

	Med	=	163	KN.M/m
		=	163000000	N.mm/m
Unit width	b	=	1000	mm
	k	=	0.0401	
	z/d	=	0.9632	
	z	=	274.52	mm
	As,req/m	=	1356.66	mm2/m
Provide H20@200 B1				
Bar diameter		=	20	mm
Spacing		=	200	mm
Number of bars/m		=	5	
As,pro		=	1570.8	mm2/m
ρ,req		=	0.48%	
ρ,pro		=	0.55%	
Check			ok	

Table C5 (Worked Examples to Eurocode 2: Volume 1, 2009)

3.deflection (consider the worst in column and middle strip)

Check Span-to-depth(deff) ratio Allowable L/d=N×K×F1×F2×F3

Factors refer to

$$\sigma_S = \sigma_{su}(A_{S,\,req}/A_{S,prov})1/\delta$$

N	k	F1	F2	F3	δ	G_k/Q_k	ψ_2	γ_G	σ_{su}	σ_s	F3
45	1.2	1	1	310	1.03	0.975	0.3	1.35	196.84	189.53	1.5

Span/depth ratio check

Span Number	Allowable L/d	Actual L/d	Check
Span 1	81	27.54	ok
Span 2	81	27.54	ok
Span 3	81	27.54	ok
Span 4	81	27.54	ok

Hence, provide H20@250 B1 for column strip and H20@200 B1 for middle strip is ok

Appendix B Cl.7.4.2(2) (Worked Examples to Eurocode 2: Volume 1, 2009)

4. Flexture: Column strip, hogging

Table 10-13 Appendix C

	Med	=	250.68	KN.M/m
		=	250682352	N.mm/m
Unit width	b	=	1000	mm
	k	=	0.0617	
	z/d	=	0.9422	
	z	=	268.52	mm
	As,req/m	=	2147.19	mm2/m
Provide H20@125 T1				
Bar diameter		=	20	mm
Spacing		=	125	mm
Number of bars/m		=	8	
As,pro		=	2513.27	mm2/m
ρ,req		=	0.75%	
ρ,pro		=	0.88%	
Check			ok	

5. Flexture: Middle strip, hogging

Table C5, (Worked Examples to Eurocode 2: Volume 1, 2009)

	Med	=	107.44	KN.M/m
		=	107435294	N.mm/m
Unit width	b	=	1000	mm
	k	=	0.0265	
	z/d	=	0.9761	
	z	=	278.18	mm
	As,req/m	=	888.27	mm2/m
Provide H16@200 T1				
Bar diameter		=	16	mm
Spacing		=	200	mm
Number of bars/m		=	5	
As,pro		=	1005.3	mm2/m
ρ,req		=	0.31%	
ρ,pro		=	0.35%	
Check			ok	

6. Requirement

A) In column strip, inside middle 1500mm

As required, 50% of At within a width equal to 0.125 of the panel width on either side of the column.

Total Area of reinforcement in tension(to resist hogging)	As,tot,t	=	6450.3	mm2
50%	As,tot,t	=	3225.2	mm2

Required width to place the reinforcement	Width(one sides)	=	781.25	mm
Required 50% reinforcement /m		=	4128.2	mm2/m
Provide H20@75 T1				
Bar diameter		=	20	mm
Spacing		=	75	mm
Number of bars/m		=	13.33	
As,pro		=	4188.8	mm2/m
ρ,req		=	1.45%	
ρ,pro		=	1.47%	
Check			ok	

Cl. 9.4.1(2)

B) in column strip outside middle 1500 mm (Remaining Column Strip)

column strip (remaining)		=	1343.8	mm
In the range	As,req	=	1337.6	mm
	As,req/m	=	995.43	mm2/m
Provide H20@300 T1				
Bar diameter		=	20	mm
Spacing		=	300	mm
Number of bars/m		=	3.33	
As,pro		=	1047.2	mm2/m
ρ,req		=	0.35%	
ρ,pro		=	0.37%	
Check			ok	

C) In the middle strip

As,req/m	=	888.27	mm2/m
Provide H16@200 T1			
ρ,req	=	0.31%	
ρ,pro	=	0.35%	
Check		ok	

6.2.3 Flat slab design for Grid line B (internal)

Material properties and Section dimensions are the same as grid line A

Equivalent Frame Model(EFM) with dimensions

Cl.9.4.2(1)
1.1.2(5)
Fig.9.9

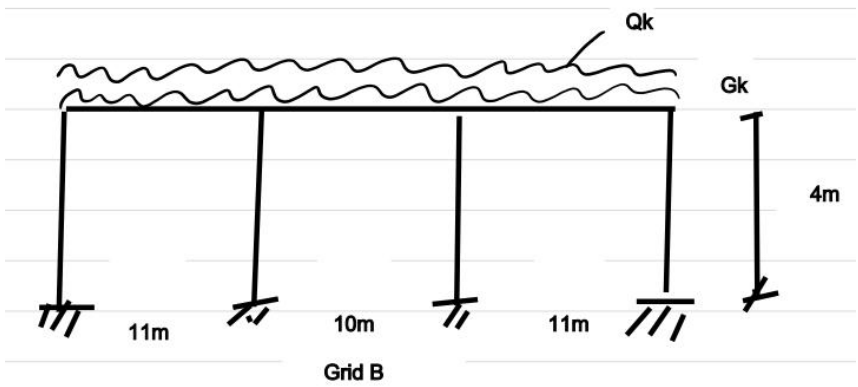


Figure 6- 5 EFM for Grid line B level 2

For comparing with FEA models, all-span-load cases are applied in here.

Span and dimensions

Span Number	Original Span	C_{left}	C_{right}	a_{left}	a_{right}	$Span_{effective}$
	mm	mm	mm	mm	mm	mm
1	11000	500	500	175	175	10850
2	8000	500	500	175	175	9850
3	11000	500	500	175	175	10850
The typical effective span					=	10850

Cl. 5.1.3(1) & NA: Table NA.1 (option b)

Consider Grid line= 4.25 m wide (Edge Perimeter, consider half of column width)

Bay width=(Slab width+column width)	Ly	=	8	m
Column Strip Width =(Ly/2)	Ly/2	=	4	m
Middle Strip Width		=	4	m

Actions,load combination and arrangement

The action condition is same as grid line A

Cl. 5.3.2.2(1)

Critical moment and shear forces calculation

In this continuous span, the design moment and shear force is the critical one in the all actions. EFM moment and shear diagrams are shown below, and the coefficient method and is the same as grid A. FEA results are extract from ETABS manually

Table I.1; (Technical Report 64,Guide to the design and construction of RCflat slabs, 2007) Table I.1 NA.3;Fig. I.1

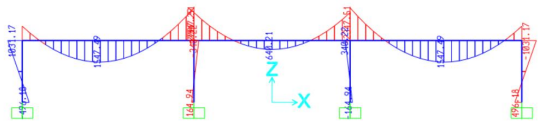


Figure 6- 6 Moment diagram

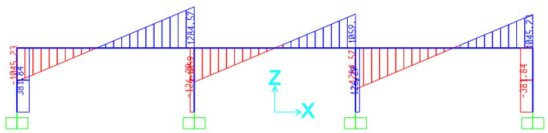


Figure 6- 7 Shear force diagram

Analysis Grid line B---Identify as Long span

Apportionment of moments between column strips and middle strips:

		Apportionment(as%)		
		Column Strip	Middle Strip	
-ve(hogging)	Long Span	0.7	0.3	
	Short Span	0.75	0.25	
+ve (sagging)		0.5	0.5	

Comparison between EFM and whole floor model results

Coefficient Method

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	450	160
+ve (sagging)	200	202.45
SUM	650	362.45

EFM-SPA 2000

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	437.5	187.5
+ve (sagging)	205.75	205.75
SUM	643.25	393.25

Whole Model--ETABS

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	516	148
+ve (sagging)	180	205.75
SUM	696	311

Design value for flexure

	Med (Kn.m/m)	Med (Kn.m/m)
	Column Strip, /m	Middle strip, /m
-ve(hogging)	516	187.5
+ve (sagging)	205.75	205.75
SUM	721.75	393.25

Design Grid line B

1. Flexure: Column strip, Sagging

Med	=	205.75	KN.M/m
As,req/m	=	1742.16	mm2/m
Provide H20@175 B1			
As,pro	=	1795.2	mm2/m
ρ,req	=	0.61%	
ρ,pro	=	0.63%	
Check		ok	

2. Flexure:Middle Strip, Sagging

Med	=	205.75	KN.M/m
As,req/m	=	1742.16	mm2/m
Provide H20@175 B1			
As,pro	=	1795.2	mm2/m
ρ,req	=	0.61%	
ρ,pro	=	0.63%	
Check		ok	

3. deflection (column strip+middle strip)

Span/depth ratio check

Span Number	Allowable L/d	Actual L/d	Check
Span 1	45.36	38.07	ok
Span 2	45.36	34.56	ok
Span 3	45.36	38.07	ok

Hence, provide H20@175 B1 is ok

4. Flexure: Column strip, hogging

Med	=	516	KN.M/m
As,req/m	=	4779.23	mm2/m
Provide H20@50 T1			
$A_{s,pro}$	=	6283.18	mm2/m
ρ_{req}	=	1.68%	

ρ_{pro}	=	2.20%	
Check		ok	

5. **Flexure: Middle strip, hogging**

M_{ed}	=	187.5	KN.M/m
$A_{s,req}$/m	=	1795.2	mm2/m
Provide H20@175 T1			
$A_{s,pro}$	=	1795.2	mm2/m
ρ_{req}	=	0,55%	
ρ_{pro}	=	0.63%	
Check		ok	

6. Requirement

In column strip, inside middle 1500mm

As required, 50% of At within a width equal to 0.125 of the panel width on either side of the column.

$A_{s,tot,t}$	50%$A_{s,tot,t}$	Width(two sides)	50%$A_{s,tot,t}$/m	Provide H22@50 T1	$A_{s,pro}$	ρ_{req}	ρ_{pro}	Check
mm^2	mm^2	mm	mm^2/m		mm^2			
25439	12719	2000	6359.6		7603	2.23%	2.67%	ok

in column strip outside middle 1500 mm (Remaining Column Strip)

r mm	mm^2	mm	mm2			
2000	6397.6	3198.8	4188.8	1.12%	1.47%	ok

In the middle strip

As,req/m mm^2	Provide H20@175 T1	As,pro mm^2	ρ,req	ρ,pro	Check
1580.4		1580.4	0.55%	0.63%	ok

Perpendicular to edge of slab at edge column

Column width in x direction	c_x	=	500	mm
Column width in y direction	c_y	=	500	mm
Slab thickness	c_z	=	350	mm
$b_e = c_x + y$	b_e	=	300	mm
Design transfer moment to column $M_t=0,17b_ed^2f_{ck}$	M_t	=	207.12	Kn.m
	k	=	0.17	
	z/d	=	0.8162	
	z	=	232.61	mm
	$A_{s,req}$/m	=	1350	mm^2/m
This reinforcement to be placed within c_z+2c_y		=		
Provide U-bars H22@200 T1				
Bar diameter		=	22	mm
Spacing		=	350	mm
Number of bars/m		=	5.71	
As,pro		=	2172.2	mm^2/m
ρ,req		=	0.72%	
ρ,pro		=	0.76%	
Check			ok	

Cl. 9.4.2(1), I.1.2(5)

Perpendicular to edge of slab generally

Assuming that there is partial fixity along the edge of the slab, top reinforcement capable of resisting 25% of the moment in the adjacent span should be provided

Partial factor	0.25
$A_{s,req}$/m (mm^2/m)	1194.8
ρ_{req}	0.42%

Check minimum area of reinforcement

$0.3 \times f_{ck}^{0.666}$	f_{ctm}	=	4.061	Mpa
Width of tension zone	b_t	=	1000	mm
	$A_{s,min}$/m	=	601.84	mm^2/m
This reinforcement to be placed within c_z+2c_y		=		
Provide H12@200 T1				
Bar diameter		=	12	mm
Spacing		=	150	mm
Number of bars/m		=	6.67	
As,pro		=	753.98	mm^2/m
ρ,req		=	0.21%	
ρ,pro		=	0.26%	
Check			ok	

Cl. 9.3.1.2(2), 9.2.1.4(1) & NA

6.2.4 Sum of simplified design calculation and information for level 2

Cl. 9.3.1.1, 9.2.1.1

EFMs for Level 2

Table 3.1

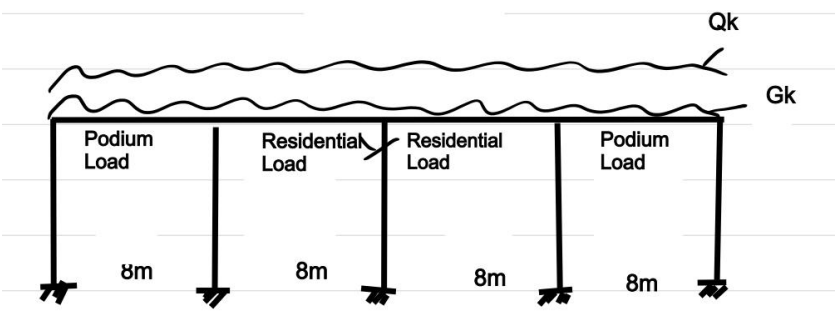


Figure 6- 8 EFM for grid line C

Cl. 9.3.1.4(2)

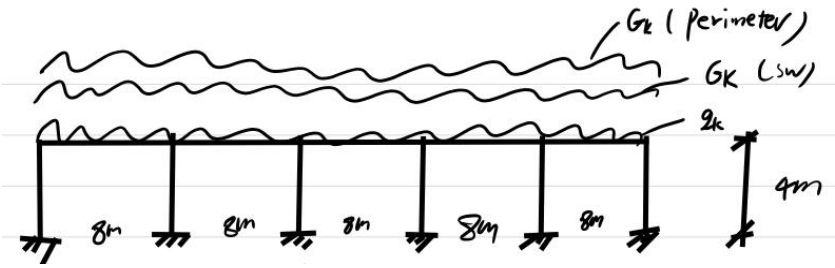


Figure 6- 9 EFM for grid line 13

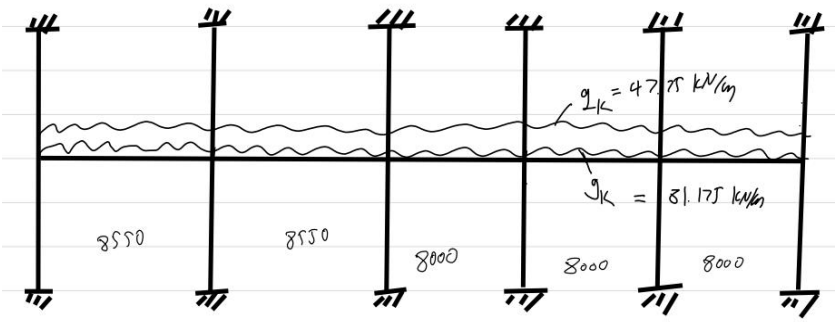


Figure 6- 10 EFM for grid line (11+10)

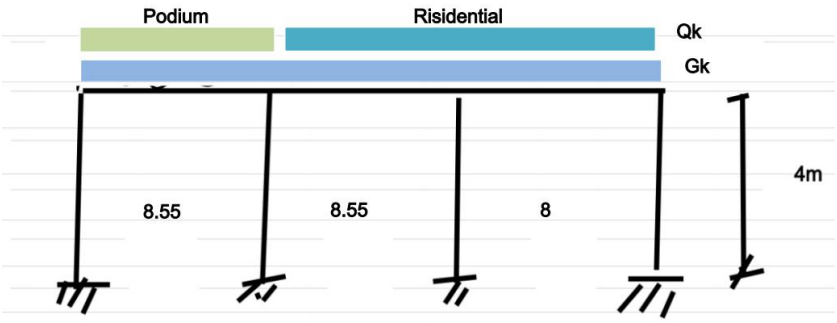


Figure 6- 11 EFM for grid line (7)

Moment results in coefficient, EFM and FEM

	Hogging in column strip (Kn.m/m)					
	Grid A	Grid B	Grid C	Grid 13	Grid (11+10)	Grid 7
Coefficient	213.66	450	240	201.79	256	240
EFM	250.68	437.5	221.9	197.86	315.79	256.34
FEA	262	516	330	220	262	74
Design value	262	516	330	220	315.78	256.34

	Sagging in column strip (Kn.m/m)					
	Grid A	Grid B	Grid C	Grid 13	Grid (11+10)	Grid 7
Coefficient	149.25	200	120	140.96	170.89	165
EFM	104.07	205.75	117	103.02	129.34	170.89
FEA	94	163	143	94	94	117
Design value	149.24	205.75	143	140.95	170.89	170.89

	hogging in Middle strip (Kn.m/m)					
	Grid A	Grid B	Grid C	Grid 13	Grid (11+10)	Grid 7
Coefficient	91.57	160	90	297.29	86.45	90
EFM	107.44	187.5	95.1	184.8	105.26	85.45
FEA	150	148	51	100	121	115
Design value	150	187.5	95.1	100	121	115

	Sagging in Middle strip (Kn.m/m)					
	Grid A	Grid B	Grid C	Grid 13	Grid (11+10)	Grid 7
Coefficient	149.24	202.45	120.23	158.57	170.89	165
EFM	104.07	205.75	117	115.9	129.34	170.89
FEA	163	163	135	163	163	109.9
Design value	163	205.75	135	163	170.89	170.89

1. Flexture: Column strip, Sagging

Grid	d_{eff} mm	M_{ed} Kn.m/m	b mm	z mm	$A_{s,req}$/m mm^2/m	provide	ρ_{req}	ρ_{pro}	Check
GC	310	143	1000	301.63	1090.4	H20@275 B1	0.35%	0.37%	ok
G13	290	140.96	1000	281.15	1153.12	H20@200 B2	0.40%	0.54%	ok
G(10+11)	290	170.89	1000	279.2	1407.82	H20@175 B2	0.49%	0.62%	ok
G7	290	170.9	1000	279.2	1407.82	H20@200 B2	0.49%	0.54%	ok

2. Flexture:Middle Strip, Sagging

Grid	M_{ed} Kn.m/m	b mm	z mm	$A_{s,req}$/m mm^2/m	provide	ρ_{req}	ρ_{pro}	Check
GC	135	1000	302.11	1027.76	H20@300 B1	0.33%	0.34%	ok
GC	163	1000	279.71	1340.29	H20@300 B2	0.46%	0.54%	ok
G(10+11)	170.89	1000	279.2	1407.82	H20@175 B2	0.49%	0.62%	ok
G7	170.9	1000	279.2	1407.82	H20@200 B2	0.49%	0.54%	ok

3. deflection (column strip+middle strip)
Span/depth ratio check

	Span Number	Allowable L/d	Actual L/d	Check
GC	Span 1	106.778	25.16	ok
	Span 2	106.778	24.35	ok
	Span 3	106.778	24.35	ok
	Span 4	106.778	25.16	ok
G13	Span 1	57.6	27.07	ok
	Span 2	57.6	27.07	ok
	Span 3	57.6	27.07	ok
	Span 4	57.6	27.07	ok
	Span 5	57.6	27.07	ok
G(10+11)	Span 1	45.36	28.97	ok
	Span 2	45.36	28.97	ok
	Span 3	45.36	27.07	ok
	Span 4	45.36	27.07	ok
	Span 5	45.36	27.07	ok
G7	Span 1	51,48	28.97	ok
	Span 2	51,48	28.97	ok
	Span 3	51,48	28.97	ok

4. Flexture: Column strip, hogging

Grid	M_{ed} Kn.m/m	b mm	z mm	$A_{s,req}$/m mm^2/m	provide	ρ_{req}	ρ_{pro}	Check
GC	330	1000	289.91	2618.06	H20@100 T1	0.84%	1.01%	ok
G13	220	1000	275.93	1833.82	H20@150 T2	0.63%	0.72%	ok
G(10+11)	315.79	1000	269.3	2697.02	H20@150 T2	0.93%	1.08%	0k
G7	256.3	1000	273.5	2513.27	H20@150 T2	0.74%	0.87%	ok

5. Flexture: Middle strip, hogging

Grid	M_{ed} Kn.m/m	b mm	z mm	$A_{s,req}$/m mm^2/m	provide	ρ_{req}	ρ_{pro}	Check

GC	95.1	1000	304.49	718.35	H20@300 T1	0.23%	0.34%	ok
G13	100	1000	283.78	810.49	H20@200 T2	0.28%	0.36%	ok
G(10+11)	121	1000	282.44	1047	H20@300 T2	0.34%	0.36%	ok
G7	115	1000	282.8	935.21	H20@200 T2	0.32%	0.36%	ok

6. Requirement

In column strip, inside middle 1500mm

As required, 50% of At within a width equal to 0.125 of the panel width on either side of the column.

Grid	$A_{s,tot,t}$ mm^2	50%$A_{s,tot,t}$ mm^2	Width mm	50%$A_{s,tot,t}$/m mm^2/m	Provide	$A_{s,pro}$ mm^2	ρ_{req}	ρ_{pro}	Check
GC	13346	6672.8	2000(two sides)	63336.4	H20@125 T1	7603	2.23%	2.67%	ok
G13	5747.1	2873.5	1000(one side)	2873.5	H20@100 T2	3141.59	0.99%	1.08%	ok
G(10+11)	17491	8745.6	2000(two sides)	4372.8	H20@100 T2	6283.19	1.51%	2.17%	ok
G7	7728	3864	2000(two sides)	3864	H20@200 T2	2094.4	0.67%	0.72%	ok

in column strip outside middle 1500 mm (Remaining Column Strip)

Grid	column strip (remaining) mm	$A_{s,req}$ mm^2	$A_{s,req}$/m mm	Provide	$A_{s,pro}$ mm^2	ρ_{req}	ρ_{pro}	Check
GC	2000	3799.4	1899.7	H20@150 T1	2094	0.61%	0.68%	ok
G13	1250	1252.6	1002	H20@250 T2	1256.63	0.35%	0.43%	ok
G(10+11)	2750	4065.2	1478.3	H20@200 T2	1570.80	0.51%	0.54%	ok
G7	500	1526	3052	H20@100 T2	3141.59	1.05%	1.08%	ok

In the middle strip

Grid	$A_{s,req}$/mm mm	Provide	$A_{s,pro}$ mm^2	ρ_{req}	ρ_{pro}	Check
GC	718.35	H20@300 T1	1580.4	0.55%	0.63%	ok
G13	810.49	H12@200 T2	1047	0.28%	0.36%	ok
G(10+11)	985.35	H20@300 T2	1570.8	0.34%	0.36%	ok
G7	935.2	H20@200 T2	2094.34	0.32%	0.36%	ok

6.2.5 Punching shear design for level 2

One detailed punching calculation will be provided for both a column and a wall corner.

Detailed design for column 2

For design simplicity, only most critical edge and internal columns will be design and applied in the Level 2 which is the edge columns in two orthogonal directions C2 and C7, corner column C1 and internal column C12 and C14.

A) Shear stress around columns

In this part, both hand calculations and FEA results will be presented. ULS load combination and all floor loaded condition are applied in this design.
A detailed calculation for one column will be presented in here.

For column 2(C2)-(Edge column)

Loading width	W	=	5.5	m	
Loading length	L	=	8	m	
Loading area	Area	=	44	m2	
Dead load(consider finishes)		=	9.75	Kn/m2	
Live load(in Podium Deck area)		=	10	Kn/m2	
Applied shear force(Hand)	V_{ed}	=	1239.15	Kn	
factor dealing with eccentricity; recommended 1.4 for edge 1.5 for corner and 1.15 for internal	β	=	1.4		
Column width in x direction	c_x	=	500	mm	
Column width in y direction	c_y	=	500	mm	
Effective depth in y direction	d_y	=	285	mm	
Effective depth in x direction	d_x	=	290	mm	
Average depth	d	=	287.5	mm	Cl.6.4.3(2) 6.4.5(3)
control perimeter under consideration $u_0 = 2(c_1 + c_2)$ for internal columns $= c_2 + 3d \leq c_2 + 2c_1$ for edge columns $= 3d \leq c_2 + 2c_1$ for corner columns	u_i	=	1362.5	mm	Fig.6.21N & NA
	v_{ed}	=	4.43	Mpa	Cl.6.4.5(3)
Finite element Analysis*Kn/m in ETABS Shear force	V_{ed}	=	1382	Kn/m	
Shear stress(FEA)	v_{ed}	=	4.81	Mpa	Exp. (6.32)
	Error	=	7.87%		Cl.6.4.5(3)Note
Design Ved(convert from ved in FEA)	V_{ed}	=	1344.98	Kn	
Design ved	v_{ed}	=	4.81	Mpa	

B) Check at perimeter of columns

	v	=	0.46	
$V_{R_{d,\,max}} = 0.5vf_{cd}$	$v_{rd,max}$	=	6.8	MPa
Check	$v_{ed}<v_{rd,max}$?	=	√	
No punching Shear RF required				

C) At control perimeter u1(2d from column)

Control perimeter under consideration $u_1 = u_0 + 2\pi \times 2d$	u_1	=	3168.9	m	
$V_{Ed} = \beta V_{Ed}/u_1 d < V_{Rd,c}$	v_{ed}	=	2.07	m	
Grid B Reinforcement ratio of bonded steel in the y direction	ρ_{ly}	=	2.2%	m^2	
Grid 13 Reinforcement ratio of bonded steel in the x direction	ρ_{lx}	=	0.72%	Kn	Cl. 6.4.2
$\rho_l = (\rho_{ly}\rho_{lz})^{0.5} =$	ρ_l	=	1.26%		
$k = 1 + (200/d)^{0.5} \leq 2$	k	=	1.8341	mm	
$v_{Rd,c} = 0.18/\gamma_C k(100\rho_l f_{ck})^{0.333}$	$v_{Rd,c}$	=	0.875	mm	
Check Shear stress	$v_{ed}<v_{rd,c}$?	=	×	mm	
Punching shear reinforcement required					

Fig. 6.13

D) Perimeter at which punching shear links are no longer required

$u_{out,ef} = \beta V_{Ed}/(v_{Rd,c}d)$	u_{out}	=	7485.5	mm	
Length attributable to column faces		=	1500	mm	
Radius to u_{out} from face of column	r_{out}	=	1905.3	mm	Exp. (6.47) & NA
	1.5d	=	431.25	mm	

Perimeter of shear reinforcement may stop		=	1474	mm	Cl. 6.4.4.1(1)

E) Reinforcement design

Spacing

Shear reinforcement(assuming rectangular arrangement of links 0.75×d			S_{rmax}	=	215.63	mm	Exp. (6.54)
			Say	=	175	mm	
Inside 2d control perimeter 1.5×d	Within 2d	(in calculation)	$S_{t,max}$	=	431.25	mm	Cl. 6.4.5(4) & NA
St is the spacing of shear links		(Take as)	$S_{t,max}$	=	350	mm	
in the tangential direction	Without 2d	(in calculation)	$S_{t,max}$	=	575	mm	
Outside control perimeter		(Take as)	$S_{t,max}$	=	500	mm	

Area of reinforcement(assuming vertical reinforcement)

Effective design strength of reinforcement (250+0.25d)<fyd	$f_{ywd,ef}$	=	321.88	MPa	Cl. 9.4.3(1)
At the basic control perimeter u1, 2d from the column $A_{Sw} \geq (v_{Ed} - 0.75v_{Rd,c})s_r u_1/1.5f_{ywd,ef})$	A_{sw} ≥	=	1620.2	Mm2	
	α vertical	=	90	°	
Minimum area of a single leg of link $A_{sw,min} \geq 0.08f_{ck}^{0.5}(s_r s_t)/(1.5f_{yk}\sin\alpha + \cos\alpha)$	$A_{sw,min}$≥	=	46.198	mm	
Try H8 legs of links (50mm2)					
	d_{bar}	=	8	mm	
	Number of legs	=	1		Cl. 9.4.3(2)
	$A_{sw,pro}$	=	50.265	mm2	
	A_{sw}/u_1	=	0.5113	mm2/mm	Exp. (6.52)
Using H8 max. Spacing = min [area per leg/Asw/u1; 1.5d]	Max Spacing	=	98	Mm cc	Cl. 6.4.5(1)
Use min.H8 legs of links at **mm cc around perimeter u_1					
Perimeter at 0.75d		=	215.63	mm	Exp. (9.11)
Say for H8	Spacing	=	175	<max spacing	Cl. 9.4.3

Detailed design for a wall corner

For wall corner design, proper hand calculation is hard to implement. Therefore, design shear stress determination highly relies on FEA model results.

As introduced in the report, the FEA model has been validated by comparing hand calculation results and FEA results for shear stress around columns.

It is reliable using FEA results to design.

Cl. 9.4.3(1)

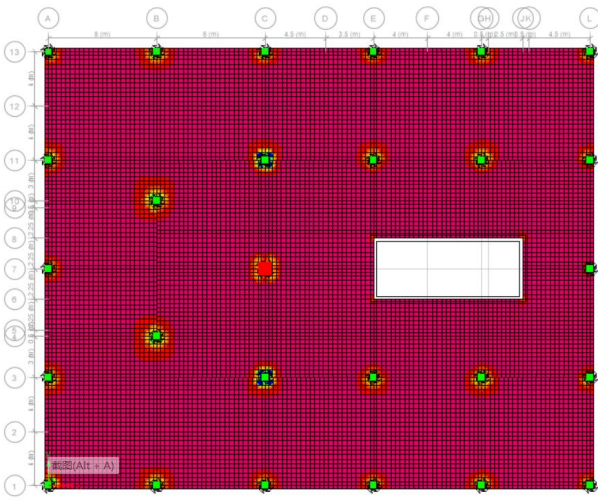


Figure 6- 12 Shear stress contour in Level 2

As the shear force contour shows on LHS, the shear stress around wall corners is very small compared with the stress around columns.
However, parametric analysis is still needed to ensure the structure's safety.
The principal stress is only concentrated on wall corners in a very limited range. The wall corner can be simplified as a corner column(two faces connected with the slab).
*Note in ETABS that the max shear force unit is Kn/m. It has been divided by the column perimeter. It needs to be divided by effective depth for getting shear stress.

Design shear stress calculation

β	c_x mm	c_y mm	d_y mm	d_x mm	d mm	u_i mm	(FEA)V_{ed} Kn/m	Design v_{ed} MPa	Design V_{ed} Kn
1.50	400	400	285	290	287.5	1600	315	1.10	336

At perimeter of column			At control perimeter u1(2d from column)							
v	$v_{rd,max}$ MPa	$v_{ed}<v_{rd,max}$	u_1 mm	v_{ed} MPa	ρ_{ly}	ρ_{lx}	ρ_l	k	$v_{Rd,c}$ MPa	$v_{ed}<v_{rd,c}$
0.48	6.8	√	2142.4	0.818	1.1%	1.08%	1.09%	1.83	0.834	√

No punching Shear reinforcement required at both control perimeter

<u>Design for the rest typical grid line in Level 2(compacted version)</u>

<u>Shear stress calculation</u>

Columns type:C2, C7-Edge column; C1- Corner column; C12, C14-internal column;
W-loading width; L-loading length; Area- loading area

(1) For Hand calculation v_{ed}

Items	W	L	A	Dead	Live	V_{ed}	β	c_x	c_y	d	u_i	v_{ed}
	m	m	M^2	Kn/m^2	Kn/m^2	Kn		mm	mm	mm	mm	MPa
C2	5.5	8	44	9.75	10	1239	1.4	500	500	287.5	1362.5	4.43
C7	8	4	32	9.75	10	901.2	1.4	500	500	287.5	1362.5	3.2
C1	4	4	16	9.75	10	450.6	1.5	500	500	287.5	862.5	2.7
C12	10.5	8	84	9.75	6.25	1893.15	1.15	500	500	287.5	2000	3.79
C14	8	4.25	34	9.75	2.5	575	1.15	1000	1000	287.5	4000	0.58
Wall corner												

Only provide shear stress from FEA model for wall corners.

(2) FEA results

	(FEA)V_{ed} Kn/m	Error	Design v_{ed} MPa	Design V_{ed} Kn

C2	1382	7.87%	4.81	1344.98
C7	1127	17.83%	1096.8	3.92
C1	1075	27.10%	618.13	3.74
C12	1207	9.81%	2099.13	4.20
C14	870	81%	3026.09	3.03
Wall corner	315		1.10	336

At perimeter of column

	v	$v_{rd,max}$ MPa	$v_{ed}<v_{rd,max}$
C7	0.48	6.8	√
C1	0.48	6.8	√
C12	0.48	6.8	√
C14	0.48	6.8	√
Wall corner	0.48	6.8	√

√-punching shear reinforcement do not required
×-Punching shear reinforcement required

At control perimeter u1(2d from column)

	u_1 mm	v_{ed} MPa	ρ_{ly}	ρ_{lx}	ρ_l	k	$v_{Rd,c}$ MPa	$v_{ed}<v_{rd,c}$
C7	3168.9	1,385	0.88%	1.08%	0.98%	1.83	0.804	√
C1	2668.9	1.208	2.88%	0.72%	0.80%	1.834	0.751	×
C12	5612.8	1.349	2.20%	1.08%	1.55%	1.834	0.9361	√
C14	7612.8	0.302	1.10%	1.08%	1.09%	1.83	0.834	√
Wall corner	2142.4	0.818	1.10%	1.08%	1.09%	1.834	0.834	√

√-punching shear reinforcement do not required
×-Punching shear reinforcement required

Perimeter at which punching shear links are no longer required
L-Length attributable to column faces
P_{stop}-Perimeter of shear reinforcement may stop

	u_{out} mm	L mm	r_{out} mm	1.5d mm	P_{stop} mm	$S_{r,max}$ mm	say mm	$S_{t,max}$(<2d) mm	say mm	$S_{t,max}$(<2d) mm	say mm
C1	4293	1500	889	431	457.9	215.63	175	431.25	350	575	500

Area of reinforcement

	$f_{ywd,ef}$ MPa	A_{sw} ≥ mm^2	$A_{sw,min}$≥ mm^2	d_{bar} mm	No.legs	$A_{sw,pro}$ mm^2	A_{sw}/u_1 mm^2/mm	S_{max} mm	0.75d mm	S-say mm
C1	321.9	612.95	46.2	8	1	50.27	0.2338	215	215	175

7 Retaining wall calculation

7.1 Retaining wall analysis and basic calculation parameters

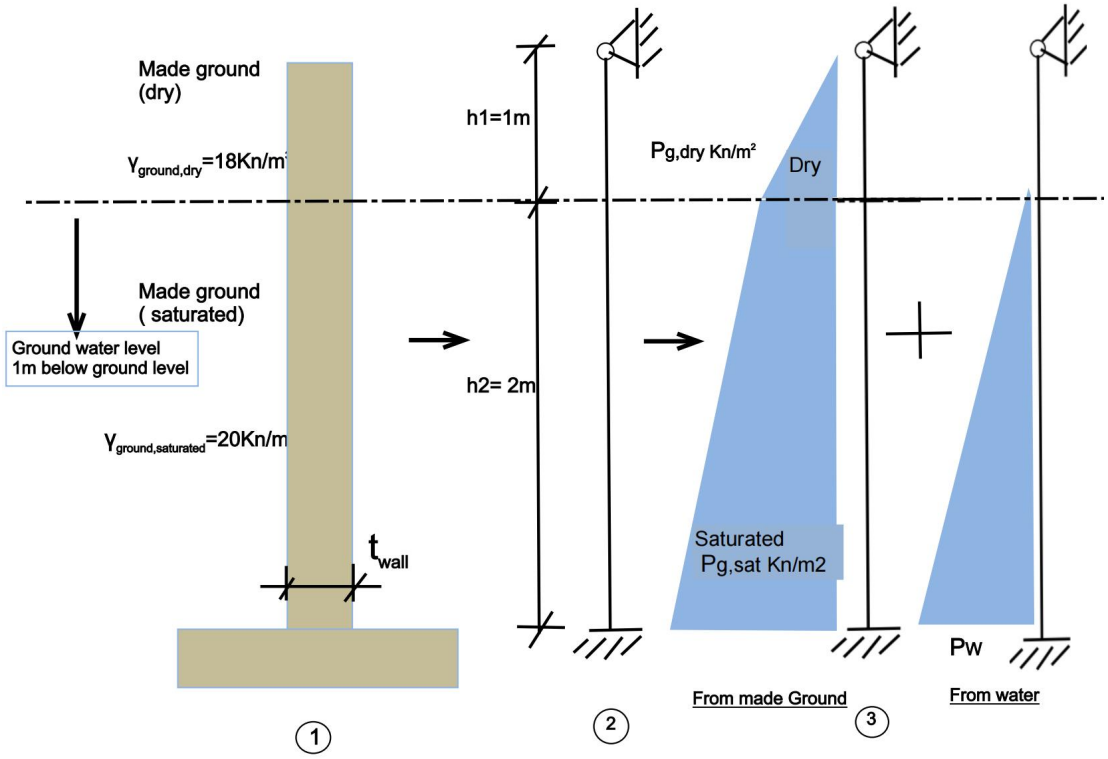


Figure 7- 1 Detailed illustration of retaining wall load conditions

As introduced in the design report, the retaining walls will be designed to resist later forces according to the ground and underground water conditions.

As groundwater is 1 m below the ground level, different soil bulk densities will be calculated; the saturated ground bulk density will be higher than dry ground.

The lateral load brought by water is 2 m high from the foundation.

The wall at the bottom will be fixed as the raft foundation will restrain its rotation while the upside is pinned (slabs will provide lateral restraints).

In this calculation, the retaining will mainly be designed to resist bending moments and shear forces (like beam behaviour). The reinforcement to resist tensile stress will be provided.

In basements and retaining structures, the primary concern is leaking "through" cracks that traverse the full thickness of the member. For this reason, crack control becomes important in basement wall design, especially in a groundwater environment.

At the serviceability limit, crack widths must be regulated within predetermined limitations. In building constructions, this is often accomplished by ensuring reinforcement remains elastic under service loads and setting maximum reinforcement bar spacing based on the design crack width.

7.2 Basic parameters for calculations

The density of water	ρ_{water}	=	1000	Kg/m³
Bulk density(water) $\gamma_{water}=\rho_{water}\times g$ assumes g=10N/kg	γ_{water}	=	10	Kn/m³
Bulk density for dry made ground	$\gamma_{ground,\ dry}$	=	18	Kn/m³
Bulk density for saturated made ground	$\gamma_{ground,\ saturated}$	=	20	Kn/m³
Height for Made Ground(dry)	h_1	=	1	m
Height for Made Ground(saturated)	h_2	=	2	m
Coefficient of static earth pressure	K	=	0.5	

Nonuniform pressure from saturated Made Ground at the bottom of h1 $P_1 = K \times \gamma_{ground,dry} \times h_1$	$P_{g,dry}$	=	9	Kn/m^2
Nonuniform pressure from water at the bottom of h2 $P_2 = K \times \gamma_{g,dry} \times h_1 + K \times (\gamma_{g,saturated} - \gamma_{water}) \times h_2$	$P_{g,sat}$	=	19	Kn/m^2
Nonuniform surface pressure from water at the bottom of h2 $P_w = \gamma_w \times h_2$	P_w	=	20	Kn/m^2
Design basement wall per meter				
Nonuniform line load from saturated Made Ground at the bottom of h1	$Q_{g,dry}$	=	9	Kn/m
Nonuniform line load from water at the bottom of h2	$Q_{g,sat}$	=	19	Kn/m
Nonuniform line load from water at the bottom of h2	Q_w	=	20	Kn/m

7.3 Design moment and shear force calculations

Conservatively consider the made ground as a dead load and the underground water as a live load. Thus, the ULS load combination is applied in this design. Meanwhile, the wall is designed as per meter width.

Load combination 1.35 × ground pressure+1.5 × water pressure

The load condition is complicated shown in Figure 7-1, the bending moments and shear forces will be solved by using SAP2000 as shown in Figure 7-2.

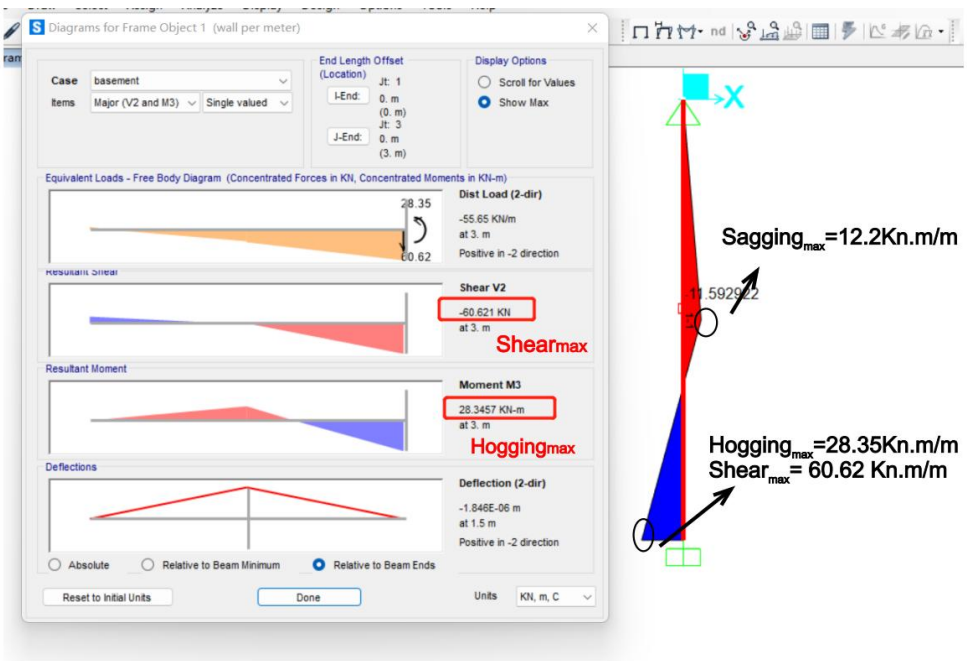


Figure 7-2 Bending moment and shear force on retaining walls (unit width)

7.1 Reinforcement design for shear walls

Cl. 4.4.1.2(3) Exp. (4.1)

The maximum sagging moment will induce tension stress on the inner side of the wall, while the outer side will induce tension stress on the outer side of the wall.

7.1.1 Material and dimension information for retaining walls

EC2-1-2: 5.6.3(1), Table 5.6

considering unit width of wall to be designed

f_{ck}	f_{cd}	E_{cm}	ρ_c	f_{yk}	f_{yd}	h=	t_{wall}	b_{wall}	E_s	m=α_e
MPa	MPa	MPa	(Kn/m^3)	MPa	MPa	m	mm	mm	MPa	MPa
30	17	32.84	25	500	434.8	3	250	1000	30	17

$Modular\ ratio: m = \alpha_e = E_s/E_c$

Cl. 5.1.3(1) & NA

7.1.2 Cover

Table NA.1 (option b)

assuming 16 mm diameter reinforcement	$c_{min,b}$	=	16	mm	
Minimum cover due to environmental conditions. For S4 and XS1	$c_{min,dur}$	=	45	mm	
C_{min}=max($C_{min,b}$;$C_{min,dur}$;10mm)	fixed value	=	10	mm	
	c_{min}	=	45	mm	
C_{nom} =C_{min}+ ΔC_{dev}	ΔC_{dev}	=	10	mm	
	C_{nom}	=	55	mm	
For 2 hours of resistance Minimum thickness for cover	a_{min}	=	51	mm	Satisfy

7.1.3 Flexural design sagging moment for internal vertical reinforcement design

Assumed links diameter	d_{links}	=	10	mm	
Effective depth	d_{eff}	=	177	mm	
Flexture in span	k	=	0.013		
	K'	=	0.153		
False→Compression reinforcement requirement True→no compression reinforcement required	k>k'?	=	True		
Lever arm	Z	=	174.9	mm	
	$A_{s,t,req}$	=	160.4	mm^2	Satisfy
Provide 16H@500 internal					
	bar diameter	=	16	mm	
	Spacing	=	500	mm	
	Number of bars	=	8		
	$A_{s,pro}$	=	402.1	mm^2	

C1.7.3.3(2) C1.7.3.1(5)& NA

7.1.4 Flexural design-hogging moment for outer vertical reinforcement design

Table 7.2N& NA

Assumed links diameter	d_{links}	=	10	mm	
Effective depth	d_{eff}	=	177	mm	
Flexure in span	k	=	0.013		
For simply supported(no redistribution)	K'	=	0.153		
false→Compression reinforcement requirement True→no compression reinforcement required	k>k'?	=	True		
Lever arm	Z	=	174.9	mm	
	$A_{s,t,req}$	=	372.7	mm^2	Satisfy
Provide 16H@500 external					
	bar diameter	=	16	mm	
	Spacing	=	500	mm	
	Number of bars	=	8		
	$A_{s,pro}$	=	402.1	mm^2	

7.1.5 Deflection check

EC2 expression 7.16.a 7.16.b

Check the span-to-depth ratio.

Table 7.4N & NA

	ρ_0	=	0.55%
Basic span: effective depth ratio for	ρ	=	0.23%
Factor for simply-supported	K	=	1
As h<7m	reduction factor	=	1
	$(l/d)_{premissible}$		60.18
	$d>d_{permis}$		Satisfy

7.1.6 Shear design

Conservative consideration	$V_{ed}=V_{max}$	=	60.62	KN
$ved=V_{ed}/bh$	v_{ed}	=	0.242	MPa
maximum shear capacity:	As cotθ	=	2.5	
For fck=30MPa	$v_{Rd,max}$	=	3.64	
	$v_{Rd,max}>v_{ed}$	=	Satisfy	
Shear reinforcement				
Assuming z=0.9d	z	=	159.3	mm
Asw/s≥Ved/(z×fywd×cotθ)	Asw/s	≥	0.35	
Minimum shear links,Asw,min/s=0.08bwfck0.5/fyk	Asw,min/s	=	0.876	CRITICAL
Provide H8@100				
	bar diameter	=	8	mm
	As,2legs	=	100.53	mm2
	Spacing	=	100	mm
	Asw/s,pro	=	1	
Asw,pro/s≥Asw,min/s		=	Satisfy	
$Spacing<Spacing_{max}=0.75d$, ok	$Spacing_{max}$	=	132.8	mm

7.1.1 Summary of design

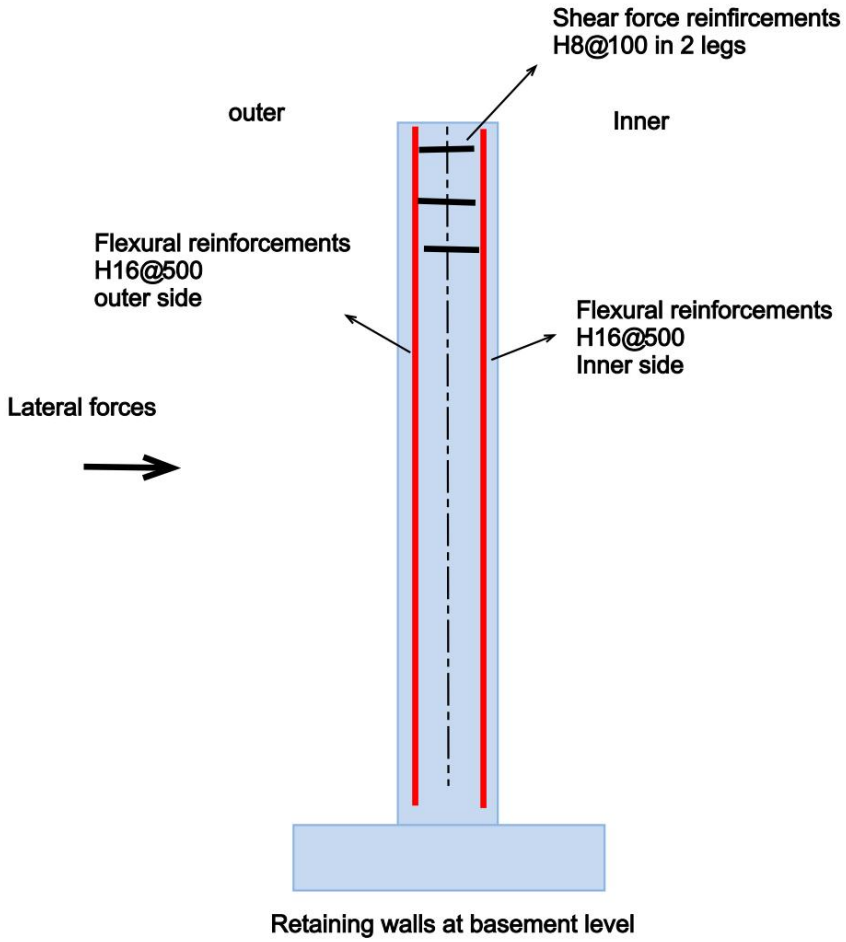


Figure 7- 3 Summary of transfer beam design

Flexural reinforcements	
internal	external
H16@500	H16@500
Shear reinforcements	
H8@100	

7.1.2 Detailing check

a) Minimum areas

$f_{ctm} = 0.30 \times f_{ck} 0.666$	f_{ctm}	=	2.89	MPa
Width of the tension zone	bt	=	1000	mm
$A_{s,min} = 0.26(f_{ctm}/f_{yk})b_t d \geq 0.0013 b_t d$	$A_{s,min}$	=	266	mm^2
For simply supported(no redistribution)	$A_{s,pro}$	=	402.1	mm^2
	Check	=	Satisfy	

b) Curtailment of main bars

Bottom: Curtail

75% main bars 0.08L from end support	0.08L	=	240	Say	300	mm
70% main bars 0.3L- al From left support (al=1.125d)		=	700.9	Say	700	mm

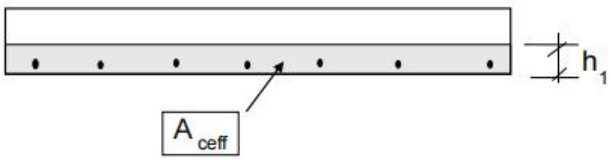
At support

25% of As to be anchored at supports	25% As	=	40.1	mm^2
Provide 4H22				
	bar diameter	=	8	mm
For simply supported(no redistribution)	Number of legs/m	=	2	
	As,pro	=	100.53	mm^2
	Check		Satisfy	

EC2 7.3

7.1.3 Calculation of crack width -long-term crack width

For cracked section

The depth to neutral axis	x	=	52	mm
The second moment of area $I_2 = bx^3/3 + mA(s-x)^2 + mA_s'(x-d')^2$	I2	=	2.17E+08	mm^4
For flexure	K1	=	266	mm^2
	K2	=	402.1	mm^2
For half of the section				
	cover	=	55	mm
Diameter of bars	ϕ	=	16	mm
For flexure $h_{cri} = \min(2.5(c+0.5\phi), (h-x)/3)$	h_{cri}	=	24.33	mm
 Figure 7- 4 illusion for effective area	A_{ceff}	=	24333	mm^2

EC2 Figure 7.1

EC2 expression 7.10

	As	=	402.1	mm^2
$\rho_{peff} = A_s/A_{ceff}$	ρ_{eff}		2%	
Crack spacing $S_{max} = 3.4c + 0.425k_1k_2\phi/\rho$	S_{max}	=	351.6	mm

7.1.4 Calculation of crack width

The critical moment for walls	M_{cri}	=	28.35	Kn.m/m
$\sigma_s = ME_s(d-x)/(E_cI_2)$	σ_s	=	1000	MPa
For flexure	kt	=	266	
$A_{s,min} = 0.26\left(\frac{f_{ctm}}{f_{yk}}\right)b_td \geq 0.0013b_td$	$A_{s,pro}$	=	402.1	mm^2
$\varepsilon_{sw} - \varepsilon_c = \frac{\sigma_s - k_t\frac{f_{ctef}}{\rho_{peff}}(1+\alpha\rho_{eff})}{E_s}$	$\varepsilon_{sw} - \varepsilon_c$		4E-04	
check≥$0.6\frac{\sigma_s}{E_s}$		=	no	

EC2 expression 7.9

Section will not crack due to moments.

7.1.5 Crack width due to restrained imposed deformation

Coefficient of thermal expansion	α	=	1.2E-05	MPa
Temperature drop	ΔT	=	40	℃
Free shrinkage strain	ε_{sh}	=	300	μs
Restraint factor	R	=	0.5	mm^2
Reinforcement factor	ρ	=	0.002	
$\varepsilon_{sm} - \varepsilon_{cm} = R_{free} = R(\alpha\Delta T + \varepsilon_{sh})$	$\varepsilon_{sm} - \varepsilon_{cm}$	=	4E-04	
Crack width $w_k = S_{max}(\varepsilon_{sm} - \varepsilon_{cm})$	W_k	=	0.137	mm
The cracking due to restraint is 0.137mm				

EC2 Expression 7.8

8 Reference

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