ENGM041 COURSEWORK

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1. Structural System (*page 2*)

In response to The Concrete Centre Student Structural Competition 2025 (TCC, 2025), the structural scheme appraised, shown in Page 2, features;

- a concrete framed structure braced using reinforced concrete shear walls and moment frames founded on piled foundations.
- columns are generally spaced at 10 m in the W-E direction and both 9 m and 12 m in the N-S direction.
- 390 mm biaxial voided "BubbleDeck" (BD) slab employed on all floors including ground floor which is suspended. This slab type was selected based on {1} large spans required {2} flexibility for partitions and {3} minimising the storey height of the building.
- 2 Y-shaped columns, aligned in a single row, employed below the restaurant floor for aesthetic appeal and reduce the large spans in that area. 4 m of the slab edge is cantilevered, based on the allowable span for the 390 mm BD slab thickness (BubbleDeck UK, 2024a).
- shear wall bracing in the N-S direction and moment frame bracing in the E-W direction. Hence, columns have been aligned with the largest dimension parallel to the E-W direction.

1.1. Vertical load transfer (page 2)

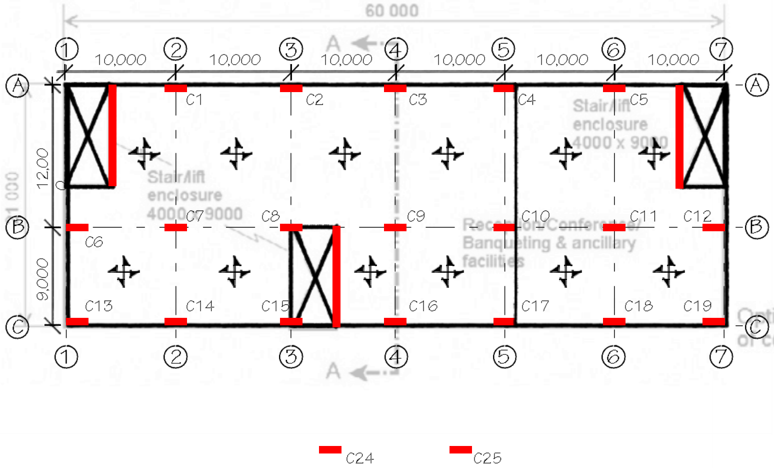
The gravity loads are supported on the BD slabs that transfer the loads to the reinforced concrete columns and walls through bending and shear action. The columns and walls via bending and shear actions transfer the loads to pile caps. Pile caps then transfer the loads to cast in-situ bored piles through bending and shear action. Cast in-situ bored pile foundation transfer the loads to the ground by friction and bearing actions. Hydrostatic water pressure on the building created by water uplift during construction phase is resisted by the piles acting in tension. The slab portion in between the Y-column members acts in tension exerting a thrust force to the inclined members.

1.2. Lateral load transfer (page 2)

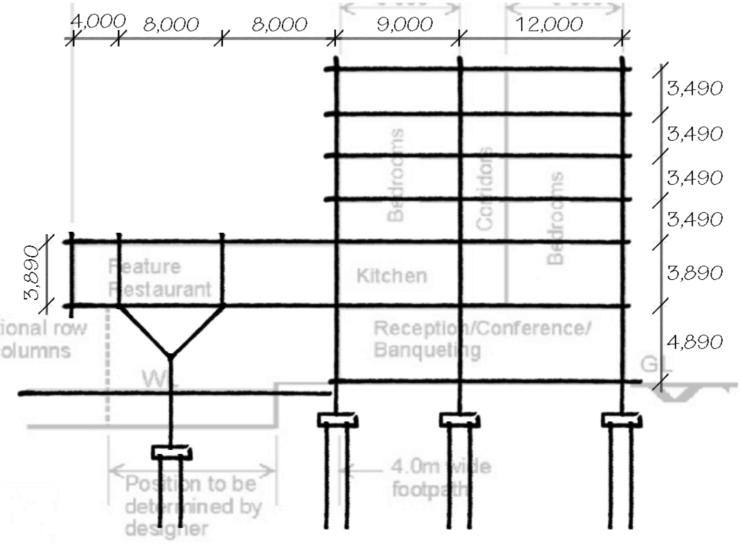
Lateral wind loads are initially resisted by the building's cladding, which transfers these forces to the floors and roof through a combination of bending and shear. The floors and roof act as horizontal diaphragms, distributing wind-induced forces to the structural cores in the north-south direction and to the columns in the east-west direction, again primarily through bending and shear mechanisms. The cores and columns work together to resist the overall wind forces, channelling them downwards. These forces are then delivered to the pile caps, which are supported by cast in-situ bored pile foundations. The foundations ultimately transfer the loads safely into the ground via both skin friction along the pile shaft and end bearing at the pile tip.

1.3. Movement joints

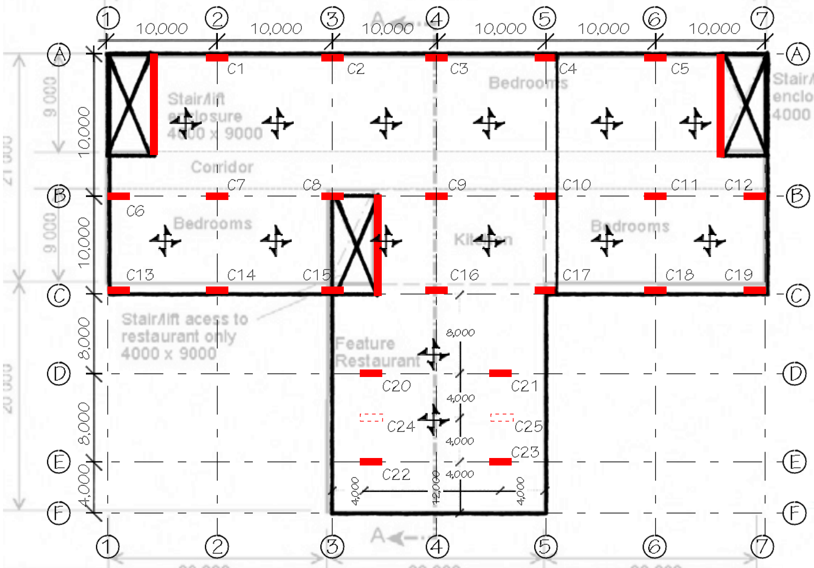
Due to the large E-W plan length of 60 m which is larger than the UK standard practice of 50 m, a movement joint is placed on Grid 5, **Fig 1** (CIRIA, 2014). A movement joint, to account for the differential settlement due to the difference in building height between the projecting restaurant area and the rest of the building, is adopted along Grid D. The foundations ought to be designed to settle similarly. To avoid scheduling issues and safety concerns a typical pour strip is not selected. Instead a PS=0 stress relief joint shown in **Fig 1** is used (Reigstad, n.d.). This avoids {1} having 3 separate structural systems and {2} double columns which are and eyesore and require maintenance.



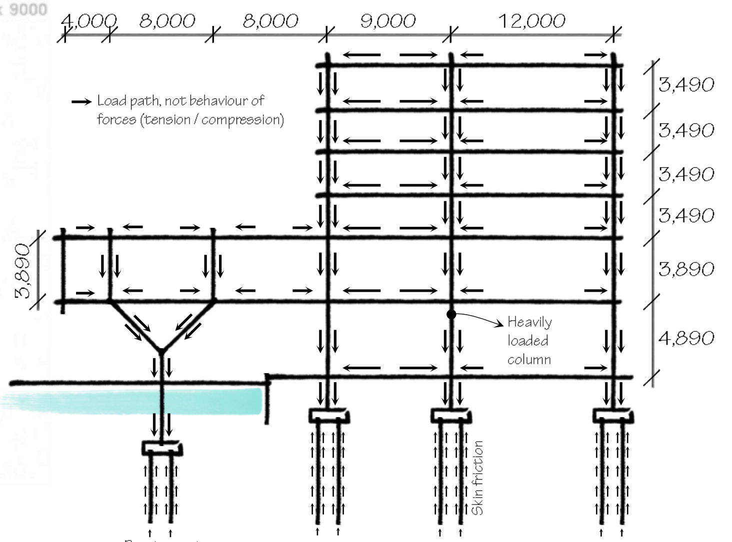
390 MM SUSPENDED GROUND FLOOR SLAB



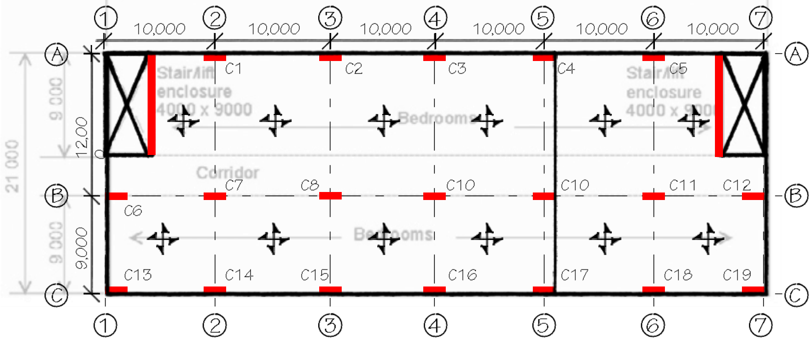
ELEVATION



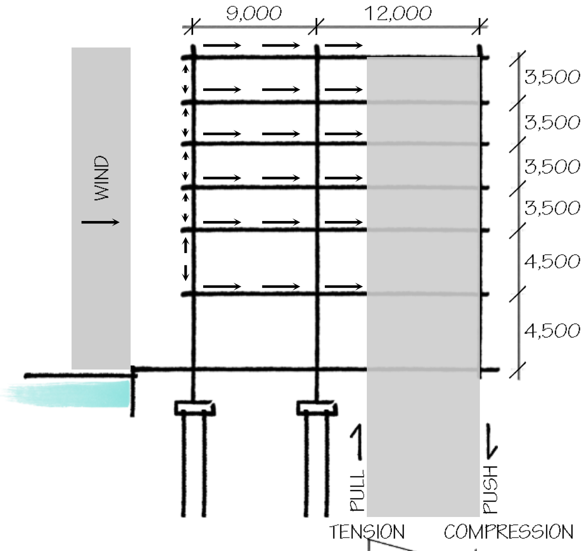
390 MM 1ST & 2ND FLOOR BUBBLEDECK SLAB



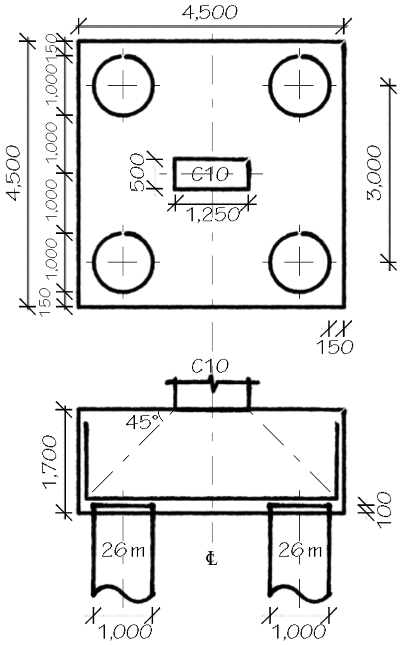
GRAVITY LOAD PATH



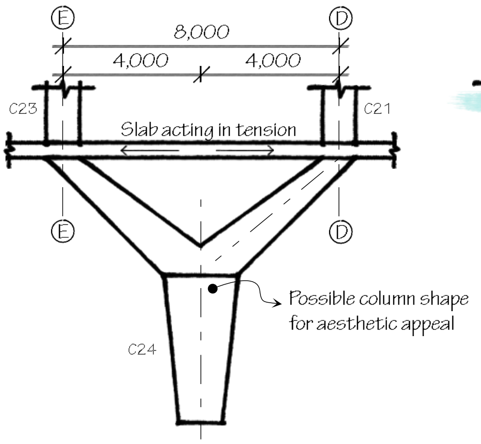
390 MM TYPICAL FLOOR BUBBLEDECK SLAB



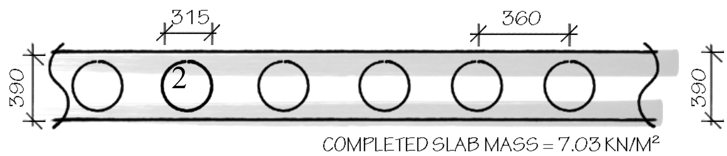
LATERAL LOAD PATH



FOUNDATION SYSTEM



Y - COLUMN



BUBBLEDECK® SLAB

2. Durability requirements

2.1. Cover

Table 1. Cover

Parameter	Members		
	Slab	Column	Foundation
f_{ck}	30/37	35/45	35/45
c_{nom}	30 mm[a]	40 mm	75 mm

Note.
[a]This is for an typical interior slab. Due to the semi-precast process , it is assumed that special quality control will be applied.

2.2. Fire resistance

The design of concrete slabs is not generally affected by fire design requirements, as opposed to columns (The Concrete Centre, 2015). Fire resistance of the structural members were assessed using Method A of *BS EN 1992-1-2* table 5.2a where $e < 0.15b$ which give the below minimum dimensions summarised in Table 1. The large covers provided for the structural members also increased their axis distance and in turn lowered the minimum dimension b_{min} required. At this stage conservative fire exposure conditions were selected for a rapid assessment.

Table 2. Minimum dimensions required for fire resistance

	Structural member	b_{min} / a
REI 90	Column exposed on more than one side	350/53
	Wall exposed on two sides	170/25

3. Foundation

The pile foundation system ,cased continuous flight auger in-situ piles, was selected to bypass the compressible thick 2.5 m made-ground layer and 2.5 m thick river terrace deposits onto the stiff clay. The high groundwater level {1} reduces bearing capacity of the possible bearing layers {2} complicates excavation, requiring costly dewatering, shoring, and management of potentially contaminated water (Norman et al., 2020). The illustration on Page 2 shows a preliminary sizing for column C10 whose sizing is done on Appendix A.1.1. Piles should be designed to act in tension during the construction phase of the building i.e. when self-weight is less than uplift forces.

4. Slab

Table 3. Slab system

Section	Parameter	Comment
GF – ROOF	Slab system	Biaxial voided
	Construction method	Semi-precast
	Slab depth	390.00 m
	Precast depth	80.00 mm

Notes: Minimising storey height, the large spans and were the governing factors used to select the floor solution (TCC, 2009, tbl. 2.1). Ground floor slab was suspended as the made-ground layer does not provide a suitable bearing layer.

5. Column

Table 4. Vertical members summary

	C1	C2	W1
Size (mm)	1000 x 500	1300 x 500	250 x 9000

Notes: To enable reuse of formwork the same section was maintained from the foundation to the roof.

6. Method statement

Pre-Construction Planning {1} Induct staff with site-specific risk assessment highlighting key safety areas e.g. projecting restaurant area, and working at large heights {2} Develop traffic management and delivery strategy as the city centre site may need single lane closure and splices in elements larger than 12 m to allow normal vehicle without an escort.

Site preparation and enabling works {1} Clear site and secure using perimeter hoarding to prevent unauthorized access by members of the public {2} Setup a dry dock using sheet piles to facilitate the construction of the projecting restaurant area as shown in **Fig 1** {3} provide a working platform over the whole site of 300 mm thick hardcore laid on geomembrane to allow piling to commence {4} the made ground must be compacted and levelled to provide hard standing for the storage and assembly of the superstructure, and for crane and mobile elevating working platforms (MEWPs) to operate within and adjacent to the footprint of the building. {5} full PPE must be worn, including fall arrest equipment and safe methods of work must be established and followed. {6} suitable equipment must be provided and maintained for use in the event of a fall into the water.

Substructure works Setup dewatering wellpoints to facilitate construction of piles and pile caps. Excavate pile caps, trim sides and blind base. Cut off piles at correct level ensuring sufficient projection into pile caps. Integrity of the masonry blocks must be maintained through monitoring.

Superstructure works The construction of the BD slab shall follow the combined column and slab erection method listed in (BubbleDeck UK, 2024b) for the following advantages {1} condenses the two-stage process (separate erection/casting of columns/walls and BD slabs) into a single stage {2} provides a stable and firm platform for casting columns and walls {3} eliminates the need for separate concrete deliveries for columns/walls and slabs and {4} ensures a good bond between the column/wall and the BD slab site concrete. However, do not overpour columns or walls; pour only up to the underside of the BD flat slab to avoid reducing the slab's effective depth at supports. If columns or walls are poured above this level, the excess concrete may need to be removed around their perimeter to ensure proper connection with the BD slab. To accommodate early-age contractions, a pour sequence of three pours, each 20 m by 21 m (Clarke, 2020). Around the shear walls, a 1 m pour strip will be left and cast four to six weeks later (CIRIA, 2014). Concrete will be supplied by a ready mix supplier and pumped to the working level. The location of machinery is shown in **Fig 2**.

MEP & cladding MEP to start when 3 floors of frame have been completed for non-sensitive items. Once cladding complete (watertight building) commissioning and installation of services.

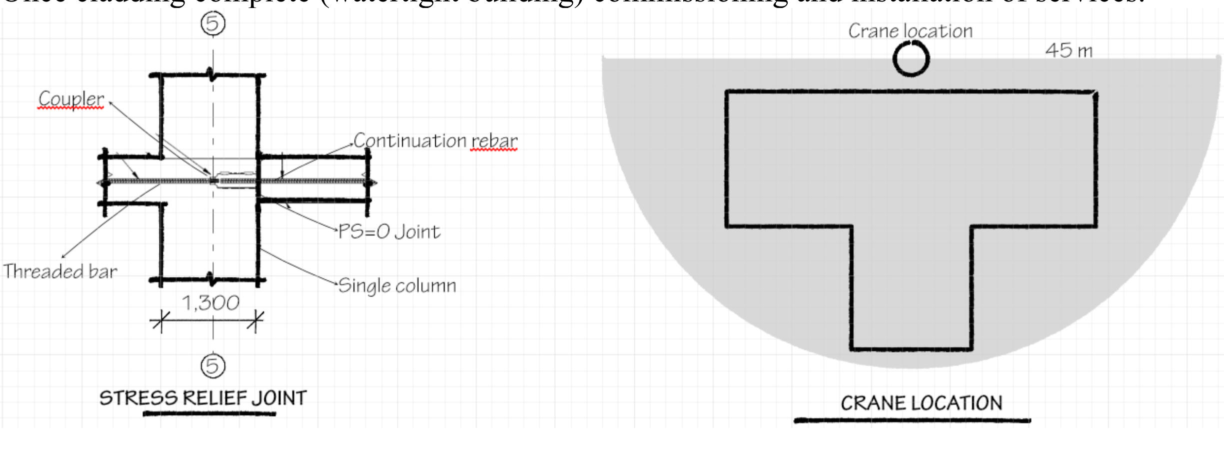


Fig 1. (Left) Detail of movement joint adopted on Grid 5 and Grid D of the building. (Right) Crane location and the required boom length.

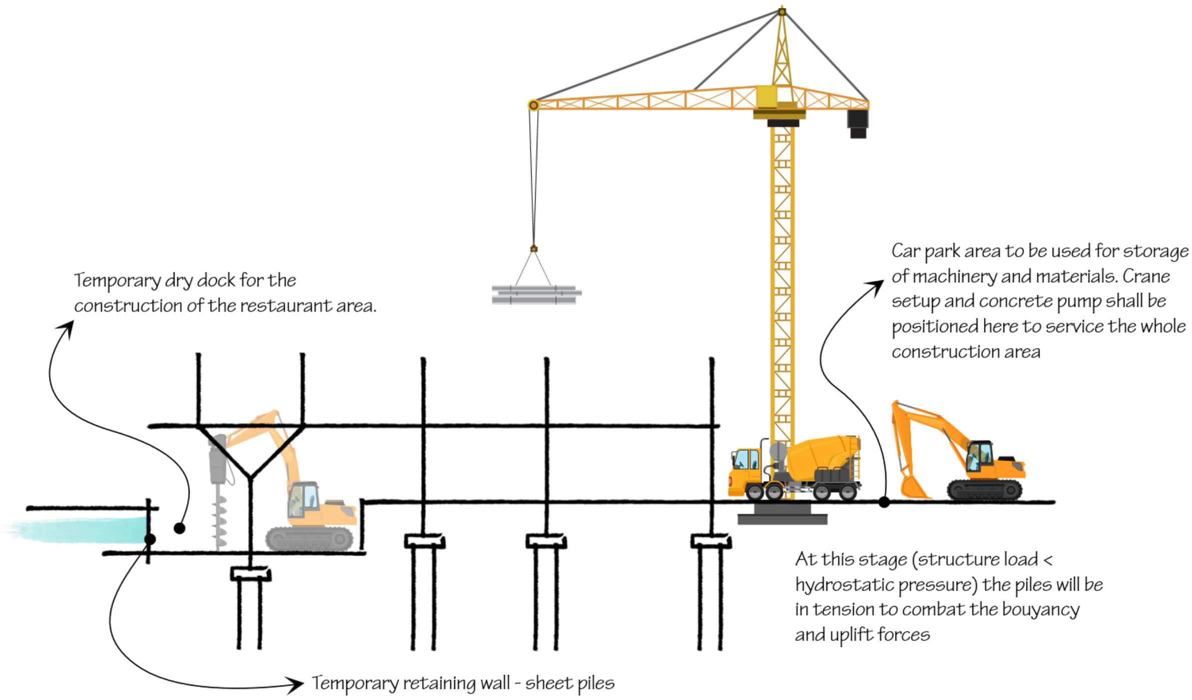


Fig 2. Section of building at construction stage

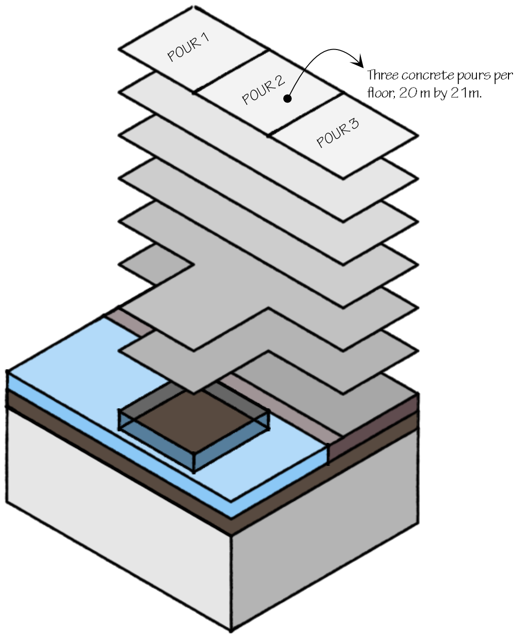


Fig 3. Location of concrete pours. In the dry dock, horizontal bracing props are required to support the sheet piles.

7. Robustness

The building falls under consequence class 2b hence horizontal ties and vertical ties need to be provided ("BS EN 1991-1-7:2006+A1:2014," 2014; *Approved Document A*, 2013). Well-detailed in situ concrete is inherently robust and in most cases, checks for compliance with tying rules will show that tying requirements have already been met through the standard reinforcement provided (IStructE, 2023). Notional column removal check should be done for the Y-columns in the projecting restaurant area, to ensure that structure remains stable and collapse does not exceed 15% of the floor area of that storey or 100m^2, whichever is smaller, and does not extend further than the immediate adjacent storeys. Key element design may be required if damage extent exceeds limit.

8. Wind

The Port Aspdin Hotel is located on a former docks site on an estuary close to the centre of a large UK town. Therefore the terrain category is IV. The wind peak velocity pressure is 0.791 kN/m^2. The wind load will be divided onto the three frames in the E-W direction based on the area covered. The frames are assumed to be of equal stiffness. In this report wind effect on the loads is considered minimal and therefore is neglected for the frame analysis. However a detailed analysis will be required for the structural walls.

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Appendix A. Calculations

A.1. Preliminary

A.1.1. Slab

The span / effective depth ratio for multiple spans of a BubbleDeck is 39 (BubbleDeck UK, 2024a).
$l / d \quad < 39$
$= 12000 / 39 = 307.69$ mm $= 308$ mm
For 90 mm fire resistance a cover of 25 mm is added (BubbleDeck UK, 2024a).
$= 308$ mm $+ 25$ mm
$= 332.69$ mm
From the available modules provided by BubbleDeck, a size of 390 mm is selected.

A.1.2. Column sizing

The axial load P on column C10 is 9,782.3925 kN from the load rundown calculated using the tributary area method in **Table 5**. The results of the rundown are summarized in **Table 6**.

Table 5. Load rundown calculation

		Panels				
		1 (30 m^2)	2 (30 m^2)	3 (22.5 m^2)	4 (22.5 m^2)	P, kN
Roof	g_k	7.03	7.03	7.03	7.03	996.5025
	q_k	0.75	0.75	0.75	0.75	118.125
Typical (×4)	g_k	7.03	7.03	7.03	7.03	3986.01
	q_k	3.00	3.00	2.00	2.00	1620
1st	g_k	7.03	7.03	7.03	7.03	996.5025
	q_k	3.00	3.00	2.00	3.00	438.75
GF	g_k	7.03	7.03	7.03	7.03	996.5025
	q_k	4.00	4.00	4.00	4.00	630
						9782.3925

From (IStructE, 2019, tbl. 4.3), for C35/45 a 2% reinforcement percentage gives an equivalent stress of 22 MPa.

$$A_{col} = P / 22 \text{ MPa} = 9{,}782.3925 \text{ kN} / 22 \text{ MPa} = 555{,}817.76 \text{ mm}^2$$

Assuming a 400 mm thickness, 550,704.12 kN / 400 mm = 1,389.54 mm and rounding up gives 1400 mm. However, the columns were limited 1300 mm to avoid very long columns. This the reinforcement percentage will increase. The shear walls are sized as 250 mm thick using the maximum clear height of 4.1 m at the ground floor (TCC, 2009, tbl. 2.24).

Table 6. Load rundown summary

Col. No	Type	Factor	P SLS (kN)	P ULS (kN)	Area (mm^2)	/ 400 mm (mm)	⌈/ 400 mm⌉ (mm)
1	Edge	1.50	3,502.08	4,834.01	289,827.82	724.57	750
2	Edge	1.50	4,269.60	5,896.71	362,284.77	905.71	950
3	Edge	1.50	4,269.60	5,896.71	362,284.77	905.71	950
4	Edge	1.50	4,269.60	5,896.71	362,284.77	905.71	950
5	Edge	1.50	3,502.08	4,834.01	289,827.82	724.57	750
6	Edge	1.50	2,380.80	3,284.96	184,210.57	460.53	500
7	Interior	1.25	6,140.28	8,523.94	484,314.80	1,210.79	1250
8	Interior	1.25	6,978.30	9,692.39	550,704.12	1,376.76	1400
9	Interior	1.25	7,000.80	9,726.14	552,621.73	1,381.55	1400
10	Interior	1.25	6,978.30	9,692.39	550,704.12	1,376.76	1400
11	Interior	1.25	6,140.28	8,523.94	484,314.80	1,210.79	1250

12	Edge	1.50	2,470.80	3,419.96	193,415.11	483.54	500
13	Corner	2.00	1,871.10	2,575.77	181,142.39	452.86	500
14	Edge	1.50	3,310.20	4,568.33	271,713.58	679.28	700
15	Edge	1.50	4,079.33	5,642.93	344,981.76	862.45	900
16	Edge	1.50	6,758.95	9,280.11	592,971.39	1,482.43	1500
17	Edge	1.50	4,028.70	5,567.00	339,804.20	849.51	850
18	Edge	1.50	3,310.20	4,568.33	271,713.58	679.28	700
19	Corner	2.00	1,871.10	2,575.77	181,142.39	452.86	500
20	Interior	1.25	2,320.00	3,216.38	182,748.58	456.87	500
21	Interior	1.25	2,320.00	3,216.38	182,748.58	456.87	500

A.1.3. Foundation

Column C10, Fig 1 with a axial load of 9,692.39 kN, is used to size the pile and pile cap. Assumptions made were {1} all 26 m of piling rig and the largest case of 1 m, the pile is embedded 21 m in the stiff clay (SKANSKA, n.d.) {2} made ground and river terrace deposits are neglected, hence do not contribute to the resistance of the pile and {3} cautious values for ground conditions were provided hence a conservative factor of 3 was adopted.

$\gamma_b = 19.4$ kN/m^3; $\gamma_w = 10$ kN/m^3; $N = 20$; $\alpha = 0.45$; $c_u = 150$ kN/m^2; $\phi_p = 1$ m; $l_p = 21$ m

Allowable load; $Q_a = (Q_b + Q_s) / 3$
Shaft resistance; $Q_s = \alpha \times c_u \times A_s$
Base resistance; $Q_b = 9 \times c_u \times A_b$

$$
\begin{aligned}
Q_a &= (Q_b + Q_s) / 3 \\
&= (\alpha \times c_u \times l_p \times \pi \times \phi_p + 9 \times c_u \times \pi \times \phi_p^2) / 3 \\
&= (0.45 \times 150 \text{ kN/m}^2 \times 21 \text{ m} \times 3.142 \times 1 \text{ m} + 9 \times 150 \text{ kN/m}^2 \times 3.142 \times (1 \text{ m})^2) / 3 \\
&= 2898.119 \text{ kN} \therefore 4 \text{ piles provided to resist the column C10 load of } 9{,}692.39 \text{ kN.}
\end{aligned}
$$

The plan area of the pile cap, shown in Figure 1, was sized by {1} spacing the piles at 3 × diameter of the pile in both directions and {2} adding applying 150 mm minimum distance from edge of pile cap to edge of pile to account for 75 mm pile location tolerance (IStructE, 2013). The depth of the pile cap employed a 45° load spread angle from the edge of column C10 (IStructE, 2013).

A.2. Detailed design

A.2.1. Cover BS EN 1992-1-1:2004

Table 4.3N	Structural class = S1.
Table 4.4N	$c_{min,dur} = 10.0$ mm, for structural class S1 and exposure class X0
§4.4.1.2(3)	$c_{min,b} = 20.0$ mm,
§4.4.1.3(1)P	$c_{min} = \max\{c_{min,b}, c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}, 10 \text{ mm}\}$
§4.4.1.2(6)	$\Delta c_{dur,\gamma} = 0.0$ mm, $\Delta c_{dur,st} = 0$ mm, $\Delta c_{dur,add} = 0$ mm.
§4.4.1.3(1)P	$c_{min} = \max\{20.0 \text{ mm}, 10.0 \text{ mm} + 0.0 \text{ mm} - 0 \text{ mm} - 0 \text{ mm}, 10 \text{ mm}\} = 20.0$ mm
	$c_{nom} = c_{min} + \Delta c_{dev} = 20.0 \text{ mm} + 10.0 \text{ mm} = 30.0$ mm

A.2.2. Slab, Zone CBS EN 1992-1-1:2004

Completed slab mass $g_k = 7.03$ kN/m^2
Bedroom & corridor $q_{k,b\&c} = 3.0$ kN/m^2
Kitchen $q_{k,kitchen} = 3.0$ kN/m^2 (BubbleDeck UK, 2024a)
Restaurant $q_{k,r} = 2.0$ kN/m^2

Slab thickness, $d = 390$ mm
Nominal reinf dia, $\phi_{nom} = 12$ mm
Cover, $c_{nom} = 35$ mm
$d_{eff} = d - c_{nom} - \phi_{nom} = 390 \text{ mm} - 30 \text{ mm} - 20 \text{ mm} = 350$ mm

The minimum span < 0.85 maximum (design) span 9000 mm / 12000 mm = 0.75. However the coefficients are still used knowing that the moments will be conservative. Also effective spans are ignored to provide a conservative design.

$\gamma_g = 1.35$, $\gamma_q = 1.5$, $\gamma_s = 1.15$, $\gamma_c = 1.5$

$n = \gamma_g \times g_k + \gamma_q \times (q_{k,b\&c}) = 1.35 \times 7.03$ kN/m^2 + 1.5 × (3 kN/m^2) = 13.991 kN/m^2 Cl. 5.1.3(1)

b_{slab} = 10 m, l_{slab} = 12 m

$A = b_{slab} \times l_{slab}$ = 10 m × 12 m = 120 m^2

$F = n \times A$ = 13.9905 kN/m^2 × 120 m^2 = 1678.86 kN

Middle strip width, $b_{x,mid} = b_{slab}$ / 2 = 10 m / 2 = 5 m

Coefficient_midspan = 0.09, Coefficient_support = 0.106 (Goodchild, 2009, tbl. C2)

$M_{Ed,mid}$ = 0.090 ×F × 12 m = 0.09 × 1678.86 kN × 12 m = 1813.169 kNm

$M_{Ed,sup}$ = 0.090 ×F × 12 m = 0.106×1678.86 kN×12 m = 1678.869 kNm

(TCC, 2007, tbl. 3)

	Column strip	Middle strip
-ve (hogging)	$= 0.7 \times M_{Ed,sup}$ = 1281.306 kNm	$= 0.3 \times M_{Ed,mid}$ = 725.268 kNm
+ve (sagging)	$= 0.5 \times M_{Ed,sup}$ = 906.584 kNm	$= 0.5 \times M_{Ed,mid}$ = 906.584 kNm

Designing for flexure at column strip and middle strip, sagging

f_{ck} = 30 N/mm^2, f_y = 500 N/mm^2

f_{yd} $= f_y / \gamma_s$ = 500 N/mm^2 / 1.15 = 434.783 N/mm^2

K $= M_{x,mid} / (b_{x,mid} \times d_{eff}^2 \times f_{ck})$
= 906.584 kNm / (5 m × (350 mm)^2×30 N/mm^2)
= 0.0493

z = Min(0.95 × d_{eff}, d_{eff} × (0.5 + √ (0.25 – K / 1.133)))
= Min(0.95 × 350 mm, 350 mm × (0.5 + √ (0.25 - 0.0411 / 1.133)))
= 332.5 mm

$A_{s,req}$ $= M_{x,mid}$ ×1 m / (0.87× f_y × z × $b_{x,mid}$)
= 906.584 kNm × 1 m / (0.87×500 N/mm^2 × 332.5 mm × 5 m)
= 1254.22 mm^2

Provide H20 @ 180 mm ($A_{s,prov}$ = 1745 mm^2). This 180 mm spacing is an average value to account for the uneven spacing of reinforcement spacing in BD slabs. (230 mm + 130 mm) / 2 = 180 mm. This is demonstrated in the reinforcement detailing.

Deflection: column strip and middle strip

$g_k / q_{k,b\&c}$ = 7.03 kN/m^2 / 3 kN/m^2 = 2.343

θ_2 = 0.2

$\gamma_g = 1.35$, $\sigma_s = 245$, δ = 1.03, $K = 1.2$, $F1 = 1$, $F2 = 1.0$

F3 = Min(310 / σ_s, 1.5) = Min(310 / 245, 1.5) = 1.265 Cl. 7.4.2, Exp. (7.17) Table 7.4N, & NA, Table NA.5 Note 5

ρ $= A_{s,prov}$ / (1000 mm × 390 mm)
= 1745 mm^2 / (1000 mm × 390 mm)
= 0.00447

N = 39.2

$(l/d)_{allow.}$ $= N \times K \times F1 \times F2 \times F3$ = 39.2 × 1.2 × 1 × 1 × 1.265 = 59.52 Cl. 7.4.2(2)

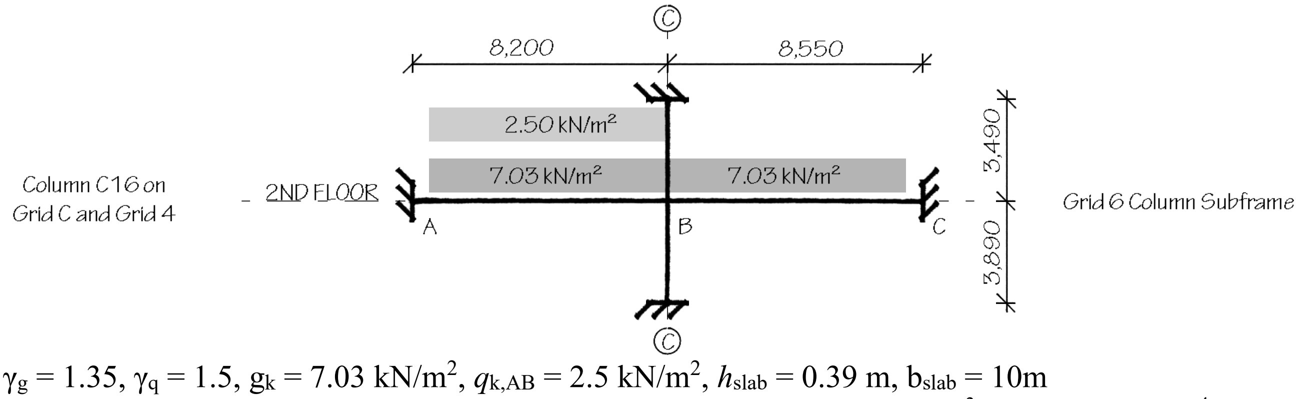
$(l/d)_{actual}$ = l_{slab} / d_{eff} = 12 m / 343 mm = 34.286
f_{ctm} = 0.3 × $(f_{ck})^{1/3}$ × 1 N/mm^2 = 0.3 × (30 N/mm^2)$^{1/3}$ ×1 N/mm^2
= 0.932 N/mm^2
b_t = 1000 mm
$A_{s,min}$ = 0.26 × (f_{ctm} / f_{yk}) × b_t × d_{eff}
= 0.26 × (0.93 N/mm^2 / 500 N/mm^2) × 1000 mm × 350 mm
= 169.665 mm^2
Provide A193 mesh in compression zones (193 mm^2/m^2).
Packing of reinforcement at the column strip is not possible in BD slab. The same spacing is maintained across the entire slab.
The rest of the design is summarised in
Table 7, together with the design of the cantilevered zone A. The analysis for Zone A is described below
g_k = 7.03 kN/m^2, q_k = 3.0 kN/m^2, l_1 = 4 m, l_2 = 8 m
w = 10 m× (1.35 × g_k +1.5 × q_k) = 139.905 kN/m
$M_1 = w \times L_1^2 / 2$ = 1119.24 kNm, $M_2 = w \times L_2^2 / 12$ = 746.16 kNm
$M_{1,k} = M_1 - (M_1 - M_2)\times(1/L_1)/(1/L_1 + 1/L_2)$ = 870.52 kNm,

Table 7. Slab design summary

	-ve, middle	-ve, column	+ve, column	-ve, middle	Zone A
M_x (kNm)	725.26	1281.30	906.58	906.58	870.52
K	0.039	0.070	0.049	0.049	0.047
z (mm)	332.5	326.94	332.5	332.5	332.5
$A_{s,req}$ (mm^2)	1003.38	1802.78	1254.22	1254.22	1204.33
Provide	H20 @ 180	H20 @ 1800	H20 @ 180	H20 @ 180	H20 @ 180
A_{prov} (mm^2)	1745	1745	1745	1745	1745

Notes. Despite the provided reinforcement being less than required, a detailed analysis will provide less conservative moments

A.2.3. Subframe analysis, Zone B



γ_g = 1.35, γ_q = 1.5, g_k = 7.03 kN/m^2, $q_{k,AB}$ = 2.5 kN/m^2, h_{slab} = 0.39 m, b_{slab} = 10m
L_{AB} = 8.5 m / 2 = 4.275 m, L_{BC} = 8.2 m / 2 = 4.1 m, $I_{AB} = 0.9 \times (b_{slab} \times h_{slab}^3 / 12)$ = 0.0445 m^4
$k_{AB} = I_{AB} / L_{AB}$ = 0.0104 m^3, $k_{BC} = I_{AB} / L_{BC}$ = 0.0109 m^3
h_{col} = 1300 mm, b_{col} = 400 mm, $I_{col} = h_{col} \times b_{col}^3 / 12$ = 0.00693 m^4
L_U = 3.49 m, L_L = 3.89 m, $k_U = I_{col} / L_U$ = 0.00199 m^3, $k_L = I_{col} / L_L$ = 0.00178 m^3
$\Sigma k = k_{AB} + k_{BC} + k_U + k_L$ = 0.0250 m^3
$w_{AB} = (\gamma_g \times g_k + \gamma_q \times q_{k,AB})\times b_{slab}$ = 132.405 kN/m, $w_{BC} = (\gamma_g \times g_k) \times b_{slab}$ = 94.905 kN/m
$M_{AB} = w_{AB} \times L_{AB}^2 / 12$ = 201.649 kNm, $M_{BC} = w_{BC} \times L_{BC}^2 / 12$ = 132.946 kNm
$M_U = (M_{AB} - M_{BC}) \times k_U / \Sigma k$ = 5.454 kNm, $M_L = (M_{AB} - M_{BC}) \times k_L / \Sigma k$ = 4.893 kNm

The same procedure was repeated along Grid 4, and the moments calculated were M_U = 25.094 kNm and M_L= 22.513 kNm. The axial load of 6788 kN was calculated from the load rundown in **Table 6**.

A.2.3 Punching Shear Check, Zone C (BS EN 1992-1-1:2004)

Shear force was calculated as the axial load on the column from the tributary area of the 1st floor.

$$g_k = 7.03\ \frac{kN}{m^2},\ q_k = 3.0\ \frac{kN}{m^2},\ \gamma_g = 1.35,\ \gamma_q = 1.5$$

$$V_{Ed} = 105\ m^2 \times \left(\gamma_g \times g_k + \gamma_q \times q_k\right) = 105\ m^2 \times \left(1.35 \times 7.03\ \frac{kN}{m^2} + 1.5 \times 3.0\ \frac{kN}{m^2}\right)$$

$= 1469.003\ kN$

$$\beta = 1.15,\ c_x = 1300\ mm,\ c_y = 400\ mm,\ d_{eff,1} = 350\ mm,\ d_{eff,2} = 320\ mm,\ f_{ck} = 30\ \frac{N}{mm^2}$$

Cl. 6.4.5(3) $u_0 = 2 \times c_x + 2 \times c_y = 2 \times 1300\ mm + 2 \times 400\ mm = 3400\ mm$

Exp. (6.32) $d = \dfrac{d_{eff,1} + d_{eff,2}}{2} = \dfrac{350\ mm + 320\ mm}{2} = 335\ mm$

$$v = 0.6 \times \left(1 - \frac{f_{ck}}{250\ MPa}\right) = 0.6 \times \left(1 - \frac{30\ \frac{N}{mm^2}}{250\ MPa}\right) = 0.528$$

$\alpha_{cc} = 1.0,\ \lambda = 1.0,\ \gamma_c = 1.5$

$$f_{cd} = \frac{\alpha_{cc} \times \lambda \times f_{ck}}{\gamma_c} = \frac{1.0 \times 1.0 \times 30\ \frac{N}{mm^2}}{1.5} = 20\ \frac{N}{mm^2}$$

Cl. 6.4.5(3) $v_{Rd,max} = 0.5 \times v \times f_{cd} = 0.5 \times 0.528 \times 20\ \frac{N}{mm^2} = 5.28\ \frac{N}{mm^2}$

Cl. 6.4.3(2) $v_{Ed} = \min\left(\dfrac{\beta \times V_{Ed}}{u_0 \times d}, v_{Rd,max}\right) = \min\left(\dfrac{1.15 \times 1469.003\ kN}{3400\ mm \times 335\ mm}, 5.28\ \dfrac{N}{mm^2}\right) = 1.483\ MPa$ OK

Cl. 6.4.2 Check shear stress at control perimeter u_1 (2d from face of column)

$$u_1 = 2 \times \left(c_x + c_y\right) + 2 \times \pi \times 2 \times d = 2 \times (1300\ mm + 400\ mm) + 2 \times 3.142 \times 2 \times 335\ mm$$

$= 7609.734\ mm$

$$k = \min\left(1 + \left(\frac{200\ mm}{d}\right)^{0.5}, 2\right) = \min\left(1 + \left(\frac{200\ mm}{335\ mm}\right)^{0.5}, 2\right) = 1.773$$

Cl. 6.4.4.1(1) $\rho_1 = (0.00447 \times 0.00447)^{0.5} = 0.00447$

Exp. (6.47) & NA
$$v_{Rd,c} = \frac{0.18 \times k \times \left(100 \times \rho_1 \times \left(\frac{f_{ck}}{1\ MPa}\right)\right)^{\left(\frac{1}{3}\right)} \times 1\ MPa}{\gamma_c} = 0.505\ MPa$$

$$\frac{\beta \times V_{Ed}}{u_1 \times d} = \frac{1.15 \times 1469.003\ kN}{7609.734\ mm \times 335\ mm} = 0.663\ MPa$$

$$v_{Ed,u1} = \max\left(\frac{\beta \times V_{Ed}}{u_1 \times d}, v_{Rd,c}\right) = \max\left(\frac{1.15 \times 1469.003\ kN}{7609.734\ mm \times 335\ mm}, 0.505\ MPa\right) = 0.663\ MPa$$

The maximum allowable shear stress in the hollow areas of a BD slab is: $v_{BDRd,c} = 0.6 \times v_{Rd,c}$ Thus, the control perimeter that defines the extent of the solid zone around the column can be derived from the following expression: $u_{solid} = \dfrac{V_{max}}{v_{BDRd,c\ d}}$

Perimeter at which punching shear links are no longer required

Exp. (6.54) $u_{out} = \dfrac{V_{Ed} \times \beta}{d \times 0.6 \times v_{Rd,c}} = \dfrac{1469.003\ \text{kN} \times 1.15}{335\ \text{mm} \times 0.6 \times 0.505\ \text{MPa}} = 16630.535\ \text{mm}$

Length of column faces value1 = u_0 = 3400 mm

Radius to u_{out} value2 = $\dfrac{u_{out} - \text{value1}}{2 \times \pi} = \dfrac{16630.535\ \text{mm} - 3400\ \text{mm}}{2 \times 3.142}$ = 2105.705 mm from face of column

Perimeters of shear reinforcement may stop at value2 - 1.5 × d = 2105.705 mm - 1.5 × 335 mm = 1603.205 mm from face of column.

Shear reinforcement assuming rectangular arrangement of links

Cl. 9.4.3(1) $s_{r,max} = d \times 0.75 = 335\ \text{mm} \times 0.75 = 251.25\ \text{mm}$, say 175 mm

Inside 2d control perimeter, $s_{t,max} = d \times 1.5 = 335\ \text{mm} \times 1.5 = 502.5\ \text{mm}$

Outside control perimeter, $s_{t,max2} = d \times 2 = 335\ \text{mm} \times 2 = 670\ \text{mm}$

Assuming vertical reinforcement:

At the basic control perimeter, u_i, 2d from the column:

Cl. 6.4.5(1) $f_{ywd,ef} = 250\ \text{MPa} + 0.25 \times d \times 1\ \dfrac{\text{N}}{\text{mm}^3} = 250\ \text{MPa} + 0.25 \times 335\ \text{mm} \times 1\ \dfrac{\text{N}}{\text{mm}^3} = 333.75\ \text{MPa}$

Exp. (6.52) $A_{sw} = \dfrac{\left(v_{Ed} - 0.75 \times v_{Rd,c}\right) \times s_{r,max} \times u_1}{1.5 \times f_{ywd,ef}}$

$= \dfrac{(1.483\ \text{MPa} - 0.75 \times 0.505\ \text{MPa}) \times 251.25\ \text{mm} \times 7609.734\ \text{mm}}{1.5 \times 333.75\ \text{MPa}} = 4216.896\ \text{mm}^2$ per perimeter

$\alpha = 90\ °$, $f_{yk} = 500\ \text{MPa}$

$$A_{sw,min} = \frac{1\ \text{MPa} \times 0.08 \times \left(\dfrac{f_{ck}}{1\ \text{MPa}}\right)^{0.5} \times \left(s_{r,max} \times s_{t,max}\right)}{1.5 \times f_{yk} \times \left(\sin(\alpha) + \cos(\alpha)\right)}$$

$$= \frac{1\ \text{MPa} \times 0.08 \times \left(\dfrac{30\ \dfrac{\text{N}}{\text{mm}^2}}{1\ \text{MPa}}\right)^{0.5} \times (251.25\ \text{mm} \times 502.5\ \text{mm})}{1.5 \times 500\ \text{MPa} \times \left(\sin(90\ °) + \cos(90\ °)\right)} = 73.762\ \text{mm}^2$$

Try H10 legs of links (78.5 mm^2)

Cl. 9.4.3 value3 = $\dfrac{A_{sw}}{u_1} = \dfrac{4216.896\ \text{mm}^2}{7609.734\ \text{mm}} = 0.554\ \dfrac{\text{mm}^2}{\text{mm}}$

Using H10 max, spacing

$$\min\left(\frac{78.5\ \text{mm}^2}{\text{value3}}, 1.5 \times d\right) = \min\left(\frac{78.5\ \text{mm}^2}{0.554\ \dfrac{\text{mm}^2}{\text{mm}}}, 1.5 \times 335\ \text{mm}\right) = 141.660\ \text{mm}$$

Use min H10 legs of links at 150 mm cc around perimeter u_1

Perimeters at 0.75 × d = 0.75 × 335 mm = 251.25 mm say 250 mm centres

Check area of reinforcement > A_{sw} = 4216.896 mm^2 perimeters inside u_1

Cl. 9.4.3(4) 1st perimeter to be > 0.3d but < 0.5d from face of column. Say 0.4 × d = 0.4 × 335 mm = 134 mm from face of column. say 130 mm. To ensure reinforcement is > A_{sw}, links are placed at all possible locations across all perimeters. This is illustrated in the drawing sheet 3.

A.2.4 Column Design, Zone B (EN1992-1-1:2004)

$f_{ck} = 35 \dfrac{N}{mm^2}$, $b = 1300$ mm, $h = 400$ mm, $E_{cm,b} = 32.8 \dfrac{kN}{mm^2}$, $E_{cm} = 34.1 \dfrac{kN}{mm^2}$, $l_y = 3500$ mm, $l_z = 3500$ mm

$\gamma_c = 1.5$, $\phi_v = 8$ mm, $\phi = 16$ mm, $c_{nom} = 40$ mm, $N_y = 6$, $N_z = 3$, $N = 14$, $E_s = 200 \dfrac{N}{mm^2}$, $f_{yd} = \dfrac{f_{yk}}{\lambda_s} = 434.783 \dfrac{N}{mm^2}$,

$\lambda_s = 1.15$, $f_{yk} = 500 \dfrac{N}{mm^2}$, $A_s = \dfrac{N \times \pi \times \phi^2}{4} = 2814.867\ mm^2$

where $N_{Ed} = 6788$ kN, $A_c = b \times h = 520000\ mm^2$, $f_{cd} = \dfrac{\alpha_{cc} \times f_{ck}}{\gamma_c} = 19.833 \dfrac{N}{mm^2}$, $\alpha_{cc} = 0.85$

$M_{topy} = 5.5$ kN m, $M_{btmy} = 4.9$ kN m, $M_{topz} = 25.1$ kN m, $M_{btmz} = 22.5$ kN m

Joint details (PD 6687 Cl. 2.10)

$h_{A1y} = 390$ mm, $b_{A1y} = 10000$ mm, $l_{A1y} = 8550$ mm | $h_{B1y} = 390$ mm, $b_{B1y} = 10000$ mm, $l_{B1y} = 8200$ mm
$h_{A1z} = 390$ mm, $b_{A1z} = 8375$ mm, $l_{A1z} = 10000$ mm | $h_{B1z} = 500$ mm, $b_{B1z} = 8375$ mm, $l_{B1z} = 10000$ mm
$h_{A2y} = 390$ mm, $b_{A2y} = 10000$ mm, $l_{A2y} = 8550$ mm | $h_{B2y} = 390$ mm, $b_{B2y} = 10000$ mm, $l_{B2y} = 8200$ mm
$h_{A2z} = 390$ mm, $b_{A2z} = 8375$ mm, $l_{A2z} = 10000$ mm | $h_{B2z} = 390$ mm, $b_{B2z} = 8375$ mm, $l_{B2z} = 10000$ mm

$I_y = 0.00693\ m^4$, $I_{A1y} = \dfrac{b_{A1y} \times h_{A1y}{}^3}{12} = 0.0494\ m^4$, $I_{B1y} = \dfrac{b_{B1y} \times h_{B1y}{}^3}{12} = 0.0494\ m^4$, $k_{1y} = 0.1$

$I_{A2y} = \dfrac{b_{A2y} \times h_{A2y}{}^3}{12} = 0.0494\ m^4$, $I_{B2y} = \dfrac{b_{B2y} \times h_{B2y}{}^3}{12} = 0.0494\ m^4$, $k_{2y} = 0.1$

$I_z = \dfrac{h \times b^3}{12} = 0.0732\ m^4$, $I_{A1z} = \dfrac{b_{A1z} \times h_{A1z}{}^3}{12} = 0.0414\ m^4$, $I_{B1z} = \dfrac{b_{B1z} \times h_{B1z}{}^3}{12} = 0.0872\ m^4$, $k_{1z} = 0.846$

$I_{A2z} = \dfrac{b_{A2z} \times h_{A2z}{}^3}{12} = 0.0414\ m^4$, $I_{B2z} = \dfrac{b_{B2z} \times h_{B2z}{}^3}{12} = 0.0414\ m^4$, $k_{2z} = 1.314$

Effective length

Cl. 5.8.3.2(3), $l_{0y} = 0.5 \times l_y \times \left[\left(1 + \dfrac{k_{1y}}{0.45 + k_{1y}}\right) \times \left(1 + \dfrac{k_{2y}}{0.45 + k_{2y}}\right)\right]^{0.5} = \left[\ 2068.182\ \text{mm}\ \right]$

Cl. 5.8.3.2(3), $l_{0z} = 0.5 \times l_z \times \left[\left(1 + \dfrac{k_{1z}}{0.45 + k_{1z}}\right) \times \left(1 + \dfrac{k_{2z}}{0.45 + k_{2z}}\right)\right]^{0.5} = \left[\ 2971.706\ \text{mm}\ \right]$

Column slenderness about y axis

Cl. 5.8.3.2(1) $i_y = \dfrac{h}{\sqrt{12}} = 115.470$ mm, $\lambda_y = \dfrac{l_{0y}}{i_y} = \left[\ 17.911\ \right]$, $e_{iy} = \dfrac{l_{0y}}{400} = \left[\ 5.170\ \text{mm}\ \right]$

$i_z = \dfrac{b}{\sqrt{12}} = 375.278$ mm, $\lambda_z = \dfrac{l_{0z}}{i_z} = \left[\ 7.919\ \right]$, $e_{iz} = \dfrac{l_{0z}}{400} = \left[\ 7.429\ \text{mm}\ \right]$

$M_{01z} = \text{Min}\left(\left|M_{topz}\right|, \left|M_{btmz}\right|\right) + e_{iz} \times N_{Ed} = \left[\ 72.930\ \text{kN m}\ \right]$

$M_{02z} = \text{Max}\left(\left|M_{topz}\right|, \left|M_{btmz}\right|\right) + e_{iz} \times N_{Ed} = \left[\ 75.530\ \text{kN m}\ \right]$

Limiting slenderness ratio λ_{lim}

Cl. 5.8.3.1(1) A = 0.7, as per default. B = 1.1 as per default. $C = 1.7 - r_m = 0.7$, where $r_m = 1$

Cl. 5.2(7), 5.2.9, 5.8.8.2(1) $\lambda_{lim} = \dfrac{20 \times A \times B \times C}{(n)^{0.5}} = 13.288$, $n = \dfrac{N_{Ed}}{A_c \times f_{cd}} = 0.658$

Min end moment about y axis; $M_{01y} = \text{Min}\left(\left|M_{topy}\right|, \left|M_{btmy}\right|\right) + e_{iy} \times N_{Ed} = \left[\ 39.997\ \text{kN m}\ \right]$

Max end moment about y axis; $M_{02y} = \text{Max}\left(\left|M_{topy}\right|, \left|M_{btmy}\right|\right) + e_{iy} \times N_{Ed} = \left[\ 40.597\ \text{kN m}\ \right]$

Using simplified method by

$\frac{l_{0y}}{h} = \left[\ 5.170\ \right]$, $e_{add} = 0.04 \times h = 16$ mm, $M_{add} = N_{Ed} \times e_{add} = 108.608$ kN m

Cl. 5.8.8.2(1), 5.8.8.2(3)

$M_{0ey} = \mathrm{Max}\left(0.6 \times M_{02y} + 0.4 \times M_{01y}, 0.4 \times M_{02y}\right) = 40.357$ kN m

$M_{Edy} = \mathrm{Max}\left(M_{02y}, M_{0ey} + M_{add}, M_{01y} + 0.5 \times M_{add}, N_{Ed} \times \mathrm{Max}\left(\frac{h}{30}, 20 \text{ mm}\right)\right) = 148.965$ kN m

$M_{Edz} = \mathrm{Max}\left(M_{02z}, N_{Ed} \times \mathrm{Max}\left(\frac{b}{30}, 20 \text{ mm}\right)\right) = 294146.667$ kN mm

Rectangular stress block factors

Cl. 3.1.7(3); $\lambda_{sb} = 0.8$, $\eta = 1.0$, $\varepsilon_{cu3} = 0.00350$, $\varepsilon_{c3} = 0.00175$, $A_{bar} = 201.062$ mm^2

Effective depths of bars for bending about y axis

Spacing of bars in faces parallel to z axis (c/c); $s_z = \frac{h - 2 \times \left(c_{nom} + \phi_v\right) - \phi}{N_z - 1} = 144$ mm

$d_{y1} = h - c_{nom} - \phi_v - \frac{\phi}{2} = 344$ mm, $d_{y2} = d_{y1} - s_z = 200$ mm, $d_{y3} = d_{y2} - s_z = 56$ mm

I of reinft about y axis; $I_{sy} = 2 \times A_{bar} \times \left(N_y \times \left(d_{y1} - \frac{h}{2}\right)^2\right) = 50030642.123$ mm^4, $i_{sy} = \sqrt{\frac{I_{sy}}{A_s}} = 133.318$ mm

Cl. 5.8.8.3(2) $d_y = \frac{h}{2} + i_{sy} = 333.318$ mm

Effective depths of bars for bending about z axis

Spacing of bars in faces parallel to y axis (c/c); $s_y = \frac{b - 2 \times \left(c_{nom} + \phi_v\right) - \phi}{N_y - 1} = 237.6$ mm

$d_{z1} = b - c_{nom} - \phi_v - \frac{\phi}{2} = 1244$ mm, $d_{z2} = d_{z1} - s_y = 1006.4$ mm, $d_{z3} = d_{z2} - s_y = 768.8$ mm, $d_{z4} = d_{z3} - s_y = 531.2$ mm, $d_{z5} = d_{z4} - s_y = 293.6$ mm, $d_{z6} = d_{z5} - s_y = 56.000$ mm

Moment of resistance

Taking the position of neutral axis as $y = 305.0$ mm

Cl 3.1.7(3) $F_{yc} = b \times f_{cd} \times \mathrm{Min}\left(\lambda_{sb} \times y, h\right) = 6291.133$ kN, $M_{Rdyc} = F_{yc} \times \left[\frac{h}{2} - \frac{\mathrm{Min}\left(\lambda_{sb} \times y, h\right)}{2}\right]$

$= \left[\ 490.708 \text{ kN m}\ \right]$

Table A.2.3.1. Moment of resistance of reinforcement

	Layer 1	**Layer 2**	**Layer 3**
Strain	$\varepsilon_{y1} = \varepsilon_{cu3} \times \left(1 - \frac{d_{y1}}{y}\right)$ $= -0.000448$	$\varepsilon_{y2} = \varepsilon_{cu3} \times \left(1 - \frac{d_{y2}}{y}\right)$ $= 0.00120$	$\varepsilon_{y3} = \varepsilon_{cu3} \times \left(1 - \frac{d_{y3}}{y}\right)$ $= 0.00286$
Stress	$\sigma_{y1} = \mathrm{Max}\left(-1 \times f_{yd}, E_s \times \varepsilon_{y1}\right)$ $= -0.0895 \frac{N}{mm^2}$	$\sigma_{y2} = \mathrm{Min}\left(f_{yd}, E_s \times \varepsilon_{y2}\right) - \eta \times f_{cd}$ $= -19.592 \frac{N}{mm^2}$	$\sigma_{y3} = \mathrm{Min}\left(f_{yd}, E_s \times \varepsilon_{y3}\right) - \eta \times f_{cd}$ $= -19.262 \frac{N}{mm^2}$
Force	$F_{y1} = N_y \times A_{bar} \times \sigma_{y1}$ $= -0.108$ kN	$F_{y2} = 2 \times A_{bar} \times \sigma_{y2}$ $= -7.879$ kN	$F_{y3} = N_y \times A_{bar} \times \sigma_{y3}$ $= -23.237$ kN
Moment of resistance of layer	$M_{Rdy1} = F_{y1} \times \left(\frac{h}{2} - d_{y1}\right)$ $= 0.0155$ kN m	$M_{Rdy2} = F_{y2} \times \left(\frac{h}{2} - d_{y2}\right)$ $= -0$ kN m	$M_{Rdy3} = F_{y3} \times \left(\frac{h}{2} - d_{y3}\right)$ $= -3.346$ kN m
Resultant concrete / steel force			$F_y = F_{yc} + F_{y1} + F_{y2} + F_{y3}$ $= 6259.910$ kN

			$M_{Rdy} = M_{Rdyc} + M_{Rdy1} + M_{Rdy2} + M_{Rdy3}$ $= \left[\ 487.378\ \text{kN m}\ \right]$

Taking position of neutral axis as z = 991.3 mm

Cl. 3.1.7(3) $F_{zc} = \eta \times f_{cd} \times \text{Min}\left(\lambda_{sb} \times z, b\right) \times h = 6291.451\ \text{kN}$

Moment of resistance; $M_{Rdzc} = F_{zc} \times \left[\frac{b}{2} - \frac{\text{Min}\left(\lambda_{sb} \times z, b\right)}{2} \right] = \left[\ 1594.757\ \text{kN m}\ \right]$

Table A.2.3.2. Moment of resistance of reinforcement

Layer	Strain	Stress	Force	Moment
1	$\varepsilon_{z1} = \varepsilon_{cu3} \times \left(1 - \frac{d_{z1}}{z}\right)$ $= -0.000892$	$\sigma_{z1} = \text{Max}\left(-1 \times f_{yd}, E_s \times \varepsilon_{z1}\right)$ $= -0.178\ \frac{\text{N}}{\text{mm}^2}$	$F_{z1} = N_z \times A_{bar} \times \sigma_{z1}$ $= -0.108\ \text{kN}$	$M_{Rdz1} = F_{z1} \times \left(\frac{b}{2} - d_{z1}\right)$ $= 0.0639\ \text{kN m}$
2	$\varepsilon_{z2} = \varepsilon_{cu3} \times \left(1 - \frac{d_{z2}}{z}\right)$ $= -0.0000533$	$\sigma_{z2} = \text{Max}\left(-1 \times f_{yd}, E_s \times \varepsilon_{z2}\right)$ $= -0.0107\ \frac{\text{N}}{\text{mm}^2}$	$F_{z2} = 2 \times A_{bar} \times \sigma_{z2}$ $= -0.00429\ \text{kN}$	$M_{Rdz2} = F_{z2} \times \left(\frac{b}{2} - d_{z2}\right)$ $= 0.00153\ \text{kN m}$
3	$\varepsilon_{z3} = \varepsilon_{cu3} \times \left(1 - \frac{d_{z3}}{z}\right)$ $= 0.000786$	$\sigma_{z3} = \text{Min}\left(f_{yd}, E_s \times \varepsilon_{z3}\right) - \eta \times f_{cd}$ $= -19.676\ \frac{\text{N}}{\text{mm}^2}$	$F_{z3} = 2 \times A_{bar} \times \sigma_{z3}$ $= -7.912\ \text{kN}$	$M_{Rdz3} = F_{z3} \times \left(\frac{b}{2} - d_{z3}\right)$ $= 0.940\ \text{kN m}$
4	$\varepsilon_{z4} = \varepsilon_{cu3} \times \left(1 - \frac{d_{z4}}{z}\right)$ $= 0.00162$	$\sigma_{z4} = \text{Min}\left(f_{yd}, E_s \times \varepsilon_{z4}\right) - \eta \times f_{cd}$ $= -19.508\ \frac{\text{N}}{\text{mm}^2}$	$F_{z4} = 2 \times A_{bar} \times \sigma_{z4}$ $= -7.845\ \text{kN}$	$M_{Rdz4} = F_{z4} \times \left(\frac{b}{2} - d_{z4}\right)$ $= -0.932\ \text{kN m}$
5	$\varepsilon_{z5} = \varepsilon_{cu3} \times \left(1 - \frac{d_{z5}}{z}\right)$ $= 0.00246$	$\sigma_{z5} = \text{Min}\left(f_{yd}, E_s \times \varepsilon_{z5}\right) - \eta \times f_{cd}$ $= -19.341\ \frac{\text{N}}{\text{mm}^2}$	$F_{z5} = 2 \times A_{bar} \times \sigma_{z5}$ $= -7.777\ \text{kN}$	$M_{Rdz5} = F_{z5} \times \left(\frac{b}{2} - d_{z5}\right)$ $= -2.772\ \text{kN m}$
6	$\varepsilon_{z6} = \varepsilon_{cu3} \times \left(1 - \frac{d_{z6}}{z}\right)$ $= 0.00330$	$\sigma_{z6} = \text{Min}\left(f_{yd}, E_s \times \varepsilon_{z6}\right) - \eta \times f_{cd}$ $= -19.173\ \frac{\text{N}}{\text{mm}^2}$	$F_{z6} = N_z \times A_{bar} \times \sigma_{z6}$ $= -11.565\ \text{kN}$	$M_{Rdz6} = F_{z6} \times \left(\frac{b}{2} - d_{z6}\right)$ $= -6.869\ \text{kN m}$

Resultant concrete/steel force; $F_z = F_{zc} + F_{z1} + F_{z2} + F_{z3} + F_{z4} + F_{z5} + F_{z6} = 6256.240\ \text{kN}$ OK

This is within half of one percent of the applied axial load

$M_{Rdz} = M_{Rdzc} + M_{Rdz1} + M_{Rdz2} + M_{Rdz3} + M_{Rdz4} + M_{Rdz5} + M_{Rdz6} = \left[\ 1585.189\ \text{kN m}\ \right]$ OK

Biaxial bending

Cl. 5.8.9(3) $\text{ratio}_\lambda = \frac{\text{Max}\left(\lambda_y, \lambda_z\right)}{\text{Min}\left(\lambda_y, \lambda_z\right)} = 2.262,\ e_y = \frac{M_{Edz}}{N_{Ed}} = 43.333\ \text{mm},\ e_z = \frac{M_{Edy}}{N_{Ed}} = 0.0219\ \text{m},\ h_{eq} = i_y \times \sqrt{12}$

$= 400\ \text{mm},\ b_{eq} = i_z \times \sqrt{12} = 1300\ \text{mm},\ e_{rel,y} = \frac{e_y}{b_{eq}} = 0.0333,\ e_{rel,z} = \frac{e_z}{h_{eq}} = 0.0549,\ \text{ratio}_e = \frac{\text{Min}\left(e_{rel,y}, e_{rel,z}\right)}{\text{Max}\left(e_{rel,y}, e_{rel,z}\right)} = 0.608,$

$\text{ratio}_\lambda > 2$ & $\text{ratio}_e > 0.2$ therefore biaxial bending check is required

Cl. 5.8.9(4) $N_{Rd} = \left(A_c \times f_{cd}\right) + \left(A_s \times f_{yd}\right) = 11537188.558\ \text{N}$

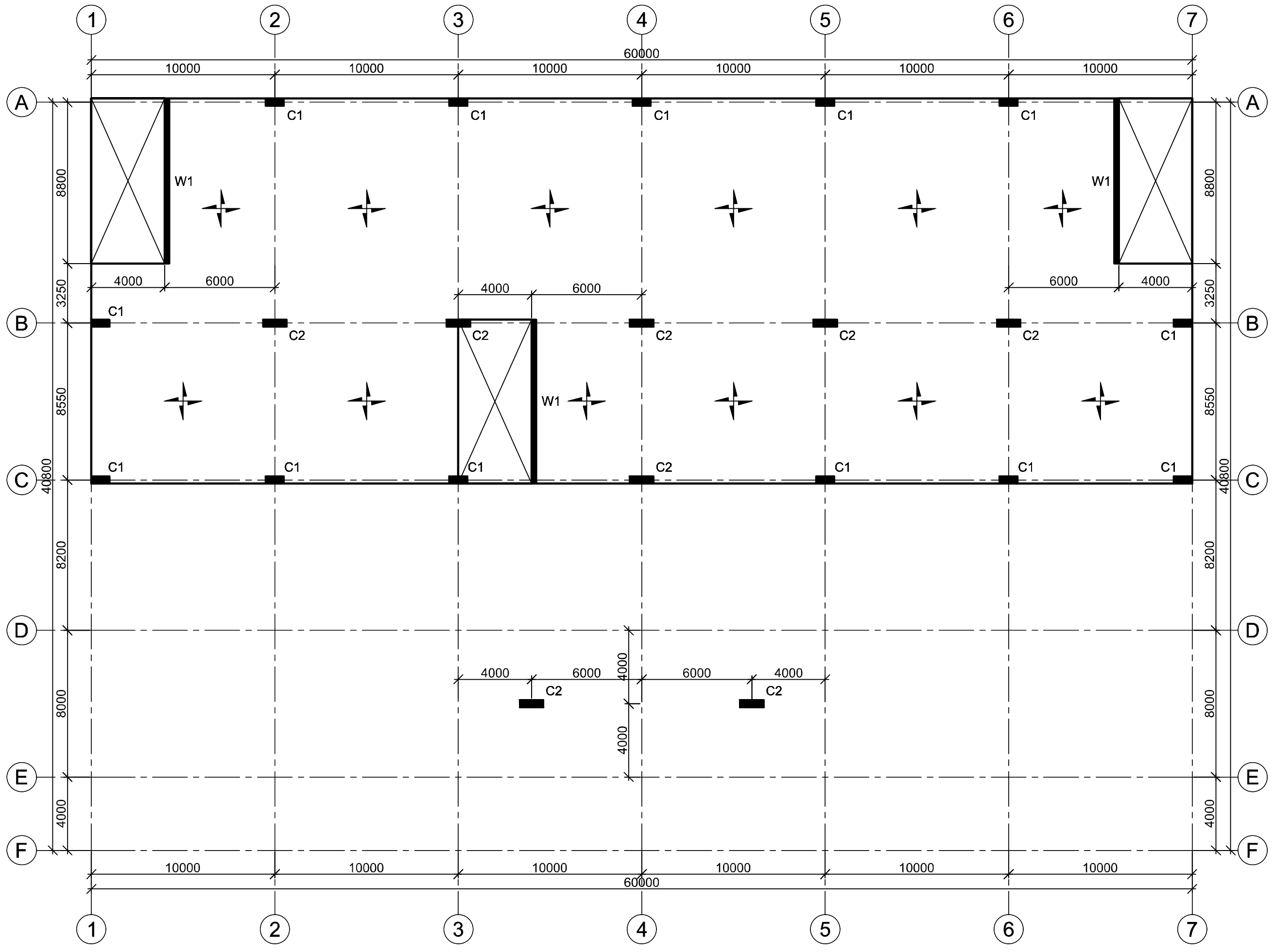
Ratio of applied to resistance axial loads; $\text{ratio}_N = \frac{N_{Ed}}{N_{Rd}} = 0.588$

Exp. (5.39) $a = 1.41,\ UF = \left(\frac{M_{Edy}}{\text{Table1}.M_{Rdy}}\right)^a + \left(\frac{M_{Edz}}{M_{Rdz}}\right)^a = \left[\ 0.281\ \right]$ OK

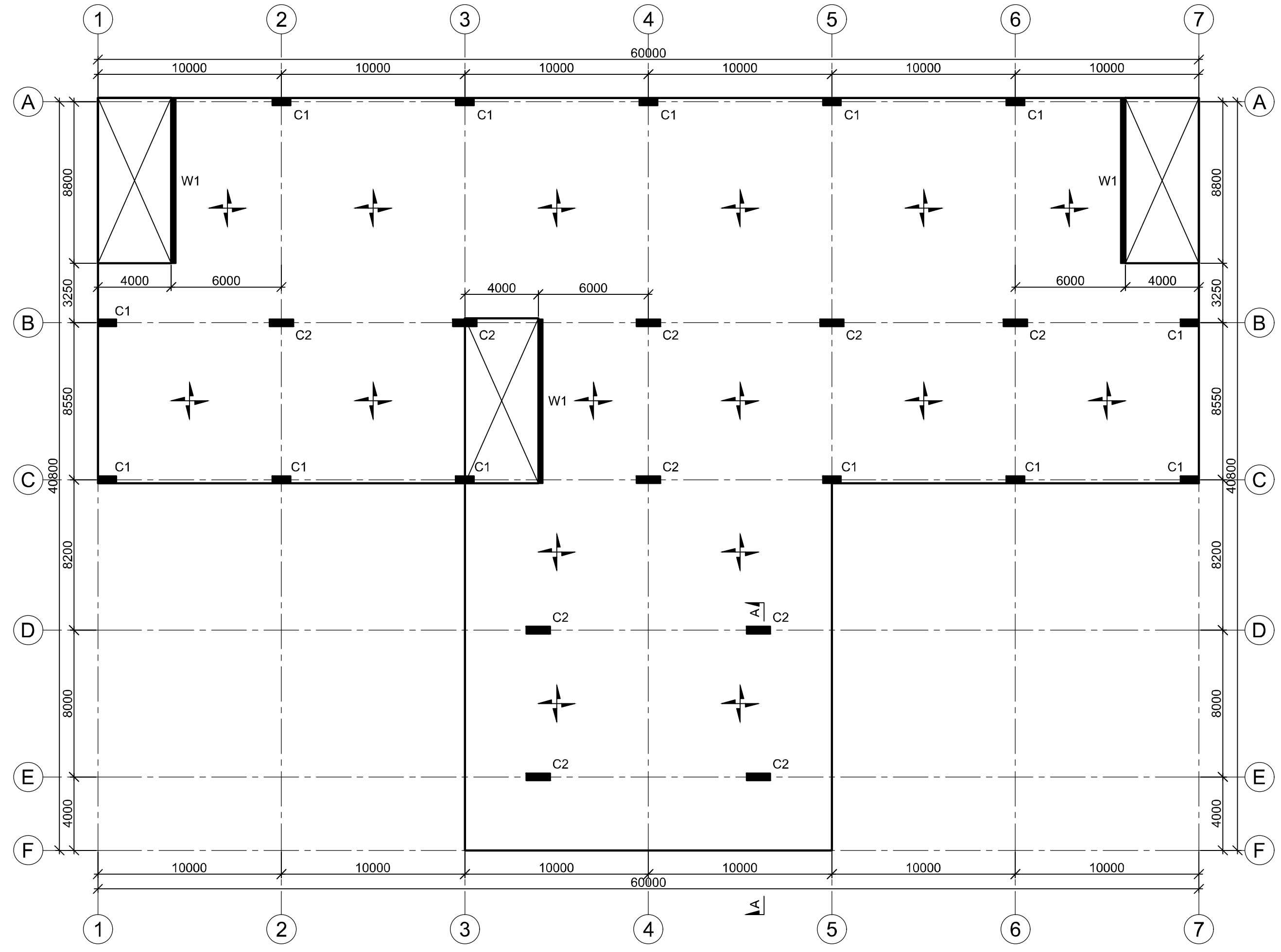
Links

Cl. 9.5.3(3), (4) $\frac{16\ \text{mm}}{4} = 4\ \text{mm}$ say 8 mm. Min spacing @$\text{Min}\left(0.6 \times 20 \times 16\ \text{mm}, 0.6 \times 300\ \text{mm}, 0.6 \times 400\ \text{mm}\right)$

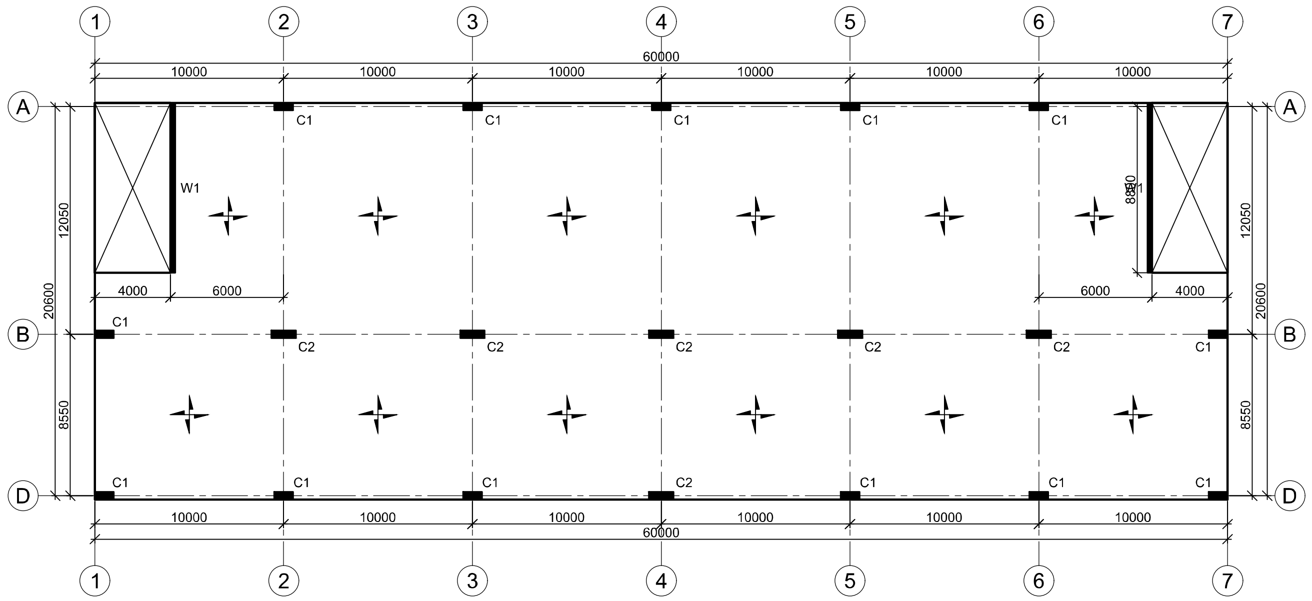
$= 180\ \text{mm}$



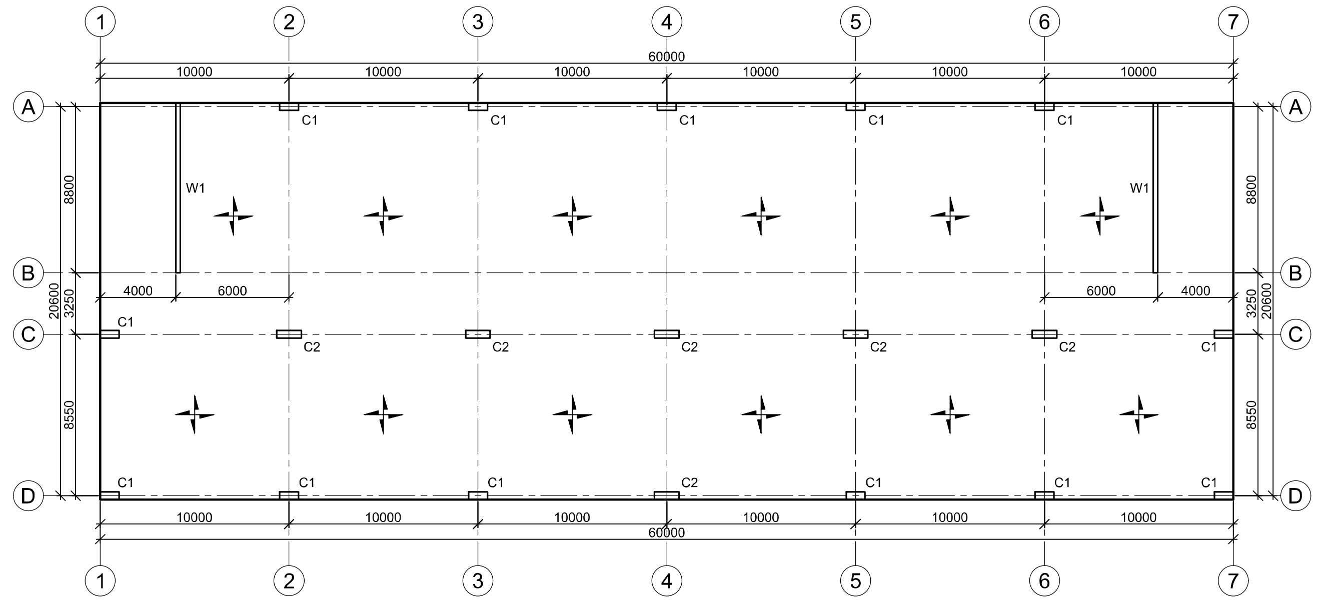
GROUND FLOOR LAYOUT PLAN
SCALE 1:200



FIRST FLOOR & SECOND FLOOR LAYOUT PLAN
SCALE 1:200



TYPICAL FLOOR LAYOUT PLAN
SCALE 1:200



ROOF LAYOUT PLAN
SCALE 1:200

COLUMN SIZES	
NO.	SIZE
C1	1000 X 400
C2	1300 X 400

RC WALL SIZES	
NO.	SIZE
W1	250 X 9000

SLAB TYPES	
FLOOR	TYPE
GROUND	390 MM THICK SEMI-PRECAST BUBBLEDECK
FIRST	390 MM THICK SEMI-PRECAST BUBBLEDECK
SECOND	390 MM THICK SEMI-PRECAST BUBBLEDECK
THIRD	390 MM THICK SEMI-PRECAST BUBBLEDECK
FOURTH	390 MM THICK SEMI-PRECAST BUBBLEDECK
FIFTH	390 MM THICK SEMI-PRECAST BUBBLEDECK
ROOF	390 MM THICK SEMI-PRECAST BUBBLEDECK

NOTES

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5. Cover to main reinforcement to be as follows:
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6. "H" Denotes Reinforcement bars of ductility classes A, B and C to BS 4461 with a yield strength of 500N/mm^2 to BS 4449-2005 to be used.
7. Reinforcement in walls and columns must be inspected by the Engineer before being enclosed in the framework.
8. All mortar used to be of cement sand mix 1:3, with all the stone walling being laid in 200 mm courses with 12 mm mortar joints.
9. PS = 0 expansion joint is located on Grid 5.

CLIENT

NAME:

SIGN:............................ DATE:.............................

PROJECT

ENGM041 CONCRETE BUILDING DESIGN

DRAWING TITLE

FLOOR LAYOUT PLANS

DRAWN BY:

ALLAN NDUBI MAUGO, 6857626

CHECKED BY:

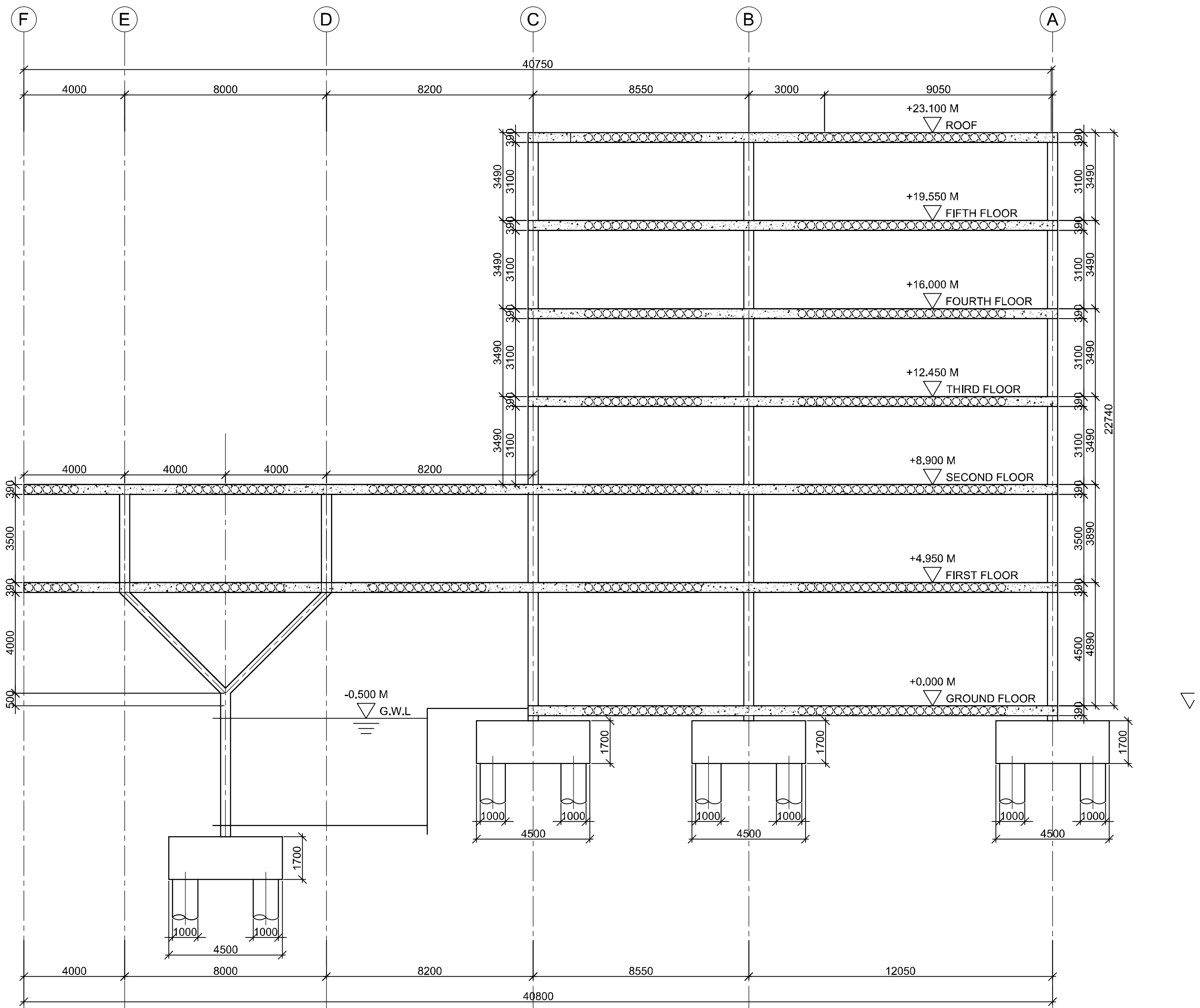
NAME: ....

DATE: .... SIGN: ....

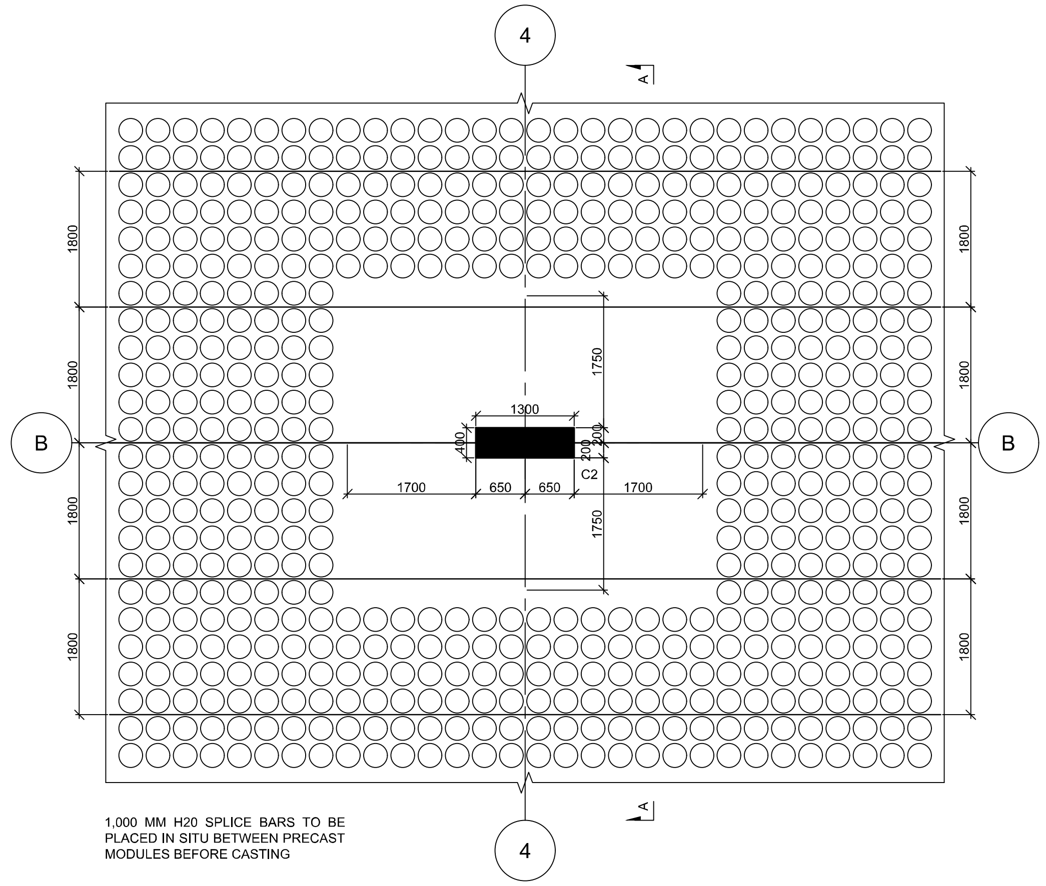
GENERAL DRAWINGS

UNIVERSITY OF SURREY

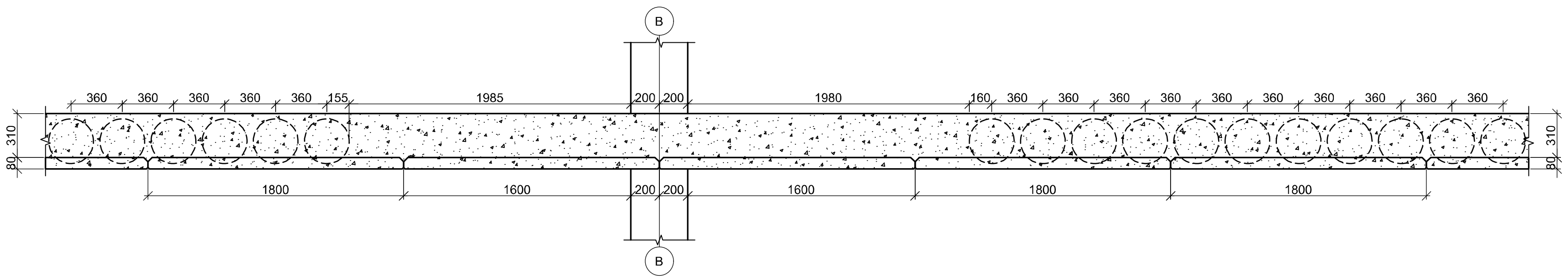
ISSUE DATE	MAY, 2025
DRAWING NO:	001 - SHT 001



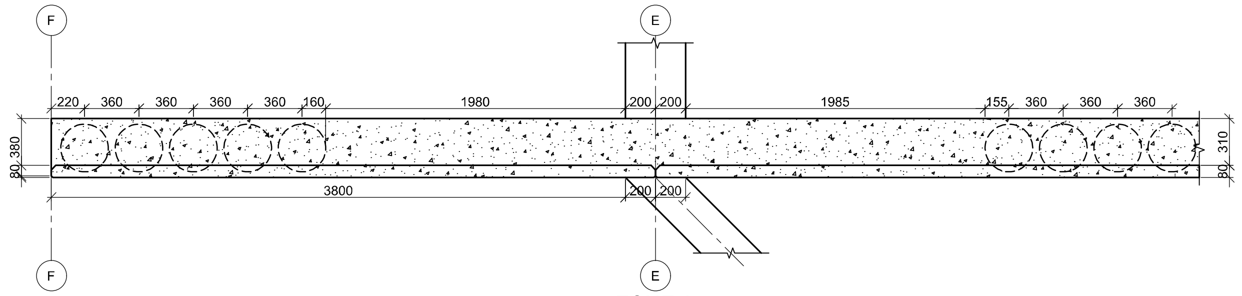
GRID 4 ELEVATION
SCALE 1:125



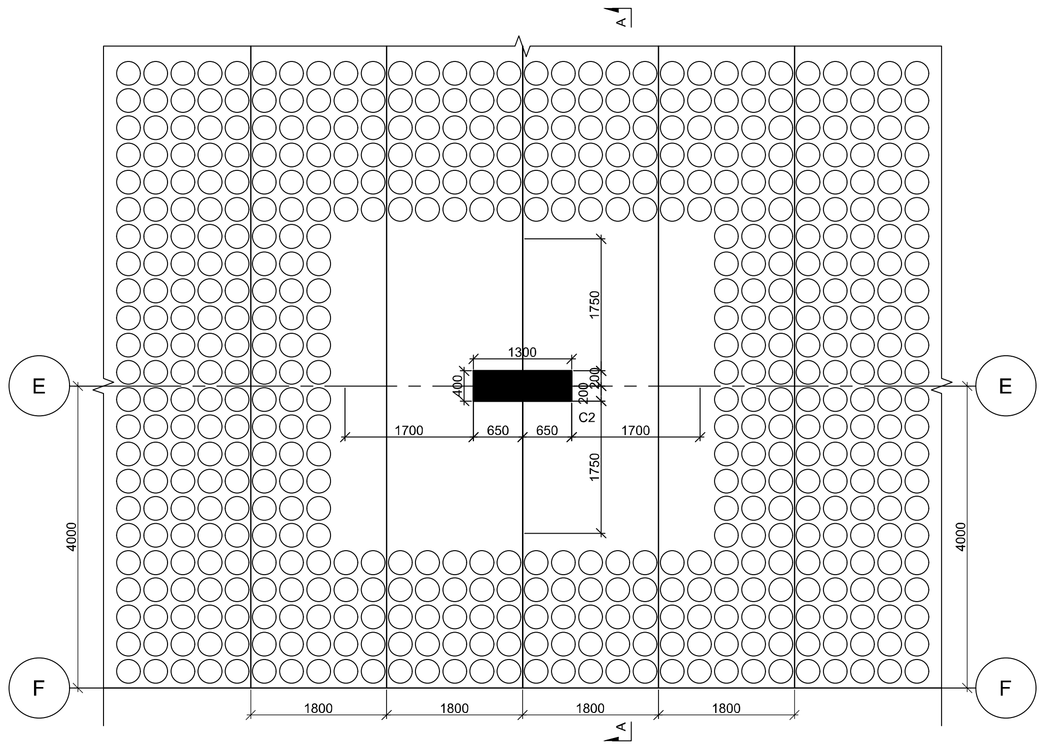
ZONE C
SCALE 1:50



ZONE C
SECTION A-A
SCALE 1:25



ZONE A
SECTION A-A
SCALE 1:25



ZONE A
SCALE 1:50

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CLIENT

NAME:

SIGN:............................ DATE:.............................

PROJECT

ENGM041 CONCRETE BUILDING DESIGN

DRAWING TITLE

GENERAL DRAWINGS

DRAWN BY:

ALLAN NDUBI MAUGO, 6857626

CHECKED BY:

NAME: ....

DATE: .... SIGN: ....

GENERAL DRAWINGS

UNIVERSITY OF SURREY

ISSUE DATE	MAY, 2025
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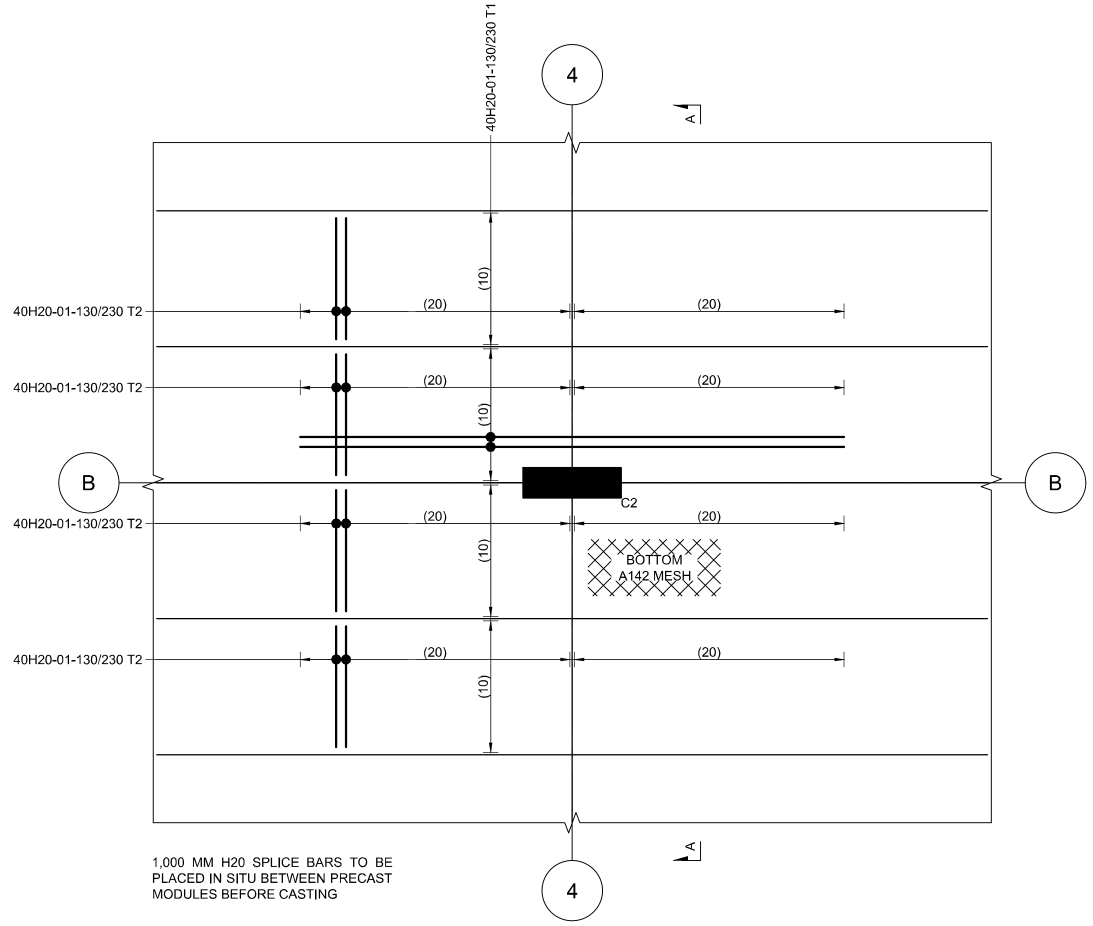
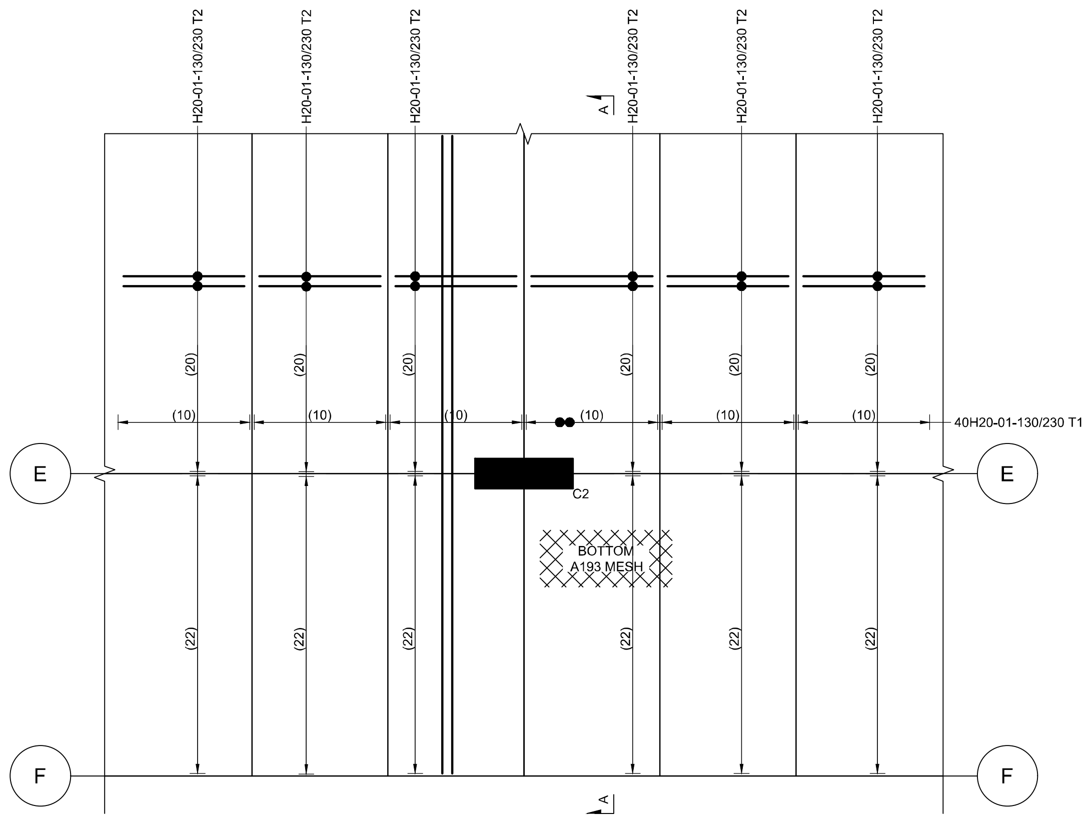
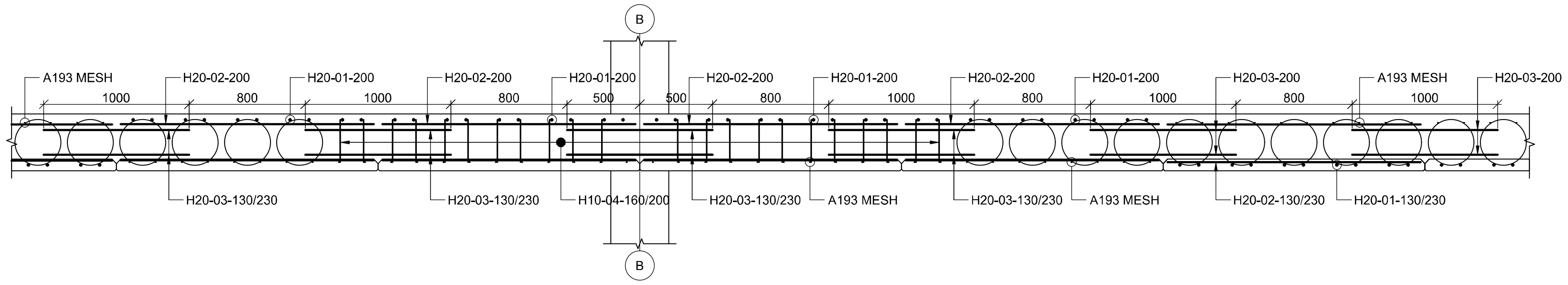


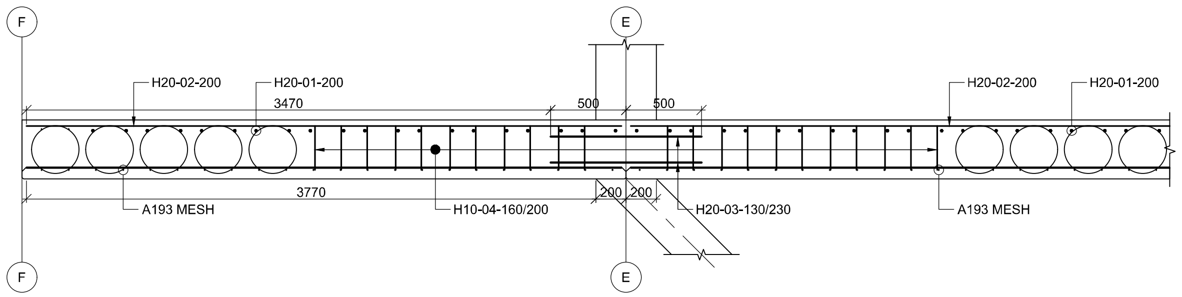
ZONE C
SCALE 1:50



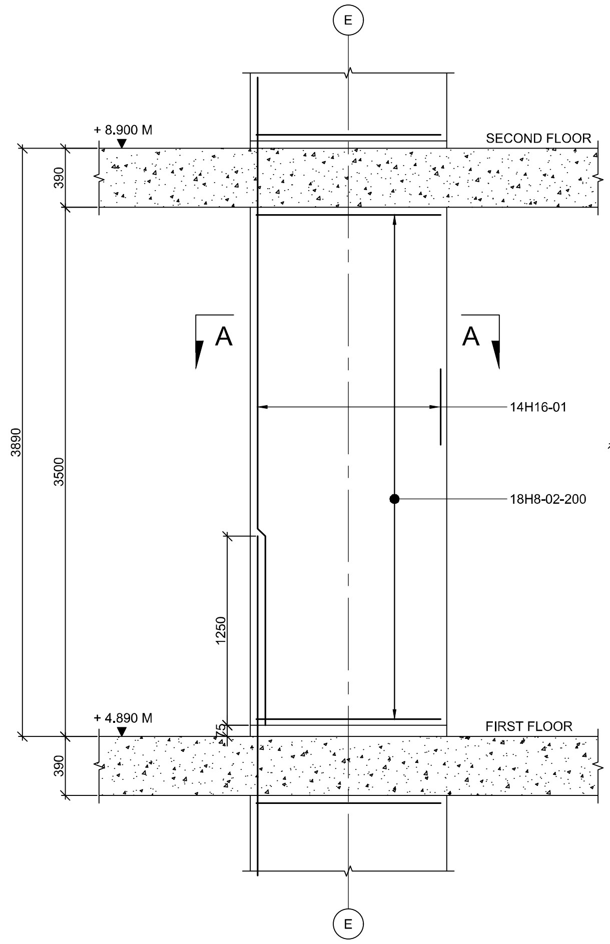
ZONE A
SCALE 1:50



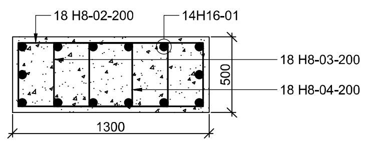
ZONE C
SECTION A-A
SCALE 1:25



ZONE A
SECTION A-A
SCALE 1:25



ZONE B
500 X 1300 MM COLUMN C2 5NO. OFF
SCALE 1:25



ZONE B
SECTION A-A
SCALE 1:25

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CLIENT

NAME:

SIGN:............................ DATE:............................

PROJECT

ENGM041 CONCRETE BUILDING DESIGN

DRAWING TITLE

REINFORCED CONCRETE DETAILS

DRAWN BY:

ALLAN NDUBI MAUGO, 6857626

CHECKED BY:

NAME: ....

DATE: .... SIGN: ....

GENERAL DRAWINGS

UNIVERSITY OF SURREY

ISSUE DATE	MAY, 2025
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