Preliminary structural design for Monier Tower

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Content:

1.Design brief and general assumptions.

Design brief

Develop a structural design scheme for a 22-storey residential tower. Building to be located in the waterside redevelopment area of a regional city centre.

Building functions.

Level -1	Basement
Level 0,1	Retail area Requirements: 8m column span and 4m height, 2 lifts, one stair core and 2 escalators
Level 2-17	Residential tower. Requirements: one staircase and two lifts.
Level 18	garden area and residential
Level 19-22	penthouse apartments

Loadings

Given by design brief. These loadings include an allowance for services. All values are characteristic values.

Cladding: precast concrete cladding including glazing	2.0kN/m2
Car parking:	2.5 kN/m2
Retail:	5.0 kN/m2
Residential:	2.5 kN/m2
Circulation:	5.0 kN/m2
Podium deck:	10.0 kN/m2
Garden deck:	10.0 kN/m2
Roof:	2.5 kN/m

Fire safety

The REI120 class required in the design brief may be revised to REI 90 based on Building Code 2010 Approved Document B Table B4.
Based on *BS EN 1992-1-2 Table 5.2a* this would set minimum sizes to:

columns	350mm
distance a	45mm
solid slab	100mm
beams	40mm for 300mm width and 35mm for 400mm width3 (Table 5.5)

Durability

Considering a life span of 50 years, exposure class XS1, structural class S4 (in accordance with *EN1992-1-1 Table 4.3N*) the cover required is:

for durability	35mm + 10 mm	*EN1992-1-1 Table 4.4N*
for bonding (given 20mm rebar)	20mm	
for fire protection	45mm	*EN1992-1-2 Table 5.2a*

2. Wind loads calculation

Basic wind velocity $v_b := 23.5\frac{m}{s}$

Dimension of plan parallel to wind $d := 16m$

Dimension of plan perpendicular to wind $b := 25m$

Building height $h := 70.6m$

Terrain roughness for Terrain category 0
(*EN1991-1-4 eq4.5*):

$z_0 := 0.003m \qquad z_{min} := 1m \qquad z_{0.II} := 0.05m$

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

Roughness factor
(*EN1991-1-4 eq4.4*) $c_r := k_r\cdot ln\left(\frac{h}{z_0}\right) = 1.571$

Orographic factor *(EN 1991-1-1 4.3.3)*
considered as 1 due to flatness of site: $c_0 := 1$

Mean wind velocity: $v_m := c_r\cdot c_0\cdot v_b = 36.911\frac{m}{s}$

Wind turbolence *(EN1991-1-4 eq.4.7)*
k.I considered as 1 as per *EN1991-1-4 sec4.4(1)*:

$$k_I := 1 \qquad I_v := \frac{k_I}{c_0\cdot ln\left(\frac{h}{z_0}\right)} = 0.099$$

Basic velocity pressure *(EN1991-1-4 sec4.5(1):*

$$\rho := 1.25\frac{kg}{m^3} \qquad q_b := \frac{1}{2}\rho\cdot v_b^{\,2} = 0.345\cdot\frac{kN}{m^2}$$

Peak velocity pressure *(EN1991-1-4 eq.4.8):*

$$q_p := \left(1 + 7I_v\right)\cdot\frac{1}{2}\cdot\rho\cdot v_m^{\,2} = 1.444\cdot\frac{kN}{m^2}$$

Pressure coefficient type *(EN1991-1-4 §7.2.1(1))*
External coefficient c.pe defined based on *Table 7.1* for the differen zones A,B,C,D,E.

$c_{peA} := -1.200 \qquad c_{peC} := -0.500$

$c_{peB} := -0.800 \qquad c_{peD} := 0.800 \qquad c_{peE} := -0.671$

Internal pressure coefficients chosen according to
EN1991-1-1-4 §7.2.9 as:

$c_{pi.min} := -0.300 \qquad c_{pi.max} := 0.200$

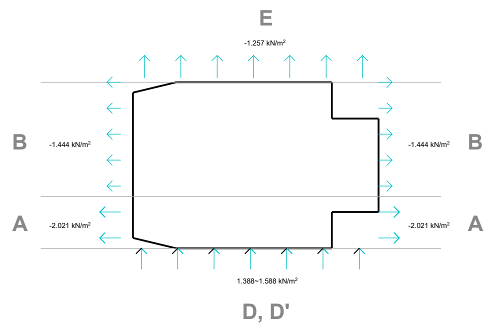
External wind pressure: $w_e := q_p\cdot c_{pe}$

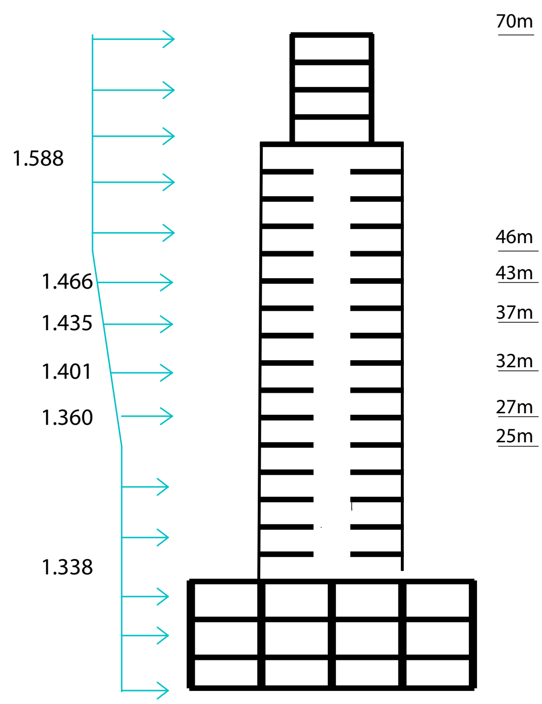
Internal wind pressure: $w_i := q_p\cdot c_{pi}$

Net wind pressure
(EN1991-1-1-4§7.2.10) $w_{net} := w_e - w_i$

Pressure zones determined as
EN1991-1-1-4 Fig 7.4 and 7.5: $e := min(b, 2\cdot h) = 25\,m$

Net wind pressure: Zone A: -2.021 kN/m2
Zone B: -1.444 kN/m2
Zone D: 1.588 kN/m2
Zone D': 1.338 kN/m2
Zone E: -1.257 kN/m2





3. General considerations on the structural scheme

The scheme proposed consist of a braced frame structure to a central core. A smaller core runs from foundation to rooftop (shaded in blue in Figure 3.1 and 3.2) and a larger core running from basement to level 18 (shaded in yellow inFig. 3.2).

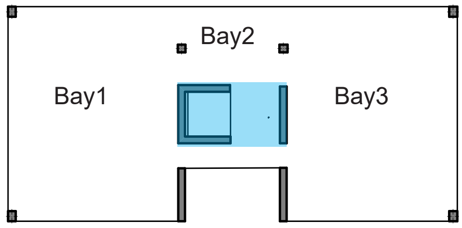


Figure 3.1: diagram of structural scheme for levels 18-22. Shaded area in blue shows the central core

Level 18-22

Considering that the upper levels will be used as a penthouse the structural scheme tried to optimize flexibility of use by creating two uninterrupted spaces (bay 1 and 3). Columns are placed at the edges and braced to the central core via flat slabs (Figure3.1).
Flat slabs seemed the better option in terms of floor to ceiling clearance, construction and integration with MEP systems.

Columns E5 and E6 were moved internally to avoid a transfer beam on level 17. This produces a simpler and stronger structure without too much interference in Bay 2 that is assumed to be used as a transition space.

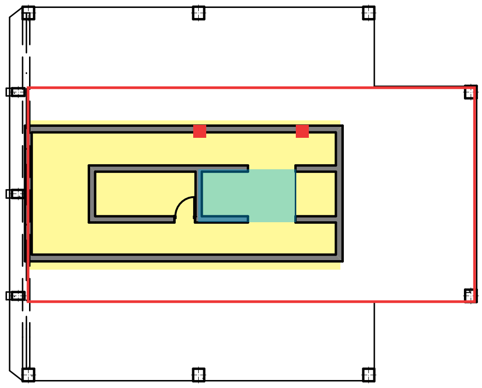


Figure 3.2: diagram of structural scheme for level 17. The red outline represent the layout of the upper floors. Blue shaded area shows the core reaching the roof while yellow shade indicated the core f up to level 18.

Level 3-18

Below the upper penthouse the central core expands and incorporates both vertical and horizontal circulation. Slabs and outer columns are braced to it.

On level 17, at the connection of the penthouse levels and the medium part of the building (Fig 3.3), a transfer beam is proposed for:
- solve the connection between slanted columns and vertical structural elements coming from upper floors.
- avoid punching shear reinforcement in the slab
- transfer the axial load of column E4 and C4

Flat slabs are proposed with the exception of lower levels (3-6) between grid 2 and 4 where a two way slab supported by beam is proposed. This is due to the span reaching values above 9m up to 12m. This was considered uneconomical and creating unnecessary complications in comparison to the benefits of a slimmer slab (15-20cm).

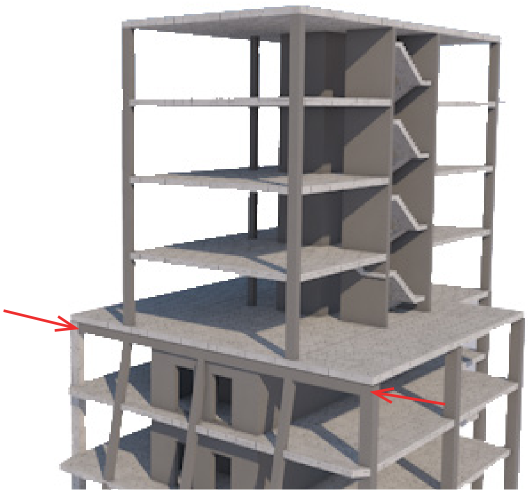


Figure 3.3: position of the transfer beam on level 17

Level -1 to 3

The structural layout for the lowest level is driven by the request of obtaining spans of minimum 8m. Due to spans above 11m between grid 2 and 4, a system of slab supported by beams was preferred to flat slabs that would generate excessive moments on the columns.

An additional note on columns B7 and F7. The current position was preferred to B8 F8 for structural reasons: the current position is aligned with upper floors while locating them on grid 8 would require a transfer beam. Ultimately the space created at B7 and F7 would not be so of much value considering the location relative to the building.

Due to determined report length, in the following sections only calculation of key structural elements are shown. These were considered the elements that are under the highest structural stress.
After this preliminary design a more detailed and optimized design should be carries.

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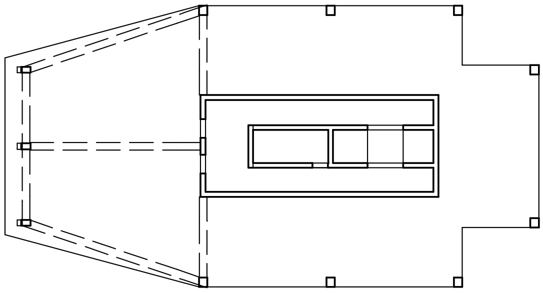


Figure 3.4: diagram of structural scheme for level 3. The structure in front part of the building is made by beam and columns supporting a two way slab. The back of the building hosts flat slabs directly connected to columns.

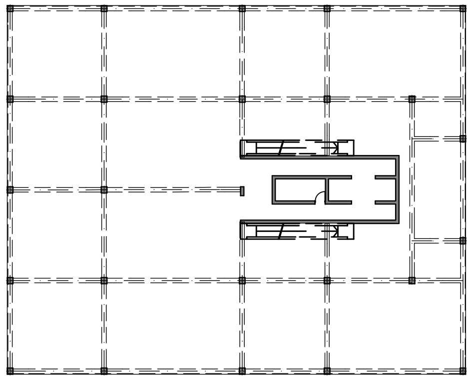


Figure 3.5: diagram of structural scheme for ground and basement levels. One way and two way slabs supported by beams was the chosen structural scheme for this levels due to the longer span in one section

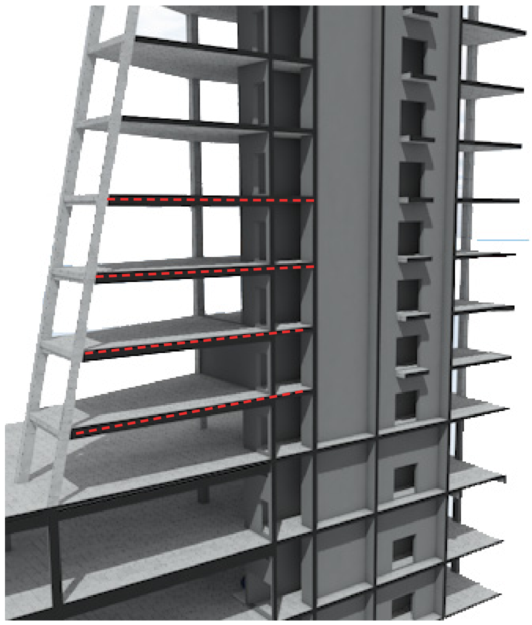


Figure 3.6: the front part of level 3-6 have spans that reach 12m. A flat slab would not be a convenient solution. Two way slabs supported by beams was chosen instead (red dashed lines).

4.1 Slab analysis / levels 19-22

The proposed design divides each level into 3 bays: the central one including the core and circulation and two side ones (7.5m x 9m) without any column interruption to allow for maximum flexibility of use. A flat slab of 300mm is proposed to maximize floor height.
For construction simplicity the thickness of the slab is constant across the whole floor.

ELEMENT Level 22 , Slab, Bay 1 long span

INITIAL PARAMETERS (EC2 Table 3.1)

$h_{slab} := 300mm$ $b_{slab} := 7500mm$ $span_{slab} := 9m$

$f_{yd} := \frac{500}{1.5} MPa$ $f_{ck} := 30\ Mpa$ $f_{cd} := 20$

$f_{yk} := 500 \cdot 0.9\text{!}\ MPa$ $f_{ctm} := 0.3 \cdot f_{ck}^{.66} = 2.832$

$dia_{rebar} := 12mm$ $w_{column} := 400mm$

CONCRETE COVER

Fire protection *(EN1992-1-2 Table 5.8,* REI 120*)*
45mm
Bond (assuming 20 mm bars)
20mm
Durability for XS1 (BS 8500-1:Table A4, IIB-V, IIIA)
35+10=45mm

Cover required for slab is 45mm $cover := 45mm$

EFFECTIVE DIMENSIONS

Effective depth:
$d := h_{slab} - cover - dia_{rebar} \cdot 0.5 = 249 \cdot mm$
Effective spans:
$L_x := 9000mm$ $L_y := 7500mm$

LOADINGS

Permanent load: $g_k := h_{slab} \cdot 25 \frac{kN}{m^3} \cdot L_x \cdot L_y = 506.25 \cdot kN$

Variable load $q_k := 2.5 \frac{kN}{m^2} \cdot L_x \cdot L_y = 168.75 \cdot kN$

Cladding load $clad := 2 \frac{kN}{m^2}$ $g_{clad} := clad \cdot 0.25m \cdot L_x$

Ultimate load according to EN 1990:2002

$$n := 1.35\left(g_k + g_{clad}\right) + 1.5 \cdot q_k = 942.638 \cdot kN$$

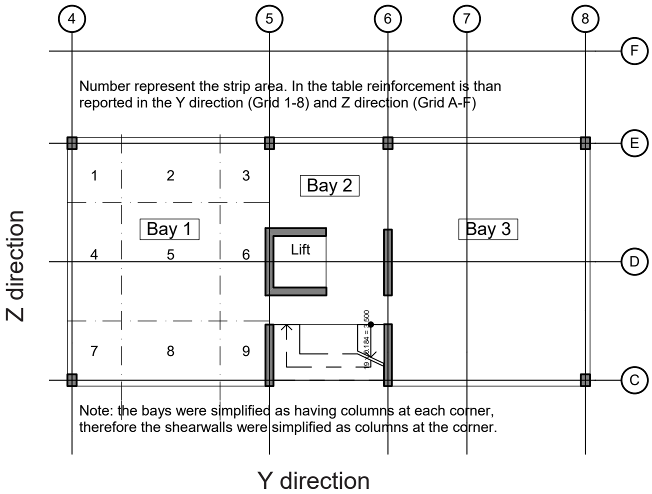


Fig: 2.1 Typical plan for levels 19-22

MOMENT MID SPAN MIDDLE

Bending moment at the middle strip end span:
(*Ref: "How to..." Table 3)*

$M_{m\_span} := 0.075 \cdot n \cdot L_x = 636.28\, m \cdot kN$

$\boxed{M_{m\_span} := 516kN \cdot m}$ from moment distribution

Middle strip take 0.4 of the load

$\boxed{M_{mid} := 0.4 \cdot M_{m\_span} = 206.4 \cdot kN \cdot m}$

$$K := \frac{M_{mid}}{\left(b \cdot d^2 \cdot f_{ck}\right)} = 0.025$$

$$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 243.457 \cdot mm$$

$\boxed{\frac{Z}{d} = 0.978}$ use $Z := 0.95 \cdot d = 236.55 \cdot mm$

$$A_s := \frac{M_{mid}}{0.87 \cdot f_{yk} \cdot Z} = 2.111 \times 10^3 \quad mm^2$$

$$\boxed{A_{s\_mm} := \frac{A_s \cdot mm^2}{L_x \cdot 0.5} = 469.204 \cdot \frac{mm^2}{m}}$$

$A_{s\_eff} := 770 \frac{mm^2}{m}$ **H14 at 200 c/c (770 mm2)**

$$\rho := \frac{A_{s\_eff}}{h_{slab} \cdot 1} = 0.257 \cdot \%$$

MOMENT MID SPAN COLUMN

Column strip takes 0.6

$M_{col} := 0.6 \cdot M_{m\_span} = 309.6 \cdot kN \cdot m$

$K := \dfrac{M_{col}}{b \cdot d^2 \cdot f_{ck}} = 0.037$

$Z := d \cdot \left[0.5 + \left(0.25 - \dfrac{K}{1.133} \right)^{0.5} \right] = 240.587$

$\dfrac{Z}{d} = 0.966$ use $Z := 0.95 \cdot d = 236.55$

$A_{s\_tot} := \dfrac{M_{col}}{0.87 \cdot f_{yk} \cdot Z} = 3.167 \times 1C\, mm^2$

$\boxed{A_{s\_mc} := \dfrac{A_{s\_tot} \cdot mm^2}{L_x \cdot 0.5} = 703.805 \cdot \dfrac{mm^2}{m}}$

H14 at 175c/c (880 mm2)

MOMENT SUPPORT SPAN MIDDLE

$\boxed{M_{m\_support} := 536 kN \cdot m}$ from moment distribution

Middle strip take 0.4 of the load

$\boxed{M_{mid} := 0.4 \cdot M_{m\_support} = 214.4 \cdot kN \cdot m}$

$K := \dfrac{M_{mid}}{\left(b \cdot d^2 \cdot f_{ck} \right)} = 0.026$

$Z := d \cdot \left[0.5 + \left(0.25 - \dfrac{K}{1.133} \right)^{0.5} \right] = 243.237 \cdot mm$

$\dfrac{Z}{d} = 0.977$ use $Z := 0.95 \cdot d = 236.55 \cdot mm$

$A_s := \dfrac{M_{mid}}{0.87 \cdot f_{yk} \cdot Z} = 2.084 \times 10^3 \quad mm^2$

$\boxed{A_{s\_mm} := \dfrac{A_s \cdot mm^2}{L_x \cdot 0.5} = 463.02 \cdot \dfrac{mm^2}{m}}$

H12 at 200 c/c (565 mm2)

$A_{s\_eff} := 616 \dfrac{mm^2}{m}$ $\quad \rho := \dfrac{A_{s\_eff}}{h_{slab} \cdot 1} = 0.205 \cdot \%$

Note that H14@250 was not chosen t control cracking above 0.3mm as shown below:

BAR SPACING

$f_{yk} := 500 MPa \quad \gamma_{ms} := 1.14 \quad \psi_2 := 0.3 \quad \delta := 1$

$A_s := A_{s\_mm} \qquad A_{s\_eff} := 616 \dfrac{mm^2}{m}$

$\sigma_z := \dfrac{f_{yk}}{\gamma_{ms}} \left(\dfrac{\psi_2 \cdot q_k + g_k}{1.5 q_k + 1.35 g_k} \right) \cdot \left(\dfrac{A_s}{A_{s\_eff}} \right) \cdot \dfrac{1}{\delta} = 196.022 \cdot MPa$

From Table 5.6 from BCA2006, maximum distance between bars to avoid crack > 0.3mm:

$MAX := 250 mm$

MOMENT SUPPORT, COLUMN

$\boxed{M_{m\_support} := 536 kN \cdot m}$ from moment distribution

Middle strip take 0.6 of the load

$\boxed{M_{col} := 0.6 \cdot M_{m\_support} = 321.6 \cdot kN \cdot m}$

$K := \dfrac{M_{col}}{\left(b \cdot d^2 \cdot f_{ck} \right)} = 0.038$

$Z := d \cdot \left[0.5 + \left(0.25 - \dfrac{K}{1.133} \right)^{0.5} \right] = 240.248$

$\dfrac{Z}{d} = 0.965$ use $Z := 0.95 \cdot d = 236.55$

$A_s := \dfrac{M_{col}}{0.87 \cdot f_{yk} \cdot Z} = 3.125 \times 10^3 \quad mm^2$

$\boxed{A_{s\_mm} := \dfrac{A_s \cdot mm^2}{L_x \cdot 0.5} = 694.53 \cdot \dfrac{mm^2}{m}}$

H14 at 175 c/c (880 mm2)

$A_{s\_eff} := 880 \dfrac{mm^2}{m}$ $\quad \rho := \dfrac{A_{s\_eff}}{h_{slab} \cdot 1} = 0.293 \cdot \%$

DEFLECTION

Span to effective ratio, ρ=0.3%, fck=30 *"How to..." Table C10* $\quad N := 39.2$

Flat slab, EN1992-1-1 Table 7.4N $\quad K := 1.2$

Gk/Qk=2.5, ψ=0.3, γ=1.25 *"How to..." Fig.C3* $\quad \sigma_{su} := 255 MPa$

Redistribution ratio $\quad \delta := 1.08$

A_{s_mm}

$\sigma_s := \sigma_{su} \cdot \dfrac{A_{s\_mm}}{A_{s\_eff} \cdot \delta} = 212.969 \dfrac{1}{m} \cdot MPa$

b_eff/b_w=1 *"How to.." Table C12*

No brittle partition "*How to.." Table C13*

$F1 := 1 \quad F2 := 1 \quad F3 := \dfrac{310 MPa}{\sigma_s} = 1.456 m$

Allowable deflection $\quad N \cdot K \cdot F1 \cdot F2 \cdot F3 = 68.472 m$

Actual $\quad \dfrac{span_{slab}}{d \cdot mm} = 36.145$ **OK**

<u>**Crack control**</u>

Steel stress under quasi permanent loading:

$\sigma_z := \dfrac{f_{yk}}{\gamma_{ms}} \left(\dfrac{\psi_2 \cdot q_k + g_k}{1.5 q_k + 1.35 g_k} \right) \cdot \left(\dfrac{A_{s\_mm}}{A_{s\_eff}} \right) \cdot \dfrac{1}{\delta} = 217.803 MPa$

Maximum diameter 20mm or distance 225mm.
H28@122 acceptable.

PUNCHING SHEAR

$w_{col} := 350mm$ $\qquad$ $b_{col} := 450mm$

$u_0 := (w_{col} + b_{col}) \cdot 2$ $\qquad$ $\beta := 1.5$

As per Figure 6.21N in EN 1992-1-1-2004

$$n_{m2} := \frac{n}{L_x \cdot L_y} = 13.965 \cdot \frac{kN}{m^2}$$

$$V_{Ed} := \left(\frac{L_x}{2} \cdot \frac{L_y}{2} - w_{col} \cdot b_{col} \right) \cdot n_{m2} \cdot \beta = 350.19 \cdot kN$$

$$\boxed{\frac{V_{Ed}}{u_0 \cdot d} = 0.879 \cdot MPa}$$

$$\upsilon := 0.6 \cdot \left(1 - \frac{f_{ck}}{250} \right) = 0.528 \quad f_{cd} := 1 \cdot 1 \cdot \frac{f_{ck}}{1.5} = 20$$

$$\boxed{v_{Rd} := 0.5 \cdot \upsilon \cdot f_{cd} = 5.28}$$

No additional reinforcement needed

PUNCHING SHEAR AT 2d

$$u_1 := 2 \cdot (b_{col} + w_{col}) + 2 \cdot \pi \cdot 2 \cdot d = 4.729m \qquad \gamma_c := 1.5$$

$$v_{USF} := n_{m2} \cdot \frac{L_x}{2} \cdot \frac{L_y}{2} \ ... \\ + n_{m2} \cdot \left[w_{col} \cdot 2 \cdot d \ ... \\ + b_{col} \cdot d \cdot 2 + \pi \cdot (2 \cdot d)^2 \right] = 252.104 \cdot kN$$

$$\upsilon_{u1} := \frac{1.15 \cdot v_{USF}}{u_1 \cdot d} = 0.246 \cdot MPa$$ Stress at u1

$$\rho_{lx} := \frac{1130}{d \cdot 1000} = 4.538 \times 10^{-3}$$

$$\rho_{ly} := \frac{1130}{d \cdot 1000} = 4.538 \times 10^{-3}$$

$$C_{Rd\_c} := \frac{0.18}{\gamma_c} = 0.12$$

$$k := \left(1 + \frac{200}{d} \right)^{0.5} = 1.343 \qquad \rho_1 := (\rho_{lx} \cdot \rho_{ly})^{0.5}$$

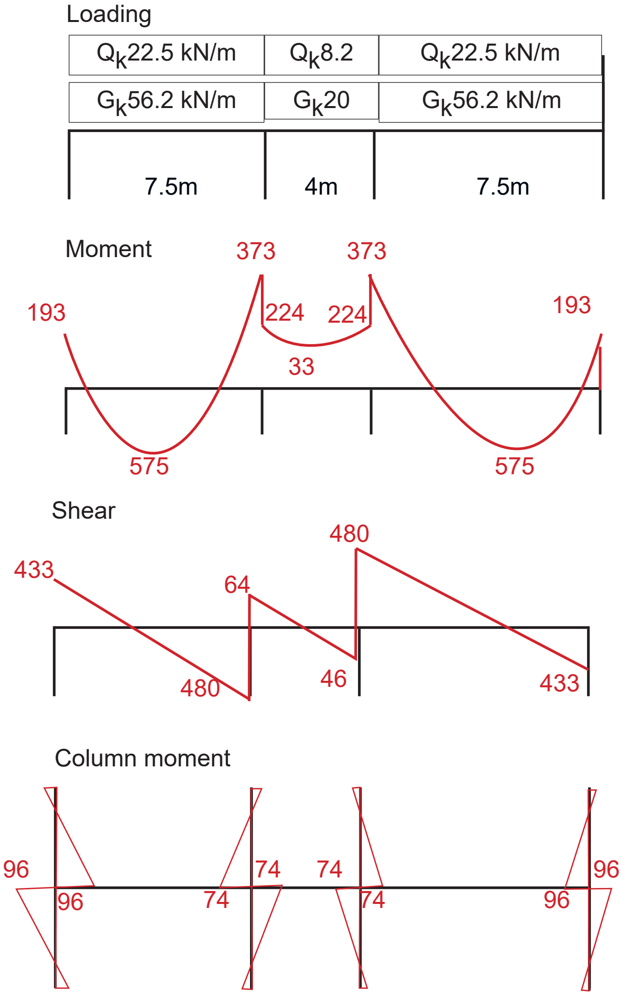
$$\upsilon_{Rd\_c} := C_{Rd\_c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{.33} \cdot MPa = 0.381 \cdot MPa$$

$$\boxed{V_{Rd\_c} := \upsilon_{Rd\_c} \cdot u_1 \cdot d = 449.155 \cdot kN}$$

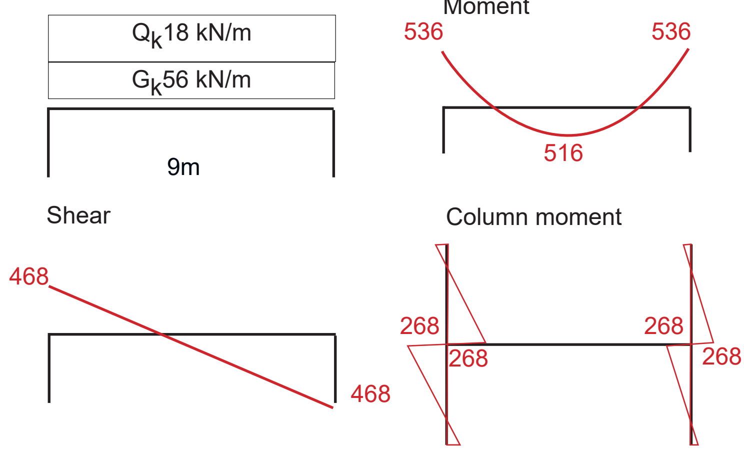
$$v_{USF} = 252.104 \cdot kN$$

Punching shear reinforcement not needed

Grid E / Level 19-22



Grid 4 / Level 19



Summary for slab analysis levels 19-22

Slab type: flat slab across the whole level

Thickness: 300mm across the whole level for ease of construction

Punching shear reinforcement is not needed near the supports. Reinforcement is estimated as follows. Refer to Figure 2.1 for strip numbering. Note: redistributing reinforcement within the strips and according to code would be needed.

Bay	Direction	Strip			
		1,3,7,9	2,8	4,6	5
1,3	Y	H10@200	H18@150	H8@200	H14@125
	Z	H16@150	H10@150	H10@125	H10@150
2	Y	H8@125	H8@150	H8@150	H8@150
	Z	H8@150	H8@150	H8@150	H8@150

4.2 Column design levels 19-22

Column E5 and E6

Analysis for internal column E5 and E6. The structural layout (Fig. 4.2.1) was simplified. The single bays (indicated as 45 and 56 in the calculation) are simplified as as shown in Fig 4.2.2.This simplification was only used for the moment calculation. The structure is still considered braced to the central core. The size of the column (350mm) is limited by fire regulation.

COLUMN Level 18 Column E6

DIMENSIONS AND MATERIAL CHARACTERISTICS

Column

$y := 350mm \qquad z := 400mm \qquad h_{col} := 3.5m$

Slab Grid 4-5

$y_{45} := 7.5m \qquad z_{45} := 9m \qquad h_{slab} := .30m$

Slab Grid 5-6

$y_{56} := 4m \qquad z_{56} := 3.3m$

$dia_{bar} := 12mm \qquad dia_{link} := 8mm$

$f_{cd} := \frac{30}{1.5}MPa \qquad f_{yd} := \frac{500}{1.5}MPa$

COVER

As per EN1992-1-2 TAble 5.2a and REI120 45mm

$cover := 45mm$

ACTIONS

Load from slab weight, variable load and cladding.
Y direction grid 4-6, Z direction grid E to F

$$g_k := 25\frac{kN}{m^3} \cdot h_{slab} = 7.5 \cdot \frac{kN}{m^2}$$

$$q_k := 2.5\frac{kN}{m^2} = 2.5 \cdot \frac{kN}{m^2} \qquad q_{k2} := 5\frac{kN}{m^2}$$

$$g_{k45} := z_{45} \cdot 1.35 \cdot g_k = 91.125 \cdot \frac{kN}{m}$$

$$q_{k45} := z_{45} \cdot 1.5 \cdot q_k = 33.75 \cdot \frac{kN}{m}$$

$$g_{k56} := z_{56} \cdot 1.35 \cdot g_k = 33.413 \cdot \frac{kN}{m}$$

$$q_{k56} := z_{56} \cdot 1.5 \cdot q_{k2} = 24.75 \cdot \frac{kN}{m}$$

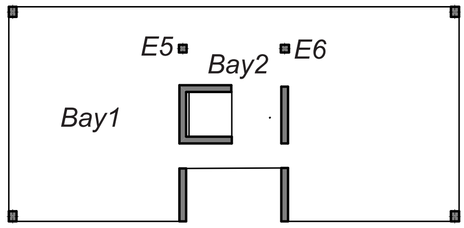


Figure 4.2.1: diagram of structural scheme for levels 18-22

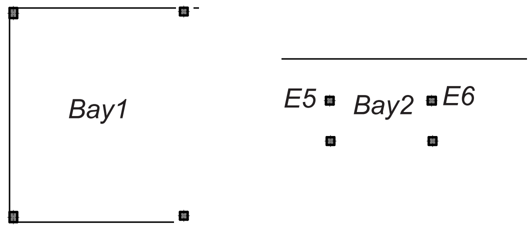
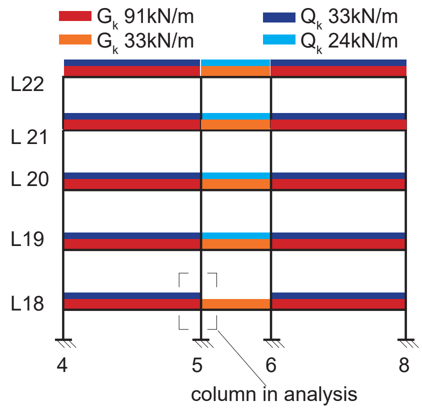


Figure 4.2.2: simplified scheme for bay 4-5 (left) and bay 5-6 and column analysis. Note the frame is still considered braced.



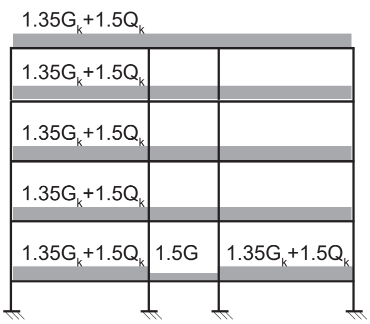


Figure 4.2.3 combination of actions

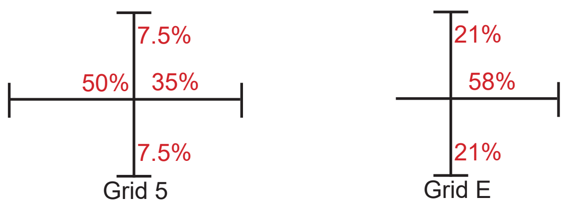


Figure 4.2.4 distribution factors along Y (Grid 5) and Z (Grid E)

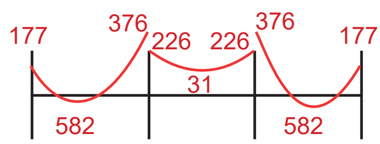


Figure 4.2.5: moment distribution along Y (Grid 5)

Axial Load

Area supported by the column on bay1 (grid 4-5):

$$A_{45} := z_{45} \cdot y_{45} \cdot 0.25 = 16.875\,m^2$$

$$A_{56} := z_{56} \cdot y_{56} \cdot 0.25 = 3.3\,m^2$$

Load per floor:

$$UDL_{56} := A_{56} \cdot \left(1.5 g_k + 1.5 q_{k2}\right) = 61.875 \cdot kN$$

$$UDL_{45} := A_{45} \cdot \left(1.35 g_k + 1.5 q_k\right) = 234.141 \cdot kN$$

Load from column

$$g_{col} := y \cdot z \cdot 4 \cdot 3.5m \cdot 25 \frac{kN}{m^3} = 49 \cdot kN$$

Load from cladding:

$$g_{clad} := 2.5 \frac{kN}{m^2} \cdot \left(\frac{y_{45} + y_{56}}{2}\right) \cdot 3.5 \cdot 4m = 201.25 \cdot kN$$

$$N_{Ed} := 5 \cdot \left(UDL_{56} + UDL_{45}\right) \ldots + -\left(A_{56} \cdot 1.5 q_{k2}\right) + g_{col} \ldots + g_{clad} = 1.706 \times 10^3 \cdot kN$$

DISTRIBUTION FACTORS

Stiffness, upper column $\quad E_{uc} := \frac{y^3 \cdot z}{h_{col}} = 4.9\,L$

Stiffness, lower column $\quad E_{lc} := E_{uc}$

Stiffness, slab grid 45:

$$E_{sl45} := z_{45} \cdot \frac{h_{slab}^{\;3}}{y_{45}} = 32.4\,L$$

Stiffness, slab grid 45:

$$E_{sl56} := z_{56} \cdot \frac{h_{slab}^{\;3}}{y_{56}} = 22.275\,L$$

Distribution factor upper and lower column

$$DF_{uc} := \frac{E_{uc}}{E_{lc} + E_{uc} + E_{sl45} + E_{sl56}} = 0.076$$

Distribution factor Ydirection slab 45

$$DF_{sl45} := \frac{E_{sl45}}{E_{lc} + E_{uc} + E_{sl45} + E_{sl56}} = 0.503$$

Distribution factor Y direction slab 56

$$DF_{sl56} := \frac{E_{sl56}}{E_{lc} + E_{uc} + E_{sl45} + E_{sl56}} = 0.345$$

Distribution factor Z direction column

$$DF_{uc\_z} := \frac{E_{uc}}{E_{lc} + E_{uc} + E_{sl45}} = 0.116$$

SLENDERNESS CHECK

$factor := 0.85$ $\quad$ From BS8110:Part 1:1997

$$l_0 := (3500mm - 300mm) \cdot factor = 2.72\,m$$

$$\lambda_z := 3.46 \cdot \frac{l_0}{z} = 23.528 \qquad \lambda_y := 3.46 \cdot \frac{l_0}{y} = 26.889$$

$A := 0.7$

A.s taken from end of calculation:

$$\omega := \frac{A_s \cdot f_{yd}}{y \cdot z \cdot f_{cd}} = 0.292 \qquad B := (1 + 2 \cdot \omega)^{0.5} = 1.259$$

Worst case for braced structures $\quad C := 1.7$

$$n := \frac{N_{Ed}}{y \cdot z \cdot factor \cdot f_{cd}} = 0.717$$

$$\lambda_{lim} := 20 \cdot \frac{A \cdot B \cdot C}{n^{0.5}} = 35.395$$

<u>Column not slender since λlim > λy and λz</u>

MOMENTS IN Y DIRECTION

From moment distribution (see Table 4.2.1):

$$M_{Edyy} := 74kN \cdot m$$

On the z direction

$$UDL_{z56} := y_{56} \cdot \left(1.35 \cdot g_k + 1.5 \cdot q_{k2}\right) = 70.5 \cdot \frac{kN}{m}$$

On the Z direction the cantilever produces a moment opposite and greater than the internal part of the slab

$$M_{Edzz} := \frac{\left(\frac{z_{56}}{2} \cdot UDL_{56}\right)}{12} \ldots + \frac{-z_{56}}{2} \cdot UDL_{56} = -93.586 \cdot kN \cdot m$$

$$M_{Edzz} := -M_{Edzz}$$

DESIGN REINFORCEMENT

$f_{ck} := 30MPa \qquad f_{yk} := 500\frac{N}{mm^2}$

$\boxed{\frac{M_{Edzz}}{y \cdot z^2 \cdot f_{ck}} = 0.056} \qquad \boxed{\frac{N_{Ed}}{y \cdot z \cdot f_{ck}} = 0.406} \qquad z = 0.4m$

$d_2 := cover + dia_{link} + dia_{bar} \cdot 0.5 = 0.059m$

$\frac{d_2}{z} = 0.147$

From chart with d/z of 0.15 As F.yk /bhf.yk
(Harrison2007a, p39)
$ratio := 0.05$

$\boxed{A_s := \frac{ratio \cdot y \cdot z \cdot f_{ck}}{f_{yk}} = 420 \cdot mm^2}$

8 bars H12 (905 mm^2)

Crushing load of a truly loaded column
may be taken as: (Mosley, p.267)

$N_{ud} := 0.567 \cdot f_{ck} \cdot y \cdot z + 0.87 \cdot A_s \cdot f_{yk} = 2.564 \times 10^3 \cdot kN$

$N_{Ed} = 1.706 \times 10^3 \cdot kN$

Minimum area of steel needs to be above 0.002Ac

$As_{min} := 0.002 \cdot y \cdot z = 280 \cdot mm^2 \qquad \frac{A_s}{z \cdot y} = 0.3 \cdot \%$

DESIGN FIRE RESISTANCE

check that effective length <3m
$l_0 := (3.1m - 0.3m) \cdot .85 = 2.38m$ OK

check e< e.max $\quad e := \frac{M_{Edyy}}{N_{Ed}} = 0.043m$

$e_{max} := 0.15 \cdot z = 0.06m$ OK

Check reinforcement <4% $\quad \frac{A_s}{z \cdot y} = 0.3 \cdot \%$ OK

CHECK BIAXIAL BENDING

$N_{Ed} = 1.706 \times 10^6 N$

$e_z := \frac{M_{Edzz}}{N_{Ed}} = 54.871 \cdot mm \qquad e_y := \frac{M_{Edyy}}{N_{Ed}} = 43.387 \cdot mm$

$\frac{e_y}{e_z} = 0.791$ between 0.2 and 5, design for biaxial bending needs to be checked

From chart obtain ratio of As f.yk/bhf.ck

Previously calculated reinforcement not enough,
8 H12 (905 mm2) used instead

$A_{s\_eff} := 905mm^2 \qquad A_{s\_eff} \cdot \frac{f_{yk}}{y \cdot z \cdot f_{ck}} = 0.108$

$\frac{N_{Ed}}{z \cdot y \cdot f_{ck}} = 0.406 \qquad \frac{d_2}{y} = 0.169$

$\boxed{f_{chart} := 0.088}$

$M_{Rd} := f_{chart} \cdot y \cdot z^2 \cdot f_{ck} = 147.84 \cdot kN \cdot m$

$N_{Rd} := z \cdot y \cdot f_{cd} \cdot 0.85 + A_{s\_eff} \cdot f_{yd} = 2.682 \times 10^3 \cdot kN$

Value for a obtained from chart (Harrison2007a, Table 5):

$\frac{N_{Ed}}{N_{Rd}} = 0.636 \qquad a := 1 + \left(\frac{N_{Ed}}{N_{Rd}} - 0.1\right) \cdot \frac{0.5}{0.6} = 1.447$

$\boxed{\left(\frac{M_{Edzz}}{M_{Rd}}\right)^a + \left(\frac{M_{Edyy}}{M_{Rd}}\right)^a = 0.884}$ *Acceptable*

CHECK REINFORCEMENT

As minimum $\quad \frac{0.10N_{Ed}}{f_{yd}} = 511.673 \cdot mm^2$

Reinforcement to area ratio $\quad \frac{A_{s\_eff}}{y \cdot z} = 0.646 \cdot \%$

Reinforcement is above 0.2% and below 4% and above As minimum.

TRANSVERSE REINFORCEMENT
Ref. EC2 9.5.3&NA

Diameter of bar shoudl exceed 6m of 1/4 of reinforcement bars

$dia_{min} := dia_{bar} \cdot 0.25 = 3 \cdot mm$ **Diameter 6mm**

Spacing given by the minimum

$20 \cdot dia_{bar} = 240 \cdot mm$

$y = 350 \cdot mm$ **H6 @ 240 mm**

$400mm = 400 \cdot mm$

Column E4, C4, E8, C8

Analysis for corner columns. The structural layout (Fig. 4.2.1) was simplified. Unlike for the internal columns, no simplification was done. Moments were analysed using equivalent frame method.

COLUMN Level 18 Column E4

DIMENSIONS AND MATERIAL CHARACTERISTICS

Column

$y := 350mm$ $\quad z := 400mm$ $\quad h_{col} := 3.5m$

Slab Grid 4-5

$y_{45} := 7.5m$ $\quad z_{45} := 9m$ $\quad h_{slab} := .30m$

$dia_{bar} := 14mm$ $\quad dia_{link} := 6mm$

$f_{cd} := \frac{30}{1.5}MPa$ $\quad f_{yd} := \frac{500}{1.5}MPa$

COVER

As per EN1992-1-2 TAble 5.2a and REI120 45mm

$cover := 45mm$

ACTIONS

Load from slab weight, variable load and cladding.
Y direction grid 4-6, Z direction grid E to F

$$g_k := 25\frac{kN}{m^3} \cdot h_{slab} = 7.5 \cdot \frac{kN}{m^2}$$

$$q_k := 2.5\frac{kN}{m^2} = 2.5 \cdot \frac{kN}{m^2} \qquad q_{k2} := 5\frac{kN}{m^2}$$

$$g_{k45} := z_{45} \cdot 1.35 \cdot g_k = 91.125 \cdot \frac{kN}{m}$$

$$q_{k45} := z_{45} \cdot 1.5 \cdot q_k = 33.75 \cdot \frac{kN}{m}$$

Axial Load

Area supported by the column on bay1 (grid 4-5):

$$A_{45} := z_{45} \cdot y_{45} \cdot 0.25 = 16.875 m^2$$

Load per floor:

$$UDL_{45} := A_{45} \cdot \left(1.35g_k + 1.5q_k\right) = 234.141 \cdot kN$$

Load from column

$$g_{col} := y \cdot z \cdot 4 \cdot 3.5m \cdot 25\frac{kN}{m^3} = 49 \cdot kN$$

Load from cladding:

$$g_{clad} := 2.5\frac{kN}{m^2} \cdot \left(\frac{y_{45} + z_{45}}{2}\right) \cdot 3.5 \cdot 4m = 288.75 \cdot kN$$

$$N_{Ed} := 5 \cdot \left(UDL_{45}\right) + g_{col} + g_{clad} = 1.508 \times 10^3 \cdot kN$$

SLENDERNESS CHECK

$factor := 0.85$ From BS8110:Part 1:1997

$$l_0 := (3500mm - 300mm) \cdot factor = 2.72 m$$

$$\lambda_z := 3.46 \cdot \frac{l_0}{z} = 23.528 \qquad \lambda_y := 3.46 \cdot \frac{l_0}{y} = 26.889$$

$A := 0.7$

A.s taken from end of calculation:

$$A_s := 1.23 \cdot 10^3 mm^2$$

$$\omega := \frac{A_s \cdot f_{yd}}{y \cdot z \cdot f_{cd}} = 0.146 \qquad B := (1 + 2 \cdot \omega)^{0.5} = 1.137$$

Worst case for braced structures $\quad C := 1.7$

$$n := \frac{N_{Ed}}{y \cdot z \cdot factor \cdot f_{cd}} = 0.634$$

$$\lambda_{lim} := 20 \cdot \frac{A \cdot B \cdot C}{n^{0.5}} = 33.992$$

<u>Column not slender since λlim > λy and λz</u>

MOMENTS

From moment distribution (see table 4.2.1)
the column would receive a moment in Y direction of:

$$M_{Edyy} := 96kN \cdot m$$

On the Z direction the moment would be:

$$M_{Edzz} := 133kN \cdot m$$

Transfer to edge and corner column , both upper and lower,is limited to Mmax
(and therefore lower column to Mmax/2):

$$d_{slab} := 245mm$$

$$M_{max} := 0.17 \cdot (z + y) \cdot d_{slab}^{\;2} \cdot f_{ck} = 229.596 \cdot kN \cdot m$$

Therefore none of the moments is greater than maximum value.

DESIGN REINFORCEMENT

$f_{ck} := 30MPa \qquad f_{yk} := 500 \frac{N}{mm^2}$

$\boxed{\frac{M_{Edzz}}{y \cdot z^2 \cdot f_{ck}} = 0.079} \qquad \boxed{\frac{N_{Ed}}{y \cdot z \cdot f_{ck}} = 0.359} \qquad z = 0.4 m$

$d_2 := cover + dia_{link} + dia_{bar} \cdot 0.5 = 0.058 m$

$\frac{d_2}{z} = 0.145$

From chart with d/z of 0.15 As F.yk /bhf.yk (Harrison2007a, p39)

$ratio := 0.06$

$\boxed{A_s := \frac{ratio \cdot y \cdot z \cdot f_{ck}}{f_{yk}} = 504 \cdot mm^2}$

<u>8 bars H14 (905mm^2)</u>

Crushing load of a truly loaded column may be taken as: (Mosley, p.267)

$N_{ud} := 0.567 \cdot f_{ck} \cdot y \cdot z + 0.87 \cdot A_s \cdot f_{yk} = 2.601 \times 10^3 \cdot kN$

$N_{Ed} = 1.508 \times 10^3 \cdot kN$

Minimum area of steel needs to be above 0.002Ac

$As_{min} := 0.002 \cdot y \cdot z = 280 \cdot mm^2 \qquad \frac{A_s}{z \cdot y} = 0.36 \cdot \%$

DESIGN FIRE RESISTANCE

check that effective length <3m

$l_0 := (3.1m - 0.3m) \cdot .85 = 2.38 m$ OK

check e< e.max $\quad e := \frac{M_{Edyy}}{N_{Ed}} = 0.064 m$

$e_{max} := 0.15 \cdot z = 0.06 m$ OK

Check reinforcement <4% and above 0.2% $\quad \frac{A_s}{z \cdot y} = 0.36 \cdot \%$ OK

CHECK BIAXIAL BENDING

$N_{Ed} = 1.508 \times 10^3 \cdot kN$

$e_z := \frac{M_{Edzz}}{N_{Ed}} = 88.17 \cdot mm \qquad e_y := \frac{M_{Edyy}}{N_{Ed}} = 63.641 \cdot mm$

$\frac{e_y}{e_z} = 0.722$ between 0.2 and 5, design for biaxial bending needs to be checked

From chart obtain ratio of As f.yk/bhf.ck

Previously calculated reinforcement not enough, 8 H14 (1230 mm2) used instead

$A_{s\_eff} := 1230mm^2 \qquad A_{s\_eff} \cdot \frac{f_{yk}}{y \cdot z \cdot f_{ck}} = 0.146$

$\frac{N_{Ed}}{z \cdot y \cdot f_{ck}} = 0.359 \qquad \frac{d_2}{y} = 0.166$

$\boxed{f_{chart} := 0.11}$

$M_{Rd} := f_{chart} \cdot y \cdot z^2 \cdot f_{ck} = 184.8 \cdot kN \cdot m$

$N_{Rd} := z \cdot y \cdot f_{cd} \cdot 0.85 + A_{s\_eff} \cdot f_{yd} = 2.79 \times 10^3 \cdot kN$

Value for a obtained from chart (Harrison2007a, Table 5):

$\frac{N_{Ed}}{N_{Rd}} = 0.541 \qquad a := 1 + \left(\frac{N_{Ed}}{N_{Rd}} - 0.1\right) \cdot \frac{0.5}{0.6} = 1.367$

$\boxed{\left(\frac{M_{Edzz}}{M_{Rd}}\right)^a + \left(\frac{M_{Edyy}}{M_{Rd}}\right)^a = 1.046}$ *Acceptable*

CHECK REINFORCEMENT

As minimum $\quad \frac{0.10 N_{Ed}}{f_{yd}} = 452.536 \cdot mm^2$

Reinforcement to area ratio $\quad \frac{A_{s\_eff}}{y \cdot z} = 0.879 \cdot \%$

Reinforcement is above 0.2% and below 4% and above As minimum.

TRANSVERSE REINFORCEMENT
Ref. EC2 9.5.3&NA

Diameter of bar shoudl exceed 6m of 1/4 of reinforcement bars

$dia_{min} := dia_{bar} \cdot 0.25 = 3.5 \cdot mm$ **Diameter 6mm**

Spacing given by the minimum

$20 \cdot dia_{bar} = 280 \cdot mm$

$y = 350 \cdot mm$ **H6 @ 240 mm**

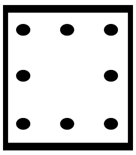
$400mm = 400 \cdot mm$

Table 4.2.1: moment distribution along Y direction (Grid E)

Load G /m2			7.25				7.25				7.25		
Load Q/m2			2.5				5				2.5		
Depth			9				3.3				9		
span			7.5				4				7.5		
Load kN/m			121.8				32.3				121.8		
Load kN			913.8								913.8		
Shear			913.8				129.2				913.8		
support	E8				E6				E5				E4
	Column	Slab 86		slab 68	Column	Slab 65		slab 56	Column	Slab 54		slab 45	Column
distribution	24%	76%		50%	15%	35%		35%	15%	50%		76%	24%
FEM			571.1				43.1				571.1		
Moment		571.1		-571.1		43.1		-43.1		571.1		-571.1	
Balance	-137.07	-434.0		264.0	79.2	184.8		-184.8	-79.2	-264.0		434.0	137.1
Distribute													
Carry over		132.0		-217.0		-92.4		92.4		217.0		-132.0	
Balance													
Distribute	-31.68	-100.3		154.7	46.4	108.3		-108.3	-46.4	-154.7		100.3	31.7
Carry over		77.4		-50.2		-54.2		54.2		50.2		-77.4	
Balance													
Distribute	-18.57	-58.8		52.2	15.6	36.5		-36.5	-15.6	-52.2		58.8	18.6
Carry over		26.1		-29.4		-18.3		18.3		29.4		-26.1	
Balance													
Distribute	-6.26	-19.8		23.8	7.1	16.7		-16.7	-7.1	-23.8		19.8	6.3
Total	**-193.57**	**193.6**		**-373.0**	**148.4**	**224.6**		**-224.6**	**-148.4**	**373.0**		**-193.6**	**193.6**

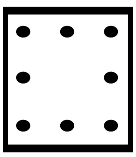
Column design summary

Column E4,C4, E8,C8



8 H14
H6 links at 200
350 x 400mm
f_{ck}=30MPa

Column E5,E6



8 H12
H6 links at 240
350 x 350mm
f_{ck}=30MPa

4.3 Slab analysis / levels 3-18

Slabs between grid 2 and 4 were initially evaluated as flat slabs. The spans required in lower levels (3 to 6) are in a range (9.5-12m) outside the optimal. The difference in height between a flat slab and slab+beam would not be significant with the disadvantage of having a smaller clearance within the bay.

Bay 1 was than changed to a slab and beam system and analysed as a two way slab. The geometry was simplified into a rectangular shape of 6.5 x 11.5m. It was assumed that the slab has fixity at the corners to prevent torsion and uplifting. Calculation are based on Example 8.5.2 from Mosley (2012). A summary of reinforcement is shown in Table 4.3.

Bays within grid 4 and 8 are proposed as flat slabs considering the reduced spans that allow to take full advantage of the reduced thickness of this type of structure. Reinforcement for Bay 2 is shown in Table 4.3. Calculation follows the same procedure as for upper levels and are not shown in this document. Refer to previous pages.

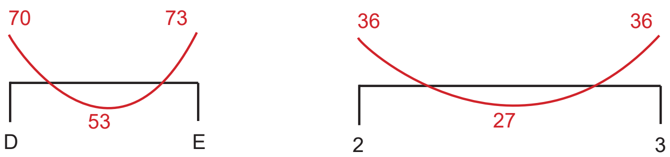


Fig: 4.3.2: diagram of moments for Bay 1 (two way slab)

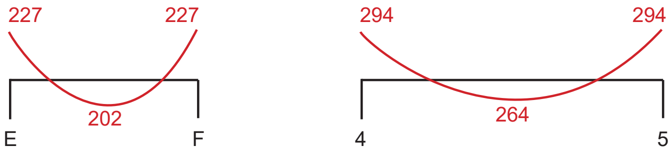


Fig: 4.3.3: diagram of moments for Bay 2 (flat slab)

Table: 4.3: summary of reinforcement for Bay 1 and 2

Bay	Direction	Strip 1,3,7,9	2,8	4,6	5
1	Y	H14@200	H10@225	H14@275	H10@225
	Z	H14@200	H14@200	H14@250	H14@250
2	Y	H10@200	H10@150	H10@200	H10@150
	Z	H8@150	H8@150	H8@150	H8@150

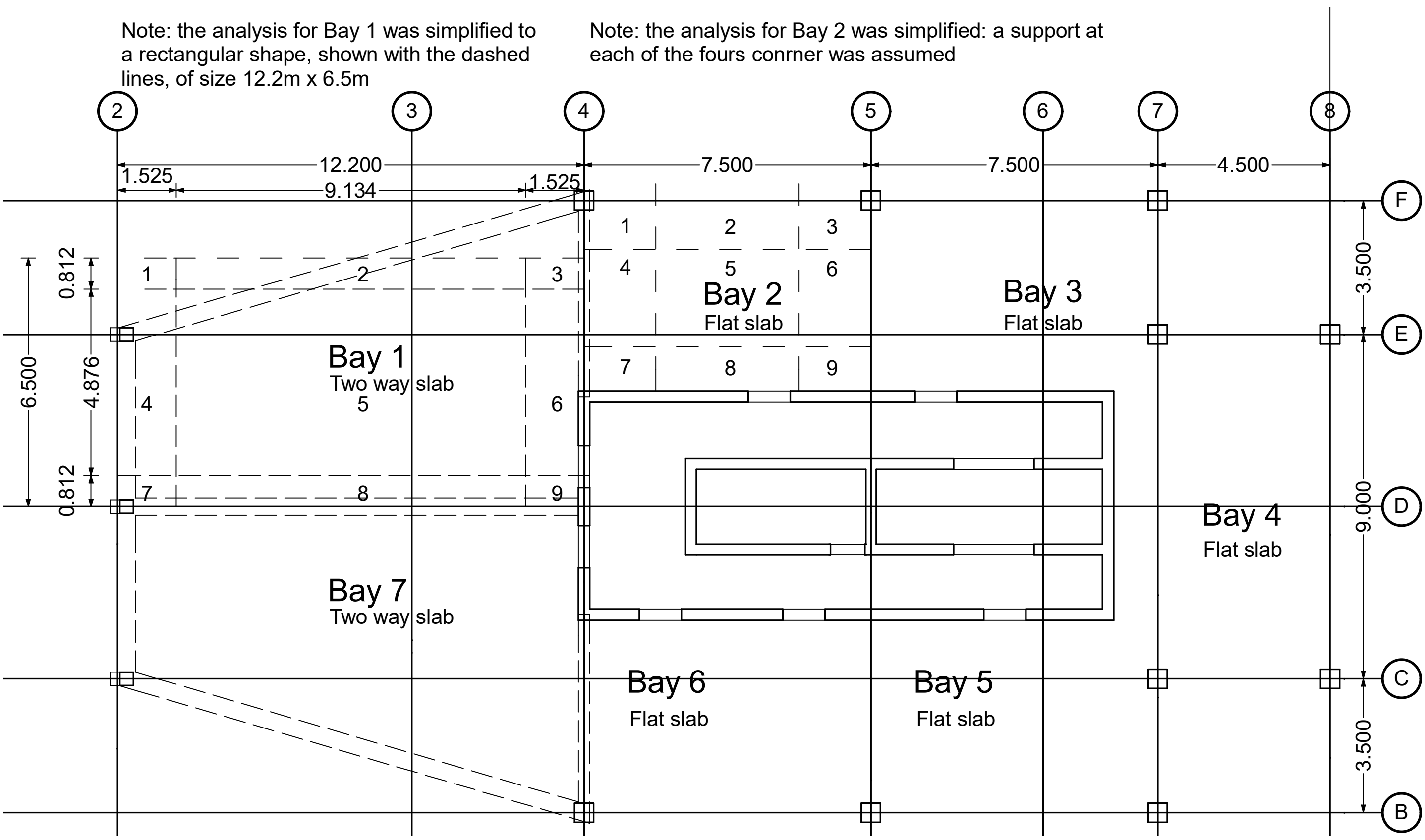


Figure 4.3.1: ceiling plan of level 2

TWO WAY SLAB LEVEL 3 (as per Brooker2006)

The irregular shape of the slab is approximated to a rectangular one of:

Dimensions:

$l_x := 6.5m \quad l_y := 12.2m \quad h_{sl} := \frac{l_y}{36} = 0.339\,m \quad \frac{l_y}{l_x} = 1.877$

Effective depth:

$h_{sl} := 350mm \qquad d := h_{sl} - 45mm - 8mm = 0.297\,m$

Coefficients for restrained slab (T3.14 BS8110)

Short span,continuos edge: $\beta_{xe} := 0.087$

Short span,mid-span: $\beta_{xm} := 0.065$

Long span,continuos edge: $\beta_{ye} := 0.045$

Long span,mid-span: $\beta_{ym} := 0.034$

ULTIMATE LOAD

Imposed load

$g_k := h_{sl} \cdot 25\frac{kN}{m^3} = 8.75 \cdot \frac{kN}{m^2} \qquad q_k := 5\frac{kN}{m^2}$

Ultimate load: $n := 1.35 \cdot g_k + 1.5 \cdot q_k = 19.313 \cdot \frac{kN}{m^2}$

MOMENT AT EDGE, SHORT SPAN

$M := \beta_{xe} \cdot n \cdot l_x^{\ 2} = 70.988 \cdot kN \cdot m \qquad b := 1000$

$f_{ck} := 30MPa \quad f_{yk} := \frac{500}{1.15}MPa$

$K := \frac{M}{\left(b \cdot d^2 \cdot f_{ck}\right)} = 0.027$

$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 289.7\,mm$

$\frac{Z}{d} = 0.976$ Design for 0.95 $\quad Z := 0.95 \cdot d$

$\boxed{A_s := \frac{M}{0.87 \cdot f_{yk} \cdot Z} = 665.14 \frac{s^2}{kg} \cdot \frac{mm^2}{m}}$

<u>**H14 @200 (770mm2)**</u>

MOMENT AT MIDSPAN SHORT SPAN

$M := \beta_{xm} \cdot n \cdot l_x^{\ 2} = 53.037 \cdot kN \cdot m \qquad b := 1000$

$f_{ck} := 35MPa \quad f_{yk} := \frac{500}{1.15}MPa$

$K := \frac{M}{\left(b \cdot d^2 \cdot f_{ck}\right)} = 0.017$

$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 292.\,mm$

$\frac{Z}{d} = 0.985$ Design for 0.95 $\quad Z := 0.95 \cdot d$

$\boxed{A_s := \frac{M}{0.87 \cdot f_{yk} \cdot Z} = 496.944 \cdot \frac{mm^2}{m}}$

<u>**H14 @250 (616mm2)**</u>

MOMENT AT EDGE, LONG SPAN

$M := \beta_{ye} \cdot n \cdot l_x^{\ 2} = 36.718 \cdot kN \cdot m \qquad b := 1000$

$f_{ck} := 30MPa \quad f_{yk} := \frac{500}{1.15}MPa$

$K := \frac{M}{\left(b \cdot d^2 \cdot f_{ck}\right)} = 0.014$

$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 293.3\,mm$

$\frac{Z}{d} = 0.988$ Design for 0.95 $\quad Z := 0.95 \cdot d$

$\boxed{A_s := \frac{M}{0.87 \cdot f_{yk} \cdot Z} = 344.038 \frac{s^2}{kg} \cdot \frac{mm^2}{m}}$

<u>**H14 @275 (560 mm2)**</u>

MOMENT AT MIDSPAN LONG SPAN

$M := \beta_{ym} \cdot n \cdot l_x^{\ 2} = 27.742 \cdot kN \cdot m \qquad b := 1000$

$f_{ck} := 35MPa \quad f_{yk} := \frac{500}{1.15}MPa$

$K := \frac{M}{\left(b \cdot d^2 \cdot f_{ck}\right)} = 8.986 \times 10^{-3}$

$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 294.\,mm$

$\frac{Z}{d} = 0.992$ Design for 0.95 $\quad Z := 0.95 \cdot d$

$\boxed{A_s := \frac{M}{0.87 \cdot f_{yk} \cdot Z} = 259.94 \cdot \frac{mm^2}{m}}$

<u>**H10 @225 (349 mm2)**</u>

SHEAR

Coefficient (Table 3.15 BS8110) $\beta_{vx} := 0.5$

$V_{sx} := \beta_{vx} \cdot n \cdot l_x = 62.766 \cdot \frac{kN}{m}$

$v := \frac{V_{sx}}{b \cdot d} = 0.211 \cdot \frac{N}{mm^2}$

$v_c := \frac{100A_s}{b \cdot d} = 0.088$

v.c > v No shear reinforcement needed

DEFLECTION

Initial data : $b := 1000$ $f_{yk} := 475MPa$ $f_y := 500$

$f_{ck} := 35MPa$ $d = 297$

Maximum sagging moment:

$M_{sag} := \beta_{xe} \cdot n \cdot l_x^2 = 70.988 m^2 \cdot \frac{kN}{m}$

$K := \frac{M_{sag}}{\left(b \cdot d^2 \cdot f_{ck}\right)} = 0.012$

$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 293\ mm$

$\boxed{\frac{Z}{d} = 0.989}$ Design for 0.95 $Z := 0.95 \cdot d$

Reinforcement required:

$A_s := \frac{M_{sag}}{0.87 \cdot f_{yk} \cdot Z} = 608.824 m \cdot mm^2$

Try: $A_{s\_eff} := 753mm^2$ H12 @150

Actual span/depth: $\frac{l_x}{d} = 21.886$

$f_s := \frac{2 \cdot f_y \cdot A_s}{3 \cdot A_{s\_eff}} = 269.51 m$ $\frac{M_{sag}}{b \cdot d^2} = 0.805m$

Modification factor: (Table C7 in Brooker 2006)

$MF := 1.37$

Allowable span/depth $MF \cdot 26 = 35.62$

$\frac{l_x}{d} = 21.886$

greater than l.x/d therefore ok

4.4 Beam analysis level 2

BEAM REINFORCEMENT E2-F4

The irregular shape of the slab is approximated to a rectangular one of:

Dimensions: $l_x := 6.5m$ $l_y := 12.2m$

$h_{sl} := \frac{l_y}{36} = 0.339 m$ $\frac{l_y}{l_x} = 1.877$

Effective depth:

$h_{sl} := 350mm$ $h_{beam} := 600mm$

$f_{ck} := 30 \frac{N}{mm^2}$ $f_{yk} := 500MPa$

$d := h_{beam} - 40mm - 32mm = 0.528m$ $b := 400mm$

ULTIMATE LOAD

Imposed load

$g_k := h_{sl} \cdot 25 \frac{kN}{m^3} = 8.75 \cdot \frac{kN}{m^2}$ $q_k := 2.5 \frac{kN}{m^2}$

Ultimate load/m2: $n := 1.35 \cdot g_k + 1.5 \cdot q_k = 15.563 \cdot \frac{kN}{m^2}$

Ultimate load: $UDL := n \cdot l_x \cdot l_y \cdot 0.25 = 308.527 \cdot kN$

<u>Reinforcement required at support:</u>

$M_{sup} := UDL \cdot \frac{l_y}{12} = 313.669 \cdot kN \cdot m$

$K := \frac{M_{sup}}{b \cdot d^2 \cdot f_{ck}} = 0.094$ below 0.167, no compression reinforcement required

$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133}\right)^{0.5}\right] = 479.929 \cdot mm$

$\frac{Z}{d} = 0.909$ OK , above 0.82

$A_s := \frac{M_{sup}}{0.87 \cdot f_{yk} \cdot Z} = 1.502 \times 10^3 \cdot mm^2$

$\rho := \frac{A_s}{b \cdot d} = 0.711 \cdot \%$ *Acceptable*

Try 3 H26 (1590 mm2) $A_{s\_eff} := 1590mm^2$

Spacing $250 - 2 \cdot 40 - 2 \cdot 10 - 26 = 124$ mm

Steel stress under quasi permanent loading:

$\sigma_z := \frac{f_{yk}}{\gamma_{ms}} \left(\frac{\psi_2 \cdot q_k + g_k}{1.5q_k + 1.35g_k}\right) \cdot \left(\frac{A_s}{A_{s\_eff}}\right) \cdot \frac{1}{\delta} = 252.998 \cdot MPa$

Maximum diameter 16mm or distance 175mm.
H28@122 acceptable.

Reinforcement in the span

$$M_{span} := UDL \cdot \frac{l_y}{24} = 156.834\, m \cdot kN$$

$$K := \frac{M_{span}}{b \cdot d^2 \cdot f_{ck}} = 0.047$$ below 0.167, no compression reinforcement required

$$Z := d \cdot \left[0.5 + \left(0.25 - \frac{K}{1.133} \right)^{0.5} \right] = 505.165 \cdot mm$$

$$\frac{Z}{d} = 0.957$$ OK , above 0.82

$$A_s := \frac{M_{sup}}{0.87 \cdot f_{yk} \cdot Z} = 1.427 \times 10^3 \cdot mm^2$$

$$\rho := \frac{A_s}{b \cdot d} = 0.676 \cdot \% \qquad Acceptable$$

Try 3 H26 (1590 mm2) $A_{s\_eff} := 1590mm^2$

Spacing $250 - 2 \cdot 40 - 2 \cdot 10 - 26 = 124$ mm

Steel stress under quasi permanent loading:

$$\sigma_z := \frac{f_{yk}}{\gamma_{ms}} \left(\frac{\psi_2 \cdot q_k + g_k}{1.5q_k + 1.35g_k} \right) \cdot \left(\frac{A_s}{A_{s\_eff}} \right) \cdot \frac{1}{\delta} = 240.359 \cdot MPa$$

Maximum diameter 16mm or distance 200mm.
H28@122 acceptable.

Deflection

Reinforcement required $A_s := 1387mm^2$

Reinforcement provided $A_{s\_eff} := 2410mm^2$

Reinforcement ratio $\rho := \dfrac{A_s}{h_{beam} \cdot h_{sl}} = 0.66 \cdot \%$

Length to span ratio from Table NA.5, with fck=30 and considering this beam as end span of a continuos beam:

$K := 1.3$

From table (Mosley Fig 6.3), ratio $R := 21.7$

Final ratio $R := K \cdot R = 28.21$

To avoid damages beyond 7m: $R := R \cdot \dfrac{7m}{l_y} = 16.186$

Modification for steel area provided:

$$R := R \cdot \frac{f_{yk}}{500MPa} \cdot \frac{A_{s\_eff}}{A_s} = 28.124$$

Span to effective depth provided:

$$\frac{l_y}{d} = 23.106$$ deflection requirements likely to be satisfied

**Revise reinforcement at mid span as:
3H32 (2410mm^2) due to deflection.**

Spacing <200 is acceptable to avoid cracking above 0.3mm

Shear

$V_{Ed} := 145kN$

$$\upsilon_{Ed} := \frac{V_{Ed}}{h_{sl} \cdot h_{beam}} = 0.69 \cdot MPa$$

v.Rd maximum considering cotangent as 2.5 amd f.yk as 30 MPa is:

$v_{Rd} := 3.64MPa$

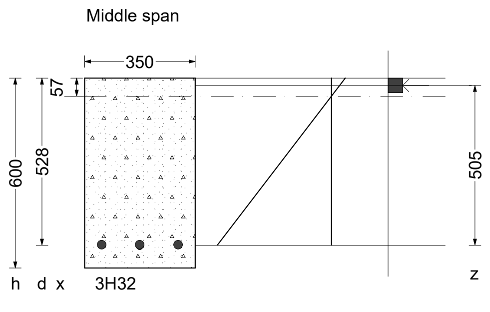
Max shear capacity greater than shear at support.

Shear reinforcement above:

$cot\theta := 2.5 \qquad f_{ywd} := 435$

Reinforcement area / spacing ratio (EN1992-1-1 6.2.3(3)) $\dfrac{V_{Ed} \cdot 10^3}{Z \cdot f_{ywd} \cdot cot\theta} = 0.264$

Use H8 at 250mm



Strain diagram for the beam on level 2 at mid span

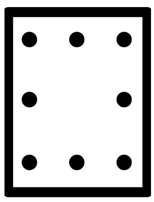
4.5 Column (F4) analysis level 2

Column F4 on level 2 can be analysed under the following stress:

Axial load from upper structure:
$N_{Ed} = 4.7\ 10^3$ kN
Crushing load N_{ult}=5.5 10^3 kN

Moment on Y direction: the moment from the beam E2-F4 is in almost equilibrium (M=5kNm) with the moment from the slab F4-F5. The total moment is therefore:
M_{Edyy}= 37kNm

Moment on Z direction is the sum of moment produced by the flat slab (120kNm), 0.05% of moment from lateral load and the eccentricity of the axial load.
M_{Edzz}=108kNm



Summary for column F4

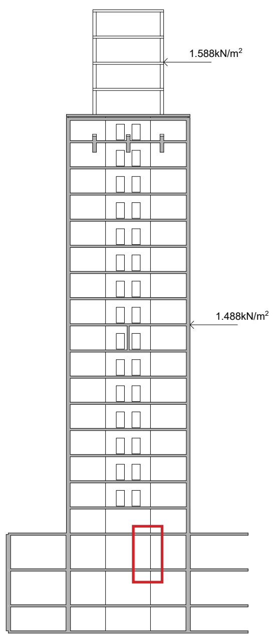


Figure 4.5.1: diagram showing the wind loading acting on the building

COLUMN Level 2 Column F4

DIMENSIONS AND MATERIAL CHARACTERISTICS

Column

$y := 500mm$ $z := 550mm$ $h_{col} := 3.5m$

$dia_{bar} := 22mm$ $dia_{link} := 8mm$

$f_{cd} := \frac{30}{1.5} MPa$ $f_{yd} := \frac{500}{1.5} MPa$

Beam E2-F4

$y_{24} := 12.2m$ $z_{24} := .35m$ $h_b := .55m$

Slab Grid 5-6

$y_{45} := 7.5m$ $z_{45} := 5.1m$ $h_{slab} := .30m$

COVER

As per EN1992-1-2 TAble 5.2a and REI120 45mm

$cover := 45mm$

ACTIONS

Load from slab weight, variable load and cladding.
Y direction grid 2-4, Z direction grid E to F

Equation 6.10 is used, with factor obtained from National Annex of:

Permanent actions unfavourable:	$\gamma_{pu} := 1.35$
Permanent actions favourable:	$\gamma_{pf} := 1$
Leading variable actions unfavourable:	$\gamma_{vu} := 1.5$
Leading variable actions favourable:	$\gamma_{vf} := 0$
Leading variable actions unfavourable:	$\gamma_{au} := 1.5 \cdot 0.7$
Accompaigning variable actions favourable:	$\gamma_{af} := 0$

AXIAL LOADING

$$g_k := 25\frac{kN}{m^3}\cdot h_{slab} = 7.5\cdot\frac{kN}{m^2} \qquad g_{clad} := 2\frac{kN}{m^2}$$

$$q_k := 2.5\frac{kN}{m^2} = 2.5\cdot\frac{kN}{m^2}$$

Area supported on levels 18-22:

$$A_{1922} := 4m\cdot 5.3\cdot m = 21.2m^2$$

Sum of Area supported on levels 3-18:

$$A_{318} := 102m^2$$

Permanent loading:

$$G_{1922} := A_{1922}\cdot\left(\gamma_{pu}\cdot g_k\right)\cdot 4 = 858.6\cdot kN$$

$$G_{318} := A_{318}\cdot\left(\gamma_{pu}\cdot g_k\right) = 1.033\times 10^3\cdot kN$$

For cladding

$$G_{1922c} := \left(7m\cdot 3.5m\cdot 2\frac{kN}{m^2}\right)\cdot 4\cdot\gamma_{pu} = 264.6\cdot kN$$

$$G_{318c} := \gamma_{pu}\left(3.75m\cdot 3.5m\cdot 15\ \ldots + 3.5m\cdot 15\cdot\frac{12m}{2}\right)\cdot 2\frac{kN}{m^2} = 1.382\times 10^3\cdot kN$$

Columns :

$$G_{1922cl} := \gamma_{pu}\cdot 3.5m\cdot 4\cdot .35m\cdot .45m\cdot 25\frac{kN}{m^3} = 74.419\cdot kN$$

$$G_{318cl} := \gamma_{pu}\cdot 3.5m\cdot 15\cdot .45m\cdot .55m\cdot 25\frac{kN}{m^3} = 438.539\cdot kN$$

$$G_{tot} := G_{1922} + G_{318} + G_{1922c}\ \cdots + G_{318c}\ \cdots + G_{1922cl} + G_{318cl} = 4.051\times 10^6 N$$

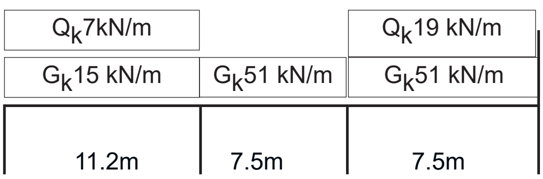


Figure 4.5.2 load distribution along E2-F4-F7 on level 2

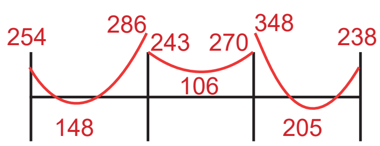


Figure 4.5.3: moment distribution along E2-F4-F7 on level 2

Leading variable action:
The leading variable is assumed to be variable loads on the structure. Wind is consider accoimpaigning variable due to shear walls taking most of lateral force and leaving each column with 0.05% of the load.

$$Q_{1922} := A_{1922}\cdot\left(\gamma_{vu}\cdot q_k\right)\cdot 4 = 318\cdot kN$$

$$Q_{318} := A_{318}\cdot\left(\gamma_{vu}\cdot q_k\right) = 382.5\cdot kN$$

$$Q_{tot} := Q_{1922} + Q_{318} = 7.005\times 10^5 N$$

$$\boxed{N_{Ed} := Q_{tot} + G_{tot} = 4.751\times 10^3\cdot kN}$$

Accompaining variable (wind)
Wind pressure is simplified as:

Levels 19-22 $\qquad P_{1922} := 1.588\frac{kN}{m^2}$

Levels 19-22 $\qquad P_{318} := 1.488\frac{kN}{m^2}$

The application point in respect to the column base on level 2 is place halfway through the height section considered:

$$L_{1922} := 72.6m - 9.6m = 63m$$

$$L_{318} := 37.7m - 9.6m = 28.1m$$

Facade area:

$$A_{1922} := 14m\cdot 19.8m = 277.2m^2$$

$$A_{318} := 1447m^2$$

Force applied in each seciton of the building:

$$F_{w1922} := A_{1922}\cdot P_{1922} = 4.402\times 10^5 N$$

$$F_{w318} := A_{318}\cdot P_{318} = 2.153\times 10^6 N$$

Portion (0.05%) of the moment produced at the base of the column by wind:

$$M_w := \left(F_{w1922}\cdot L_{1922}\ \ldots + F_{w318}\cdot L_{318}\right)\cdot .05\% = 44.118\cdot kN\cdot m$$

DISTRIBUTION FACTORS

Stiffness, upper column $E_{uc} := \frac{y^3 \cdot z}{h_{col}} = 19.643\,L$

Stiffness, lower column $E_{lc} := E_{uc}$

Stiffness, beam E2-F4

$$E_{b24} := z_{24} \cdot \frac{h_b^{\ 3}}{y_{24}} = 4.773\,L$$

Stiffness, slab grid 45,

$$E_{sl45} := z_{45} \cdot \frac{h_{slab}^{\ 3}}{y_{45}} = 18.36\,L$$

Distribution factor upper and lower column in F4

$$DF_{uc} := \frac{E_{uc}}{E_{lc} + E_{uc} + E_{b24} + E_{sl45}} = 0.315$$

Distribution factor for beam 24

$$DF_{sl24} := \frac{E_{b24}}{E_{lc} + E_{uc} + E_{b24} + E_{sl45}} = 0.076$$

Distribution factor Y direction slab 56

$$DF_{sl45} := \frac{E_{sl45}}{E_{lc} + E_{uc} + E_{b24} + E_{sl45}} = 0.294$$

Distribution factor upper and lower column F5

$$DF_{uc} := \frac{E_{uc}}{E_{lc} + E_{uc} + E_{sl45} + E_{sl45}} = 0.258$$

Distribution factor upper and lower column F7

$$DF_{uc} := \frac{E_{uc}}{E_{lc} + E_{uc} + E_{sl45}} = 0.341$$

Distribution factor upper and lower column E2

$$DF_{uc} := \frac{E_{uc}}{E_{lc} + E_{uc} + E_{b24}} = 0.446$$

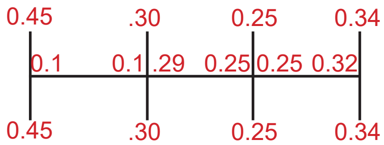


Figure 4.5.4: distribution factors along E2-F4-F7 on level 2

SLENDERNESS CHECK

$factor := 0.85$ From BS8110:Part 1:1997

$l_0 := (3500mm - 300mm) \cdot factor = 2.72\,m$

$\lambda_z := 3.46 \cdot \frac{l_0}{z} = 17.111$ $\lambda_y := 3.46 \cdot \frac{l_0}{y} = 18.822$

$A := 0.7$

A.s taken from end of calculation:

$\omega := \frac{A_s \cdot f_{yd}}{y \cdot z \cdot f_{cd}} = 0.152$ $B := (1 + 2 \cdot \omega)^{0.5} = 1.142$

Worst case for braced structures $C := 1.7$

$$n := \frac{N_{Ed}}{y \cdot z \cdot factor \cdot f_{cd}} = 1.016$$

$$\lambda_{lim} := 20 \cdot \frac{A \cdot B \cdot C}{n^{0.5}} = 26.948$$

Column not slender since λlim > λy and λz

LOADS FROM SLAB ON LEVEL 2

$$UDL_{24} := z_{24} \left(1.35 \cdot g_k + 1.5 \cdot q_k\right) = 4.856 \cdot \frac{kN}{m}$$

$$UDL_{45} := z_{45} \left(1.35 \cdot g_k + 1.5 \cdot q_k\right) = 70.763 \cdot \frac{kN}{m}$$

$$UDL_{45min} := z_{45} 1.5 \cdot g_k = 57.375 \cdot \frac{kN}{m}$$

$$UDL_{z24} := y_{24} \cdot \left(1.35 \cdot g_k + 1.5 \cdot q_k\right) = 169.275 \cdot \frac{kN}{m}$$

$$UDL_{z45} := y_{45} \cdot \left(1.35 \cdot g_k + 1.5 \cdot q_k\right) = 104.063 \cdot \frac{kN}{m}$$

MOMENTS FROM STRUCTURE ON LEVEL 3

In Y direction the total moment is given by moment created by horizontal structure in level 3 and axial load:

$M_{str.y} := 21kN \cdot m$

$$\boxed{M_{Edyy} := M_{str.y} + N_{Ed} \cdot \frac{l_0}{400} = 53.31 \cdot kN \cdot m}$$

In Z direction the moment from the flat slab is limited to 120kNm since F4 is an edge column. Worst case, with wind in the same direction was considered

$M_{str.z} := 120kN \cdot m$ $M_w = 44.118 \cdot kN \cdot m$

$$\boxed{M_{Edzz} := M_{str.z} + M_w + N_{Ed} \cdot \frac{l_0}{400} = 196.428 \cdot kN \cdot m}$$

DESIGN REINFORCEMENT

$f_{ck} := 30MPa$ $\qquad f_{yk} := 500\frac{N}{mm^2}$

$\boxed{\frac{M_{Edzz}}{y \cdot z^2 \cdot f_{ck}} = 0.043}$ $\qquad \boxed{\frac{N_{Ed}}{y \cdot z \cdot f_{ck}} = 0.576}$ $\qquad z = 0.55m$

$d_2 := cover + dia_{link} + dia_{bar} \cdot 0.5 = 0.064m$

$\frac{d_2}{z} = 0.116$

From chart As F.yk /bhf.yk (Harrison2007a, p39)

$ratio := 0.11$

$\boxed{A_s := \frac{ratio \cdot y \cdot z \cdot f_{ck}}{f_{yk}} = 1.815 \times 10^3 \cdot mm^2}$

8 bars H18 (2040 mm^2) $\quad A_s := 2040mm^2$

Crushing load of a truly loaded column
may be taken as: (Mosley, p.267)

$N_{ud} := 0.567 \cdot f_{ck} \cdot y \cdot z + 0.87 \cdot A_s \cdot f_{yk} = 5.565 \times 10^3 \cdot kN$

$N_{Ed} = 4.751 \times 10^3 \cdot kN$

Minimum area of steel needs to be above 0.002Ac

$As_{min} := 0.002 \cdot y \cdot z = 550 \cdot mm^2$ $\qquad \frac{A_s}{z \cdot y} = 0.742 \cdot \%$

DESIGN FIRE RESISTANCE

check that effective length <3m

$l_0 := (3.1m - 0.3m) \cdot .85 = 2.38m$ OK

check e< e.max $\qquad e := \frac{M_{Edyy}}{N_{Ed}} = 0.011m$

$e_{max} := 0.15 \cdot z = 0.083m$ OK

Check reinforcement <4% $\qquad \frac{A_s}{z \cdot y} = 0.742 \cdot \%$ OK

CHECK BIAXIAL BENDING

$N_{Ed} = 4.751 \times 10^6 N$

$e_z := \frac{M_{Edzz}}{N_{Ed}} = 41.34 \cdot mm$ $\qquad e_y := \frac{M_{Edyy}}{N_{Ed}} = 11.22 \cdot mm$

$\frac{e_y}{e_z} = 0.271$ between 0.2 and 5, design for biaxial bending needs to be checked

From chart obtain ratio of As f.yk/bhf.ck

Revise reinforcement to 8 H22 (3040 mm^2)

$A_{s\_eff} := 3040mm^2$ $\qquad A_{s\_eff} \cdot \frac{f_{yk}}{y \cdot z \cdot f_{ck}} = 0.184$

$\frac{N_{Ed}}{z \cdot y \cdot f_{ck}} = 0.576$ $\qquad \frac{d_2}{y} = 0.128$

$\boxed{f_{chart} := 0.05}$

$M_{Rd} := f_{chart} \cdot y \cdot z^2 \cdot f_{ck} = 226.875 \cdot kN \cdot m$

$N_{Rd} := z \cdot y \cdot f_{cd} \cdot 0.85 + A_{s\_eff} \cdot f_{yd} = 5.688 \times 10^3 \cdot kN$

Value for a obtained from chart (Harrison2007a, Table 5):

$\frac{N_{Ed}}{N_{Rd}} = 0.835$ $\qquad a := 1 + \left(\frac{N_{Ed}}{N_{Rd}} - 0.1\right) \cdot \frac{0.5}{0.6} = 1.613$

$\boxed{\left(\frac{M_{Edzz}}{M_{Rd}}\right)^a + \left(\frac{M_{Edyy}}{M_{Rd}}\right)^a = 0.889}$ *Acceptable*

CHECK REINFORCEMENT

As minimum $\qquad \frac{0.10N_{Ed}}{f_{yd}} = 1.425 \times 10^3 \cdot mm^2$

Reinforcement to area ratio $\qquad \frac{A_{s\_eff}}{y \cdot z} = 1.105 \cdot \%$

Reinforcement is above 0.2% and below 4% and above As minimum.

TRANSVERSE REINFORCEMENT
Ref. EC2 9.5.3&NA

Diameter of bar shoudl exceed 6m of 1/4 of reinforcement bars

$dia_{min} := dia_{bar} \cdot 0.25 = 5.5 \cdot mm$ $\qquad$ **Diameter 6mm**

Spacing given by the minimum

$20 \cdot dia_{bar} = 440 \cdot mm$

$y = 500 \cdot mm$ $\qquad$ **H6 @ 400 mm**

$400mm = 400 \cdot mm$

$z \cdot y \cdot f_{cd} \cdot 0.85 = 4.675 \times 10^6 N$

5. Horizontal loading, shear walls

Horizontal loading is mostly caused by wind. A preliminary analysis on the tensile stress applied to the shear wall was done in three point of the building. it was possible to have a first estimate of the amount of shear wall needed in different part of the building.

Figures 5.1, 5.2 and 5.3 show a possible layout for shear walls. As previsouly stated, upper levels, used as a penthouse are keps as free of vertical elements as possible. In the lower levels (2-18) partitions between apartmetments were assumed as shear walls. This seemed a reasonable assumption considering that the layout of the different units is unlikely to change.

A summary of result is show in Table 5.1 and calculation for one of the three levels in the following page.

Figure 5.1: structural diagram with shear walls on level 18

Level	Wall	N_{Ed} (kN)	DF	"Wk (kNm)	σ (MPa)	As (mm2)	Reinf
17	D5	1254	49%	3397	10	7497	H25@125 x2
10	D5	1591	4%	2652	8.4	5853	H24@150 x2
2	D5	2600	4%	4230	12.2	8400	H28@125 x2

An additional source for horizontal loading is created by the slanted structure on grid 2. In Figure 5.4 shows a diagram visualizing the sub-frame on level 2. The horizontal component of the axial loading acting on the column will produce a force F= axial loading * cos(77 °).
This force should be considered in the analysis of the vertical structure on grid 4 especially at the point D4 where the beams meet the vertical shear wall.

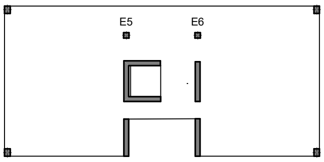


Figure 5.1: structural diagram with shear walls on level 18

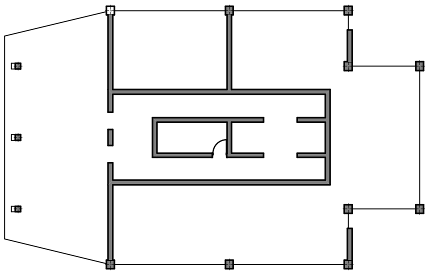


Figure 5.2: structural diagram with shear walls on level 10

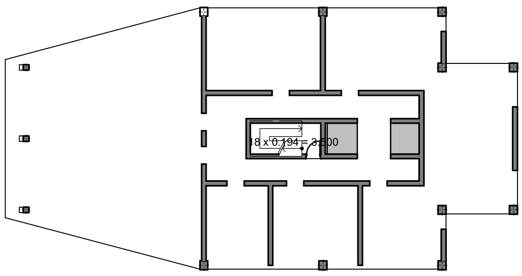


Figure 5.3: structural diagram with shear walls on level 3

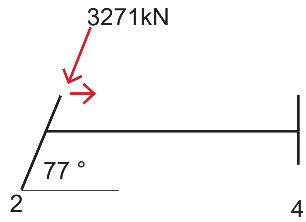


Figure 5.4: diagram showing the slanted column on

SHEAR WALL (D5) ON LEVEL 2

Axial load:

Tributary area on supported by the wall: $A := 8m^2$

Load per m2 $g_k := 7.5\frac{kN}{m^3}\cdot .3m = 2.25\times 10^3\,Pa$

$Floors := 20$

Wall dimensions $t := 0.3m$ $L := 2.5m$

Load from wall:

$$g_{wall} := L\cdot t\cdot 3.5m\cdot Floors\cdot 25\frac{kN}{m^3} = 1.313\times 10^3\cdot kN$$

Load from variable load:

$$q_k := 5\frac{kN}{m^2}\cdot A\cdot Floors = 800\cdot kN$$

Permanent actions unfavourable: $\gamma_{pu} := 1.35$

Leading variable actions unfavourable: $\gamma_{vu} := 1.5$

Accompaigning variable actions unfavourable: $\gamma_{au} := 1.5\cdot 0.7$

$$N_{Ed} := \left(g_{wall} + A\cdot g_k\right)\cdot\gamma_{pu}\ \ldots + \gamma_{au}\cdot q_k = 2.636\times 10^3\cdot kN$$

<u>Moment generated by wind action:</u>

Upper part, levels 18-22

$$P := 1.588\frac{kN}{m^2}$$

$$A_w := 4\cdot 3.5m\cdot 19.8m = 277.2m^2$$

$$L_{wind} := 68m$$

$$W_{k1} := P\cdot A_w\cdot L_{wind} = 2.993\times 10^4\cdot kN\cdot m$$

Lower part, levels 2-18

$$P := 1.588\frac{kN}{m^2}$$

$$A_w := 1415m^2$$

$$L_{wind} := 31.5m$$

$$W_{k2} := P\cdot A_w\cdot L_{wind} = 7.078\times 10^4\cdot kN\cdot m$$

$$W_{k\_tot} := W_{k1} + W_{k2} = 1.007\times 10^5\cdot kN\cdot m$$

Moment acting on the column:

Portion of moment take $f := .028$

$$W_k := W_{k\_tot}\cdot f = 2.82\times 10^3\cdot kN\cdot m$$

SHEAR WALLS

Ultimate axial load $N_{Ed} = 2.636\times 10^3\cdot kN$

Ultimate in-plane moment $W_k = 2.82\times 10^3\cdot kN\cdot m$

Length of the wall $L := 2.5m$

Thickness of the wall $t_{wall} := .3m$

Maximum applied tensile stress:

$$M := W_k\cdot\gamma_{vu} = 4.23\times 10^3\cdot kN\cdot m$$

$$\sigma := \frac{N}{L\cdot t} + \frac{M}{\left(\frac{t_{wall}\cdot L^2}{6}\right)} = 13.536\cdot MPa$$

Area of reinforcement required:

Length of wall in tension $L_t := 1m$

$f_y := 500MPa$

$$A_{min} := \frac{\sigma\cdot L_t\cdot t_{wall}}{0.87\cdot f_y} = 9.335\times 10^3\cdot mm^2$$

<u>H28 @125 (4928 mm2) on both face of the wall</u>

Ultimate compressive load should be less than:

Area of concrete per unit length of wall $A_c := 1m\cdot t = 3\times 10^5\cdot mm^2$

$f_{cu} := 30MPa$

$$F_c := 0.35f_{cu}\cdot A_c + 0.67f_y\cdot A_{min} = 6.277\times 10^3\cdot kN$$

6.Health and Safety

The CDM 2015 regulation requires the designer to (amongst other duties):
-avoid foreseeable risks
-provide adequate information about significant risk
-co-ordinate their work with others

At the moment, being at a conceptual level of the design and with scarce site information, it becomes difficult to describe specific solutions. The main activities that would be carried are therefore described.

The first obligation, regarding the reduction of risks, can be addressed during different phases of the project. During pre-construction phase, together with client, gather available information related to health. Sources may be previous health and safety files, local conditions and regulations, assessment of the site. Fill gaps and provide information to designer and contractors. The preparation of a Construction Method Statement and a Construction Phase Plan (by principal contractor) will help in better understanding the construction process and related risks. These documents will inform on possible site rules and arrangements and requirements (CDM2015, Schedule 2) to be taken to ensure safety.

During the design phase, evaluate and foresee if particular details or moments during the construction may cause an hazard to workers. Possible hazards may be created by the movement of heavy pieces, use of dangerous tools, working at height, handling hazardous materials (see also CDM 2015, Schedule 3). Consider possible solutions by assessing alternatives and applying the general principles of preventions (CDM215, Appendix).

The main document to provide information on risk is the Health and Safety File (CDM 2015, Appendix 4). This document shall be prepared by the principal designer. Feedback and coordination with client and contractors is essential.

The third obligation, to be fulfilled, requires the creation of a health and safety file. It is essential to involve client and contractors in the process and to make sure workers are aware of its content. It is important to establish a line of communication across client, designers, contractors and workers. It may be noted that a project of this size will involve numerous contractors and many of them will join the project at a later stage. The transmission of information should also consider this. Additionally, the file should be thought as a “live document” that can be updated and adjusted during the process. Feedback from workers is essential therefore a system supporting it should be created.

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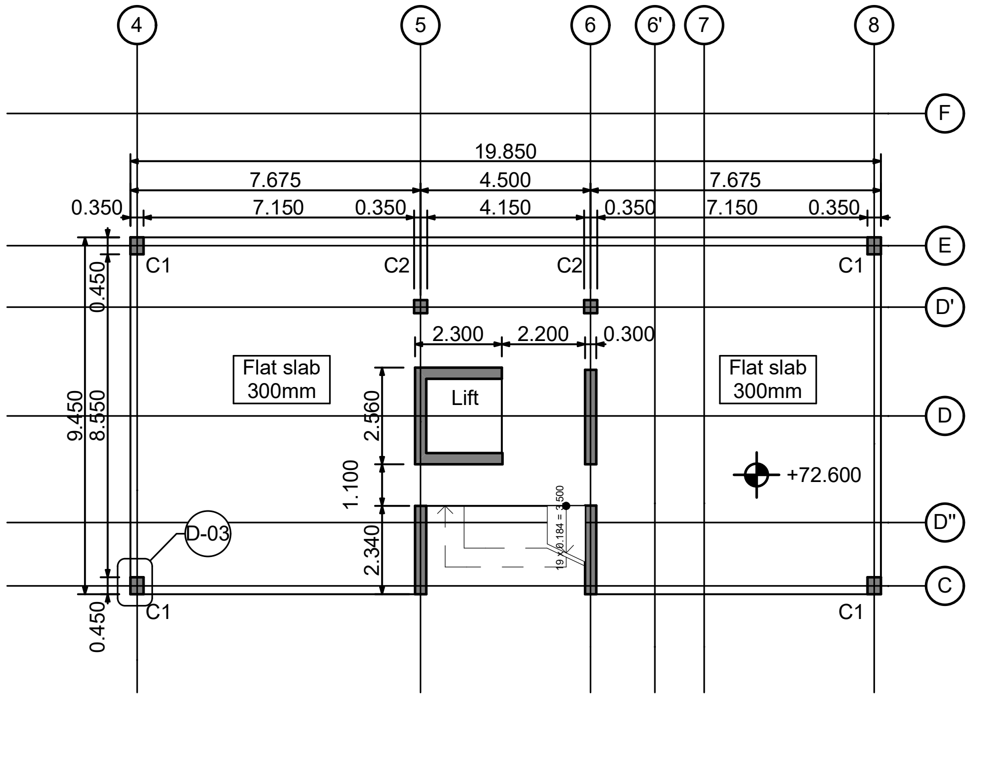
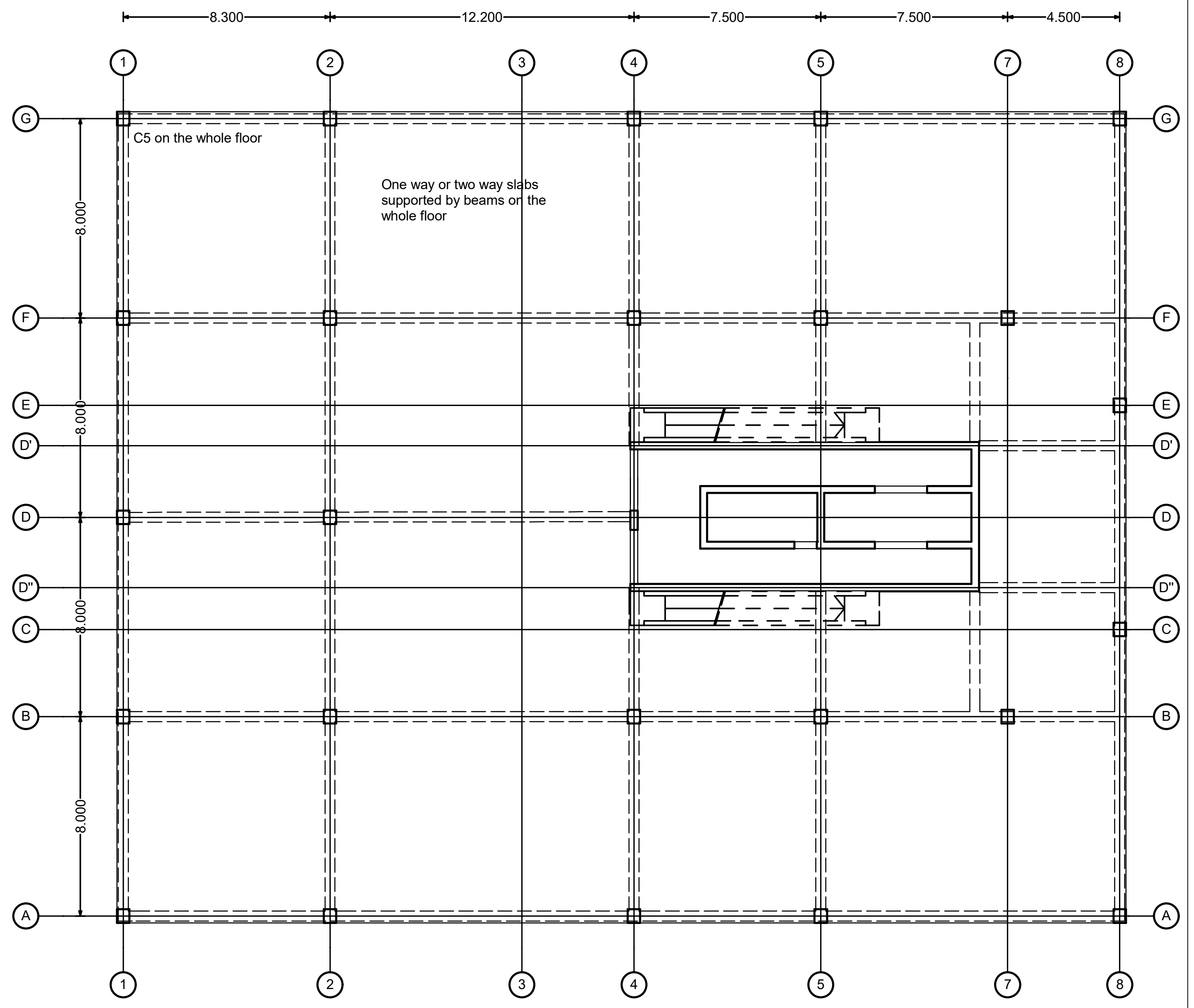
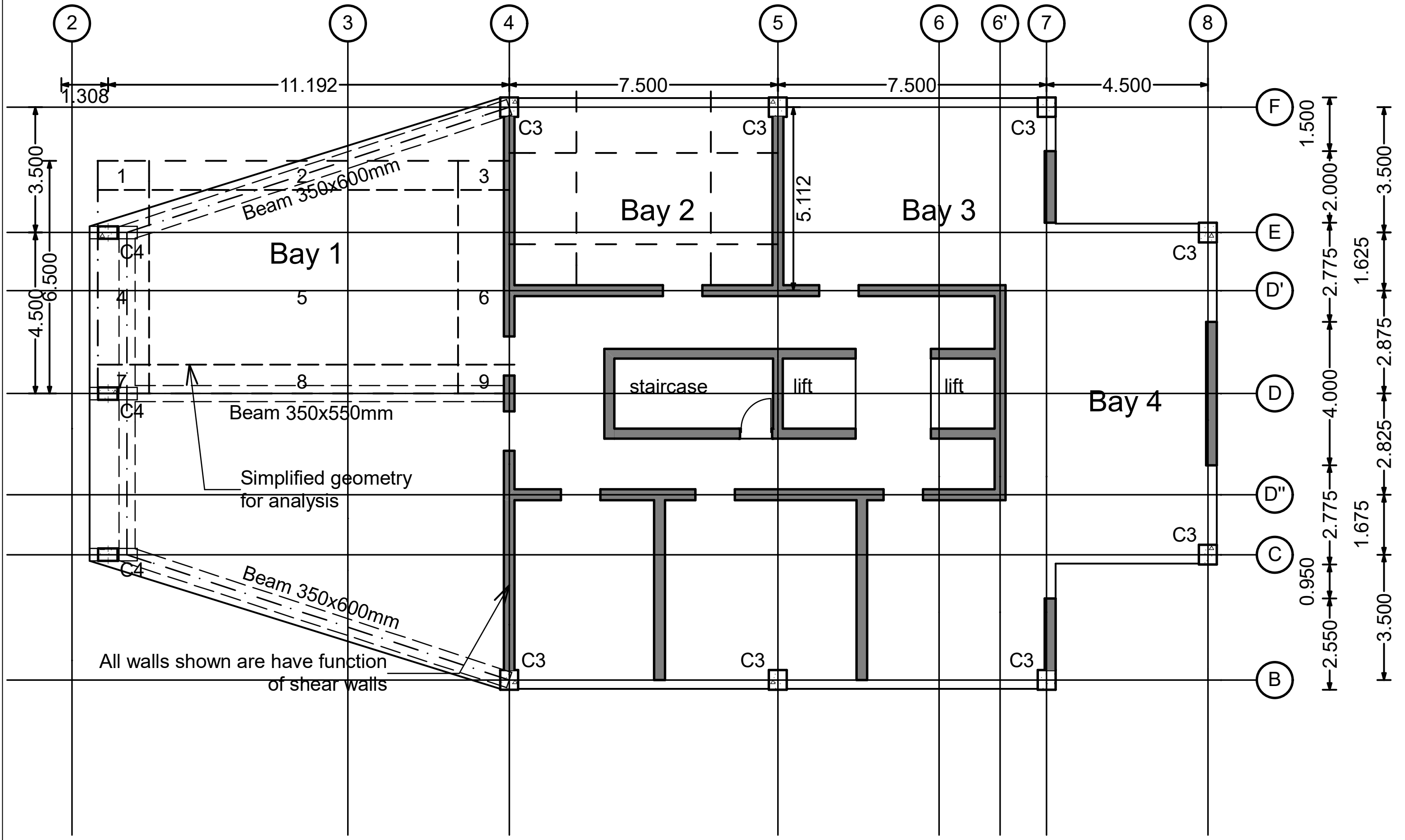


20. Plan L18-22 1:200



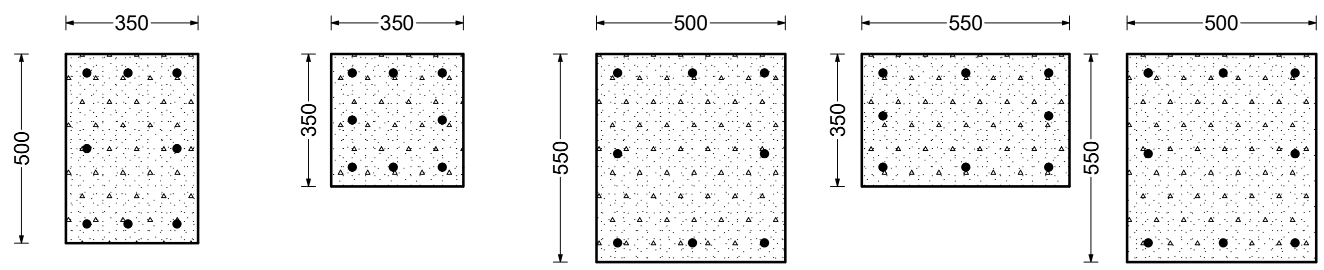
0. Ground Floor 1:2

Note: the analysis for Bay 2 was simplified: a support at each of the fours conrner was assumed



Level 3 1:200

Column type C1	Column type C2	Column type C3	Column type C4	Column type C5
8 H14	8 H12	8 H22	8 H28	8 H20

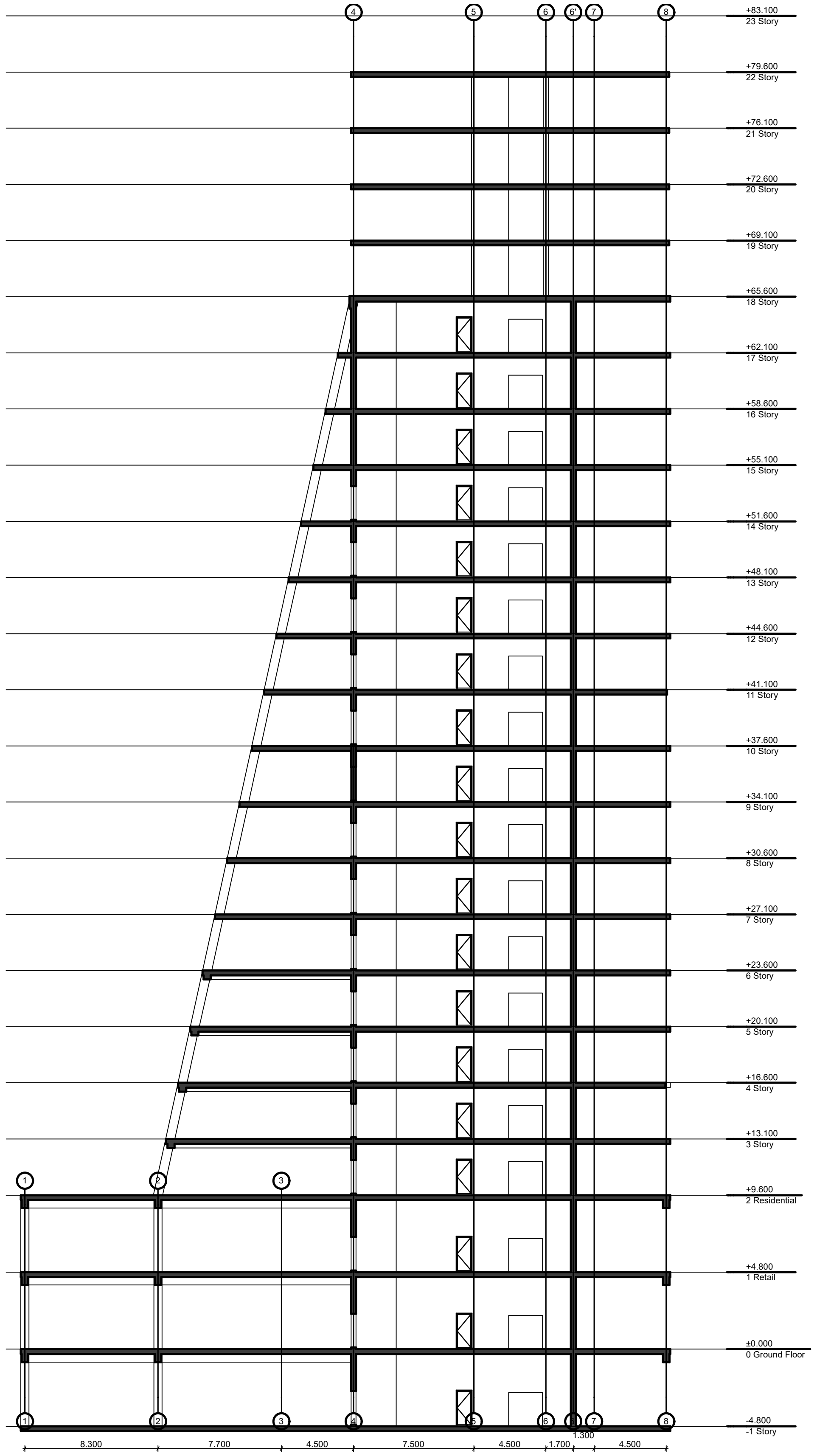


Reinforcement and should be optimized every 3/4 floors

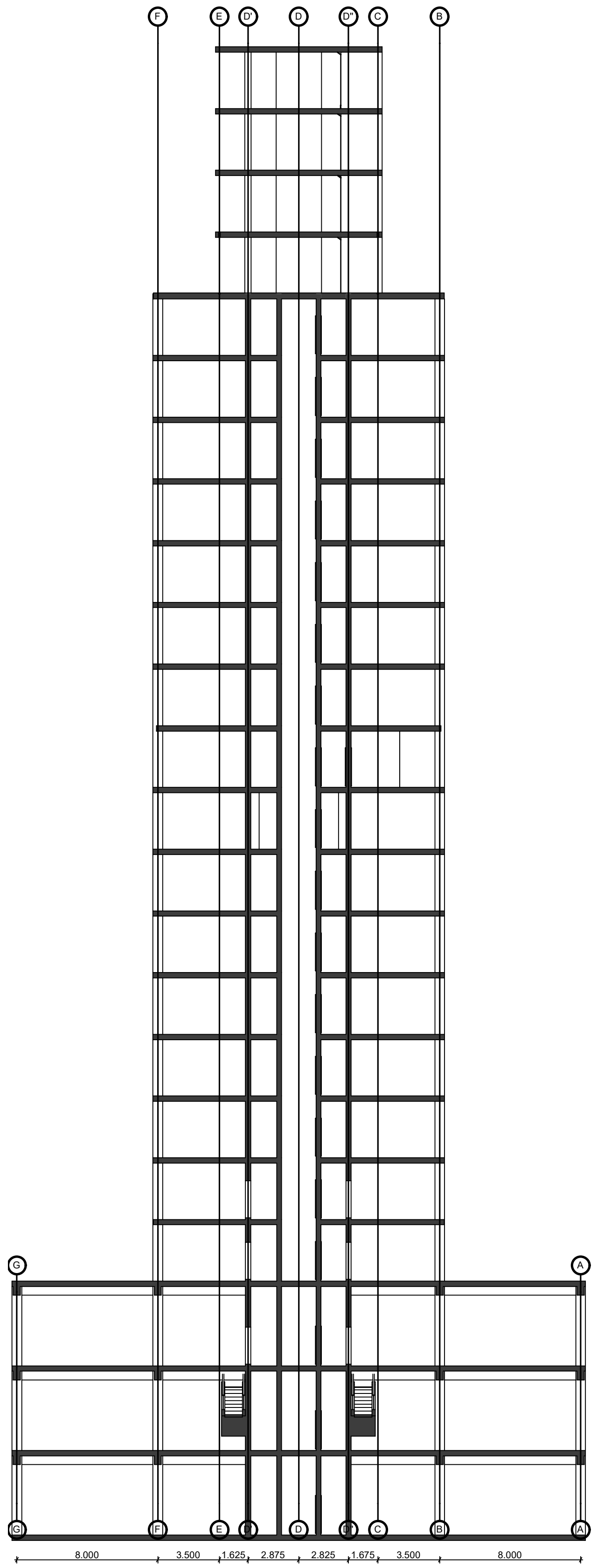
Internal columns may have less reinforcement

Drawing Scale
1:200, 1:250, 1:20

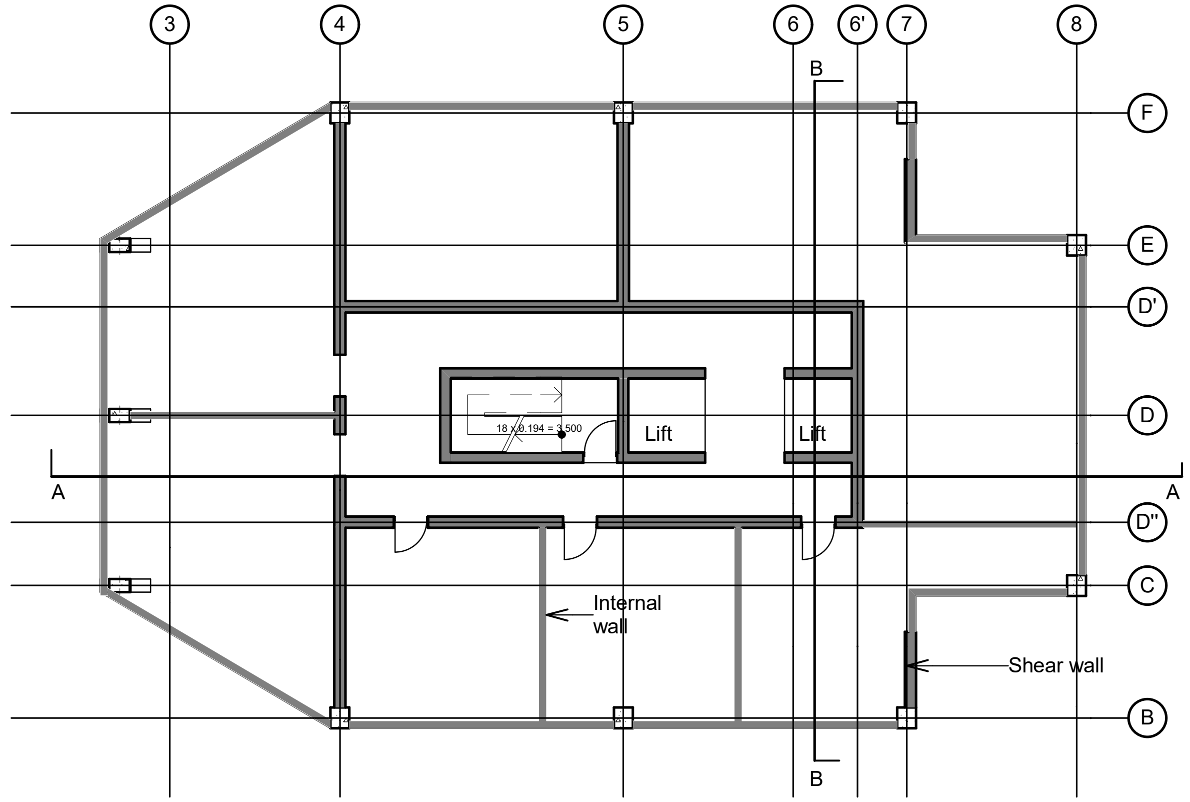
Layout ID
Structural plans



AA Building Section 1:400

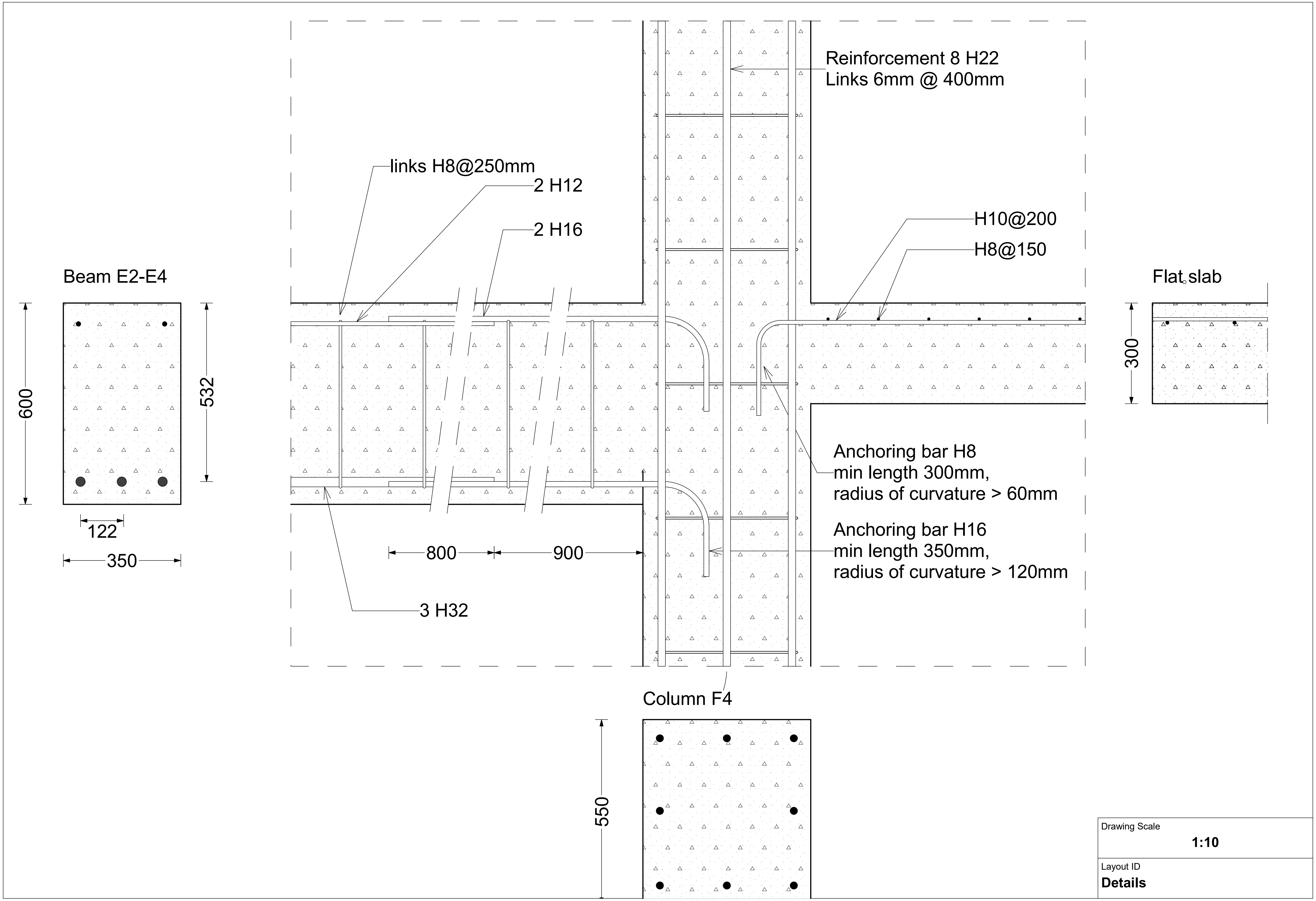


BB Building Section 1:400



10. Layout 1:200

Drawing Scale	**1:400, 1:200**
Layout ID	**A3:2**



Beam E2-E4
600
532
122
350
links H8@250mm
2 H12
2 H16
Reinforcement 8 H22
Links 6mm @ 400mm
H10@200
H8@150
Flat slab
300
Anchoring bar H8
min length 300mm,
radius of curvature > 60mm
Anchoring bar H16
min length 350mm,
radius of curvature > 120mm
800
900
3 H32
Column F4
550
Drawing Scale
1:10
Layout ID
Details