plateaus

a tectonic alteration

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Report content

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VISUALISATIONS

The project report was focused on the theoretical method of Tectonic Alteration and testing it in praxis. The report gave a picture of how the method was used and how it affected the design process. The design in it self was described through a series of iterations, in order to show how the knowledge learned was used and helped shape the project. The project report gives an understanding of how the design works on a conceptual level in regards to different considerations.

This presentation folder is then the visual representation of the design that was developed during the project and it will show what the building could look like, if it was to be erected. The visualisations are meant to give an impression of the spaces in the building, their character, atmosphere and materials.
These are then accompanied by technical drawings of details, plans, sections and elevations, to give a more elaborate understanding of the final design for the building.

The exterior of the building has a subtle shift from new in old by change in material colour and geometry, as the floor plates extend outside the column grid, breaking the vertical mass and guiding the view towards the plateaus.
The facade is pulled back to the middle of columns but still exposed to sun and heavy rain that over time will result in a silver gray surface. This will from distance blend into the concrete environment but as one approaches the building, the less exposed areas below the deck and windows will reveal the warm wood and its true construct.

The forest is a space playing on the balance of inside outside, particularly inviting in times of heavy rain to seek shelter. One should look through and be able to see that the space is not inclosed however the planting should break the wind and obstruct the view in eyeheight, creating smaller spaces inside. By allowing a view to the staircase it is intended to guide them to ascend in the building revieling prospect of the building, then the area and finally the city.

Render from Alexander Foss Gade

Render from parkinglot

Render of the forest

Render of gallery


Render of apartment interior

DRAWINGS

Apartment plans and sections

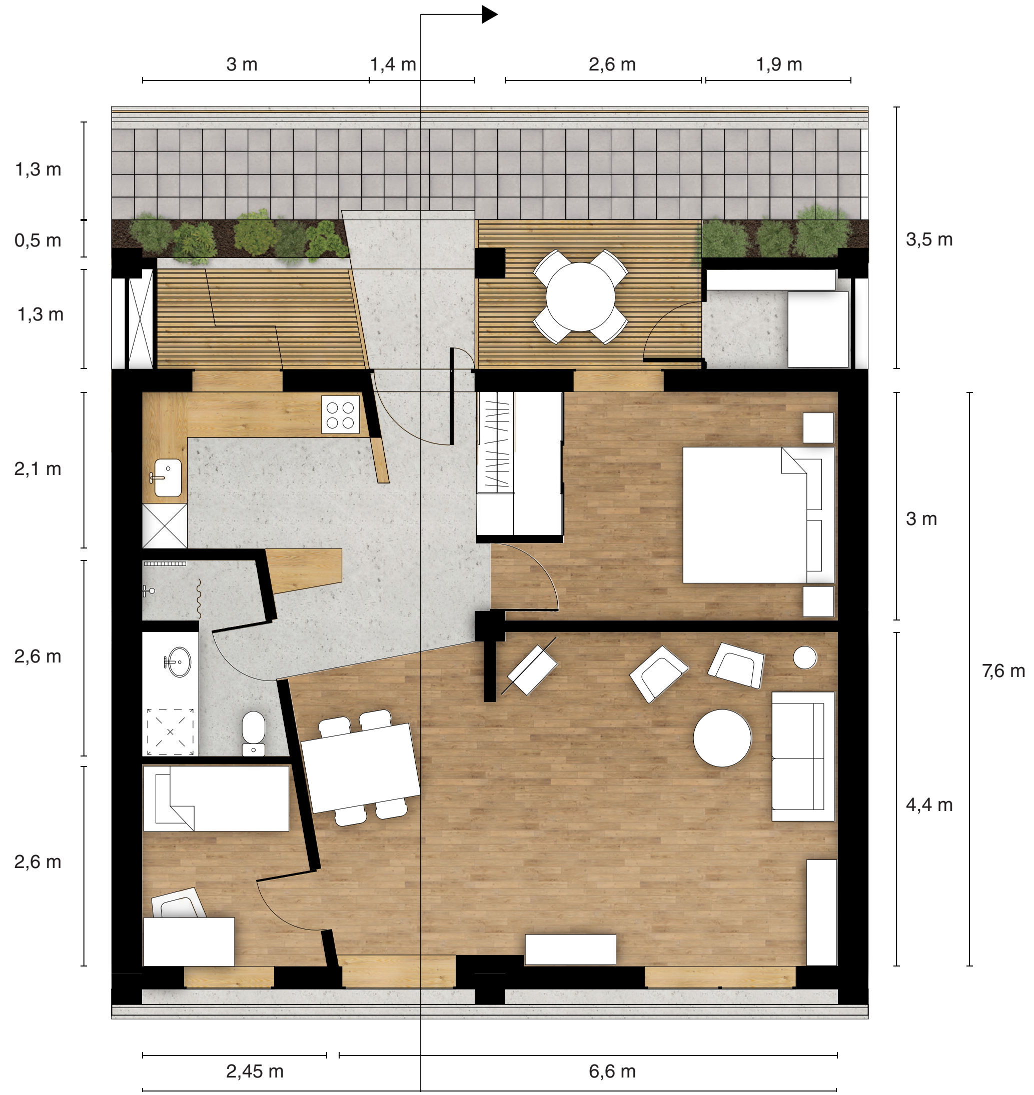
The galleries are seperating access and stay, by the concrete disc connected between the columns. The planting boxes outside allows for the dwellers to control the privacy of their porch through vegation. The porch is covered in non exposed larch battens creating a warm sheltering semi private space, coherent but with a contrast to the exposed skin towards the public.

The small wood covered porch is seperated in a small private space with a storage bench and a open deck towards the gallery. The entrence is marked by a concrete ramp, that seperates porch, and flows into the apartment defining the exposed interior of entrence, kitchen and bathroom and for easy maintenance while the rest of apartment has woodflooring.

Apartment section 1:100

Spacial composition

Apartment GFA	78,7 m2
Apartment NSA	67,6 m2
Entrence	4,7 m2
Kitchen	9,5 m2
Bathroom	4,6 m2
Livingroom	29,7 m2
Master bedroom	13,1 m2
Bedroom	6 m2
Porch	6 m2
Shed	2,5 m2



Apartment plan 1:100

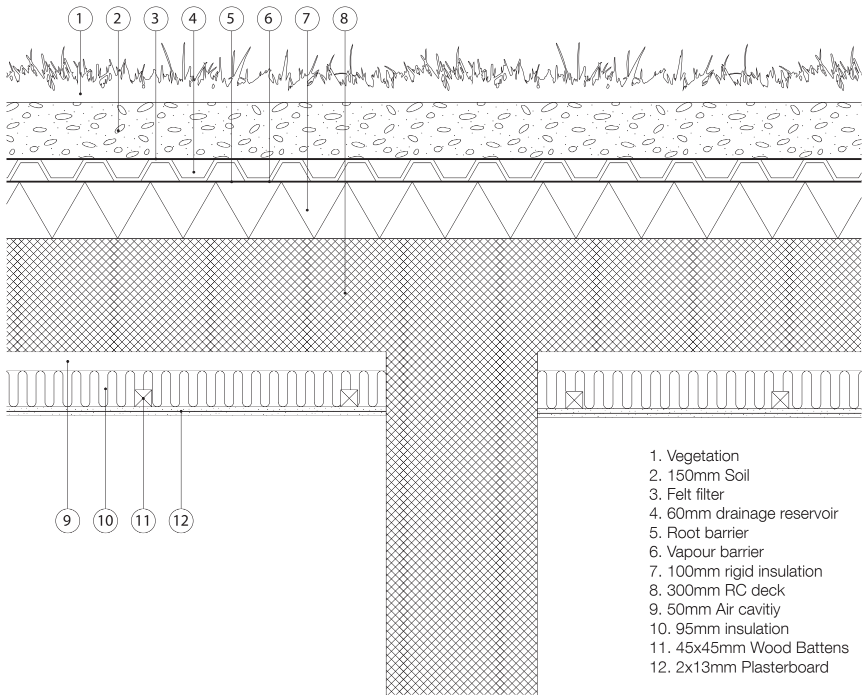
Details

The three details are showing the apartments relation to the structure.

The detail of the roof detail is taken from the center column in the apartment showing the greenroof construction.

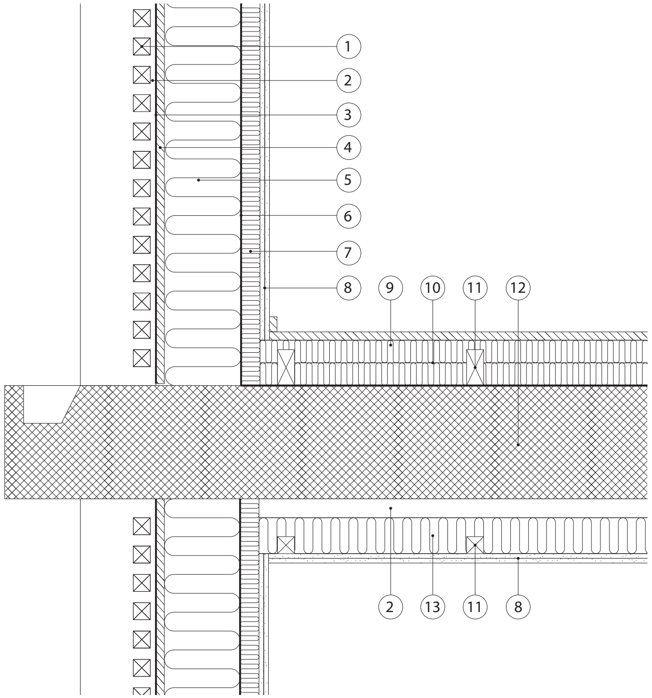
The deck and facade detail is showing the floor, ceiling and facade construction of the apartments.

The third is showing the seperation between the deck above the forest, and the column shoe minimizing the coldbridge to the apartment columns. The column shoe is structurally only taking the vertical loads as the disc between the apartments are considered to stabilise the structure above in terms of lateral loads.



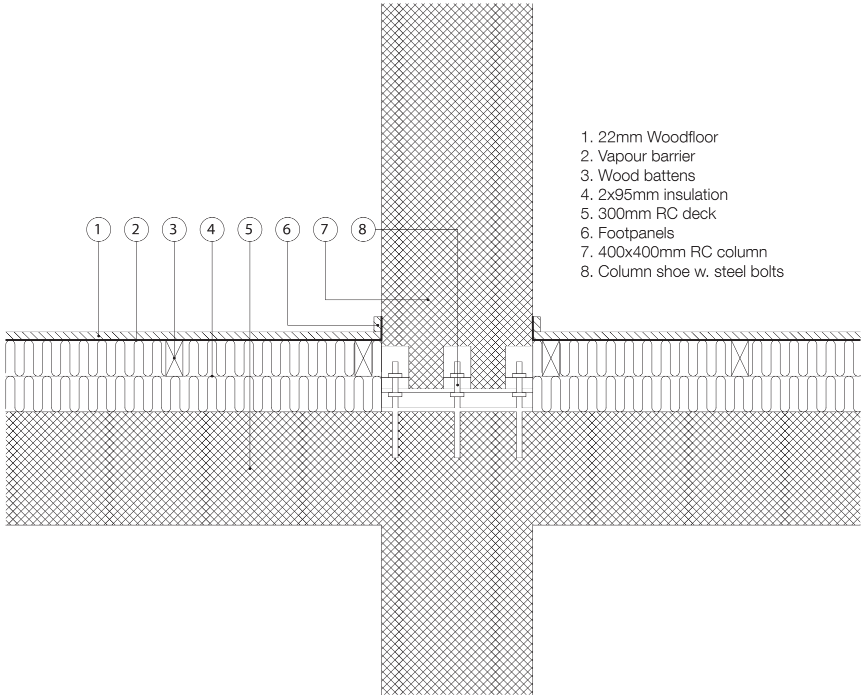
Roof detail 1:20

1. 45x45mm Larch battens
2. Air cavity
3. Felt - wind barrier
4. 22mm Plywood board
5. 140mm Insulaton
6. Vapour barrier
7. 50mm insulation
8. 2x13mm Plaster board
9. 22mm Larch Flooring
10. 2x60mm Insulation
11. Wood battens
12. 300mm RC deck
13. 95mm Insulation



Apartment facade 1:20

1. 22mm Woodfloor
2. Vapour barrier
3. Wood battens
4. 2x95mm insulation
5. 300mm RC deck
6. Footpanels
7. 400x400mm RC column
8. Column shoe w. steel bolts



Column shoe 1:20

APPENDIX

Robot analysis

Structure

The structural analysis carried out through the design process can be divided into the initial analysis and final calculation. The first analysis concerning the main deck and its behaviour with different location of column supports through robot analysis.

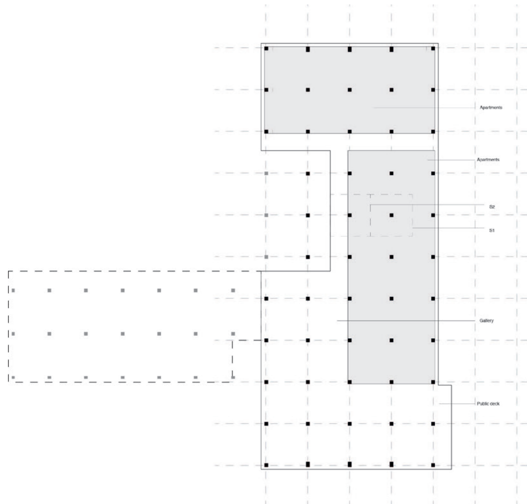
The second part consists of selected calculations to verify deck, cantilevered gallery and dimensioning of columns. Since actual documentation of these are quite complex and the documentation of structure is outside the scope of the project - the columns have been calculated as centrally loaded columns, and the deck as a beam with simple support.

These simplifications do not correspond with the reality, as the columns do not take the momentum created from the fixation of in situ joining of column and deck.

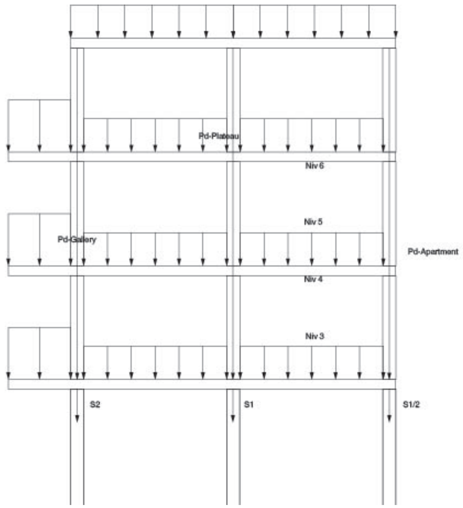
Column location

The initial idea was to create a forest of columns by scattering the columns throughout the hall, however by doing so it raised questions regarding the structural behaviour as the loads from the decks and apartments were offset. From the robot analysis we see that by locating the columns offset from the line loads above, they are generating large momentum in the deck, particularly above the columns and below the loads as one might expect.

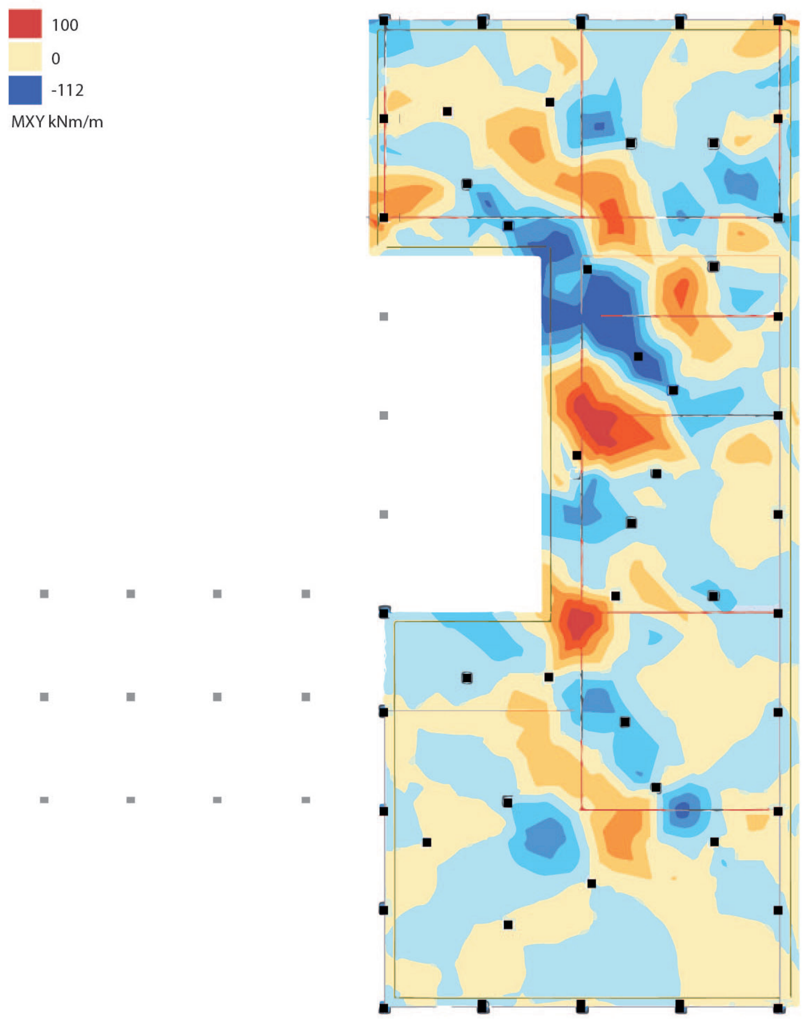
By locating the columns along the line load, the momentum is primarily located in the centre of a span of the line loads. Which lead to the concept of grid-space columns, working to distribute the loads more evenly and theoretically avoiding that the columns would be eccentrically loaded.



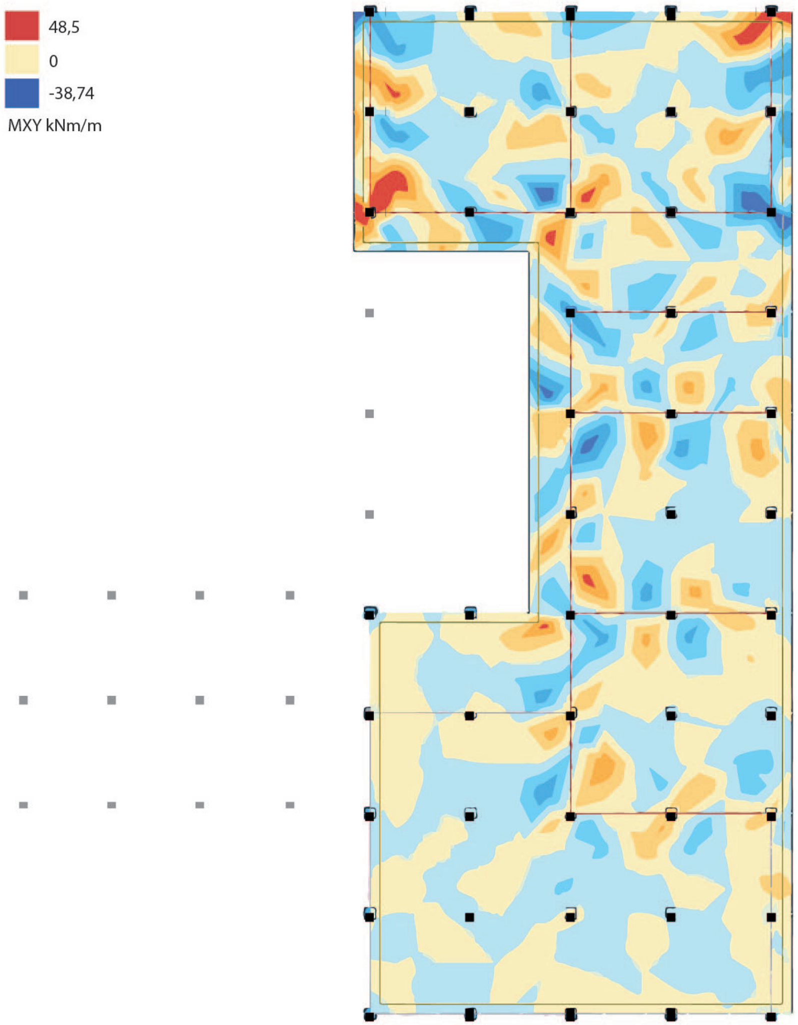
Load area diagram “plan”



Load area diagram “section”



Moment distribution in deck with “random” columns

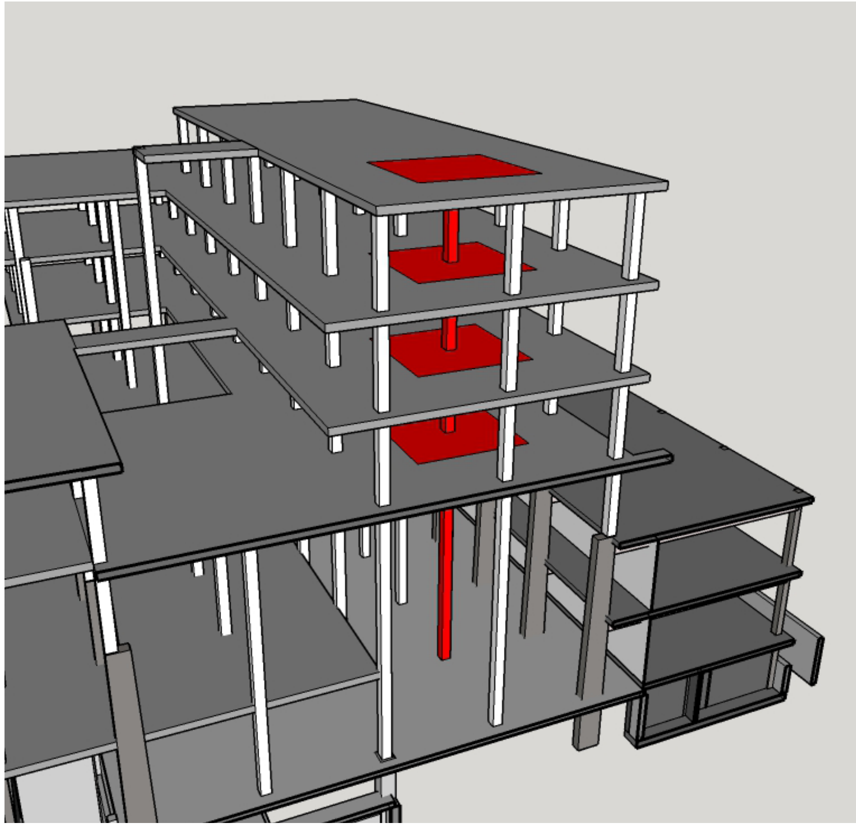


Moment distribution in deck with “grid” columns

Column load calculation

In order to dimension the lower columns in the hall, the sum of all loads must be found. This is done by looking at the load area that it has to handle, and the loads that affect that area.

Having the column grid laid out will to some extend make the loads more evenly distributed and therefore it is assumed that the columns can be seen as centrally loaded instead of eccentrically which would be the actual case. Finding the load combination is the first step and then dimensioning the column accordingly.



Load combination from each floor, transferred down to the ground

Load calculation

Conctrol class: CC3 (DS/EN 1990-DK-NA 2010-5)
Building in multiple storeys more than 12 meters
Exposure class: Aggressive (DS/EN 1992-1-1)
Deck thickness is set at 300mm
Soil – 250 *mm*

Dead load:

$$g_{deck} = h \cdot w \cdot l \cdot \rho \cdot g \ \rightarrow 0,\ 3\ m \cdot 1\ m \cdot 1\ m \cdot 2,\ 4 \frac{kg}{m^2} \cdot 9,\ 82 \frac{m}{s^2}$$

$$= 7,\ 07\ \frac{kN}{m^2}$$

$$g_{soil} = 0,\ 25\ m \cdot 7,\ 5 \frac{kN}{m^2} \cdot 9,\ 82 \frac{m}{s^2} = 3,\ 75 \frac{kN}{m^2}$$

$$g_{column} = h \cdot w \cdot l \cdot \rho \cdot g \ \rightarrow$$

$$3,\ 2\ m \cdot 0,\ 4\ m \cdot 0,\ 4\ m \cdot 2400\ \frac{kg}{m^3} \cdot 9,\ 82\ \frac{N}{kg} = 12\ kN$$

h = height, w = width, l = length, ρ = density*, g = a gravity*

Variable loads:

Live load:

$$q_{k,\mathbf{in}} = 1,\ 5\ \frac{kN}{m^2}\ (EC-1\ table\ \ 6.2\ Cat.\ A1)$$

$$q_{k,\ o} = 5\ \frac{kN}{m^2}\ \ (EC-1\ table\ \ 6.2\ Cat.\ B-C1)$$

Snow load:

$s = \mu_i \cdot C_e \cdot C_t \cdot s_k \quad \rightarrow \quad s = 0,8 \cdot 1 \cdot 1 \cdot 0,9 \frac{kN}{m^2} = 0,72 \frac{kN}{m^2}$

μ_i – *shape coefficient*

C_e – *exposure coefficient*

C_t – *thermal coefficient*

s_k – *characteristic value of snow load on the ground*

(acc. DS/EN 1991-1-3 FU:2010)

Column Load area:

$A = 23\ m^2$

$G_{kj,\ inf} = n_{floors+1} \cdot g_{deck} \cdot A + n_{floors} \cdot g_{column}$

$= 4 \cdot 7,07 \frac{kN}{m^2} \cdot 23\ m^2 + 3 \cdot 12\ kN = 686,44\ kN$

$G_{kj,\ sup} = g_{soil} \cdot A = 3,75 \frac{kN}{m^2} \cdot 23\ m^2 = \underline{86,25\ kN}$

$Q_{snow} = s \cdot A = 0,72 \frac{kN}{m^2} \cdot 23\ m^2 = \underline{16,56\ kN}$

$Q_{live} = 3 \cdot q_{k,\ i} \cdot A + q_{k,\ o} \cdot A$

$= 3 \cdot 1,5 \frac{kN}{m^2} \cdot 23\ m^2 + 5 \frac{kN}{m^2} \cdot 23\ m^2 = \underline{218,5\ kN}$

Where:

A – *load area on each floor*

n – *number of floors*

Q_{live} *is considered dominant of variable loads*

Ultimate limit state (ULS):

Load combination:

EQU(*teknisk ståbe table* 4.1)

$P_d = K_{FI} \cdot \gamma_{Gj,\ sup} \cdot G_{kj,\ sup} + \gamma_{Gj,\ inf} \cdot G_{kj,\ inf}$

$+ K_{FI} \cdot \gamma_{Q,\ 1} \cdot Q_{k,\ 1} + \gamma_{Q,\ i} \cdot \psi_{Q,\ i} Q_{k,\ i}$

$P_d = 1,1 \cdot 1,1 \cdot 86,25\ kN + 0,9 \cdot 686,44\ kN$

$+ 1,1 \cdot 1,5 \cdot 218,5\ kN + 1,1 \cdot 1,5 \cdot 0,5 \cdot 16,56\ kN$

$P_d = 1110,0075\ kN$

$K_{FI} = 1,1$ (*Teknisk ståbi* 4.2.3)

Partial Coefficients

$\gamma_{Gj,\ sup} = 1,1$

$\gamma_{Gj,\ inf} = 0,9$

$\gamma_{Q,\ 1} = \gamma_{Q,\ i} = 1,5$

Reduction factor

$\psi_0 = 0,5$

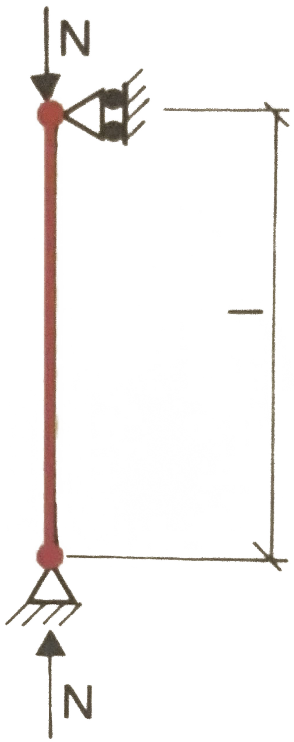
Diagram of a centrally loaded column

Concrete column

Centrally loaded column

One of the columns in the warehouse must be verifyed in terms of load capasity. In the previous calculation it was found, that the column has to support $P_d = 1110,\ 0075$ kN

The ultimate limit state is bound by calculating the load capacity for a centrally loaded concrete column N_{cr}



<u>Data given:</u>

Dimension - 400 x 400 mm

Height - 10.700 mm

Normally reinforced

Concrete yield strain:	$\varepsilon_{c,\ u} = 0,\ 35\ \%$
Reinforcement strain:	$\varepsilon_y \leq \varepsilon_s < \varepsilon_{uk}$
Concrete compressive strengt:	$f_{ck} = 35\ MPa$
Reinforcement yield point:	$f_{yk} = 550\ MPa$
Steel elasticity module:	$E_{sk} = 2 \cdot 10^5 MPa$
Partial coefficient:	$\gamma_c = 1,\ 45$ − *in situ concrete*
	$\gamma_s = 1,\ 2$ − *reinforcement*
Reinforcement diameter:	4psc. $ø = 25\ mm$ → 4 x Ø25 (Teknisk Ståbi - 5.3.2)

Calculation of the columns compressive strenght and the reinforcements yield strength.

Concrete Compression:

$$f_{cd} = \frac{35\ MPa}{1,\ 45} = 24,\ 1\ MPa$$ (table 5.14 - Teknisk Ståbi)

Steel Tension:

$$f_{yd} = \frac{550\ MPa}{1,\ 2} = 458,\ 33\ MPa$$ (table 5.15 - Teknisk Ståbi)

Elasticity module:(For C25, chosen for simplicity)

$$E_{sd} = \frac{E_{sk}}{\gamma_s} = \frac{2 \cdot 10^5 MPa}{1,2} = 166666,66 \approx 1,6 \cdot 10^5 MPa$$

Elasticity ratio between concrete and steel:

$$\alpha = \frac{E_{sd}}{E_{cd}} = \frac{E_{sd}\varepsilon_{cl}}{f_{cd}} \rightarrow \frac{200 \cdot 2,1}{24,1} = 17,4$$

This means that the elasticity module of the steel is 17 times stronger than that of the concrete.

Where: ε_{cl} – *Concrete yield strain at maximum limit.*
E_{sd} – *elasticity module for reinforcement steel.*
f_{cd} – *compressive strenght for concrete.*

Reinforcement ratio:
According to DS/EN 1992-1-1 the upper limit for reinforcement ratio is 4% and the lower 0,2%. The ratio is therefore calculated.

$$\rho = \frac{A_{sc}}{A_c} = \frac{r^2 \cdot \pi \cdot 4}{b \cdot h}$$

$$\rho = \frac{1963,5\ mm^2}{400\ mm \cdot 400\ mm} \cdot 100\ \% = 1,23\ \%$$

0,2% < 1,23% < 4%

The ratio is therefore within the allowable limit.

Where: A_{sc} – *area of reinforcement section in concrete*
A_c – *area of concrete section*

Slenderness ratio:

$$\lambda = \frac{l_0}{i}\ ,\ i = \sqrt{\frac{I}{A_c}}\ ,\ I = \frac{b \cdot h^3}{12}$$

$$i = \sqrt{\frac{I}{A_c}} = \sqrt{\frac{\frac{b \cdot h^3}{12}}{b \cdot h}} = \sqrt{\frac{b \cdot h^3}{12} \cdot \frac{1}{b \cdot h}} = \sqrt{\frac{h^2}{12}}$$

$$= \frac{h}{\sqrt{12}} = \frac{\sqrt{12} \cdot l_0}{h}$$

$$\lambda = \frac{10.700\ mm \sqrt{12}}{400\ mm} = 92,\ 7$$

Where: l_0 – *effective lenght of column*

i – *section's inertia radius, no regards to reinforcement*

I – *section's moment of inertia in* mm^4

A_c – *section of concrete in mm*

b – *width*

h – *height*

Elasticity module:

$$E_{0\,cr} \leq \begin{cases} 1000\ f_{cd} \\ 0,\ 75 E_{cod} \end{cases}$$

$$E_{0d} = \frac{51.000}{\gamma_m} \cdot \frac{f_{ck}}{f_{ck} + 13}$$

$$E_{0\,cr} \leq \begin{cases} 1000 \cdot 24,\ 1\ MPa \\ 0,\ 75 \cdot \dfrac{51.000}{1,\ 4} \cdot \dfrac{35\ MPa}{35\ MPa + 13} \end{cases}$$

$$E_{0\,cr} \leq \begin{cases} \underline{24100\ MPa} \\ 19921,\ 9\ MPa \end{cases}$$

Where: γ_m – *partial coefficient Teknisk Ståbi table 9.4*

$E_{0\,cr}$ – *critical elasticity module*

$E_{0\,d}$ – *design elasticity module*

E_{cod} – *design elasticity module for concrete*

f_{ck} – *characteristic compression strenght for concrete*

f_{cd} – *design compression strenght for concrete*

Elasticity module:

$$E_{0\,cr} \leq \begin{cases} 1000\,f_{cd} \\ 0,\ 75E_{cod} \end{cases}$$

$$E_{0d} = \frac{51.000}{\gamma_m} \cdot \frac{f_{ck}}{f_{ck} + 13}$$

$$E_{0\,cr} \leq \begin{cases} 1000 \cdot 24,\ 1\ MPa \\ 0,\ 75 \cdot \dfrac{51.000}{1,\ 4} \cdot \dfrac{35\ MPa}{35\ MPa + 13} \end{cases}$$

$$E_{0\,cr} \leq \begin{cases} \underline{24100\ MPa} \\ 19921,\ 9\ MPa \end{cases}$$

Where: γ_m – *partial coefficient Teknisk Ståbi table 9.4*
$E_{0\,cr}$ – *critical elasticity module*
$E_{0\,d}$ – *design elasticity module*
E_{cod} – *design elasticity module for concrete*
f_{ck} – *characteristic compression strenght for concrete*
f_{cd} – *design compression strenght for concrete*

Columns critical strength is found:

$$\sigma_{cr} = \frac{f_{ck}}{1 + \dfrac{f_{ck}}{\pi^2 \cdot E_{c0k}} \cdot \left(\dfrac{l_0}{i}\right)^2} = \frac{f_{ck}}{1 + \dfrac{f_{ck}}{\pi^2 \cdot E_{c0k}} \cdot \lambda^2}$$

$$\sigma_{cr} = \frac{24,\ 1\ MPa}{1 + \dfrac{24,\ 1\ MPa}{\pi^2 \cdot 24100\ MPa} \cdot 92,\ 7^2} = 12,\ 9\ MPa$$

Where: E_{c0k} – *concrete's characteristic starting elasticity module*
λ – *slenderness ratio*
l_0 – *effective lenght*
i – *inertia radius*

Reinforcement tensile strength is found:
The reinforcements tensile strength is found
so that it can be compared to its yield strength.

$\sigma_s = \alpha \cdot \sigma_{cr} \;\rightarrow\; \sigma_s = 17,4 \cdot 12,9\ MPa = 224,5\ MPa$

$\sigma_s < f_{yd} \;\rightarrow\; 224,5\ MPa < 458,33\ MPa$

The reinforcement tensile strength is thereby lower than
the yield strenght and the reinforcement will therefore hold.

The columns load capacity is found:
The concretes contribution to the load capacity
is found by N_c vha. $N_c = A_c \cdot \sigma_{cr}$

$N_c = A_c \cdot \sigma_{cr} \;\rightarrow\; N_c = 400\ mm \cdot 400\ mm \cdot 12,9 \cdot 10^{-3} Pa = 2064\ kN$

The reinforcements contribution is found by

N_s by the use of $N_s = min \begin{cases} A_{sc} \cdot \sigma_s \\ k_c \cdot N_c \end{cases}$

$$N_s = min \begin{cases} A_{sc} \cdot \sigma_s \\ k_c \cdot N_c \end{cases} \;\rightarrow\; N_s = min \begin{cases} 1963,5\ mm \cdot 224,5 \cdot 10^{-3}\ Pa \\ 1,0 \cdot 2064\ kN \end{cases}$$

$$= min \begin{cases} 440,8\ kN \\ 2064\ kN \end{cases}$$

$$440,8\ kN < \frac{1}{2} \cdot 2064\ kN$$

Where: k_c *- column factor is 1, 0 with continous reinforcement*

with divided reinforcement the factor is set to $k_c = \frac{1}{2}$.

The columns central load capacity can now be found by
$N_{cr} = N_c + N_s$

$N_{cr} = N_c + N_s \;\rightarrow\; N_{cr} = 2064\ kN + 440,8\ kN = 2504,8\ kN$

Compared to the load found in the previous calculation,
the column has to handle $P_d = 1110,0075\ kN$
This is under half of what the column can handle at breaking point,
so it is over dimensioned in relation to load capacity.

Access Gallery

Cantilever reinforced concrete balcony

The access galleries on the building are a part of the in situ cast decks for each floor. The galleries are supported by beam/discs that run between the columns supporting the deck. It is assumed that the galleries act like cantilever beams, and is therefore calculated as such by setting the depth as 1000mm making the load calculated prior sufficient. The decks for each floor has been estimated at 300mm and must now be demonstrated as being adequate.

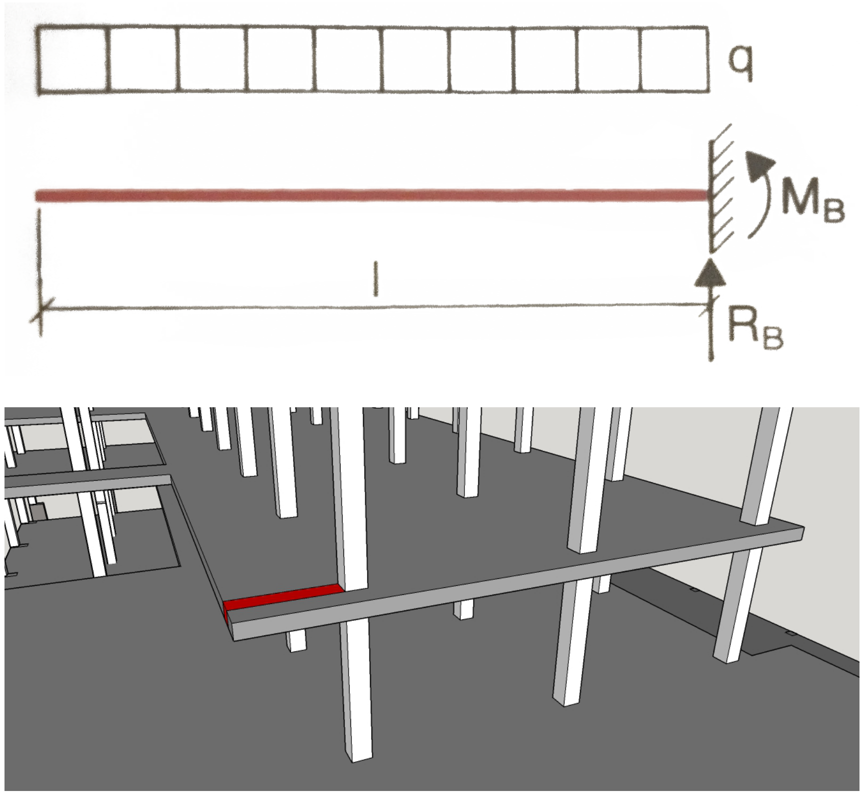


Diagram of cantilevered column and location of balcony

First step is to find the loads that the beam has to handle.

Dead load:

$$g_{beam} = g_{deck} = 7,07 \frac{kN}{m^2}$$

$$g_{total} = g_{beam} + g_{soil} \rightarrow g_{total} = 7,07 \frac{kN}{m^2} + 3,75 \frac{kN}{m^2}$$

$$= 10,82 \frac{kN}{m^2}$$

<u>Data given:</u>

Normally reinforced

Concrete yield strain:	$\varepsilon_{c,u} = 0,35\ \%$
Reinforcement strain:	$\varepsilon_y \leq \varepsilon_s < \varepsilon_{uk}$
Concrete compressive strengt:	$f_{ck} = 35\ MPa$
Reinforcement yield point:	$f_{yk} = 550\ MPa$
Steel elasticity module:	$E_{sk} = 2 \cdot 10^5 MPa$
Partial coefficient:	$\gamma_c = 1,45$ – *in situ concrete*
	$\gamma_s = 1,2$ – *reinforcement*

Concrete Compression strength:

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{35\ MPa}{1,\ 45} = 24,\ 13 \approx 24,\ 1\ MPa$$

Reinforcement Tension strengt:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{550\ MPa}{1,\ 2} = 458,\ 33 \approx 458\ MPa$$

Elasticity module:

$$E_{sd} = \frac{E_{sk}}{\gamma_s} = \frac{2\cdot10^5 MPa}{1,\ 2} = 166666,\ 66 \approx 1,\ 6\cdot10^5 MPa$$

Data given:

Concrete cover above reinforcement: 30mm (EC2 - Tabel 4.4)
Tolerance addition: 5mm
Concrete cover thickness: $c = 30\ mm + 5\ mm = 35\ mm$
Reinforcement diameter: $ø = 25\ mm \rightarrow$ 10pc. (Teknisk Ståbi - 5.3.2)

Determining *d* and A_s :

$$d = 300\ mm - 35 - \frac{25}{2} = 252,\ 5\ mm$$

$$A_s = 10\cdot\left(\left(\frac{25}{2}\right)^2\cdot\pi\right) = 4908,\ 7\ mm^2 \sim 0,\ 0049087\ m^2$$

Where:

d – *distance from top of beam to lowest reinforcement*
A_s – *area of reinforcement section*

Horizontal equilibrium:

$$F_c = F_s \Rightarrow 0,\ 8\ x\cdot b\cdot f_{cd} = A_s\cdot f_{yd} \Rightarrow x = \frac{A_s\cdot f_{yd}}{0,\ 8\cdot b\cdot f_{cd}}$$

$$x = \frac{A_s\cdot f_{yd}}{0,\ 8\cdot b\cdot f_{cd}} \Rightarrow x = \frac{4908,\ 7\ mm^2\cdot 458\ MPa}{0,\ 8\cdot 1000\ mm\cdot 24,\ 1\ MPa} = 116,\ 6\ mm$$

Where:

x - distance from top of beam to zero line.

Service limit state (SLS):

The beam is set to be 1m wide, so the center distance is also 1m as the deck is continues in "depth".

Load combination:

$$P_d = \gamma_G\cdot g_k + \gamma_Q\cdot q_k$$

$$= 1\cdot\left(10,\ 82\frac{kN}{m^2}\cdot 1\right) + 1\cdot\left(5,\ 72\frac{kN}{m^2}\cdot 1\right) = 16,\ 54\ \frac{kN}{m^2}$$

Where:

$\gamma_G = 1,\ 0$ – *dimensioning value* for *loads*
$\gamma_Q = 1,\ 0$ – *dimensioning value* for *loads*
c/c = *1000mm - center distance between beams*
g_k – *dead load · center distance*
q_k – *live load · center distance*

Moment of inertia:

$$I = \frac{1}{12} \cdot b \cdot h^3 \quad \Rightarrow \quad I = \frac{1}{12} \cdot 1000\, mm \cdot (300\, mm)^3 = 22,\, 5 \cdot 10^8\, mm^4$$

Bending due to dead load g

$$u_{inst\,g} = \frac{g \cdot L^4}{8 \cdot E \cdot I} \quad \Rightarrow \quad u_{inst\,g} = \frac{\left(10,\, 82\, \frac{kN}{m} \cdot (2000\, mm)^4\right)}{8 \cdot 160000 \frac{N}{mm^2} \cdot 22,\, 5 \cdot 10^8\, mm^4}$$

$$= 0,\, 06\, mm$$

Bending due to live load q

$$u_{inst\,q} = \frac{q \cdot L^4}{8 \cdot E \cdot I} \quad \Rightarrow \quad u_{inst\,q} = \frac{\left(5,\, 72\, \frac{N}{m} \cdot (2000\, mm)^4\right)}{8 \cdot 160000 \frac{N}{mm^2} \cdot 22,\, 5 \cdot 10^8\, mm^4}$$

$$= 0,\, 032\, mm$$

Total bending for the beam is calculated.

$$u_{\max} = u_{inst\,g} + u_{inst\,q} \quad \rightarrow \quad 0,\, 06\, mm + 0,\, 032\, mm = 0,\, 092\, mm$$

Largest acceptable amount of deformation differs depending on t constructiontype and its context and can be calculated in many w Refering to DS/EN 1992-1-1 there are two simplified calculation f deformations from quasipermenant loads.

1. Apperance and general use $\frac{l}{250}$

if the deformation does not have critical consequence

2. If joining other constructionparts and the deformation is critical $\frac{l}{500}$

As the gallery is not supported or joining other constructionparts below the first will suffice

$$0,\, 092\, mm \le \frac{l}{250} = 8\, mm$$

So the amount of bending is easily acceptable.

Ultimate limit state (ULS):

Load combination:

$P_d = \gamma_G \cdot g_k + \gamma_Q \cdot q_k$

$$= 1 \cdot \left(10,82 \frac{kN}{m^2} \cdot 1\right) + 1,5 \cdot \left(5,72 \frac{kN}{m^2} \cdot 1\right) = 19,4 \ \frac{kN}{m^2}$$

Where:

$\gamma_G = 1,0$ – *dimensioning value* for *loads*

$\gamma_Q = 1,5$ – *dimensioning value* for *loads*

c/c = *1000mm - center distance between beams*

g_k – *dead load · center distance*

q_k – *live load · center distance*

Permissible moment for the beam:

The total moment for the beam is calculated.

$$M(x) = \frac{1}{2} \cdot P_d \cdot L^2 = \frac{21,95 \ \frac{kN}{m} \cdot (2\,m)^2}{2} = 38,8 \ kNm$$

Where:

P_d – *load combination*

L – *lenght of beam*

Yield moment:

$z = d - 0,4\,x \quad \Rightarrow \quad z = 252,5\,mm - 0,4 \cdot 116,6$ mm

$= 206$ mm

$$M = A_s \cdot f_{yd} \cdot z \quad \Rightarrow \quad 0,0049087\,m^2 \cdot 458 \cdot 10^3 \ \frac{kN}{m^2} \cdot 0,206\,m$$

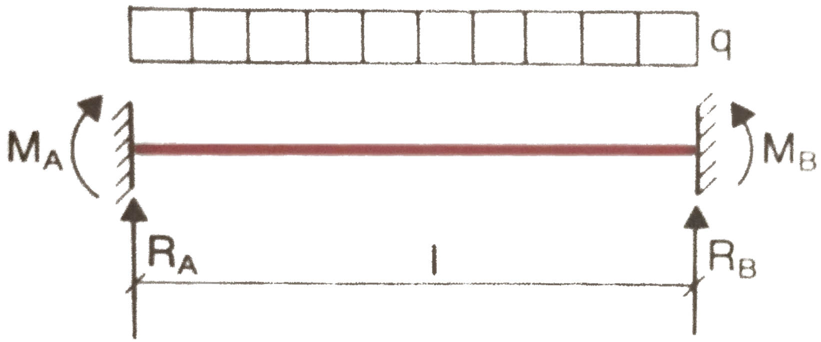
$= 463\ kNm$

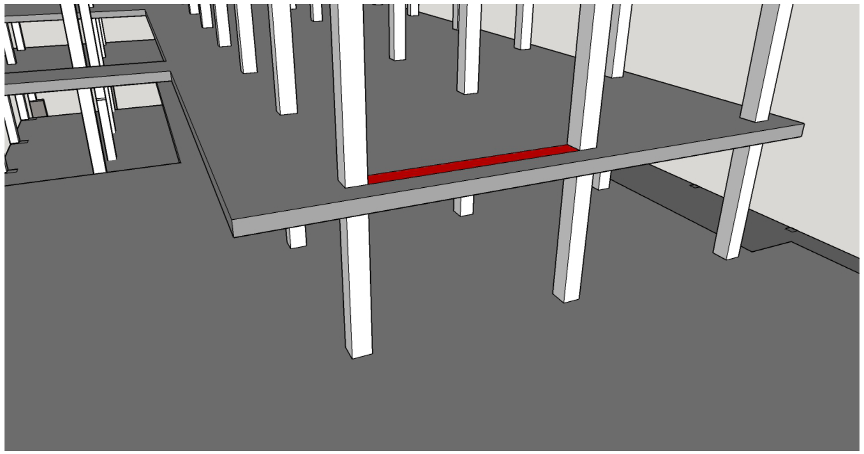
The beam is subjected to 38,8 kNm, but it can handle up to 463 kNm. The beam will therefore not break, since the beams maximum moment is larger than the yield moment.

Floor structure

Reinforced in situ cast concrete decks

The decks for each floor are in situ cast, and estimated at a thickened of 300mm. The supporting columns are spaced in a 4,8x4,8m grid, so the maximum span for the decks is 4,8m. To look at the most extreme situation for the deck, an outdoor scenario is chosen.
The deck is restrained in both ends because it is cast into the structure and will be calculated as fixed in both ends.





<u>Data given:</u>

Concrete compressive strengt: $f_{ck} = 35\ MPa$

Reinforcement yield point: $f_{yk} = 550\ MPa$

Steel elasticity module: $E_{sk} = 2 \cdot 10^5 MPa$

Partial coefficient: $\gamma_c = 1,45$ – *in situ concrete*

$\gamma_s = 1,2$ – *reinforcement*

Calculation of the beams compressive and tensile strenght.

Compression:

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{35\ MPa}{1,45} = 24,13 \approx 24,1\ MPa$$

Tension:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{550\ MPa}{1,2} = 458,33 \approx 458\ MPa$$

Elasticity module:

$$E_{sd} = \frac{E_{sk}}{\gamma_s} = \frac{2 \cdot 10^5 MPa}{1,2} = 166666,66 \approx 1,6 \cdot 10^5 MPa$$

Data given:

Concrete cover above reinforcement: 30mm (EC2 - Tabel 4.4)
Tolerance addition: 5mm
Concrete cover thickness: $c = 30 + 5 = 35\ mm$
Reinforcement diameter: $ø = 25\ mm \rightarrow$ 10pc.
(Teknisk Ståbi - 5.3.2)

Determining d and A_s :

$$d = 300\ mm - 35 - \frac{25}{2} = 252,\ 5\ mm$$

$$A_s = 10 \cdot \left(\left(\frac{25}{2} \right)^2 \cdot \pi \right) = 4908,\ 7\ mm^2 \sim 0,\ 0049087\ m^2$$

Where:

d – distance from top of beam to lowest reinforcement
A_s – area of reinforcement section

Horizontal equilibrium:

$$F_c = F_s \ \Rightarrow\ 0,\ 8\ x \cdot b \cdot f_{cd} = A_s \cdot f_{yd} \ \Rightarrow\ x = \frac{A_s \cdot f_{yd}}{0,\ 8 \cdot b \cdot f_{cd}}$$

$$x = \frac{A_s \cdot f_{yd}}{0,\ 8 \cdot b \cdot f_{cd}} \ \Rightarrow\ x = \frac{4908,\ 7\ mm^2 \cdot 458\ MPa}{0,\ 8 \cdot 1000\ mm \cdot 24,\ 1\ MPa} = 116,\ 6\ mm$$

Where:

x - distance from top of beam to zero line.

Service limit state (SLS):

The beam is set to be 1m wide, so the center distance is also 1m.

Load combination:

$$P_d = \gamma_G \cdot g_k + \gamma_Q \cdot q_k$$

$$g_k = 14,\ 57 \frac{kN}{m^2}$$

$$q_k = 5,\ 72$$

$$= 1 \cdot \left(14,\ 57 \frac{kN}{m^2} \cdot 1 \right) + 1 \cdot \left(5,\ 72 \frac{kN}{m^2} \cdot 1 \right) = 20,\ 29\ \frac{kN}{m^2}$$

Where:

$\gamma_G = 1,\ 0$ – *dimensioning value* for *loads*
$\gamma_Q = 1,\ 0$ – *dimensioning value* for *loads*
c/c = *1000mm - center distance between beams*
g_k – *dead load · center distance*
q_k – *live load · center distance*

Moment of inertia on fixed beam:

$$I = \frac{1}{12} \cdot b \cdot h^3 \ \Rightarrow\ I = \frac{1}{12} \cdot 1000\ mm \cdot (300\ mm)^3 = 22,\ 5 \cdot 10^8\ mm^4$$

Bending due to dead load g

$$u_{inst\,g} = \frac{1}{384} \cdot \frac{g \cdot L^4}{E \cdot I}$$

$$\Rightarrow u_{inst\,g} = \frac{1}{384} \cdot \frac{\left(14,57\,\frac{kN}{m} \cdot (4800\,mm)^4\right)}{160000\frac{N}{mm^2} \cdot 22,5 \cdot 10^8\,mm^4} = 0,0559488\ \mathrm{mm}$$

Bending due to live load q

$$u_{inst\,q} = \frac{1}{384} \cdot \frac{q \cdot L^4}{E \cdot I}$$

$$\Rightarrow u_{inst\,q} = \frac{1}{384} \cdot \frac{\left(5,72\,\frac{N}{m} \cdot (4800\,mm)^4\right)}{160000\frac{N}{mm^2} \cdot 22,5 \cdot 10^8\,mm^4} = 0,0219648\ mm$$

Total bending for the beam is calculated.

$$u_{\max} = u_{inst\,g} + u_{inst\,q} \quad \rightarrow \quad 0,0559488\ \mathrm{mm} + 0,0219648\ \mathrm{mm}$$
$$= 0,078\ \mathrm{mm}$$

Largest acceptable amount of bending in this case $\frac{l}{500}$.

$$0,078\ mm \leq \frac{l}{500} = 9,6\ mm$$

So the amount of bending is easily acceptable.

Ultimate limit state (ULS):

The beam is set to be 1m wide, so the center distance is also 1m.

Load combination:

$$P_d = \gamma_G \cdot g_k + \gamma_Q \cdot q_k$$

$$= 1,5 \cdot \left(14,57\frac{kN}{m^2} \cdot 1\right) + 1 \cdot \left(5,72\frac{kN}{m^2} \cdot 1\right) = 27,575\ \frac{kN}{m^2}$$

Where:

$\gamma_G = 1,5$ – *dimensioning value* for *loads*
$\gamma_Q = 1,0$ – *dimensioning value* for *loads*
c/c = *1000mm - center distance between beams*
g_k – *dead load · center distance*
q_k – *live load · center distance*

Permissible moment for the beam:

The total moment for the beam is calculated.

$$M(x) = \frac{1}{12} \cdot P_d \cdot L^2$$

$$= \frac{27,575 \frac{kN}{m} \cdot (4,8\,m)^2}{12} = 52,94\ kNm$$

Where:

P_d - *load combination*

L - *lenght of beam*

Yield moment:

$$z = d - 0,4\,x \quad \Rightarrow \quad z = 252,5\ mm - 0,4 \cdot 116,6\ \text{mm} = 0,206\ \text{m}$$

$$M = A_s \cdot f_{yd} \cdot z$$

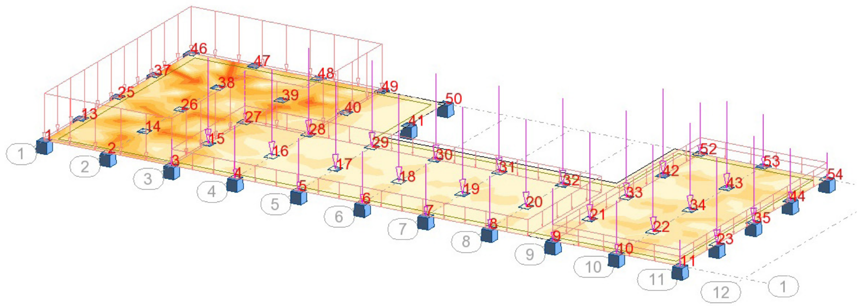
$$\Rightarrow \quad 0,0049087\ m^2 \cdot 458 \cdot 10^3 \frac{kN}{m^2} \cdot 0,206\ m = 463\ kNm$$

The beam is subjected to 52,94 kNm, but it can handle up to 463 kNm. The beam will therefore not break, since the beams maximum moment is larger than the yield moment.

Column load distribution

Several assumptions were made in relation to the design. The final concept is assuming that the disc located throughout the building will ensure horizontal stability thus allowing the construction to carry the loads directly down into the columns.

In this analysis the load of the deck and dwellings are added as nodal loads along with the evenly distributed loads in the deck it self. The results as expected shows that by carrying the load directly down into the columns, the plate is primarily affected by the evenly distributed loads thus minimizing the moments across the deck. Furthermore it is possible to see how the deck is distributing the loads down to the columns making it possible to dimension these so they correspond to their respective loads.



Node	Case	FZ (kN)	MX (kNm)	MY (kNm)
1	ULS+	43,43	21,83	-11,08
2	ULS+	106,73	59,83	1,73
3	ULS+	388,61	28,37	7,85
4	ULS+	683,82	11,82	-0,05
5	ULS+	687,24	12,16	0,1
6	ULS+	686,98	13,03	-0
7	ULS+	687,05	11,66	1,67
8	ULS+	686,22	11,33	0,75
9	ULS+	697,25	13,07	-1,72
10	ULS+	708,19	21,3	-1,39
11	ULS+	352,2	9,39	6,44
13	ULS+	91,45	-1,54	-25,74
14	ULS+	215,6	7,84	10,55
15	ULS+	1920,51	-5,01	27,26
16	ULS+	1137,74	0,2	-1,03
17	ULS+	1141,96	1,49	-0,22
18	ULS+	1142,46	0,42	0,44
19	ULS+	1142,47	1,69	0,31
20	ULS+	1141,67	0,14	4,3
21	ULS+	1163,82	0,95	-5,31
22	ULS+	1187,86	2,23	-1,73
23	ULS+	1840,1	-0,36	15,03
25	ULS+	87,56	-3,98	-26,21
26	ULS+	206,42	-6,81	6,49
27	ULS+	824,12	14,55	16,57
28	ULS+	1180,33	19,94	14,9
29	ULS+	1132,09	-0,1	0,66
30	ULS+	1134,03	-0,04	0,36
31	ULS+	1134,02	-0,02	-0
32	ULS+	1133,13	-0,09	1,76
33	ULS+	2296,78	-0,12	-6,16
34	ULS+	1185,68	-1,25	-1,04
35	ULS+	1839,17	-0,82	15,15
37	ULS+	90,93	2,12	-22,23
38	ULS+	233,95	7,16	2,82
39	ULS+	203,27	-9,43	-3,52
40	ULS+	108,35	2,06	43,92
41	ULS+	-1,89	0,77	-0,76
42	ULS+	1154,42	2,87	-6,76
43	ULS+	1188,95	-2,1	-0,27
44	ULS+	706,76	1,11	14,42
46	ULS+	46,41	-12,33	-12,26
47	ULS+	91,93	-22,96	1,35
48	ULS+	90,01	-28,21	-3,61
49	ULS+	47,66	-13,76	15,84
50	ULS+	-1,38	0,26	-0,71
52	ULS+	684,26	-2,79	-3,39
53	ULS+	707,61	-6,94	-1,24
54	ULS+	352,37	-3,15	5,95

Case ULS+	ULS+		
Sum of val.	35710,31	157,77	80,19
Sum of reac.	27030,61	202650,53	-815220,94
Sum of forc.	-27030,61	-202650,53	815220,94
Check val.	-0	-0	-0
Precision	2,00300e-015	3,86647e-032	