# STRUCTURAL DESIGN OF A CANTILEVERED BUILDING

Presented by

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**Master's Thesis:**

# Structural Design of a Cantilevered Building

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# TABLE OF CONTENT

# 1 SUMMARY

The Architectural Engineering graduation project presented in this report had for main focus the integration of the architectural issues associated with the design of a mixed-use building in a high-density urban context with the technical aspects related to very large cantilevered structures. From an architectural point of view, the proposed building aims to reinstate a connection between all parts of Spaarnwoude by providing a new central hub where all forms of transports (rail, bus, pedestrian, cycle, vehicular and maritime traffics) meet in a high-density area with 24h activity. The main building – the heart of this new hub - captures the essence of the new development with its auditoriums for conferences and film projections, exhibition halls, shops, offices, meeting rooms as well as its train station hall and bus station and even rowing facilities (figure 1). The spiral configuration of the 42m-long cantilevered structure – seated on a "plinth" (figure 2) - above Spaarnwoude's new train station is made possible by the development of a unique way of combining steel trusses and this formed the core of the technical analysis part of the graduation project.

This report explains the general structural approach adopted to support this 42m-long cantilevered building and insure its stability. Then, the calculation methods used and results obtained for the main structural elements are presented. These are based on the Eurocodes and the corresponding National Annexes for The Netherlands.

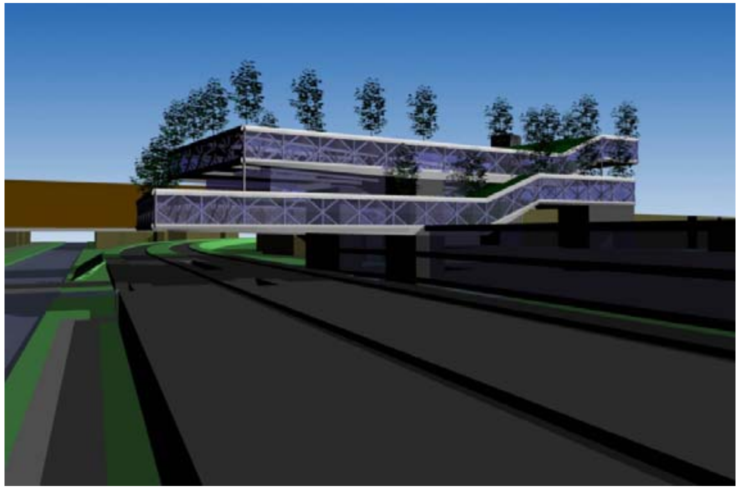

Figure 1 - External view of building

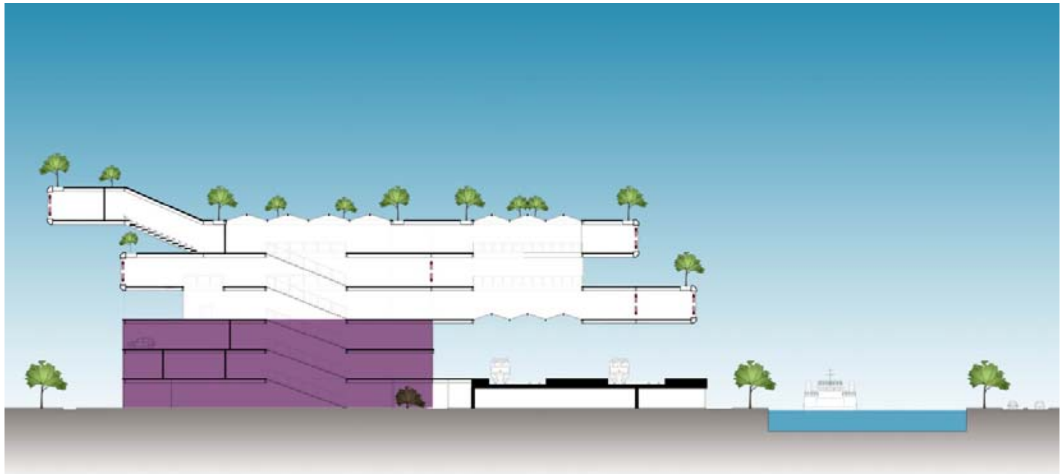

Figure 2 - Longitudinal section - Upper cantilevered structure seated on plinth (purple)

# 2 DESIGN APPROACH - OVERVIEW OF BUILDING'S STRUCTURE

The approach used to design the cantilevered building consists of two main parts:

1) Determination of the nature and configuration of the main structural elements.

2) Sizing of the members with hand calculations and finite element analyses in accordance to the Eurocodes. Checks for stresses and deflections for the worst loading cases.

Figure 3 shows in a schematic manner the main steps of the design process with the corresponding sections of this report. In the first stage, possible floor construction types were selected for further study as well as the general configuration of the upper cantilevered structure of the building. Then, an approach to eliminate any possible uplift force at the supports was developed in parallel with a strategy to insure the stability of the whole structure.

Calculations were then performed in order to select and refine the design of the structure and determine the sizes of the main elements. First of all, the live and dead loads encountered were determined based on the type of spaces and materials used. Knowing the overall dimensions of the structure, various floor options could be found which had various overall depths and construction methods. A definitive choice regarding the floor system was not made at this stage but this gave a more precise idea of the dead loads which would need to be supported as well as the overall depth of the floor which would connect to the main structure.

The primary cantilevered structure was then designed from the study of a cantilevered beam, to the individual trusses and the whole upper structure at ultimate and service limit states. The dimensions of the main structure, a final decision was then made regarding the floor system to be used.

Finally, the lower structure (the "plinth") was added to the cantilevered part of the building and checks were performed to make sure no uplift forces were encountered at the supports for the worst loading cases - including transverse loads – and that the structure would be stable. Once again, maximum stresses and deflections were verified.

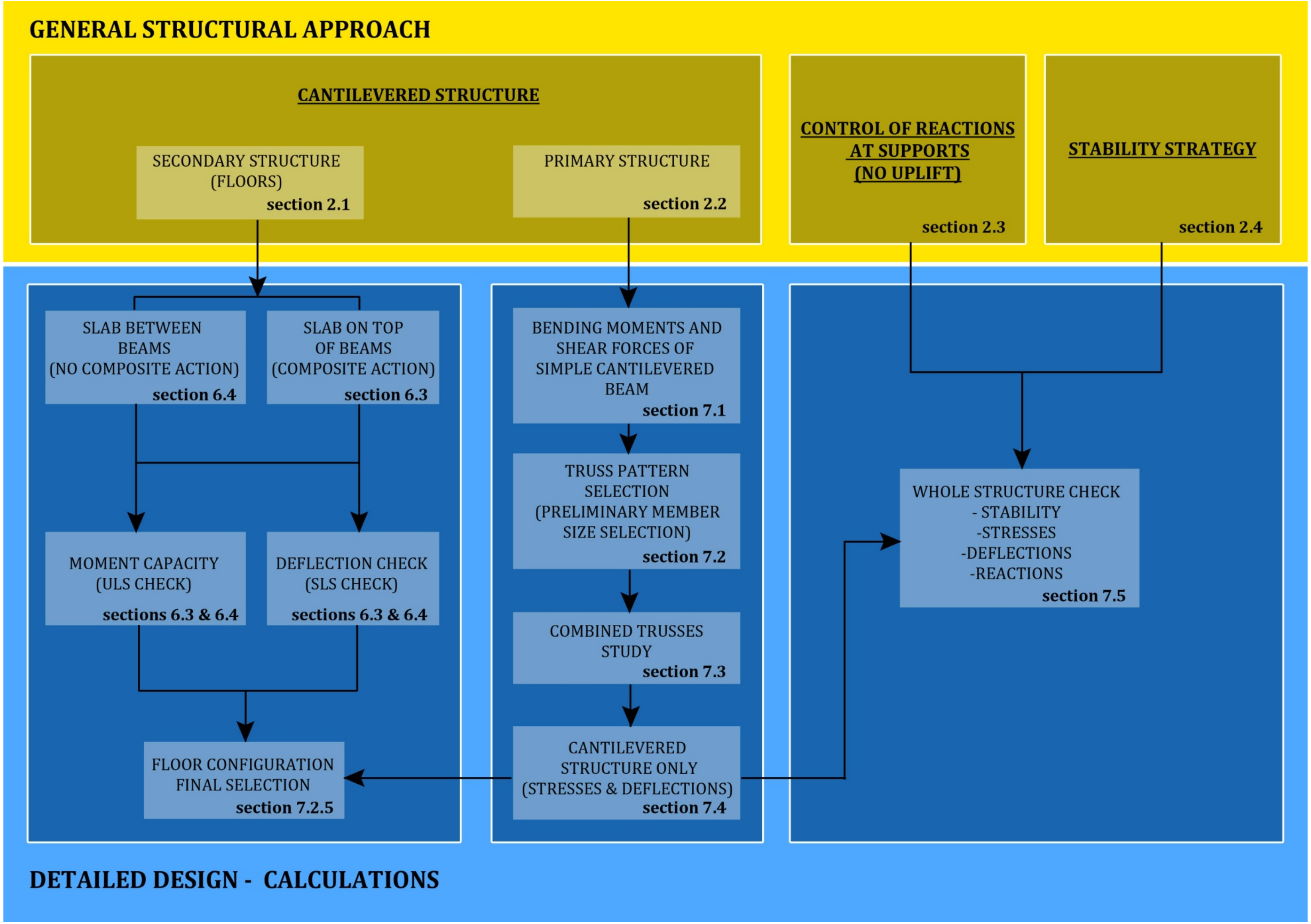


Figure 3 - Diagram of the structural design process

## 2.1 SECONDARY STRUCTURE APPROACH

Because the building has a very large 42m-long and 3-storey-high cantilever, it was important to make decisions which would minimize the weight (dead load) of the structure, thus reducing as much as possible the stresses, deflections and therefore the structural members' sizes.

For this reason, a composite floor system was selected as it results in a much lighter construction than an all-concrete solution. This type of floor system also reduces the construction periods. A composite deck slab consists in a profiled galvanized decking spanning between support beams and act as the permanent formwork for the reinforced concrete slab. Under service loading, after the concrete has set, the decking acts compositely with the concrete slab (figure 4). In order to reduce the overall depth, it is also possible to place the decking between the beams, also eliminating the possibility of composite action between the concrete slab and the beams (figure 5). Both options have been studied.

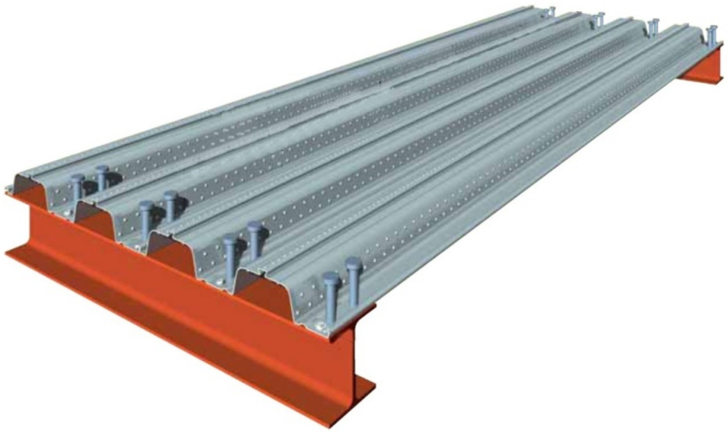

Figure 4 Floor structure with steel decking [6]

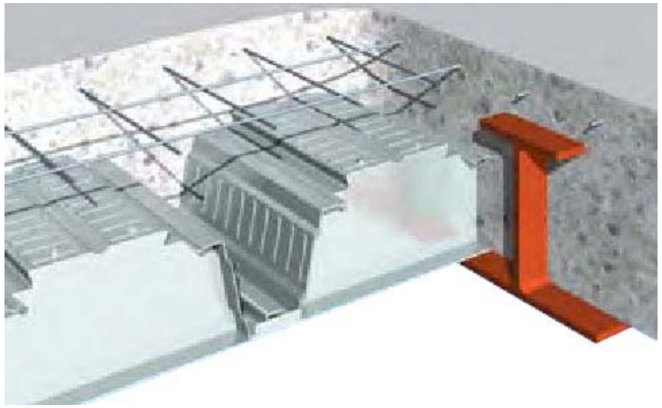

Figure 5 - Slab within beam depth [6]

The metal decking has – amongst others – the following roles [1]:

1) Supports the load of wet concrete during construction
2) Acts as a working platform
3) Transfers in-plane loads by diaphragm action to the vertical bracing or shear walls
4) Stabilizes the beams against lateral torsional buckling if fixed with shear studs to the top flange and at an angle of at least 45°
5) Distributes shrinkage strains preventing serious cracking

The main economy sought in buildings is speed of construction and for this reason slabs and beams are generally designed to be unpropped during construction. However, this reduces the spans that can be achieved. For this particular building, unpropped construction is also sought to reduce the extent of temporary works above the railway tracks and disturbance to the train service.

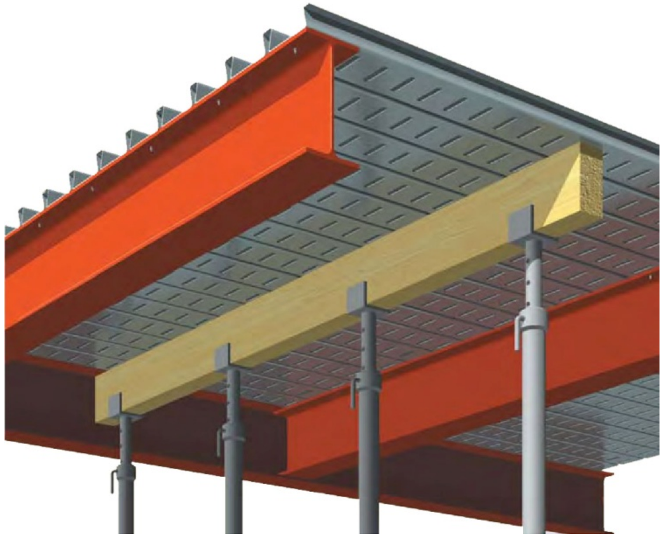

Figure 6 - Propped decking during construction [6]

## 2.2 PRIMARY CANTILEVERED STRUCTURE APPROACH

The primary cantilevered structure "sitting" on the plinth is made of a series of steel trusses. Maximum stiffness is achieved by keeping the ratio cantilever/backspan as small as possible and having the maximum truss depth. In this case, the site constraints and the shape of the building which had to be achieved dictated these values.

First of all, a cantilever/backspan ratio of 1 was provided, placing the four "cores" on the outer edges of the plinth. This allows for a car park free of very large vertical supports within the plinth and a column-free platform area around the railway tracks. Both the cantilever and backspan have a length of 42m.

Then, because the cantilevered upper structure had to have a "spiral" shape, it was not possible to use trusses which were the full depth of the upper structure, i.e. 3-storey high. Pairs of 1-storey high were therefore used to create the desired shape. As it was expected that deflections would be a major issue, it was important to find a way to limit them. This has been done by simply connecting the ends of the upper and lower trusses together with a steel tie. This principle was initially developed when both trusses are in the same plane. In order to achieve the spiral shape, the support points were connected to different sides of the cores, rotating both trusses about the steel ties connecting them (figure 7).

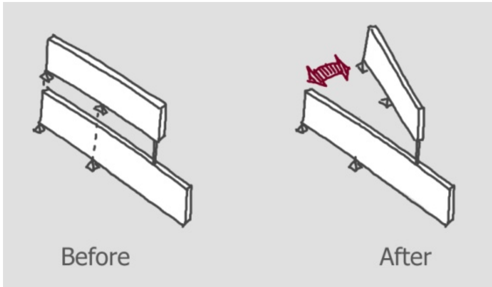


Figure 7 - Strategy used for combined cantilevered trusses

Finally, the width of the building being approximately 35m, a way of limiting the floor spans had to be found in order to reduce their depth – and weight - as much as possible. This has been achieved through the layout of the pairs of combined trusses. This way, a truss may start at the edge of the building at one end and be located at its centre at the other end. The upper and/or lower floors can then sit or hang from this truss. The following figures show the exact layout of the main trusses supported by four cores as well as the columns and hangers for the floors in the upper cantilevered structure.

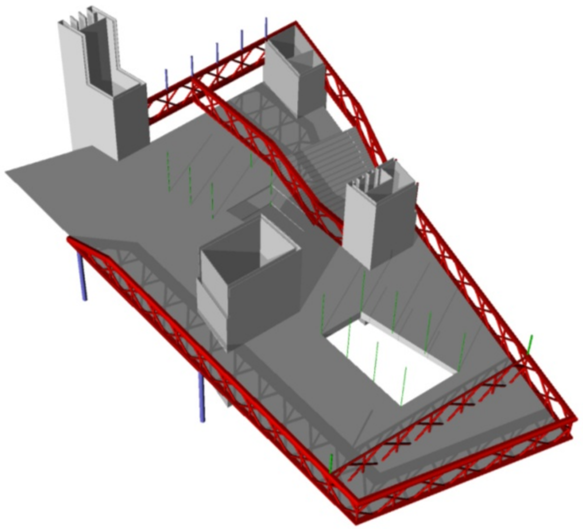

Figure 8 - Main trusses at third (part fourth) floor – plinth level

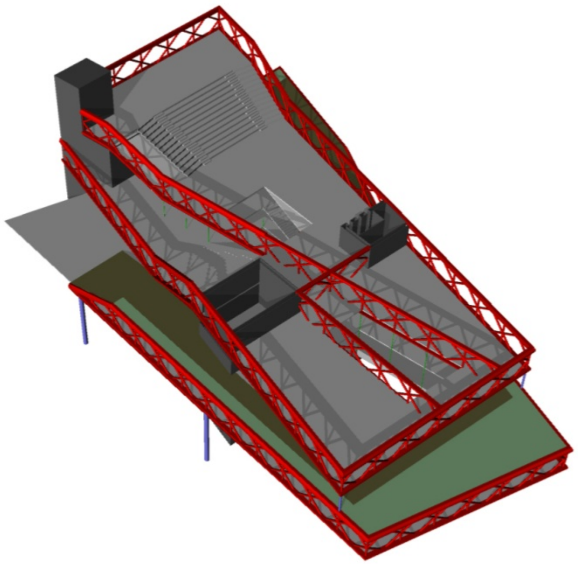

Figure 9 - Main trusses at fifth (and sixth) floor

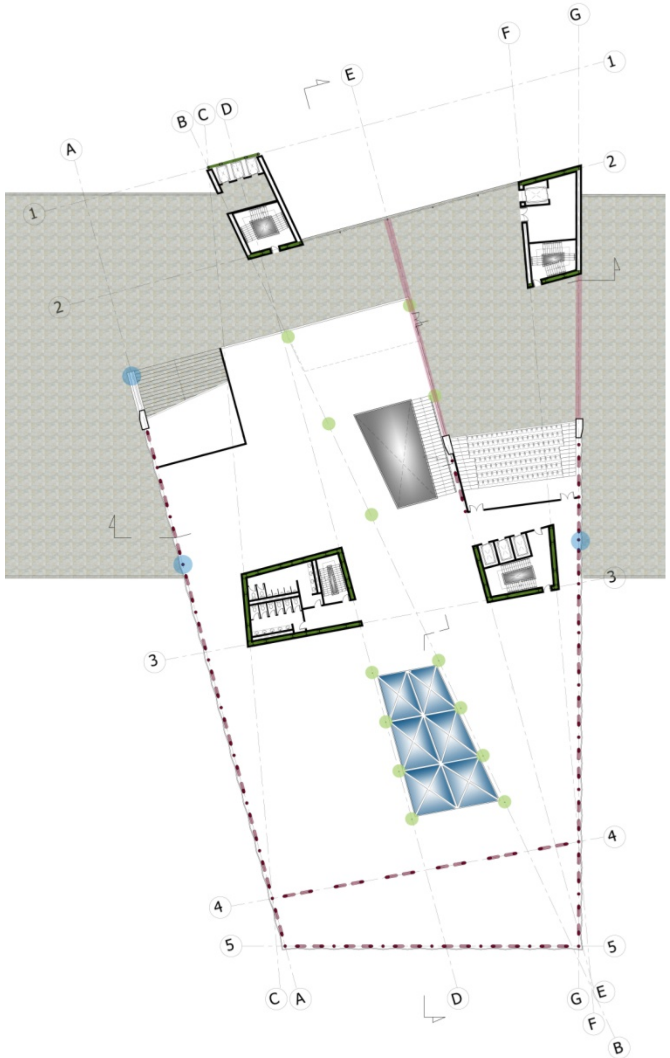


Figure 10 - Third floor plan (plinth level)

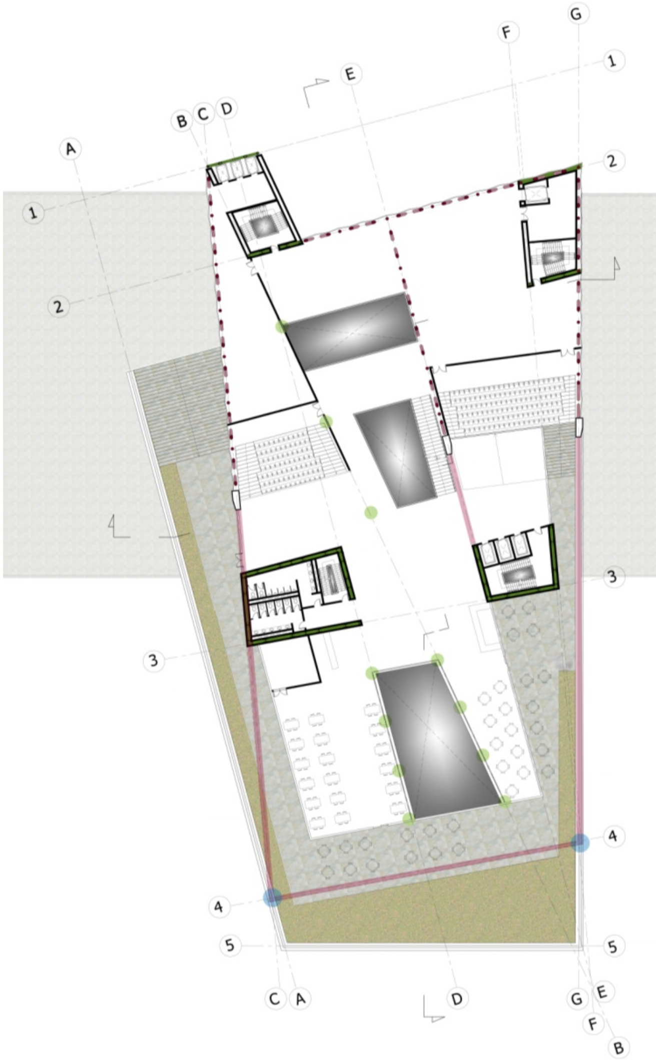

Figure 11 - Fourth floor plan

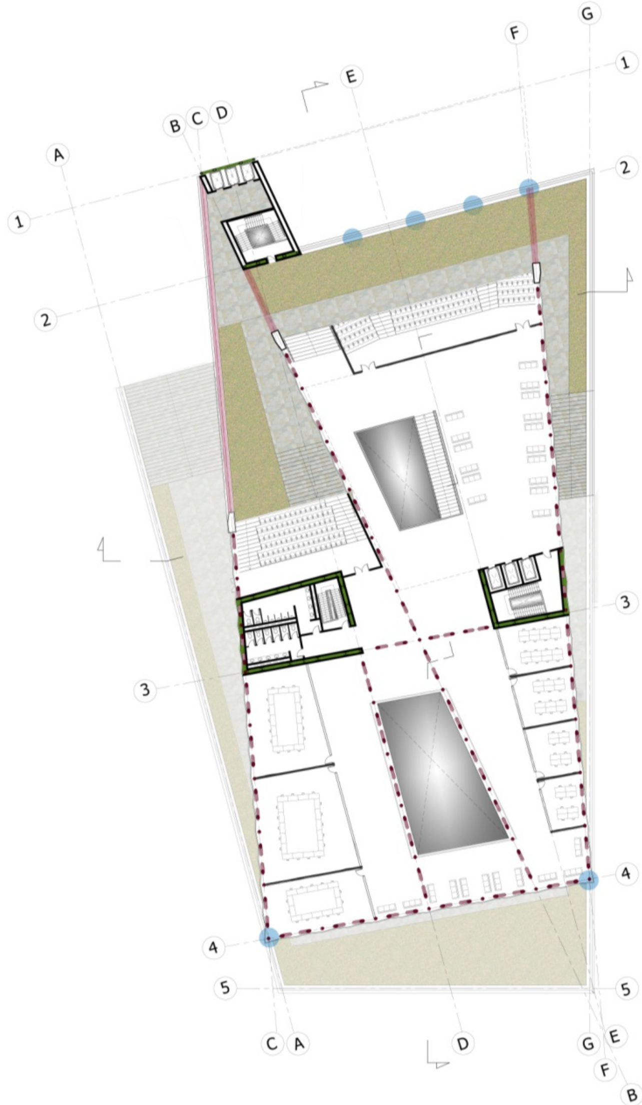

Figure 12 - Fifth floor plan

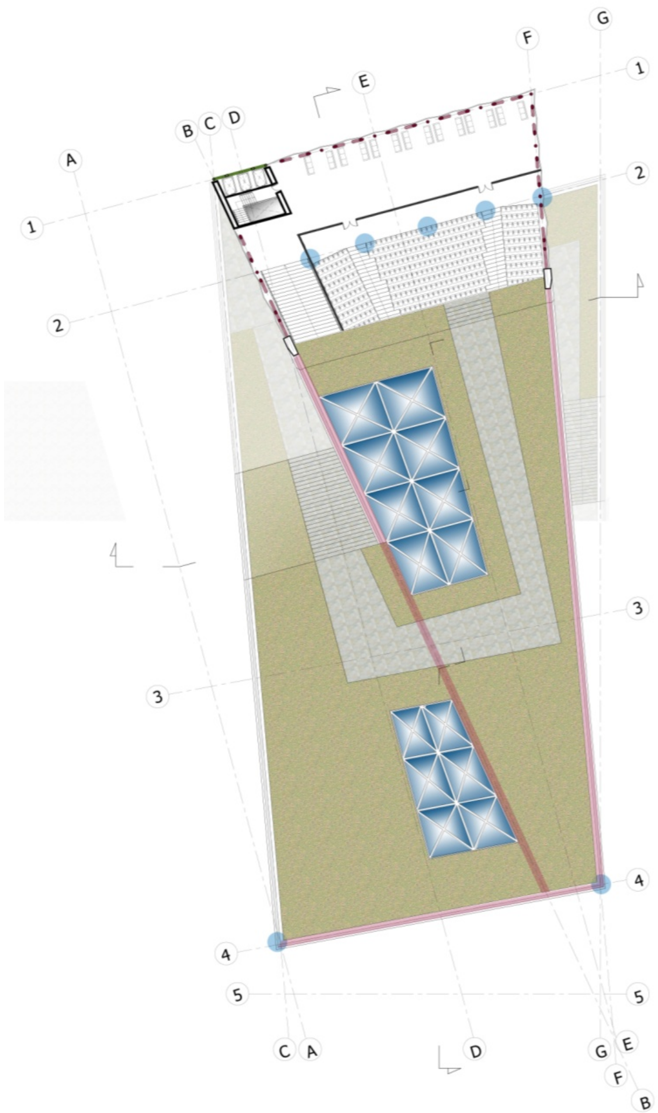


Figure 13 - Sixth floor plan

## 2.3 CONTROL OF REACTION FORCES AT SUPPORTS

One of the main issues which may arise when building a cantilevered structure is the presence of uplift forces at the back supports. Uplift forces at the foundations have to be avoided as they require very costly and time-consuming systems. They can be eliminated by increasing the downward load on the supports. This can be done either by adding material on the cores (for example concrete cores with thicker walls), or by designing the structure of the building in such a way that the loads are directed as much as possible towards the main supports. The latter option was used as it makes a more efficient use of the materials.

The plinth is the key element in eliminating the uplift forces at the back supports. Initially, the structure of the plinth was almost independent from the upper structure; the floors were sitting on a series of columns which transferred the loads directly to the ground. This resulted in low reactions at the main "core" supports. These reactions were increased simply by "flipping" the structure upside-down, i.e. by hanging the floors of the plinth from the backspan of the upper cantilevered structure (figure 14). This way the floor spans of the car park remain fairly small, limiting their depth, but the loading transferred to the main supports is greatly increased, eliminating any uplift force.

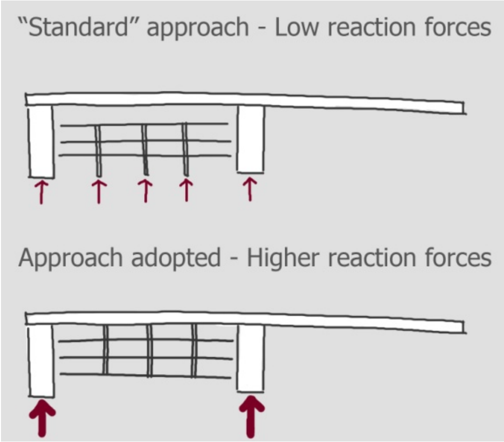


Figure 14 - Strategy to eliminate uplift forces at supports

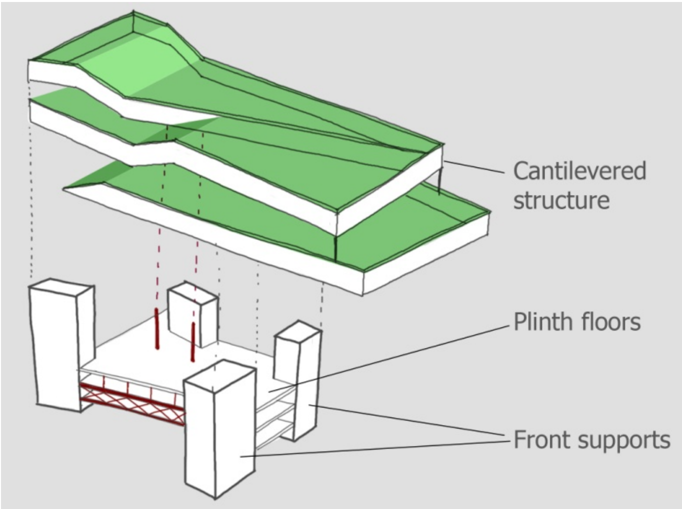


Figure 15 - Sketch of plinth floors hung from cantilevered structure

## 2.4 STABILITY STRATEGY

The main support or "cores" have to be designed to limit their deflection at the top, to which are directly related the maximum stresses. This deflection has three components:

1) Due to temperature variations
2) Flexion due to side winds
3) Torsion which is dictated by the centre of stiffness position in relation to the wind load resultant

First of all, the upper steel structure will expand (mainly longitudinally) or contract under temperature variations, inducing stresses into the vertical supports. These stresses can be minimized by strategically adjusting the stiffness of the supports. In this case, the largest stiffness in the longitudinal direction has been placed at the front supports and the lowest one at the back, allowing for a free movement of the upper structure (figure 16).

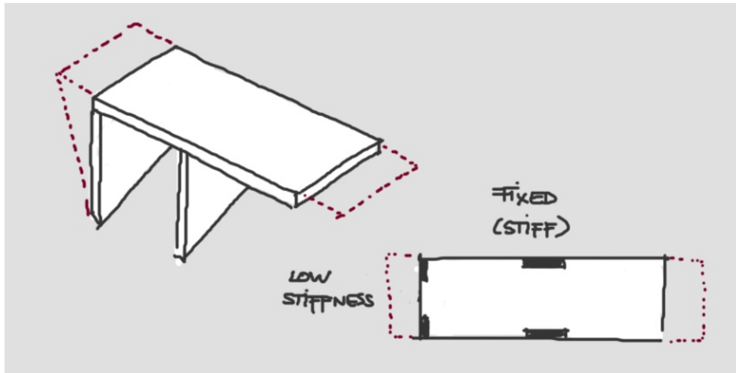


Figure 16 - Strategy for temperature only

Then, the deflection due to the lateral wind load is inversely proportional to the stiffness of the supports in the wind direction. Because we have to cater for winds coming from all directions, we have to provide adequate bracing for all loading cases (figure 17).

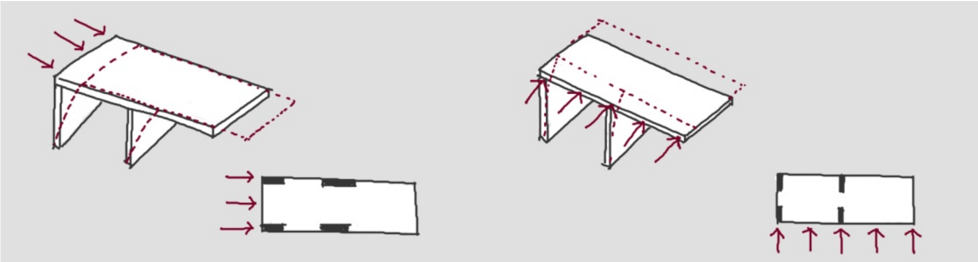

Figure 17 - Strategy for flexion due to lateral wind loads

The total deflection is also due to the torsion of the structure in plan. The intensity of the moment is equal to the value FL, where F is the wind resultant, and L the distance between this force and the centre of stiffness of the structure. The moment is minimized by reducing as much as possible the distance L, i.e. by placing the maximum stiffness near the resultant F which is at the geometric centre - and the front supports - of the building in this case (figure 18).

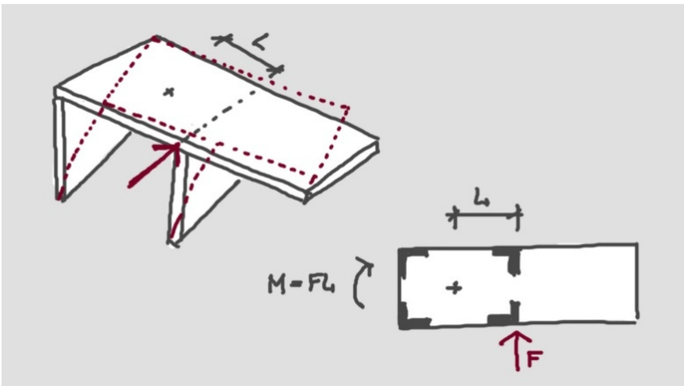


Figure 18 - Torsion due to lateral wind loads

The final configuration of the supports was determined by combining the strategies related to temperature variations as well as flexion and torsion due to wind loadings (figure 19). At the back supports, the stiffness is provided in the transverse direction. On the other hand, bracing is present in both transverse and longitudinal directions, hence an overall stiffness which is greater than at the back supports. This way, the lever arm L is reduced, minimizing the torsion and deflections due to lateral wind loading.

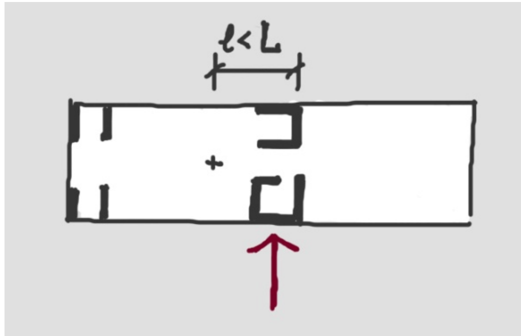


Figure 19 - Combination of strategies for temperature, flexion and torsion

Figure 20 shows the final configuration for the core structure. The walls corresponding to the ones marked on figure 19 are braced with steel members. In order to further improve the stiffness under the lateral wind loads applied on the large side of the building, two “portal frames” have been created at the front and back supports by combining the pairs of vertical trusses in the cores with a truss from the main upper structure. This way, by linking the ends for the vertical trusses, we combine their stiffness and reduce the deflections, as has been done for the upper cantilevered structure.

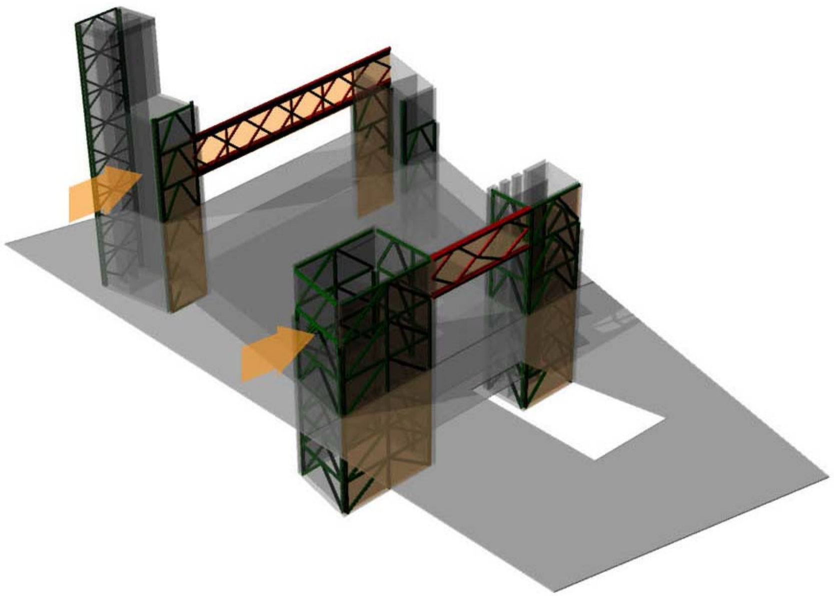

Figure 20 - Configuration of vertical trusses in cores

# 3 LOADING CALCULATIONS

The limit state design philosophy is used for structural design in Europe, on which are based the Eurocodes. This philosophy considers two limit states:

1) ***Ultimate limit state (stresses check) - ULS*:**
The collapse of all or part of the structure

2) ***Serviceability limit state (deflections check) - SLS*:**
A state before collapse at which deformation, appearance or condition of the structure becomes unacceptable or cause discomfort to users

To satisfy the ultimate state requirements, it must be shown that there is an adequate margin of safety against collapse of any significant element of the structure for the worst combination of loading and material properties that can occur.

According to the Eurocode, these states have to be checked for various design situations [3]:

1) ***Persistent design situations*:**
Refers to the condition of normal use

2) ***Transient design situations*:**
Temporary conditions, for example during construction

3) ***Accidental design situations*:**
Exceptional conditions applicable to the structure or its exposure, e.g. to fire, explosion, impact or the consequences of localized failure

Therefore, for each design situation, both limit states have to be checked when applying to the various types of loadings the corresponding safety factors provided in the Eurocode.

The Eurocode classifies the loads based on their variation in time:

1) ***Permanent actions (G)*****:**
   Self-weight of structures and fixed equipment

2) ***Variable actions (Q):***
   Imposed (live) loads on building floors, beams and roofs, wind actions or snow loads

3) ***Accidental actions (A)*****:**
   Explosions or impact from vehicles

In this case, to simplify the calculations, only the permanent and variable actions will be considered.

The applied load for limit state checks purposes is calculated using the following formula:

$$W = \gamma_g G + \gamma_q Q$$

With the safety factors from the table X below:

| CASE | PERM. ACTIONS $\gamma_g$ | VARIABLE ACTIONS $\gamma_q$ | |
|---|---|---|---|
| | | Unfavourable | Favourable |
| ULS | 1,35 | 1,5 | 0 |
| SLS | 1 | 1 | 0 |

Table 1 - Safety factors [3]

## 3.1 Live Loads on Floors

The live loads on floors (variable actions) values are found in the National Annex of the Eurocode 1

| **Klasse van belaste oppervlakte** | $q_k$ kN/m$^2$ | $Q_k$ kN |
|---|---|---|
| **Klasse A** (wonen en huishoudelijk gebruik) | | |
| A-vloeren | 1,75 [a] | 3 [a] |
| A-trappen | 2,0 | 3 |
| A-balkons | 2,5 | 3 |
| **Klasse B** (kantoorruimten) | | |
| B-kantoorruimten | 2,5 | 3 |
| **Klasse van belaste oppervlakte** | $q_k$ kN/m$^2$ | $Q_k$ kN |
| **Klasse C** (bijeenkomstruimten) | | |
| C1-tafels | 4,0 | 7 |
| C2-vaste zitplaatsen | 4,0 | 7 |
| C3-zonder obstakels voor rondlopende mensen | 5,0 | 7 |
| C4-fysieke activiteiten | 5,0 | 7 |
| C5-grote mensenmassa's | 5,0 | 7 |
| **Klasse D** (winkelruimten) | | |
| D1-kleinhandel | 4,0 | 7 |
| D2-warenhuizen | 4,0 | 7 |

[a] Deze waarden moeten ook zijn gebruikt voor constructies van ondergeschikte betekenis.

Table 2 - Imposed loads on floors table 6.2 EC1 [4]

| **Klasse van belaste oppervlakte** [b] | $q_k$ kN/m$^2$ | $Q_k$ kN |
|---|---|---|
| F (lichte voertuigen lichter dan 25 kN) | 2 | 10 |
| G (middelzware voertuigen 25 kN tot 120 kN) | 5 | 40 |
| G (voertuigen zwaarder dan 120 kN) | $G_v / A_v$ [a] | maximale krikbelasting |

[a] $G_v$ is het gewicht van het voertuig, in kN en $A_v$ is de oppervlakte ingenomen door het voertuig, in m$^2$.

[b] Voor banen en hellingen van parkeergarages moet een extra horizontale remkracht op het wegoppervlak zijn toegepast. Deze belasting moet zijn beschouwd als een statische belasting. Voor voertuigen met een gewicht tot 25 kN moet een horizontale kracht van 10 kN zijn gebruikt. Voor voertuigen met een gewicht groter dan 25 kN moet de horizontale kracht per baan zijn bepaald met $Q_k = m \cdot a$, in N, waarbij $m$ is de massa van het volledig beladen voertuig, in kg en $a$ is de vertraging ten gevolge van de remvertraging, in m/s$^2$.

Table 3 - Imposed vehicle loading table 6.8 EC1 [4]

Table 4 summarizes the variable loads on floors used for this particular building.

| SPACE TYPE | CATEGORY OF USE | LIVE LOAD [kN/m²] |
|---|---|---|
| Offices<br>Meeting rooms | B | 2,5 |
| Auditoriums | C2 | 4 |
| Exhibition hall | C5 | 5 |
| Restaurant<br>Café | C1 | 4 |
| Shops | D1 | 4 |
| Car park | G | 5 |

Table 4 - Live loads on floors used

## 3.2 Wind Loads

The wind load acting on the building can be calculated using Eurocode 1, part 1-4 [5] and its corresponding National Annex for The Netherlands. To simplify the problem, we will only consider the transverse wind load acting on the windward vertical façades and ignore the succion forces on the roofs and leeward façades.

As one might expect, the wind load will depend on the wind velocity, which in turn depends – amongst other things - on the orientation of the façade according the prevailing wind direction, the terrain roughness, the exposure and altitude. All these variables are taken into account in the Eurocode when calculating the wind pressure.

The building has a height of h=35m and a length of approximately b=84m. Because h<b, the Eurocode recommends using a constant wind pressure along the whole height of the building with z=h=35m as shown on the figure below.

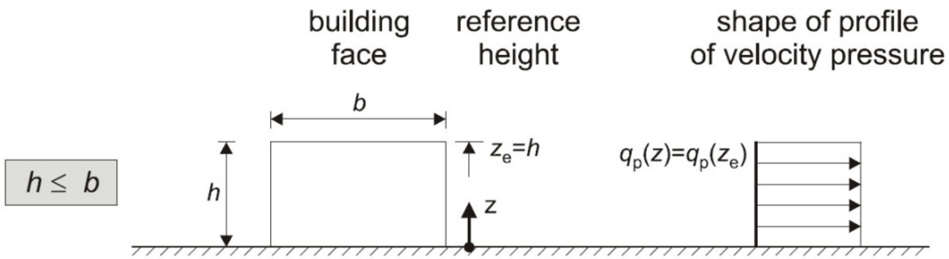


Figure 21 - Wind profile EC1 [5]

### 1) *Calculate terrain roughness factor*

The building is located in an urban area, a terrain category III.

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \qquad \text{for } z_{min} \leq z \leq z_{max}$$

where:

$z_0$ : roughness length = 0,5m
$z_{0,II}$ : 0,2m (terrain category II, table 5)
$z_{min}$ : minimum height table 5 = 7m
$z_{max}$ : 200m
$k_r = 0{,}19 \cdot \left(\frac{z_0}{0{,}05}\right)^{0{,}07}$ : terrain factor = 0,223

Hence:

$$c_r(35) = 0{,}223 \cdot \ln\left(\frac{35}{0{,}5}\right) = 0{,}947$$

| Terreincategorie | | $z_0$ m | $z_{min}$ m |
|---|---|---|---|
| 0 | Zee of kustgebied aan zee | 0,005 | 1 |
| II | Onbebouwd gebied | 0,2 | 4 |
| III | Bebouwd gebied | 0,5 | 7 |

**Table 5 - Terrain categories and parameters, table 4.1 EC1 [12]**

### *2) Calculate mean wind velocity*

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_{b,0}$$

where:

$c_r(z)$ : roughness factor given by eq. 2 = 0,947
$c_0(z)$ : orography factor = 1,0
$v_{b,0}$ : fundamental wind velocity from National Annex Table NB.1 for zone II = 27,0 $m/s^2$

Hence:

$$v_m(17{,}5) = 0{,}947 \cdot 1{,}0 \cdot 27 = 25{,}57 \text{ m/s}$$

| Windgebied | $v_{b,0}$ m/s |
|---|---|
| I | 29,5 |
| II | 27,0 |
| III | 24,5 |

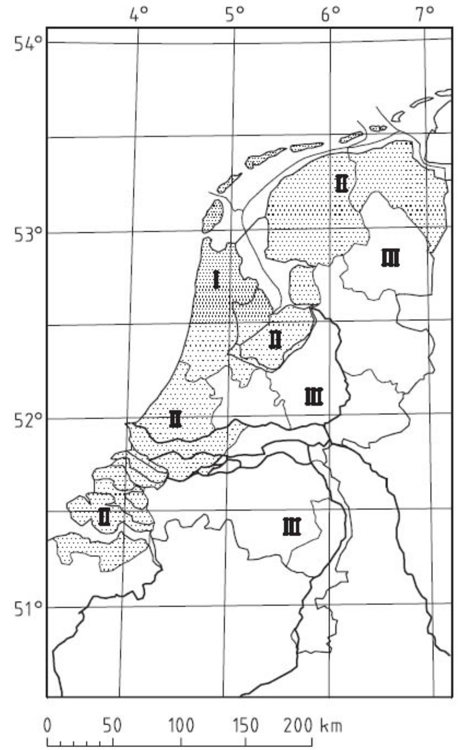


**Figure 22 - Wind zones and corresponding wind speeds [12]**

### 3) *Calculate wind turbulence intensity*

$$I_v(z) = \frac{k_l}{c_0(z) \cdot \ln(z/z_0)} \quad \text{for } z_{min} \leq z \leq z_{max}$$

where:

| | | |
|---|---|---|
| $k_l$ | : turbulence factor | $= 1{,}0$ |
| $c_0(z)$ | : orography factor | $= 1{,}0$ |
| $z_0$ | : roughness length | $= 0{,}5$ |

Hence:

$$I_v(35) = \frac{1{,}0}{1{,}0 \cdot \ln(35/0{,}5)} = 0{,}235$$

### 4) *Calculate peak velocity pressure*

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot {v_m}^2(z)$$

where :

| | | |
|---|---|---|
| $\rho$ | : air density | $= 1{,}25\ kg/m^3$ |
| $I_v(z)$ | : turbulence calculate with eq. 4 | $= 0{,}235$ |
| $v_m(z)$ | : meanwind velocity from eq. 3 | $= 25{,}57\ m/s$ |

The characteristic wind load used is therefore:

$$q_p(17{,}5) = [1 + 7 \cdot 0{,}235] \cdot \frac{1}{2} \cdot 1{,}25 \cdot 25{,}57^2$$
$$= \mathbf{1{,}08\ kN/m^2}$$

This value is almost equal to the maximum (indicative) wind pressure in Table NB.4 of the National Annex of Eurocode 1 for a 35m-high building in an urban area.

Tabel NB.4 (informatief) — Extreme stuwdruk in kN/m² als functie van de hoogte

| Hoogte | Gebied I | | | Gebied II | | | Gebied III | |
|---|---|---|---|---|---|---|---|---|
| m | kust | onbe-bouwd | be-bouwd | kust | onbe-bouwd | be-bouwd | onbe-bouwd | be-bouwd |
| 1 | 0,93 | 0,71 | 0,69 | 0,78 | 0,60 | 0,58 | 0,49 | 0,48 |
| 2 | 1,11 | 0,71 | 0,69 | 0,93 | 0,60 | 0,58 | 0,49 | 0,48 |
| 3 | 1,22 | 0,71 | 0,69 | 1,02 | 0,60 | 0,58 | 0,49 | 0,48 |
| 4 | 1,30 | 0,71 | 0,69 | 1,09 | 0,60 | 0,58 | 0,49 | 0,48 |
| 5 | 1,37 | 0,78 | 0,69 | 1,14 | 0,66 | 0,58 | 0,54 | 0,48 |
| 6 | 1,42 | 0,84 | 0,69 | 1,19 | 0,71 | 0,58 | 0,58 | 0,48 |
| 7 | 1,47 | 0,89 | 0,69 | 1,23 | 0,75 | 0,58 | 0,62 | 0,48 |
| 8 | 1,51 | 0,94 | 0,73 | 1,26 | 0,79 | 0,62 | 0,65 | 0,51 |
| 9 | 1,55 | 0,98 | 0,77 | 1,29 | 0,82 | 0,65 | 0,68 | 0,53 |
| 10 | 1,58 | 1,02 | 0,81 | 1,32 | 0,85 | 0,68 | 0,70 | 0,56 |
| 15 | 1,71 | 1,16 | 0,96 | 1,43 | 0,98 | 0,80 | 0,80 | 0,66 |
| 20 | 1,80 | 1,27 | 1,07 | 1,51 | 1,07 | 0,90 | 0,88 | 0,74 |
| 25 | 1,88 | 1,36 | 1,16 | 1,57 | 1,14 | 0,97 | 0,94 | 0,80 |
| 30 | 1,94 | 1,43 | 1,23 | 1,63 | 1,20 | 1,03 | 0,99 | 0,85 |
| 35 | 2,00 | 1,50 | 1,30 | 1,67 | 1,25 | 1,09 | 1,03 | 0,89 |
| 40 | 2,04 | 1,55 | 1,35 | 1,71 | 1,30 | 1,13 | 1,07 | 0,93 |
| 45 | 2,09 | 1,60 | 1,40 | 1,75 | 1,34 | 1,17 | 1,11 | 0,97 |
| 50 | 2,12 | 1,65 | 1,45 | 1,78 | 1,38 | 1,21 | 1,14 | 1,00 |
| 55 | 2,16 | 1,69 | 1,49 | 1,81 | 1,42 | 1,25 | 1,17 | 1,03 |
| 60 | 2,19 | 1,73 | 1,53 | 1,83 | 1,45 | 1,28 | 1,19 | 1,05 |
| 65 | 2,22 | 1,76 | 1,57 | 1,86 | 1,48 | 1,31 | 1,22 | 1,08 |
| 70 | 2,25 | 1,80 | 1,60 | 1,88 | 1,50 | 1,34 | 1,24 | 1,10 |
| 75 | 2,27 | 1,83 | 1,63 | 1,90 | 1,53 | 1,37 | 1,26 | 1,13 |
| 80 | 2,30 | 1,86 | 1,66 | 1,92 | 1,55 | 1,39 | 1,28 | 1,15 |
| 85 | 2,32 | 1,88 | 1,69 | 1,94 | 1,58 | 1,42 | 1,30 | 1,17 |
| 90 | 2,34 | 1,91 | 1,72 | 1,96 | 1,60 | 1,44 | 1,32 | 1,18 |
| 95 | 2,36 | 1,93 | 1,74 | 1,98 | 1,62 | 1,46 | 1,33 | 1,20 |
| 100 | 2,38 | 1,96 | 1,77 | 1,99 | 1,64 | 1,48 | 1,35 | 1,22 |
| 110 | 2,42 | 2,00 | 1,81 | 2,03 | 1,68 | 1,52 | 1,38 | 1,25 |
| 120 | 2,45 | 2,04 | 1,85 | 2,05 | 1,71 | 1,55 | 1,41 | 1,28 |
| 130 | 2,48 | 2,08 | 1,89 | 2,08 | 1,74 | 1,59 | 1,44 | 1,31 |
| 140 | 2,51 | 2,12 | 1,93 | 2,10 | 1,77 | 1,62 | 1,46 | 1,33 |
| 150 | 2,54 | 2,15 | 1,96 | 2,13 | 1,80 | 1,65 | 1,48 | 1,35 |
| 160 | 2,56 | 2,18 | 2,00 | 2,15 | 1,83 | 1,67 | 1,50 | 1,38 |
| 170 | 2,59 | 2,21 | 2,03 | 2,17 | 1,85 | 1,70 | 1,52 | 1,40 |
| 180 | 2,61 | 2,24 | 2,06 | 2,19 | 1,88 | 1,72 | 1,54 | 1,42 |
| 190 | 2,63 | 2,27 | 2,08 | 2,20 | 1,90 | 1,75 | 1,56 | 1,44 |
| 200 | 2,65 | 2,29 | 2,11 | 2,22 | 1,92 | 1,77 | 1,58 | 1,46 |

**Figure 23 - Indicative maximum wind pressure as a function of the building height [12]**

# 4 MATERIAL PROPERTIES

The table below lists the steel and lightweight concrete properties used in the calculations.

| PROPERTY | STEEL | CONCRETE C40 |
|---|---|---|
| Young's modulus E [N/mm²] | 210000 | 10000 (long-term) |
| Shear modulus G [N/mm²] | 80770 | - |
| Strength [N/mm²] | $f_y = 355$ | $f_{cu} = 40$ |

Table 6 - Material properties

In an attempt to reduce as much as possible the dead loads and the size of the load bearing elements, lightweight concrete (approx. 17 kN/m²) has been chosen instead of normal concrete (approx. 24 kN/m²). This lower density is achieved by using lightweight aggregates which are typically expanded shale, clay or slate materials that have been fired in a rotary kiln to develop a porous structure. The structural lightweight concrete mixtures can therefore be designed to achieve similar strengths, mechanical properties and durability as normal concrete.

In buildings, structural lightweight concrete provides a higher fire-rated concrete structure. It also benefits from energy conservation considerations as it provides higher R-values for improved insulation properties.

Finally, even though lightweight concrete is generally more expensive than its normal equivalent, the higher cost is generally offset by size reduction of structural elements, less reinforcing steel and reduced volume of concrete, resulting in lower cost overall.

# 5 FIRE CONSIDERATIONS

Although a full assessment of the behaviour of the structure in fire is beyond the scope of this project, some design decisions were made with this eventuality in mind.

The strength of all materials reduces as their temperature increases and this is particularly true for steel. It begins to lose strength at about 200°C and continues to lose strength at an increasing rate up to a temperature of about 750°C when the rate of strength loss flattens off. This relationship is shown in the figure below.

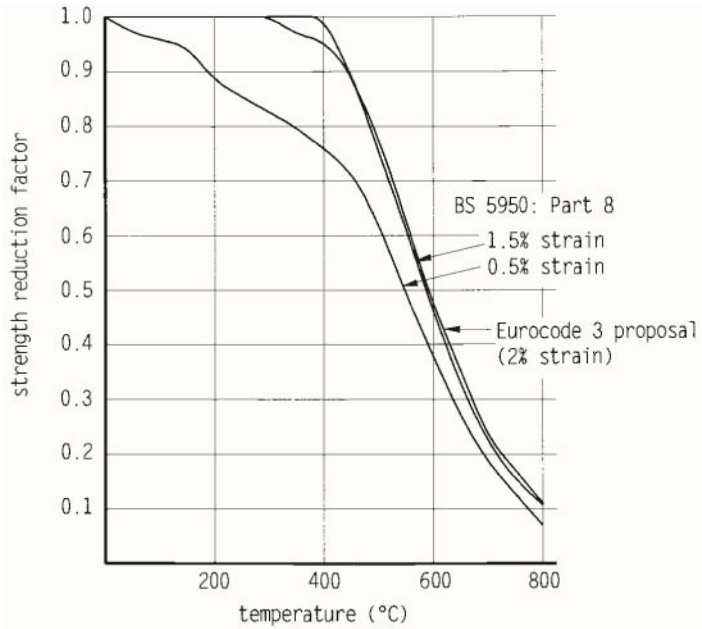


Figure 24 - Strength retention factor for steel at high temperatures [1]

In every country, all buildings therefore have to comply with building regulations which are aimed at reducing the danger for people who are in or around the building when a fire occurs, by containing the fire and ensuring the stability of the structure for sufficient time to allow the occupants to reach safety. The degree of fire resistance required of a structural member is governed by the function, the building height and whether or not sprinklers are installed.

Fire resistance provisions are expressed in units of time: 1, 1.5 and 3 hours in The Netherlands. It is important to note that these times are not allowable escape times for building occupants or even survival times for the structure. They are simply a way of grading different categories of buildings by fire load, from those in which a fire is likely to be relatively small, such as low-rise offices, to those in which a fire might result in a major conflagration.

Various fire protection methods and products are available depending on the type of structural element considered (floor beam or slab, columns, etc) and the fire rating required.

In the present case, the main steel trusses which are located on the inside of the building will be protected with a 2h intumescent paint. When exposed to fire, the paint expands to form a tough char barrier that the fire cannot penetrate, thus keeping the steel at a lower temperature and limiting its deflection under high temperatures.

Spray-applied protection is used around the perimeter of the floor beams. It is the cheapest fire-protection method with the fastest application. However, the appearance is poor and a suspended ceiling will be required also for acoustic performance purposes.

# 6 FLOOR DESIGN

Depending on the main trusses' pattern used, it would be possible to provide main steel beams every 3 or 6m, dictating the span of the floor slabs. These distances correspond approximately to the maximum unpropped spans which can be achieved with slim and deep decks and also give acceptable member angles in the truss to be efficient. Each floor span would require a specific truss

The calculations below have been performed for the worst floor location, i.e. where the live loading and clear span necessary are maximum. This is located in the exhibition hall where a live load of 5kN/m$^2$ and a maximum span of 15m need to be catered for.

## 6.1 Floor Slab Selection

The floor slabs depth were first selected in order to know the magnitude of dead load both during construction and service that the main beams would have to support. This was achieved simply by using the tables provided by Corus, the manufacturer of the steel decking. The spans provided for each case-scenario insure that the stresses and deflections in the slab and deck remain within permissible limits during and after construction.

For both 3m and 6m spans, the selection was made based on the following criteria:

- Single span slab and deck
- 2h fire rating
- No temporary props during construction
- Live load of 5 kN/m$^2$
- Lightweight concrete

## 6.1.1 3m Span

| Props | Span | Fire Rating | Slab Depth (mm) | Mesh | MAXIMUM SPAN (m) Deck Thickness (mm) 0.9 | | | 1.0 Total Applied Load (kN/m²) | | | 1.1 | | | 1.2 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 3.5 | 5.0 | 10.0 | 3.5 | 5.0 | 10.0 | 3.5 | 5.0 | 10.0 | 3.5 | 5.0 | 10.0 |
| No Temporary props | Single span slab & deck | 1 hr | 130 | A142 | 3.5 | 3.2 | 2.3 | 3.6 | 3.3 | 2.3 | 3.7 | 3.4 | 2.4 | 3.9 | 3.4 | 2.5 |
| | | | 130 | A252 | 3.5 | 3.5 | 2.6 | 3.6 | 3.6 | 2.7 | 3.7 | 3.7 | 2.7 | 3.9 | 3.9 | 2.8 |
| | | | 160 | A252 | 3.2 | 3.2 | 2.9 | 3.4 | 3.4 | 3.0 | 3.5 | 3.5 | 3.0 | 3.6 | 3.6 | 3.1 |
| | | 1.5 hr | 140 | A193 | 3.4 | 2.9 | 2.1 | 3.5 | 3.0 | 2.2 | 3.6 | 3.1 | 2.2 | 3.7 | 3.1 | 2.3 |
| | | | 170 | A252 | 3.1 | 3.1 | 2.4 | 3.3 | 3.3 | 2.5 | 3.4 | 3.4 | 2.5 | 3.5 | 3.5 | 2.6 |
| | | 2 hr | 150 | A193 | 2.9 | 2.5 | 1.9 | 3.0 | 2.5 | 1.9 | 3.0 | 2.5 | 1.9 | 3.0 | 2.6 | 1.9 |
| | | | 180 | A252 | 3.1 | 3.0 | 2.1 | 3.2 | 3.0 | 2.1 | 3.3 | 3.0 | 2.2 | 3.5 | 3.0 | 2.2 |
| | Double span slab & deck | 1 hr | 130 | A142 | 3.6 | 3.6 | 2.7 | 3.9 | 3.8 | 2.8 | 4.2 | 3.9 | 2.9 | 4.5 | 3.9 | 2.9 |
| | | | 130 | A252 | 3.6 | 3.6 | 3.2 | 3.9 | 3.9 | 3.2 | 4.2 | 4.2 | 3.3 | 4.5 | 4.5 | 3.3 |
| | | | 160 | A252 | 3.3 | 3.3 | 3.3 | 3.7 | 3.7 | 3.7 | 4.0 | 4.0 | 3.8 | 4.2 | 4.2 | 3.8 |
| | | 1.5 hr | 140 | A193 | 3.5 | 3.5 | 2.6 | 3.8 | 3.6 | 2.6 | 4.1 | 3.6 | 2.7 | 4.1 | 3.6 | 2.7 |
| | | | 170 | A252 | 3.2 | 3.2 | 3.2 | 3.6 | 3.6 | 3.2 | 3.9 | 3.9 | 3.3 | 4.1 | 4.1 | 3.3 |
| | | 2 hr | 150 | A193 | 3.4 | 3.0 | 2.3 | 3.5 | 3.1 | 2.3 | 3.5 | 3.1 | 2.4 | 3.5 | 3.1 | 2.4 |
| | | | 180 | A252 | 3.1 | 3.1 | 2.8 | 3.5 | 3.5 | 2.8 | 3.8 | 3.8 | 2.9 | 4.1 | 3.9 | 2.9 |
| 1 Line of Temporary props | Double span slab | 1 hr | 130 | A393 | 4.6 | 4.1 | 3.2 | 4.7 | 4.2 | 3.3 | 4.8 | 4.3 | 3.3 | 4.8 | 4.3 | 3.4 |
| | | | 160 | 2xA252 | 5.0 | 4.5 | 3.6 | 5.1 | 4.6 | 3.7 | 5.2 | 4.7 | 3.7 | 5.2 | 4.7 | 3.8 |
| | | 1.5 hr | 140 | A393 | 4.1 | 3.7 | 2.9 | 4.1 | 3.7 | 2.9 | 4.2 | 3.8 | 2.9 | 4.2 | 3.8 | 3.0 |
| | | | 170 | 2xA252 | 4.3 | 3.9 | 3.1 | 4.4 | 4.0 | 3.2 | 4.5 | 4.1 | 3.2 | 4.5 | 4.1 | 3.3 |
| | | 2 hr | 150 | A393 | 3.7 | 3.3 | 2.6 | 3.7 | 3.4 | 2.6 | 3.8 | 3.4 | 2.7 | 3.8 | 3.4 | 2.7 |
| | | | 180 | 2xA252 | 3.9 | 3.5 | 2.8 | 3.9 | 3.6 | 2.9 | 4.0 | 3.6 | 2.9 | 4.0 | 3.6 | 2.9 |

Table 7 - Span table for Comflor 60 with lightweight concrete [6]

| Slab Depth (mm) | Concrete volume ($m^3/m^2$) | Weight of Concrete ($kN/m^2$) Normal weight Concrete Wet | Dry | Lightweight Concrete Wet | Dry |
|---|---|---|---|---|---|
| 120 | 0.087 | 2.05 | 2.00 | 1.62 | 1.53 |
| 130 | 0.097 | 2.28 | 2.23 | 1.81 | 1.71 |
| 140 | 0.107 | 2.52 | 2.46 | 1.99 | 1.89 |
| 150 | 0.117 | 2.75 | 2.69 | 2.18 | 2.06 |
| 160 | 0.127 | 2.99 | 2.93 | 2.36 | 2.24 |
| 170 | 0.137 | 3.22 | 3.16 | 2.55 | 2.42 |
| 180 | 0.147 | 3.46 | 3.39 | 2.74 | 2.59 |
| 190 | 0.157 | 3.69 | 3.62 | 2.92 | 2.77 |
| 200 | 0.167 | 3.93 | 3.85 | 3.11 | 2.95 |
| 250 | 0.217 | 5.11 | 5.00 | 4.04 | 3.83 |

Table 8 - Weight of concrete table for various floor slab depths [6]

## 6.1.2 6m Span

| Props | Span | Fire Rating | Slab Depth (mm) | Mesh | MAXIMUM SPAN (m) Total Applied Load (kN/m²) 3.5kN/m² | | | | 5kN/m² Bar Size (mm) | | | | 10kN/m² | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 16 | 20 | 25 | 32 | 16 | 20 | 25 | 32 | 16 | 20 | 25 | 32 |
| No Temporary props | Single span slab | 1 hr | 285 | A142 | 6.5 | 6.5 | 6.5 | 6.5 | 6.0 | 6.5 | 6.5 | 6.5 | 4.7 | 5.7 | 6.2 | 6.5 |
| | | | 320 | A193 | 6.1 | 6.1 | 6.1 | 6.1 | 6.1 | 6.1 | 6.1 | 6.1 | 4.9 | 6.0 | 6.1 | 6.1 |
| | | | 350 | A252 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.1 | 5.8 | 5.8 | 5.8 |
| | | 1.5 hr | 295 | A193 | 6.4 | 6.4 | 6.4 | 6.4 | 5.9 | 6.4 | 6.4 | 6.4 | 4.6 | 5.7 | 6.4 | 6.4 |
| | | | 320 | A193 | 6.1 | 6.1 | 6.1 | 6.1 | 6.0 | 6.1 | 6.1 | 6.1 | 4.8 | 5.9 | 6.1 | 6.1 |
| | | | 350 | A252 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.8 | 5.0 | 5.8 | 5.8 | 5.8 |
| | | 2 hr | 305 | A193 | 5.4 | 6.3 | 6.3 | 6.3 | 4.9 | 6.0 | 6.3 | 6.3 | 3.9 | 4.8 | 5.9 | 6.3 |
| | | | 350 | A252 | 5.6 | 5.8 | 5.8 | 5.8 | 5.1 | 5.8 | 5.8 | 5.8 | 4.1 | 5.1 | 5.8 | 5.8 |
| | | | 400 | A393 | 5.3 | 5.3 | 5.3 | 5.3 | 5.2 | 5.3 | 5.3 | 5.3 | 4.3 | 5.3 | 5.3 | 5.3 |
| 1 Line of Temporary props | Single span slab | 1 hr | 285 | A252 | 6.8 | 7.7 | 7.9 | 8.2 | 6.1 | 7.3 | 7.5 | 7.8 | 4.7 | 5.8 | 6.2 | 6.5 |
| | | | 320 | A393 | 7.0 | 7.5 | 7.5 | 7.5 | 6.3 | 7.5 | 7.5 | 7.5 | 4.9 | 6.1 | 6.8 | 7.2 |
| | | | 350 | 2xA252 | 6.8 | 6.8 | 6.8 | 6.8 | 6.5 | 6.8 | 6.8 | 6.8 | 5.1 | 6.3 | 6.8 | 6.8 |
| | | 1.5 hr | 295 | A393 | 6.7 | 7.8 | 8.1 | 8.3 | 6.0 | 7.4 | 7.6 | 7.9 | 4.6 | 5.8 | 6.7 | 6.7 |
| | | | 320 | A393 | 6.8 | 7.5 | 7.5 | 7.5 | 6.1 | 7.5 | 7.5 | 7.5 | 4.8 | 6.0 | 6.8 | 7.2 |
| | | | 350 | 2xA252 | 6.8 | 6.8 | 6.8 | 6.8 | 6.3 | 6.8 | 6.8 | 6.8 | 5.0 | 6.2 | 6.8 | 6.8 |
| | | 2 hr | 305 | A393 | 5.5 | 6.9 | 8.0 | 8.0 | 5.0 | 6.2 | 7.6 | 8.0 | 3.9 | 4.8 | 6.0 | 6.9 |
| | | | 350 | 2xA252 | 5.7 | 6.8 | 6.8 | 6.8 | 5.2 | 6.4 | 6.8 | 6.8 | 4.1 | 5.1 | 6.3 | 6.8 |
| | | | 400 | 2xA393 | 5.8 | 5.9 | 5.9 | 5.9 | 5.3 | 5.9 | 5.9 | 5.9 | 4.3 | 5.4 | 5.9 | 5.9 |
| | Continuous Slab | 1 hr | 285 | A252 | 7.9 | 8.2 | 8.4 | 8.6 | 7.0 | 7.7 | 7.9 | 8.2 | 5.4 | 6.4 | 6.6 | 7.0 |
| | | | 320 | A393 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 6.0 | 7.0 | 7.3 | 7.5 |
| | | | 350 | 2xA252 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.4 | 6.8 | 6.8 | 6.8 |
| | | 1.5 hr | 295 | A393 | 7.9 | 8.3 | 8.3 | 8.3 | 7.1 | 7.9 | 8.1 | 8.3 | 5.5 | 6.4 | 6.8 | 7.1 |
| | | | 320 | A393 | 7.5 | 7.5 | 7.5 | 7.5 | 7.3 | 7.5 | 7.5 | 7.5 | 5.7 | 6.7 | 7.2 | 7.5 |
| | | | 350 | 2x A252 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.8 | 6.1 | 6.8 | 6.8 | 6.8 |
| | | 2 hr | 305 | A393 | 6.8 | 8.0 | 8.0 | 8.0 | 6.1 | 7.1 | 8.0 | 8.0 | 4.8 | 5.6 | 6.6 | 7.3 |
| | | | 350 | 2xA252 | 6.8 | 6.8 | 6.8 | 6.8 | 6.6 | 6.8 | 6.8 | 6.8 | 5.2 | 6.1 | 6.8 | 6.3 |
| | | | 400 | 2xA393 | 5.9 | 5.9 | 5.9 | 5.9 | 5.9 | 5.9 | 5.9 | 5.9 | 5.7 | 5.9 | 5.9 | 5.9 |

Table 9 - Span table for Comflor 225 with lightweight concrete [6]

| Slab Depth (mm) | Concrete volume ($m^3/m^2$) | Weight of Concrete ($kN/m^2$) Normal weight Concrete Wet | Dry | Lightweight Concrete Wet | Dry |
|---|---|---|---|---|---|
| 285 | 0.116 | 2.74 | 2.68 | 2.17 | 2.05 |
| 290 | 0.121 | 2.85 | 2.79 | 2.26 | 2.14 |
| 295 | 0.126 | 2.97 | 2.91 | 2.35 | 2.23 |
| 300 | 0.131 | 3.09 | 3.02 | 2.45 | 2.32 |
| 305 | 0.136 | 3.21 | 3.14 | 2.54 | 2.41 |
| 310 | 0.141 | 3.32 | 3.26 | 2.63 | 2.49 |
| 320 | 0.151 | 3.56 | 3.49 | 2.82 | 2.67 |
| 350 | 0.181 | 4.27 | 4.18 | 3.38 | 3.20 |
| 380 | 0.211 | 4.97 | 4.87 | 3.94 | 3.73 |
| 400 | 0.231 | 5.44 | 5.33 | 4.31 | 4.08 |

Table 10 - Weight of concrete table for various floor slab depths [6]

### 6.1.3 Floor Slab Selection Summary

The table below summarizes the floor types and corresponding dead loads which are going to be used in the following calculations.

| | | DEAD LOAD | |
|---|---|---|---|
| **Beam spacing [m]** | **Floor type** | **During construction (wet concrete) [kN/m²]** | **During service (dry concrete) [kN/m²]** |
| 3m | ComFlor 60<br>0,9mm thick steel decking<br>180mm deep concrete slab | 2,74 | 2,59 |
| 6m | Comflor 225<br>20mm dia rebars<br>305mm deep concrete slab | 2,54 | 2,41 |

Table 11 - Floor type and dead load summary

## 6.2 Beam Calculations

There are two ways of connecting the steel deck to the supporting steel beams. Each of these has an effect on the total floor depth and the structural behaviour of the beams at various stages of the construction and use.

In the first case, the steel decking is connected to the top flange of the beams via shear studs. By restraining the compression flange, the decking prevents lateral torsional buckling of the beams – a phenomenon explained in a later section - during and after construction, increasing its moment capacity and therefore size required. However, placing the slab on top of the beams contributes to increase the overall floor depth.

Figure 25 - Slab fixed to top flange [6]

The second solution studied is where the slab is placed within the depth of the beams with the intention of reducing the overall floor depth. However, in this case, the beams can be prone to lateral torsional buckling during construction, thus requiring a large section size.

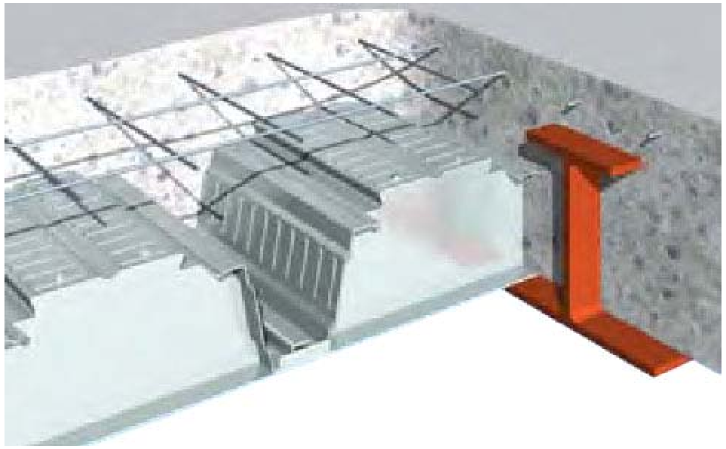

Figure 26 - Slab within beam depth [6]

Both case-scenarios are studied in the following sections for 3m and 6m spans to find out the smallest floor depth which can be achieved.

### 6.2.1 Moment Capacities and Deflections

The minimum moment capacities and maximum allowed deflections of the beams were first calculated during construction and service knowing the dead and live loads as well as the span (15m).

The moment capacity was simply calculated using the formula for a simply supported beam under a uniformly distributed load.

$$M_{\text{midspan}} = \frac{WL}{8}$$

Where:

W: total load
L : beam span

The total load is calculated using eq.1 and the necessary factors of safety. During construction, only the weight of the wet concrete is taken into account whilst during service both the weight of the concrete slab and the live load of 5kN/m². The loading is also dependent on the spacing of the main beams; beams at 3m spacing have to support half the floor area of beams at 6m spacing.

Finally, in all cases, the maximum deflection allowed is L/250, thus 60mm. Values calculated for all cases are presented in the table below.

| | DURING CONSTRUCTION | | DURING SERVICE | |
|---|---|---|---|---|
| Beam spacing [m] | Moment capacity required [kN.m] | Max deflection allowed [mm] | Moment capacity required [kN.m] | Max deflection allowed [mm] |
| 3m | 312,10 | 60 | 927,83 | 60 |
| 6m | 578,64 | 60 | 1814,65 | 60 |

Table 12 - Moment capacities required and allowed max deflections permitted

### 6.2.2 Lateral Torsional Buckling

Open sections such as I-beams have low lateral bending and torsional stiffnesses compared to their vertical bending stiffness. Whenever a slender element is loaded in a stiff plane, there is always a tendency for it to fail in a more flexible plane (lateral torsional buckling). Square and circular beams are not susceptible to lateral torsional buckling.

It is important to remember that during erection the beam may receive less support for example from floors or bracings and this condition may prove to be the critical design case.

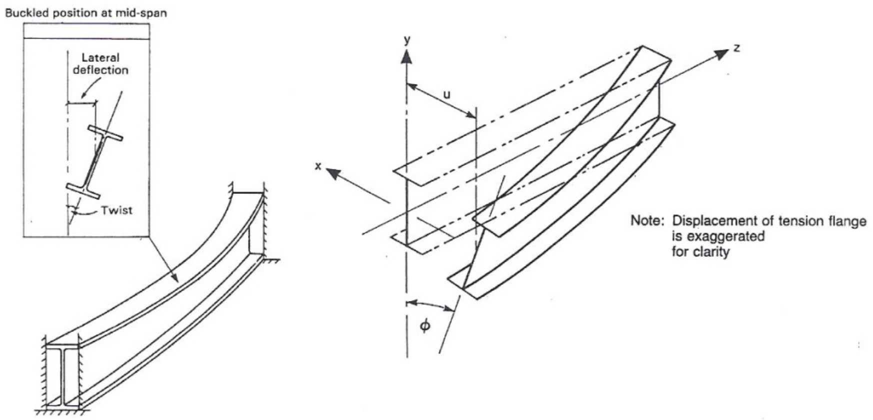


Figure 27 - Lateral torsional buckling [7]

A beam made from an open section and not being fully restrained will have a lower moment capacity and this has to be taken into consideration. Lateral torsional buckling being a fairly complex phenomenon, the best way to introduce it in the calculations of the structures is to follow the method given in the Eurocode 3. The steps are presented below. In order to accelerate the design process and study various solutions, this was programmed in an Excel spreadsheet.

#### 1) *Calculate the elastic critical moment $M_{cr}$*

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \left\{ \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g \right\}$$

Where:

$I_z$ : second moment about the weak axis
$I_t$ : torsion constant
$I_w$ : warping constant
L: beam length between lateral restraints
$Z_g$ : distance between the point of load application and neutral axis
$C_1$: 1,127
$C_2$ : 0,454

#### 2) *Calculate the dimensionless slenderness $\lambda_{LT}$*

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

Where:
$W_y$ : plastic moment of section about the strong axis
$f_y$ : yield strength of steel
$M_{cr}$ : elastic critical moment for lateral torsional buckling

3) ***Calculate the lateral torsional buckling factor***

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{{\Phi_{LT}}^2 - \beta{\bar{\lambda}_{LT}}^2}} \text{ but } \begin{cases} \chi_{LT} \leq 1 \\ \chi_{LT} \leq \dfrac{1}{{\bar{\lambda}_{LT}}^2} \end{cases}$$

$$\Phi_{LT} = 0{,}5 \left[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}) + \beta{\bar{\lambda}_{LT}}^2\right]$$

Where:

$\chi_{LT}$ : lateral torsional buckling factor for rolled sections
$\lambda_{LT0}$ : 0,4
$\beta$ : 0,75
$\alpha_{LT}$ : imperfection factor (table 6.3 EC3, see annex)

4) ***Calculate the moment capacity of the beam***

The moment capacity is then calculated with

$$M_{b,Rd} = \chi_{LT} W_y f_y$$

If there's no possibility of lateral torsional buckling, $X_{LT}$ = 1.

Finally, the beam will be adequate at ULS if its design moment (from table 12) is lower than $M_{b,Rd}$

### 6.2.3 Moment Capacity of Composite Sections

When the slab is placed on the top flange of the supporting beam and fixed to it via shear studs, it acts *compositely* with the beam to support the loads. For this reason, it generally leads to shallower beams and lower deflections than the non-composite equivalent configuration. The Eurocode 4 is used to calculate the moment capacity of composite sections [9].

The first step is to find out where the neutral axis of the composite section lies, in the slab or in the beam. This is done by calculating the strength of each component:

$$R_c = 0{,}45 \cdot A_c \cdot f_{cu}$$

where:

$R_c$: slab strength (force)
$A_c$: area of the concrete slab
$f_{cu}$: concrete strength (stress)

Then the strength of the beam is given by:

$$R_s = 0{,}95 \cdot A_s \cdot f_y$$

where:

$R_s$: beams strength (force)
$A_s$: section area of the beam
$f_y$: steel strength (stress)

### *1) If $R_c > R_s$ the neutral axis lies in the concrete slab*

The moment capacity is then equal to:

$$M_{pl,Rd} = R_s \left(h_f + \frac{h}{2} - \frac{R_s}{R_c}\frac{h_f}{2}\right)$$

Where:

$h_f$ : slab depth
h : beam depth

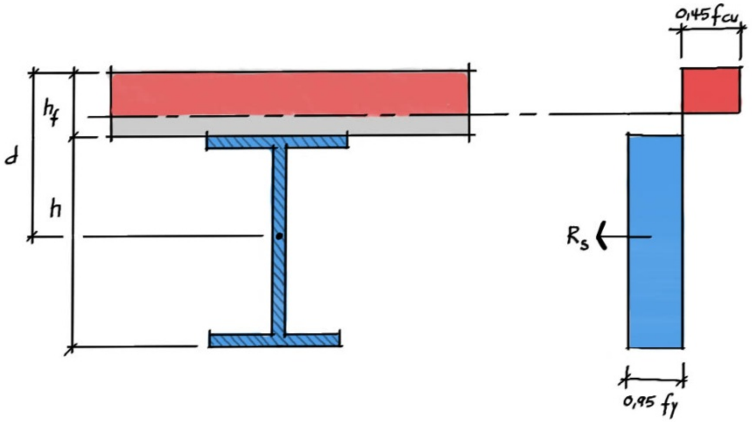


Figure 28 - Neutral axis in the slab

### *2) If $R_c = R_s$ the neutral axis lies at the interface*

$$M_{pl,Rd} = R_s \left(\frac{h}{2} + \frac{h_f}{2}\right)$$

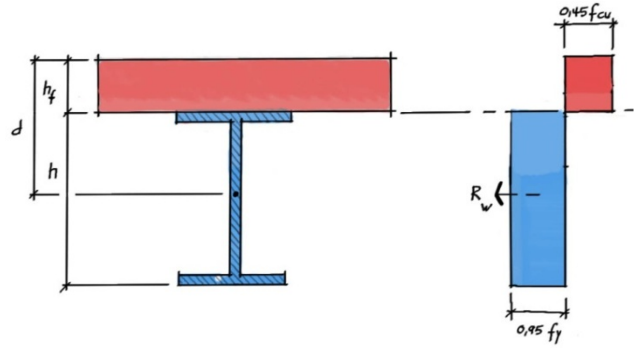


Figure 29 - Neutral axis at interface

### *3) If $R_c < R_s$ & $R_c > R_w$ the neutral axis is in the top flange*

$$M_{pl,Rd} = R_s \frac{h}{2} + R_c \left(\frac{h_f}{2} + \frac{t_f}{2}\right)$$

Where:

$t_f$ : top flange thickness

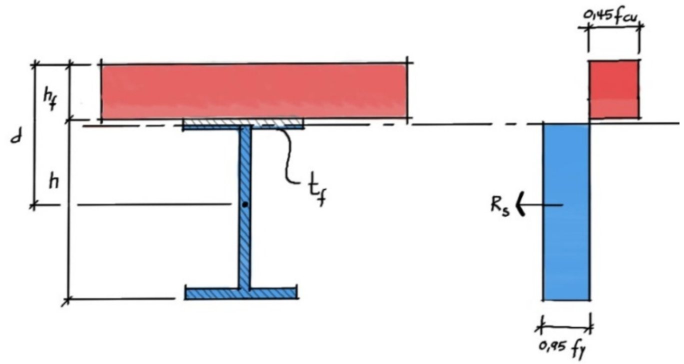


Figure 30 - Neutral axis in the top flange

#### 4) *If $R_c < R_w$ the neutral axis is in the web*

$$M_{pl,Rd} = 0{,}95 f_y W_y + R_c\left(\frac{h}{2} + \frac{h_f}{2}\right) - \frac{R_c{}^2}{R_w}\frac{h}{4}$$

Where:

$W_y$ : steel section plastic modulus about strong axis
$R_w$ : strength of the top flange (force)

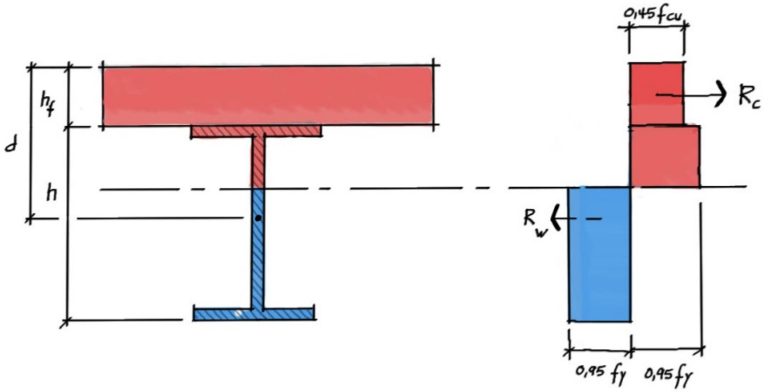


Figure 31 - Neutral axis in the web

### 6.2.4 Deflection of Composite Sections

First of all, in such sections, the effect of *shear lag* has to be taken into account. This phenomenon consists in the actual longitudinal compressive bending stress in the concrete slab varying as shown in the figure below. The maximum value is at the beam/slab intersection and its minimum value midway between beams and this is inconsistent with simple bending theory in which it is assumed that plane sections remain plane.

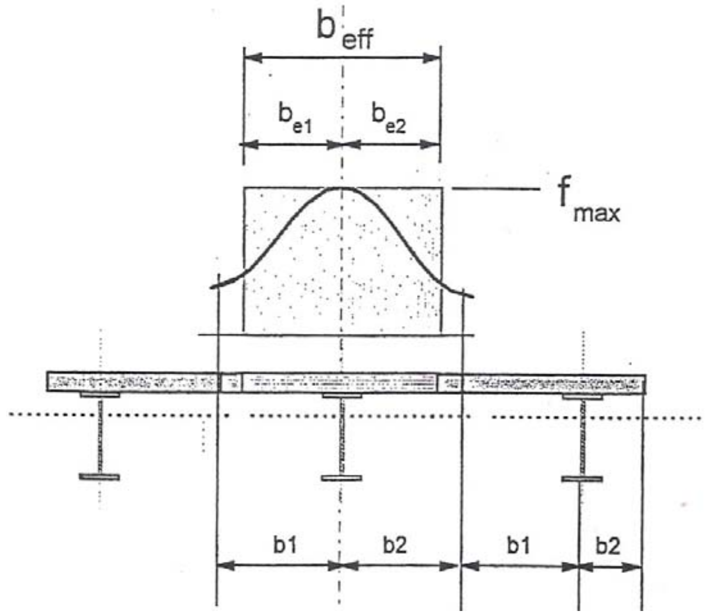


Figure 32 - Compression stress distribution in concrete slab [10]

This variation of bending stress is due to in-plane shear stresses in the slab resulting from the difference of the in-plane stiffness of the slab at the beam positions (well-restrained by the shear connectors) and midway between the girders (where it is less restrained). These differences distort what would otherwise be a uniform distribution of compressive stress.

Eurocode 4 deals with this problem by introducing the concept of effective width $b_{eff}$. It is normally taken as span/8 on each side of the beam (but not greater than half of the distance to the next adjacent beam. Thus for internal beams:

$$b_{eff} = 2 \times L/8$$

The discovery of shear lag was made during testing of steel sections under bending in the elastic range, and it is therefore reasonable to assume that it applies only within that range. The effective width is thus only used for the calculation of the section properties for determining serviceability (deflections) limits only. Otherwise, for moment capacity calculations, the actual slab width is used.

In order to calculate the deflection, the section properties of the combined concrete slab and beam have to be found. One must first find the depth of the neutral axis which may occur in the slab or in the steel beam. This is done by comparing the left had side (LHS) with the right hand side (RHS) of this equation:

$$A_s(d - h_f) = (A_s/\alpha_e)\, h_f/2$$

Where:

d: distance of the beam's neutral axis from top of slab
$A_s$: section area of steel beam
$h_f$ : slab depth
$\alpha_e = E_{steel}/E_{concrete}$ : modular ratio

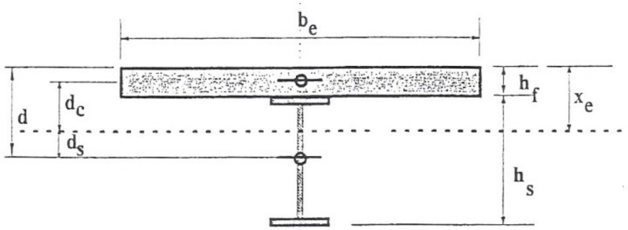

Figure 33 – Definitions

***1) If LHS < RHS the neutral axis lies in the slab***

The depth of the neutral axis is found by solving the following quadratic equation:

$$x_e^2 + Kx_e - Kd = 0$$

$$K = 2\,\alpha_e\, A_s/b_e$$

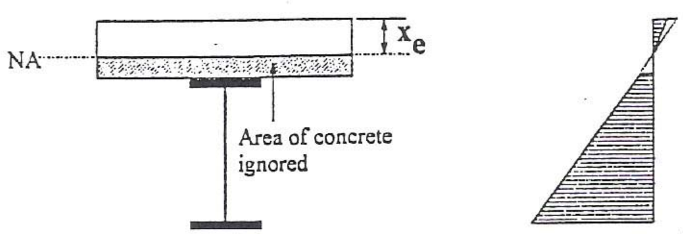

Figure 34 - Neutral axis in the slab

The second moment of area is then equal to:

$$I = I_s + \left(\frac{b_e x_e^3}{3\alpha_e}\right) + A_s\,(d - x_e)^2$$

***2) If LHS > RHS the neutral axis lies in the beam***

$$x_e = \frac{Kd + h_f^2}{K + 2h_f}$$

$$I = I_s + A_s d_s^2 + \frac{I_c}{\alpha_e} + A_c \frac{d_c^2}{\alpha_e}$$

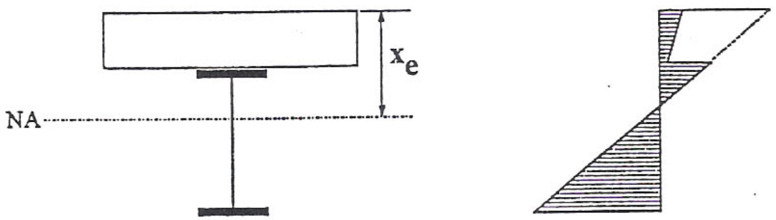


**Figure 35 - Neutral axis in the beam**

Finally, the deflection is calculated with the formula for a simply-supported beam under a uniformly-distributed load:

$$\delta_{midspan} = \frac{5WL^3}{384E_{steel}I}$$

The maximum deflection allowed in beam span/250 = 60mm

## 6.3 Beam Selection – Shear Connection Slab-Beam (Composite action)

In the first case studied, the deck is fixed to the top flange for the steel beams, eliminating the possibility of lateral torsional buckling both during and after construction. With the method described in sections 6.2.1 to 6.2.3, the minimum beam (and floor) depth could be calculated for floor spans of 3m and 6m. For each case, two options are proposed: without and with precamber of the beams. The figure below illustrates this method with a simply supported beam. Without precamber, the final deflection measured is equal to the sum of the deflections due to the dead and live loads. By "pre-bending" the beam with a "deflection" in the opposite direction and of equal intensity to the deflection due to the dead load, we can eliminate the dead load effect. Indeed, after all the dead loads are applied, the beam exhibits no deflection, it is straight. Therefore, as shown in the table below, precambering gives to possibility of using shallower beams. The gain in floor depth versus the increase of workmanship to precamber the beams then has to be assessed in order to make a decision regarding the best option.

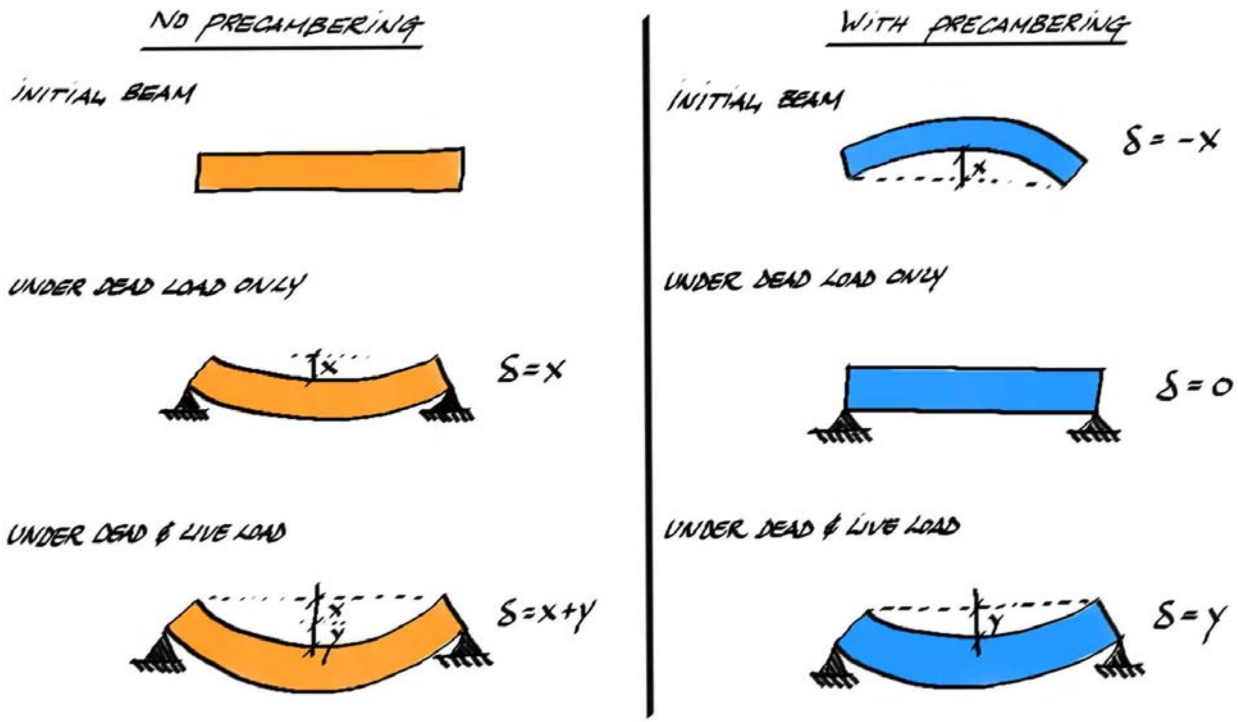


Figure 36 - Precambering method

| Beam spacing [m] | DURING CONSTRUCTION Beam | Moment capacity [kN.m] | Deflection (dead load only) [mm] | Total floor depth [mm] | DURING SERVICE Beam | Moment capacity [kN.m] | Deflection (dead load only) [mm] | Total floor depth [mm] |
|---|---|---|---|---|---|---|---|---|
| 3m | HE 180 M | 313,61 | 344,8 | 380 | HE 180 M | 889,15 | 167,6 (57,2) | 380 |
| | HE 300 M | 1447,69 | 43,6 | 520 | HE 240 M | 1556,48 | 81,1 (27,67) | 450 |
| | | | | | HE 300 M | 2802,09 | 44,9 (15,16) | 520 |
| 6m | HE 240 M | 751,54 | 196,9 | 575 | HE 240 M | 2459,57 | 75,7 (24,6) | 575 |
| | HE 360 M | 1771,10 | 56,4 | 700 | HE 360 M | 4115,87 | 37,5 (12,2) | 700 |

| LEGEND | |
|---|---|
| (red) | Moment capacity too low or deflection too high |
| (orange) | Moment capacity adequate and deflection adequate with precamber |
| (green) | Moment capacity adequate and deflection adequate |

The calculation method is applied in detail for the HE 300 M beam (3m spacing) in section 10.1. The same procedure was used to find the other beam sizes.

## 6.4 Beam Selection – No Shear Connection Slab-Beam

By placing the steel decking within the beams' depth, it could be possible to reduce the overall floor depth. However, because the steel decking is not fixed to the top flange of the beams, lateral torsional buckling can occur during construction before the concrete slab is set. With the present floor configuration, there are two options to prevent lateral torsional buckling: increasing the beam sizes or using a secondary beam which is providing a lateral restraint at midspan of the main beams. Both options have been studied and the results are presented in the table below. Precambering of the beams has also been taken into consideration.

It can be seen that for both 3m and 6m floor spans, placing a transverse secondary beam to prevent lateral torsional buckling during construction is doesn't provide any benefits. Indeed, in all cases, the beam depths have to be increased anyway in order to have a sufficient moment capacity or acceptable deflection during service. As predicted, a shallower floor is obtained with a span of 3m instead of 6m.

| | | DURING CONSTRUCTION | | | | DURING SERVICE | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Beam spacing [m]** | **Lateral restraint [m]** | **Beam** | **Moment capacity [kN.m]** | **Deflection (dead load only) [mm]** | **Floor depth [mm]** | **Beam** | **Moment capacity [kN.m]** | **Deflection (dead load only) [mm]** | **Floor depth [mm]** |
| **3m** | **No lateral restraint** | HE 220 M | 331,69 | 176,6 | 240 | HE 220 M | 503,75 | 489,5 (167,5) | 240 |
| | | HE 300 M | 1126,35 | 43,6 | 340 | HE 300 M | 1447,69 | 120,7 (41,2) | 340 |
| | **1 lateral restraint (midspan)** | HE 220 M | 428,84 | 176,6 | 240 | HE 360 M | 1771,1 | 84,2 (28,7) | 395 |
| | | HE 300 M | 1126,35 | 43,6 | 340 | HE 450 M | 2247,5 | 54,3 (18,5) | 478 |
| **6m** | **No lateral restraint** | HE 260 M | 652,9 | 152,8 | 290 | HE 240 M | 751,54 | 574,5 (186,7) | 270 |
| | | HE 360 M | 1283,1 | 56,4 | 395 | HE 260 M | 896,02 | 445,7 (145) | 290 |
| | | | | | | HE 360 M | 1771,09 | 164,4 (53,5) | 395 |
| | **1 lateral restraint (midspan)** | HE 240 M | 664,97 | 197,0 | 270 | HE 400 M | 1977,71 | 134,1 (43,6) | 432 |
| | | HE 360 M | 1283,10 | 56,4 | 395 | HE 500 M | 2518,37 | 86,2 (28,0) | 572 |
| | | | | | | HE 600 M | 3114,06 | 58,79 (19,11) | 620 |

| LEGEND | |
|---|---|
| Red | Moment capacity too low or deflection too high |
| Orange | Moment capacity adequate and deflection adequate with precamber |
| Green | Moment capacity adequate and deflection adequate |

Detailed calculation procedure presented in section 10.2 for beam HE 450 M with 3m spacing.

## 6.5 Summary

Table 13 summarizes the possible floor solutions with and without precamber of the beams for floor spans of 3m and 6m. For all cases, precambering the beams leads to a floor depth reduction of 8 to 18%. The shallowest floor (395mm) is obtained for a span of 3m when the steel decking is placed within the beams' depth. This solution requires the maximum workmanship; the beams have to be precambered and the 3m spacing doubles the number of beams to be installed. Having the floor slab between the beams also reduces the space available to the service ducts.

If a depth of 395mm for the top and bottom chords of the main structure trusses (to which to floor beams connect) is not possible, the floor depth is then not the driving element for the overall visual aspect of the building structure. In this case, it does not make sense to involve more workmanship to get a shallower floor. We might then opt for a 6m span with decking on top of the beams without precambering, which gives a depth of 700mm and maximum space for the service ducts. The final decision regarding the floor solution will therefore be made when the truss pattern and truss member sizes will be known.

| | | WITHOUT PRECAMBER | | | WITH PRECAMBER | | | |
|---|---|---|---|---|---|---|---|---|
| **Floor construction type** | **Beam spacing [m]** | **Beam** | **Maximum deflection during service [mm]** | **Floor depth [mm]** | **Beam** | **Precamber at construction stage [mm]** | **Maximum deflection during service [mm]** | **Floor depth [mm]** |
| With shear connection | 3m | HE 300 M | 44,9 | 520 | HE 240 M | 27,7 | 53,4 | 450 |
| | 6m | HE 360 M | 37,5 | 700 | HE 240 M | 24,6 | 51,1 | 575 |
| Without shear connection | 3m | HE 450 M | 54,3 | 478 | HE 360 M | 28,7 | 55,4 | 395 |
| | 6m | HE 600 M | 59,8 | 620 | HE 500 M | 28,0 | 58,2 | 572 |

Table 13 - Floor configurations summary

# 7 MAIN TRUSSES DESIGN

The approach used to design the main trusses was to start with a simple model of a cantilevered beam with various loading conditions and then to gradually increase the complexity of the problem until the whole structure of the building was considered.

The studies have been made with Autodesk's Robot Structural Analysis Professional 2011, which allows to perform comprehensive analysis of large and complex structures.

## 7.1 Load Cases and Corresponding Bending Moment and Shear Force Diagrams of Cantilevered Beam

The magnitude of the bending moments and shear forces in a truss as well as its support reactions will vary depending on the loading distribution along its length. Obviously the dead loads will be constant, but the live loads will vary in position during the use of the building. Some loading patterns will cause higher bending moments and shear forces than others.

The bending moments and shear force diagrams have been calculated for the three main loading cases on a simply supported cantilevered beam and these will be then used to make initial estimations of the member sizes for more detailed analysis.

In all cases, the truss has to support 2 times 20m-width of floors. A value of 2,6 kN/m$^2$ has been used as dead load and 5,0 kN/m$^2$ for live loads. The cantilever and backspan both measure 42m. Factors of safety of 1,35 and 1,5 have been used for dead and live loads respectively.

Hence:

Dead Load UDL = (1,35 x 2,6) x 20 x 2 = 140 kN/m
Live Load UDL = (1,5 x 5) x 20 x 2 = 300 kN/m

### 7.1.1 Load Case 1: Live Load Along Whole Length

With a live load and dead load along the whole length of the beam, we obtain the following bending moment and shear force diagrams. Forces are in kN and moments in kN.m.

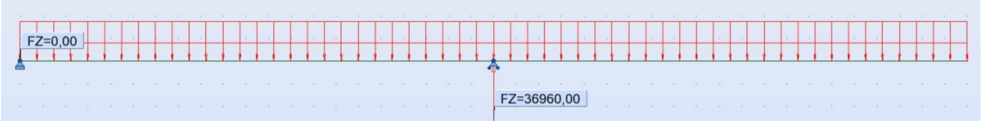


Figure 37 - Support reactions for load case 1

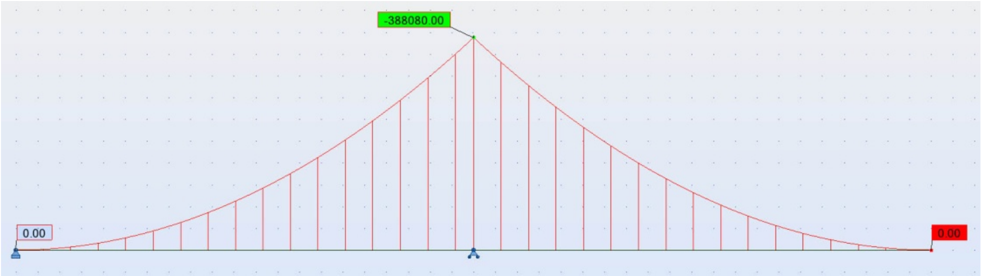


Figure 38 - Bending moment diagram for load case 1


Figure 39 - Shear force diagram for load case 1

### 7.1.2 Load Case 2: Live Load on Cantilever Only

For the second load case, the live load is placed only on the cantilever.

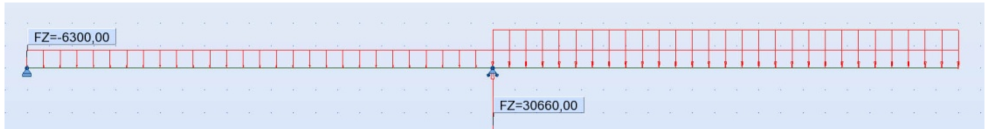


Figure 40 - Support reactions for load case 2

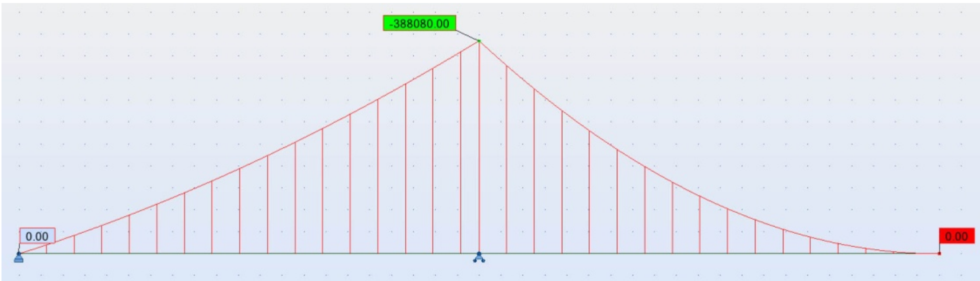


Figure 41 - Bending moment diagram for load case 2

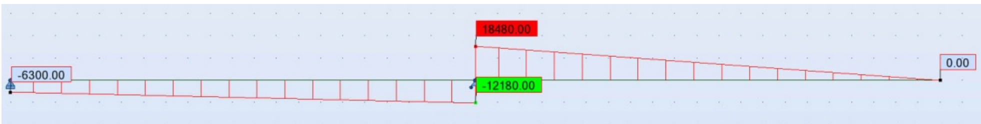


Figure 42 - Shear force diagram for load case 2

### 7.1.3 Load Case 3: Live Load on Backspan Only

Finally, for the last load case, the live load in located on the backspan only.

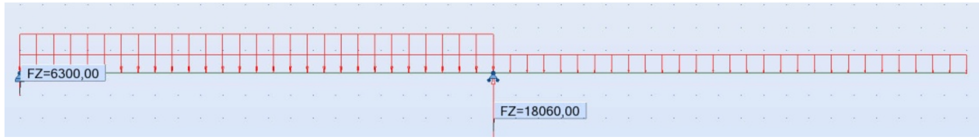


Figure 43 - Support reactions for load case 3

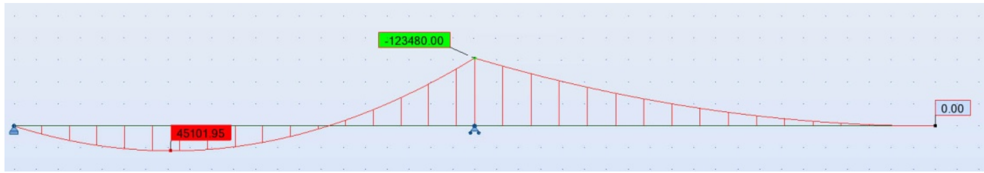


Figure 44 - Bending moment diagram for load case 3

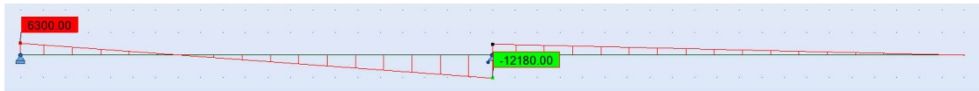


Figure 45 - Shear force diagram for load case 3

### 7.1.4 Summary

The table below presents the reactions as well as the critical bending moments and shear forces for all three load cases studied.

| | LOAD CASE | FRONT SUPPORT | BACK SUPPORT | BACKSPAN |
|---|---|---|---|---|
| Reaction [kN] | 1 | 36960 | 0 | N/A |
| | 2 | 30660 | -6300 (uplift) | N/A |
| | 3 | 18060 | 6300 | N/A |
| Bending moment [kN.m] | 1 | -388080 (hogging) | 0 | Varies (hogging) |
| | 2 | -388080 (hogging) | 0 | Varies (hogging) |
| | 3 | -123480 (hogging) | 0 | 45102 (sagging) |
| Shear force [kN] | 1 | ±18480 | 0 | Varies (-) |
| | 2 | 18480 | -6300 | Varies (-) |
| | 3 | -12180 | 6300 | Varies (+ and -) |

Table 14 - Reactions, bending moments and shear forces for all load cases

Firstly, the maximum reaction is always located in the front supports, near the cantilever; the highest value being reached when full live and dead loads are applied on both the cantilever and backspan. For all load cases, the front supports will be in compression, which is the desired situation. However, an uplift force can be found on the back supports when the live load is applied on the cantilever only. This would require anchoring the foundations at this location - a more expensive and labour-intensive solution – which should ideally be avoided. When refining the analysis, the live loads will be applied more precisely according to the type of space and this might correct this problem. If not, further actions could be taken such as increasing the weight of the back cores.

Then, as expected, for all load cases, the maximum moment and shear forces are also located at the front supports. It is important to note that although a hogging moment is always found on the cantilever, the backspan can experience both hogging and sagging moments depending on the load case. This will have implications on the type of truss pattern which is going to be the most efficient, thus requiring the smallest member sizes and leads to the smallest deflections.

The relationship between the load case, truss pattern, forces in members and deflections is therefore studied in the next section.

## 7.2 Main Trusses Pattern Selection

The aim of this study is to determine how the truss pattern may influence the forces in its members and the deflection magnitude. Once again, a cantilever of 42m with a backspan of the same length was used. This corresponds to the largest cantilever in the actual building. The same loadings as for the cantilevered beam were used (2,6 kN/m$^2$ dead load from floor and 5 kN/m$^2$ live load). These loadings were applied as equivalent nodal forces on the truss, explaining the slight discrepancies in the reaction magnitudes. Only when the live load is placed solely on the cantilever (loadcase 2) was considered as this lead to the maximum bending moment, shear force and deflection.

The truss has a 5m-height and bays of 3m or 6m-length, which corresponds to the possible floor spans.

### 7.2.1 Howe Truss – 6m Bays

The Howe truss was chosen as is can be designed in such a way that the diagonals are always in tension and can therefore have a minimal section area.

Initial calculations were made manually to find member sizes which could then be adjusted to meet the final stress and deflection requirements if required.

From the previous cantilevered beam study, we have:

$M_{max}$ = 388080 kN.m
$V_{max}$ = 30660 kN

Knowing that the height of the truss is h = 5m, the maximum compression force in the chord at the front support can be approximated by:

$F_{max} = M_{max} / h = 388080 / 5 = 77616$ kN

Then, the member should have a sufficient size to prevent buckling and yield of steel. The minimum second moment of area I required to prevent buckling is (from the Euler formula):

$$F_{max} = \frac{\pi^2 EI}{{L_e}^2}$$

Where:

E: Young's modulus of steel = 210 GPa
$L_e$ : effective length of chord = 1 x 6m = 6m (pinned-pinned)

Hence:

$$I_{min} = \frac{7616000 \text{ x } 6^2}{\pi^2 \text{ } (210 \text{ x } 10^9)} = 1{,}34 \text{ x } 10^{-4} \text{ m}^4$$

To prevent yield of steel, the minimum section area is:

$$A_{min} = \frac{F_{max}}{f_y} = \frac{77616000}{355} = 218637 \text{ mm}^2$$

Let's say we use a square section of 900x900mm with a 70mm thick wall, which has the following section properties:

A = 232400 $mm^2$
I = 2,68 x $10^{-2}$ $m^4$

Knowing that the diagonals are at an angle of 39.8 degrees fom the horizontal, the maximum tensile force in a diagonal is:

$$F_{diag} = V_{max} / \sin 39{,}8 = 47928 \text{ kN}$$

The area required to prevent yield is:

$$A_{min} = \frac{F_{diag}}{f_y} = \frac{47928000}{355} = 135008 \text{ mm}^2$$

This gives a solid rod of diameter 415mm

Using these section properties, we get the following results:

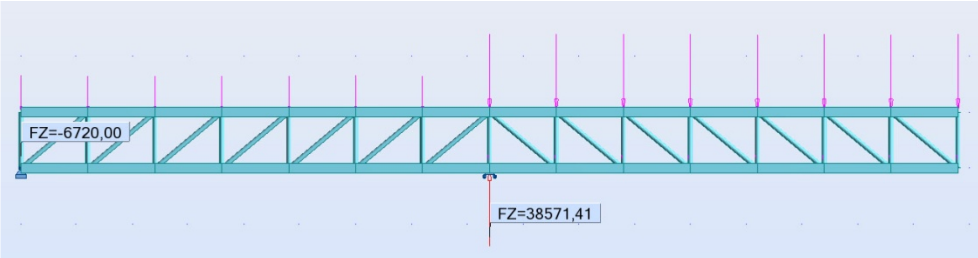


Figure 46 - Support reactions for Howe truss with 6m-bays

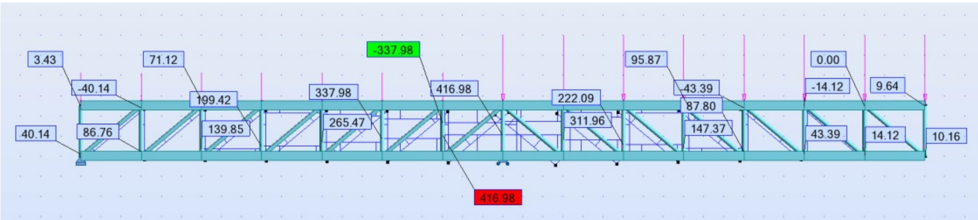


Figure 47 - Maximum member stresses for Howe truss with 6m-bays

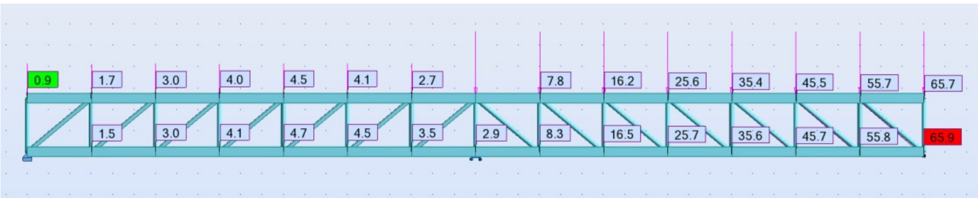


Figure 48 - Displacements for Howe truss with 6m-bays

The stresses are slightly too high but the members sections will be kept as it is for this study. They will be adjusted when two trusses are tied together at their ends, resulting in different stresses and deflections.

### 7.2.2 Howe Truss – 3m Bays

The bay size was then reduced by half to 3m in order to see if this resulted in a significant reduction in stresses and deflection.

The top and bottom chords remain the same. However, the angle of the diagonals being increased to 59 degrees, we have a new tensile force of:

$$F_{diag} = V_{max} / \sin 59 = 35769 \text{ kN}$$

The area required to prevent yield is:

$$A_{min} = \frac{F_{diag}}{f_y} = \frac{35769000}{355} = 100758 \text{ mm}^2$$

This gives a solid rod of diameter 360mm

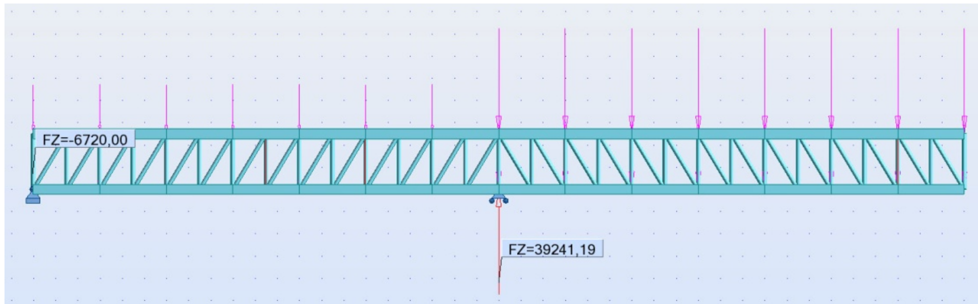


Figure 49 - Support reactions for Howe truss with 3m-bays

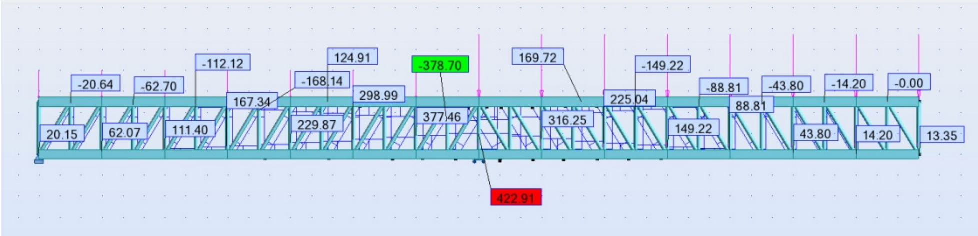


Figure 50 - Maximum member stresses for Howe truss with 3m-bays

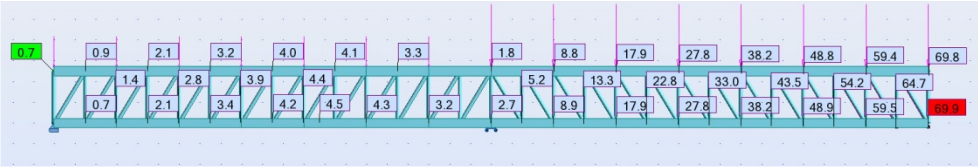


Figure 51 - Displacements for Howe truss with 3m-bays

The maximum axial stresses in the chords are almost the same and the deflection is slightly increased by doubling the number of diagonal elements. It is therefore preferable to use 6m bays which will require a lot less workmanship and gives a lighter aspect.

### 7.2.3 Diamond Truss – 6m Bays

The diamond pattern which has diagonals both in tension and compression was also studied. This pattern gives the possibility of using 6m bays but have connection points every 3m where the floor beams can be connected. This way, a shallower and lighter floor can be used.

As initial estimation, the maximum bending moment and shear force are still:

$M_{max}$ = 388080 kN.m

$V_{max}$ = 30660 kN

The height of the truss remaining at 5m, the top and bottom chord sections calculated for the Howe truss can still be used (900mm x 900mm x 70mm thick).

Then, the diagonals should have a sufficient size to prevent buckling and yield of steel. The maximum compression force is calculated by:

$$F_{diag} = V_{max}/ \sin 39{,}8 = 47928 \text{ kN}$$

The minimum second moment of area I required to prevent buckling is (from the Euler formula):

$$F_{max} = \frac{\pi^2 EI}{{L_e}^2}$$

Where:

E: Young's modulus of steel = 210 GPa
$L_e$ : effective length of diagonal = 1 x 7,8m = 7,8m (pinned-pinned)

Hence:

$$I_{min} = \frac{47928000 \text{ x } 7{,}8^2}{\pi^2 \text{ } (210 \text{ x } 10^9)} = 1{,}41 \text{ x } 10^{-3} \text{ m}^4$$

To prevent yield of steel, the minimum section area is:

$$A_{min} = \frac{F_{max}}{f_y} = \frac{47928000}{355} = 135008 \text{ mm}^2$$

Let’s say we use a solid rod of diameter 415mm of with the following section properties:

A = 135265 $mm^2$
I = 1,45 x $10^{-3}$ $m^4$

Using these section properties, we obtain the following results:

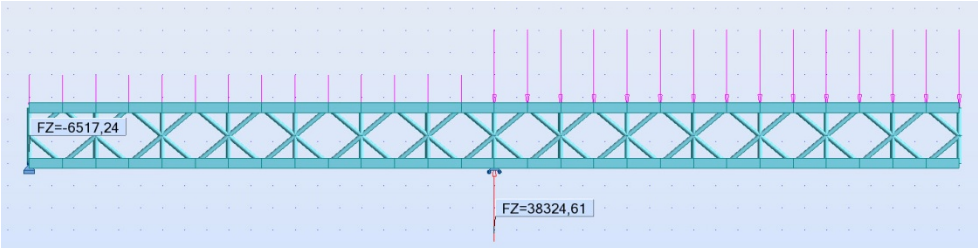


Figure 52 - Support reactions for Diamond truss with 6m-bays

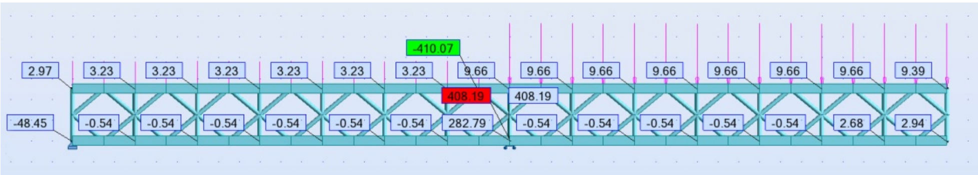


Figure 53 - Maximum member stresses for Diamond truss with 6m-bays

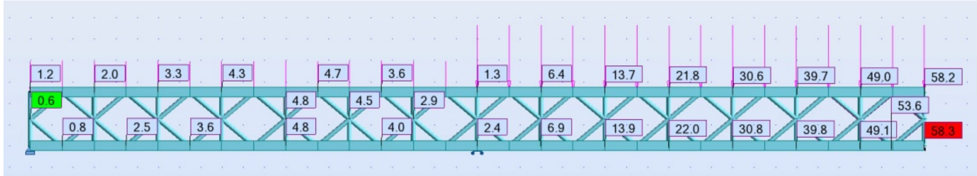


Figure 54 - Displacements for Diamond truss with 6m-bays

Compared with the 6m-bay Howe truss, the deflection is reduced by approximately 8%.

## 7.2.4 Diamond Truss – 3m Bays

As for the Howe truss, we reduce the bay size to 3m. The member sizes remain unchanged.

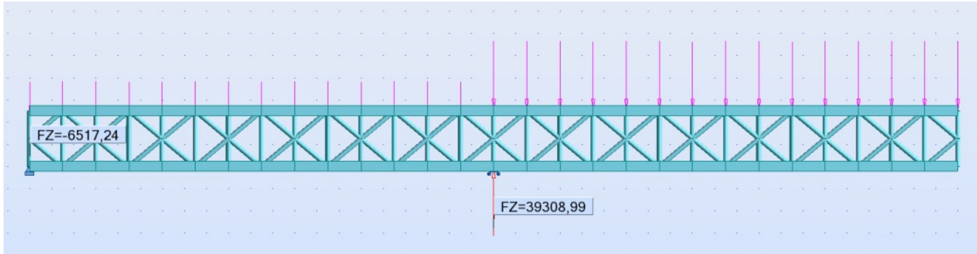


Figure 55 - Support reactions for Diamond truss with 3m-bays

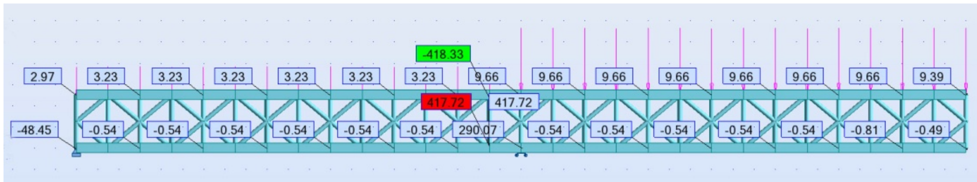


Figure 56 - Maximum member forces for Diamond truss with 3m-bays

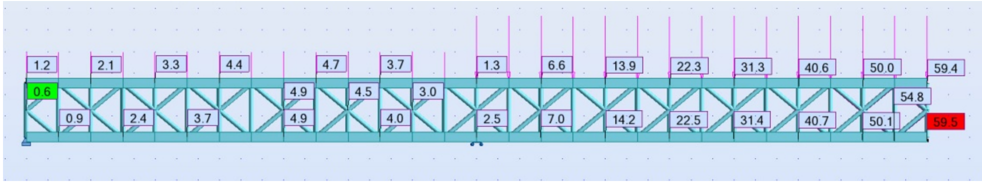

Figure 57 - Displacements for Diamond truss with 3m-bays

Reducing the bay size by half has no positive effect. The stresses and deflection even increase due to the added weight of the additional members.

### 7.2.5 Pattern Adopted and Final Floor Selection

A diamond truss with 6m bays was adopted as it presents deflections which are slightly lower than the Howe truss and is also presents a more uniform pattern visually. The 6m diamond truss also offers the possibility to connect floor beams every 3m, reducing the floor slab depth necessary.

Having a first approximation of the chord depth, it is possible to make a final decision regarding the floor structure. The chord being fairly large with a depth of 900mm, it is not necessary to choose the shallowest floor. We can therefore select the cheapest solution which would be HE 300 M beams every 3m with ComFlor60 decking fixed to the top flange with shear studs and a 180mm-deep concrete slab.

## 7.3 Combined Trusses

Even though the diamond pattern requires more workmanship than the Howe truss in comparison to the deflection reduction, it was selected due based on its more homogeneous appearance and the possibility of using smaller panes of glass for the façade.

Further refinement of the member sizes has been done by linking two trusses has it will be found in the final structure. This simplified model allowed for quick amendments thanks to very short calculation times.

Firstly, in order to assess the benefits of combining trusses, the same members were used as previously, i.e. 900mm x 900mm x 70mm for top and bottom chords and diameter 415mm rod for the diagonals. The same loadings were used. The results are illustrated in the following figures.

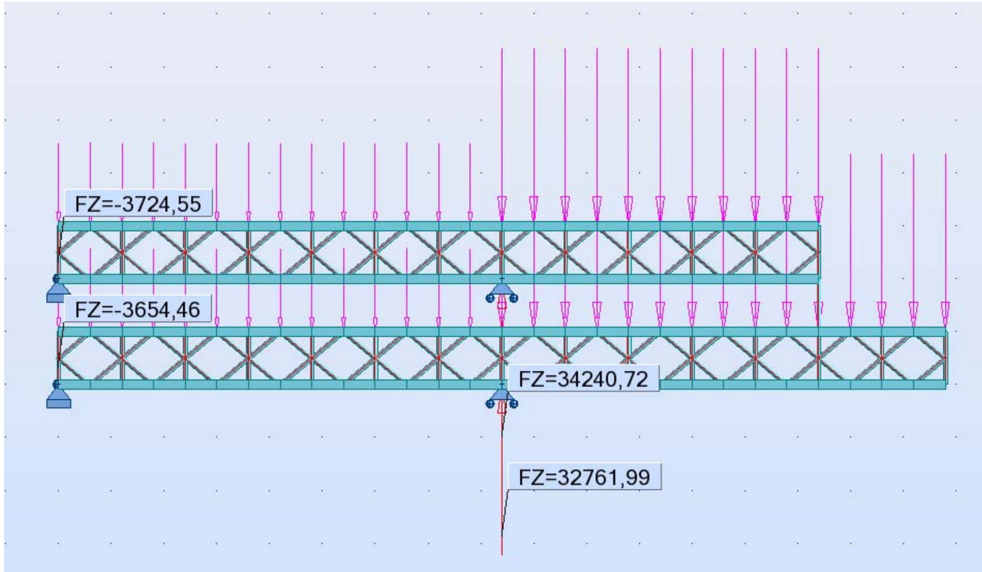

Figure 58 - Support reactions for linked trusses with initial

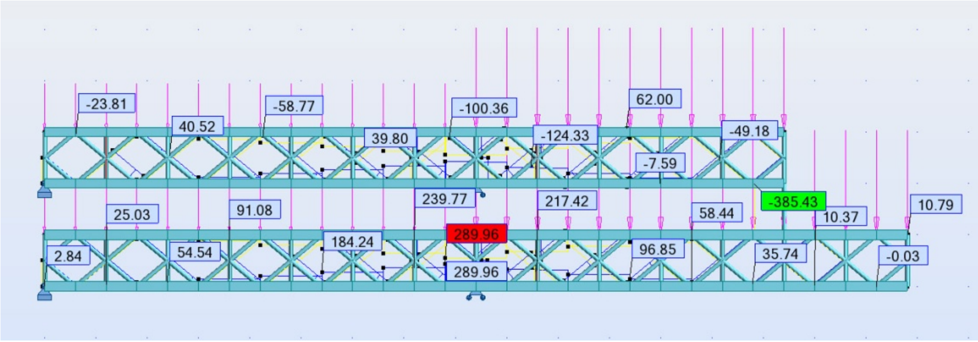

Figure 59 - Maximum member axial stresses for linked trusses

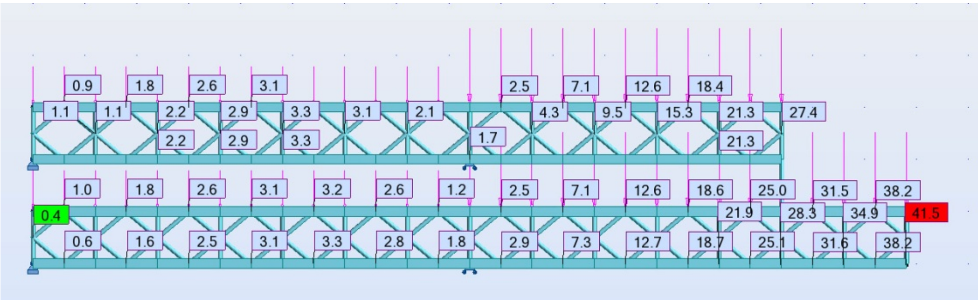

Figure 60 - Total deflection for linked trusses

By linking two trusses at their ends, the maximum stresses are reduced as well as the deflection, which is reduced by nearly 30%. A way to further reduce the deflection within the limit of 0.003(2 x cantilever length) which is 25,2cm is to precamber the trusses in the shape opposite to the deflection profile under dead load only. This corresponds to considering the deflection due to live load only. If we do so, we obtain the following maximum deflection:

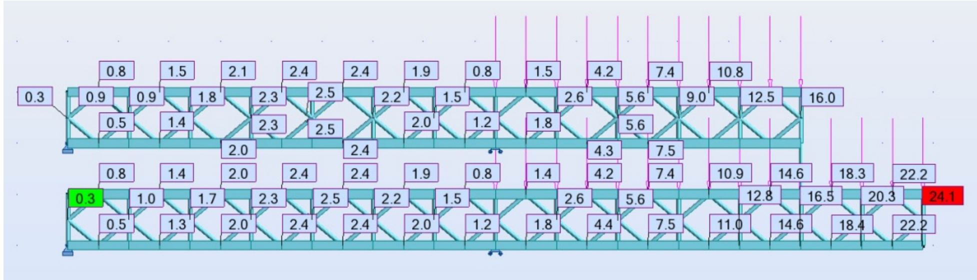

Figure 61 - Total deflection for precambered linked trusses

The top and bottom chords are further refined to reduce the plate thickness used and give a more streamlined profile to the structure. By using a tapered profile for the bottom chord which is 1,8m deep at the front support where the bending moment is maximal, it is possible to reduce the plate thickness to 55mm and still have acceptable stresses and deflections.

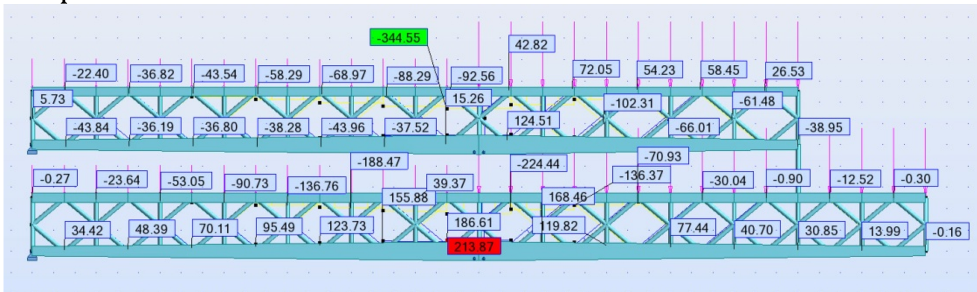

Figure 62 - Stress distribution with tapered bottom chord

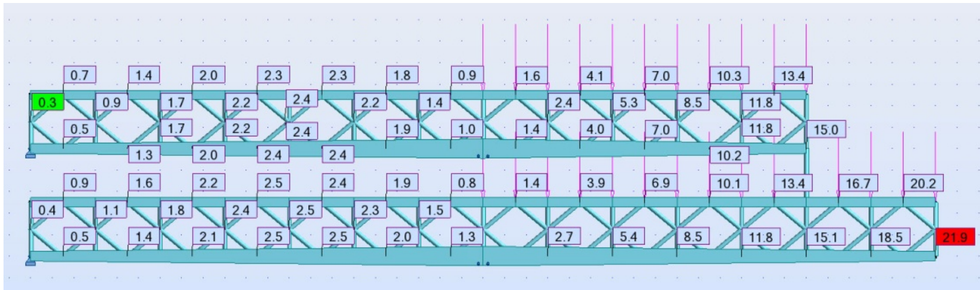

Figure 63 - Maximum deflection with tapered bottom chord

## 7.4 Whole Cantilevered Structure Check

With the members sizes calculated in the previous section, the whole cantilevered structure was checked for stresses and deflection. In this case, the live loads were applied more precisely according to the internal functions based on the values in table 4.

### 7.4.1 Initial Member Sizes

#### *7.4.1.1 Axial Stresses*

For all load cases the maximum tensile stress is located at the front support where the maximum bending moment is located. A maximum tensile stress of 241,19 MPa was calculated for when the live load is placed on the cantilever only, which is well below the steel's yield strength of 355 MPa.

The structure is therefore overdesigned at ultimate and service limit states. Because the chords (floors' edge beams) have a minimum depth of 900mm, we can use a deeper floor which does not require precambering and fix the steel decking to the top flange of the beams with shear studs. This gives a floor depth of 520mm with a span of 3m. This option also offers the maximum space for the building services ducts.

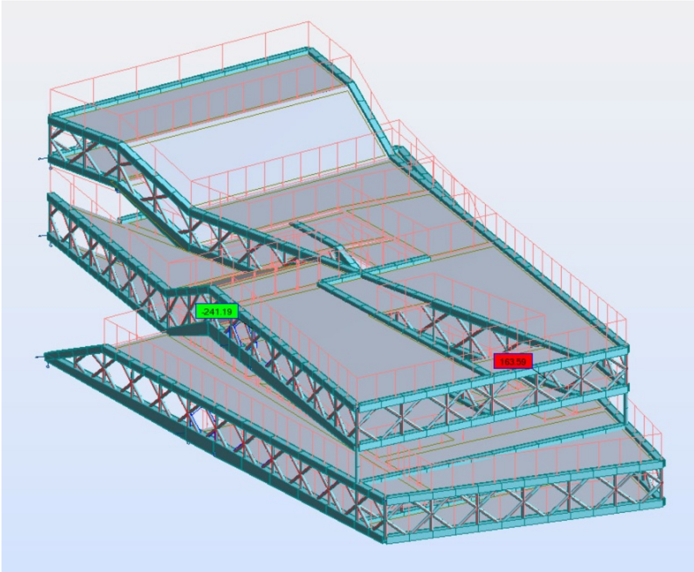

**Figure 64 - Maximum stress when live load applied on cantilever and backspan**

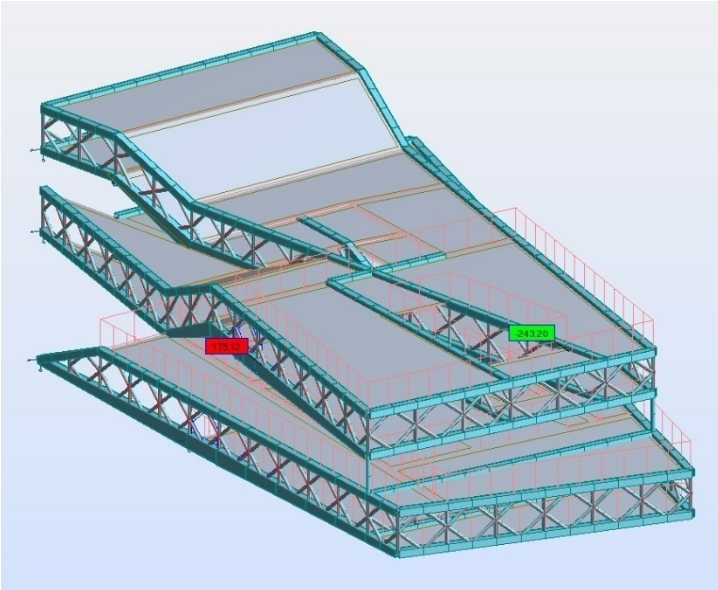

**Figure 65 - Maximum stress when live load on cantilever only**

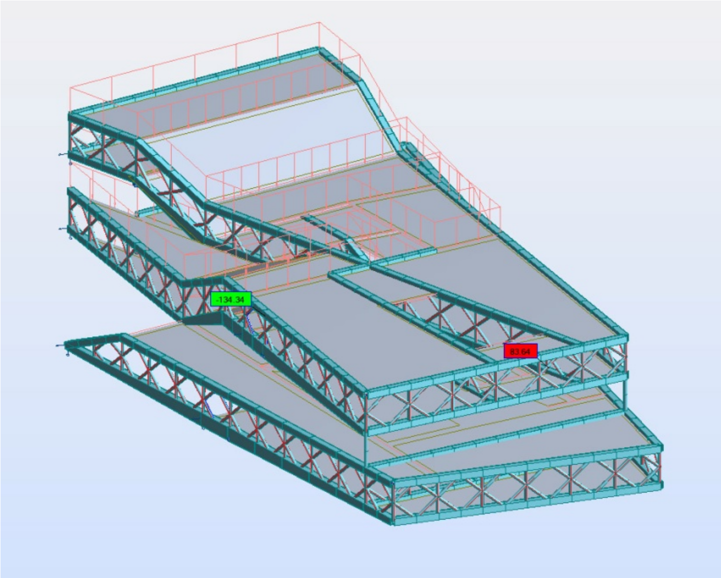

Figure 66 - Maximum stress when live load on backspan only

## 7.4.2 Deflection

As mentioned previously, a maximum deflection of 25,2cm is allowed at the tip of the cantilever. A deflection of 23,4cm when the live load is applied on the cantilever only, within the limit prescribed by the Eurocode. By precambering the trusses to compensate for the dead load, it is possible to reduce this deflection to 10 cm.

Because the stresses and potentially the deflections are below the limits, an attempt was made to reduce the thickness and size of the members. This would lead to a lighter and cheaper structure.

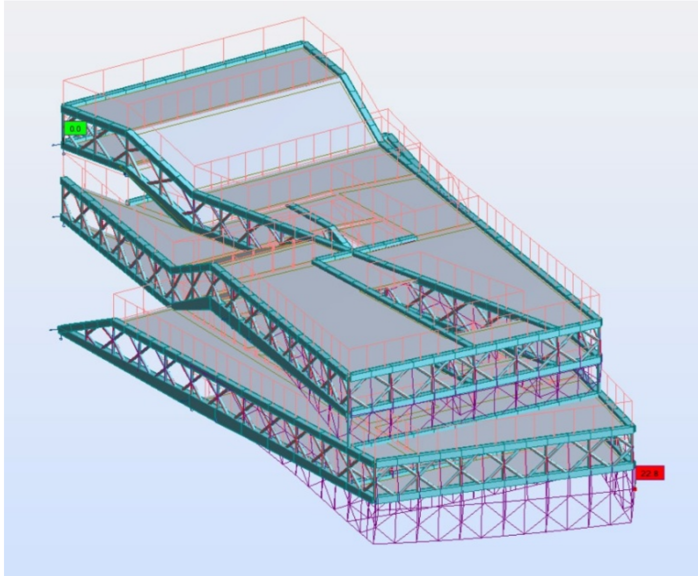

Figure 67 - Total deflection with live load on cantilever and backspan

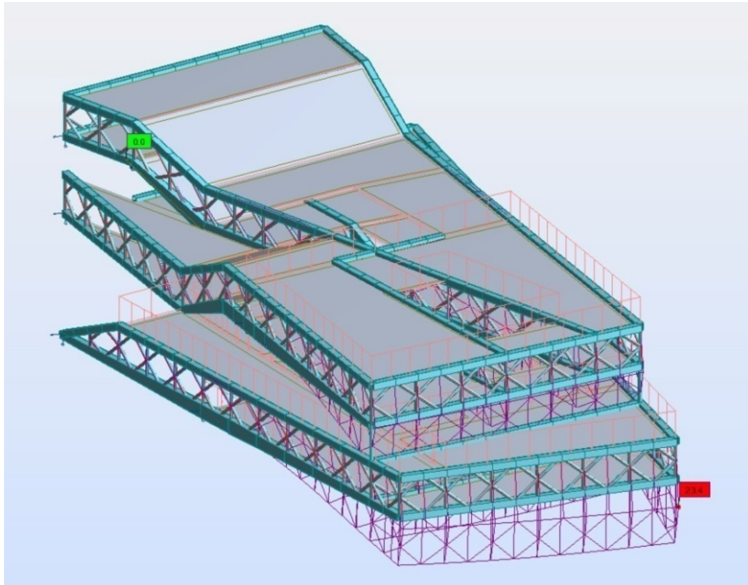

Figure 68 - Total deflection with live load on cantilever only

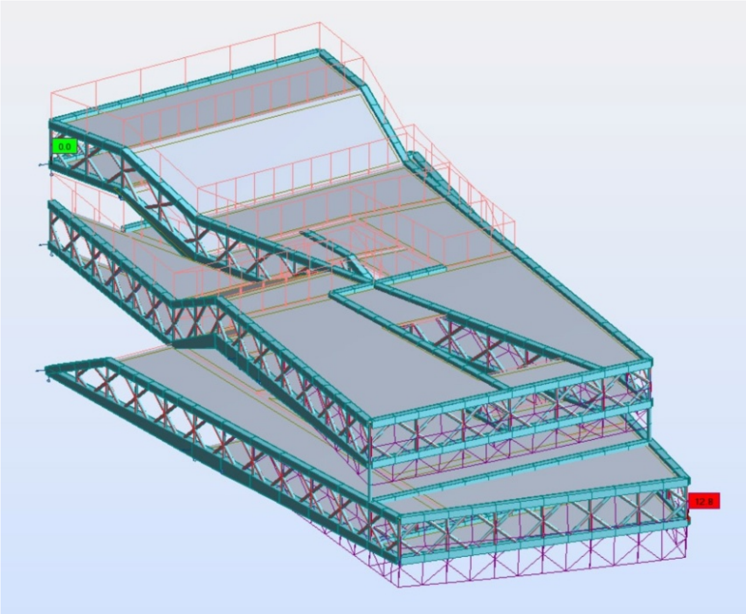

Figure 69 - Total deflection with live load on backspan only

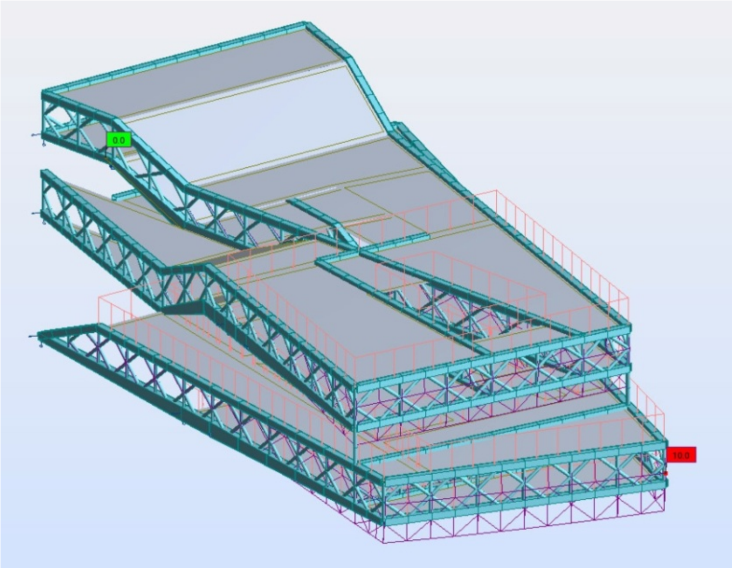

Figure 70 - Maximum deflection with precambering and live load on cantilever only

### 7.4.3 Structure with Reduced Member Sizes

#### *7.4.3.1 Axial Stresses*

The 425mm-diameter diagonals were replaced by 300mm-diameter hollow steel tubes with 40mm thick walls. The outside dimensions of the top and bottom chords were kept the same, but the plate thickness was reduced by nearly half to 40mm. This will greatly reduce the stresses and deflection due to the self-weight of the structure. On the other hand, the smaller cross sections areas should increase the stresses and deflections.

The figures below present the maximum axial stresses calculated for all load cases. Once again, the maximum stresses are located at the front support and have the largest magnitude when the live load is placed on the cantilever only. In this particular case, a maximum tensile stress of 337,12 MPa was obtained, very near the yield stress of 355 MPa. The structure is therefore not overdesigned from a stress point of view.

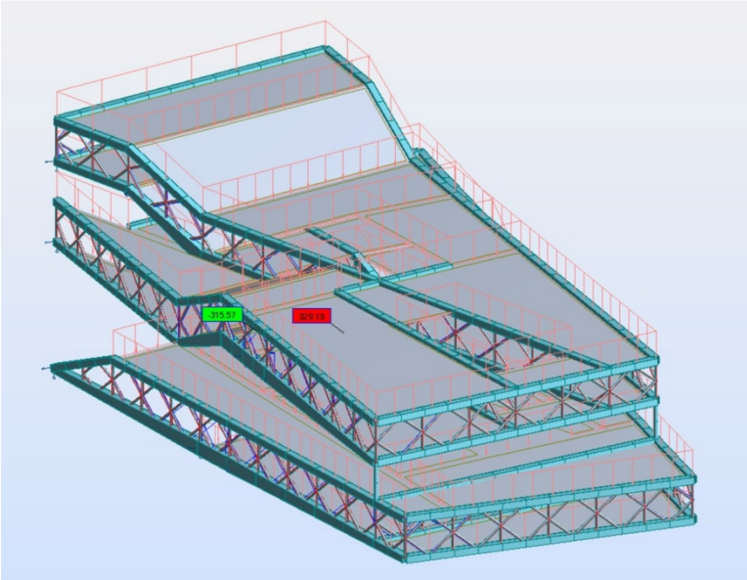

Figure 71 - Maximum stress when live load applied on cantilever and backspan

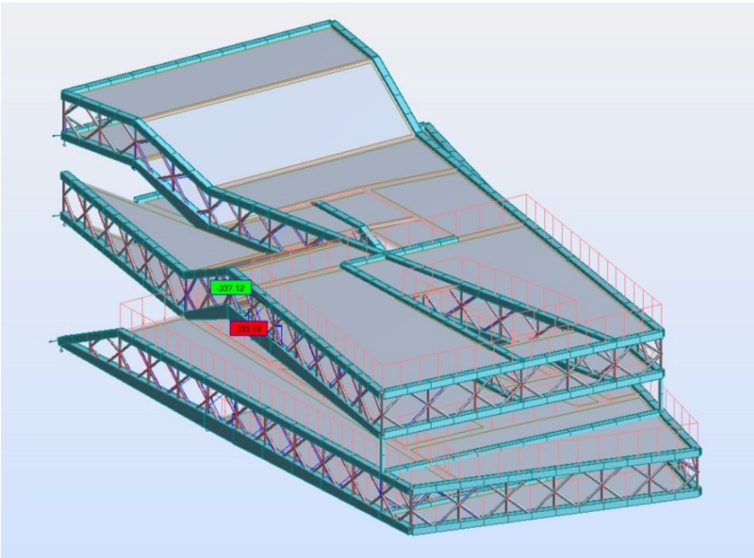

Figure 72 - Maximum stress when live load on cantilever only

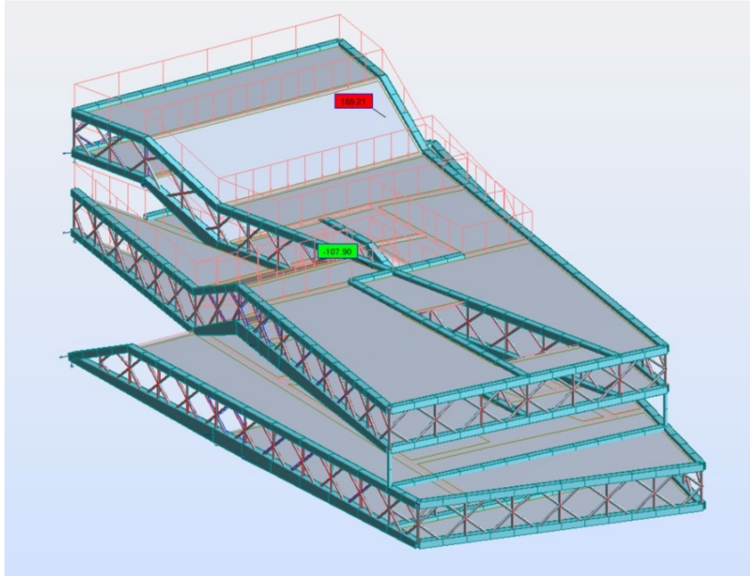

Figure 73 - Maximum stress when live load on backspan only

### *7.4.3.2 Deflection*

Applying the live load only on the cantilevered part of the building results in a deflection of 25,7cm, thus very close to the target value of 25,2cm. Precambering the trusses reduces this deflection to 15,1cm.

Because it is very unlikely that this type of loading will occur during the life of the building and should it happen, would still be adequate at ULS, we consider that a deflection of 25,7cm is acceptable for the purpose of this exercise. Precambering such a structure with 40mm thick plates would be very difficult and costly.

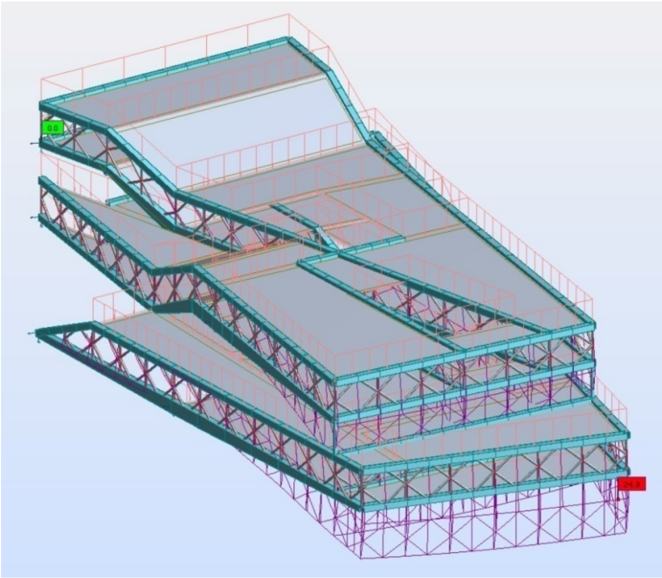

Figure 74 - Total deflection with live load on cantilever and backspan

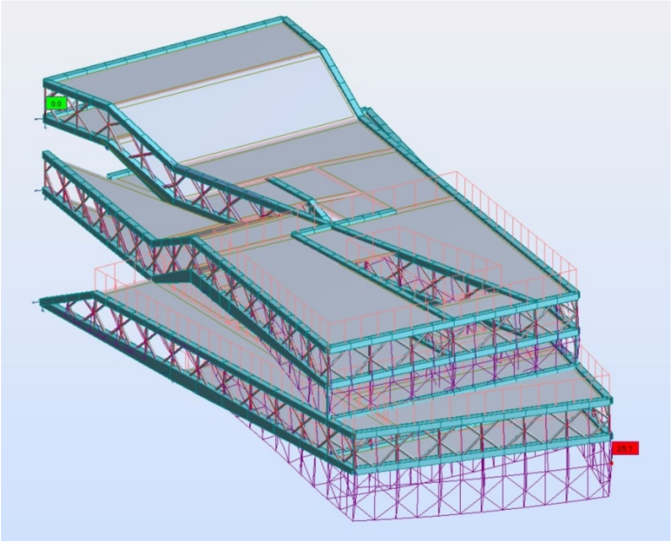

Figure 75 - Total deflection with live load on cantilever only

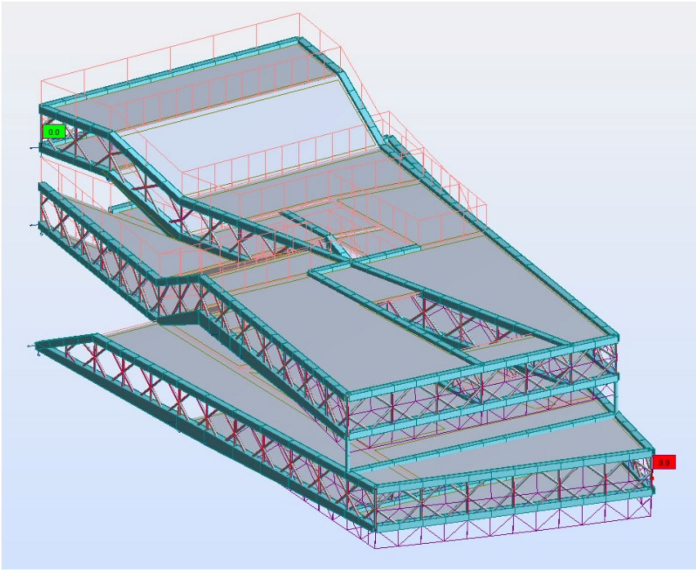

Figure 76 - Total deflection with live load on backspan only

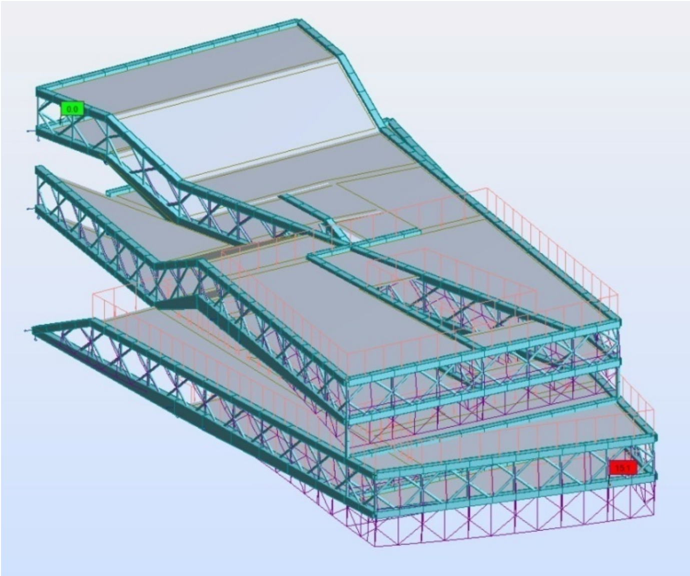

Figure 77 - Maximum deflection with precambering and live load on cantilever only

## 7.5 STABILITY CHECKS

The last step in the design of the building's structure is to check its stability under various load combinations, including the wind loading (transverse). As shown below, the cantilevered structure is supported by vertical steel trusses. The stresses in these should not at any time exceed the steel's yield strength and the deflection at the top be larger than h/500, where h is the height of the vertical truss. Verifications are also made to ensure that the foundations do not have to cater for any uplift force.

### 7.5.1 Loading cases considered

The table below presents all the loading cases which have to be considered based on the Eurocode as well as the safety factors used.

| LOADING CASE | SAFETY FACTOR (ULS) | | |
|---|---|---|---|
| | Dead Load | Live Load | Wind Load |
| Dead + Live | 1.35 | 1.5 | 0 |
| Dead + Wind | 1.35 | 0 | 1.5 |
| Dead + Live + Wind | 1.2 | 1.2 | 1.2 |

Table 15 - Loading cases and factors

These ULS factors are used for the stresses and support reactions checks. For the deflections (SLS), no safety factor is used.

For each load case which includes the wind, two wind directions have been studied: from the side which potentially creates the largest moment in plan and from the back, which leads to the largest reaction reduction at the back supports. The dead + live + wind cases are illustrated below.

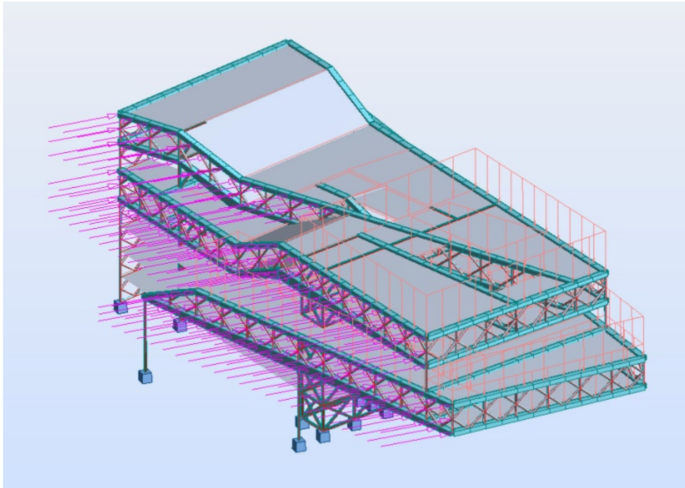

Figure 78 - Dead + live + side wind loading

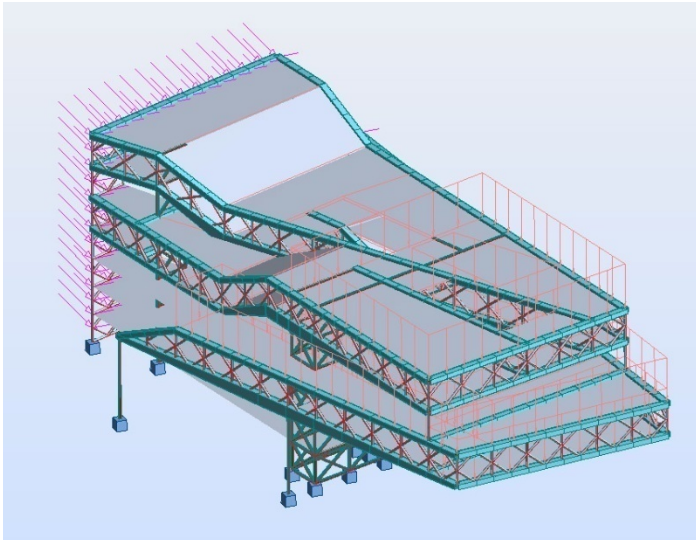

Figure 79 - Dead + live + back wind loading

### 7.5.2 Results

For each of the five loading cases (two wind directions considered), the reactions and deflections have been calculated at various points indicated on the figure below. The vertical trusses connecting these points are highlighted in green.

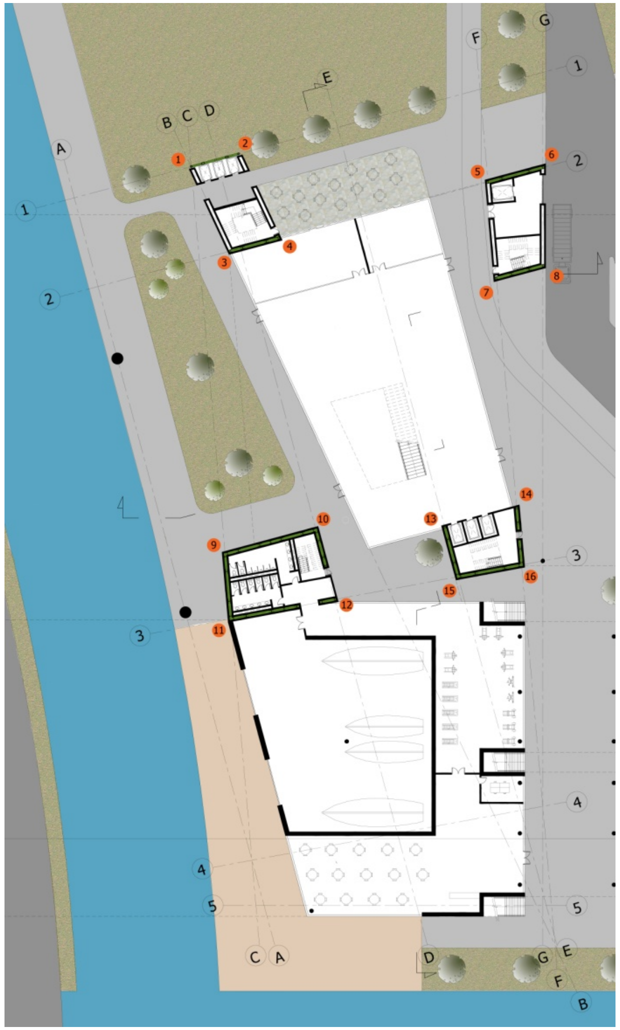

Figure 80 - Support points

As a starting point, HE 300 M steel profiles are used for the vertical supports as for the floor structure.

The results obtained for the reactions and deflections at the top point of the vertical trusses at the locations corresponding to the reactions points are presented in table 16.

It can be observed that for all loading cases, no uplift force is obtained at any support point. Also, the deflections of the top points of the vertical supports are lower than the limit prescribed by the Eurocode.

In figures 81 to 85, the maximum deflection of the tip of the cantilever for all load cases at SLS are shown, this time taking into account the configuration of the vertical supports. The maximum deflection increases from 25,7cm to 27,3cm, 2,1 cm over the limit value of 25,2cm. Further refinements would be required in the thickness of the steel elements of the truss as well as their overall sizes should be made. Also, the member sizes for the vertical trusses would have to be increased. However, as the deflection obtained is still very close to the limit value, the actual configuration is considered at this stage acceptable for the purpose of this exercise.

Finally, as shown in figures 86 to 90, the stresses for all load cases are all below 355 N/mm$^2$, the yield strength of steel.

| SUPPORT POINT | TRUSS HEIGHT (m) | ALLOWED DEFLECTION (mm) | DEAD + LIVE | | DEAD + WIND | | | | DEAD + LIVE + WIND | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Side Wind | | Back Wind | | Side Wind | | Back Wind | |
| | | | Reaction (kN) | Deflection (mm) | Reaction (kN) | Deflection (mm) | Reaction (kN) | Deflection (mm) | Reaction (kN) | Deflection (mm) | Reaction (kN) | Deflection (mm) |
| 1 | 38.5 | 77 | 1164.74 | 8 | 1315.83 | 11 | 1327.02 | 8 | 1060.68 | 9 | 1069.63 | 9 |
| 2 | 38.5 | 77 | 928.02 | 14 | 1080.31 | 15 | 1076.18 | 15 | 823.33 | 14 | 820.02 | 14 |
| 3 | 28.5 | 57 | 1317.39 | 7 | 2259.52 | 8 | 2275.64 | 6 | 1256.08 | 8 | 1268.97 | 7 |
| 4 | 28.5 | 57 | 845.75 | 8 | 1835.85 | 8 | 1794.34 | 7 | 830.08 | 8 | 796.88 | 8 |
| 5 | 23.5 | 47 | 1216.47 | 9 | 1731.73 | 6 | 1727.14 | 7 | 1142.09 | 9 | 1138.42 | 10 |
| 6 | 23.5 | 47 | 1390.92 | 10 | 1904.04 | 6 | 1840.58 | 7 | 1302.48 | 8 | 1251.71 | 10 |
| 7 | 23.5 | 47 | 909.33 | 12 | 2594.80 | 6 | 2658.69 | 11 | 915.75 | 11 | 966.86 | 12 |
| 8 | 23.5 | 47 | 739.40 | 10 | 2773.93 | 6 | 2593.34 | 7 | 917.51 | 8 | 773.03 | 10 |
| 9 | 28.5 | 57 | 10392.19 | 25 | 6145.04 | 12 | 6921.31 | 16 | 8266.56 | 25 | 8887.58 | 25 |
| 10 | 28.5 | 57 | 7563.52 | 24 | 5472.72 | 12 | 5035.55 | 15 | 6727.15 | 25 | 6377.42 | 25 |
| 11 | 28.5 | 57 | 10754.50 | 46 | 6571.82 | 17 | 7218.94 | 29 | 8859.79 | 46 | 9377.49 | 46 |
| 12 | 28.5 | 57 | 12160.65 | 38 | 7992.96 | 19 | 7434.86 | 24 | 10815.90 | 38 | 10369.43 | 38 |
| 13 | 28.5 | 57 | 9528.48 | 17 | 6096.95 | 13 | 6079.74 | 14 | 8084.03 | 22 | 8070.26 | 23 |
| 14 | 28.5 | 57 | 8058.56 | 22 | 5573.29 | 13 | 4949.85 | 14 | 7252.50 | 21 | 6753.74 | 22 |
| 15 | 28.5 | 57 | 8469.67 | 32 | 5375.80 | 20 | 5993.40 | 21 | 6967.38 | 32 | 7461.46 | 32 |
| 16 | 28.5 | 57 | 7589.96 | 34 | 5183.23 | 21 | 5134.32 | 22 | 6675.93 | 34 | 6636.80 | 34 |

**Table 16 - Vertical support reactions and deflections**

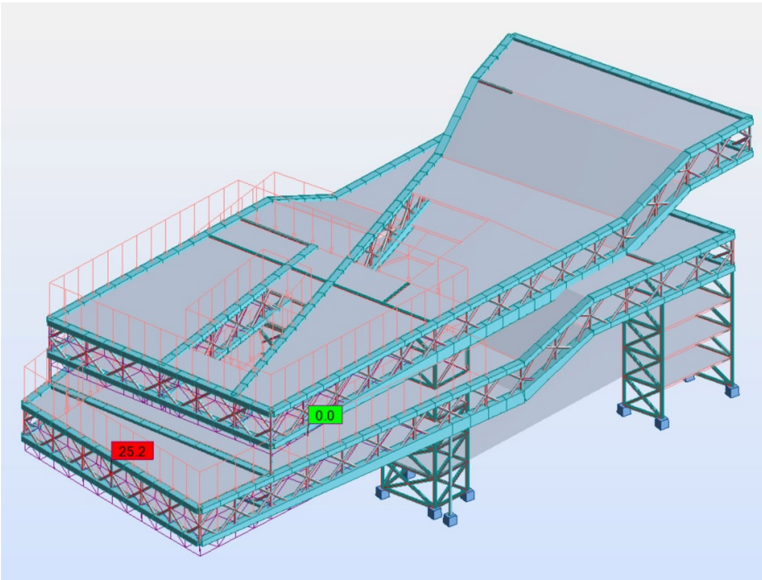


**Figure 81 - Dead + live load deflection**

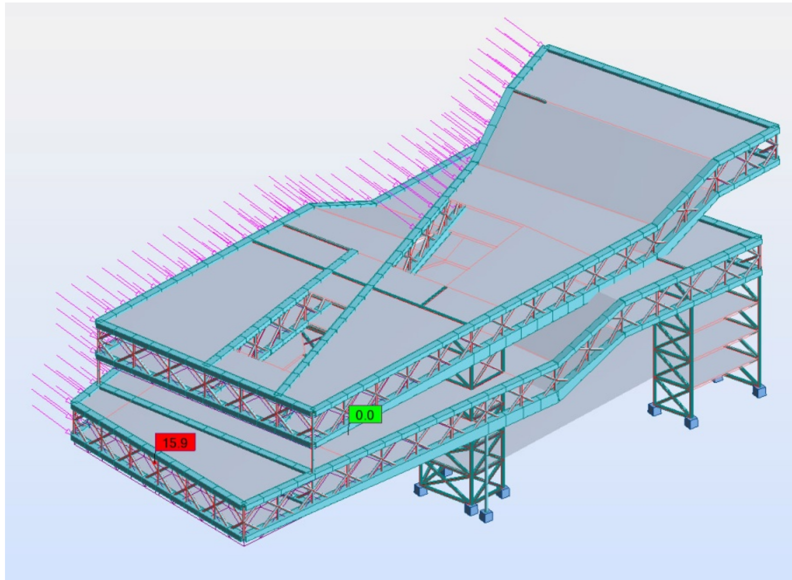


**Figure 82 - Dead + side wind deflection**

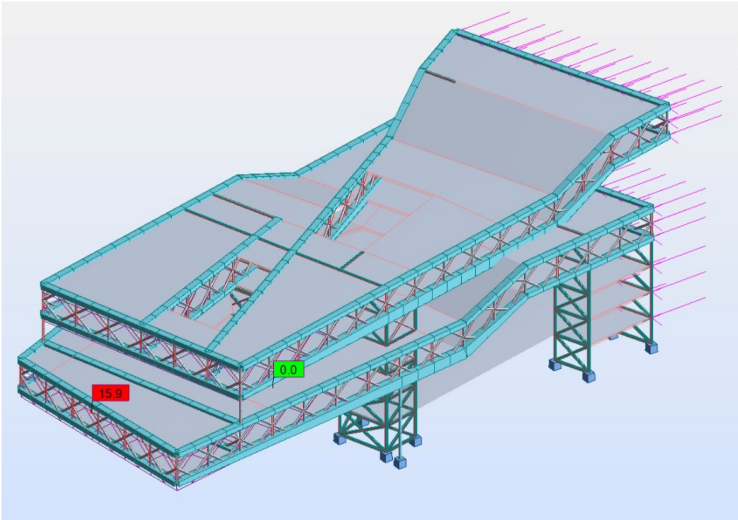


Figure 83 - Dead + back wind deflection

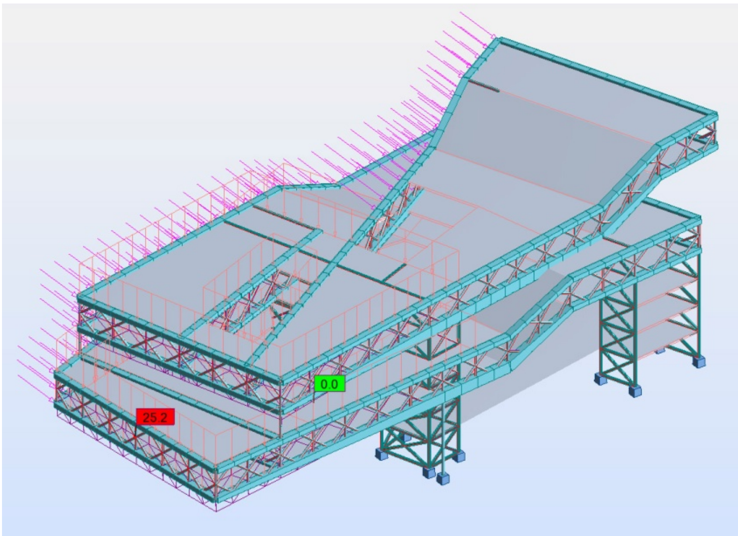


Figure 84 - Dead + side wind + live load deflection

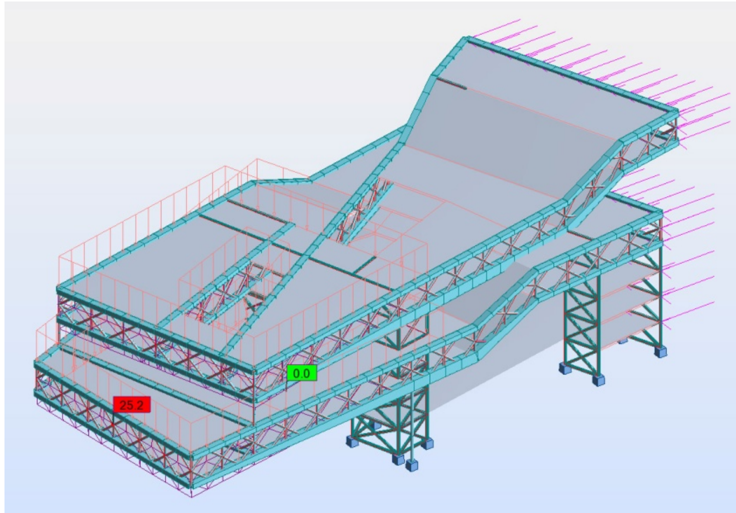


Figure 85 - Dead + back wind + live load deflection

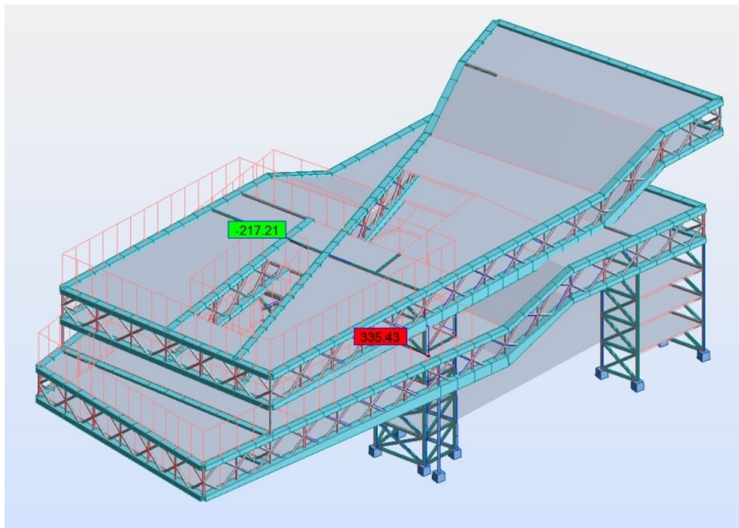


Figure 86 - Dead + live load maximum stresses

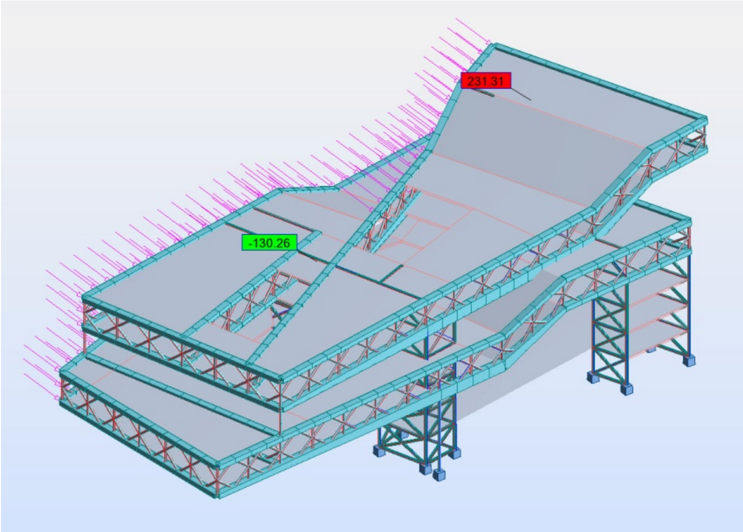


Figure 87 - Dead + side wind maximum stresses

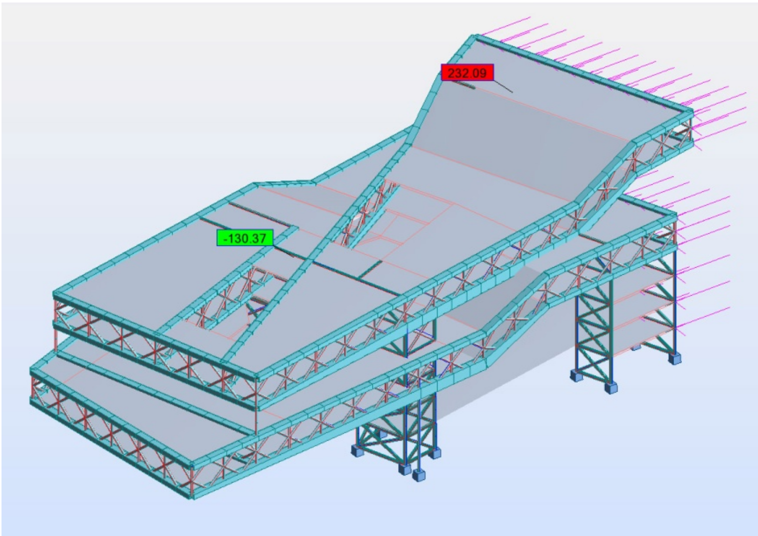


Figure 88 - Dead + back wind maximum stresses

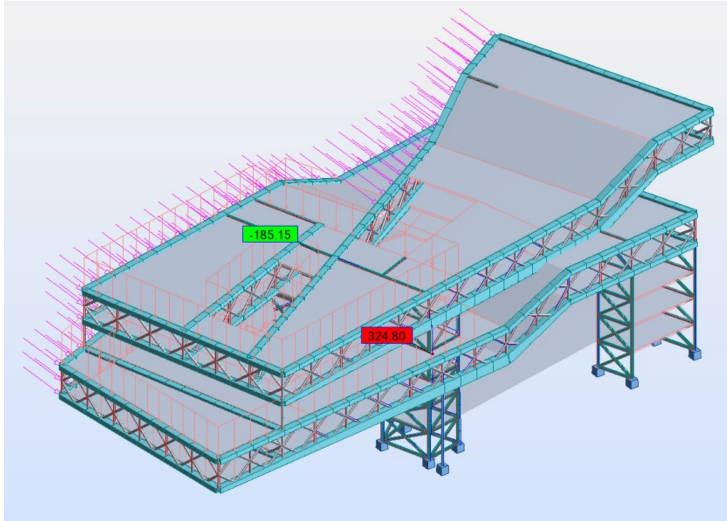


Figure 89 - Dead + side wind + live load maximum stresses

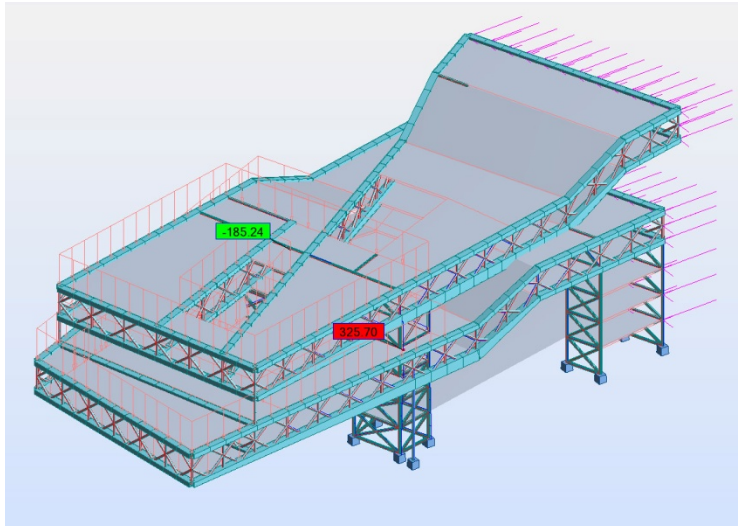


Figure 90 - Dead + back wind + live load maximum stresses

### 7.5.3 Optimisation

The next step, which is outside the scope of this project, would be to optimize the size of all members of the structure. At the moment, all members of the vertical trusses in the cores are HE 300 M, which is certainly unnecessarily large for the cross bracings. The next iteration would be to take the maximum force in one of the braces and determine the section area required to be slightly below the maximum stress permitted of 355 N/mm$^2$ and then run another simulation. We would then check the deflections and stresses and readjust the member sizes based on the new axial forces. We would repeat this process until we are very near the maximum allowed stress and deflection, thus having the smallest members necessary.

Obvisouly, reducing the member sizes in the cores would result in an increase of the maximum deflection at the tip of the cantilever. A study would have to be made to see what would be the cheapest solution: precambering the cantilevered trusses, increasing the member sizes in the upper structure or increasing the member sizes in the core supports.

# 8 CONCLUSION

The structure of a building presenting a 42m-long cantilever with a backspan to cantilever ratio of 1 has been designed. Due to the "spiral" shape which was desired from an architectural point of view, the full height of the building could not be used to span this distance. A unique way of combining 1-storey-high steel trusses has been developed.

The proposed structure has been checked and dimensioned in terms of stresses and deflection as well as for stability based on the Eurocode using both basic hand calculations and finite element analysis with Autodesk Robot Professional.

Various studies and refinements have been made in order to obtain maximum stresses and deflections which are near the limit values allowed by the Eurocode, resulting in a structure which is not overdesigned and doesn't use more material than needed.

Further developments would require optimisation of the member sizes, detailed design of the connections and also perform dynamics checks on the structure.

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# 10 ANNEX A

## 10.1 Calculation of floor beam – composite action

This section presents the calculations made for the floor system chosen, i.e. HE 300 M beams every 3m with a span of 15m. A comflor 80 with 180mm deep concrete slab is used.

As demonstrated below, the loading values vary depending if it is during or after construction. During construction, only the dead load due to the wet concrete needs to be considered, whilst after construction, the live load due to the potential aggregation of users was added to the self-weight of the floor system, increasing the moment capacity required.

### 10.1.1 Loading

**During construction**:

Q = 0 kN/m²
G = 2,74 kN/m²

Ultimate limit state:

$$\begin{aligned} W_{\text{construction ULS}} &= \gamma_g G + \gamma_q Q \\ &= (1{,}35 \text{ x } 2{,}74) + (1{,}5 \text{ x } 0) \\ &= \mathbf{3,70\ kN/m^2} \end{aligned}$$

Serviceability limit state:

$$\begin{aligned} W_{\text{construction SLS}} &= \gamma_g G + \gamma_q Q \\ &= (1{,}0 \text{ x } 2{,}74) + (1{,}0 \text{ x } 0) \\ &= \mathbf{2,74\ kN/m^2} \end{aligned}$$

**After construction**:

Q = 5 kN/m²
G = 2,59 kN/m²

Ultimate limit state:

$$\begin{aligned} W_{\text{service ULS}} &= \gamma_g G + \gamma_q Q \\ &= (1{,}35 \text{ x } 2{,}59) + (1{,}5 \text{ x } 5) \\ &= \mathbf{11,0\ kN/m^2} \end{aligned}$$

Serviceability limit state:

$$\begin{aligned} W_{\text{service SLS}} &= \gamma_g G + \gamma_q Q \\ &= (1{,}0 \text{ x } 2{,}59) + (1{,}0 \text{ x } 5) \\ &= \mathbf{7,59\ kN/m^2} \end{aligned}$$

### 10.1.2 Design Moments

The design (applied) moments will only be used for the ultimate limit state calculations.

The span is:

L = 15m

**<u>During construction</u>**:

$$M_{construction} = \frac{W_{construction\ ULS}\, L^2}{8}$$
$$= \frac{(3{,}70 \text{ x } 3\text{m}) \text{ x } 15^2}{8}$$

= **312 kN.m**

**<u>After construction</u>**:

$$M_{service} = \frac{W_{service\ ULS}\, L^2}{8}$$
$$= \frac{(11{,}0 \text{ x } 3) \text{ x } 15^2}{8}$$

= **928,14 kN.m**

### 10.1.3 Moment Capacities

**<u>During construction:</u>**

Because the steel deck is fixed to the top flange of the beam with shear studs, no lateral torsional buckling is possible during construction. The moment capacity is therefore calculated as follows:

$$M_{b,Rd} = \chi_{LT}\, W_y\, f_y$$

$\chi_{LT}$ : lateral torsional buckling factor for rolled sections = 1
$W_y$ : plastic moment of section about the strong axis = 4078000 $mm^3$
$f_y$ : yield strength of steel = 355 $N/mm^2$

Hence:

$$M_{b,Rd} = 1 \text{ x } 4078000 \text{ x } 355$$

= **1447,69 kN.m > $\mathbf{M_{construction}}$**

**<u>After construction:</u>**

After construction, the concrete of the slab will have set and will contribute to the moment capacity of the floor system. There is therefore a composite action between the beam and the concrete slab, increasing the moment capacity of the floor.

In order to find where the neutral axis of the composite section lies, the strength of the beam and the concrete slab are calculated.

Then the strength of the steel beam is given by:

$$R_s = 0{,}95 \cdot A_s \cdot f_y$$

where:

$A_s$: section area of the beam = 30310 $mm^2$
$f_y$: steel strength (stress) = 355 $N/mm^2$

Therefore:

$$R_s = 0{,}95 \cdot 30310 \cdot 355$$
= **10222,05 kN**

Then the strength of the web steel beam is given by:

$$R_w = 0{,}95 \cdot A_w \cdot f_y$$

where:

$A_w$: section area of the web = 7140 $mm^2$
$f_y$: steel strength (stress) = 355 $N/mm^2$

Therefore:

$$R_w = 0{,}95 \cdot 7140 \cdot 355$$
= **2407,97 kN**

The strength of the concrete slab is:

$$R_c = 0{,}45 \cdot A_c \cdot f_{cu}$$

where:

$A_c$: area of the concrete slab = 540000 $mm^2$
$f_{cu}$: concrete strength (stress) = 40 $N/mm^2$

Hence:

$$R_c = 0{,}45 \cdot 540000 \cdot 40$$
**= 9720 kN**

**In this case, *$R_c$ < $R_s$ & $R_c$ > $R_w$, thus the neutral axis is in the top flange***

$$M_{pl,Rd} = R_s \frac{h}{2} + R_c \left(\frac{h_f}{2} + \frac{t_f}{2}\right)$$

Where:

$t_f$ : top flange thickness = 39 mm
h: beam depth = 340 mm
$h_f$: slab depth = 180 mm

Therefore:

$$M_{pl,Rd} = \left(10222{,}05 \text{ x } \frac{0{,}34}{2}\right) + \left(9720\left(\frac{0{,}18}{2} + \frac{0{,}039}{2}\right)\right)$$
= **2802,09 kN.m > $M_{service}$**

The moment capacities during service and construction are well above the minimum values required. However, as will be demonstrated below, the deflections are leading the design, requiring deeper floor beams and increasing the moment capacities.

### 10.1.4 Deflections

**During construction:**

During construction, only the beam will support the weight of wet concrete. The deflection is calculated using the formula for a simply supported beam under UDL:

$$\delta_{midspan} = \frac{5W_{constructionSLS}L^4}{384E_{steel}I}$$

Where:

L: beam span = 15 m
$E_{steel}$: steel's Young's modulus = 210000 N/mm$^2$
I: second moment of area of beam = 59200 x $10^4$ mm$^4$

Therefore:

$$\delta_{\text{midspan}} = \frac{5 \text{ x } (2740 \text{ x } 3) \text{ x } 15^4}{384 \text{ x } 210 \text{ x } 10^9 \text{ x } 59200 \text{ x } 10^{-8}}$$

**= 43,6 mm < 60mm**

The maximum deflection allowed in beam span/250 = 60mm

**After construction:**

From the calculations for the moment capacity of the composite section, we know that the neutral axis lies in the steel beam. For this case, we use the following formulas:

Effective width:

$b_{eff}$ = 2 x L/8
= 2 x 15000/8
= 3750mm

Because this value is greater than the floor beams' spacing of 3m, we have to take:

$b_{eff}$ = 3000 mm

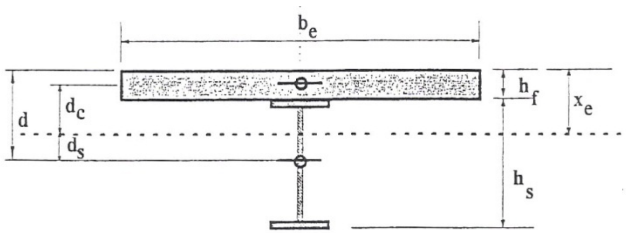

Figure 91 - Definitions

With:

$$K = 2\,\alpha_e\ A_s/b_{eff}$$

where:

$A_s$: section area of steel beam = 30310 $mm^2$
$\alpha_e = E_{steel}/E_{concrete}$ : modular ratio = 210000/10000 = 21

K = 2 x 21 x 30310/3000
= **424,34**

Then,

$$x_e = \frac{Kd + h_f^2}{K + 2h_f}$$

Where:

$h_f$ : slab depth = 180 mm
$d = (h_s/2) + h_f = (340/2) + 180 = 350$ mm

Hence:

$$x_e = \frac{(424{,}34 \text{ x } 350) + 180^2}{424{,}34 + (2 \text{ x } 180)}$$
= 230,66mm

We can then calculate the second moment of area of the composite section:

$$I = I_s + A_s d_s^2 + \frac{I_c}{\alpha_e} + A_c \frac{d_c^2}{\alpha_e}$$

$I_s$: second moment of area of the beam = 59200 x $10^4$ $mm^4$
$I_c$: second moment of area of the slab = 1219449600 $mm^4$
$A_s$: section area of the beam = 30310 $mm^2$
$A_c$: section area of the slab = 540000 $mm^2$
$d_s = d - x_e = 350 - 230{,}66$ = 119,34 mm
$d_c = x_e - (h_f/2) = 230{,}66 - (180/2)$ = 140,66 mm

By replacing these values into the equation, we get:

I = **159050,83 x $10^4$ $mm^4$**

Finally, once again, the deflection is calculated with the formula for a simply-supported beam under a uniformly-distributed load:

$$\delta_{midspan} = \frac{5(W_{serviceSLS} \text{ x } 3)L^4}{384 E_{steel} I}$$

$$= \frac{5 \text{ x } (7590 \text{ x } 3) \text{x } 15^4}{384 \text{ x } 210 \text{ x } 10^9 \text{ x } 159050{,}83 \text{ x } 10^{-8}}$$

**= 44,9 mm < 60 mm**

## 10.2 Calculation of floor beam – No shear connection

This section presents the calculations made for a floor system without shear connection between the steel deck and the beams. There is therefore possibility of lateral torsional buckling of the beam during construction but this phenomenon is eliminated during service when the concrete slab restrains the compression (top) flange of the beams.

For this example, HE 450 M beams every 3m with a span of 15m are used. A comflor 80 with 180mm deep concrete slab is used.

As for the previous floor type, various loading intensities are used during construction and service. The addition of live loading during service will nearly triple the moment capacity required.

### 10.2.1 Loading

**During construction**:

Q = 0 kN/m²
G = 2,74 kN/m²

Ultimate limit state:

$$
\begin{aligned}
W_{\text{construction ULS}} &= \gamma_g G + \gamma_q Q \\
&= (1{,}35 \text{ x } 2{,}74) + (1{,}5 \text{ x } 0) \\
&= \mathbf{3,70\ kN/m^2}
\end{aligned}
$$

Serviceability limit state:

$$
\begin{aligned}
W_{\text{construction SLS}} &= \gamma_g G + \gamma_q Q \\
&= (1{,}0 \text{ x } 2{,}74) + (1{,}0 \text{ x } 0) \\
&= \mathbf{2,74\ kN/m^2}
\end{aligned}
$$

**After construction**:

Q = 5 kN/m²
G = 2,59 kN/m²

Ultimate limit state:

$$
\begin{aligned}
W_{\text{service ULS}} &= \gamma_g G + \gamma_q Q \\
&= (1{,}35 \text{ x } 2{,}59) + (1{,}5 \text{ x } 5) \\
&= \mathbf{11,0\ kN/m^2}
\end{aligned}
$$

Serviceability limit state:

$$
\begin{aligned}
W_{\text{service SLS}} &= \gamma_g G + \gamma_q Q \\
&= (1{,}0 \text{ x } 2{,}59) + (1{,}0 \text{ x } 5) \\
&= \mathbf{7,59\ kN/m^2}
\end{aligned}
$$

### 10.2.2 Design Moments

The design (applied) moments will only be used in the ultimate limit states.

The span is:

L = 15m

**During construction**:

$$
\begin{aligned}
M_{\text{construction}} &= \frac{W_{\text{construction ULS}}\, L^2}{8} \\
&= \frac{(3{,}70 \text{ x } 3\text{m}) \text{ x } 15^2}{8}
\end{aligned}
$$

**= 312 kN.m**

**After construction**:

$$
\begin{aligned}
M_{\text{service}} &= \frac{W_{\text{service ULS}}\, L^2}{8} \\
&= \frac{(11{,}0 \text{ x } 3) \text{ x } 15^2}{8}
\end{aligned}
$$

**= 928,14 kN.m**

### 10.2.3 Moment Capacities

**During construction:**

Because the steel deck is placed between the beams, there is a possibility of lateral torsional buckling of the beams, reducing their moment capacity.

Calculate the elastic critical moment $M_{cr}$:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \left\{ \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z} + \left(C_2 z_g\right)^2} - C_2 z_g \right\}$$

Where:

$I_z$ = 19340 x $10^4$ mm$^4$
$I_t$ = 1529 x $10^4$ mm$^4$
$I_w$ = 9251 x $10^9$ mm$^6$
L = 15000 mm
$Z_g$ = (height of beam/2) = (478/2) = 239 mm
$C_1$ = 1,127
$C_2$ = 0,454
E = 210000 N/mm$^2$
G = 80770 N/mm$^2$

Replacing these values into the formula, we obtain:

$M_{cr}$ = **1524,20 kN.m**

Calculate the dimensionless slenderness $\lambda_{LT}$:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

Where:
$W_y$ = 6331 x $10^3$ mm$^3$
$f_y$ = 355 N/mm$^2$

$$\bar{\lambda}_{LT} = \sqrt{\frac{6331 \text{ x } 10^3 \text{ x } 355}{1524197063}}$$

= **1,214**

Calculate the lateral torsional buckling factor:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{\,2} - \beta\bar{\lambda}_{LT}^{\,2}}} \text{ but } \begin{cases} \chi_{LT} \leq 1 \\ \chi_{LT} \leq \dfrac{1}{\bar{\lambda}_{LT}^{\,2}} \end{cases}$$

$$\Phi_{LT} = 0{,}5 \left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}\right) + \beta\bar{\lambda}_{LT}^{\,2}\right]$$

Where:

$\lambda_{LT0}$ : 0,4
$\beta$ : 0,75
$\alpha_{LT}$ : imperfection factor = 0,21

$$\Phi_{LT} = 0{,}5\,[1 + 0{,}21(1{,}214 - 0{,}4) + (0{,}75 \text{ x } 1{,}214^2)]$$
$$= \mathbf{1{,}138}$$

$$\chi_{LT} = \frac{1}{1{,}138 + \sqrt{1{,}138^2 - (0{,}75 \text{ x } 1{,}214^2)}}$$
$$= \mathbf{0{,}635}$$

The moment capacity is then calculated with

$$M_{b,Rd} = \chi_{LT}\, W_y\, f_y$$
$$= 0{,}635 \text{ x } 6331 \text{ x } 10^3 \text{ x } 355$$
$$= \mathbf{1427{,}17\ kN.m > M_{construction}}$$

**After construction:**

After construction, the concrete slab has set and eliminates any possibility of lateral torsional buckling. The factor $\chi_{LT} = 1$ and the moment capacity during service is simply calculated:

$$M_{b,Rd} = \chi_{LT}\, W_y\, f_y$$
$$= 1 \text{ x } 6331 \text{ x } 10^3 \text{ x } 355$$
$$= \mathbf{2247{,}5\ kN.m > M_{service}}$$

Once again, the floor design is driven by the deflection limit, hence the unnecessarily high moment capacities provided.

### 10.2.4 Deflections

**During construction:**

During construction, only the beam will support the weight of wet concrete. The deflection is calculated using the formula for a simply supported beam under UDL:

$$\delta_{midspan} = \frac{5W_{constructionSLS}L^4}{384E_{steel}I}$$

Where:

L: beam span = 15 m
$E_{steel}$: steel's Young's modulus = 210000 N/mm$^2$
I: second moment of area of beam = 131500 x $10^4$ mm$^4$

Therefore:

$$\delta_{midspan} = \frac{5 \text{ x } (2740 \text{ x } 3) \text{ x } 15^4}{384 \text{ x } 210 \text{ x } 10^9 \text{ x } 131500 \text{ x } 10^{-8}}$$

$$= \mathbf{19{,}6\ mm < 60\ mm}$$

The maximum deflection allowed in beam span/250 = 60mm

**After construction:**

Because there is no shear connection between the slab and the beam, only the beam will have to support the dead and live loads and the slab does not contribute in reducing the deflections.

The method used for the deflection calculation during service is therefore the same as during construction, but with the addition of live loads.

$$\delta_{midspan} = \frac{5W_{serviceSLS}L^4}{384E_{steel}I}$$

L: beam span = 15 m
$E_{steel}$: steel's Young's modulus = 210000 N/mm$^2$
I: second moment of area of beam = 131500 x $10^4$ mm$^4$

Therefore:

$$\delta_{\mathrm{midspan}} = \frac{5 \text{ x } (7590 \text{ x } 3) \text{ x } 15^4}{384 \text{ x } 210 \text{ x } 10^9 \text{ x } 131500 \text{ x } 10^{-8}}$$

= **54,3mm < 60 mm**