MEGA
Structural
Design

The Pine
LOI 130, Brussels, Belgium

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Group 4

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I. Preface

The report that lies in front of you is the result of the multidisciplinary project of MEGA. The course MEGA is a ten week long project concerned with the design, computation, engineering, and construction management of a high-rise building. The design project in 2019 deals with a city block in Brussels. This block is based on the current international restricted interdisciplinary architectural competition in two phases for the construction of a “Real estate complex for the European Commission in Brussels -Project Loi 130”. The competition is organized by the European Commission. The last ten weeks our multidisciplinary team aimed to create an integral design respecting all the requirements as given in the brochure. We would like to thank Steven van Eck for the insights and help he provided during the consults.

Enjoy reading this report!

Brouwer, Joost
Gaaij de, Joris

Delft, July 4th 2019

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III. Introduction

Location

The LOI130 project is situated in Brussels. The aim of the project is the construction of a real estate complex for the European Commission in Brussels. Although Europe does not want to declare an official political capital, Brussels can be seen as this capital of Europe. Because, it hosts the official seats of the European Commission, Council of the European Union, European Council, and a seat of the European Parliament. (Maufort 2016)

The site located in the "European Quarter" has an approximate size of 24.000 m2. The goal of the project competition is to redevelop the site into an efficient building complex for the European Commission with an approximate gross floor area above ground between 175.000 –190.000 m2. The mixed-use complex should provide office space for at least 5.000 employees, conference facilities, two childcare centres, the Commission´s visitor centre, retail facilities, a car park and a new entrance to the Maelbeek subway station (Dobbelsteen, 2019)

The idea

Our team specified two main aims. The first is to create a collaborative design by working together with the different disciplines and designing in a realistic environment. The second aim is to create a sustainable design. In this report the structural part of the design is explained, and this design is based on three main concepts.

These three main concepts are: timelessness, flexibility and climate responsive. The sizes of commissions in the European commission are subject to change over time. By creating a flexible building, it can be used for a longer time. By focussing on timelessness, we will create a more sustainable building because the materials can be used for a longer time. By being climate responsive we are already taking into account the possibility of extreme weather conditions caused by global warming.

To fulfil all the requirements and to realise two outstanding towers, the Pine is presented in this report. The Pine will give Brussels, and thus Belgium, a boost of confidence. With the use of the two towers and the redesigned plinth it will give a huge possibility for companies and the inhabitants of Brussels.

With the flexibility of the change of functions of the two towers, the focus on timelessness and being climate responsive, the Pine will be useful for the upcoming 100 years and aims to become a national monument.

The set-up of the report

In this report the first chapter will be about the structural systems, which will be chosen and calculated. The next chapter will be about the realisation of the Plinth. Furthermore, the dimensions of the floors, beams and columns are calculated. When the plinth and towers are dimensioned, the bridge will be discussed. As final chapters the soil, foundation piles and building pit are examined. The report will finish with a conclusion and discussion.

IV. Structural systems

In this paragraph the structural systems of both towers will be explained and elaborated. At first the structural possibilities will be shown with their own (dis)advantages. The structural system will be chosen to fit best with all the disciplines. Due to the many discussions and constant optimising for each discipline the design has changed multiple times. In this chapter all the calculations are based on the higher tower, due to the fact that bigger forces act on the bigger tower. The same structure is then also applied for the lower tower. This can be adjusted in a later stage of design.

Design phase

The project consists of two towers connected with a sky bridge. The two towers both have different heights. The higher tower will be 150 meters high and the lower tower is 122 meters high. For high rise towers, multiple structures are possible. In Figure 1 the different possibilities are given. The towers will be respectively 42 and 34 floors.

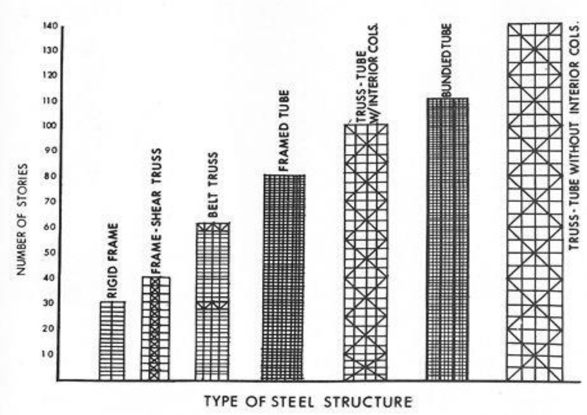


Figure 1 Structural systems (Researchgate 2018)

Three systems will be elaborated due to the fact that they meet the requirements for the high-rise structure that is demanded on the project. These systems are a *Core structure*, an *Outrigger structure* and a *Tube structure*. The *Frame Structure* is not possible due the height limit. The *Mega structure* and the *Braced truss* didn't fulfil the requirements and demands of the architect. The structure will be made of steel and concrete. The main advantage of using a steel structure and concrete core is the rapid erection process. The concrete core can be built first since it stabilizes itself. After the concrete cores reaches a certain height, the erection of the steel structure can be started. For stability reasons, horizontal temporary bracings are often necessary (Abspoel, 2013). In Table 1 the general (dis)advantages of the different structure types are shown.

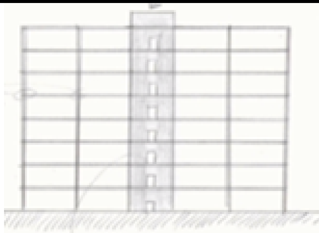
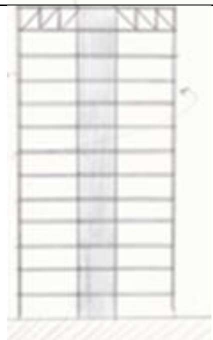
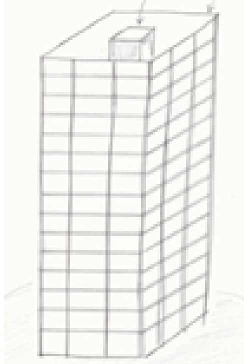
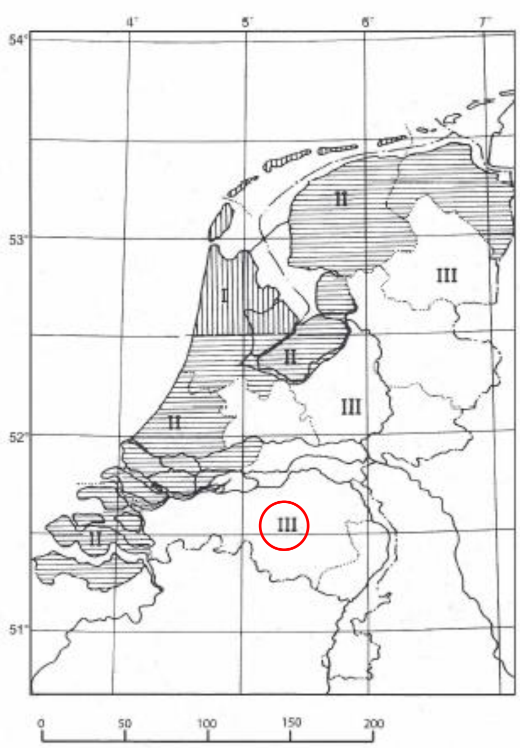
Structure	Up to … layers (height)	Characteristics	Sketch	Pros/Cons
Core	40 layers ± 150m height	-Strong concrete core -'Weak' connections possible in the outer façade.		- Only possible for small tower + According to architectural design with a flexible façade
Outrigger	50 layers ± 200m height	-Strong core -Outer columns will have to withstand strain -Stiff rigger compromises a horizontal truss -The riggers can be applied once, or on multiple levels of the building		-Sacrificing one or multiple floors +Possible for both towers
Tube	Up to 100 layers ± 400m height	-Façade is also used as a core -The inner core will be 'weak' and the outer core will be able to withstand all the forces -Rigid connections, thus façade will act as a sway frame.		+Possible for both towers + Multiple (but leight) steel members possible

Table 1 Structure types (Nijsse, 2019)

Loads

Wind load

When designing a building, not only vertical loads have to be considered. Especially with high rise buildings, horizontal loads must be considered. Since earthquakes are not very likely to occur in the area, this can be neglected. However, wind load will be an important aspect. With the use of to the Quick reference 2014, the wind load can be determined in the Netherlands. With the assumption that Brussels has sort of the same climate as the province 'North Brabant' in the Netherlands, the wind load in Brussels can be determined. As seen on Figure 2 Brussels is determined as Area III. With an urban climate, the wind loads can be determined by means of tower height. With the heights of 150m and 122m, the wind load will be respectively 1.35 kN/m^2 and 1.28 kN/m^2. These are the base values. For the other buildings of the Plinth, a value of 0.8 kN/m^2 is found at a height of 24m.



z_e [m]	$q_p(z_e)$ [kN/m²] Area I Coastal	Rural	Urban	Area II Coastal	Rural	Urban	Area III Rural	Urban
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	[illegible]
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	[illegible]
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	[illegible]
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	[illegible]
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	[illegible]
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 2 Wind loads (Pasterkamp, 2014)

However, due to the fact that the wind load is needed for a high rise building, some extra factors are needed. A factor of 1.2 is added for second order and dynamic factors. On top of this, a factor of 1.5 is added to take into account the pressure and suction on both sides of the building. With these known factors, the wind loads will be 2,43 kN/m² for the bigger tower, 2,304 kN/m² for the lower tower and 1,44kN/m² for the buildings in the Plinth. For the use of the calculations and the fact that the bridge connects both towers, the governing wind load is 2.43 kN/m² . However, for simplitcity reasons a wind load of 2.5 kN/m² will be used for the calculations in the preliminary design stage.

Floor loads

Both towers offer room for multiple functions throughout the whole building. Every function has its own specifications and thus differs from the others. With the different floor loads per function, the governing load can be determined. In Table 2 you can immediately see that the snow load is negligible.

Function	Load [kN/m²]	Type of load
Residential	1.75	Variable
Office	2.75	Variable
Restaurant	5	Variable
Wind	2.5	Variable
Parking	2	Variable
Roof (depends on angle)	maximum 1	Variable
Snow	0.7	Variable
Own load floor	2.45	Permanent
Own load beams	0.40	Permanent
Own load other structural members	1	Permanent
Services	0.5	Permanent

Table 2 Loads per function (Pasterkamp, 2014)

Load combinations

Since all the load factors of the different functions are known, the governing load combinations have to be determined. Before the load combinations can be composed, some extra load factors have to be determined.

Due to the fact of the high consequence for loss of human life or economic, social or environmental great loss, the consequence class of the towers reaches CC3. This will give a value of K_{FL}=1.1.

Besides there are some load categories with corresponding ψ-factors. This is a factor for a combination value of a variable action. So called coincidence factors.

Load	ψ
Wind	0
Residential	0.4
Office	0.5

Table 3 Coincidence factors (Pasterkamp, 2014)

Design situation	Permanent loads unfavourable	Favourable	Variable loss leading	Other
1	1,35	0,9	-	1.5
2	1.2	0.9	1.5	1.5

Table 4 Load factors tower (Pasterkamp 2014)

The load combinations can now be determined. The combinations are separated in two main parts, permanent and variable loads governing. Each situation has its favourable and unfavourable loads.

<u>Permanent loads governing</u>

Load combination 1: $K_{FL} * 1{,}35G + \sum K_{FL} * 1.5Q_i * \psi_i$

Load combination 2: $K_{FL} * 0.9G + \sum K_{FL} * 1.5Q_i * \psi_i$

<u>Variable loads governing:</u>

Load combination 3: $K_{FL} * 1{,}2G + K_{FL} * 1.5Q_{office} + K_{FL} * 1.5Q_{wind} * \psi_{wind}$

Load combination 4: $K_{FL} * 1{,}2G + K_{FL} * 1.5Q_{wind} + K_{FL} * 1.5Q_{office} * \psi_{office}$

Load combination 5: $K_{FL} * 0.9G + K_{FL} * 1.5Q_{office} + K_{FL} * 1.5Q_{wind} * \psi_{wind}$

Load combination 6: $K_{FL} * 0.9G + K_{FL} * 1.5Q_{wind} + K_{FL} * 1.5Q_{office} * \psi_{office}$

	Loads [kN/m2]	psi factors [-]
Wind	2.5	0
Residential	1.75	0.4
Office	2.5	0.5
Own load floor	2.45	
Beams	0.4	
Structural members	1	
Services	0.5	
Dead weight total	**4.35**	
Only Q, thus G =	0	
LC1	8.52225	kN/m2
LC2	6.369	kN/m2
LC3	9.867	kN/m2
LC4	11.814	kN/m2
LC5	8.4315	kN/m2
LC6	10.3785	kN/m2
LC4_Dead weight=0	6.072	kN/m2

Table 5 Load combinations (Designed by the authors)

As seen in Table 5, LC4 is governing with a value of 11.814 kN/m^2. When only the variable load is taken into account (dead load = 0), LC4 is still governing with a value of 6.072 kN/m^2. These floor loads as used can be found in Appendix 1.

Tower Geometry

The big and the small tower both consist of a series of different 14 meter high 'blocks' resting on a 24 meter high plinth. Both towers will not be structurally connected to the plinth. So, the block above the plinth will be extended through the plinth. All these blocks have different floor plans. The big tower has 9 blocks, while the small tower consists of 7 blocks. The different widths of the North and the East elevation are given in Figure 3.Since the bottom two 12 meter high floors are surrounded by the plinth, there will be no wind force on the tower.

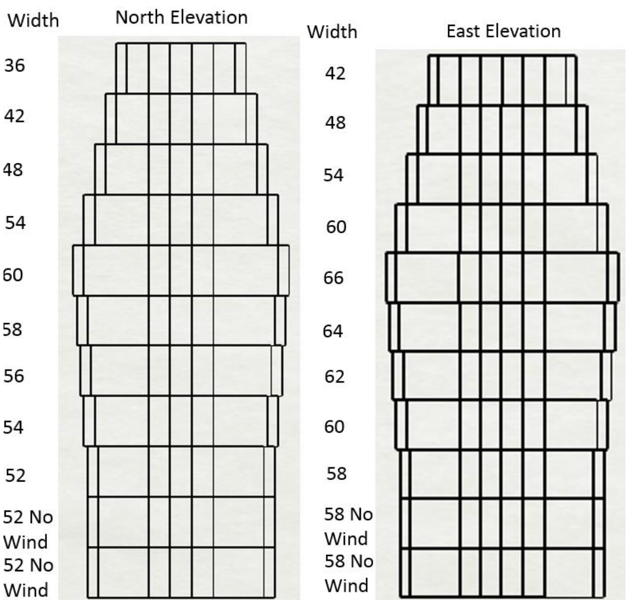


Figure 3 Different elevation widths (Designed by the authors)

Final system

For the towers, two structural stability systems are possible to use. These systems are the outrigger and the tube system. A basic calculation is done for both systems. After this basic information was found, the two possibilities were discussed with all disciplines to find the most favourable choice. At this point, the tower was 165 meter high, so the maximum allowable horizontal deflection is $w_{max} = \frac{total\ height\ of\ the\ tower}{750} = \frac{165}{750} = 0{,}22\ m$

Outrigger system

At first a basic calculation of the outrigger system is done. In this calculation the most favourable position of the outrigger and the sizes of the tension and compression columns were determined. The first calculation shows that it is necessary to use two outriggers to stabilize the building. The optimal position of these outriggers was found by making a simplified model in Matrixframe. The results of this optimisation are shown Table 6. Figure 4 shows the north elevation of the building. The vertical blue area visualizes the size of the core. The horizontal blue area visualizes the position of the outriggers. Each outrigger was approximated to be two floors high to be able to have sufficient stiffness. The red lines show the tension and compression elements in the façade. These are HL 1000x591 profiles. Figure 5 shows a floor plan of the biggest floor. The red dots show these HL 1000x591 profiles. The size of the green columns depends on the weight of the floors, the floor loads and the number of floors on top of the column. The size of the yellow columns depends on the weight of the façade and the floor loads.

horizontal displacement w[m] at top		Height outrigger 2 [m]			
		51	63	75	87
Height outrigger 1 [m]	111	0,2223	0,22	0,2235	0,2322
	123	0,2234	0,2193	0,2215	0,2293
	135	0,2291	0,223	0,2236	0,2302

Table 6 Optimisation outrigger (Designed by the authors)

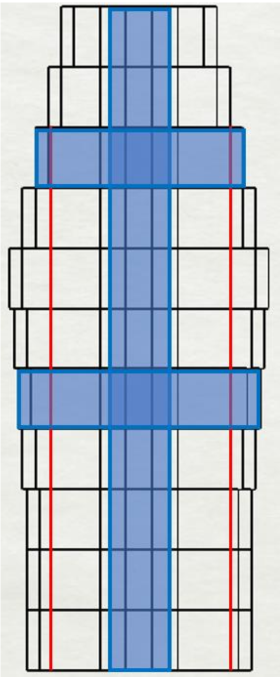


Figure 4 Elevation of the tower (Designed by the authors)

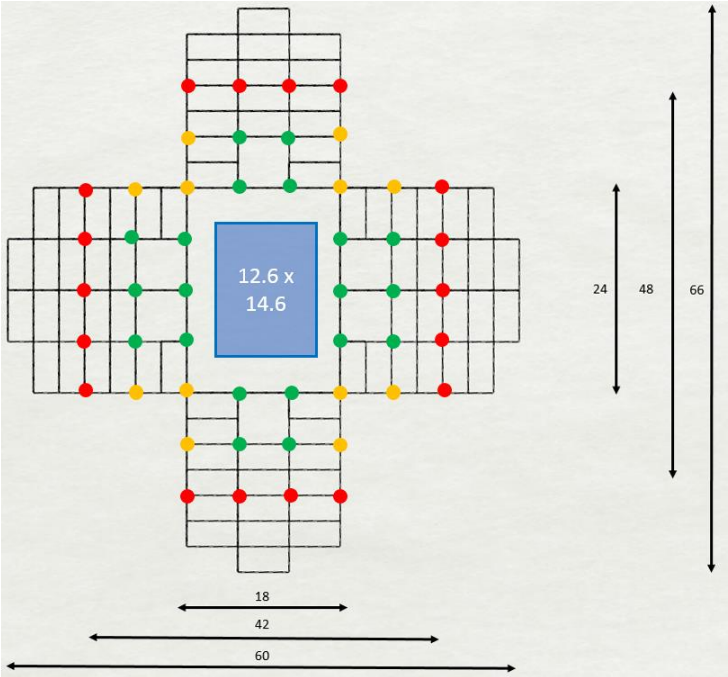


Figure 5 Floor plan of biggest floor (Designed by the authors)

Tube system

The shape of the façade makes it impossible to use a rectangular tube system. It is most optimal to use four tubes around the core. For this idea, the “Willis Tower” was used as a reference project, see Figure 6.This tower consists of nine main tubes. From the 84th floor on, five of the nine tubes continue. This results in a floor plan similar to the one in the Pine, also shown in Figure 6.

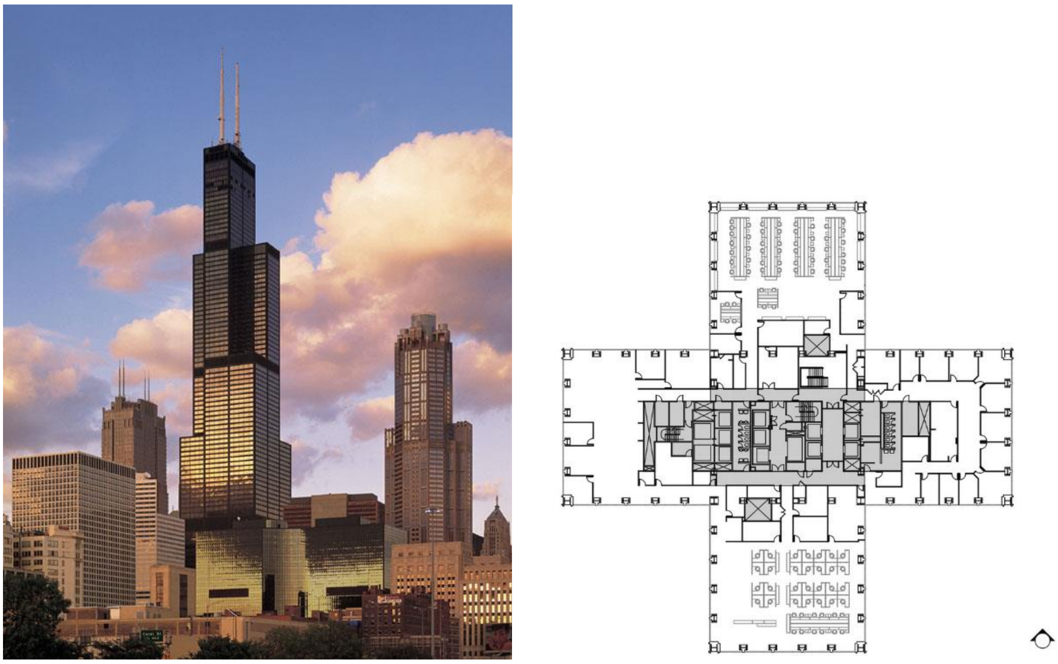
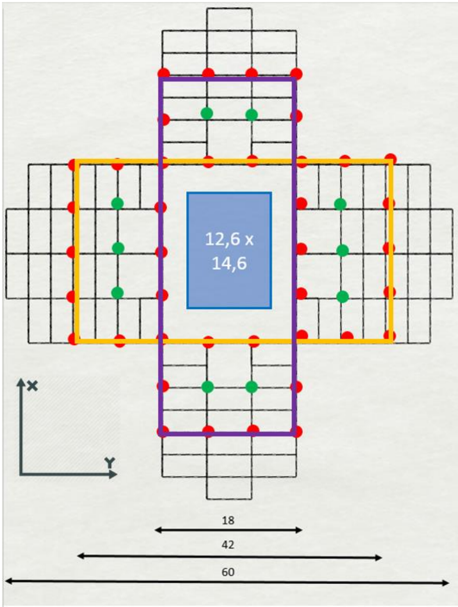


Figure 6 Willis Tower with floorplan (Wikiarquitura, 2010)

These four tubes on the floor plan can be regarded as two tubes. These two tubes can be seen in Figure 7 The purple rectangle shows the tube in X-direction and the orange rectangle shows the tube in Y-direction. The collaboration of these two tubes will always provide stiffness despite the wind direction. The horizontal beams in these tubes are shown with the red squares. These horizontal parts are situated every 3,5 meter. The columns of the tube system are placed every 6 meters. The north elevation of this building is shown in Figure 8. Again, the vertical blue part shows the structural core. As can be clearly seen, the four tubes are absent in the top two floors. In these two floors only the tube around the core is present.



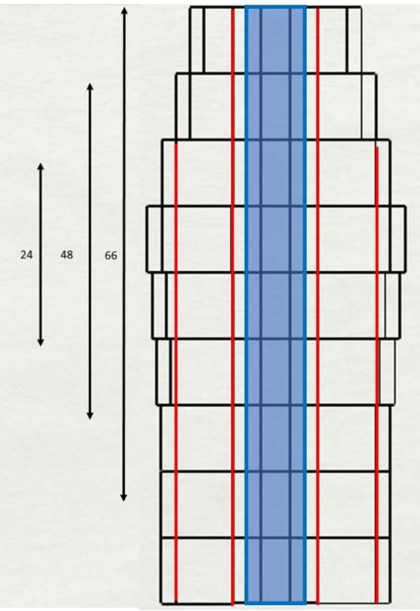


Figure 7 North elevation of the tower (Designed by the authors) *Figure 8 Two tube system (Designed by the authors)*

In the basic calculation of the tube system, there were two main questions. What is the contribution of the use of a structural core for the stability? And what are the needed dimensions of the beams and columns in the tube? The basic calculations done in SCIA showed that a structural core with a thickness of 600mm provides around 40% of the total stiffness when using a tube+core system. So, it is wise to use the core as a structural element. The basic calculation also showed that steel square hollow core sections are need for the beams and column. These hollow core sections will be 800 by 800 mm with a thickness of 25 mm.

Advantages and disadvantages

Based on this project, both structural systems have their (dis)advantages. These are shown in Table 7

Outrigger	Tube
Advantages	
+ No moment resistant connections need to be realised	+ No floors wasted
+ No thick beams in the floors	+ Will result in an easy connection for the cantilevering blocks
	+ Works better with façade elements.
	+ Provides the possibility for easy attachment of extra floors.
Disadvantages	
- 1 meter big profiles needed as the tension and compression elements	- Big beams in the floors
- around 4 floors will be wasted in the big tower were the outrigger is situated.	- Moment resistant connections needed
	- columns of the tube placed every 6 meter and every 3,5 meter high

Table 7 Outrigger vs. Tube (Designed by the authors)

By creating this table of advantages and disadvantages and discussing it with the team, it was decided that the tube system is the most integrated solution.

Optimisation process of the Tube structure

There are five separate forces acting on the model. These are the wind load, the floor loads, the selfweight, the cantilevering blocks and the bridge. As discussed in "Floor Loads, page 8" the snow loads will not be considered. All these forces are divided into three different load cases. Load case 1 is the horizontal wind load. For this load case, the wind load is 2,5 kN/m^2, see "Wind load, page 8" accordingly to the width of the building, see Figure 9.

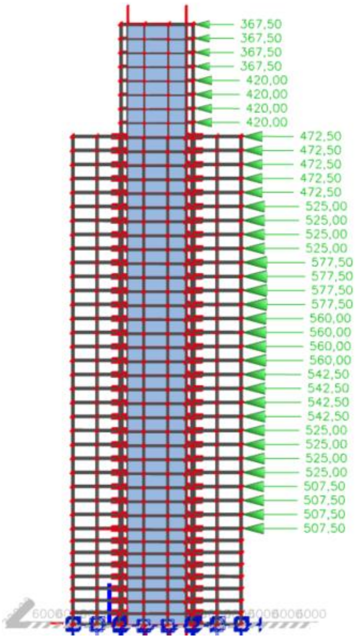


Figure 9 Load case 1, wind (Designed by the authors)

The second load case is the addition of all vertical forces. This total vertical force consists of floor loads, selfweight, cantilevering blocks and the bridge. These forces are added together to find the total weight of the building, see Appendix 2. The total weight of the building is 74.146 kN. For simplicity reasons, this total weight is divided by the number of nodes in the structure. There are 2366 nodes in the full structure, so the vertical force per node is

$$\frac{total\ force}{amount\ of\ nodes} = \frac{74146}{2366} = 31{,}5\ kN\ per\ node$$

The third load case is the moment in the tube structure generated by the connection with the bridge. Since it is not possible to add a moment on the structure in the student version of SCIA, this moment is divided into two forces of 619 kN, seven meter apart.

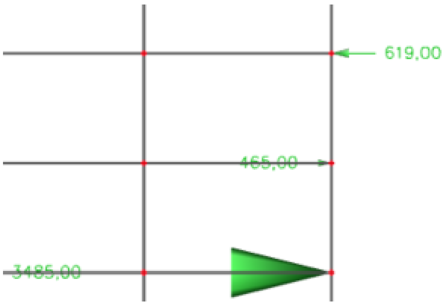


Figure 10 Third load case, moment (Designed by the authors)

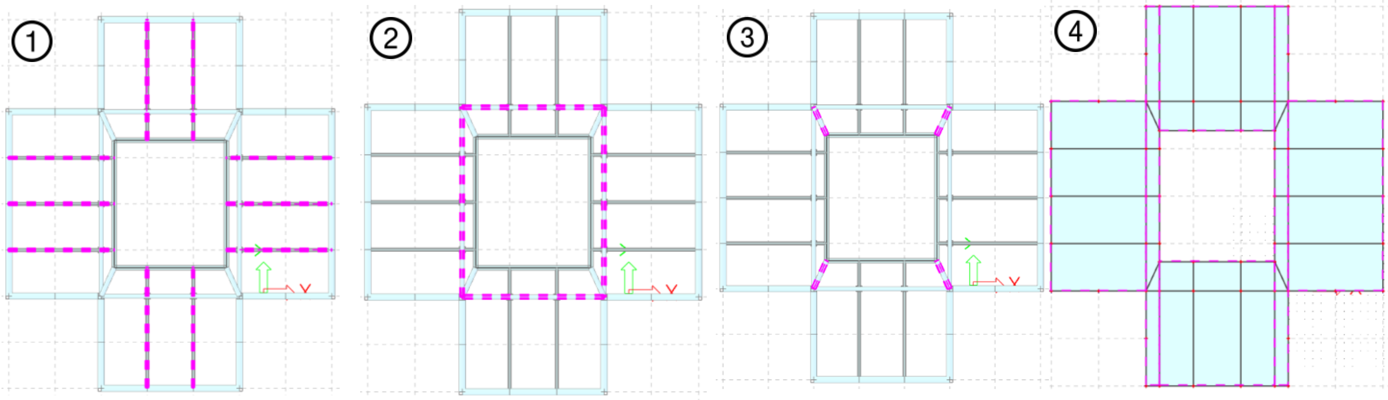


Figure 11 The four optimisation points. (Designed by the authors)

When creating the model, some properties are determined, and some can be changed to achieve certain needs. The location of the floors, the core and the geometry of the tower were given parameters. The positioning of the columns was straight forward, since the grid of the tower is a 6 meter grid. The following parameters were optimized by checking the consequences in various models. The numbers correspond with the optimisation points in see Figure 11.

1. The beams connecting the core and the tube. These beams support the floors in the part inside the tube. So, it is preferred to have the same dimension of these beams for the connection of the core and the tube
2. The inside beams of the tubes. It is preferred to use smaller beams at this position in the tube. Since climate systems from the core must pass these beams.
3. If the beams connecting the core and the tube in 1. are not sufficient, these beams can be added to provide enough stiffness between the core and the tube.
4. The concrete floors and structural topping also provide a connection between the core and the tube and they must be considered for a more exact calculation

A series of iterations were completed. The final model deformed 196,2 mm at the top. So, this gives a unity check of $\frac{196,2}{200} = 0,981$

In this calculation the following properties were used:

	Properties
Core	600mm thick, outside dimensions: 13,8 x 15,8
Concrete Core	C45/55, with a reduced Young's modulus of 20.000 N/mm2
Tube	RHS 800x800x25 except inside beams, see optimization point 2.
Grid	6 meter horizontal 3,5 meter vertical
Steel	S355

Table 8 Properties (Designed by the authors)

The tower is optimized on the following parameters. The numbers correspond with the optimisation points shown in Figure 11. This optimisation can also be found in Appendix 3.

1. The beams used for connecting the core and the tube are the SFB HE200B beams. These are the same beams as the beams supporting the floor inside the tube.
2. The inside beams of the tube are RHS 500x500x20. This provides enough room for the climate systems to pass.
3. It is necessary to use these beams to make sure that the stiffness of the tube and the stiffness of the core collaborate to provide enough stability. RHS 350x350x16 profiles are used.
4. For the calculation, 200mm floors C45/55 was used to model the 200mm thick hollow core slabs and the structural topping.

The elaboration of the floors and beams will be described in Chapter V on page 22.

Dynamic effects

To determine the dynamic response of the building SCIA was used. First, a mass group for the permanent loads and a mass group for the variable loads was created. Then a modal calculation was done to find the eigenfrequencies of the building. The properties of the three eigenfrequencies are shown in Table 9.

eigenfrequency	f [Hz]	ω [1/s]	ω^2 [1/s^2]	T [s]
1	0,35	2,21	4,89	2,84
2	0,37	2,3	5,28	2,73
3	1,06	6,67	44,43	0,94

Table 9 Eigenfrequency (Designed by the authors)

Figure 12 shows the deformations in these three Eigen frequencies. As expected, the first two Eigen frequencies are deformations in the x and y-direction, while the third Eigen frequency is a type of torsion of the building.

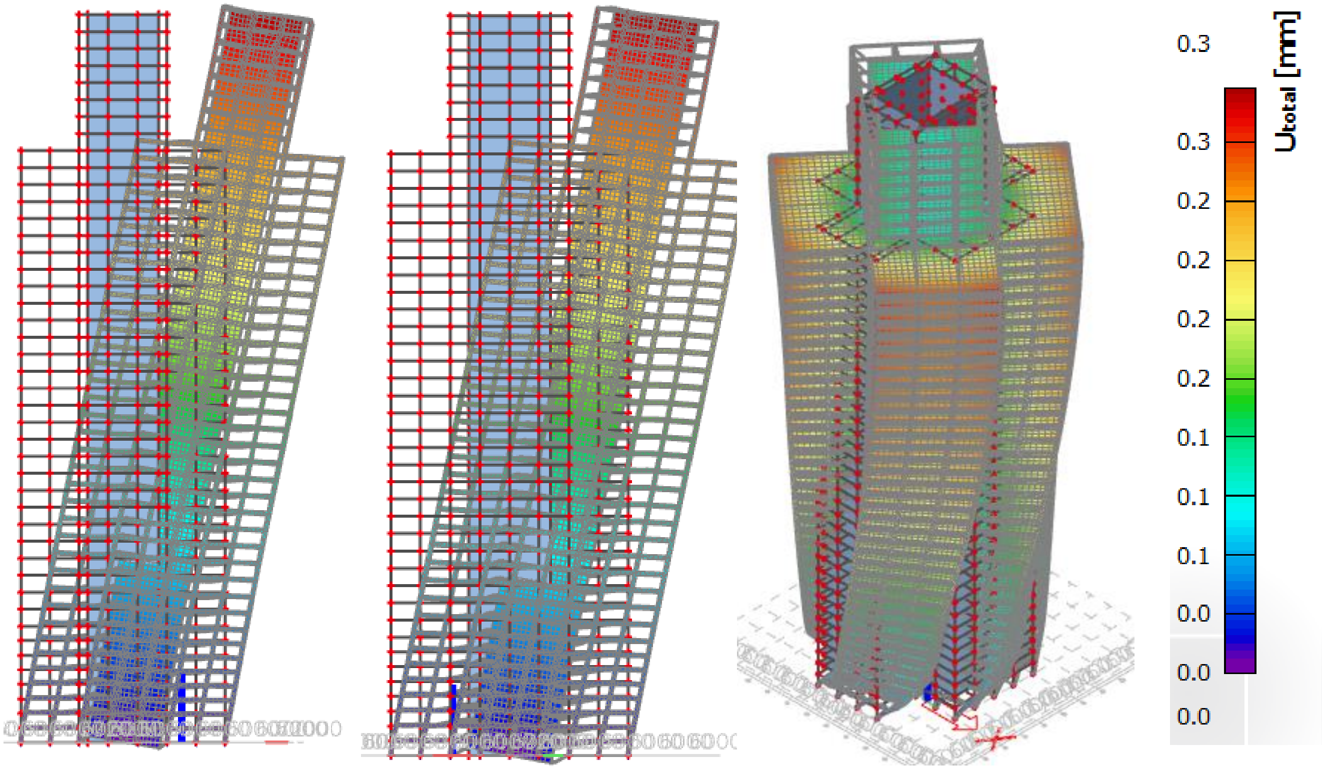


Figure 12 Deformations in Eigen frequencies (Designed by the authors)

Cantilevering blocks

In the middle parts of the tower, the structure cantilevers from the main structural tube system. These cantilevers need a stiff system that will keep the deformation within the limits given by the Belgium code. Every 14 meters the structure cantilevers a different length. Thus, it is decided to create 14 meter high cantilevering blocks which can be connected to the tube system. Figure 13 shows such a cantilevering block with the largest cantilever.

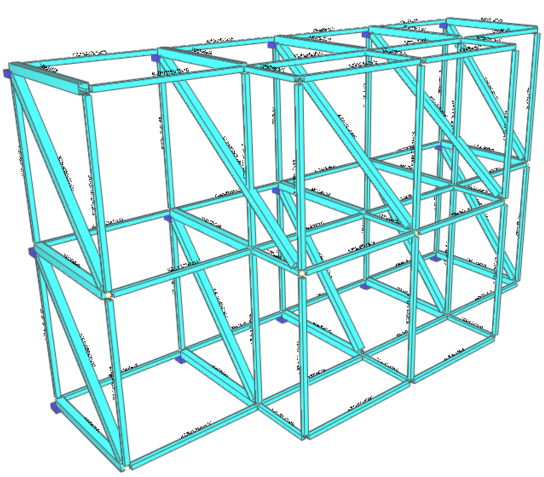


Figure 13 Cantilevering part (Designed by the authors)

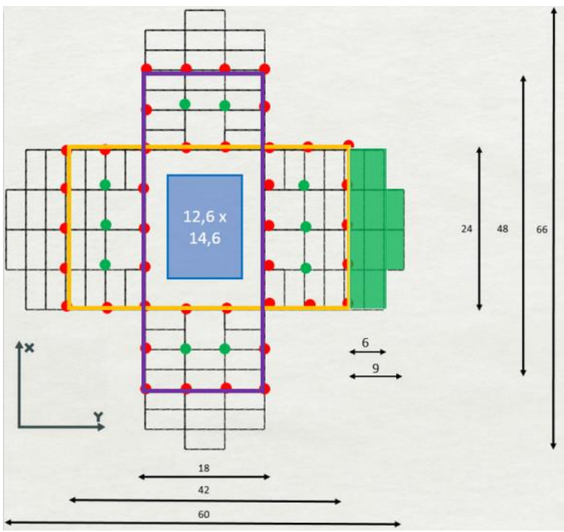


Figure 14 Cantilevering block (Designed by the authors)

The cantilevering block with the largest cantilever is used to find the most optimal geometry and profiles for such a structure.

The maximum deflection is: $Length * 0{,}03 = 6 * 0{,}03 = 0{,}018m$ for the 6 meter cantilevering part and $Length * 0{,}03 = 9 * 0{,}03 = 0{,}024m$ for the 9 meter cantilevering part, see the green marked part in Figure 14.

Firstly, the addition of diagonals in the cantilevering blocks was discussed with the architect and the façade engineer. Both disciplines did not oppose this. So, an analysis was done of the blocks to see the effect of adding diagonals and the optimal positioning of these diagonals. For this analysis the Matrixframe software was used.

Using only boxes with square hollow sections profile of 200x200x15 resulted in a maximum deflection of 1,65m. Adding diagonals with a square hollow section profile of 400x400x20 resulted in a maximum deflection of 0,172m. So, the addition of these diagonals resulted in a ten times smaller deflection.

Some following optimisation steps were done. It was found that the structure would require less material when the floor beams (shown in red) would have different sizes than the rest of the cubes. This is shown in Figure 15.

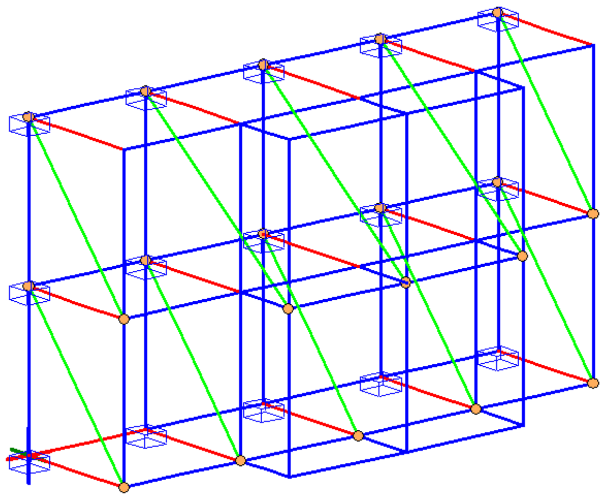


Figure 15 Cantilevering block with beams (Designed by the authors)

For this final solution, the following profiles were found:

- 200x200x12,5mm profiles for the columns and beams shown in blue.
- 350x350x16mm profiles for the beams shown in red and the diagonals shown in green.

The maximum deflection of the nodes was 0,005m and the maximum deflection of the beams is 0,013m. The design process which led to this final solution can be found in Appendix 4.

This calculation was done for the cantilevering block with the largest cantilever. For blocks with a smaller cantilever the diagonal will have less effect. For cantilevering blocks with a smaller cantilever, it is also aimed to use the same floor beam as the floor beams in the main structure. This is shown in red in Figure 15.

Details

- Corner profile

In Figure 16 a detail of a connection from the floor to the core is shown. With the use of a bolted corner profile, the beam can be attached to the core.

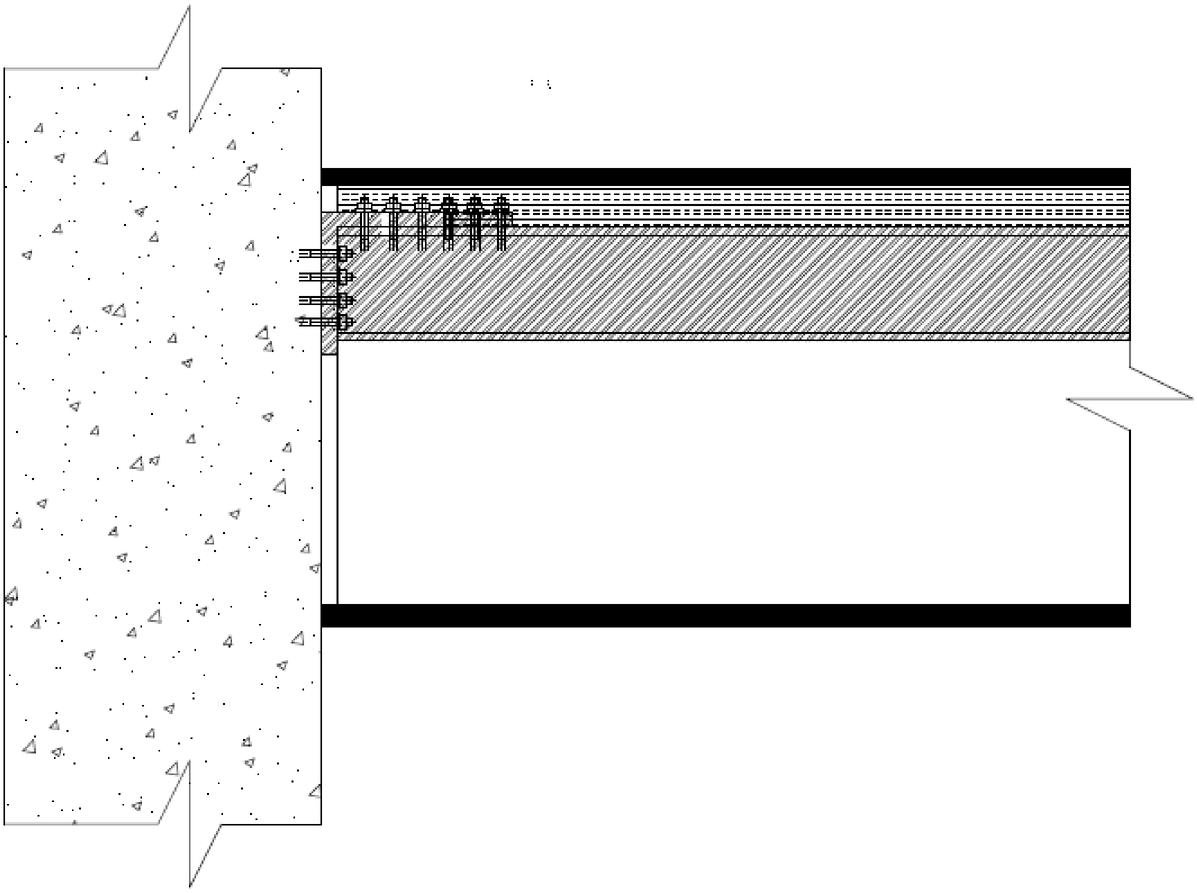


Figure 16 Detail corner profile (Designed by the authors)

V. The Plinth

Besides the two towers, also lower buildings have to be realised. They will be built in the plinth. This plinth is not attached to the towers. By realising the plinth as a self-contained part, it will not be affected by the settlements of the tower. However, the plinth will need its own structural stability.

The height of the plinth is 24m. For this height two possibilities are considered as a structural system. A *Frame Structure* or a *Core Structure* (See Chapter III, page 7). The Core structure is already briefly examined before and has his own (dis)advantages. A sketch of a frame structure is shown in Figure 17. It can be used for towers up to 6 floors (≈25m).

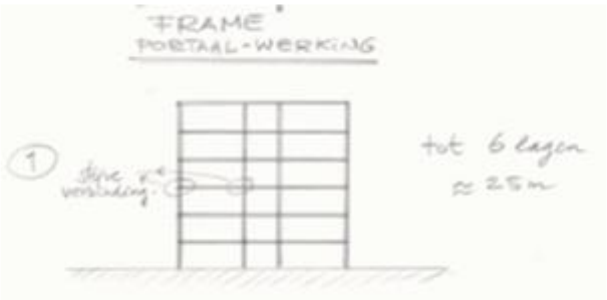


Figure 17 Frame structure (Nijsse, 2019)

The Core system is chosen to be realised since cores are needed for vertical transport. These can be used in the structural system. Since the plinth is considered to be separate of the towers it will cause a 'hole' in these buildings, which is hard to realise with a frame structure. Besides, the safe fire routes can be put into the cores.

The Plinth is divided in three parts. This is shown in Figure 18. In yellow the cores of the towers are shown, and in blue the tube structure of the towers is shown. In green the extra cores of the areas are shown. These locations are an estimation and it is up to the architect what the exact locations will be.

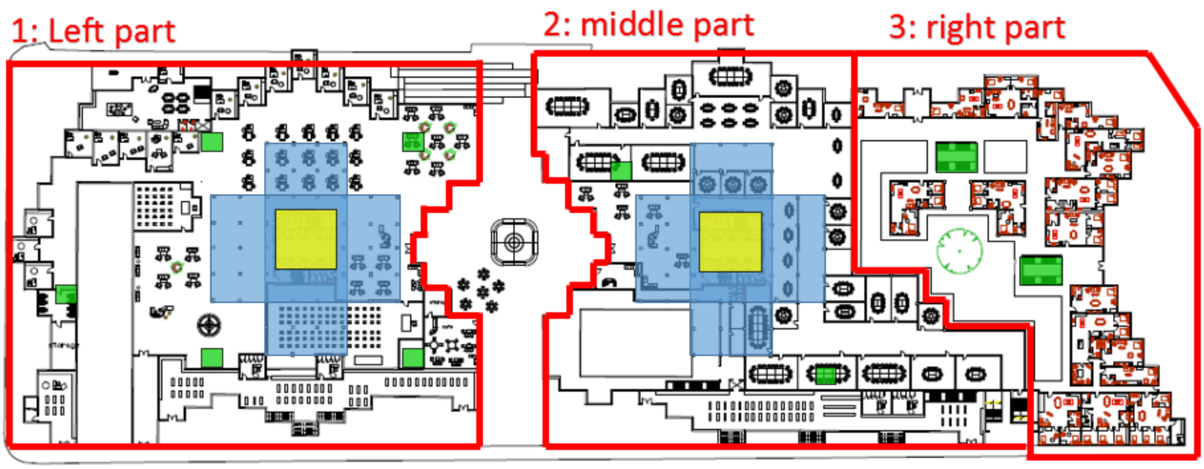


Figure 18 Plinth divided in three parts (Designed by the authors)

The calculations and input variables are shown in Appendix 5. All the cores will have an area of $16m^2$, the width and the length will be 4000x4000mm. Only the cores in the left part (including the part with the monument) have a wall thickness of 200mm. The others will be realised with a thickness of 300mm. These thicknesses are also enough to provide fire safety.

The Monument

The monument is a highly valued building at the Plinth. Obviously, it must be maintained, and the new designed buildings and towers must not do any harm to this monument. However, the foundation isn't known yet and thus only assumptions can be made in the begin of the design phase. The monument won't be attached to the Plinth. This way a different settlement of the plinth and the monument won't occur and thus cause any damage. It is assumed that the monument has its own stability system.

The most important aspect is to make sure the foundation won't get damaged. Vibration free foundations piles will become not only an option for the foundation, but due to this monument it will be demanded. Furthermore, the soil-drilling tests must be studied to know the exact soil type. As a final note, the foundation must be at a certain distance from the foundation of the monument. This way the soil will not deform too much or experience too much settlement. This can be done with a rule of thumb for the distance of 4*D. Where D is the diameter of the foundation pile. With the use of a good inspection while putting the foundation piles in the ground, the settlement can be observed. If settlement will occur, the foundation of the monument has to be improved. Foundation recovery can be divided in three types:

- **Building a new foundation with the use of a *Table structure* or an *Edge beam.***
  Table structure: A new floor on fragment piles will be able to carry the consisting walls with the use of grooves (in Dutch: "inkassingen").
  Edge beam: Close to the walls, fragment piles will be pushed into the soil. With the use of edge beams the load can be carried. The edge beams are connected to the walls with the use of the grooves. (Joostdevree, 2016)
- **Strengthen the subsoil with an injection technique.**
  With a so called two component resin, the holes can be filled, this is simplified shown in Figure 19.

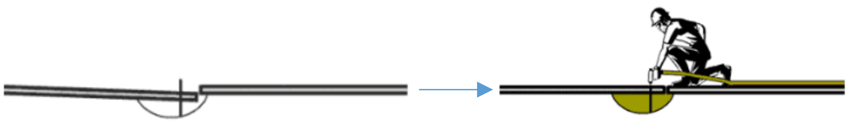


Figure 19 Injection technique (Uretek, 2016)

- **Jacking up the monument**
  When the monument gets settlement due to the Plinth, jacking up the building (In Dutch: "Opvijzelen") will cause it to get to its original position again.

1. Current Situation
2. Underneath the monument a reinforced concrete slab is placed
3. Through the floor, hollow steel piles are drilled, which will be filled with concrete
4. With the jacks, the building is pulled up at the piles

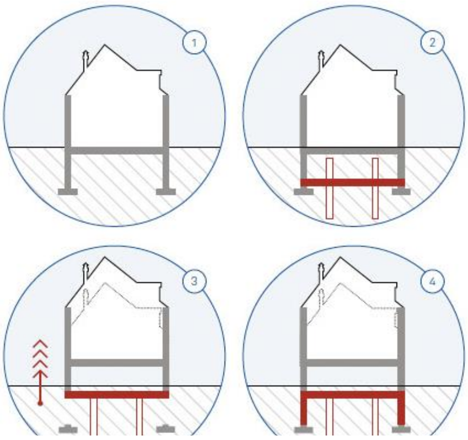


Figure 20 Jacking up (Changemagazine, 2015)

VI. Floors, beams and columns

Since the type of structural system of the towers is decided, the floors and beams must be realised. However, this does not only depend on the strength of these elements, but it also must deal with the requirements from the climate designers and the architect. The façade is load bearing which must be taken into account with the design of these elements. Different systems will be briefly shown with their advantages. This way the final beams and floors can be chosen for the towers.

Different systems

Floors

Floor systems

For the realisation of the high-rise buildings, a particular floor type is required. The floors need to be able to withstand the dead- and the variable loads of the building. Besides the fact that the floor needs to be load bearing it must also contain the climate systems. Three common floor types for high rise buildings will be compared.

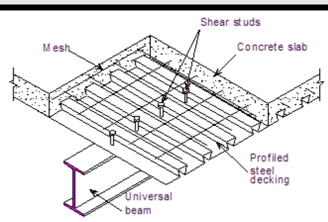
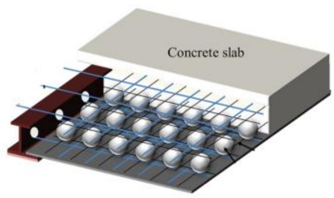
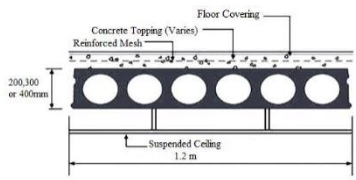
Floor type	Brief	Max Span	Sketch	Advantages and price
Composite steel-concrete floor	-System floor in the shape of a concrete formwork -Mostly used as storey floors in a steel framework	9 meters		+Able to withstand high loads +Limited use of crane capacity +Low dead load +Easy possibilities for lining of services *75€/m2 (t=150)* *100€/m2 (t=200)*
Biaxial hollow slab floor	-Monolithic floor with plastic balls which lowers the self-weight of the concrete floor	14 meters		+Low self-weight +High erection speed +Smaller construction height +Relatively cheap *110€/m2 (t=230)* *150€/m2 (t=450)*
Hollow core beam floor	-Consists of prefab reinforced concrete slabs which are provided with hollow 'canals' -The floor only needs a screed floor of ±50mm to finish it	18 meters		+Low self-weight +Can withstand high loads +Relatively cheap +High erection speed *45€/m2 (t=200)* *80€/m2 (t=400)*

Table 10 Comparison floor types (Joostdevree, 2015/ Steelconstruction, 2013/ Hollowcore, 2014)

Flexible floors

The client desires flexibility. This means that the functions of the floors in the tower must be able to change after some years. At the moment the towers consist of offices or conference rooms. To make sure that all the functions (restaurant, offices, housing etc.) can be able to be used in the tower, the biggest loads are taken into account for the calculations. To create a flexible floor, a computer floor (or raised floor) can be realised. This will perform as a second floor, above the structural floor. In between these floors there is some spacing available for cables. This raised floor will be about 250mm high. It will provide an easy access to installations; the room can be air conditioned from the floor and the tiles or floor type can easily be changed. An impression of this floor is shown in Figure 21. (ibm, 2016)

Figure 21 Computer floor (teconstrucor, 2017)

Another option is to make use of a self-levelling screed. This is mostly used for housing. In between the load bearing floor and the screed is a so-called spring layer. This floor has multiple advantages:

- *Prevents noise pollution*. Which is essential in housing
- *Flexible*. A self-levelling screed is easy to remove
- *Thermal insulation*. An underfloor heating can be placed in between the two layers.

Beams

In the previous chapter the floor systems are discussed. However, these floors have to be carried by beams, which will transfer the loads to the columns. For the beams there are multiple options. For these towers the beam choice is narrowed down to only two systems. A *stacked floor system* and an *Integrated floor system*. Both are shown in Figure 22.

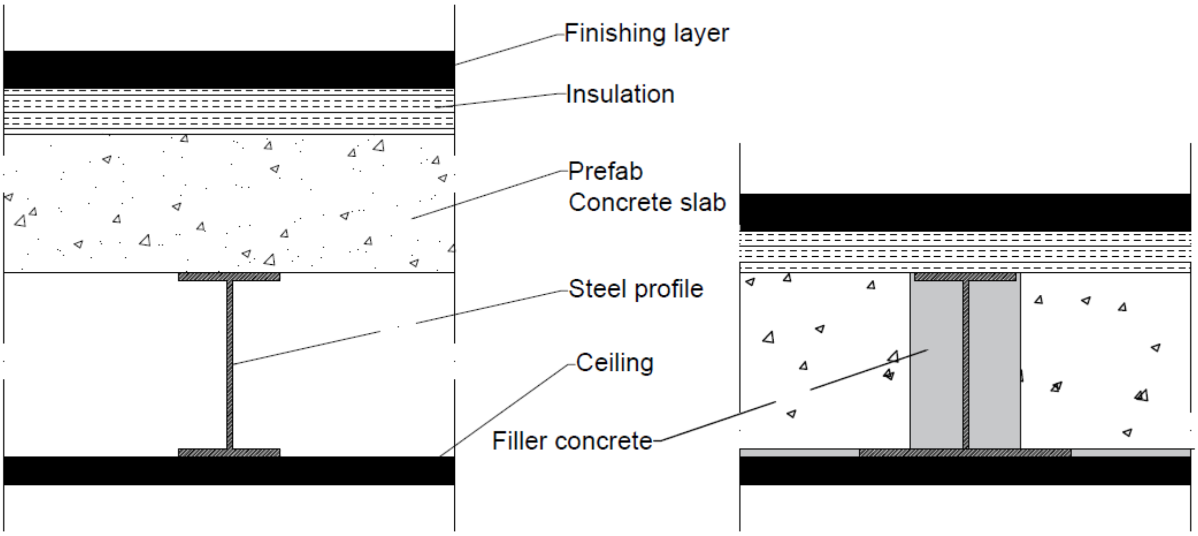


Figure 22 Stacked floor system and Integrated floor system (Designed by the authors)

For a *stacked floor system* multiple beam profiles can be used, for example: IPE beams, squared hollow beams or HEA beams. All these beams have their own (dis)advantages. However, they have some things in common. The most important ones are a low self-weight (less use of material) and their use will result in a relatively thick floor. Besides the beams, a ceiling, load-bearing floor system, insulation and room for climate tubes must be in the total floor. This will lead to a floor of about 1m thickness.

For an *Integrated floor system* two profiles can be used. HE-profiles or so-called top hat-profiles. (Both shown in Figure 23. They both have their own (dis)advantages but have in common that they are at the same height as the load-bearing floor system. This way the insulation and a finishing layer can be placed right on top and won't need hard work to finish it.

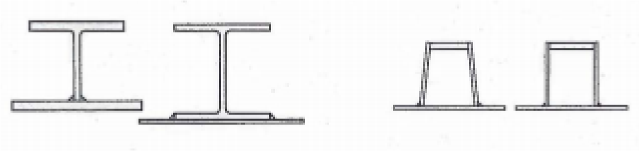


Figure 23 HE-profiles and top hat-profiles (Pasterkamp, 2014)

Final floor system

The floors will consist of hollow core slabs. Just like the other floor it has a low self-weight, but it will also be the cheapest. The hollow core slab is a prefab slab, which is commonly used in Belgium and the cranes are already at the building site for the steel frame.

On top of the load bearing floor, a screed floor is needed. With this screed the necessary floor height is realized. For practical and financial reasons, this isn't done immediately with the load bearing floor. The choice of this extra floor is not only based on the structural discipline. This screed also has to deal with requirements and demands from the climate designers and the architect. Everyone discipline has a different optimal solution and eventually a compromise is chosen for all the floors in the towers. This screed is also necessary for the floor covering. Two different screed floors will be used in the tower. A self-levelling screed and a computer floor. Both types are already discussed before. The computer floor is used in the offices. When the function of the room changes to housing, this floor can easily be removed and changed into a self-levelling screed floor. Both the computer floor and screed floor are shown in Figure 24.

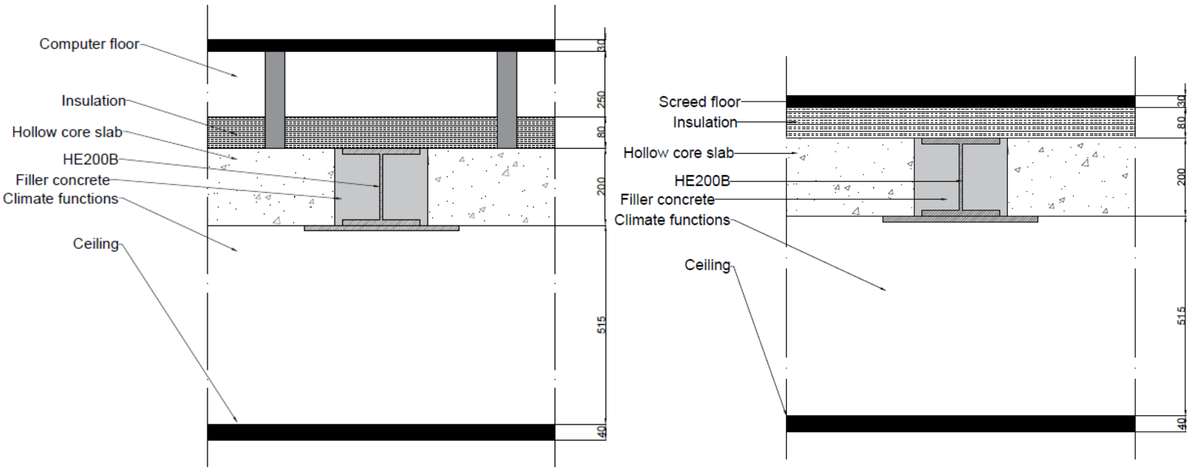


Figure 24 Computer floor and screed floor (Designed by the authors)

Final beam system

The beams have been compared to each other. The calculations and input variables are shown in Appendix 6. To decide which beam must be chosen, at first the moment distribution and the maximum shear force must be calculated. This is done with the sketch in Figure 25.

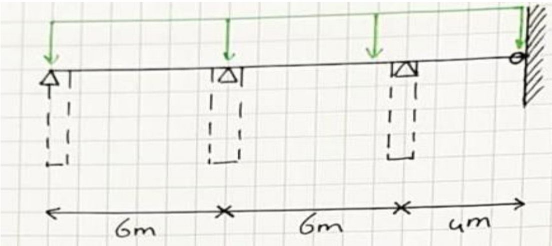


Figure 25 Sketch beam span (Designed by the authors)

In Table 11 a summary is given of the final calculations. For both beams the total deflection is levelled to the maximum deflection $w\_max = 14 * 0.03 * 1000 = 42mm$. This will give the stress that is required for the profile. For the SFB HE200B this stress is below 355, while for the TopHat200 it is above 355. Thus, the used beam will be the SFB HE200B.

SFB HE200B			**TopHat200**		
Load	27.45469	kN/m	*load*	20.19816216	kN/m
w	12.9	mm	w	9.5	mm
UC	0.307143	-	UC	0.226190476	-
W	655000	mm3	W	514000	mm3
Sigma	325.1908	N/mm2	Sigma	414.3968872	N/mm2
UC	0.916031	-	UC	1.167315175	-

Table 11 Final calculations beams (Designed by the authors)

Columns

For the columns in the towers, bigger columns are needed at the bottom of the tower than at the top. Because using a lot of different columns is expensive, the towers were divided in column groups. Since a steel tube structure is used as the structural system, the columns will also be made of steel. For each column group the load on a normative column was determined. Using this load, the minimum surface area of a column is determined. Then, the column was checked for buckling. These calculations can be found in Appendix 7.

For the columns it was determined to use different HD320 profiles. These profiles will be used because they offer a big area relative to their dimensions and because the inside depth between the flanges is 225 mm for all profiles. This way it is easy to connect the different types of columns to each other. The profiles used for each column group are given in Figure 26.

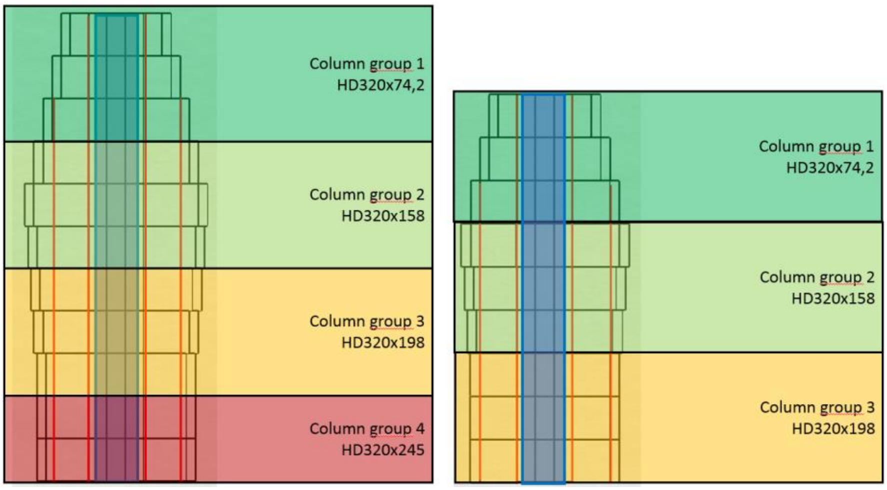


Figure 26 Column groups (Designed by the authors)

VII. The Bridge

In between the two towers a bridge will be made, which connects both towers. This bridge will provide a fire safety route, extra functions like a restaurant and offices and stability between two towers in eastern and western wind directions.

The bridge will be 59 meters long, 14 meters high and 12 meters wide. These are quite some big distances for a bridge to hang on the towers. Thus, a truss has to be realised. This truss will be attached to the tubes of both towers with a moment resisting connection. To realize a good combination for the dimensions of the truss, load case 4 has been used. The input values are shown in Table 12 in the blue boxes. The bridge is first calculated in 2D with only the variable loads.

Dimensions Truss		
Height	14	m
Width	59	m
Depth	12	m
LC4	6.072	kN/m2
LC4_per width	72.864	kN/m
#Floors	2	-
Total Variable load		
Qload_perwidth	145.728	kN/m
Qload_perdepth	89.562	kN/m

Table 12 Starting values Truss (Designed by the authors)

With the use of the program MatrixFrame and the Qload per width from Table 12 the optimal combination can be formed. As a start the truss is calculated with the maximum variable load. This way the dimensions of the truss can be calculated. These dimensions are determined with the maximum deformation. The maximum deformation is $\frac{length}{300} = \frac{59}{300} = 196.667mm$. This deformation is the total of the x- and y-direction combined (thus along the length and the width). The tube structure of the tower can deform because of the wind load, which will cause a deformation in the bridge. For this reason, the total deformation of the truss for each direction is half the maximum deformation. In both directions a maximum deformation of $\frac{196.667}{2} = 98.33mm$ is accepted.

With the dimensions of the truss the self-weight of the truss is known. With the use of a deck shear (in Dutch: "zeeg") the deformation of the self-weight can be compensated. This idea is shown in Figure 27. With the use of the deck shear, the bridge becomes the shape in red. When the truss is built, it will deform due to his own load into the blue rectangle again.

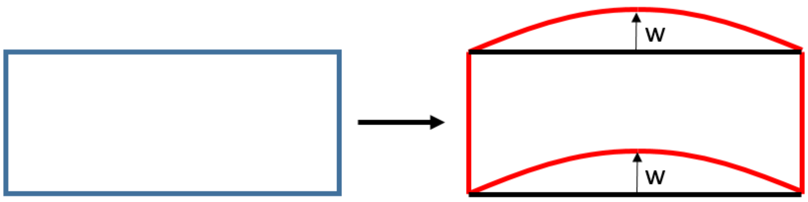


Figure 27 Sketch of deck shear (Designed by the authors)

For the optimisation to find the optimal truss, the differences between 6, 7 and 8 nodes have been tested in the first stage. An optimal combination with respect to deformation, weight of the truss and the biggest acting force has been tried to be found. This is shown in Table 13. It is easily seen that the option with 6 nodes dropped off quickly. In Appendix 8 the MatrixFrame sketches are shown to make the deflections visible. The colours show the comparison of the values. The darker the red, the more unfavourable. Lighter green means more favourable.

Optimal combination					
Horizontal x Diagonal (thickness)	Nodes	Max w [m]	[mm]	Total weight Truss [kN]	Biggest moment [kNm]
700(20)x400(15)	8	0.0774	77.4	2502	2095
700(20)x400(15)	7	0.0798	79.8	2370	1936
700(20)x400(15)	6	0.0877	87.7	2243	2232
600(15)x400(10)	7	0.1309	130.9	1565	1787
600(15)x400(10)	8	0.1229	122.9	1656	1754
700(20)x400(10)	7	0.1056	105.6	1988	2364
700(20)x400(10)	8	0.1056	105.6	2079	2270
600(15)x500(15)	8	0.0804	80.4	2631	1312
600(15)x500(15)	7	0.0921	92.1	2426	1549
500(15)x500(15)	8	0.0995	99.5	2290	1180
600(15)x400(15)	8	0.0933	93.3	2081	1384

Table 13 Optimal combinations truss (Designed by the authors)

After some optimisation, the last combination has been chosen. The horizontal tubes of the truss will have a diameter of 600mm with a thickness of 15mm and the diagonals will be 400mm with 15mm thickness. However, these dimensions also need to be checked in the other dimension. The deformations are both shown in Figure 28.

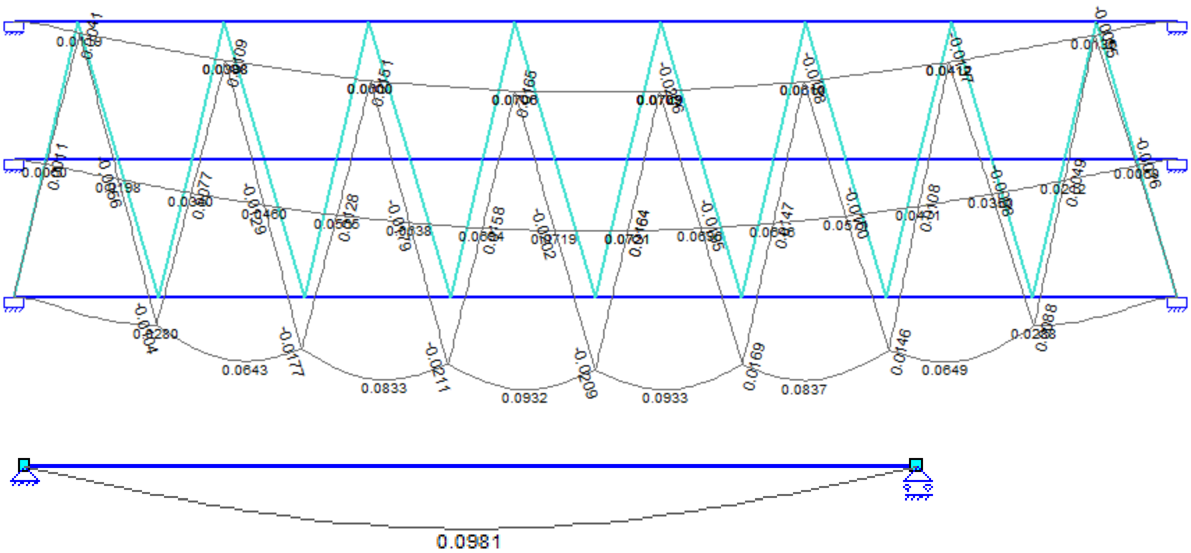


Figure 28 Deformations in x- and y-directions (Designed by the authors)

The total deformation will be $(0.0981 + 0.0933) * 1000 = 191.4mm$ which is lower than the maximum deflection of 196.667 mm.

Since the dimensions are known, the deformation/deck shear of the bridge can be calculated. The variables and calculations are shown in Appendix 9. The total loads are summarised in Table 14.

Total load G+Q		
Load_perwidth	251.1006	kN/m
Load_perdepth	607.644	kN/m
Point load	1234.578	kN
Point load/2	617.289	kN

Table 14 Total load values (Designed by the authors)

With the use of this table and MatrixFrame, a 3D-model can be made. This model is assumed with loads of the whole bridge and moment resistant connections between the bridge and the tower structure. This is shown in Figure 29. In grey the deformations are shown. The maximum deflection will be 80.4mm.

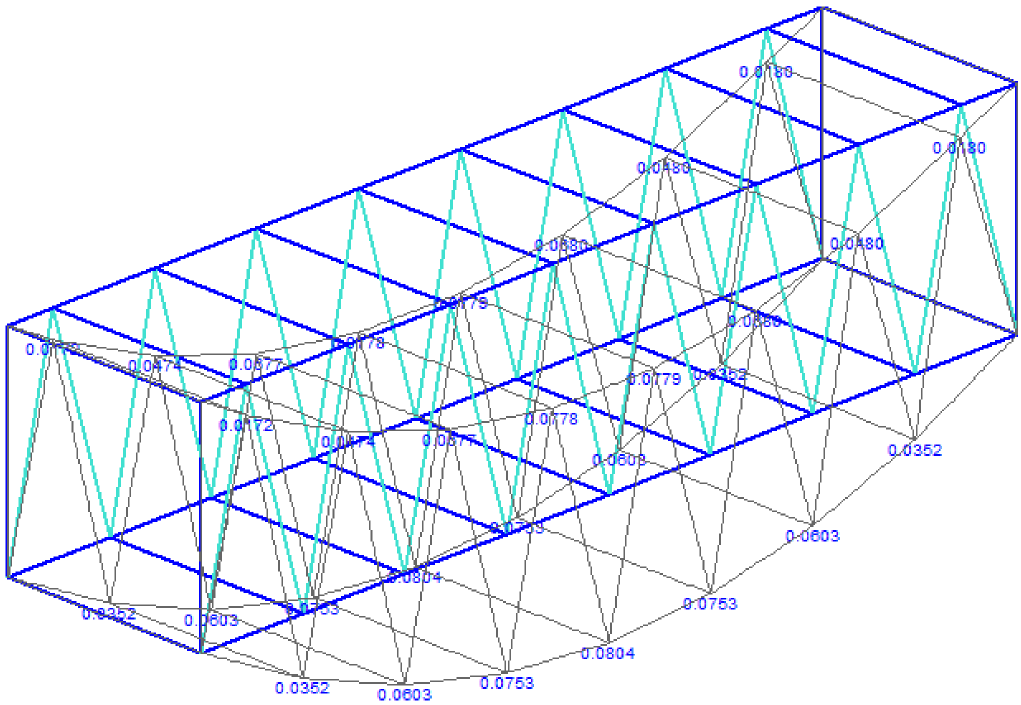


Figure 29 Final deformations in 3D (Designed by the authors)

Details

- Moment resistant connection

With a steel plate, the truss of the bridge will be connected to the tube structure. The tubes of the truss will be deformed at the end, to make it flat. This way the steel plate can be bolted to the tubes.

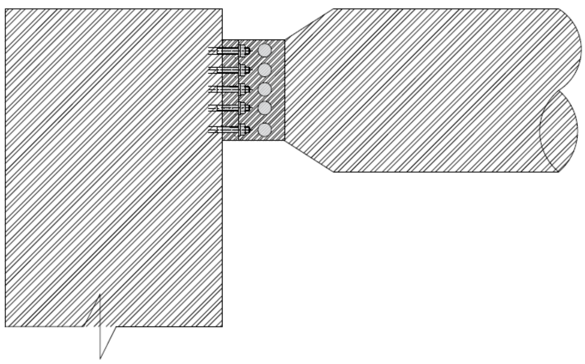


Figure 30 Moment resistant connection (Designed by the authors)

VIII. Underground

Soil

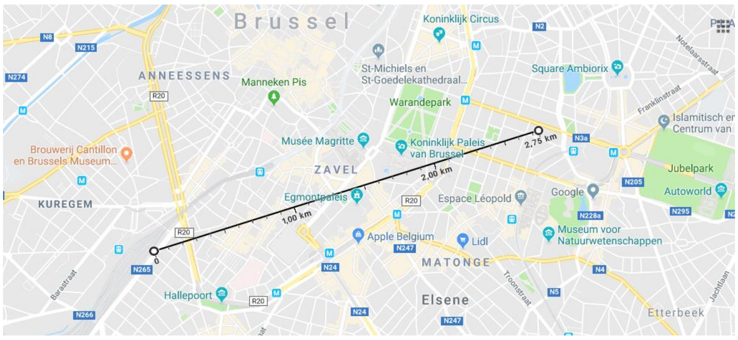


Figure 31 Distance between the plinth and the south station (GoogleMaps, 2019)

Via OREX Geotechnics, soil-drilling tests are provided for the area around the South station of Brussels. These soil-drilling tests are about 2.75km away from the plinth, as seen in Figure 31. However, these soil-drilling tests will give a good estimation and will be used as a reference for the plinth. In Figure 32 an indication of the geology of Brussels is shown. It shows that the most important area is covered with Clay (purple area). Below this clay area, a sand layer and further on a rock layer is found.

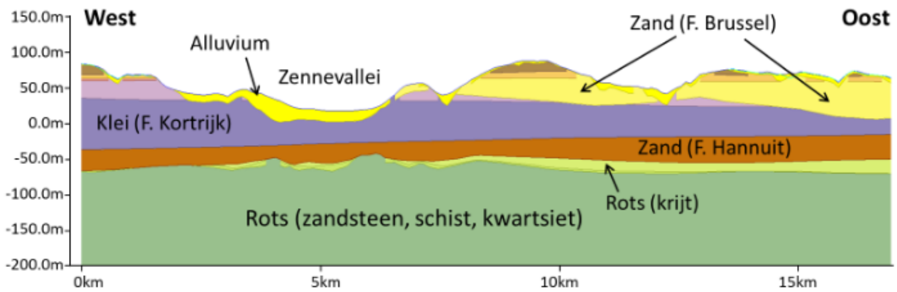


Figure 32 Geology of Brussels (OverheidVlaanderen, 2019)

Three soil-drilling tests are shown in Appendix 10. In these tests the groundwater table (GWT) is marked with a red circle. In every test this GWT is around +16.5m measured from sea level. The ground level is at +21m. This means that the GWT is 4.5m below the surface. As seen in Figure 32 the first layer in which the foundation can be placed is at about -30m. Thus, the foundation has to be at least 50m long.

Foundation piles

There are multiple types of pile foundations. In Table 15 four of them are briefly explained. *Prefab concrete, Vibro combi, Fundex, and Baretten*. In the last column it is shown how to apply each foundation pile. Each pile has his own (dis)advantages.

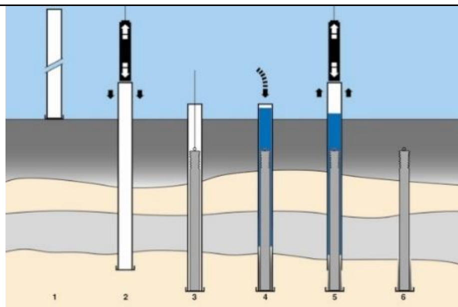
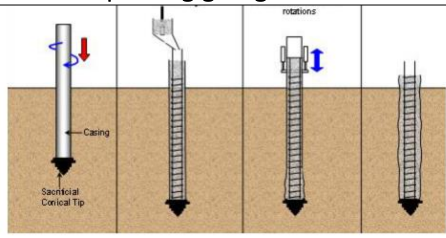
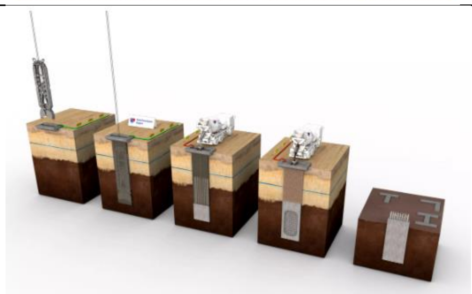
Pile names	Max length	Driven or Drilled	Prefab or cast-in-situ	Diameter	Maximum Force	Short explanation
Prefab concrete	36m	Driven	Prefab	180-500 mm	4.000kN	Concrete prefab piles driven in the soil.
Vibro combi	±85x diameter of the prefab element =(20-50m)	Driven	Both, As the prefab pile is installed, it can be topped with cast-in-situ concrete	270-610 mm	5.000kN	 Ideal for parking garages.
Fundex	40m	Drilling	Cast-in-situ	380-540 mm	6.000kN	
Diaphragm wall elements ('Baretten')[1]	70m	Drilling	Cast-in-situ	600-1500 mm	5.000-50.000 kN	

Table 15 Different types of pile foundation (Ffgb, 2016/ terracon, 2017/ Vroom, 2018/ vsf, 2018)

As the load bearing layer is at about 40-50m depth, the *prefab concrete* pile will drop off. The application of the piles is also influenced by the surroundings. A driven pile is not really desirable, due to the fact of the many nearby buildings. From Table 15 can be concluded that the Baretten are the best option. They are also used in the 'The One Tower', which is located next to the plinth. These foundation piles are especially used when on great depths, big forces have to be transferred. For example, when a high-rise is built. The only disadvantage is that the bentonite and the soil have to be removed from the area. These piles are not only quietly, but also vibration free.

The width of these 'Baretten' can vary from 0.6 – 1.5m, the length very from 2800-3200mm and each pile can withstand 5.000 to 50.000 kN. With an area of 2,8mx0,8m (=2,24m^2), a pile strength of 20.000kN can be used to calculate with. An average load of 18 kN/m^2 is used as a rule of thumb for high rise buildings. In the Table 16, with some simple basic calculations, a first indication of the amount of foundation piles that are needed is shown.

[1] Lose panels of the diaphragm walls turned into columns. The Dutch name is 'Baretten'

Foundation calculation		
Load per floor	18	kN/m2
Amount of floors	42	-
Avarage floor space	1600	m2
Total load	1209600	kN
Load capacity per pile	20000	kN
Amount of piles needed	61	-
Area ground floor	1600	m2
Area per pile	26.22951	m2

Table 16 Foundation calculation (Designed by the authors)

Since the design of the towers are more detailed and more values are known, the total amount of loads and the area per floors can be used for a better calculation of the foundation piles. This is shown in Table 17. In here the safety factors are also applied on the total load of the tower.

Foundation calculation Final		
Load Tower	618193	kN
Weight of the core	120643.2	kN
Load half the bridge	3551	kN
Total load	741416	kN
Load capacity per pile	20000	kN
Amount of piles needed	38	-
Area ground floor	1716	m2
Area per pile	45.15789	m2

Table 17 Final foundation calculation (Designed by the authors)

As seen in the table, the load of the bridge is almost negligible in comparison to the total weight of the whole tower.

Parking garage

The parking garage will have the same load bearing structure as the tower above it. This way the load can be carried the easiest way. As already said before, the GWT is 4.5m below the surface. This has to be taken into account when digging, thus damming walls must be realised.

For the parking spots is a rule of thumb used. For each 200 m^2 of office space, one parking spot is required (Dobbelsteen, 2019). At first the area of a parking spot is calculated, which will provide the total area of parking spots required. However, there is also space required for the cars to get to the parking spots. Thus, an area of 25 m^2 per parking spot (ocw.tudelft, 2018) is used as a rule of thumb. It is al shown in Table 18.

Parking garage area		
Parking spot length	5	m
Parking spot width	2.5	m
Parkingspot area	12.5	m2
1 Parking per … office space	200	m2
Total office area	152000	m2
Amount of parking spots	760	-
Total parking spot area	9500	m2
Area per spot	25	m2
Total area parking garage	19000	m2
Total area Plinth	24000	m2
Area left	5000	m2
If 2 floors, area needed:	9500	m2
If 3 floors, area needed:	6333.333	m2

Table 18 Parking garage area (Designed by the authors)

The total area of the whole parking garage will be 19.000 m^2. This includes everything, from roads till barriers. The total area of the plinth is about 24.000m^2. The parking garage will be divided in two floors of each 9.500m^2. The parking garage will be 92m by 103m. This will be big enough to reach the required area for the parking garage. The location of the parking garage is shown in Figure 33 in the left bottom.

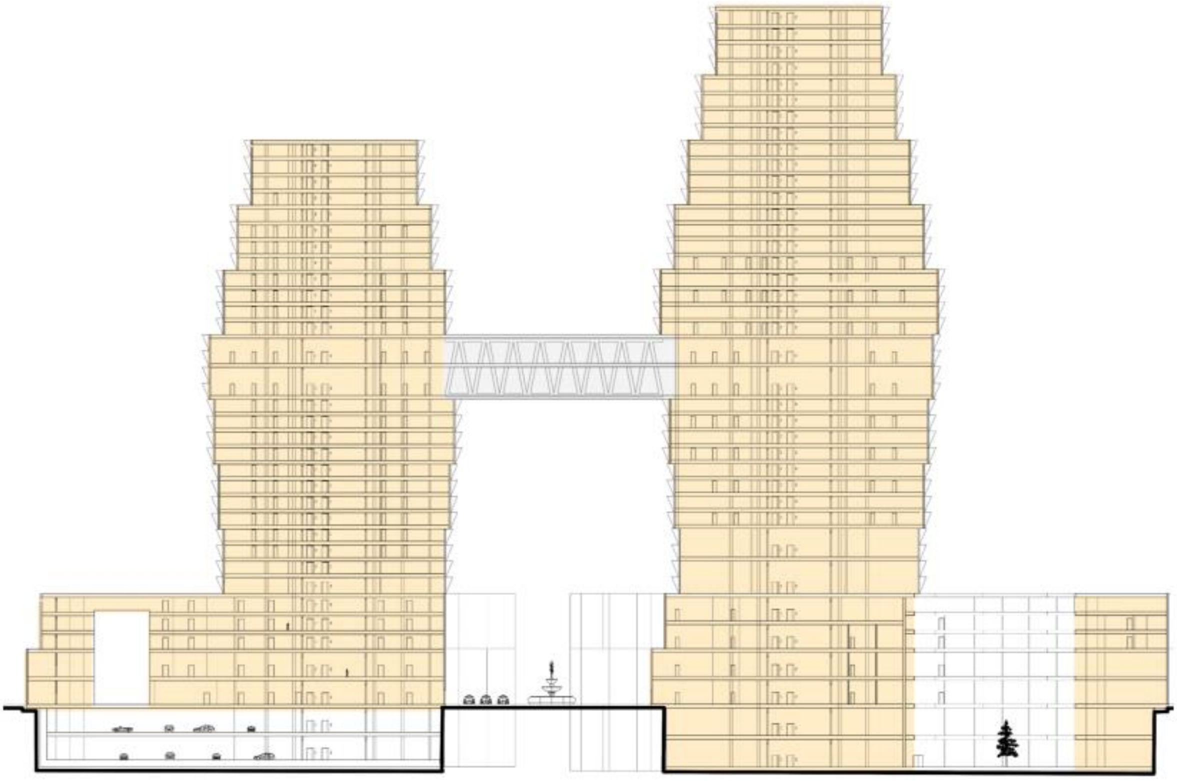


Figure 33 Side view plinth, including parking garage (Designed by the authors)

Building pit

As known from the soil analysis, the GWT is at 4.5m below the surface. This means, when the digging starts for the parking garage and the foundation piles, a huge swimming pool will be created. This is not desirable. Thus, the groundwater must be held back. For this, multiple options are available:

- **Pump out**
  By pumping the water out of the construction pit, the groundwater table will be lowered locally until below the construction pit. This way it is possible to work in the dried soil.
  Advantage: No 'useless' elements in the soil
  Disadvantages: Has influence on the surrounding soil, energy consuming, water needs to be transported.
- **Diaphragm walls**
  Steel, concrete or wooden walls can be used as a water-resistant construction. These walls consist of loose elements which can be connected with a tongue and groove joint[2].
  Advantages: Can be vibrated in the soil, reusable, can take every specific shape, high strength.
  Disadvantages: Big elements can be tough to transport
- **Underwater concrete**
  With the use of a specific concrete mixture, the concrete is able to harden underwater. When the concrete is hardened, it will be able to keep out the underlying water and the water which is on top can be pumped out. This concrete is mostly used in combination with diaphragm walls.
  Advantages: No continuous pumping of the water, the bottom layer of the garage is already placed, Serve as a brace for the diaphragm walls
  Disadvantages: Mixture needs very specific attention, needs time for hardening
- **Permeation grouting**
  An impermeable layer is formed by an injection liquid, which consists of water-glass, water and a hardener. The sand grains will attach to each other and form some kind of gel. Unfortunately, this isn't possible in the plinth, due to the fact that the top layer doesn't consist of sand. (Bodeminjectie, 2014)

Unfortunately, the exact soil-drilling tests were not available for this phase of the design stage. The calculations that are done and the conclusions that are made are based on assumptions. The options mentioned above can be considered when a more advanced research is done for the Plinth.

Settlement of the towers

The two towers are connected using a rather stiff skybridge. This presents a challenge regarding the foundation. Uneven settlement of the towers will result in deformation of the bridge. When this deformation of the bridge is restricted by the construction, this will create additional forces. Thus, measurements must be taken to minimize these deformations. The first measurement is realising the two towers in phases. The towers will already have experienced their primary settlement before the structure of the bridge is added. This way, only the secondary settlement of the two towers will result in deformation in the bridge. Another measurement that can be taken is the connection of the bridge to the towers using dilatation. This way deformation will not cause extra forces. The third measurement that will be taken is using the same type of foundation for both towers and making sure that each pile will support approximately the same weight for both towers.

---

[2] "Each piece has a slot (the groove) cut all along one edge, and a thin, deep ridge (the tongue) on the opposite edge. The tongue projects a little less than the depth of the groove. Two or more pieces thus fit together closely" (Woodwork, 2008)

IX. Structural Redundancy

The Plinth and the towers have a big risk of terrorist attacks. The Maelbeek subway station underneath the plot was hit by a terrorist attack on March 22nd of 2016. This risk must be considered for the structural design of the building. The structure must have enough structural redundancy to withstand a terrorist attack where a column gets blown up.

For the structure of the plinth two measures need to be taken for the design to have enough structural redundancy when one column gets blown away. At first it must be made sure that the floor has enough strength to span the distance between the two columns adjacent to the destroyed column. Secondly, these two columns must be able to support the full wait of the column.

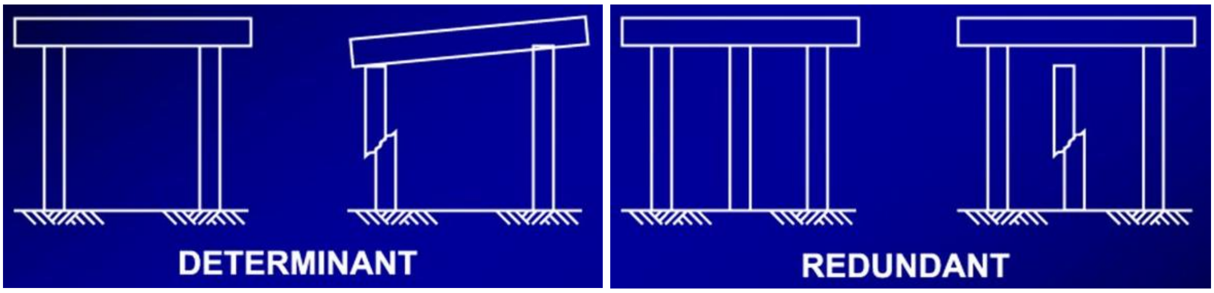


Figure 34 Determinant and redundant elements (IBM, 2015)

For the towers it first must be checked if there are any key elements. A key element is a determinant element regarding the structural redundancy of a building, see Figure 34. By deleting elements in the structural elements and checking the effect which a missing element has on the building it can be checked if the structure possesses any key elements.

X. Conclusion

Designing a high-rise building is a complex process. Multiple disciplines cooperated to finish the realisation of the Pine. Every choice made by one discipline affected the other disciplines. Thus, a good communication and teamwork was required to obtain an integrated design. The structural part of this final integrated design is shown in this report. This chapter gives a summary of the final structure.

- *Structural scheme*

The load bearing structure of the towers will be a tube structure made of steel connected to a concrete core. Since the main focus was on the towers, the plinth has been calculated with simplified rules of thumb.

- *Floors, beams and columns*

The floors will consist of 200mm thick hollow core slabs. Just like the other floor it has a low self-weight, but it will also be the cheapest. Two different screed floors will be used in the tower. A self-levelling screed and a computer floor. Both types are already discussed before. The computer floor is used in the offices, and the self-levelling screed will be used if the function changes to housing.
For the floors, an integrated floor system is used. In this system, beams with a SFB HE200B profile are used. The columns are divided into column groups. In these different column groups different HD320 profiles are used. These profiles will be used because they offer a big area relative to their dimensions and because the inside depth between the flanges is 225 mm for all profiles. This way it is easy to connect the different types of columns to each other.

- *Bridge*

The bridge will be realised with a big truss with steel members. These steel members are round hollow sections and will be used for horizontal and diagonal beams with respectively a diameter of 600mm and 400mm. Both have a thickness of 15mm.

- *Foundation*

For the foundation Baretten are used. This is necessary because of the big depth of the load-bearing layer.

XI. Discussion

This paper discussed the structure in the preliminary design stage. Due to the lack of information of the project, a lot of assumptions had to be made through the whole design, this resulted in a lack of exactness. When more information is obtained, more exact calculations can be done in the later design stages. In this discussion some personal recommendations will be given for calculations in these later design stages.

At first the structural system is calculated with the use of simplified rules of thumb. However, since the design of the tower is that complicated, these values wold deviate too much with the reality.

In this report the bridge is connected to the tube using a moment resistant connecting. When a more detailed design is made, it might be wise to use a hinged connection. Using a hinged connection will improve the ability of the bridge to take up deformations.

The foundation can be further optimised. The soil survey can be improved if more information is obtained.

For the cantilevering blocks, in further research it can be determined for which cantilevers the use of the diagonals will still be useful. For the floor beams in the cantilevering blocks it will be more optimal to use I-beams rather than rectangular hollow core sections.

In the calculation of the horizontal deformation and the dynamic effects of the tower, the tower was assumed to have a moment resistant support. However, the foundation will deform and cause some deformations and rotation in the supports. For the horizontal deformation this was considered by restricting the maximum horizontal deformation to be $w_{max} = \frac{height}{750}$

XII. References

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XIII. Figure List

XIV. Table list

XV. Appendices

Appendix 1 General Floor loads

Qconference	2,5	kN/m2
Qhousing	1,75	kN/m2
Gfloor	2,9	kN/m2
Gfacade	3	kN/m
GHollow Core slab	2,4	kN/m2
GService loads	0,5	kN/m2
Consequence Class = CC3		
K	1,1	
gammaQ	1,5	
gammaG	1,2	
Total loads on floors per 7 meter		
2 residential floors every 3,5 meter	13,431	kN/m2
1 confecerence floor every 7 meter	7,953	kN/m2
Facade	3,96	kN/m

Appendix 2 Calculation total weight of big tower

Big Tower (3,5 meter floors)				
x-direction	y-direction	floor area [m2]	perimeter[m]	weight per floor (7 meter) [kN]
36	42	1044	180	29470
42	48	1296	204	36429
48	54	1548	228	43388
54	60	1800	252	50347
60	66	2052	276	57307
58	64	1968	268	54987
56	62	1884	260	52667
54	60	1800	252	50347
52	58	1716	244	48028
52	58	1716	244	48028
52	58	1716	244	48028
		1685,454545		**519025**

Total weight of floor loads and selfweight of floors

t	25	mm
l & b	800	mm
steel weight	8050	kg/m3
Surface are of tube per m	77500	mm2/m
Surface are of tube per m	0,0775	m2/m
weight	623,875	kg/m
Tube below		
Length	122	m
N columns total	34	
beams	264	m
N floors	36	
Tube above		
Length	28	m
N columns total	14	
beams	84	m
N floors	8	
weight		
columns to 122m	2587834	kg
columns from 122m	244559	kg
beams to 122m	5929308	kg
beans from 122 m	419244	kg
total		
total weight	9180945	kg
total load	91809	kN

Total weight of tube system

height	150	m
thickness	0,6	m
weight concrete	23,6	kN/m3
Core inside width	14,6	m
Core inside depth	12,6	m
Core outside width	15,8	m
Core outside depth	13,8	m
total m3/m	34,08	m3/m
m3 of concrete	5112	m3
weight of core	120643,2	kN

Total weight of Core

support reaction 1	728	kN
support reaction 2	2106	kN
support reaction 3	4268	kN
half of the weight of the bridge	7102	kN

Total weight of half of the bridge

total weight	Profile	mass [kg/m]	total weight [kN]
Column group 1	HD320x74,2	74,2	311,64
Column group 2	HD320x158	158	663,6
Column group 3	HD320x198	198	831,6
Column group 4	HD320x245	245	1029
		total weight	**2835,84**

Total weight of the columns

	weight [kN]
Weight of floors	519.025
Weight of tube	91.809
Weight of core	120.643
Weight of half the bridge	7.102
Weight of columns	2.836
Total weight of the big tower	**741.416**

Total weight of the big tower

Appendix 3 Optimization of big tower

					Optimization point				Wmax = 200
Tube	Core	Bridge attached	Exact wind loads	Floor loads added	1	2	3	4	w[mm]
700x700x25	No	No	No	No	Same as tube	Same as tube	No	No	360
700x700x25	13,8x15,8 t=600	No	No	No	Same as tube	Same as tube	No	No	227
900x900x25	No	No	No	No	Same as tube	Same as tube	No	No	217,4
900x900x25	13,8x15,8 t=600	No	No	No	Same as tube	Same as tube	No	No	142,4
1000x1000x25	No	No	No	No	Same as tube	Same as tube	No	No	179
700x700x25	13,8x15,8 t=600	Yes	No	No	Same as tube	Same as tube	No	No	238
900x900x25	13,8x15,8 t=600	Yes	No	No	Same as tube	Same as tube	No	No	152
800x800x25	13,8x15,8 t=600	Yes	Yes	No	Same as tube	Same as tube	No	No	188,3
900x900x25	13,8x15,8 t=600	Yes	Yes	No	Same as tube	Same as tube	No	No	154,1
800x800x25	13,8x15,8 t=600	Yes	Yes	No	Same as tube	Same as tube	No	No	134,8
Dimensions changed according to latest model and fully stiff connection between tube and core									
800x800x25	13,8x15,8 t=600	Yes	Yes	No	HE200B	Same as tube	No	No	225
800x800x25	13,8x15,8 t=600	Yes	Yes	No	HE500B	Same as tube	No	No	164,5
800x800x25	13,8x15,8 t=600	Yes	Yes	No	RHS600	Same as tube	No	No	163,5
800x800x25	13,8x15,8 t=600	Yes	Yes	No	HE200B	600x600x25	RHS 350x350x16	No	183,5
800x800x25	13,8x15,8 t=600	Yes	Yes	No	HE200B	600x600x25	RHS 350x350x16	200 mm	169,1
800x800x25	13,8x15,8 t=600	Yes	Yes	Yes	HE200B	600x600x25	RHS 350x350x16	200 mm	192,3
800x800x25	13,8x15,8 t=600	Yes	Yes	Yes	HE200B	500x500x20	RHS 350x350x16	200 mm	196,2

Appendix 4 Cantilevering cubes

Floors cantilevering cubes		
Biggest floor	180	m2
number of floors in model	2	
lines per floor	39	m
total lines for line load	78	
total load	2417,58	kN
line load	30,99462	kN/m

wmax = 0,018 m and line load = 31 kN/m				
Cubes	Diagonals	Floor beams	Wtotal [m]	info
200x200x15	-	200x200x15	1,65	-
400x400x20	-	400x400x20	0,01446	-
200x200x15	400x400x20	200x200x15	1,02	only non-visible diagonals
200x200x15	400x400x20	200x200x15	0,0692	all diagonals
200x200x15	400x400x20	200x200x15	0,3825	no diagonals in facade
200x200x15	400x400x20	200x200x15	0,331	no diagonals in the middle
200x200x15	400x400x20	400x400x20	0,0108	floor beams bigger
200x200x15	350x350x20	350x350x20	0,0157	
200x200x12,5	350x350x16	350x350x16	0,018	

Appendix 5 Plinth core calculations

Core calculations right part

INPUT VARIABLES		
Height	24	m
Maximum defelction	0.032	m
windload	1.44	kN/m2
windload	57.6	kN/m
floor load	0	kN/m2
thickness walls	0.3	m
WIDTH>DEPTH		
buildingwidth	80	m
buildingdepth	40	m
Innerwidth	79.4	
Innerdepth	39.4	
E (C45/55)	15000	N/mm2
Selfweight concrete	24	kN/m3
Width core	4000	mm
Depth core	4000	mm
Thickness core	300	mm
Inside width of core	3400	mm
Inside depth of core	3400	mm

CORE STRUCTURE					
I_OuterCore	2.13333E+13	mm4	I_InnerCore	1.11E+13	mm4
W_OuterCore	10666666667	mm3	W_InnerCore	6.55E+09	mm3
2 Cores, thus everything *2					
total I	2.03944E+13	mm4	20.3944	m4	
total W	8232000000	mm3	8.232	m3	
A	8880000	mm2	8.88	m2	
Formulas	**Tension and compression stresses**				
Selfweight * A	q (selfweight core)	213.12	kN/m		
q*height	N(core at ground)	5114.88	kN		
(1/2)*qwind*width*height^2	M	33177.6	kNm		
M/W - N/A	σ (tension)	3454.321	kN/m2	3.454321	N/mm2
M/W + N/A	σ (compression)	-4606.32	kN/m2	-4.60632	N/mm2
	Deflection				
	EI	3.06E+08			
((qwind*width)*height^4)/(8*EI)	δ	0.015617	m	15.61728	mm
	w max	0.032	m	32	mm
δ/wmax	UC	0.48804			

Core calculations middle part

INPUT VARIABLES		
buildingwidth	110	m
buidlingdepth	80	m

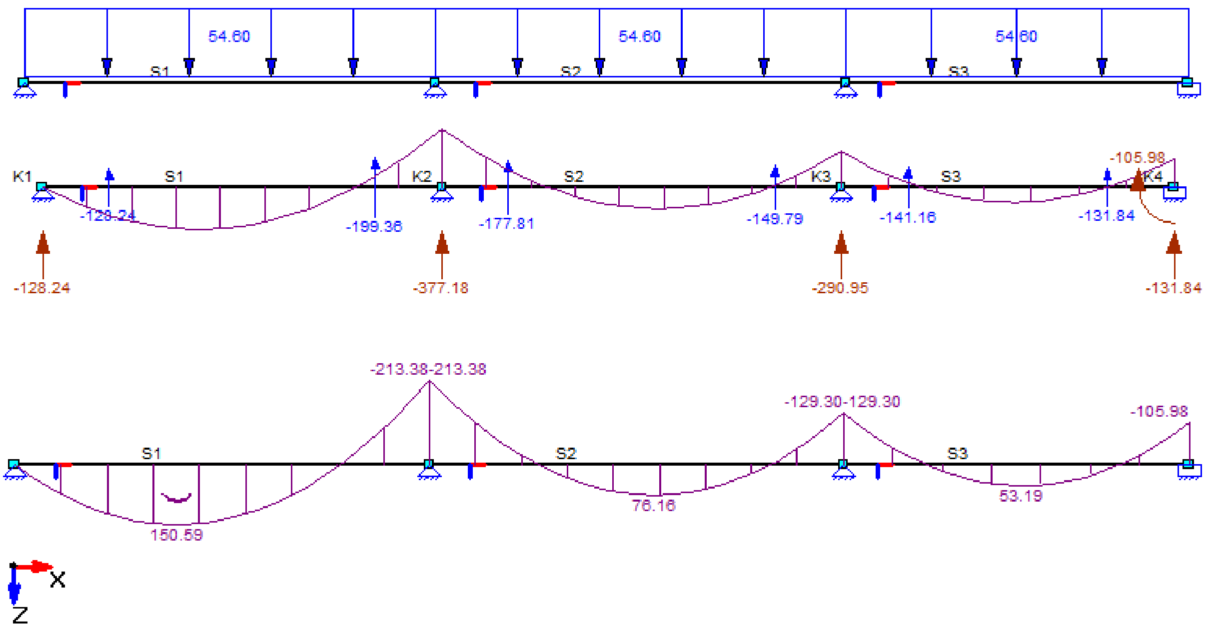
CORE STRUCTURE					
I_OuterCore	2.13333E+13	mm4	I_InnerCore	1.11E+13	mm4
W_OuterCore	10666666667	mm3	W_InnerCore	6.55E+09	mm3
2 Cores, thus everything *2					
total I	2.03944E+13	mm4	20.3944	m4	
total W	8232000000	mm3	8.232	m3	
A	8880000	mm2	8.88	m2	
	Tension and compression stresses				
Selfweight * A	q (selfweight core)	213.12	kN/m		
q*height	N(core at ground)	5114.88	kN		
(1/2)*qwind*width*height^2	M	45619.2	kNm		
M/W - N/A	σ (tension)	4965.691	kN/m2	4.965691	N/mm2
M/W + N/A	σ (compression)	-6117.69	kN/m2	-6.11769	N/mm2
	Deflection				
	EI	3.06E+08			
((qwind*width)*height^4)/(8*EI)	δ	0.021474	m	21.47375	mm
	w max	0.032	m	32	mm
δ/wmax	UC	0.671055			

Core calculations left part

INPUT VARIABLES		
Height	24	m
Maximum defelction	0.032	m
windload	1.44	kN/m2
windload	115.2	kN/m
floor load	0	kN/m2
thickness walls	0.3	m
WIDTH>DEPTH		
buildingwidth	95	m
buidlingdepth	80	m
Innerwidth	94.4	
Innerdepth	79.4	
E (C45/55)	15000	N/mm2
Selfweight concrete	24	kN/m3
Width core	4000	mm
Depth core	4000	mm
Thickness core	200	mm
Inside width of core	3600	mm
Inside depth of core	3600	mm

CORE STRUCTURE					
I_OuterCore	2.13333E+13	mm4	I_InnerCore	1.4E+13	mm4
W_OuterCore	10666666667	mm3	W_InnerCore	7.78E+09	mm3
4 Cores, thus everything *4					
total I	2.93461E+13	mm4	29.34613333	m4	
total W	11562666667	mm3	11.56266667	m3	
A	12160000	mm2	12.16	m2	
	Tension and compression stresses				
Selfweight * A	q (selfweight core)	291.84	kN/m		
q*height	N(core at ground)	7004.16	kN		
(1/2)*qwind*width*height^2	M	39398.4	kNm		
M/W - N/A	σ (tension)	2831.38	kN/m2	2.83138	N/mm2
M/W + N/A	σ (compression)	-3983.38	kN/m2	-3.98338	N/mm2
	Deflection				
	EI	4.4E+08			
((qwind*width)*height^4)/(8*EI)	δ	0.012888	m	12.8884	mm
	w max	0.032	m	32	mm
δ/wmax	UC	0.402762			

Appendix 6 Beam calculations



Integrated beams	h [mm]	I x 10^4 [mm4]	W x 10^3 [mm3]	G [kg/m]	
HE200B	200	9629	655	110.5	
TopHat200	200	5959	514	59.8	
Hollow core slab			**Moment calculation**		
g	9.81	m/s	Depth	6	m
Area	84	m2	q	18.2	kN/m2
Weight	2500	kg/m3	q	54.6	kN/m
Weight reduce	50	%	*With Matrixframe:*		
Height	200	mm	Vmax	377	kN
Total weight	206.01	kN	Mmax	213	kNm
Total weight	14.715	kN/m	Mmax	213000000	Nmm
Profile check					
ULS			Qwind	2.5	kN/m2
Width	14	m	Qoffices	3	kN/m2
Depth	6	m	psi_office	0.5	kN/m2
K	1.1	-			
LC4	$K_{FL} * 1{,}2G + K_{FL} * 1.5Q_{wind} + \sum K_{FL} * 1.5Q_{office} * \psi$				
Q_load	18	kN/m			
w_max	42	mm			
Steel	S355				

HE200B			**TopHat200**		
Own weight	15.17607	kN	Own weight	8.212932	kN
Own weight_perwidth	1.084005	kN/m	Own weight_perwidth	0.586638	kN/m
Total weight	15.79901	kN/m	Total weight	15.301638	kN/m
Load	27.45469	kN/m	*load*	20.19816216	kN/m
w	12.9	mm	w	9.5	mm
UC	0.307143	-	UC	0.226190476	-
W	655000	mm3	W	514000	mm3
Sigma	325.1908	N/mm2	Sigma	414.3968872	N/mm2
UC	0.916031	-	UC	1.167315175	-

Appendix 7 Columns of the big tower

weight per columngroup		number of floors	load on normative column [kN]
	Column group 1	6	2901
	Column group 2	6	5802
	Column group 3	6	8703
	Column group 4	4	10637

Found profiles	steel = S355	Needed A [mm2]		A of profile [mm2]	mass [kg/m]
	Column group 1	8172	HD320x74,2	9460	74,2
	Column group 2	16344	HD320x158	20100	158
	Column group 3	24516	HD320x198	25200	198
	Column group 4	29964	HD320x245	31200	245

Buckling force	Buckling force	Profile	I[mm4]	Buckling force [kN]	load on normative column [kN]	U.C.
	Column group 1	HD320x74,2	49.600.000	4.515.687	0	0,00064
	Column group 2	HD320x158	118.000.000	10.742.965	37600376	0,00054
	Column group 3	HD320x198	153.000.000	13.929.437	86353406	0,00062
	Column group 4	HD320x245	197.000.000	17.935.288	149126915	0,00059

Appendix 8 Bridge Calculations

Dimensions Truss		
Height	14	m
Width	59	m
Depth	12	m
#nodes	8	-
LC4	6.072	kN/m2
LC4_per width	72.864	kN/m
#Floors	2	-
Total Variable load		
Qload_perwidth	145.728	kN/m
Qload_perdepth	89.562	kN/m
Point load knots	307.06971	kN
P/2	153.53486	kN
Gtruss	2081	kN
Gtruss_per width	35.271186	kN/m
Gtruss_perdepth	173.41667	kN/m
Max allowable w	196.66667	mm
Weight floors		
Width floors	1200	mm
#Floors	50	-
g	9.81	m/s
#stories	2	-
Total area floors	1416	m2
Weight concrete	2500	kg/m3
Weight reduce	50	%
Height	200	mm
Total weight	3472.74	kN
Gfloors_per width	58.86	kN/m
Gfloors_per depth	289.395	kN/m
Weight beams		
HE200B	110.5	kg/m
Weigth_beams	1.084005	kN/m
#Beams	50	-
Length beams	12	m
Total weight	650.403	kN
Gbeams_per width	11.02378	kN/m
Gbeams_per depth	54.20025	kN/m

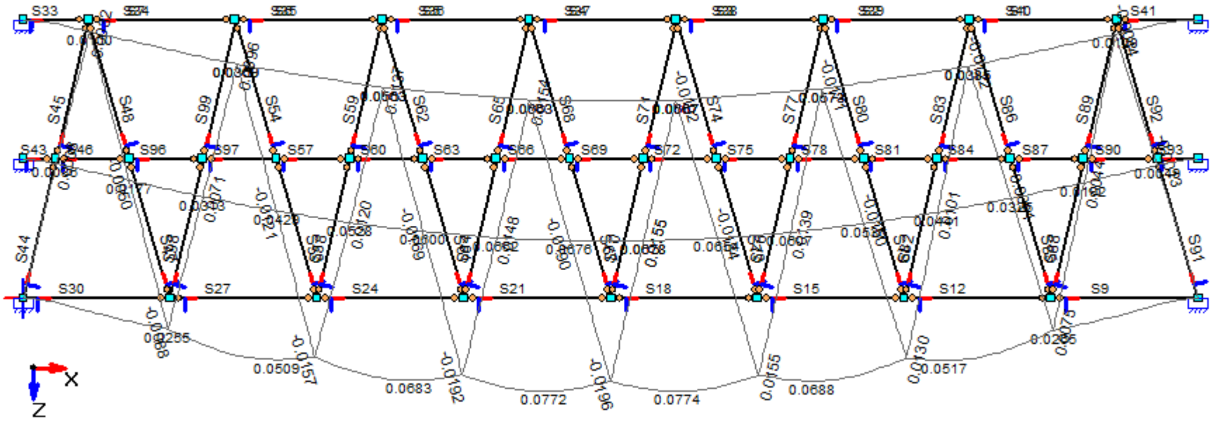
Weight Columns		
Distance between nodes	7.375	m
#Columns needed per width	8	-
#Columns_per depth	1	-
Total #columns per floor	8	-
Total #columns in the bridge	16	
Column profile	HD400x187	
Column weight	187	kg/m
Height	7	m
Weight per column	12.84129	kN
Total weight columns	205.46064	kN
Column per_width	0.217649	kN/m
Column per_depth	1.0701075	kN/m

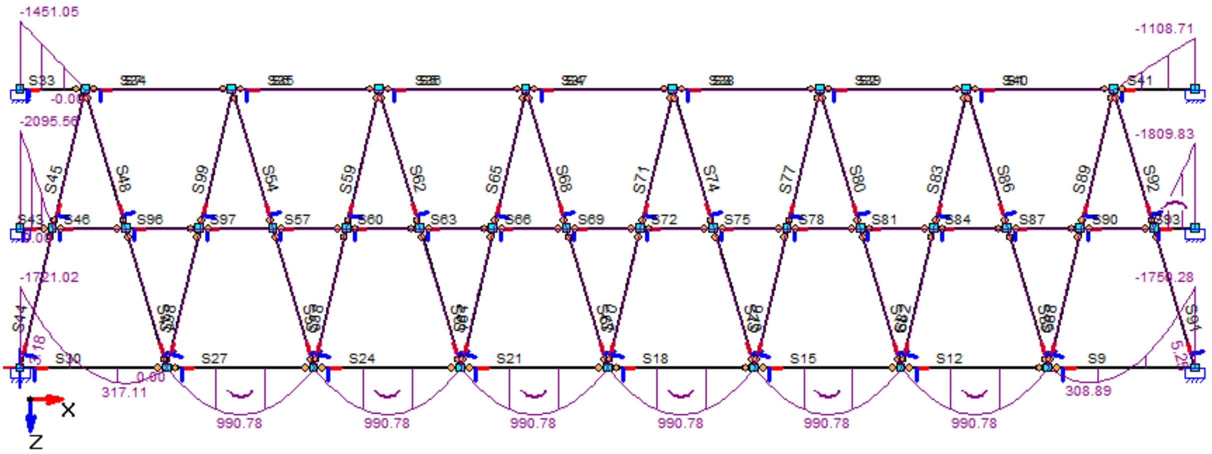
Total load G+Q		
Load_perwidth	251.1006	kN/m
Load_perdepth	607.644	kN/m
Point load	1234.578	kN
Point load/2	617.289	kN

Appendix 9 Bridge Combinations

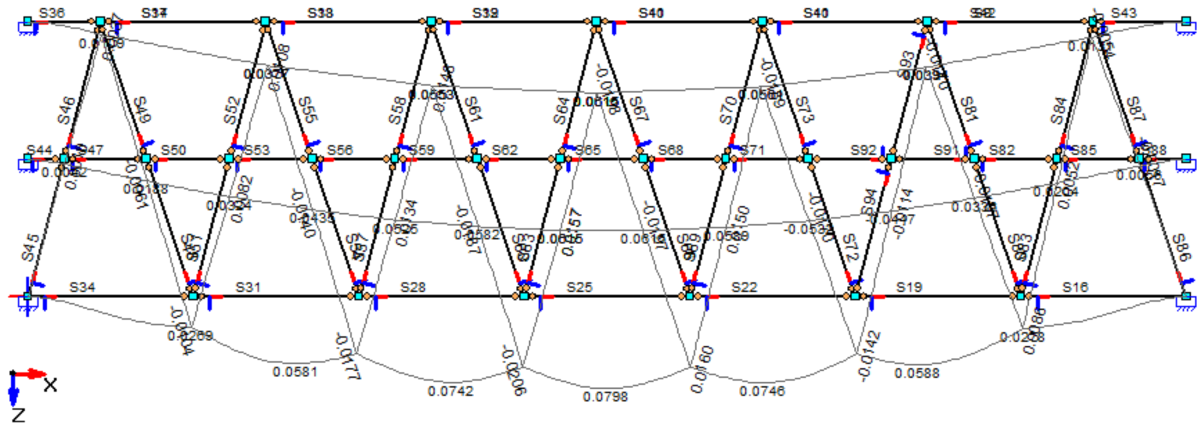
Starting combinations

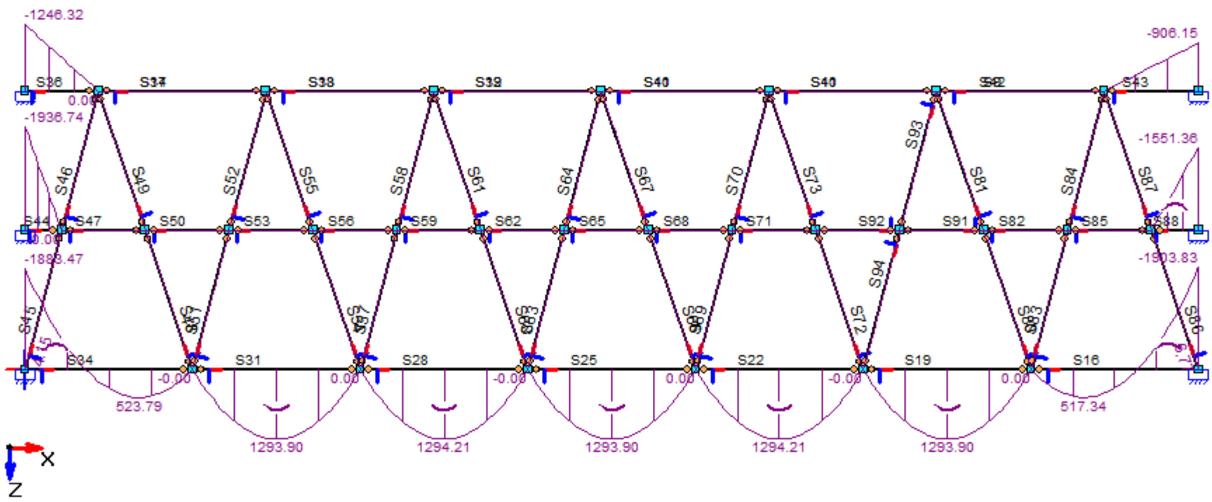
- 8 Nodes with horizontal beams of 700 mm diameter with a thickness 20mm and diagonal beams of 400mm diameter with a thickness of 15mm:



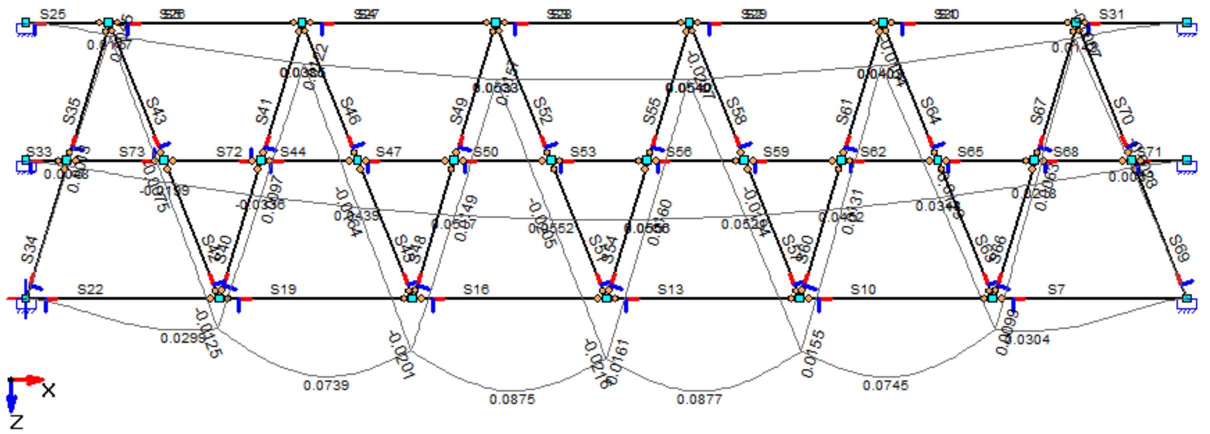


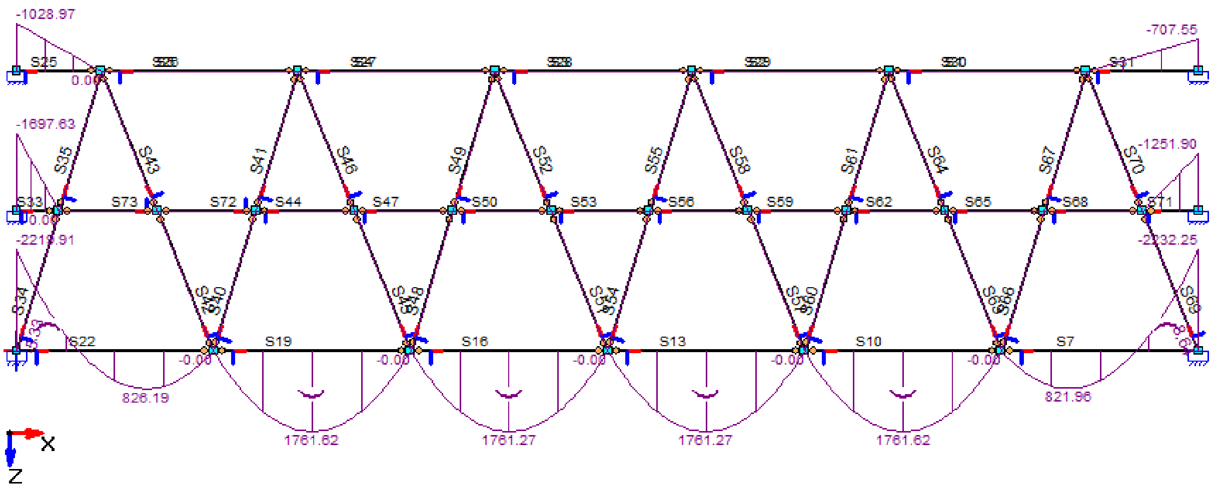
- 7 Nodes with horizontal beams of 700 mm diameter with a thickness 20mm and diagonal beams of 400mm diameter with a thickness of 15mm:





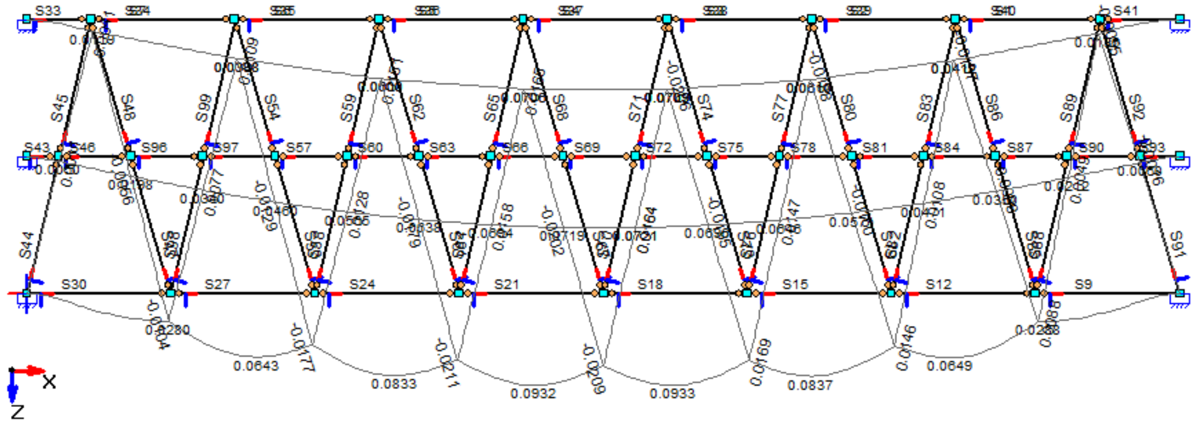
- 6 Nodes with horizontal beams of 700 mm diameter with a thickness 20mm and diagonal beams of 400mm diameter with a thickness of 15mm:

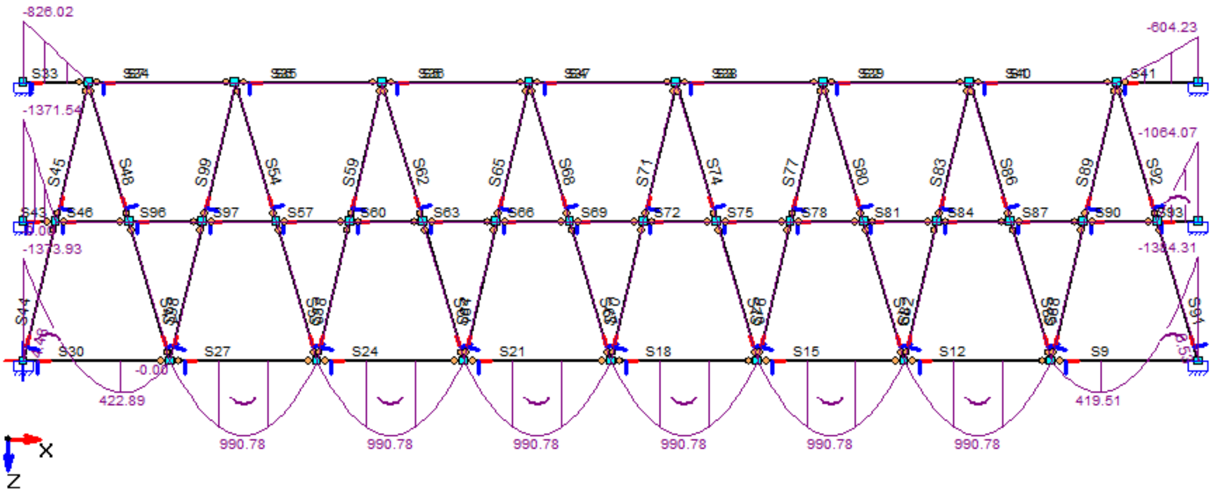




Optimal combination

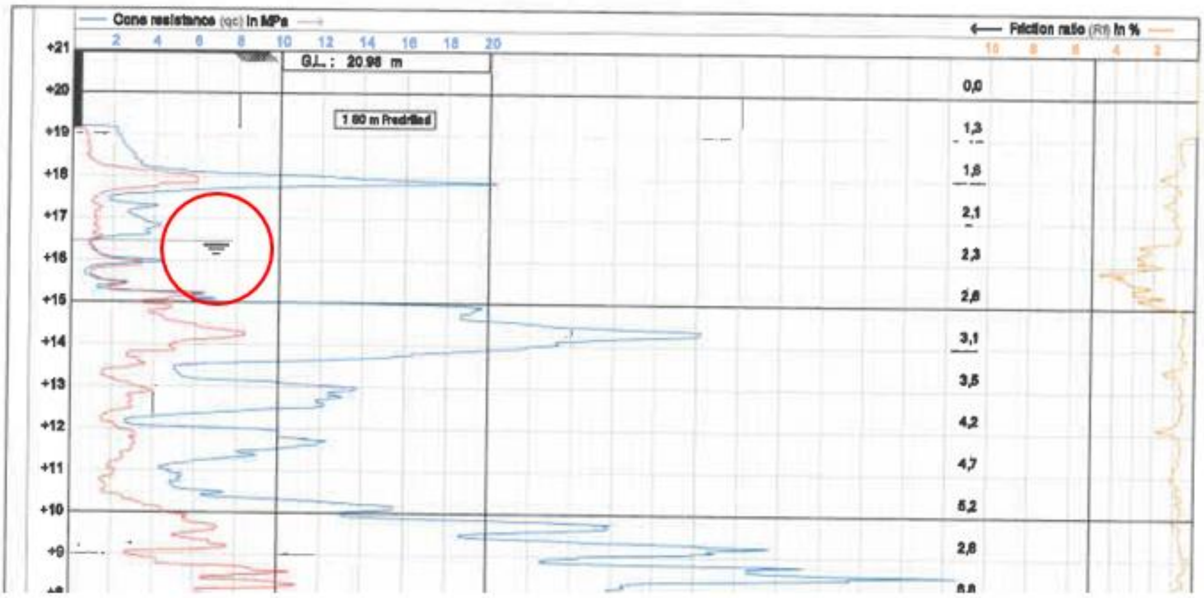
- 8 Nodes with horizontal beams of 600 mm diameter with a thickness 15mm and diagonal beams of 400mm diameter with a thickness of 15mm

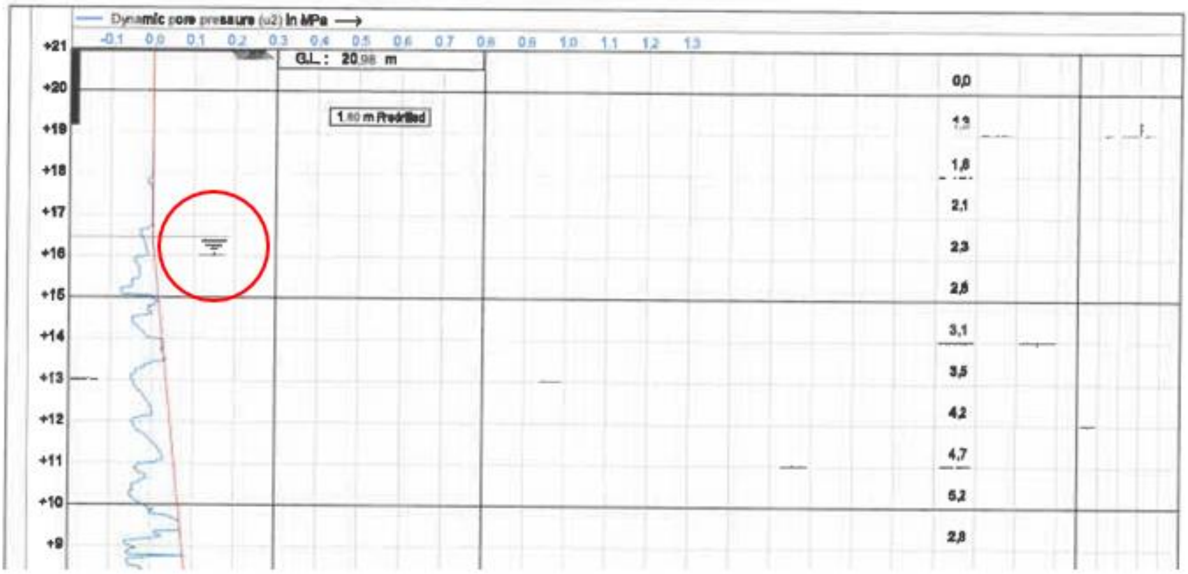


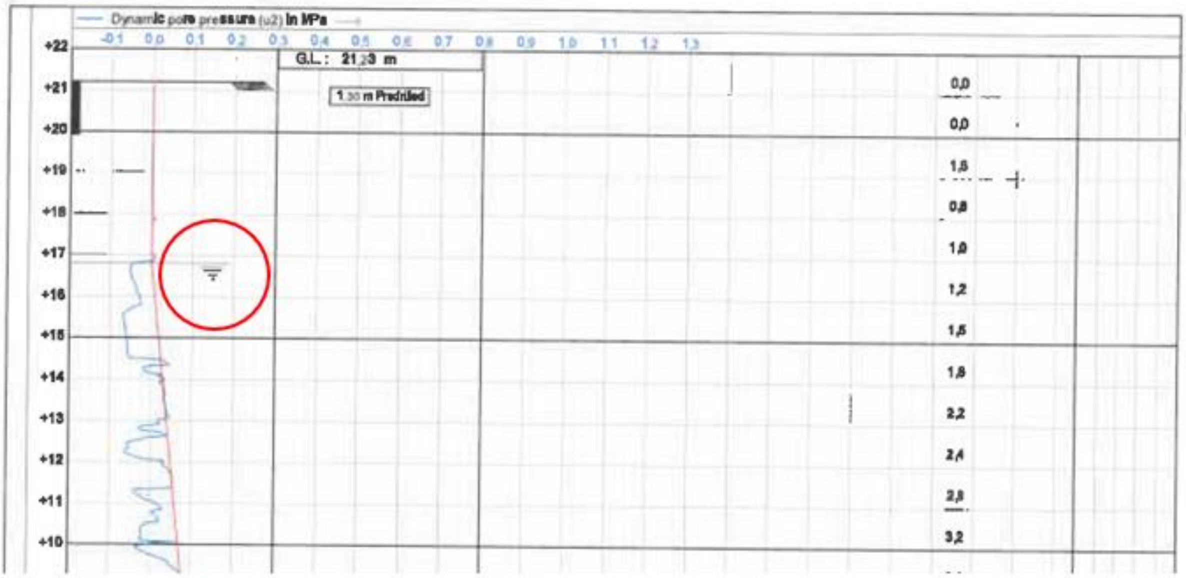


Appendix 10 Soil-drilling tests

3 soil-drilling tests from OREX, in the red circle, the GWT is visible with the red circle.






Dynamic pore pressure (u2) in MPa
G.L.: 21,23 m
1.30 m Predrilled