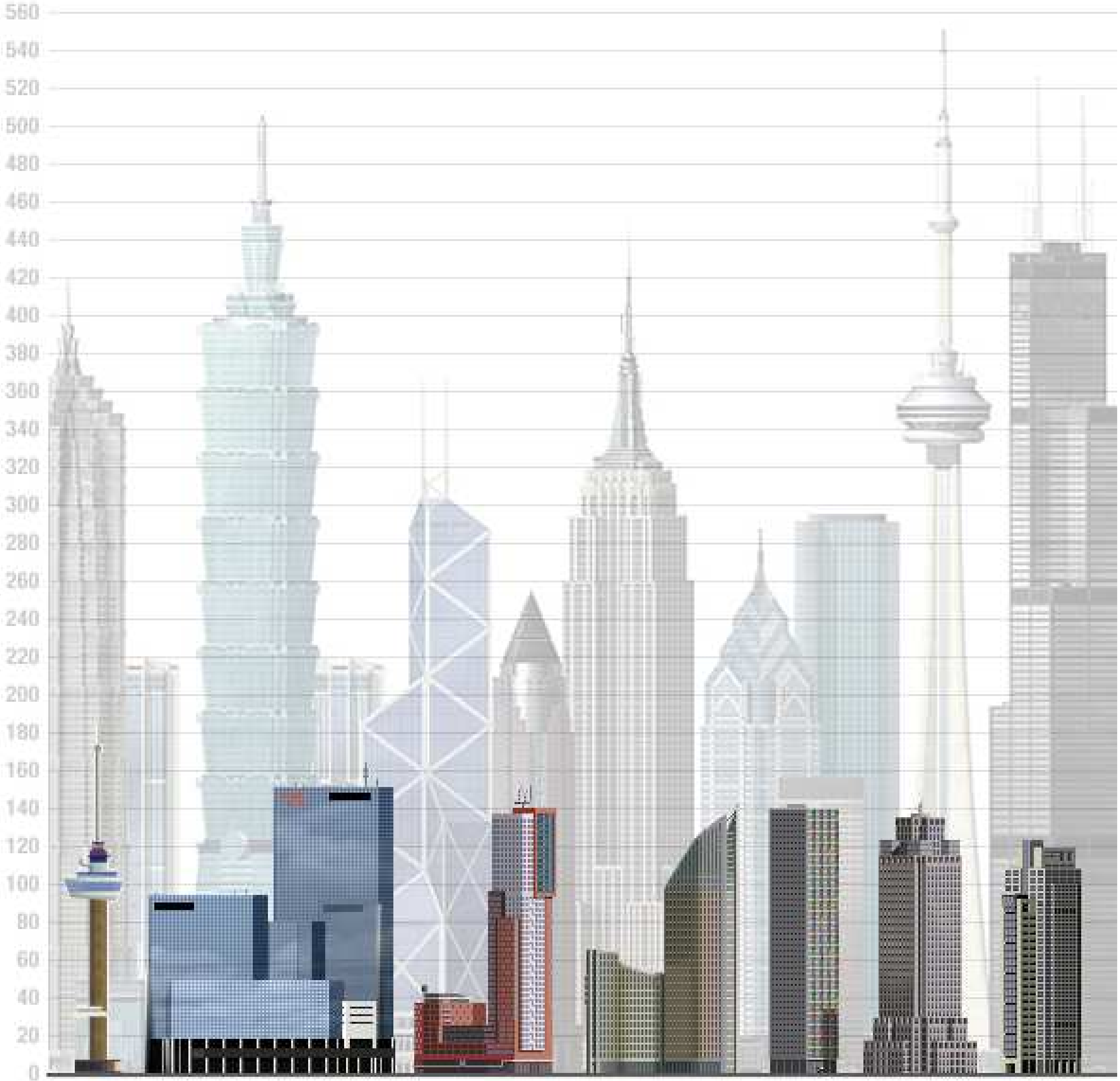
Super high-rise in Rotterdam
Part 2: Structural design



Master's Thesis Report

U.M. Winter

March 2011

Super High-Rise in Rotterdam

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Preface

This Master's Thesis is the final part of my study at the faculty of Civil Engineering at the Delft University of Engineering.
This report describes a structural concept of an 800 meter high building which can be built in the Netherlands. It consists of three parts namely:

- *Part 1: Literature study* which consists of a study on the most important aspects which come into play when designing a high-rise building.
- *Part 2: Structural design* which describes a design of the building's load-bearing structure.
- *Part 3: Appendices*
- *Part4: Addendum*

I would like to thank the members of my graduation committee who guided me through this graduation process despite their busy agendas.
Also I would like to thank ABT for providing the necessary resources and giving me the opportunity to work on my thesis in a warm environment with many knowledgeable engineers.
Finally, I would like to thank my parents, my brother and sister for their emotional and financial support.

Delft, March 2011

Uriah Winter

Abstract

<u>Design</u>

After studying the available structural systems, existing and proposed supertall projects in *part 1: literature study*, the so called compound structure was chosen as the buildings superstructure. This type of structure consists of several towers which are linked together. By doing this a lot of the aspects which become increasingly important with an increasing height are positively influenced. The openings which are created for example reduce the lateral wind load and can improve its dynamic behavior. Also, the fact that most of the buildings footprint is located at the perimeter instead of the center offers a solution for the daylight entry problem.

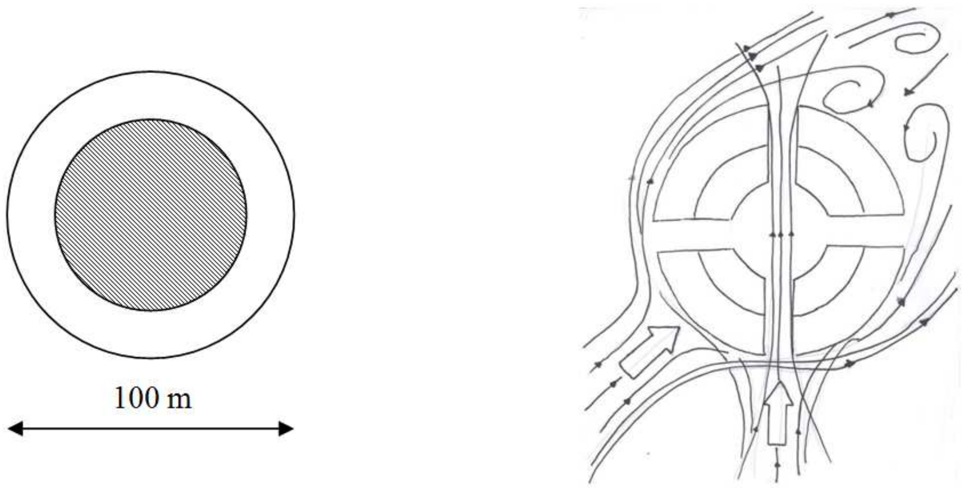


Figure 0-1 Conventional closed footprint versus open footprint.

Using tools such as net floor ratio drag coefficients and strouhal number the following footprint was created (see Figure 0-2). The shape of the floor plan is the result of trying to find a balance between reducing the wind loads and improving the dynamic behaviour as much as possible while still maintaining a good net floor surface and assuring the entry of natural daylight.

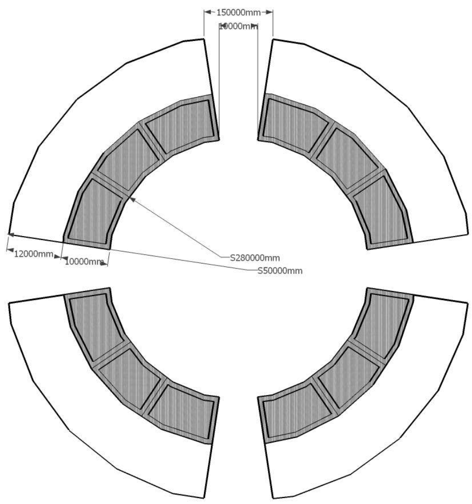


Figure 0-2 footprint

Structural engineering

One of the goals of designing a superstructure for a supertall building is creating a maximum internal lever arm by allowing the perimeter to participate in the transfer of lateral loads. Both the core-outrigger and tube alternative are suitable structural systems since the internal lever arm reaches from facade to façade. For good measure a load-bearing structure consisting only of a core was also tested in using the FEM software "ESA scia engineer".

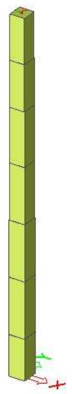
1. Core	2. Core-outrigger	3. Diagrid
		

Table 0-1 Three structural alternatives

The results showed that the building acts very stiff in the along-wind direction. However for a building with a height of 800 meter the across-wind vibrations and forces are governing. The knowledge from the literature study was used to create an aerodynamic shape which reduces the vortex shedding phenomenon. Normally a wind tunnel is used to calculate the aerodynamic response of the building. Windtunnel research however was not possible due to limited time and resources. Therefore the vortex shedding was calculated for a conventional closed cylinder after which a reduction was applied to take into account the effect gaps which allow the wind to flow through the building. These reductions were based on experiments and windtunnel research done to buildings with slots.
The reduced values of the across-wind deflection, acceleration and forces were within the limits of h/500 and ISO6897. The core-outrigger alternative was chosen out of the three alternatives in Table 0-1 for the building superstructure. This choice was based on non-structural considerations such as the erection process and the architectural obstructions.

Foundation engineering

The literature study showed that mitigation of (differential) settlements is an important aspect in the design of a supertall. Of the three foundation systems a pile-and-raft foundation showed the most promise.
Since a full design of a piled raft foundation would be very time consuming a shallow foundation was designed in order to get a grasp of the buildings settlements. The shallow foundation consist of 7 layer basement and a 3 meter thick raft which rest on the first load-bearing layer at -21 m N.A.P.

The settlements due to a uniformly distributed permanent vertical load was calculated. The total settlement are 1090 mm and the differential settlements due to the wind load is 5.6 mm. For a diameter of 140 meter this means an extra deformation of 32 mm at the top of the building. It should be noted that effect such as squeezing and the differential settlements due to non-uniformly distributed loads have not been taken into account.
It is recommended to add piles to the shallow foundation which creates a piled (raft) foundation which is able to reduce the above mentioned settlements.

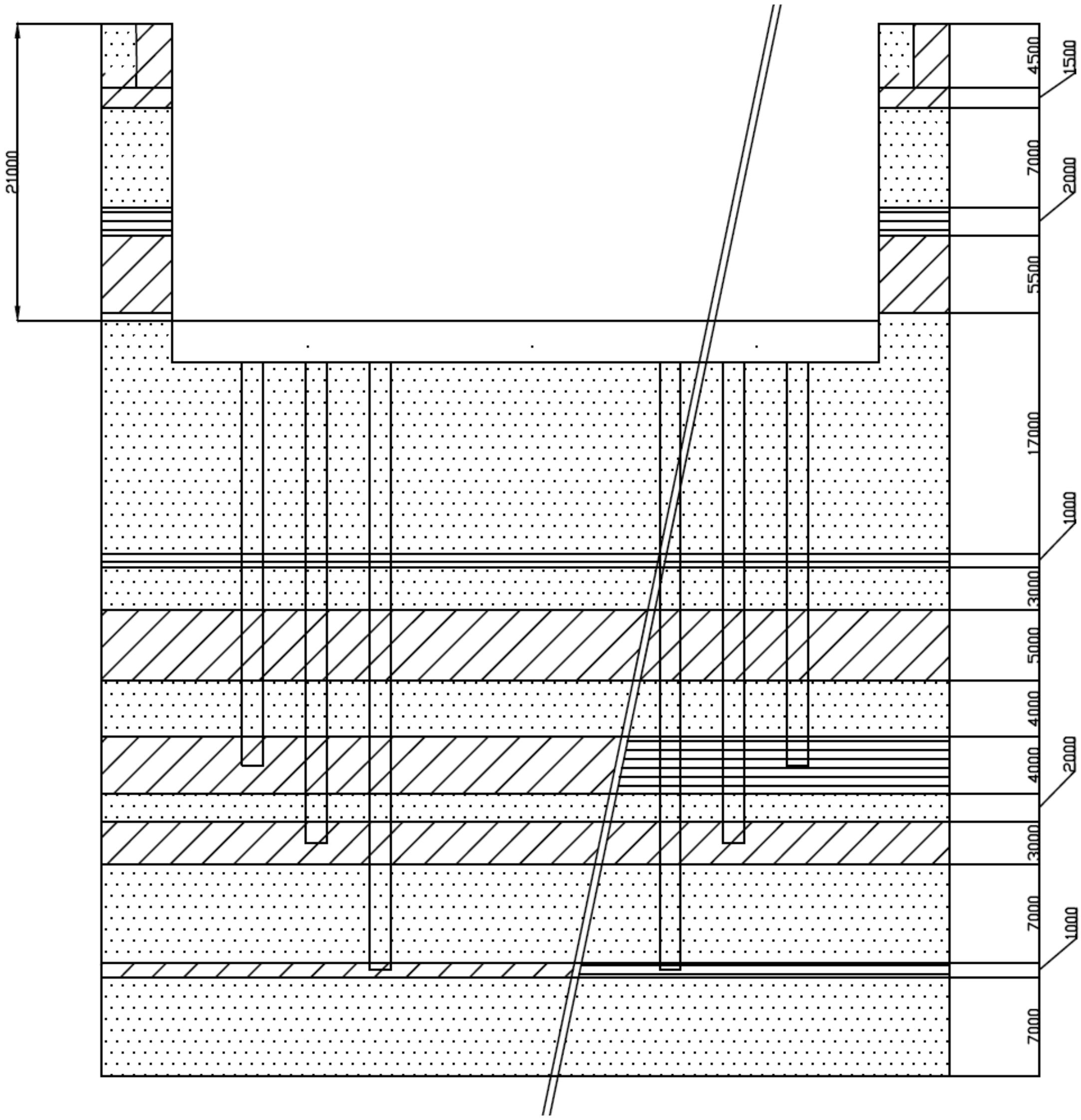


Figure 0-3 Piled raft concept scheme

<u>Conclusion</u>

The compound superstructure of the Rijnhaven Tower is performs very well structurally. The drift and acceleration of the building tower satisfy the limits. The piled raft foundation was not designed due to limited time and instead the loadbearing capacity and settlements of a shallow foundation were determined. The results showed that the settlements were quite large. Therefore it is recommended to reduce the settlements by adding piles and creating a piled raft foundation where the loads from the superstructure are transferred to both the raft and the piles. In order to make a conclusion on the structural feasibility of further research on the foundation is necessary. In chapter 6 the aspects which need to be researched are listed.

Content

Outline

In chapter 1 a brief overview is given of the structural systems which were chosen as a result of the conclusions made in chapter 4 of the literature study.

In chapter 2 the schedule of requirements of the building is given. It consists of several starting points and relevant rules and guidelines found in the building code.

Chapter 3 starts with the design of the building’s footprint. The footprint is designed using the knowledge from the literature study and is mostly based on aerodynamics. However, the economic feasibility and the influence of the footprints shape on the erection process are also examined.
After the footprint has been chosen the floor-system, structural elements and structural materials are chosen.
In paragraph 3.6 two structural alternatives for the buildings perimeters are given.

In chapter 4 the alternatives from paragraph 3.6 are tested with respect to their static and dynamic structural behavior using the finite element program “ESA SCIA engineer”.
Firstly the models which are used in ESA are described after which the along-wind and across-wind behavior is examined.

In chapter 5 the foundation is designed. Due to limited time the foundation system (piled raft) which was chosen in chapter 4 of *part 1: literature study* could not be designed and calculated. Instead the load-bearing capacity and the settlements for a shallow foundation is given in order to get a grasp of the geotechnical possibilities.

In chapter 6 conclusions are made on the structural feasibility of an 800 meter tower in the Netherlands. Also several recommendations are given on research that should be done on design aspects that have not yet have been discussed and assumptions which need to be checked.

Chapter 1 Introduction

This is part 2 of the thesis which contains the structural design of the building. Earlier in part 1 a location for the 800 meter tower was chosen and a literature study was done on supertalls. In the conclusion the most important aspects were discussed and used to choose a suitable structural system for the superstructure and foundation. Finally the so called compound structure was chosen as the load-bearing structure and a pile and raft foundation as the buildings foundation system.

A compound structure consists of several towers which are linked together. By doing this a lot of the aspects which become more and more important with an increasing height are positively influenced. The openings which are created for example have a positive impact on the wind induced behavior in the along- and across-wind direction. Also, the fact that most of the buildings footprint is located at the perimeter instead of the center offers a solution for the daylight entry problem.

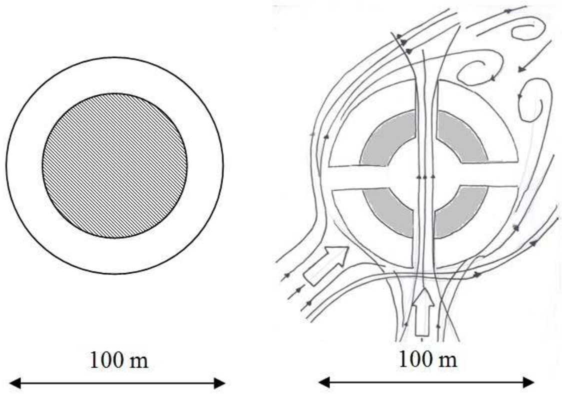


Figure 1-1 Compound structure versus conventional structure

The high concentrated loads which are expected for a tower with a height of 800 meter can require dense piling or large pile dimensions (length and diameter) for a piled foundation. Since the tower will require a basement for parking and storage it is possible to place the foundation raft on a soil layer with load-bearing capacity.
This creates a pile-and-raft foundation where the vertical loads are transferred partly via the foundation piles and partly via the foundation raft.

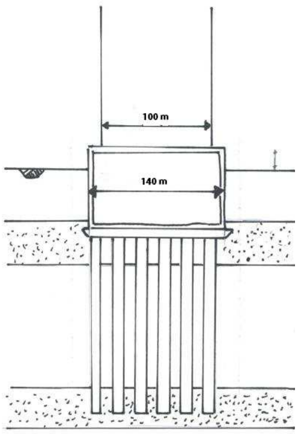


Figure 1-2 Piled raft foundation

Chapter 2 Schedule of requirements

2.1 Design philosophy

- For the foundation it is important that the difference in settlements is reduced through good design. These settlements can cause unwanted stresses in the high-rise structure and can have a negative influence on existing neighbouring structures.
- The superstructure has to be structurally and architecturally efficient. It should be designed in such a way that the vertical and horizontal loads are reduced as much as possible.
- The dynamic effects should also be taken into account.
- The economic feasibility of the structure should be considered.
- During fires or other accidents the buildings inhabitants should have enough time to flee the building.

Starting points

Height:	ca 800 meter
Gross-net floor ratio:	aim 70 %
Structural type:	Compound/linked structure
Slenderness:	1:8
Storeys:	ca 200
Location:	Rijnhaven
Function:	mixed use

2.2 Building functions

In chapter 3.4 of the literature study, it was mentioned that with the completion of a high-rise building, instantly a large amount of square meters become available on the market. In order to cope with this the building will be a mixed-use building which offers more flexibility and diversity. The Rijnhaven Tower will house the functions listed in Table 2-1.

function
Penthouses
Flexible
Residential
Offices
Hotel
Commercial
Mechanical

Table 2-1 Building functions

Mixed-use high-rise buildings however are more complex because each function has different demands. Each function has a different optimal span, column heart-to-heart distance and storey height.
The different functions can be stacked or linked.

Stacked program - The functions are stacked on top of each other.
Linked program - The individual tower each have their own function i.e. Residential tower, Office tower etc.

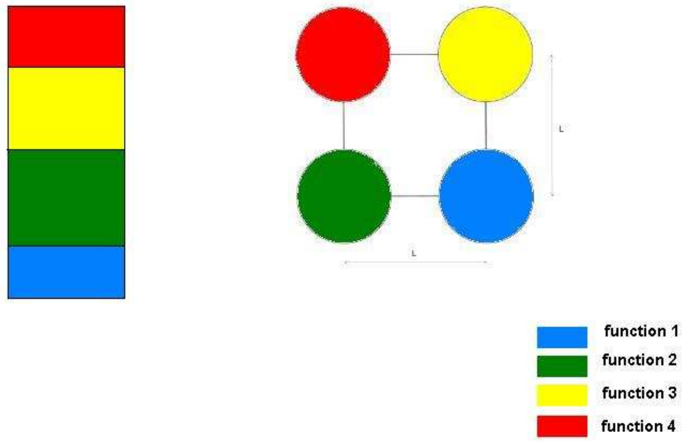


Figure 2-1 Stacked vs. linked functions

An important engineering aspect of a supertall is limiting the differential settlements. If the individual towers each have their own function the chance of unequal settlements will be larger because each tower would be subjected to different live loads. Since this results in tilting we chose a stacked program where the different functions are vertically stacked on top of each other.

2.2.1 Vertical allocation of the building functions

There are two ways to determine the vertical allocation of the building functions.
One is from the tenant's preference i.e.: How long are people willing to travel? How often do they leave the building etc.?

"Building types have characteristic traffic profiles. For example, office buildings typically have up-peak traffic in the morning when employees enter the building, intense two-way or inter-floor traffic during the lunch time, and down-peak traffic when employees exit the building. Below grade should be used for parking, the first level above grade should be commercial use, the next level for office space, the next for hotel and topmost level for residential function." [6]

The other is from a structural point of view where the columns centre to centre distance is taken into account.

"However, from the structural point of view, the smallest column space, which is hotel or residential function, always should be placed at the bottom of the building for structural efficiency to avoid special consideration in transferring loads." [6]

For Rijnhaven Tower we will use the most common configuration which is based on tenants' preference. From top to bottom we find the following functions:

Penthouses →Residential →Flexible →Offices →Hotel →Commercial →Basement

2.3 Building code

See appendix B.

2.4 Vertical transportation

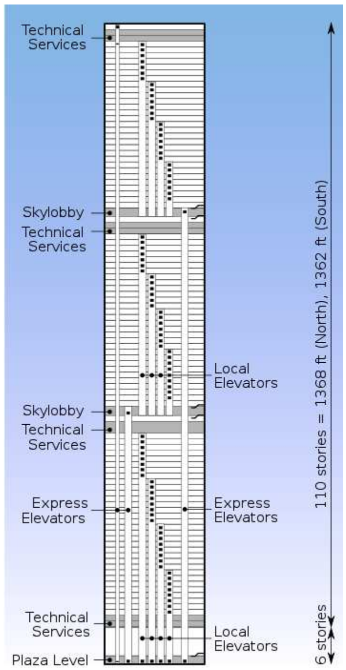


Figure 2-2 stacked zones

As mentioned in chapter 3.6 of the literature study a 200-storey tower will most likely have multiple sky lobbies that are to be serviced by double-deck shuttle lifts. Utilizing skylobbies and their shuttles, multiple local zones of lifts may be stacked on top of one another, significantly reducing the number of lift shafts that penetrate the building's lower floors.

In order to save space in the buildings footprint we will use **stacked zones** and double-decker lifts. These zones end and start at the mechanical levels making them ca 25-30 storey high.
Double deck lifts comprise two passenger cars one above the other connected to one suspension system. The upper and lower decks can thus serve two adjacent floors simultaneously. During peak periods the decks are arranged to serve even and odd floors respectively with passengers guided into the appropriate deck for their destination.

A complete design of the vertical transport is very complex and beyond the scope of this thesis. Nevertheless we need an indication of the amount of area which will be lost to vertical transport to design the buildings footprint. In this paragraph we will use several reference projects and assumptions to estimate the area necessary for vertical transportation.

Table 2-2 shows the number of elevators used in several supertalls. The large difference between the Nakheel Tower and Burj Khalifa can be attributed to the fact that the formers superstructure consist of several cores whereas Burj khalifa and Taipei 101 only have one core. Since our tower has a similar compound structure we will assume our tower needs about the same number of elevators for vertical transportation.
The Nakheel Tower however with its height of 1000 meter has a large spire (larger than the Eiffel Tower) meaning that a large part of the tower is unfit for habitation [27][28] the lifts only reach to about 800 meters and the footprint of the tower decreases significantly at the beginning of the spire.
Since our tower has the same footprint along the height of the building we will assume that the tower will have 180 lifts. The vertical transportation system consists of express lifts and normal elevators. The express lifts take the passengers to the mechanical floors where they have the possibility to transfer to a normal lift which brings them to their floor or final destination. This way several elevator hoist ways can be stacked on top of each other and space can be saved

Building	Height (m)	Number of elevators
Nakheel Tower	1000	156
Burj Khalifa	828	57
Taipei101	500	67

Table 2-2 Height and number of elevators for supertalls

Assumptions

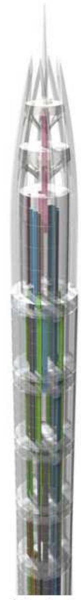


Figure 7. Diagram of the planning and stacking of the 156 tower lifts

According to [11] a single person takes up 2 m^2 or 1.3 m^2 in a crowded elevator and a double decker lift requires 150 square feet or 13.94 m^2 (150 *0.0929).

The function of the express lifts is transferring people to their own 25-30 storey high neighbourhoods after which they can transfer to a normal lift. We will assume that the express elevators are larger than the normal elevators and we reserve 20 m^2 for the express elevator and 14m^2 for the normal elevators.
The ratio express elevator to normal elevators is 1:5 meaning there are 30 express lifts and 150 normal lifts.
The normal lifts can be stacked on top of each other and since there are 7 subdivisions we have about 22 normal elevators and 30 express elevators per footprint

If we include 300 m^2 for free space and traffic and another 200 m^2 for other functions which do not need daylight, the total space taken up by vertical transportation is:

$30*20+22*14+300+200= 1408\ m^2$

Therefore ca. 1400 m^2 will be reserved for vertical and horizontal transportation traffic.

2.4.1 Fire safety and evacuation

Fire safety is an important issue in supertalls. Because of their height high-rise buildings are challenging for emergency planning.

The Dutch "eurocode" and "bouwbesluit" are not equipped to deal with fire safety for supertalls. They state that if a floor with a usage function is located more than 13 m above ground level the fire resistance corresponding to the failure of the load-bearing structure should be at least 120 minutes. This means that the requirements for a 30 meter building are the same as those for a building larger than 300 m. This is of course not the case as it is much harder to evacuate a taller building due to its height and the number of people which will try to flee the building at once.

Extra measures to protect the building structural system are:

- CHS and RHS sections filled with concrete.
- Beams protected by concrete or paint.

An advantage of a compound structure compared to conventional superstructures is that there can be several ways of exiting the building due its linked design. In case of an emergency such as fire, inhabitants can travel down or up to skybridge and transfer to unaffected tower which allows them to safely evacuate the building.
[4] Offers a more in depth view of the problems which are encountered when trying to evacuate a supertall building and gives several solutions, approaches and evacuation methods.

Chapter 3 Structural Design

3.1 Introduction

The first step will be designing the footprint of the building. A slenderness of 1:8 means that the 800 meter high building will be 100 meters wide.
The shape of the floor plan should be the result of trying to find a balance between reducing the wind loads and improving the dynamic behaviour as much as possible while still maintaining a good Nett floor area and assuring the entry of natural daylight.

Daylight

Not all the functions in a building need to have a direct source of daylight. Therefore other facilities such as corridors, meeting rooms, kitchenettes, archives and photocopying rooms can be positioned further from the perimeter of the building.
If we take an estimated floor strip of 3.6 meters for these functions and 7.2 – 9 meters for the main function of the footprint such as office or residential we get a leasable depth of 10.8-12.6 meter. This is similar to the values given in (Table 3-3).

Nett floor area

The "Nett-gross Floor Ratio", which is the ratio "Gross Floor Area" to "Lettable Floor Area", is an important factor which is used to assess the economic feasibility of a building. It expresses how much floor area is lost to functions from which no revenue can be generated. The demand for these functions increases with the height of the building and thus the space taken up by elevators stairwell building service can take up a significant amount of the footprint in supertall buildings.

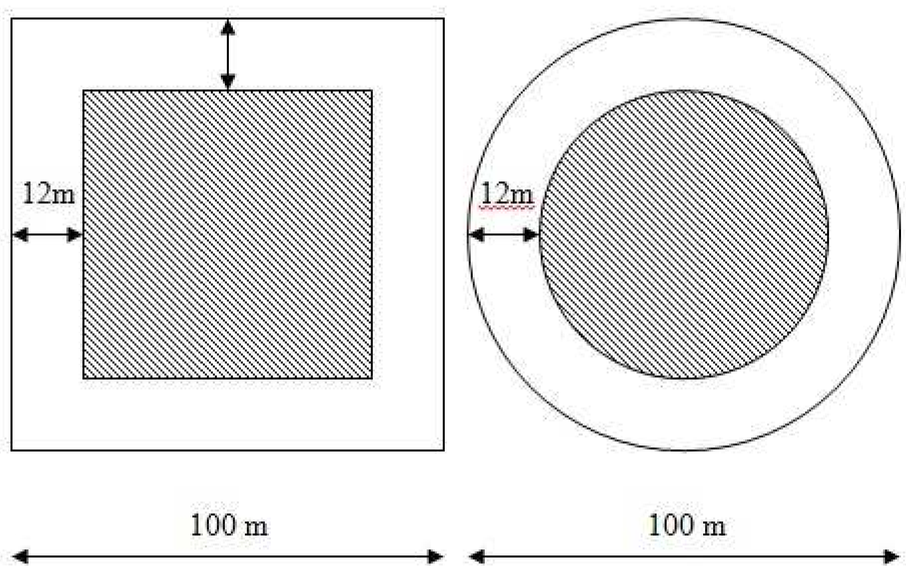


Figure 3-1 Economically unfeasible footprint.

Figure 3-1 shows a square and circular footprint with a leasable depth of 12 meter and a width of 100 meters. The Nett-gross floor ratio for the rectangle and circular cross section are 42 and 44% respectively which is very bad from an economical point of view.

Table 3-2 shows the GFA and NFA for several supertall buildings in the world. It can be seen that the average space efficiency for these 10 buildings is 68.5 %.
To keep a high-rise project feasible real estate agents aim for a nett floor area of 70 to 80 %.

3.2 Aerodynamic design

As the height of a building increases the wind starts to play a larger a role in the design and economical feasibility of the structure. The wind load en the motions of the building are susceptible to dynamic amplification in both the along-wind and across-wind direction and at a height of 800 meter wind from all directions has to be taken into account.

[49] States that:

The resultant of the aerodynamic forces experienced by a structure subjected to wind action can be resolved into a drag (along wind) force and a lift (across wind) force acting perpendicularly to that direction. Very often the design is governed by the serviceability response (peak acceleration and deflection at top floors). Crosswind vibrations are usually greater than along wind vibration for buildings with a height greater than 100 meter.
Also generally the total force, elastic and the centre of mass do not coincide resulting in torsional moments. This happens even for symmetric shapes immersed in a symmetric mean flow since the instantaneous flow will in general be asymmetric due to the randomness of flow fluctuations. These aerodynamic forces are greatly influenced by the building shape.

The wind load on tall buildings is always determined with the help of a wind tunnel. However due to a limited availability of time and resources the use of a wind tunnel is not possible for this thesis. Therefore tools such as drag coefficients and strouhal number are used to compare the behaviour of basic shapes.

Name of building	City	Year of completion	Height (m)	Number of floors
Taipei Tower	Taipei	2004	509	101
Shanghai World Financial Center	Shanghai	2008	492	101
Petronas Towers 1-2	Kuala lumpur	1998	452	88
Sears Tower	Chicago	1974	442	110
Jin Mao Tower	Shanghai	1998	421	88
Two International Finance Center	Hong Kong	2003	415	88
Citic Plaza	Guangzhou	1997	391	80
Shun Hing Square	Shenzhen	1996	384	69
Central Plaza	Hong Kong	1992	374	78
Bank of China	Hong Kong	1990	367	70

Table 3-1 General information [6]

Name of building	GFA (m^2)	NFA (m^2)	Space Efficiency (%)	Interior Columns	
				Single	Multiple
Taipei Tower	2650	1920	72	No	
Shanghai World Financial Center	2500	1750	70	No	
Petronas Towers 1-2	2150	1290	60	No	
Sears Tower	4900	3780	77	Yes	
Jin Mao Tower	2800	1940	69	No	
Two International Finance Center	2800	1904	68		Yes
Citic Plaza	2230	1500	67	No	
Shun Hing Square	2160	1450	67	No	
Central Plaza	2210	1460	66	Yes	
Bank of China	2704	1865	69	No	
Average			**68,5**	**------**	

Table 3-2 Space efficiency [6]

Name of building	Leasing depth (m)	Floor-to-floor height (m)	Floor-to-ceiling height (m)	Structural Floor material
Taipei Tower	13,9-9,8	4,20	2,80	Composite
Shanghai World Financial Center	12,5	4,20	2,75	Composite
Petronas Towers 1-2	13,0-8,3	4,00	2,65	Composite
Sears Tower	22,9	3,92	2,70	Composite
Jin Mao Tower	14,8-11,8	4,00	2,79	Composite
Two International Finance Center	14,5	4,00	2,70	Composite
Citic Plaza	11,3	3,90	2,70	Composite
Shun Hing Square	12,5-12,0	3,75	2,65	Composite
Central Plaza	13,5-9,4	3,90	2,60	Reinforced Concrete
Bank of China	17,6	4,0	2,80	Composite
Average	**12,1**	**3,98**	**2,70**	**------**

Table 3-3 Leasable depth and storey height [6]

3.2.1 Drag coefficient (along-wind behaviour)

Please note the aim of this chapter is not to make quantitative comparisons of possible footprints but to give insight into the behaviour of different shapes.

Along-wind is the term used to refer to drag forces where a structure experiences an aerodynamic force which has the same direction the wind .The structural response induced by the wind drag is referred to as the along-wind response. The along-wind motion is the result of pressure fluctuations on windward and leeward face.

The drag coefficient C_d, is a dimensionless quantity which is used to quantify the drag or resistance of an object in a fluid environment such as air or water. It is used in the drag equation, where a lower drag coefficient indicates the object will have less aerodynamic or hydrodynamic drag. The drag coefficient is always associated with a particular surface area.

$$C_d = \frac{F_d}{0,5 \cdot \rho \cdot U^2 \cdot B} \tag{1}$$

At heights of 800 meter we should expect wind from every direction. Therefore we will take a look at the most unfavourable wind loading for each shape. We will consider 3 basic shapes for the cross section of our compound structure (figure 3-2).

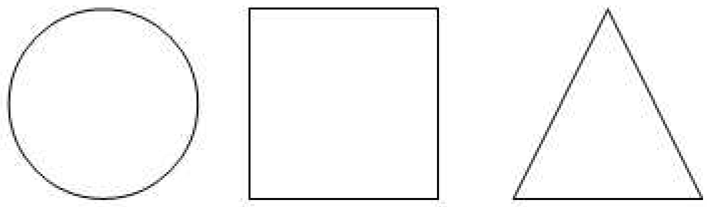


Figure 3-2 Basic shapes

These shapes all have different drag coefficients which are given in figure 3-3.

In figure 3-3 it can be seen that the circular shape has the best along wind behaviour because it allows the wind to flow across its body. Also it does not have an unfavourable side due to its round shape. Both the rectangle and the triangle have a higher drag coefficient in the most unfavourable situation where the wind load acts on one of the sides.
The most favourable situation for the triangle and square shape is wind acting on one of the corners.

It should be noted that studies have shown that rounded corners can help reduce the wind loads. This can be clearly observed in Figure 3-4, which indicates the variation of drag coefficients for gradually increasing radius of curvature of building corners as we go from an almost square to a fully circular shape. For the latter, it is interesting to note the variation of C_d with the surface roughness, which affects the location of separation and, consequently, the pressure loads on the surface.

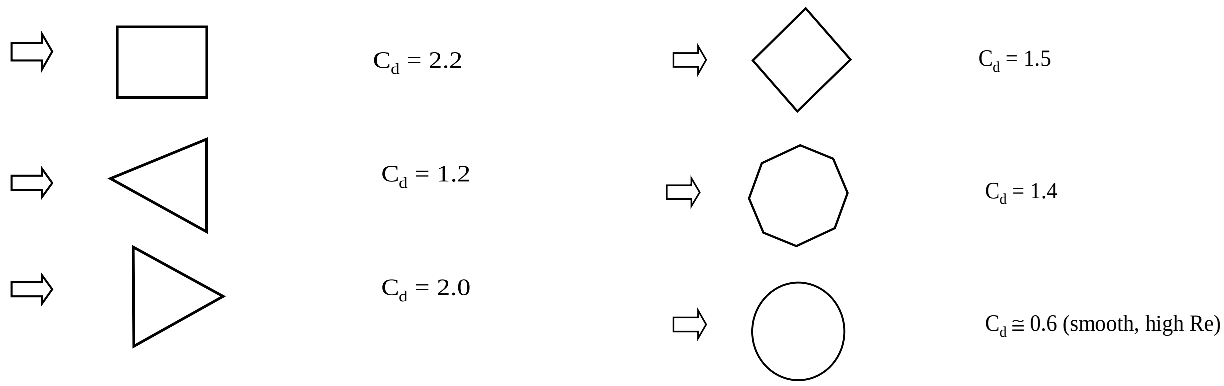


Figure 3-3 Drag coefficients

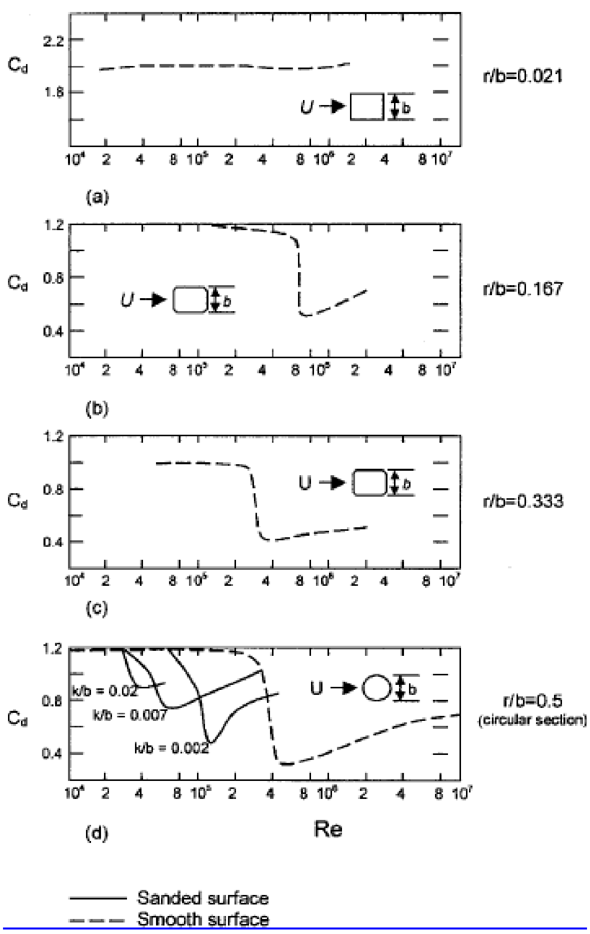


Figure 3-4 Surface roughness

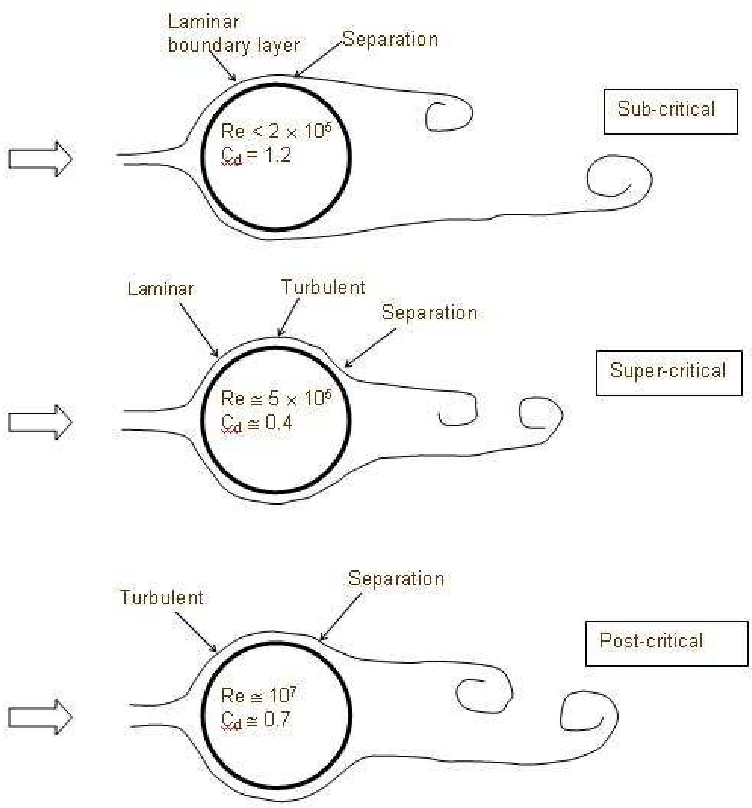


Figure 3-5 Effect Reynolds Number on drag of a circular shape

The drag of circular section is dependent on the **Reynolds number** (figure 3-7) and the **surface roughness**, the pressure distribution also changes at the sub-critical and super-critical state (figure 3-5).

Air since it has mass evidences inertial effects according to Newtons second law (or more specifically the Navier-strokes equations). The two most influential effects in an air flow are then viscous and inertial and the relation of these to each other becomes an index of the type of flow characteristics or phenomena that may be expected to occur.
The non-dimensional parameter Re (the Reynolds number) is a measure of the ratio of inertial to viscous forces.

For square cylinders or buildings with sharp corners on their outline, *Cd* is almost independent of *Re.*

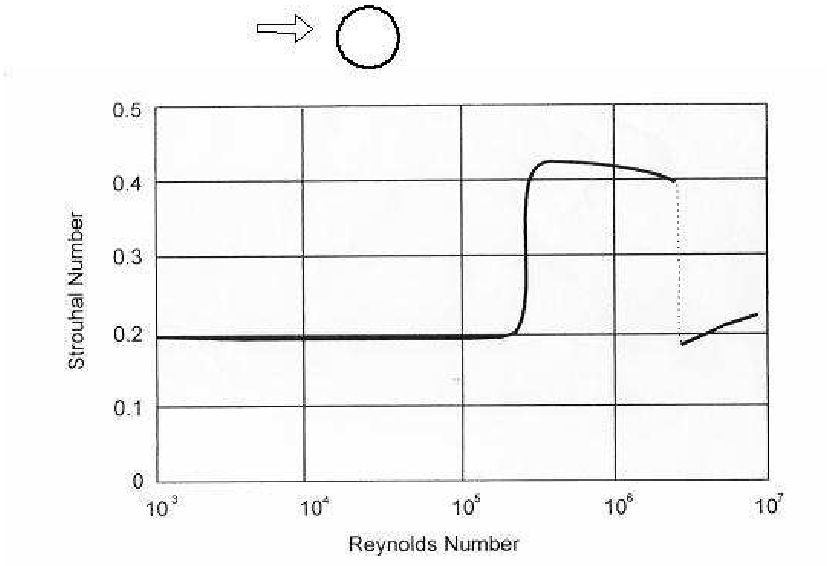


Figure 3-6 Reynolds number versus strouhal number

$$\mathrm{Re} = \frac{v \cdot b}{\nu} \qquad (2)$$

b = the diameter in m
ν= 15* 10^{-6} is the kinematic viscosity of the air in m^2/s
$v_{(Ze)}$= the peak wind velocity

3.2.2 Vortex shedding (across-wind behaviour)

In a supertall the across-wind behavior is very important and often governing in the design of the building,

"*while the maximum lateral wind loading and deflection are usually observed in the along wind direction, the maximum acceleration of a building loading to possible human perception of motion or even discomfort may occur in across wind direction*". ([9] Taranath).

The Strouhal number is a dimensionless value useful for analysing oscillating unsteady fluid flow dynamics problems. It relates the frequency of shedding of vortex around an object as a fluid, like air passes around the object.

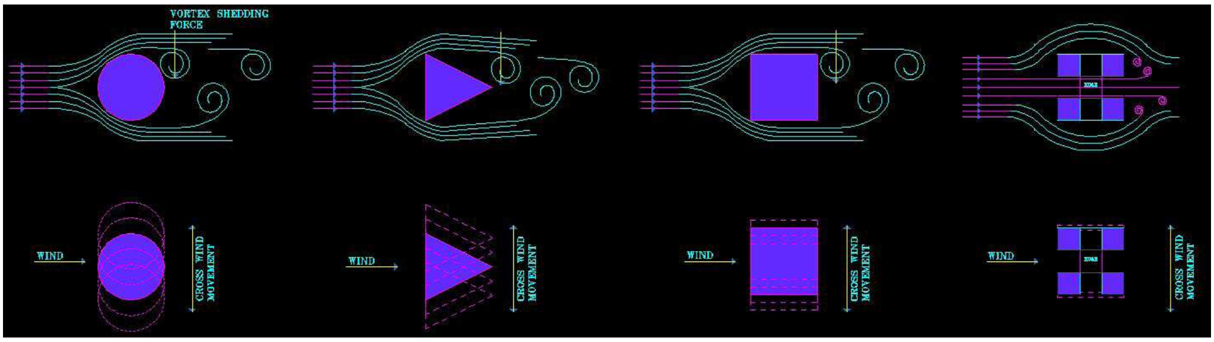


Figure 3-7 Behaviour of different shapes with respect to vortex shedding

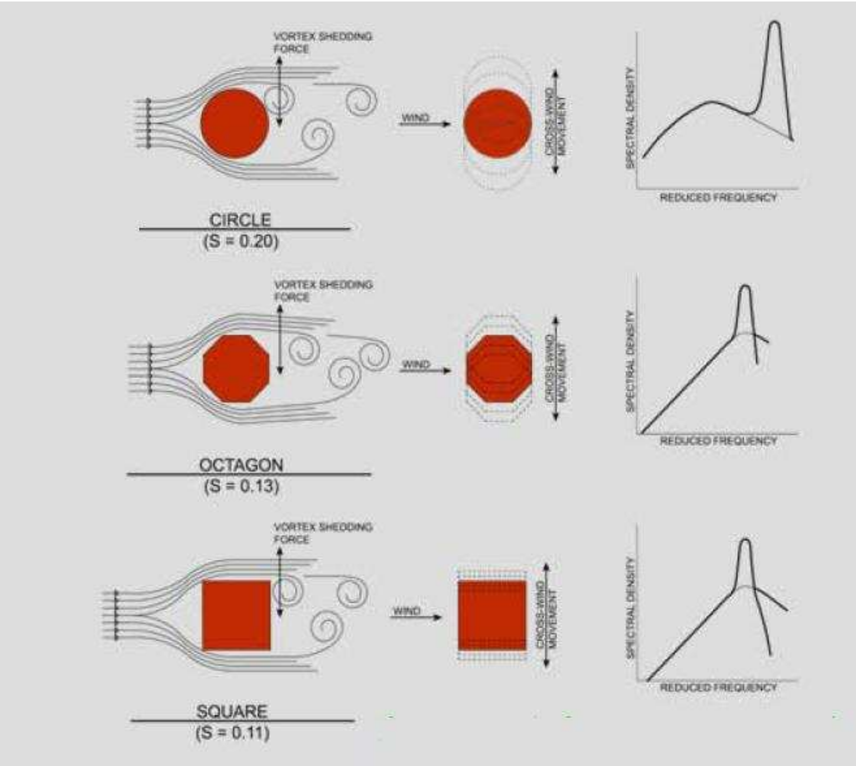


Figure 3-8 Vortex shedding and crosswind movement

From Figure 3-8 it can be seen that a circle has a larger crosswind-amplitude. The Strouhal number of a triangle lies between the value of circular and rectangular section.
Note that the end effect (i.e. 3-D effect) is extremely important in reducing response due to vortex shedding, as is turbulence.

3.2.3 Voids and slots

Now that we have taken a look at the behaviour of basic shapes subjected to wind loads we will examine the addition of voids and slots which allow the wind to flow through the building.

In [34][52][57] and [58] it is shown that slots and voids have a positive effect on the along and across wind behaviour. By allowing the air to bleed through the building via openings or porous sections the formation of the vortices becomes weakened and disrupted by the flow of air through the structure.

A tower consisting of two elements has several disadvantages. When **two towers** are connected the internal lever arm is only increased in one direction so the structure will be weak to lateral loads in the direction perpendicular to the load. If the tower internal lever arm of the tower is increased the drag coefficient on one of the closed sides will be high.

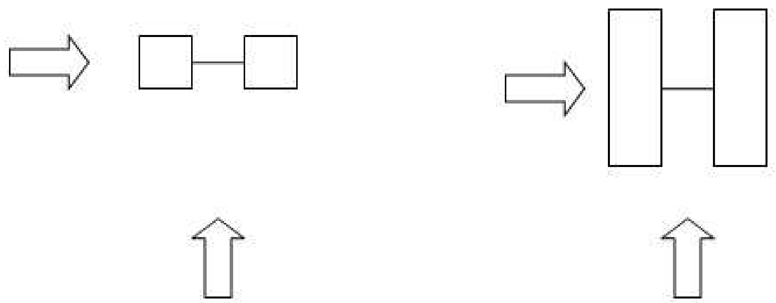


Figure 3-9 Two linked towers

A compound structure composed of **3 individual towers** does not allow the wind to flow through without flowing around one of the towers which can cause high stress on one segment of the compound structure.

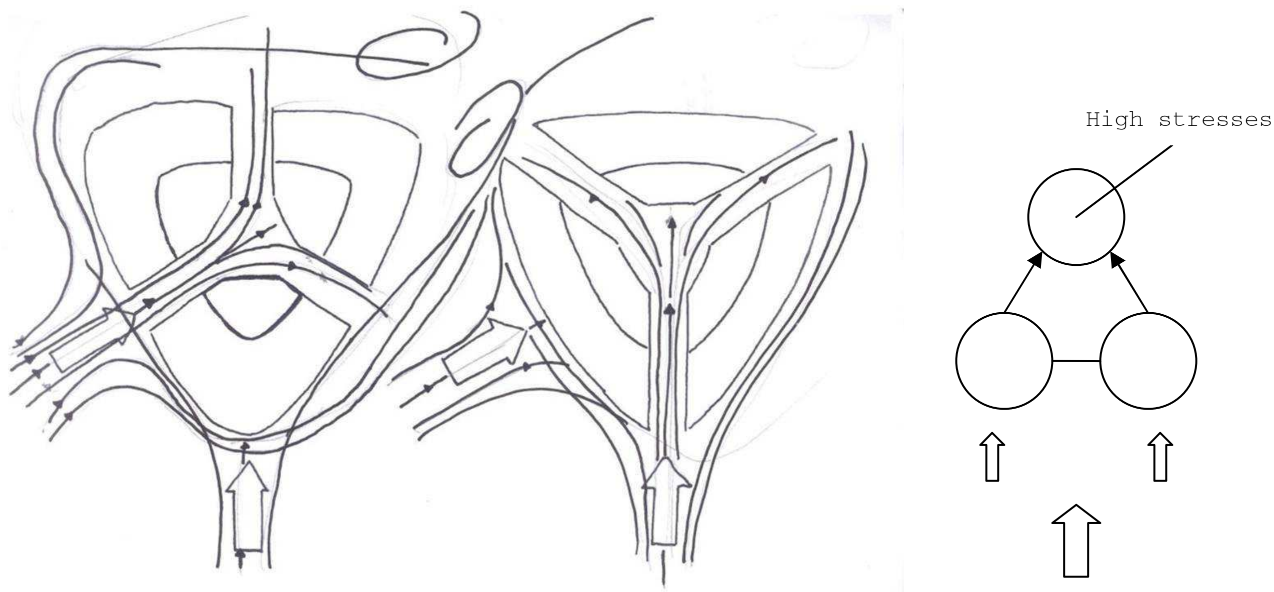


Figure 3-10 Three linked towers

For the static behaviour the most unfavourable situation would be wind loading on the two segments or components which can cause high stresses in one single tower.

A configuration with **4 slender towers** always distributes the load from one tower to one or two tower(s) and allows the wind to flow through the structure without hindrance.

Therefore a configuration consisting of 4 interconnected towers (see Figure 3-11) is chosen.

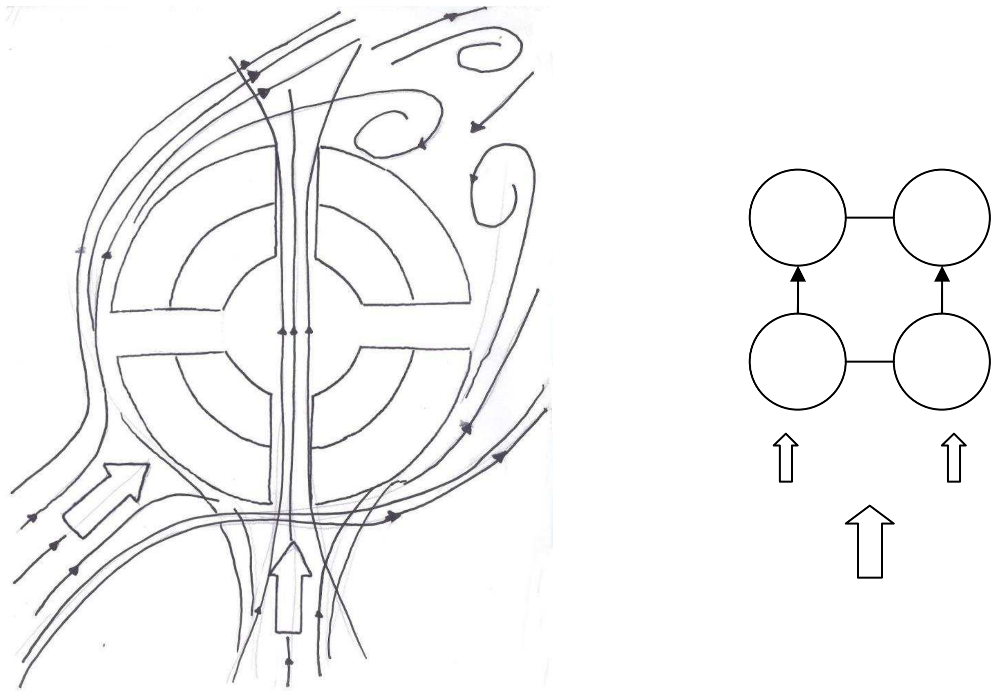


Figure 3-11 Four linked towers

3.3 Footprint

3.3.1 Net floor area

Besides the aerodynamic aspect of the footprints shape there are other important considerations which will be examined in this paragraph. As mentioned earlier the Nett-gross floor ratio is usually used to determine the economic feasibility of a supertall.

Other considerations are:

- The constructability
- Area available for vertical transportation
- Structural considerations
- Architectural considerations

In Figure 3-12 and Figure 3-13 we see 2 alternatives for a compound structure, namely:

- A basic shape with voids and slots (Figure 3-12)
- A basic shape applied on segments or individual towers (Figure 3-13)

The first option looks more like one tower with holes punched in it. The second option however is built up out of 4 separate towers who are linked together. In both configurations however each quadrant has its own core.
We will take a look at both alternatives and judge them using the abovementioned considerations. The following fixed variables se used for both alternatives.

Slot width: ca. 15 meter
Daylight entry depth: 12 meter

The daylight entry depth has been determined in paragraph 3.1. The minimal slot width is ca.1/6 D which is used in experiments and reference projects [27] [34] [57].
We get the following results for the different footprints (Table 3-4).

	Core area (m^2)	**Leasable area (m^2)**	**Total area (m^2)**	**Ratio**
Alternative 1	1628	2663	4291	62%
Alternative 2	1075	3450	5675	61%
Alternative 1 (omitted walls)	1628	3690	4291	86%
Alternative 2 (with ¼)	1075	4600	5675	81%

Table 3-4 GFA NFA alternatives

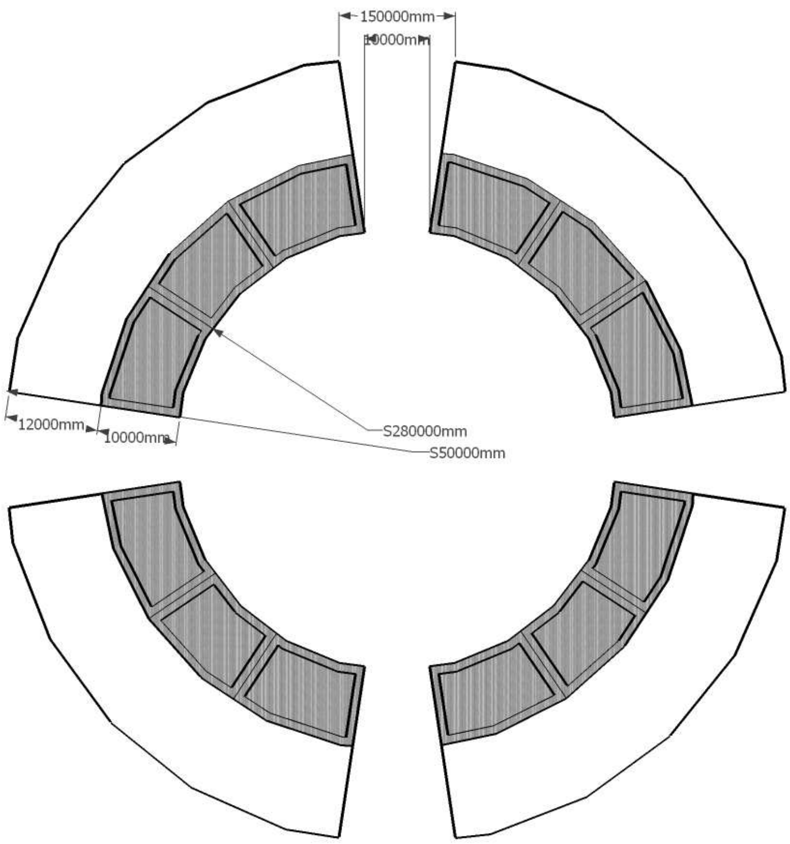


Figure 3-12 Alternative 1

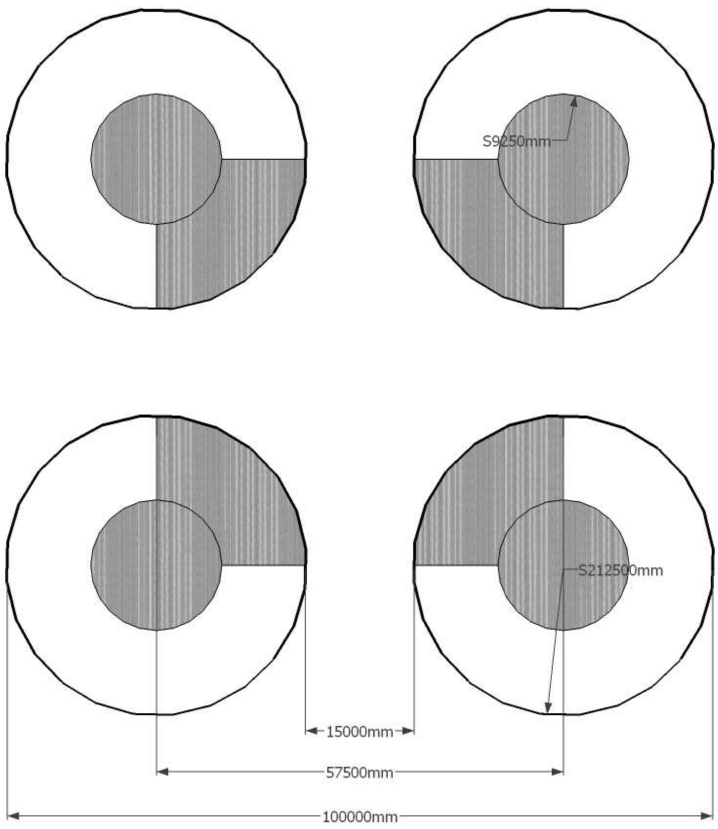


Figure 3-13 Alternative 2

Leasable area (economic feasibility)

Both alternatives have a similar Nett-gross floor ratio. Alternative 2 however has a larger leasable area which means more profit for the client. However a larger leasable area and total area also means more pressure on the foundation which, considering the subsoil, is another important part of the projects feasibility.
For alternative 1 a better Nett-gross floor ratio can be achieved by omitting certain walls as the building gets higher. If walls 1, 2, 6 and 8 are omitted the Nett-gross floor ratio increases to 86 %.

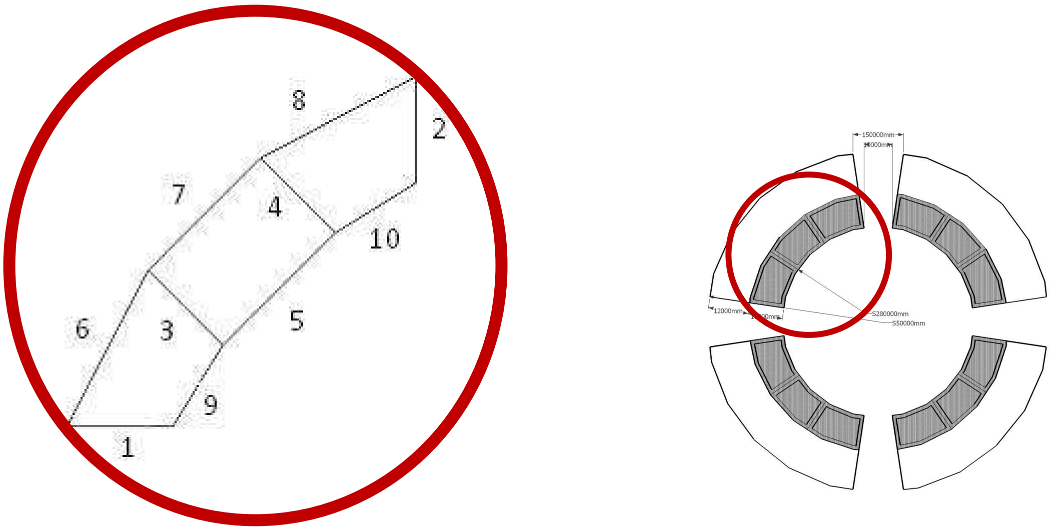


Figure 3-14 Core walls

For alternative 2 the use of a quarter of each individual tower in Figure 3-13 is debatable. Even if there is sufficient daylight the space has no or poor view on the environment surrounding the building. However even though the areas located at the inside have less value daylight can still reach these parts which makes them usable. If these areas are counted as leasable areas then the Nett-gross floor ratio increases from 62% to 81%. (see Table 3-4).

Core area (vertical transportation)

In chapter 2.4 it was decided to reserve ca. 1400 m^2 for vertical transportation. Alternative 2 does not have enough space. If the core area is increased the leasable depth becomes smaller and the building becomes less economic. A comparison of the space efficiencies for different the leasable depth is given in Table 3-5.

	Core area (m^2)	Leasable area (m^2)	Total area (m^2)	Nett –gross floor ratio
Alternative 2 Leasable depth 10,25 m	1521	4154	5675	73 %
Alternative 2 Leasable depth 15,00 m	1075	4600	5675	81%

Table 3-5 Alternative 2 space efficiency for different leasable depths

Note that making the circles bigger will also result in smaller width of the voids.

Connecting the towers

For both alternatives the slot width is similar however in alternative 2 the maximum distance between segments is significantly larger (57,5 meter) due to their round shape whereas the distance in alternative 1 is more or less constant. In the compound structure the 4 segments have to work together as a single entity and good connections are the most important in achieving this goal. The larger the span which needs to be crossed the more material and connections are necessary making alternative 2 unattractive from the view of constructability.

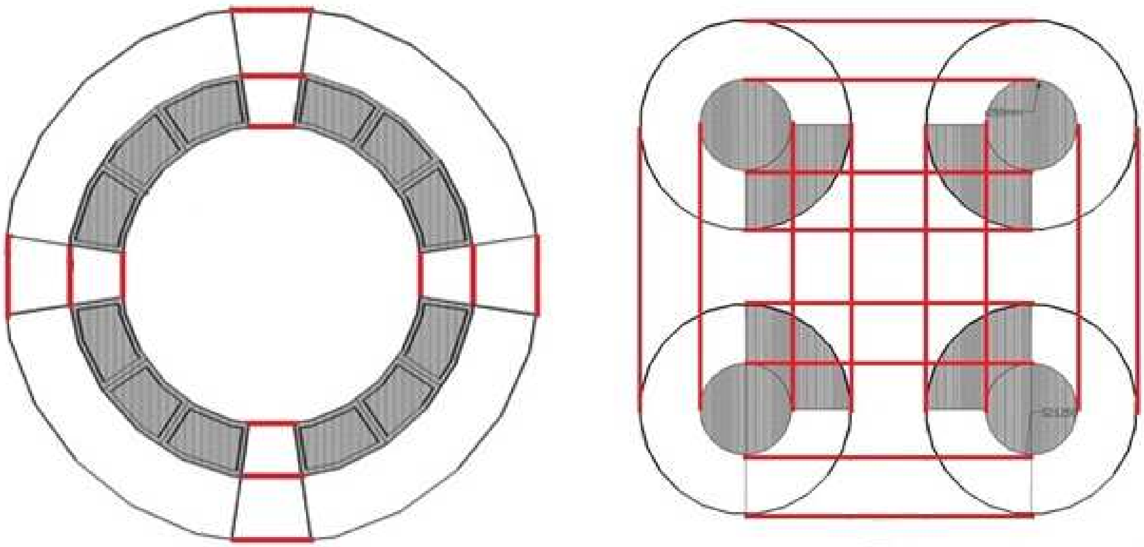


Figure 3-15 Connections in the footprint for the two alternatives.

Architectural

Alternative 2 might be harder to arrange due to the round shape of the segments. Alternative 1 is also round but because the circle is much bigger it's much easier to divide the footprint into more or less straight sections.

3.4 Final choice footprint

Different shapes (rectangle triangle and circle) have different advantages. A circular cross-section has the best behaviour in the along-wind direction while rectangular and triangular section behave better with respect to vortex shedding. The addition of slots to a round shape however reduces the formation of vortices by disrupting the wake. Resulting in a very aerodynamic shape.

Both alternatives (Figure 3-12 and Figure 3-13) have space efficiencies below the average of 69.5% seen in Table 3-2. According to (Watts et al. (2007) this can be explained as follows :

"Floor slab efficiency is adversely affected by the height of a high-rise office building, as the core and structural elements expand relatively to the overall floor slab to satisfy the requirements of vertical circulation as well as lateral-load resistance.
The problem is that as the relative height of the building increases, the proportion of the building that is devoted to elevators (particularly on the lower floors) must increase to prevent unacceptable bottlenecks as people enter and leave".

Since the height of an 800 meter building is larger we can expect to lose more to leasable area than in a 100-300 meter high building.

An important part of designing the compound structure is the connection of its segments at mechanical floors. This is done by:

- connecting the circular cores to each other
- connecting core to perimeter columns
- using a belttruss to connect perimeter columns

Connecting the quadrants to each other is more practical in alternative 1 since the distance between them is smaller 10-15 meter.

Because of the round shape of the quadrants a large span has to be crossed in alternative 2 to connect tubes. For alternative 2 a square tube is more practical because the distance the gap between the individual towers is now constant. This however means worse aerodynamic behaviour. The large gaps between the cores make it hard to ensure rigid connections which have the task of making the tower behave like one structural entity. If the connections between the towers are not stiff enough the links between the towers will be similar to an ordinary skybridge like the Petronas Towers which has no structural function.

Architecturally speaking alternative 1 looks more like a single building whereas alternative 2 clearly expresses the fact that the structure consist of 4 slender towers which are tied together. Depending on the architects preference both can be seen as positive.

The geometry of the internal void and slots are also an important consideration. To study the effect of the different geometries in alternatives 1 and 2 computational fluid dynamics analysis or wind tunnel research would be necessary.

In summary both alternatives are possible however alternative 2 is harder to erect and requires more complicated connections, thus alternative 1 (Figure 3-16) is chosen.

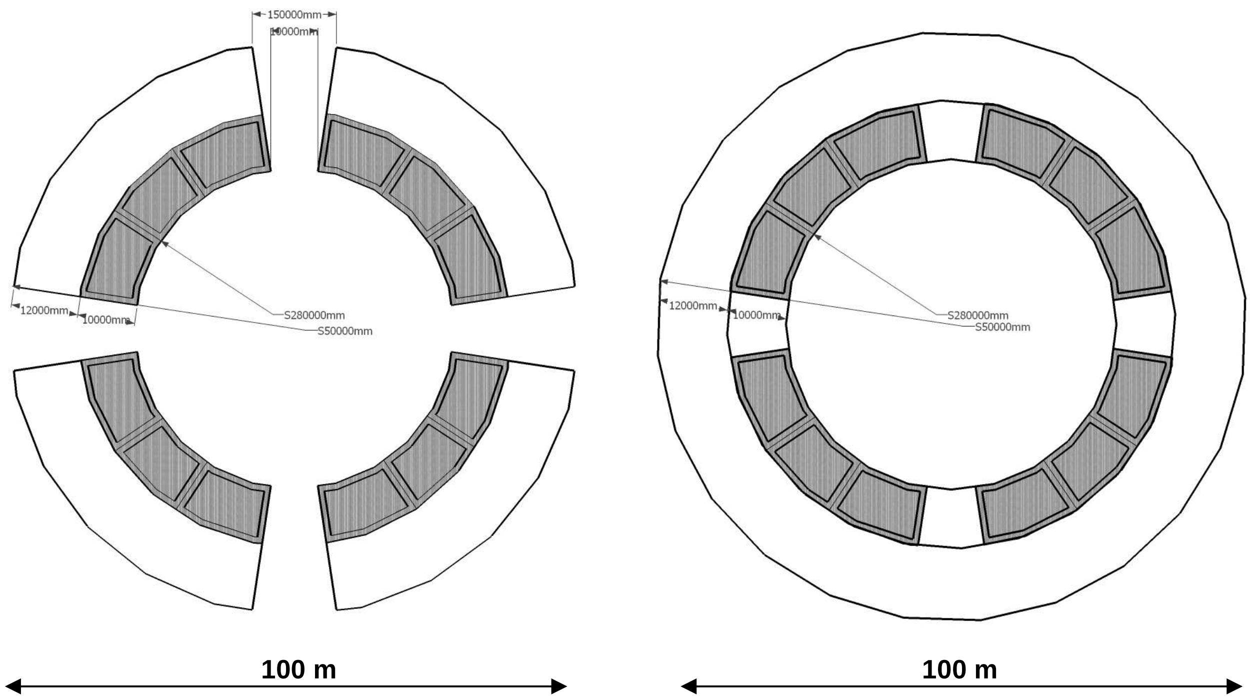


Figure 3-16 Final footprint (left normal floors, right mechanical floors)

Earlier it was mentioned that the economic feasibility of the chosen footprint can be improved by omitting certain walls as the buildings height increases (Table 3-4). For now there will be no omission of walls since the core together with the perimeter columns determines the total stiffness of the building. This means the core has the same shape along the height of the building. However after the building has been modelled in a finite element program and the deformation and forces are known it can be evaluated whether it is possible to improve the feasibility of the structure by omitting walls.

3.5 Floor-system

In chapter 3.4 floor systems in supertall buildings have been discussed. From a structural point of view the self-weight of the floor-system is very important since the large amount of storeys can cause large loads on the foundation and structural elements. The conclusion however, was that the choice for a floor-system is often determined by its non-structural characteristics. The impact a floor-system has on the erection process of a supertall building is especially important in this decision making process. In 3.4 it was found that a composite floor is very suitable for a supertall building since they reduce the dead load and storey height of the building and also ensure a feasible erection method.
For our tower we will use the composite floor: Comflor 210 in combination with an asymmetric floor beam to reduce storey height (see appendix D).

Figure 3-17 Floorsystem: Comflor 210

The chosen asymmetric floor beam can only achieve a maximum span of 9 meter. In order to achieve a span of 12 meter without additional supports adjustments can be made.

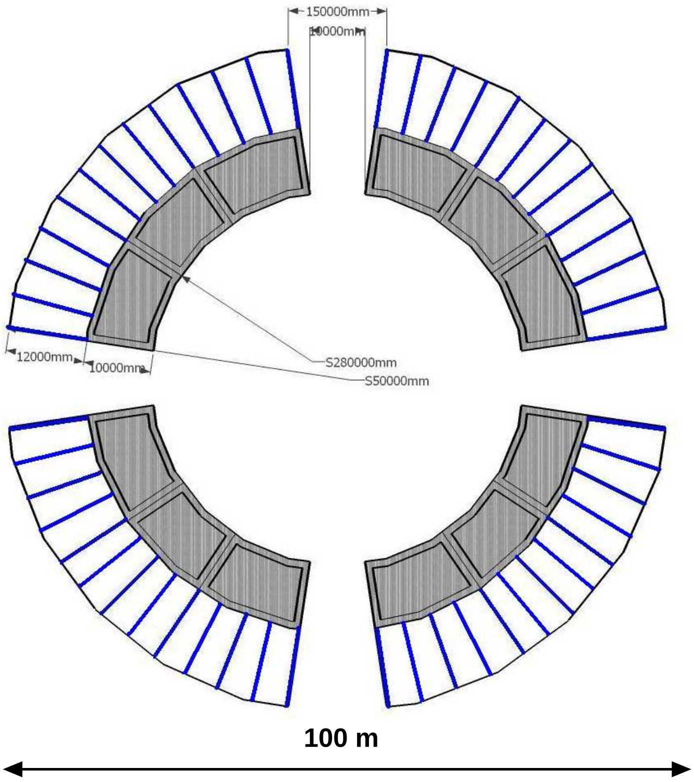


Figure 3-18 Position ASB Beams

Floor system	Span (m)	Dead load (kN/m^2)	Fire resistance (min)
Comflor 320	5.4	3.19	120
Installations and finishing	-------	0.75	------

Table 3-6 Floor-system

These adjustments increase the height of the beam and total floor-system with ca. 200 mm. Now that the floor-system is known the exact floor-to-floor height of the buildings functions can be determined. The floor-to-ceiling-height is 2700 mm for all functions except 3200 mm for penthouse.

Element	Height (mm)
Floor-system	340
Adjustments	200
Computerfloor	100
Installations	300
Finishing	60

Table 3-7 Storey height

Table 3-8 shows the floor-to-floor heights of each of the buildings function.

Function	Floor to ceiling height (mm)	Remaining height (mm)	Total height (mm)	Floor to floor height (mm)
Residential	2700	900	3600	3700
Hotel	2700	900	3600	3700
Penthouse	3200	900	4100	4100
Office	2700	1000	3700	3700
Flexible	2700	1000	3700	3700

Table 3-8 Storey height of the buildings functions

Since there is small difference (100 mm) between the storey height with the function residential, hotel, office and flexible are given the same standard height 3700 mm.
This choice is made because repetition of vertical elements such as columns façade elements etc. can have a positive effect on the planning and speed of the towers erection.

Appendix D: Floor-system shows the structural sections which were used and a table with the absolute height of every storey. In total there are 208 storeys and the maximum height of the building is 801.9 meters.

Function	Storey number
Penthouse	186-208
Residential	144-169, 173-185
Hotel	115-140
Flexible	69 -111
Office	8-10, 24-68
Commercial	1-6
Mechanical	5, 21-23, 50-52, 80-82, 112-114, 141-143, 170-172
Basement	(-1)- (-7)

Table 3-9 Building functions and storey numbers

3.6 Structural alternatives for the perimeter

One of the goals of designing a superstructure for a supertall building is creating a maximum internal lever arm. By having the perimeter participate in the transfer of lateral loads the structural behavior of the building is greatly increased. There are two ways to achieve this, namely;

- Megacolumns at the perimeter which are connected to the core at several points along the buildings height using outriggers and belt trusses.
- A perimeter tube which consists of closely spaced diagonal and /or vertical columns and is coupled to the core through floorslabs.

These alternatives are very suitable for high-rise since the internal lever arm reaches from façade to façade. A choice between these two alternatives will be made in the chapter 4.

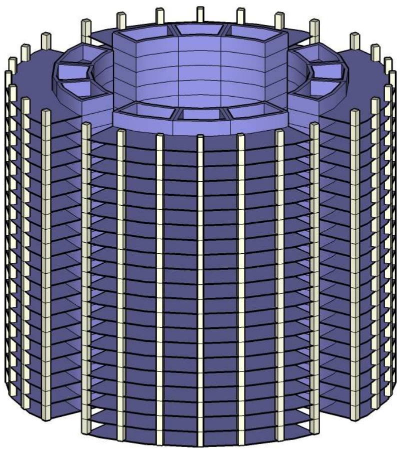


Figure 3-19 Perimeter with megacolums

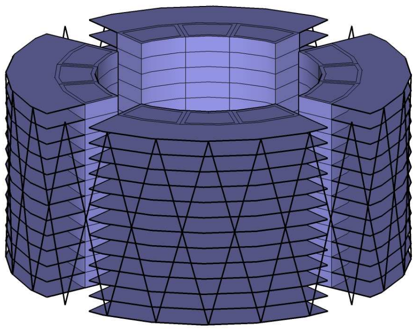


Figure 3-20 Perimeter tube

Perimeter trussed tube

A perimeter tube is very stiff and has a an almost continuous coupling to the core over the height of the building (every floor). Advantages of the tube structure are
That it can add a certain roughness to the buildings surface which has a beneficial effect on the wind-induced behaviour. Also diagonals cause a highly redundant structural network that allows multiple load paths.
Disadvantages of this system are the architectural obstruction as a result of the diagonals and a slower erection which can affects the feasibility of an 800 meter high building.

Megacolumns

The erection process of megacolumns is very practical due the ability to pump concrete up to great heights. Unlike the the tube structure the only architectural obstruction is the result of the outriggers which connect the columns to the core. This can be solved by placing the outrigger at the mechanical floors. A disadvantage of this system is that it is not as stiff as a tube structure.

3.6.1 Connecting the towers

In both alternatives the four quadrants need to work as one structural entity. In this paragraph we will discuss how this will be achieved.

The slender towers are connected at the mechanical floors at every 25 – 30 storeys. The mechanical floors will house belttrusses and outriggers. The difference between the tube alternative and the core outrigger system is that in the core-outrigger alternative the towers are connected to each other only at 6 places namely, the three-storey high mechanical levels (see table 1-2). The tube structure on the other hand has an almost continuous connection (by means of the floor slabs) to the core along the buildings height.

Tube structure

In the tube alternative the floor slabs are used as "diaphragms" to efficiently collect and distribute horizontal forces to other parts of the building lateral force resisting system. The idea being that the more this is done, the more uniformly the building will move. Both the floor slab and the horizontal members comprising the diagrid framing will act to transfer these forces.

Figure 3-21 Connection floor and diagrid Swiss Re

Diagrid design

The diagrid system is discussed in paragraph 3.3.2 of part 1 (literature study) [13, 14, 15, 16] recommends using varying angles for supertall buildings to save material. The angle used is however dependent on the storey height and the order of the different functions i.e. residential or office function.

According to [13, 14, 15, and 16] the optimal building angle lies between ca. 60 en 70 degrees. In paragraph 2.2.1 the vertical allocation of the buildings functions has been determined and in paragraph 3.5 the floor-to-floor height has been determined. Earlier it was

decided to have a constant floor-to-floor height of 3.7 meters for all the functions except penthouse and mechanical floors.
Table 3-10 gives the floor- to-floor height of the buildings different functions.

Function	Floor -to-ceiling height (mm)	Remaining height (mm)	Total height (mm)	Floor-to-floor height (mm)
Residential	2700	900	3600	3700
Hotel	2700	900	3600	3700
Penthouse	3200	900	4100	4100
Office	2700	1000	3700	3700
Flexible	2700	1000	3700	3700

Table 3-10 Floor-to-floor height

Since these variables are fixed the angle between the columns is related the floor-to floor height of the structure. Because of this relation it is not practical to use a diagrid with varying angles as recommended by [13, 14, 15, and 16]
it is more practical too use a constant angle between the columns of the diagrid.

Core-outrigger

The core and megacolumns are connected through 16 outriggers which span 12 meter.
These outriggers are 3 storey high trusses which connect the main stabilizing element corewalls to perimeter columns. The outriggers can be seen as extensions of the 16 core walls and allow the perimeter columns to participate in the transferring of horizontal forces working on the building. Axial forces in the columns create a counteracting bending moment which helps reduce the forces and deflection due to lateral wind loads. A belt-truss consisting of ties and diagonals make sure that the remaining 12 perimeter columns are also activated.

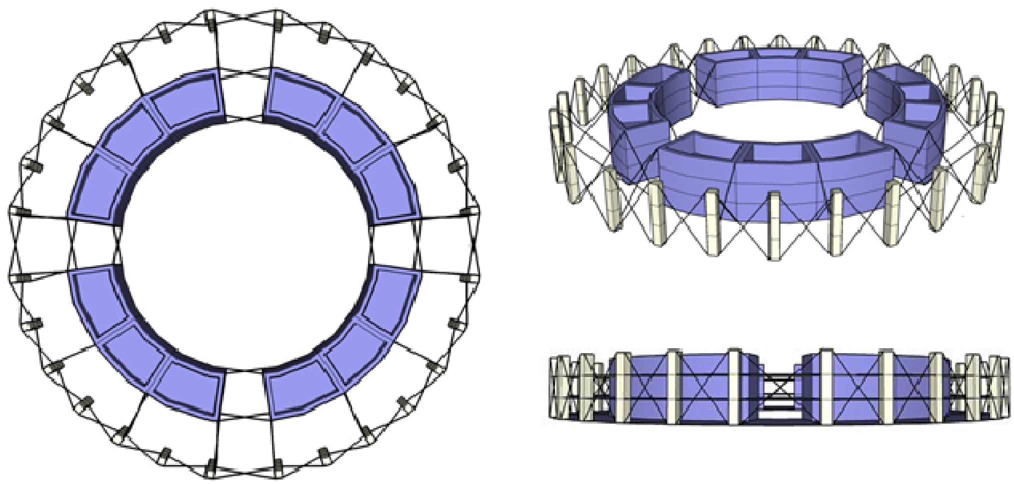


Figure 3-22 Outriggers and belttruss

The locations of these outriggers and belttrusses are given in Table 3-11.

Level	Position (m)	Number storeys	Height (m)
21-23	92.1-104.1	3	12
55-57	218.8-230.3	3	12
89-91	345.5-357.5	3	12
123-125	472.2-484.2	3	12
156-158	595.2-607.2	3	12
184-186	699.7-711.7	3	12

Table 3-11 Position outriggers (mechanical floors)

Whether the building works as a structural entity is dependent on the number and stiffness of the connection between the four quadrants. In [7] the optimum location and number of outriggers is examined and a maximum of 4 outriggers is recommended.
The number of couplings along the height of the building however is 6 (see Table 3-8), exceeding the aforementioned recommended number 4. The amount of outriggers follows from the demands of the towers vertical transportation system which separates the tower in several 25 – 30 storey neighbourhoods in order to save space with a stacked lift system.
The dimensions of our tower are however much larger therefore we will study the influence the number of couplings has on the stiffness of the tower using a finite element program.
The connections at the mechanical level also have a functional significance. The skybridge functions as transfer floors and divides the building into several vertical neighbourhoods. It would be nice to have a place where inhabitants or employees can recreate, socialize or interact in the skybridge. A structure consisting of a canopy and trusses can create meeting point for inhabitants at the 3-storey high skybridge.

3.7 Structural elements and material choice

Composite elements are chosen to try and reduce the dead load as much as possible but at the same time maintain the mass needed to resist lateral loads.

Concrete

As mentioned in the literature study high strength concrete offers many advantages for high-rise construction therefore concrete C90/105 will be used for the core and composite columns.

Material properties

Strength class	: C90/105 (f_{ck} = 105 N/mm^2)
Mass density	: ρ = 24 kN/m^3

Long term strength

Even though wind is a short term load we can't simply use the short term elasticity modulus for the concrete (C90). This is because initial displacement as a result of inevitable errors during erection can cause a bending moment which works on the building as a long term load. When concrete is subjected to long term compression stress its strength is lower than the short term strength namely, ca. 80 % (some say 70-85%). This subject is however debatable. Many people however argue that a reduction due to the long term effect is unnecessary since a building is only subjected to high/large loads when the building is at least 6 months old. Because of the on-going hydration of the cement the strength of the concrete will be higher than the 28 day compressive strength on which the strength tables are based. These two effects cancel each other out and the eurocode therefore leaves it up to each country to decide whether or not to apply a reduction but recommends that the long term effect should be neglected. In this thesis the recommendation found in the eurocode is followed meaning the long term effect is neglected.

Tensile stresses

We will assume there are no tensile stresses in the concrete due to the large vertical loads and will not take into account a reduction due to cracking of concrete. After the forces have been calculated the structural elements will be checked for tensile stresses.

Steel

The beams and the steel in the composite columns are made of S235. The tower will consist of the following structural elements:

- Concrete core
- Composite floor
- Composite megacolumns
- Steel beams
- Trussed Tube

Core

As mentioned in paragraph 3.3.1 of part 1, supertall buildings usually have concrete cores which are used for vertical transport. It makes sense to use these cores to transfer vertical and lateral loads working on the building.
The tower will have 4 concrete cores, one for each quadrant, which are tied together at the mechanical floors. In order to reduce the load on the foundation the cores will have setbacks meaning that along the height of the thickness of the core walls changes.

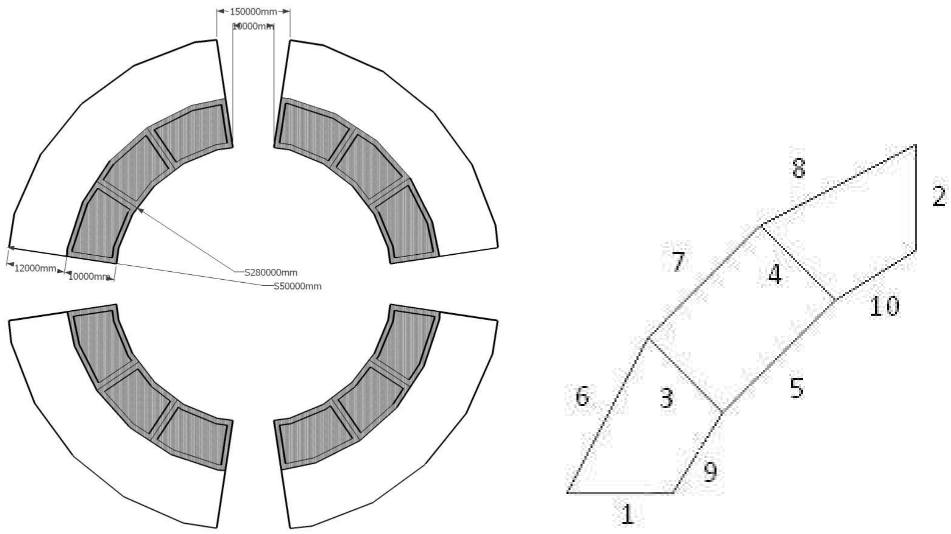


Figure 3-23 Footprint (left), core walls (right)

Columns

Composite columns are used for core outrigger alternative and circular hollow sections are used for the tube alternative.

Figure 3-24 Composite column

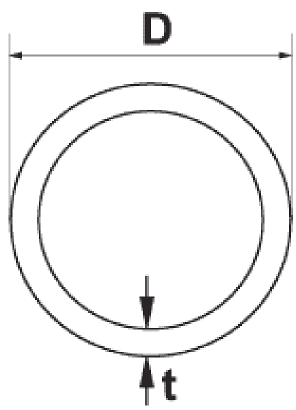


Figure 3-25 Circular hollow section

Chapter 4 Superstructure

4.1 Introduction

In this chapter, alternatives from paragraph 3.6 will be tested with respect to their static and dynamic structural behavior.

As mentioned in the conclusion of *part 1: literature study* the biggest challenge of a superstructure in a supertall is resisting lateral loads. In this paragraph we will model and analyse the behaviour of the tower using the finite element program ESA SCIA Engineer. The deformation of different alternatives will be compared to building codes in the schedule of requirements (appendix B).

When a tower is modelled as a one-dimensional element i.e. a fixed column it has:

- a bending stiffness EI
- a shear stiffness GA
- a rotation stiffness C (at the foundation)

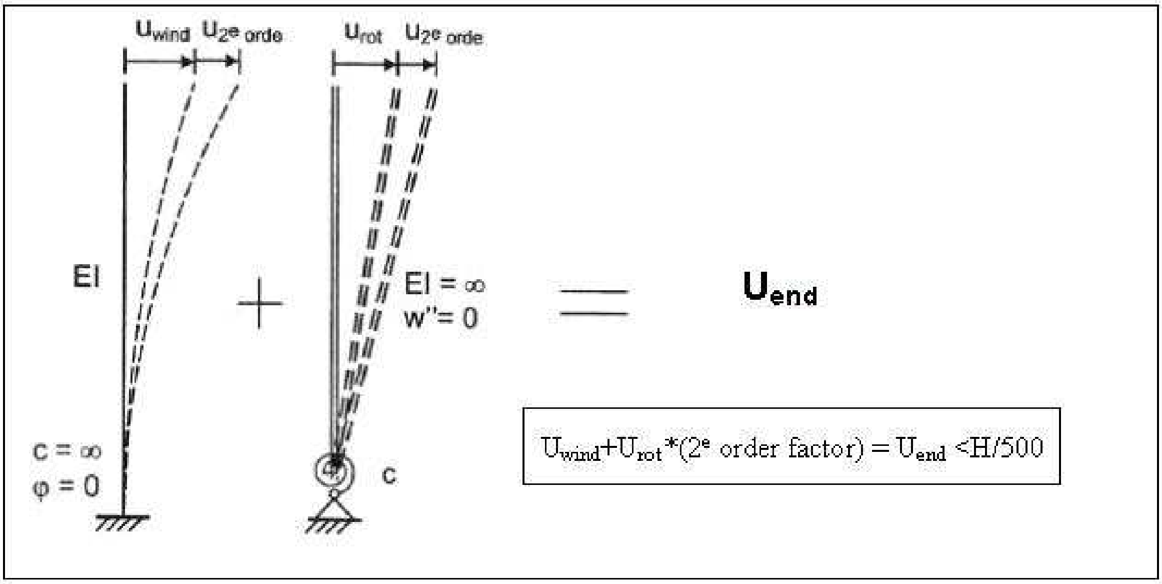


Figure 4-1 Deformation of building modeled as a one-dimensional element.

The drift is then result of the deformation due to:

- The wind load
- The second order effect
- The rotation of the foundation

The wind load has been determined in appendix B and C.

It's hard to estimate the rotational stiffness when the foundation has not yet been designed. The rotational effect is however very important since increasing the stiffness of the tower has no meaning if the foundation is not stiff enough.

In this chapter a clamped constraint will be used to model the connection between the tower, the foundation and the soil. The rotation stiffness of the foundation will be calculated later when the dimensions of the foundations structural elements are known.
When examining the superstructure we will assume the relation between the deformation due to the bending of the tower and the rotation of the foundation is 1:1. In other words the deformation due to the wind load and the second order effect is H/1000. Since our tower is 801.9 meters high this means that the deformation must be lower than ≈ 800 mm.

The second order effects have to be taken into account when determining the deformations of a tall building. This effect is the result of the extra deformation caused by the gravity loading and will be calculated using ESA SCIA Engineer.

4.2 loads

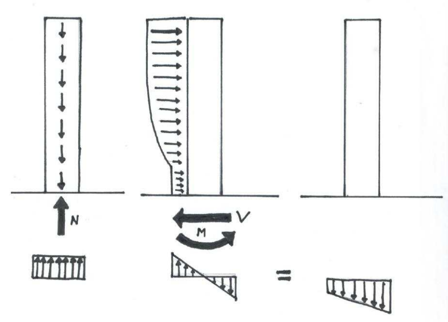


Figure 4-2 Loads working on the building

The load on the foundation consists of:

- The vertical loads caused by the building self-weight.
- The vertical live loads cause by users / inhabitants.
- The vertical loads caused by wind loads.

The loads which work on the building have been calculated in Appendix E

4.2.1 Load cases and combinations

Load cases

LC1: Dead Load; Self weight
LC2: Live Load; Imposed loads
LC3: Live Load; Wind

Load combinations

Ultimate limit state

Load combinations	Normal	Favourable	Unfavourable
fundamental	1.2	0.8	1.5

Table 4-1 fundamental load combination

Serviceability limit state

Load combinations	Normal	Favourable	Unfavourable
characteristic	1.0	1.0	1.0

Table 4-2 Characteristic load combination

4.3 Structural alternatives within a compound structure

The load bearing structure has the following goals:

- Transferring the loads which act on the building to the foundation. These loads consist of vertical loads such as the buildings self-weight and live loads and horizontal loads caused by wind forces.
- Making sure the deformations caused by these loads are kept within the limits defined in PVE
- Guaranteeing the stability of the building.

The findings from the literature study and reference projects show that a compound structure is very promising since it reduces the wind loads and offers a solution to the problem of a lack of daylight entry for large cross-sections.

The core-outrigger and tube alternative from chapter 3.6 will be tested in the following chapters. For good measure a load-bearing structure consisting only of a core will also be tested.
We will examine the following alternatives for the load-bearing structure of the Rijnhaven Tower:

- Core
- Tube structure
- Core-outrigger

A supertall is usually divided into several segments which each have their own structural elements. The elements at the bottom are the larger than those at the top because they carry the largest load. This is done to reduce the load on the foundation and to reduce cost by saving materials.
For this project the building is subdivided in 7 parts which each have a different bending stiffness EI due to the decreasing of the wall thickness as the building height increases. This gives the following subdivisions:

Subdivision	Length (m)	Position (m)
EI 1	92.1	0-92.1
EI 2	126.7	92.1-218.8
EI 3	126.7	218.8-345.5
EI 4	126.7	345.5-472.2
EI 5	123	472.2-595.2
EI 6	104.5	595.2-699.7
EI 7	102.2	699.7-801.9

Table 4-3 Subdivisions tower

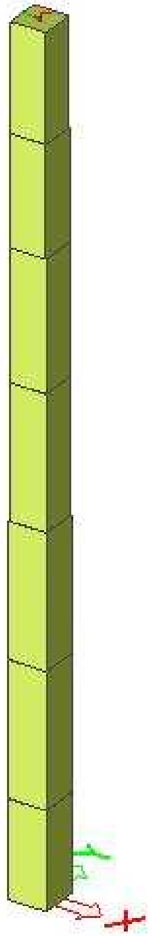
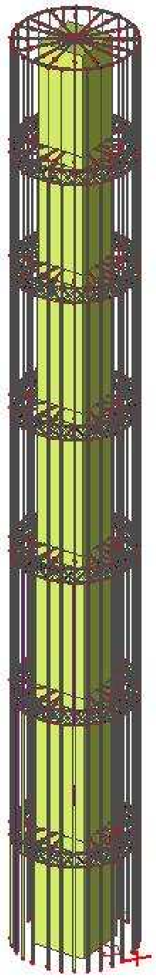
Alternative 1: Core	Alternative 2: Core-outrigger	Alternative 3: Diagrid
		

Figure 4-3 Three structural alternatives

4.3.1 Alternative 1: Core

Structural elements

In the first alternative the load-bearing structure of the tower consists out of a concrete core and megacolumns which both have setbacks along the buildings height. The megacolumns however do not participate in the transfer of lateral loads and only transfer vertical loads to the buildings foundation.
The functions of the core are therefore providing stiffness and stability to the building and transferring vertical and lateral (wind) forces to the foundation.
The buildings core consists of four quadrants which are linked to each other using belt-trusses at the positions which are given in table (Table 4-4). This is where outrigger and mechanical floors are located.

Level	Position (m)	Number storeys	Height (m)
21-23	92.1-104.1	3	12
55-57	218.8-230.3	3	12
89-91	345.5-357.5	3	12
123-125	472.2-484.2	3	12
156-158	595.2-607.2	3	12
184-186	699.7-711.7	3	12

Table 4-4 Position outriggers (mechanical floors)

Each quadrant of the footprint has its own core which consists of 10 concrete walls. Figure 4-4 shows schematization of the buildings concrete cores.

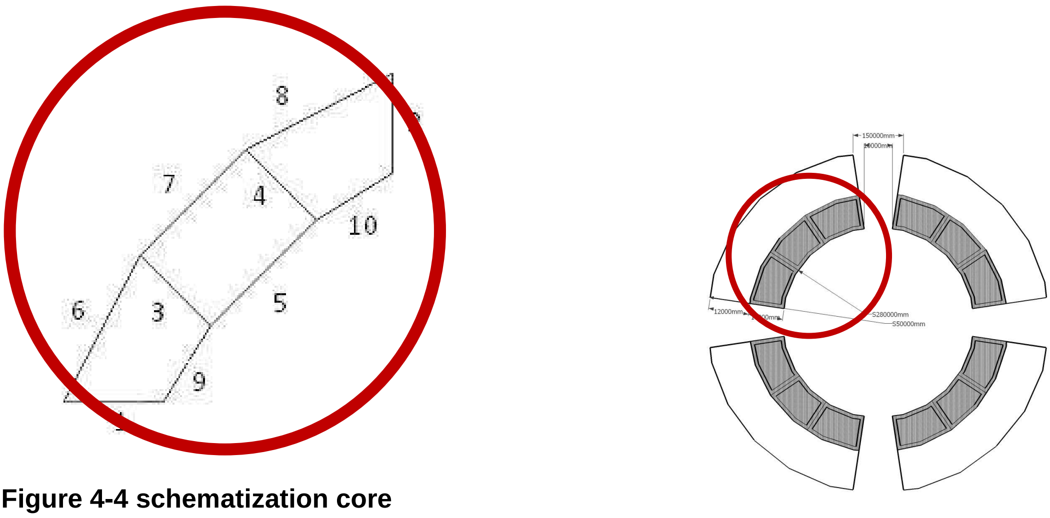


Figure 4-4 schematization core

The stiffness of the core depends on the stiffness of the belttruss and the number of couplings along the height of the building. Earlier 6 coupling were chosen based on recommendations found in reference projects [27] and [28]. A supertall is usually divided in 25-30 high storey sections in order to keep the area lost to vertical transportation limited.
In order to understand the effect which the couplings have on the deformation and bending stiffness of the tower a study was done using 8 models having respectively 1, 2 ,3, 4, 6, 8, 10 and 12 couplings along the height of the building (see Figure 4-6). These models were all subjected to a unit load "q" of 100 kN/m and the bending stiffness of the tower 7 subdivisions was kept the same for all models.

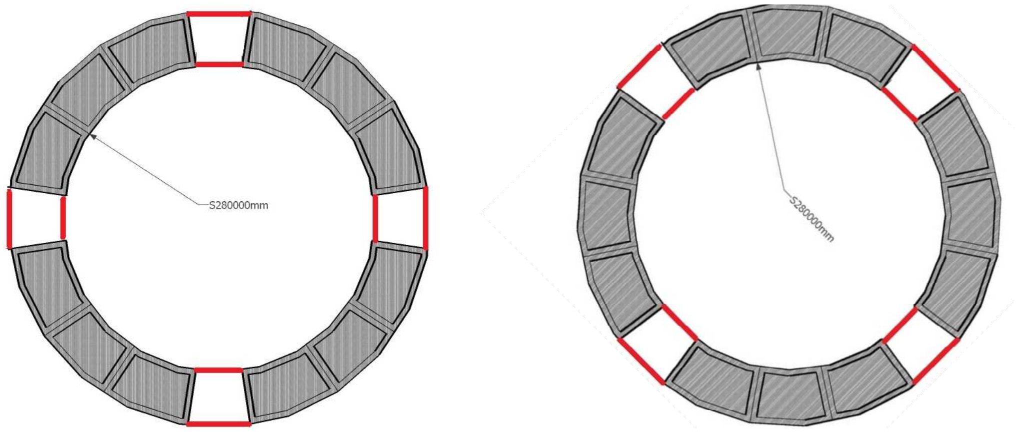


Figure 4-5 Belt trusses load case 1 (left) and load case 2 (right)

Table 4-5 and Figure 4-6 show that when the number of couplings increases the stiffness of the building also increases. The improvements in structural behaviour are significant during the jump from 1 to 2, 2 to 3 and 3 to 4 couplings along the height of the building. The increase of stiffness however is not as significant when we compare a tower with 4 couplings to a tower with 6 couplings. The deflection is reduced by 11 % whereas the jump from 3 to 4 couplings reduces the buildings deflection by 32 %. To achieve a similar reduction as seen at the jump from 3 to 4 couplings 8 couplings need to be added to the structure which consists of 4 couplings. The jump from 4 to 12 couplings along the height of the buildings results in a 27 % reduction of the deformation at the top of the building.
As mentioned in the literature study a disadvantage of the core-outrigger system is the fact the outriggers, which make sure the columns at the perimeter participate in the transfer of lateral load transfer, interfere with the occupiable and rentable space in a supertall.
This is often solved by placing the outrigger in the mechanical floors. Also extra outriggers can have a negative effect on the erection process.
As mentioned earlier the belttrusses are three storeys high which means that a tower which has 12 couplings along the height of the building has a total of 36 storeys in which outriggers interfere with the leasable space. This is 24 more storeys than a tower with 4 couplings which has only 12 storeys containing outriggers and belttrusses.
The findings in appendix F correspond well with the results found in [7].
It can be concluded that the jump from 4 to 6 couplings along the height of the building does not result in significant improvements. From a structural point of view the increase in stiffness is always good but we must also consider the economic feasibility of the tower. Since a lot of leasable space is sacrificed for a smaller increase in stiffness we choose not the use more than 6 couplings along the height of the building which is the number of couplings which was chosen in paragraph chapter 2.2.

Number of couplings	**1**	**2**	**3**	**4**	**6**	**8**	**10**	**12**
Deformation (mm)	3084	1660	786	535	478	436	404	391
Average bending stiffness EI (N/mm^2)	1.68E+21	3.11E+21	6.58E+21	9.66E+21	1.08E+22	1.19E+22	1.28E+22	1.32E+22

Table 4-5 Effect number of couplings along the height of the building

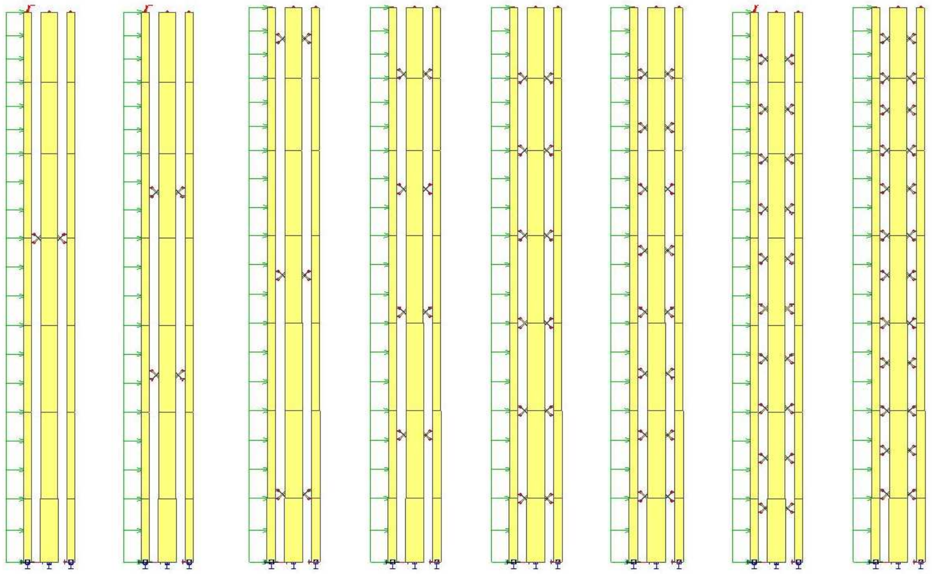
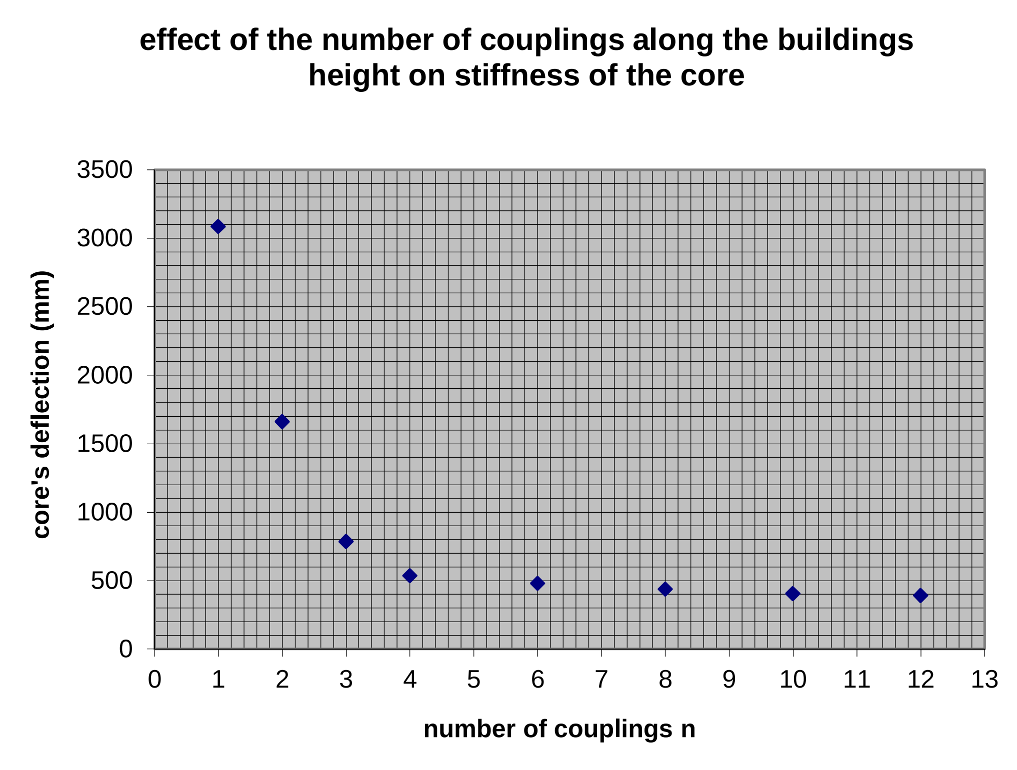


Figure 4-6 Number of couplings along the height of the tower



Graph 4-1 Effect couplings on stiffness of the core

The bending stiffness EI of the subdivisions haven been calculated using an excel spreadsheet and ESA Prima Win. A detailed explanation can be found in appendix F and the results are given in Table 4-6

subdivision	Core EI (N/mm^4)	Wall thickness (mm)
Floor 0-22	1.3639E+22	1000
Floor 21-54	1.2287E+22	900
Floor 55-88	1.1494E+22	850
Floor 89-122	1.0813E+22	800
Floor 123-155	1.0134E+22	750
Floor 156-183	9.0807E+21	700
Floor 184-208	8.4314E+21	650

Table 4-6 Bending stiffness subdivisions tower

In this alternative the columns do not participate in the transfer of horizontal forces as they only transfer vertical forces. For the calculation of the composite columns (see appendix F)

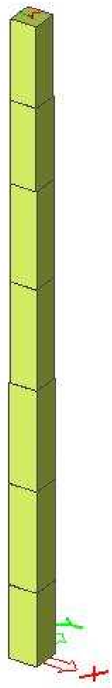
Level	Floors	Design load (kN)	Height (m)	Max. floor-to-floor height	Dimensions (mm)
0-22	208	123117	0-92.1	4	1600x1600 t =100
21-54	188	110679	92.1-218.8	4	1500x1500 t =100
55-88	154	89942	218.8-345.5	4	1300x1300 t = 80
89-122	120	68790	345.5-472.2	4	1200x1200 t = 70
123-155	86	48462	472.2-595.2	4	1000x100 t = 60
156-183	53	31244	595.2-699.7	4	800x800 t = 50
184-208	25	13706	699.7-801.9	4,1	600x600 t = 50

Table 4-7 Cross-section and location megacolumns

ESA Model

Cross-section

Table 4-8 and Table 4-9 show the dimensions of the cross section used in the ESA model of the alternative where the lateral loads are resisted only by the cores.



Subdivision	Height (m)	A (m^2)	E (N/mm^2)
EI1	0-92.1	43916x43916	44000
EI2	92.1-218.8	42785x42785	44000
EI3	218.8-345.5	42078x42078	44000
EI4	345.5-472.2	41440x41440	44000
EI5	472.2-595.2	40733x40733	44000
EI6	595.2-699.7	39670x39670	44000
EI7	699.7-801.9	38941x38941	44000

Table 4-8 Cross-sections core ESA model

Megacolumn	Height (m)	A (m^2)	E(N/mm^2)
EI1	0-92.1	1770 x1770	44000
EI2	92.1-218.8	1680 x1680	44000
EI3	218.8-345.5	1440 x1440	44000
EI4	345.5-472.2	1320 x1320	44000
EI5	472.2-595.2	1100 x1100	44000
EI6	595.2-699.7	890 x 890	44000
EI7	699.7-801.9	690 x 690	44000

Table 4-9 Cross-section megacolumns ESA model alternative 1

Material

Material	Strength (N/mm^2)	E- modulus (N/mm^2)
Concrete (C90/105)	Fcd=63	44000

Table 4-10 Materials ESA model

Constraints

For now the tower is modelled as having a clamped support at (0,0,0) in reality however the foundation piles and plate act as a rotational spring which causes an additional deformation. This rotational effect will be taken into account later when the foundation has been designed.

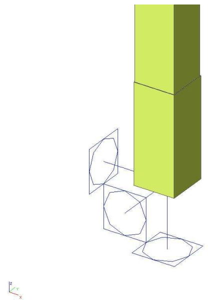


Figure 4-7 Tower constraints

Load cases

LC1: Dead Load; Self weight (vertical line load)
- 13207 kN/m^1 on the core

LC2: Live Load; Imposed loads
- 2563 kN/m^1 on the core (vertical line load) (478*4291/801,9)

LC3: Wind load; according to paragraph 3.2.1

For more details see appendix E:

Load combinations

See paragraph 4.2.1

Results

Deformation	1th order (mm)	2nd order (mm)	n/n-1
ULS	826	979	1,19
SLS	550	648	1,18

Table 4-11 Maximum deformation alternative 1

Max moment	1th order (kNm)	2nd order (kNm)	n/n-1
ULS	$61.5*10^6$	$70.4*10^6$	1,15
SLS	$41.0*10^6$	$46.6*10^6$	1,14

Table 4-12 Maximum bending moment alternative 1

Shear force	1th order (kN)	2nd order (kN)
ULS	141893	143861
SLS	94596	96188

Table 4-13 Shear forces alternative 1

The maximum deformation at the top of the building in the serviceability limit state is 648 mm or h/1235 (2nd order effect included). This value is below the limit of 800 mm or h/1000 determined in the building code.
The moment is caused by the lateral wind loads working on the building. Figure 4-8 shows the expected parabolic moment of a clamped beam. The max moment at the base is $61.5 *10^6$ kNm in the ultimate limit state and increases to $70.4 *10^6$ kNm when the second order effect is included.

The results found using the ESA model show that the alternative where the lateral loads are resisted by the core only quite stiff. This can be attributed to the fact that in the design of a compound structure the core is located closer to the perimeter than conventional superstructures and the use of high strength concrete.

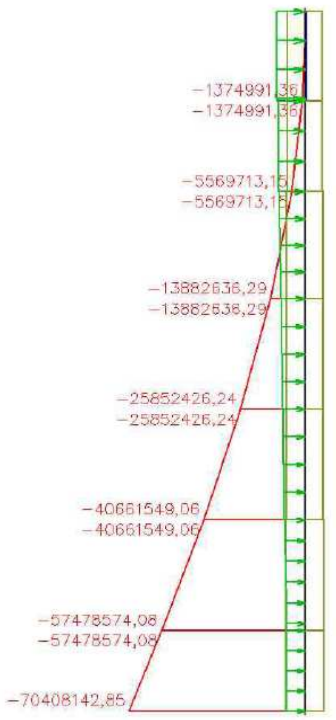


Figure 4-8 Moment diagram core alternative

Spanning

Niet-lineaire berekening, Extreem : Staaf
Selectie : Alle
Niet-lineaire combinaties : combi1

Staaf	BG	dx [m]	Normaal - [MPa]	Normaal + [MPa]	Afschuiving [MPa]	Von Mises [MPa]	Vermoeiïng [MPa]	Kappa [-]
S691	combi1	0,000	-11,6	0,0	0,1	11,6	0,0	0,00
S691	combi1	126,700	-9,0	0,0	0,1	9,0	0,0	0,00
S940	combi1	0,000	-9,4	0,0	0,1	9,4	0,0	0,00
S940	combi1	126,700	-6,9	0,0	0,1	6,9	0,0	0,00
S1101	combi1	0,000	-7,1	0,0	0,1	7,1	0,0	0,00
S1101	combi1	126,700	-4,7	0,0	0,1	4,7	0,0	0,00
S1106	combi1	0,000	-4,9	0,0	0,1	4,9	0,0	0,00
S1106	combi1	123,000	-2,8	0,0	0,0	2,8	0,0	0,00
S1159	combi1	0,000	-3,0	0,0	0,1	3,0	0,0	0,00
S1159	combi1	104,500	-1,3	0,0	0,0	1,3	0,0	0,00
S1188	combi1	0,000	-1,4	0,0	0,0	1,4	0,0	0,00
S1188	combi1	102,200	0,0	0,0	0,0	0,0	0,0	0,00
S1189	combi1	0,000	-12,7	0,0	0,1	12,7	0,0	0,00
S1189	combi1	92,100	-10,9	0,0	0,1	10,9	0,0	0,00

Table 4-14 No tensile stresses in the concrete core

Table 4-14 shows that there are no tensile stresses in the concrete core this means that the assumption made in paragraph 3.7 are correct and that a reduction of the elasticity moduus due to cracking is not necessary.

4.3.2 Alternative 2: Core-outrigger

The load-bearing structure of the core-outrigger alternative consists of concrete cores and megacolumns which have setbacks along the vertical axis. The core is connected to the perimeter megacolumns using steel outriggers which allow them to participate in the transfer of horizontal forces.

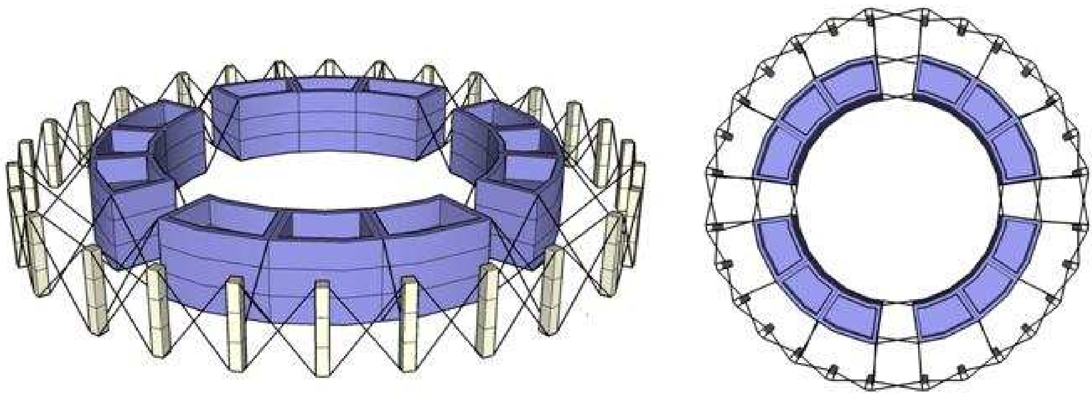


Figure 4-9 Outrigger and belttrusses

The functions of the core are:

- Providing stability
- Providing stiffness
- Transferring vertical forces to the foundation
- Transferring lateral forces (wind) to the foundation

At mechanical floors (3 storeys) the concrete core is connected to perimeter megacolumns using 16 steel outrigger trusses, 4 for each quadrant. Also belttrusses connect the columns to each other allowing more all the columns to participate in lateral load transfer see Figure 4-9 .

Structural elements

Core

For the core the same dimensions as paragraph 4.3.1 are used.

Subdivision	Core EI (N/mm^4)	Wall thickness (mm)
Floor 0-22	1,3639E+22	1000
Floor 21-54	1,2287E+22	900
Floor 55-88	1,1494E+22	850
Floor 89-122	1,0813E+22	800
Floor 123-155	1,0134E+22	750
Floor 156-183	9,0807E+21	700
Floor 184-208	8,4314E+21	650

Table 4-15 Bending stiffness core alternative 2

Megacolumns

The megaclumns are now participating in the transfer of lateral forces so the besides transferring vertical loads the also create a counteracting moment which reduces the base moment of the structure.
The megacolums are therefore subjected to the following loads:

- The variable loads from the functions such as office residential etc.
- The permanent loads such as self-weight of the floor beams and the façade.
- The vertical forces caused by the lateral forces working on the building

Half of the loads from the leasable span (6 m) are carried by the columns and the other half is transferred to the core.

Each quadrant consists of 7 megacolumns meaning there are 28 megacolumns in the entire footprint. The megacolumns are composite columns consisting of steel rectangular hollow sections filled with concrete 90/105 and have a centre-to-centre distance of 10.6 meter.

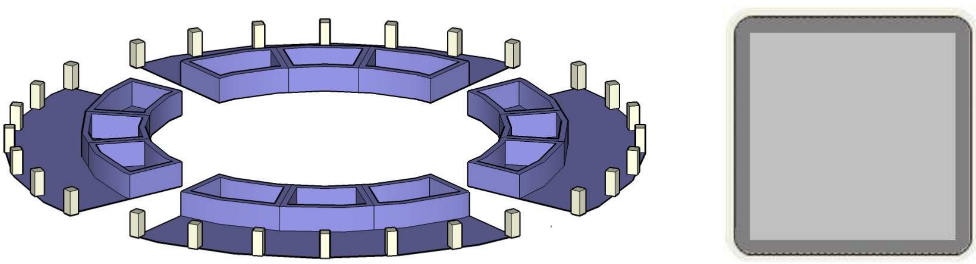


Figure 4-10 Cross-section at voids and slots

The building is subdivided in 7 parts and similar to the setbacks of the core the columns have different sizes for each subdivision.

Level	Floors	Design load (kN)	Height (m)	Max floor-to-floor height	Dimensions (mm)
0-22	208	123117	0-92.1	4	1600x1600 t =100
21-54	188	110679	92.1-218.8	4	1600x1600 t =100
55-88	154	89942	218.8-345.5	4	1500x1500 t =100
89-122	120	68790	345.5-472.2	4	1300x1300 t = 80
123-155	86	48462	472.2-595.2	4	1200x1200 t = 70
156-183	53	31244	595.2-699.7	4	1000x1000 t = 60
184-208	25	13706	699.7-801.9	4,1	800x 800 t = 50

Table 4-16 Cross-section and location megacolumns

The bending stiffness EI of the subdivisions haven been calculated using a spreadsheet and are given in Table 4-17.

Subdivision	Megacolumns EI (N/mm^2)
0-22	$3.936 * 10^6$
21-54	$3.142 * 10^6$
55-88	$1.700 * 10^6$
89-122	$1.203 * 10^6$
123-155	$5.890 * 10^5$
156-183	$2.460 * 10^5$
184-208	$9.060 * 10^4$

Table 4-17 Bending stiffness megaclumns

Outrigger and Belttruss

Height (m)	A (mm^2)
0-92.1	1200x1200 t = 70
92.1-218.8	1200x1200 t = 70
218.8-345.5	1200x1200 t = 70
345.5-472.2	1200x1200 t = 70
472.2-595.2	1200x1200 t = 70
595.2-699.7	1200x1200 t = 70
699.7-801.9	1200x1200 t = 70

ESA Model

Cross-section

The following values are used for the cross-sections of the different structural elements.

Core	Height (m)	A (mm^2)	E (N/mm^2)
EI1	0-92.1	43916x43916	44000
EI2	92.1-218.8	42785x42785	44000
EI3	218.8-345.5	42078x42078	44000
EI4	345.5-472.2	41440x41440	44000
EI5	472.2-595.2	40733x40733	44000
EI6	595.2-699.7	39670x39670	44000
EI7	699.7-801.9	38941x38941	44000

Table 4-18 Cross-section core ESA model alternative 2

Megacolumn	Height (m)	A (mm^2)	E (N/mm^2)
EI1	0-92.1	1770 x1770	44000
EI2	92.1-218.8	1680 x1680	44000
EI3	218.8-345.5	1440 x1440	44000
EI4	345.5-472.2	1320 x1320	44000
EI5	472.2-595.2	1100 x1100	44000
EI6	595.2-699.7	890 x 890	44000
EI7	699.7-801.9	690 x 690	44000

Table 4-19 Cross-section megacolumns ESA model alternative 2

Outrigger

Height (m)	A (mm^2)
0-92.1	1320x1320
92.1-218.8	1320x1320
218.8-345.5	1320x1320
345.5-472.2	1320x1320
472.2-595.2	1320x1320
595.2-699.7	1320x1320
699.7-801.9	1320x1320

Table 4-20 Cross-section outrigger elements in ESA

Material

materials	Strength(N/mm^2)	E modulus (N/mm^2)
Steel (S235)	235	210000
Concrete (c90/105)	Fcd=63	44000

Table 4-21 Materials ESA model alternative 2

The core and megacolumns are modelled as concrete and the outriggers and belltrusses are made from steel.

Constraints

All the megacolumns have hinged supports and the core has a clamped support at (0,0,0) (Figure 4-11)

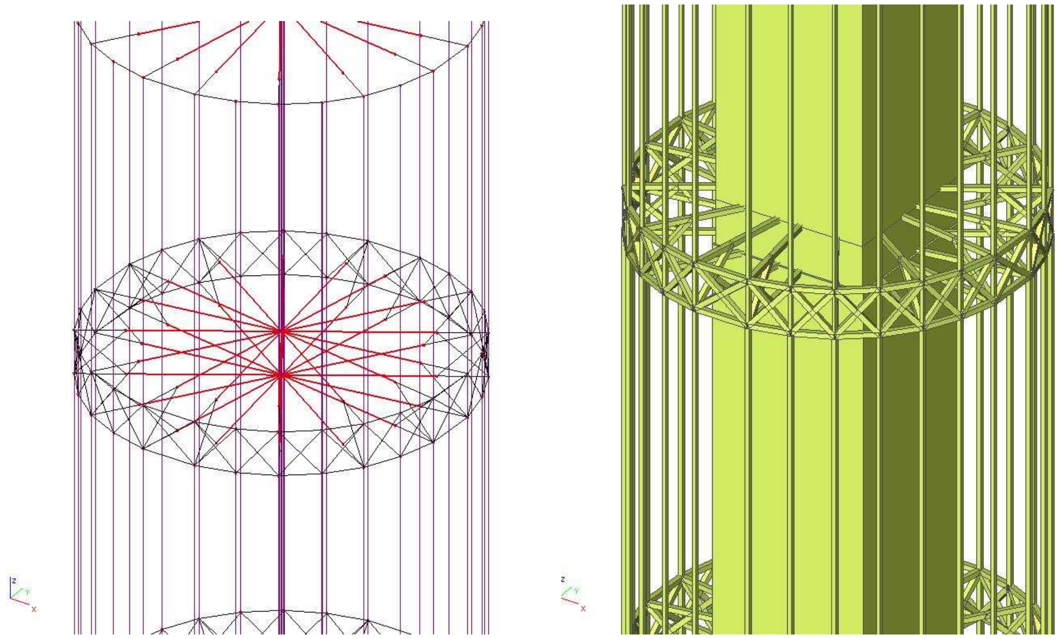


Figure 4-11 Supports FEM model Core-outrigger

For the megacolumns, hinges at the end of each section are added to ensure that the megcaolumns do not transfer shear forces and moments.

Since the core is modelled as massive rectangular element with a smaller internal lever arm the distance between the perimeter columns and the core increases. In order to keep the same distance between the columns and the tower of 12 meter beams with an infinite bending stiffness are added. These structural elements connect the outrigger to the core in the model are given an infinite bending stiffness using the master-slave method option in ESA.

Load cases

LC1: Dead Load; Self-weight
- 13207 kN/m^1 on the core
- 1469.1 kN/m^1 on the columns + self-weight of the columns added as pointloads

LC2: Live Load; Imposed loads
- 1794.1 kN/m^1 on the core
- 768.9/28 kN/m^1 on the columns

LC3: Live Load; Wind according to paragraph 3.2.1

For more details see appendix E

Load combinations
See paragraph 4.2.1

Results

Deformation	1^{th} order	2^{nd} order	n/n-1
ULS	567	598	1.05
SLS	378	411	1.09

Table 4-22 Maximum deformation alternative 2

Max moment	1^{th} order (kN/m)	2^{nd} order (kN/m)	n/n-1
ULS	$52.1 * 10^6$	$54.4*10^6$	1.04
SLS	$34.7 * 10^6$	$36.5*10^6$	1.05

Table 4-23 Maximum bending moment alternative 2

Shear moment	1^{th} order (kN)	2^{nd} order (kN)
ULS	141895	143652
SLS	94597	96017

Table 4-24 Maximum shear force alternative 2

The maximum deformation at the top of the building including the 2^{nd} order effect is 411 mm or h/1946.This within the limit of 800 mm or h/1000 and it can be concluded that the core outrigger alternative satisfies the drift limit. The core however has been assumed to be fully clamped. In reality this constraint should be modeled as a rotational spring and the extra deformation due to the effect of a rotational spring has yet to be taken into account.

The moment is caused by the lateral wind loads working on the building. At the outrigger levels a counteracting moment caused by the normal forces in the megacolumns (see Figure 4-12 and Figure 4-13) reduces the total moment working on the tower.

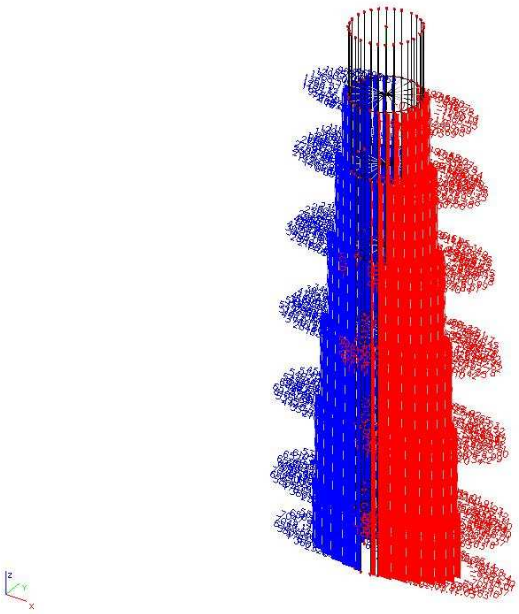


Figure 4-12 Normal forces in the megacolumns blue (tension) red (compression)

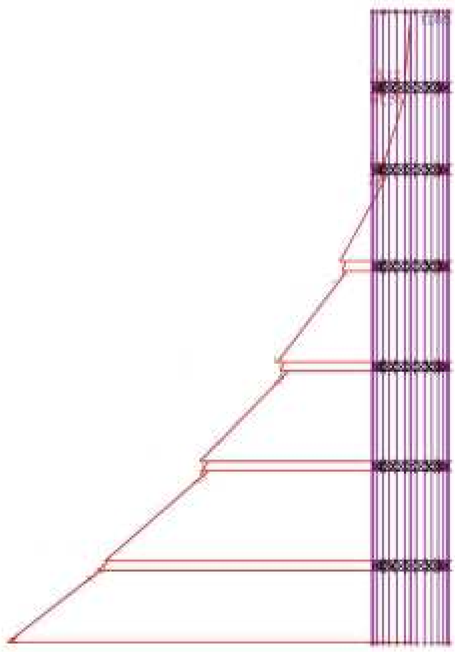


Figure 4-13 Moment diagram

The maximum moment at the base is **54.4 kNm*10^6**. In the alternative where the lateral load is only resisted by the core the max moment is **70.4 *10^6 kNm**. This means that having the megacolumns participate in the transfer of lateral loads, by connecting them to the core with outriggers and belttrusses, causes a **23 %** reduction of the maximum base moment. These results confirm the theory from the literature study (see Figure 4-14). As seen in the figure above, the total moment is reduced due to a counteracting moment created by the megacolumns.

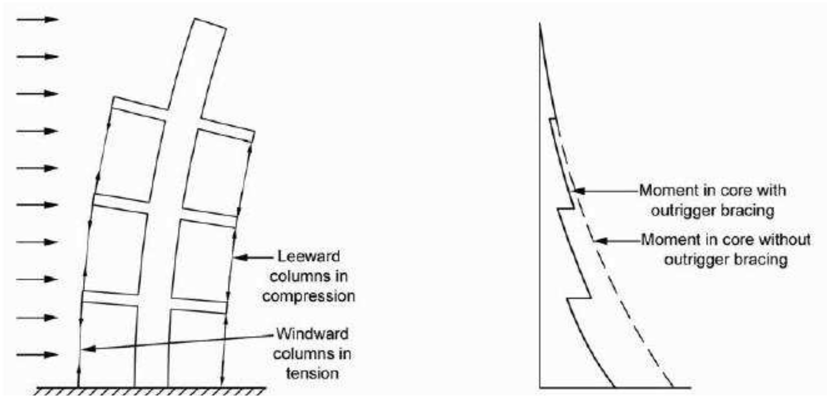


Figure 4-14 Core-outrigger theory

The slenderness ratio of the alternative "core-outrigger" is 100:801.9 ≈1:8. This value is in the range of the recommended slenderness ratio of 1:6 to 1:8. Having the perimeter megacolumns participate in the transfer of lateral loads clearly improves the structural behavior of the tower as the maximum moment is reduced by **23 %** and the deformation by **37%**. The deformation satisfies the drift limit of h/1000 which is in this case 800 mm but it should be noted that the second order effect and the effect of a rotational spring instead of a clamped core has not been taken into account.

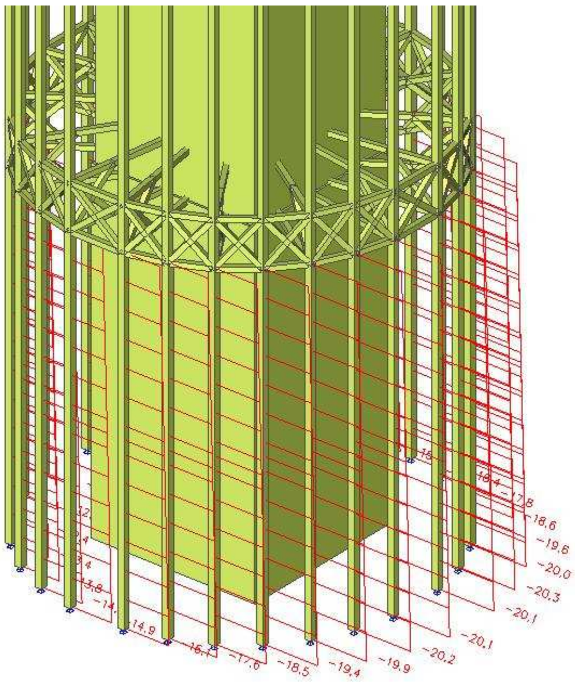


Figure 4-15 Compression in the megacolumns

Figure 4-15 shows the compression in the megacolumns for the building which is subjected to lateral loads and vertical loads. There is no tension in the megacolumns and core so an unreduced value for the elasticity modulus can be used

4.3.3 Alternative 3: Tube structure

Structural elements

Perimeter Tube

Tube in tube system

In this alternative both the tube and the core provide stability and stiffness to the building and transfer vertical lateral forces (wind) to foundation.
A tube-in-tube structure the perimeter tube possesses a large bending stiffness which is used to transfer the bending moment caused by wind to the foundation. The core is used to transfer shear forces caused by the wind loads. In order to have both the tube and the core working together a connection is needed. This connection is achieved by using the floors to couple the tube to the core.

Diagrid dimensions

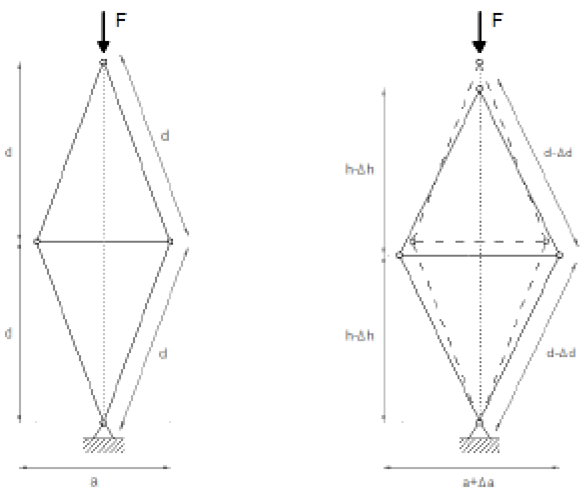


Figure 4-16 Dimensions diagrid

h (m)	a (m)
11.1	10.58

Table 4-25 Dimensions diagrid

Core

The core has the following functions:

- Transferring vertical forces
- Mitigating shear deformation

For the core the same dimensions as paragraph 4.3.1 are used.
The bending stiffness EI of the subdivisions have been calculated using a spreadsheet

Subdivision	Core EI (N/mm^4)	Wall thickness (mm)
Floor 0-22	1,3639E+22	1000
Floor 21-54	1,2287E+22	900
Floor 55-88	1,1494E+22	850
Floor 89-122	1,0813E+22	800
Floor 123-155	1,0134E+22	750
Floor 156-183	9,0807E+21	700
Floor 184-208	8,4314E+21	650

Table 4-26 Bending stiffness ESA alternative 3

ESA Model

Cross-section core

Subdivision	Height (m)	A (m^2)
EI1	0-92.1	43916x43916
EI2	92.1-218.8	42785x42785
EI3	218.8-345.5	42078x42078
EI4	345.5-472.2	41440x41440
EI5	472.2-595.2	40733x40733
EI6	595.2-699.7	39670x39670
EI7	699.7-801.9	38941x38941

Table 4-27 Cross-section core ESA model alternative 3

Cross-section diagrid column

Subdivision	Height (m)	D= (mm)	t= mm
EI1	0-92.1	1500	150
EI2	92.1-218.8	1500	150
EI3	218.8-345.5	1500	150
EI4	345.5-472.2	1500	150
EI5	472.2-595.2	1500	150
EI6	595.2-699.7	1500	150
EI7	699.7-801.9	1500	150

Table 4-28 Cross-section columns ESA model alternative 3

Constraints and connections

All the columns have hinged supports and the core has a fixed support at (0,0,0) (Figure 4-17)

The connection between the perimeter tube and the core is modelled using several hinged bars which represent the floor.

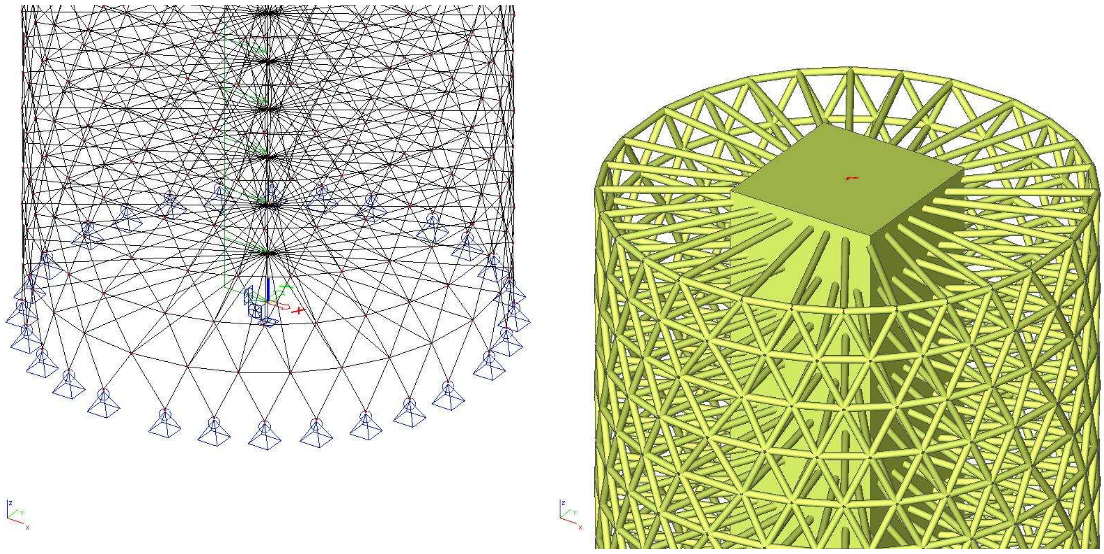


Figure 4-17 Supports FEM model diagrid

Load cases

LC1: Dead Load; Self-weight
- 13207 kN/m^1 on the core
- 1469 kN/m^1 on the columns + Self-weight of the columns added as pointloads

LC2: Live Load; Imposed loads
- 1794,1 kN/m^1 on the core
- 768,9/28 kN/m^1 on the columns

LC3: Live Load; Wind according to paragraph 4.2.1

For more details see appendix E:

Load combinations

See paragraph 4.2.1.

Results

Deformation	1th order (mm)	2nd order (mm)	n/n-1
ULS	481	505	1.07
SLS	320	339	1.04

Table 4-29 Maximum deformation alternative 2

Max moment	1th order (kNm)	2nd order (kNm)	n/n-1
ULS	$42.5 \cdot 10^6$	$47.0 \cdot 10^6$	1.10
SLS	$30.2 \cdot 10^6$	$31.5 \cdot 10^6$	1.04

Table 4-30 Maximum bending moment alternative 2

Shear moment	1th order (kN)	2nd order (kN)
ULS	147065	148689
SLS	98041	99347

Table 4-31 Maximum shear force alternative 2

The maximum displacement at the top of the structure is 339 mm or h/2360. Therefore the maximum deformation is within the limit of h/1000 or 800 mm and is reduced by 48% compared to alternative 1.

Just like the core-outrigger alternative the internal lever arm of the tower reaches from façade to façade. The diagrid however acts as a stiff perimeter tube which can transfer lateral loads and reduces the maximum moment at the base by 33 %. The moment diagram shows that unlike alternative 2 the connection to the core is almost continuous.

4.3.4 Comfort (vibrations in the along-wind direction)

The building has to comply with several comfort demands in the serviceability state, the first one being de maximum deflection of the building and the second one being the demand that the accelerations are kept beneath a certain value. In a tall building not the motion itself but the acceleration causes discomfort for its occupants. This is similar to how a person in a car feels nothing at a constant speed but does feel something when the car accelerates or decelerates.

Natural frequency of the alternatives.

The natural frequencies of the alternatives have been calculated by ESA and are given in Table 4-32.

Alternative	Natural frequency f [Hz]	T [sec]
Core	0.048	20.74
Core-outrigger	0.055	18.30
Diagrid	0.054	18.65
Eurocode 46/H	0.057	17.86
N*0,1sec	0.050	20.00

Table 4-32 natural frequency of the alternative

According to [46]

For a 50-story building, fb is typically about 0.2 Hz, corresponding to a period of 5 seconds. For a 100-story building, fb is in the range of 0.1– 0.125 Hz, corresponding to a period of 8–10 seconds, but some super-tall structures have been conceived for which the frequency is as low as 0.05 Hz, corresponding to a 20-second period.

The values from ESA correspond well with the estimations found in literature and the eurocode. The eurocode gives the formula fe= 46/H to estimate the natural frequency of a tall building for our building with a height of 801.9 meter this formula gives a frequency 0.057Hz Another estimate which is used in North-America is N*0.1 in which N is the number of storeys.

$$T = N \cdot 0{,}1$$

$$f = \frac{1}{T} \qquad (3)$$

This gives t=20 sec and a frequency of 0.050 Hz.

RMS and Peak acceleration

In the international community the negative experience due to wind-induced motion is tested using two criteria for acceleration, namely:

- Peak acceleration
- RMS acceleration (root mean square)

In the RMS method it is assumed that the negative experience due to the buildings movement is the result of sustained or ongoing motion which is described by an average effect over a period of time. The peak acceleration assumes that the negative experience due to the buildings movement is the result of large events (peaks). The RMS index is often favored due to its easy measurability and predictability. It offers a more accurate means of combining response in different directions based on their respective correlations. Advocates of peak acceleration argue that the peak resultant accelerations are difficult to estimate using RMS criteria (Isyumov 1993).
In the Netherlands there are two regulations which can be used for the comfort criterion, namely: NEN 6702 and ISO 6897.

NEN 6702 shows the limiting peak in a graph with two curves. Curve 1 applies to floors with industrial, office or educational function. Curve 2 applies to floors with a residential, gathering, health care, hotel sport or commercial function. This standard uses the peak acceleration as the limiting criteria.

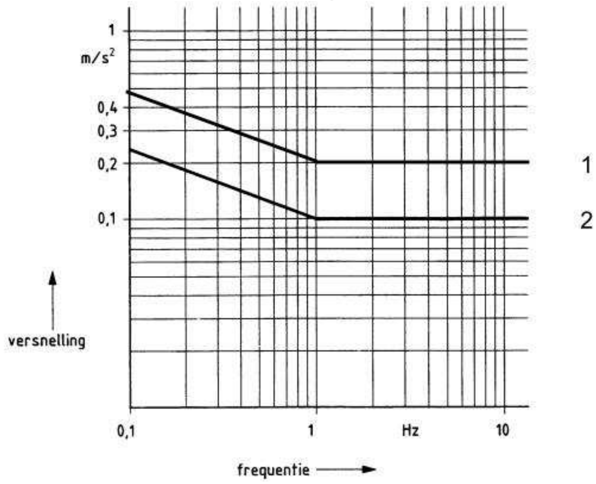


Figure 4-18 Peak acceleration according to NEN 6702

ISO 6897 is the international standard which uses the RMS index as the limiting criteria. Acceleration limits are given for natural frequencies between 0,063 and 1 Hz see table.

A relation between the RMS and peak acceleration is given by the peak factor g_{ap} found in [58]

$$g_p = \sqrt{2\ln(f_e T)} + \frac{1}{\sqrt{3}} \cdot \frac{1}{\sqrt{2\ln(f_e T)}} \tag{4}$$

Where

Fe eigenfrequency in Hz
T time in s

By multiplying the ISO 6897 we can convert acceleration RMS to the peak acceleration values (see Table 4-33).This allows us to compare the NEN 6702 to the ISO 6897 (Table 4-34).

Eigenfrequency **[Hz]**	**Acceleration r.m.s.** **[m/s^2]**	**Acceleration peak** **[m/s^2]**
0,063	0,0815	0,237
0,080	0,0735	0,220
0,100	0,0670	0,205
0,125	0,0610	0,191
0,160	0,0550	0,177
0,200	0,0500	0,164
0,250	0,0460	0,154
0,315	0,0418	0,143
0,400	0,0379	0,132
0,500	0,0345	0,122
0,630	0,0315	0,114
0,800	0,0285	0,105
1,000	0,0260	0,097

Table 4-33 Acceleration grenzen ISO 6897

Eigenfrequency **[Hz]**	**ISO 6987** **Acceleration peak** **[m/s^2]**	**NEN 6702** **Acceleration peak** **[m/s^2]**
0,063	0,237	-----
0,080	0,220	-----
0,100	0,205	0,24
0,125	0,191	0,22
0,160	0,177	0,2
0,200	0,164	0,18
0,250	0,154	0,175
0,315	0,143	0,16
0,400	0,132	0,15
0,500	0,122	0,14
0,630	0,114	0,125
0,800	0,105	0,115
1,000	0,097	0,1

Table 4-34 Comparison ISO 6897 and NEN 6702

Table shows that the values given in the ISO 6897 are stricter than NEN 6702. Therefore we will use the ISO 6897 as the limiting criteria for the buildings comfort.
[42][50][51][56]

peak acceleration according to NEN 6702

A calculation of the buildings acceleration is usually done by testing a scale model in a wind tunnel. Since it not possible to use a wind tunnel due to limited time and resources we will use a simple calculation method which is described in NEN 6702.

$$\alpha = 1,6 \cdot \frac{\left(\rho_2 \cdot p_{w1} \cdot C_t \cdot b_m\right)}{\rho_l} \tag{5}$$

Where

ρ_2: factor dependant on the eigenfrequency and damping of the building
$P_{w,1}$: variation in thrust on the building in N/m
C_t: summation of the wind factors for thrust and suction 1.2
b_m: the average width of the building
ρ_l: mass of the building per metre building height

$P_{w,1}$ is given by

$$p_{w1} = 100 \cdot Ln(\frac{h}{0,2}) \tag{6}$$

H height of the building

ρ_2 is given by equation below

$$\rho_2 = \sqrt{\frac{0,0344 \cdot f_e^{-2/3}}{D(1+0,12 \cdot f_e \cdot h)(1+0,2 \cdot f_e \cdot b_m)}} \tag{7}$$

Where

f_e eigenfrequency of the building in Hz
D damping factor
H height of the building
b_m average width of the building

To calculate the natural frequency of the building NEN 6702 gives the following formula

$$f_e = \sqrt{\frac{a}{\delta}} \tag{8}$$

a = value dependant on the distribution of the mass of the building 0.384 m/s^2. Since we have already calculated f_e in ESA we will use the values from Table 4-32.

NEN 6702	Alternative 1: Core	Alternative 2: Core-outrigger	Alternative 3: Diagrid
a (m/s^2)	**0,104**	**0,092**	**0,098**
f_e	0,048	0,055	0,050
D	0,016	0,016	0,013
ρ_2	1,22	1,07	1,29
$p_{w,1}$	829,6	829,6	829,6
C_t	1,2	1,2	1,2
b_m (m)	100	100	100
ρ_l (kg/m)	1854900	1854900	1971200

Table 4-35 Along-wind accelerations according to NEN 6702

The accelerations of all the alternatives are within the limits found in Table 4-33.

.

4.3.5 Comparison along-wind behavior of the alternatives

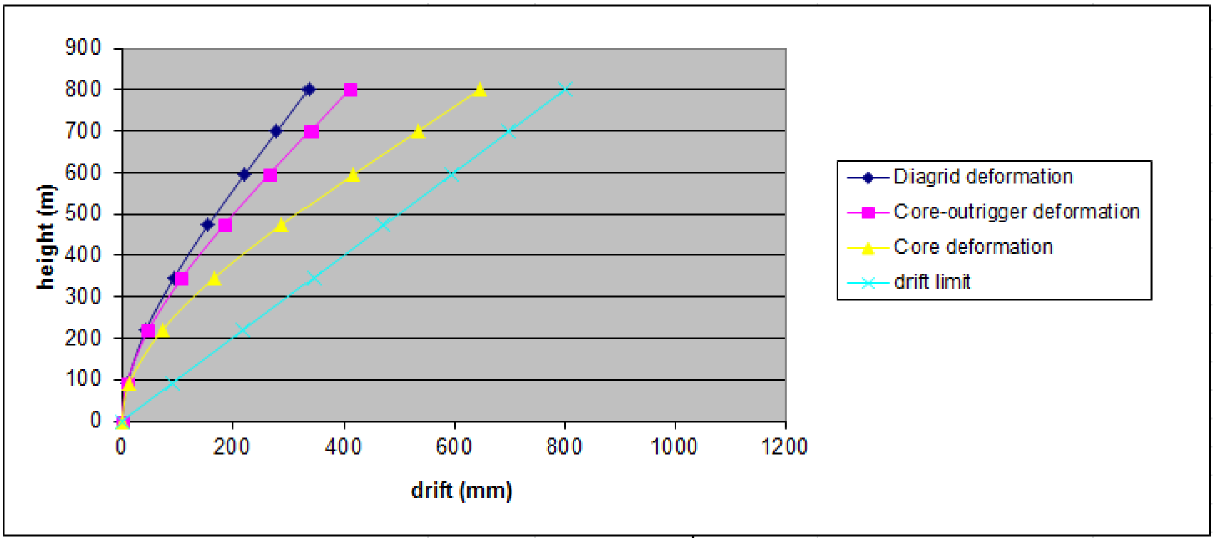
Alternative	Core	Core-outrigger	Diagrid
Deformation (mm)	648	411	339
Moment (kNm)	$70.4*10^6$	$54.4*10^6$	$47.0*10^6$
Weight perimeter (kN/m)	1219.9	1219.9	2383.3
Acceleration (m/s^2)	0.104	0.092	0.098

Table 4-36 Comparison forces and deformation

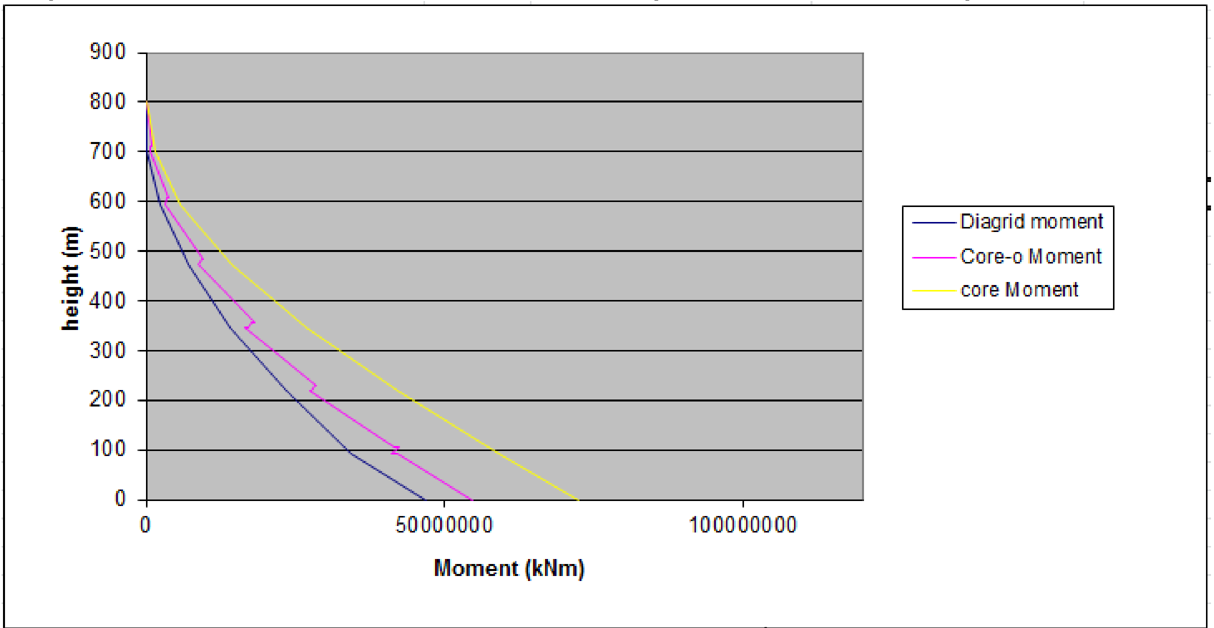
The dimensions of the core are the same for all three alternatives so the difference between alternative 1 and the others is that by allowing the perimeter to participate in the lateral load transfer a significant reduction in deformation and the base moment is achieved. The reduction in deformation is 35% and 38% for the core-outrigger and the tube alternative respectively and the reduction of the maximum moment at the base is 22% and 33% for the core-outrigger and the tube alternative respectively.
Both the core-outrigger and the tube alternative satisfy the drift limit of h/1000 as seen in Graph 4-2. However, the perimeter of the core-outrigger and the tube alternative do not have the same mass. The perimeter of the diagrid is circa twice as heavy as the core outrigger alternative.

Note that in the three alternatives the dimensions of the core have been kept the same. This is done in order to make a good comparison between the different alternatives. It is possible to optimize both the core-outrigger and the tube alternative by decreasing the dimensions of the core which results in a smaller load on the foundation.

The results show that the buildings has enough stiffness to deal with the lateral loads in the along wind direction. The design of a supertall however is usually governed by wind-induced across-wind vibrations. In the following chapter we will examine the behaviour of the alternatives with respect to vortex shedding.



Graph 4-2 Deformation of the structural alternative (2nd order effect included)



Graph 4-3 Moment Diagram of the structural alternatives (2nd order effect included)

4.4 Across-wind behavior

Introduction

Wind plays a dominant role in the structural design of a high-rise structure. Comfort (i.e. acceleration and vibrations) and vortex shedding are important and often governing design aspects.

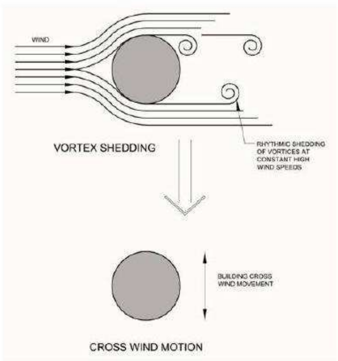


Figure 4-19 Vortex shedding

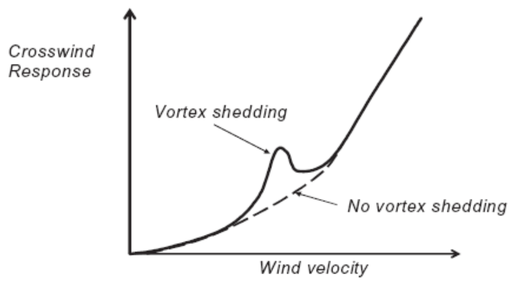


Figure 4-20 Across-wind Response

In chapter 3.5 of the literature study the importance of aerodynamic design at an early stage was discussed. Several methods such as changing the mass stiffness damping and shape of the building which can be used to influence the buildings dynamic behavior were mentioned such as changing the mass stiffness damping and shape of the building.

Changing the mass and stiffness can be very costly if significant improvements are needed. Also they can have adverse effects such as an increase of the jerk component.

Damping is achieved by adding one or two mass dampers or several energy dissipating devices throughout the building.
Our goal is however to design the building so that damping is not necessary. This is done by considering aerodynamics early in the design of the building. Out of the different beneficial aerodynamic shapes which are available to reduce vortex shedding (tapering, twisting, openings and corner cuts) a compound structure which includes slots was chosen.
The slots allow the wind to blow through the building which disrupts the vortices resulting in a reduction of lateral loads.

In this chapter the alternatives introduced in chapter 3.6 will be tested with respect to vibrations, acceleration and the forces created by the phenomenon vortex shedding. Normally wind tunnel research is used in order to calculate the reduction of the vortex shedding forces. However because of limited time and resources this is not an option. In order to take into account the reduction of the across-wind vibration due to the addition of slots we will firstly calculate the vortex shedding as if the building where a conventional solid structure. After this we apply a reduction which is based on experiments done in actual wind tunnels. A more detailed description of these experiments can be found in appendix A.

4.4.1 Across-wind vibration

The complex nature of the across wind loading which results from an interaction of incident turbulence, unsteady wake effects and building motion makes predicting the across wind vibrations of a tall building very difficult. For this reason the across-wind accelerations are usually determined using a wind tunnel. The NBCC (National Building Code Canada) gives a formula based on a wide range of turbulent boundary layer studies which can be used to determine the peak acceleration at the top of supertall.

$$a_w = f_e^2 g_p \sqrt{WD}\left(\frac{a_r}{\rho_B g\sqrt{\beta_W}}\right) \qquad (9)$$

Where:

f_e = eigenfrequency of the building Hz
g_p= peak factor
W = the average width of the building in m
D = the average Depth of the building in m
ρ= average density in kg/m^3
g =acceleration due to gravity
β=the structural damping (See Appendix B)

$$a_r = 78{,}5\cdot 10^{-3}\left(\frac{V_H}{n_w\sqrt{WD}}\right)^{3.3} \qquad (10)$$

Where:

f_e = eigenfrequency of the building in Hz
b_m= the average width of the building in m
v_h = the mean wind speed at the top of the building

The across-wind accelerations of the 3 alternatives according to the NBCC are given in table Table 4-37.

Canada across-wind	Core	Core-outrigger	Diagrid
a_w (m/s^2)	0.43	0.37	0.35
g_p	2.82	2.86	2.85
W (m)	100	100	100
D (m)	100	100	100
ρ (kg/m^3)	236.20	236.20	251.00
g	10	10	10
β	0.05	0.05	0.04

Table 4-37 Across-wind accelerations according to NBCC

The result show that the across wind acceleration are much larger than the along-wind accelerations.

According to [35] The NBCC method for calculating the across wind acceleration was tested using results obtained from wind tunnel studies employing aero elastic and high frequency force-balance modeling techniques of 48 buildings.
The results showed that for most buildings the NBCC was likely to significantly overestimate the actual measured accelerations, sometimes by a factor two or more.

4.4.2 Vortex shedding

A principal feature of bluff bodies is that they create separated flow regions which become the source of vortex shedding. This phenomenon is discussed in the literature study, part 1 paragraph 3.5.2.

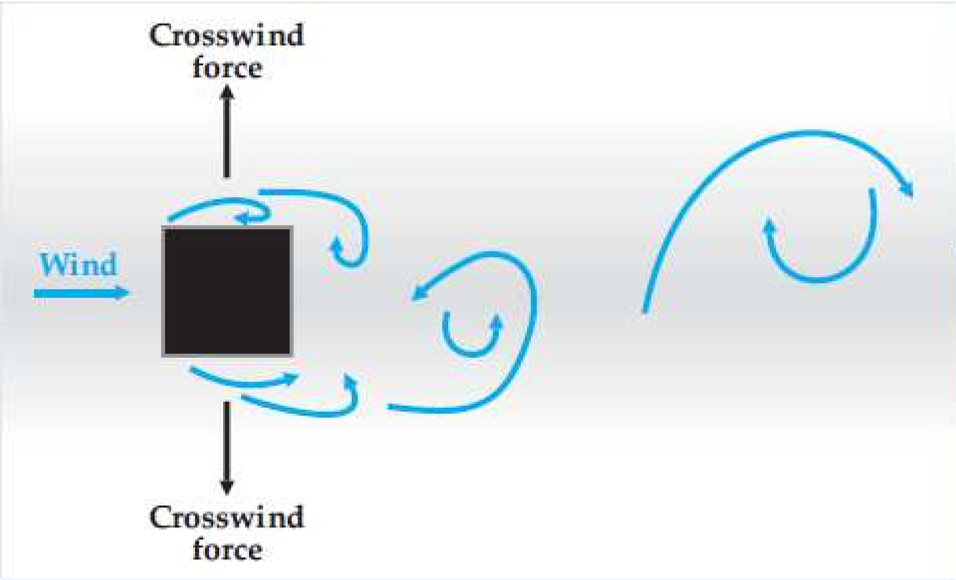


Figure 4-21 vortex shedding

Very often the highest overall wind loading on a tall slender building is due to the dynamic response the results of the across-wind vortex excitation. The motions caused by vortex shedding may cause discomfort to t occupants and it becomes a major concern of the structural designer and architect as to how they can keep these motions to within acceptable limits.

The natural frequencies of the structural alternatives are given in Table 4-32.

With the help of the formulas given in [36] we can determine the shear force and the bending moment due to vortex shedding.

Str=0.18
B = 100
D = (damping) = 0,016 for alternative 1 and 2 and 0,013 for alternative 3

The critical wind velocity is given by expression

$$v_{kr} = \frac{fe \cdot b}{Str} \qquad (11)$$

In case of a forced vibration (across wind) with a frequency equal to the towers natural frequency, the static equivalent across force is given by.

$$q_{eq} = C_L \cdot \frac{1}{2} \cdot 1,25 \cdot v^2{}_{kr} \cdot b \cdot \frac{1}{2 \cdot D} \tag{12}$$

$C_L =$ Depends on the Reynolds number Re

Usually Re> $4*10^5$ and thus vkr* b>5.7. In this case CL =0.2 [36] and q_{eq} can then be written as:

$$q_{eq} = \frac{v^2{}_{kr} \cdot b}{16 \cdot D} \tag{13}$$

In case of a clamped tower, it is mostly the load at the top of the structure which is responsible for the across-wind vibrations. For this reason the top 33 % of the wind load is used when calculating the Vortex shedding forces.

$$Q_{eq} = q_{eq} \cdot \frac{1}{3} \cdot h = \frac{v^2{}_{kr} \cdot b \cdot h}{50 \cdot D} \tag{14}$$

$$M_{eq} = Q_{eq} \cdot \frac{5}{6} \cdot h = \frac{v^2{}_{kr} \cdot b \cdot h^2}{60 \cdot D} \tag{15}$$

This approach is only valid for forced vibrations. When the drift becomes too large (more than 4 %), oscillations makes this approach invalid.
The across-wind vibrations can also be determined using a so called spectral analysis [36]:

$$Q_{eq} = 0,7 \cdot \frac{\frac{1}{2} \cdot \rho \cdot v^2{}_{kr} \cdot b \cdot h}{\sqrt{D}} \tag{16}$$

	Alternative 1	Alternative 2	Alternative 3
f_e (1/s)	0.048	0.0546	0.0536
v_{kr} (m/s)	26.67	30.33	29.78
q_{eq} (kN/m)	277.78	359.4	426.3
Q_{eq} (1) (kN)	71280	92230	109393
Q_{eq} (2) (kN)	197321	255199	272842
M_{eq} (1) (kNm)	$47.6*10^6$	$61.6*10^6$	$73.1*10^6$
M_{eq} (2) (kNm)	$131.8*10^6$	$170.5*10^6$	$182.3*10^6$

Table 4-38 vortex shedding

The formulas show that increasing the damping factor and decreasing the critical wind speed are very efficient ways to reduce the vortex shedding. A lower critical wind speed can be achieved by adjusting the strouhal number in such a way that the vortices are shed faster or slower. This can be achieved by changing the roughness of the facade of the building.
The critical wind speed can be lowered by increasing the stiffness and mass of the structure. However as mentioned in the introduction of this chapter these adjustments can be very costly.

The problem with the forces due to vortex shedding is not just their size but the fact that they are dynamic forces.

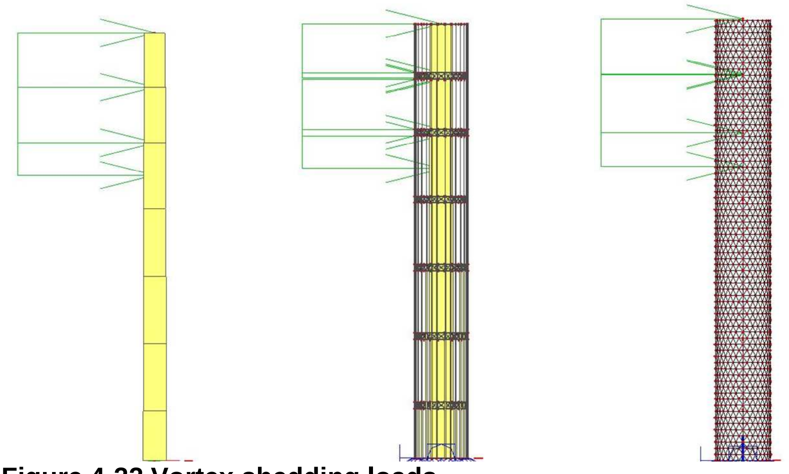


Figure 4-22 Vortex shedding loads

Table 4-37 shows that if no measures are taken, the 3 structural alternatives are subjected to considerable vortex shedding forces. In case of alternative 2 and 3 these forces are 3 to 4 times as large as the maximum base moments found in the static calculation.

	Alternative 1	Alternative 2	Alternative 3
M_{eq} (2) (kNm)	$131.8*10^6$	$170.5*10^6$	$182.3*10^6$
M_{base} (kNm)	$70.4*10^6$	$54.4*10^6$	$47.0*10^6$
q (kN/m)	738	955	1021
U (mm)	2097	1849	2014

Table 4-39 Unreduced vortex shedding forces and deformation.

Table 4-40 shows that the critical wind speeds are lower than the extreme hourly wind speed given in [63]

T=100 gives v=28.1. Therefore the occurrence of the vortex shedding is almost certain. However, if the critical wind speed is increased than the vortex shedding forces will occur less often but will also be bigger. In this case a choice has to be made between smaller and more often occurring forces and larger and less often occurring forces.

Reference period	T=12,5	T=25	T=50	T=100	T=200
Extreme hourly averaged wind speed	23.7	25.5	26.8	28.1	29.2

Table 4-40 Wind speeds

It should be noted that formula (15) is made for rectangular buildings. The forces due to vortex shedding can be very large and govern the design of the tower. The addition of slots decreases the Strouhal number and the wind flowing through the building disrupts the formation of vortex shedding.

4.4.3 Reduction of the across-wind induced vibration due to slots and openings

Looking at the results of chapter 4.3.4 and 4.3.5 and the previous paragraphs it is clear that the building is stiff enough in the along-wind direction and that the across-wind vibrations and forces govern the design of the tower. Earlier the forces and deflection due to vortex shedding were calculated for a conventional closed tower. The result was very large base moments and deflections which did not satisfy the drift limit 1/1000 H. The tower however uses openings or gaps to reduce the vortex shedding. The reductions which will be applied are based on experiments done on supertall buildings with openings found in [34][52][57] and [58].

For a detailed description of the experiments and the results see appendix A.

alternative	Reduction Deflection %	Reduction Base moment %
[58]H.Okada and Kong L	20-25	-
[52] Kikitsu H. , Okada H	45	-
[34] R Dutton and N Isyumov (1990),	73	58
[57] Miyashita K., et al (1993)	60	66

Table 4-41 Reduction across-wind forces and deformation

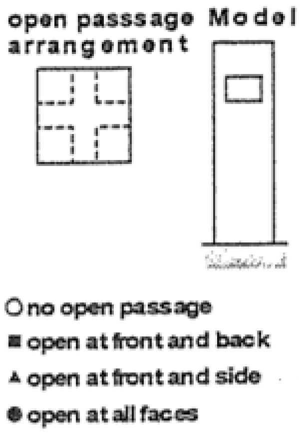
In [38] Okada and Long a horizontal slot at a single location was used. Therefore we will use the results from research [34] and [57] since the configuration of the gaps are similar to that of the Rijnhaven Tower (vertical slots along the height of the building) .

alternative	Alternative 1	Alternative 2	Alternative 3
M_{eq} (2) (kNm)	$131{,}8*10^6$	$170{,}5*10^6$	$182{,}3*10^6$
M_{red} [34] (kNm)	$55{,}4*10^6$	$71{,}6*10^6$	$76{,}6*10^6$
M_{red} [57] (kNm)	$44{,}8*10^6$	$57{,}9*10^6$	$61{,}9*10^6$
U (mm)	2097	1849	2014
U_{red} [34] (mm)	713	629	685
U_{red} [57] (mm)	839	740	806
a_w (m/s^2)	0.43	0.37	0.35
$a_{w;red}$ [34] (m/s^2)	0.18	0.16	0.15
$a_{w;red}$ [57] (m/s^2)	0.15	0.13	0.12

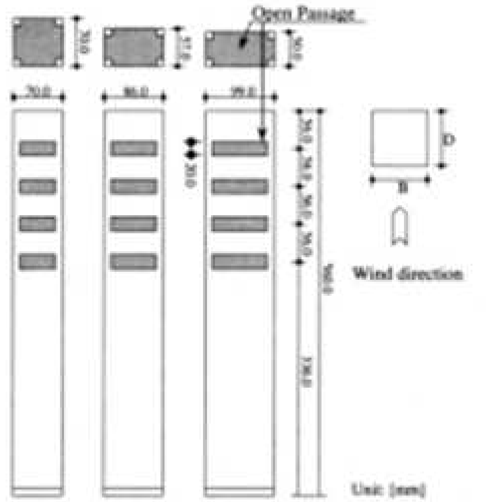
Table 4-42 Reduced values forces deflection and acceleration

The openings in the building result in a reduced across-wind deflection. Alternative 2 which is the core-outrigger satisfy the serviceability limit criteria h/1000 for both reductions.
It should be noted that all the experiments involve buildings with a square footprint.

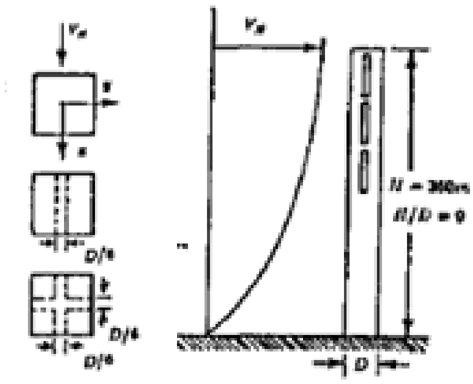
In the experiments there are no reduction values given for the across-wind acceleration. The accelerations are however related to the vortex shedding forces. If we apply the same reduction for the acceleration as we did for the forces we get values which are within the limits found in Table 4-34.



[58]



[52]



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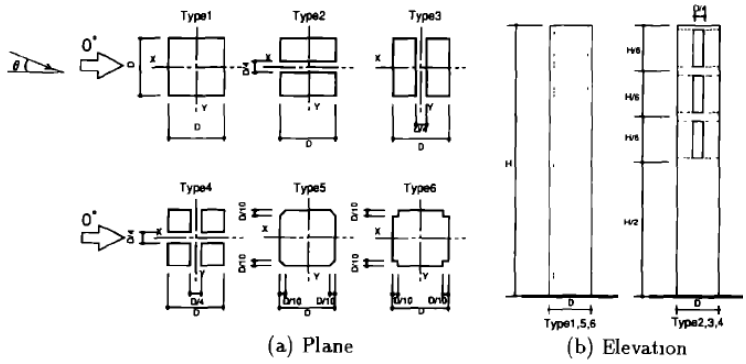


[57]

Figure 4-23 Setup for experiments with voids and slots

4.5 Choice load-bearing structure

In the previous paragraphs different alternatives for the load-bearing structure of the Rijnhaven Tower were examined. The results are shown in table 4-41.

Alternative	Core	Core-outrigger	Diagrid
Deformation (SLS) (mm)	648	411	339
Deformation vortex shedding	839	740	806
Base moment ULS (kNm)	$70.4*10^6$	$54.8*10^6$	$47.0*10^6$
Base moment Vortex shedding (kNm)	$55,4*10^6$	$71,6*10^6$	$76,6*10^6$
Along-wind acceleration NEN 6702 (m/s^2)	0.104	0.092	0.098
Across-wind acceleration NBCC (m/s^2)	0.18	0.16	0.15

Table 4-43 Comparison alternatives

Statics (deformation and Forces)

The deformation of the core alternative satisfies the drift limit of h/1000 and the increased lever arm of alternative 2 and 3 cause a significant reduction. Both alternative 2 and 3 satisfy the drift limit of h/1000. It should however be noted that the deformation due to the rotation of the foundation has not yet been taken into account. Therefore feedback is necessary when the design of the structural elements is known.
When compared to the alternative where only the core resist lateral loads the forces are also significantly reduced in the core-outrigger and tube structure alternative. By allowing the perimeter to participate in the transfer of lateral loads respectively a reduction of 23 and 33 % is achieved.

Along-wind and across-wind acceleration Dynamics (accelerations)

The accelerations shown in Table 4-34 are smaller than the limit of 0.237 m/s^2 at 0.063 Hz. If the frequency gets smaller the limit gets larger therefore all the alternatives satisfy the limit. However these accelerations have been calculated using simplified formulas found in NEN 6702 and the NBCC. It is recommended to determine the accelerations using wind tunnel testing.

Dynamics (vortex shedding)

With the wind speeds given in Table 4-40 the occurrence of the vortex shedding is almost certain. The vortex shedding forces have been determined with a simplified formula and need to be checked using a wind tunnel.

Architectural freedom

A steel diagrid causes architectural obstructions in the façade due to diagonals. However the diagonals can also be used to give the building a recognizable aesthetic image.
The core-outrigger alternative has no architectural obstructions in the façade which means more freedom for the architect. Therefore it depends on the wishes of the architect and client as both can be seen as a positive.

Self-weight of the structure

The cores of all three alternatives have the same dimensions. Therefore the only difference is the mass of the perimeter. The mass of the perimeter per meter (along the buildings height is more than twice as large as the core-outrigger alternative).

Alternative	(kN/m)
Core outrigger	1220
Tube structure	2383

Table 4-44 Weight perimeter alternatives

Erection process

Vertical pumping can be used to construct the concrete core and megacolumns. This method has proven to be very effective for supertalls.
A diagrid requires good planning and prefabrication. The nodes need to be assembled and prefabricated in the factory to minimize on site butt welding. Generally diagrid joints are more complicated than conventional orthogonal structures and therefore more expensive. Due to the triangular configuration rigid connections are not necessary and pinned connections can be used at joints /nodes.
Design and erecting a core-outrigger structure is much more practical and faster due to the ability to pump concrete up to great heights..

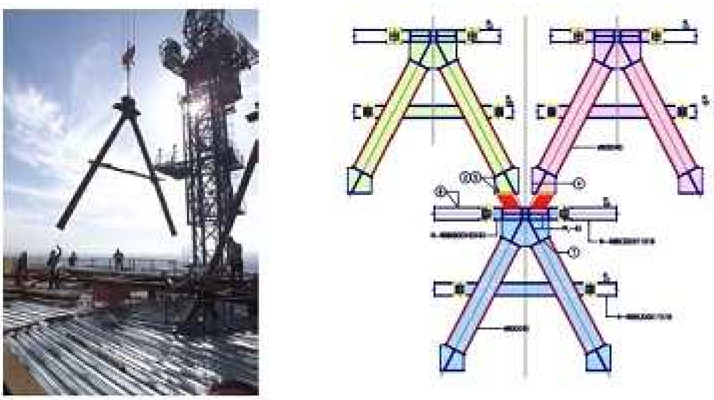


Figure 4-24 Diagrid construction

4.6 Conclusion

	Core	Core-outrigger	Diagrid
Along-wind drift	+-	+	+
Along-wind forces	+-	+	++
Along-wind accelerations	-	+	+
Across-wind drift	-	+	+
Across-wind forces	-	--	--
Across-wind accelerations	+-	+-	+-
Architectural freedom	+	+	--
Erection process	+	+	--
Self-weight	+-	+-	+-

Table 4-45 Comparison structural alternatives

Both the diagrid and core-outrigger are suitable alternatives. The diagrid performs slightly better if we look at the deformation and base moment. However, core-outrigger alternative clearly wins when the architectural freedom and erection process are considered and also has better dynamic behaviour due to the use of concrete. Because the diagrid does not structurally perform significantly better than the core-outrigger alternative, the alternative 2 which uses a core-outrigger system is chosen as towers superstructure.

Chapter 5 Foundation

5.1 Introduction

In the literature study the importance of a well-designed foundation was described.
For a building with a height of 800 meters large concentrated loads are expected. These can result in differential settlements which lead to unwanted stresses in structural members, cracking, an increase of the 2^{nd} order effect and a larger deflection at the top of the building.
In chapter a piled raft foundation was chosen because large concentrated loads were expected. This foundation system consists of a raft and piles. However unlike in a normal piled foundation the raft does not only distribute the loads but also transfers the forces from the superstructure to a soil layer with load-bearing capacity.
The interactions which take place between the pile raft and soil are:

- Transfer of loads from the superstructure to soil layers with better load bearing capacity through foundation piles via end bearing and skin friction.
- Mutual interaction between the foundation piles (group effect).
- Area load transfer from the raft to the subsoil.
- Increased axial stress on the pile jacket and hence skin friction as a result of surface pressure of the raft.

Modeling these interactions is quite complicated and requires FEM software. Due to limited time, designing a full pile and raft foundation is not possible. In order to get a feeling for the geotechnical possibilities a shallow foundation will be designed. For the shallow foundation the load-bearing capacity and the (differential settlements) will be determined.

In paragraph 5.2-5.4 the subsoil and position geometry of the foundation system is described.
In paragraph 5.5 the load-bearing capacity of the shallow foundation is determined using the soil layers given in appendix K and the equation given in (Brough, [70]).
In paragraph 5.6 the settlements are calculated according to NEN-EN 6740 2006 paragraph 13.5
In paragraph 5.7 a starting design and advantages of a piled raft foundation are given.

Finally, a conclusion is made based on the results of the shallow foundation and the expected improvements of a pile and raft foundation.

5.2 Soil profile

Determining the soil profile is one of the most important steps in the foundations design process. The soil profile in Rotterdam can generally be described as followed:

- Beneath a ca. 3 meter thick sand layer lies 14 meters of compressible soil layers.
- At -18 m NAP we find the first Pleistocene sand layer with a thickness of 15 meter. (Most of Rotterdam's high-rise buildings are founded on this layer).
- Beneath this sand layer we find the layer of Kedichem" consisting of a clay and sand.
- At circa -50 m NAP we find another Pleistocene sand layer.

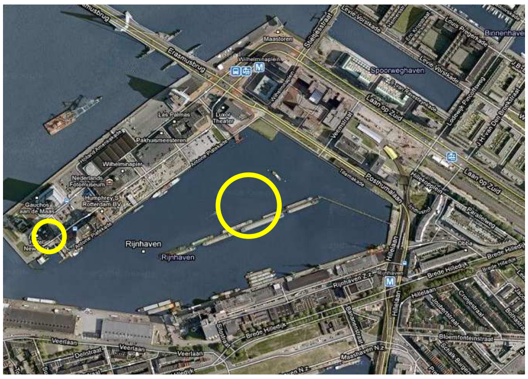


Figure 5-1 Location Montevideo (small circle) and Rijnhaven tower (large circle)

Because of limited time and resources it is not possible to know the exact soil condition of the soil beneath our tower. In order to get an indication of the subsoil beneath the tower we will use the results of a soil probing of a neighboring high-rise building "Montevideo" (see Appendix K). Figure 5-1 shows the position of the Montevideo and the Rijnhaven Tower.

The soil layers from Appendix K have been classified according to NEN EN 6743 and 6740 and are given in Table 5-1. Ground water is located at ca. -2 m NAP.

Layer	Depth NAP (m)	γ (kN/m^3)	Description
0	3.5 to -1	17	Sand / Clay
1	-1 to -2,5	17	Clay
2	-2.5 to -9.5	18	Sand
3	-9.5 to -11.5	10	Peat
4	-11.5 to -17	17	Clay
5	-17 to -34	19	Sand
6	-34 to -35	13	Peat
7	-35 to -38	20	Sand
8	-38 to -43	20	Clay
9	-43 to -47	20	Sand
10	-47 to -51	20	Peat / Clay
11	-51 to -53	20	Sand
12	-53 to -56	21	Clay / Loam
13	-56 to -63	20	Sand
14	-63 to -64	20	Peat / Clay
15	-64 to -71	20	Sand

Table 5-1 soil layer description

5.3 Design shallow foundation

The shallow foundation should rest on a soil layer with sufficient load-bearing capacity. Figure 5-2 shows the position of the raft, basement and soil layers.

The depth of the tower is 100 meter. Both the basement and raft have a circular shape with a diameter of 140 meter and a total area of 15394 m^2. The raft is 3 meters thick and basement is 21 meter deep.

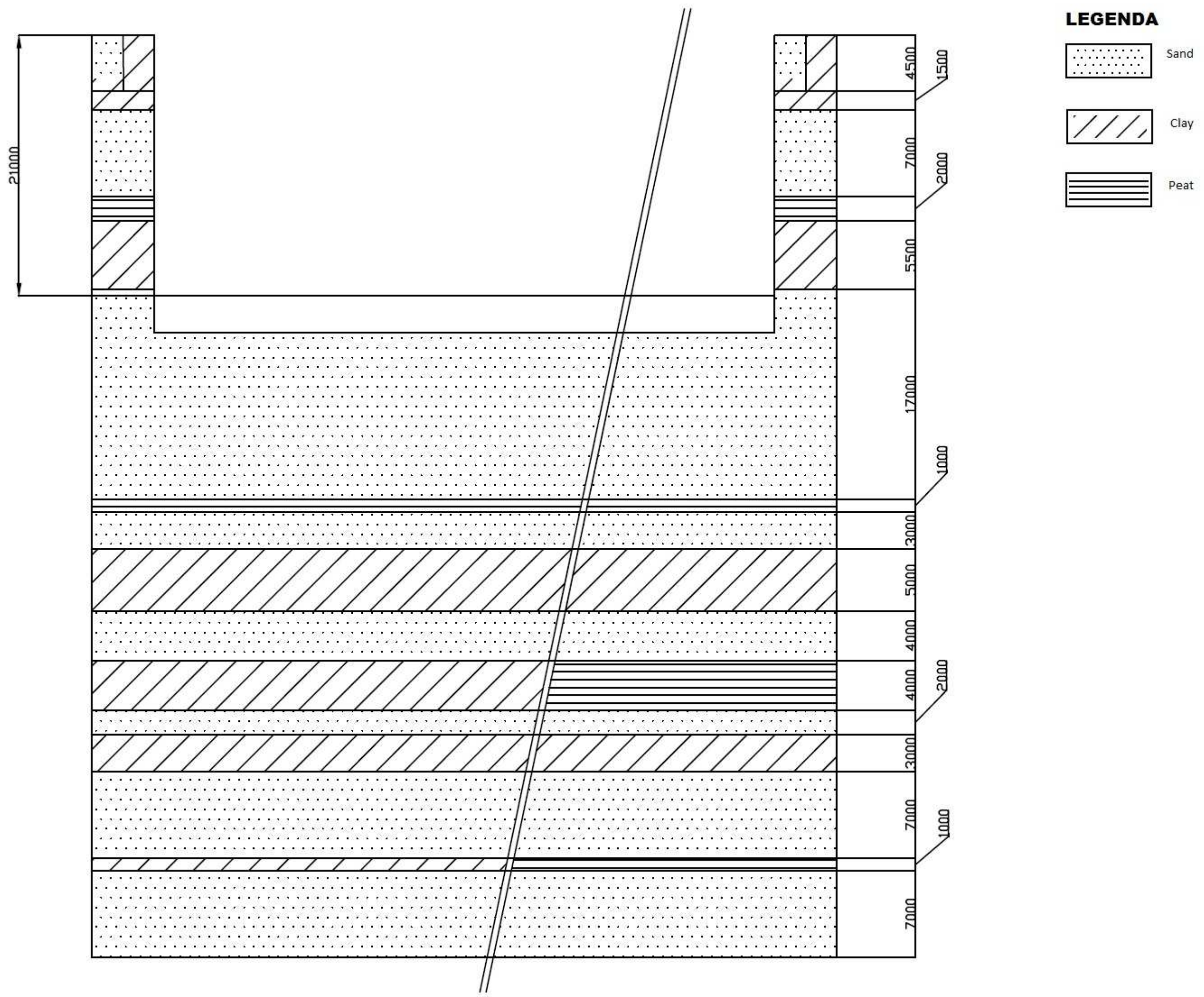


Figure 5-2 soil layers and raft

5.3.1 Basement

Like the tower, the basement and raft have a round shape. The diameter of the basement and the 3 meter thick raft is 140 meter.

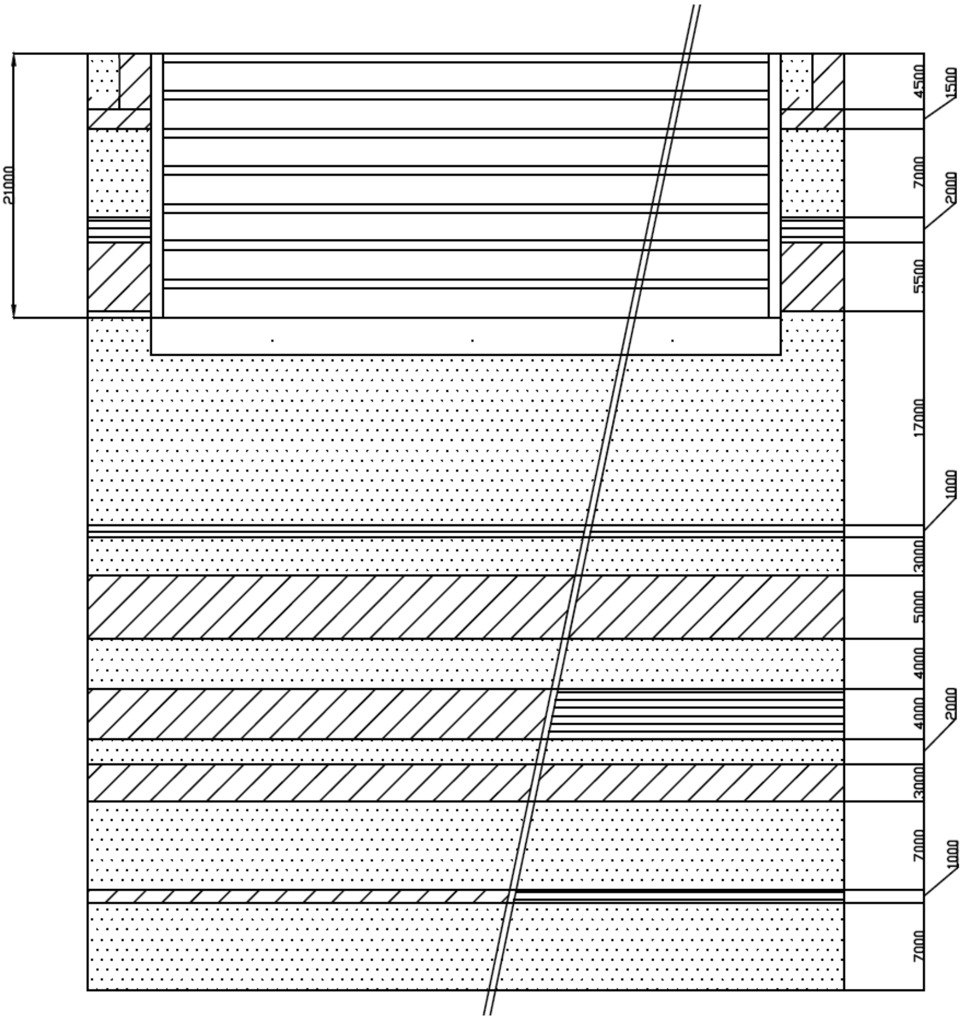


Figure 5-3 Basement Rijnhaven Tower

The basement functions are:

- parking for the tower inhabitants
- transferring the forces from the superstructure to the foundation raft

Starting points according to NEN 2443:

Width parking spot	: 2.50 m
Length parking spot	: 5.00 m
Width parking road	: 6.00 m
Load basement function	: 3.50 kN/m^2
Minimal free height	: 2.30 m
Maximum height vehicle	: 2.20 m

5.3.2 Structural design

The structural design of the basement is similar to design of the footprint of the superstructure however the composite floor-system: comflor 210 is replaced by TT-beams. The concrete TT-beams are used to cross a span of 20 meter and their dimensions have been determined using the tables and graph in appendix D. In the middle of the basement a span of 28 meter needs to be crossed.

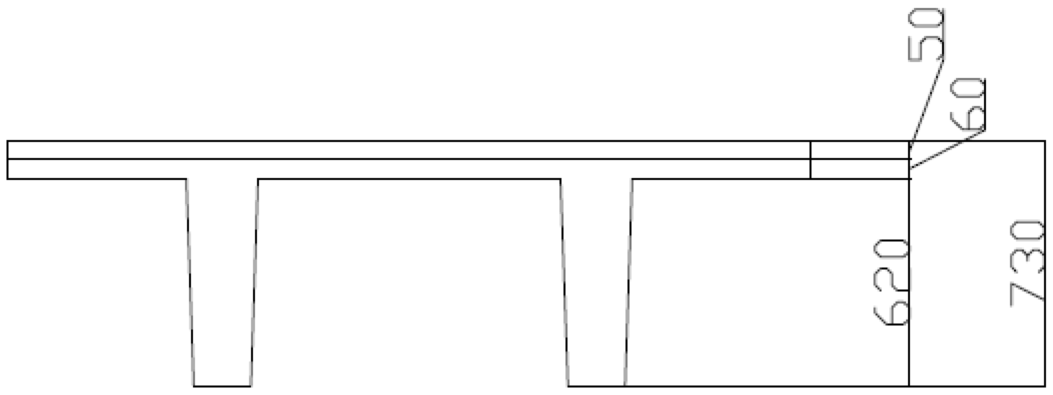


Figure 5-4 Dimensions TT Beam

The concrete walls from the super structure are extended throughout the basement and used to support the TT beams. These walls have a thickness of 1000 mm.

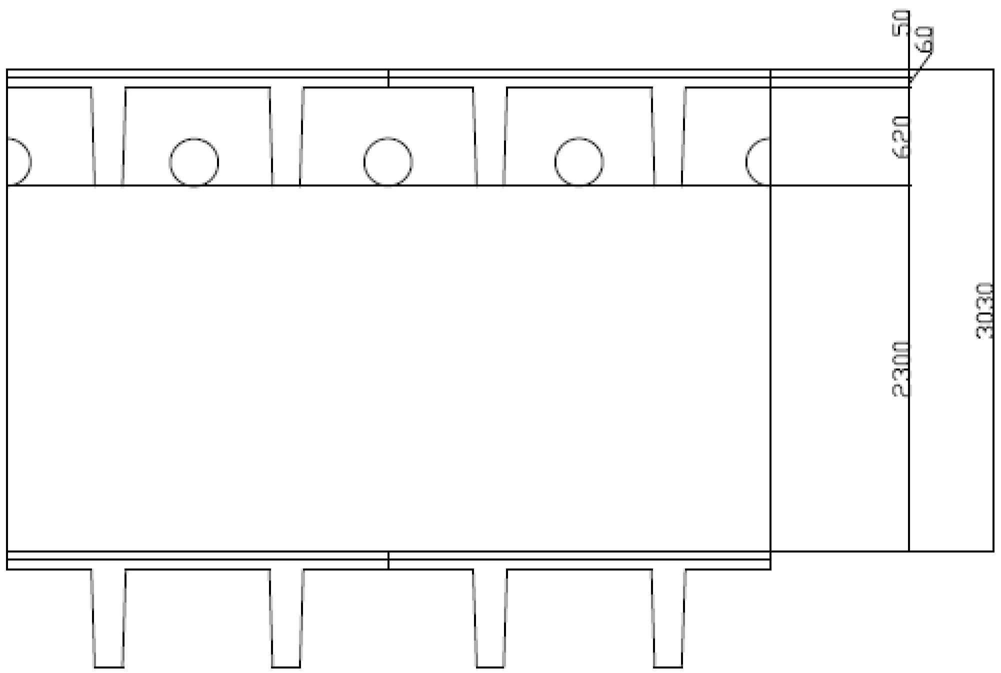


Figure 5-5 Floor-to-floor height Rijnhaven Tower

A complete overview of the structural elements and dimension of the basement is given in *Addendum: Rijnhaven Tower Basement.*

5.4 loads

The load on the foundation consists of:

- The vertical loads caused by the building self-weight.
- The vertical live loads cause by users, inhabitants and furniture.
- The vertical loads caused by wind loads.

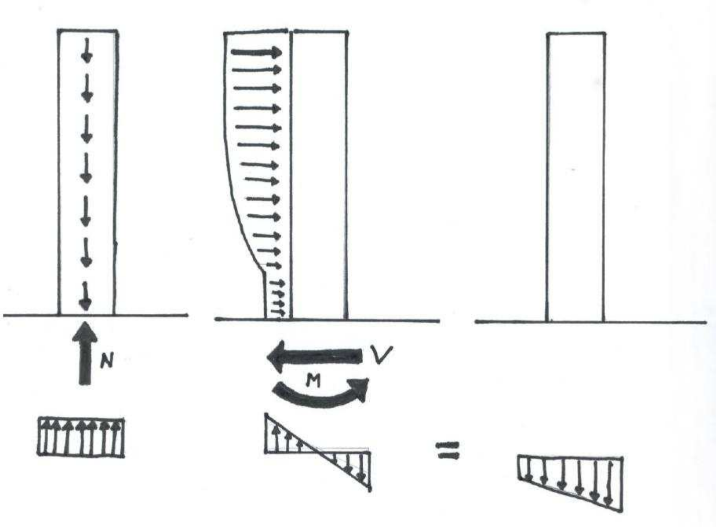


Figure 5-6 Loads working on the tower

5.4.1 Lateral loads (wind)

The wind load which acts on the building is determined using.

- NEN-EN 1991-1-4 and NEN-EN 1991-1-4/NB: 2007
- Convenanthoogbouw NTA Hoogbouw (03-A) table 03-A.1

$0 < Z < 100$	$q = 95$ kN/m
$100 < Z < 700$	$q = 95–140$ kN/m
$700 < Z < 800$	$q = 143$ kN/m

5.4.2 Vertical loads

The vertical loads from the superstructure are transferred to the foundation raft through the concrete walls of the foundation. These walls are an extension of the structural columns and core walls found in the footprint.
These loads have been calculated in appendix J and the maximum load in kN/m^2 on the raft is: 414+750.9+76.7=1241.6 kN/m^2.

5.5 Load bearing capacity of the foundation

In this paragraph we will only determine the load-bearing capacity of the raft. We assume that raft is subjected to a uniform load. Tilting and squeezing as a consequence of horizontal loads have not been taken into account.

According to NEN 6740 -6.2 the foundation is classified as GC3. The partial factor for the soil characteristics according to NEN 6740 are given in Table 5-2.

Factor		**Limit states**		
		1A/1B (ultimate)		**2 (serviceability)**
		Favourable	**Unfavourable**	
$\gamma_{m;g}$	Self-weight soil	1,1	1	1
$\gamma_{m;\varphi}$	Tangent friction angle	1,15	1	1
$\gamma_{m;c1}$	Cohesion	1,6	1	1
$\gamma_{m;cu}$	Undrained shear strength	1,35	1	1

Table 5-2 Partial factors

The soil layers characteristic are given in Table 5-1, Table 5-3 and Table 5-4.

Layer	**Depth NAP (m)**	**γ (kN/m^3)**	**γ_{sat} (kN/m^3)**	**φ**	**C_p**	**C_s**
0	3.5 to -1	17	19	30		
1	-1 to -2,5	17	17	17,5		
2	-2.5 to -9.5	18	20	32,5		
3	-9.5 to -11.5	10	10	15		
4	-11.5 to -17	17	17	17,5		
5	-17 to -34	19	21	35	1000	∞
6	-34 to -35	13	13	15	30	40
7	-35 to -38	20	22	40	1500	∞
8	-38 to -43	20	20	22,5	30	400
9	-43 to -47	20	22	40	1500	∞
10	-47 to -51	20	20	22,5	40	400
11	-51 to -53	20	22	40	1500	∞
12	-53 to -56	21	21	27,5	50	600
13	-56 to -63	18	20	32,5	450	∞
14	-63 to -64	20	20	22,5	50	600
15	-64 to -71	17	19	30	200	∞

Table 5-3 Representative soil properties

Layer	Depth NAP (m)	γ (kN/m^3)	γ_{sat} (kN/m^3)	φ	C_p	C_s
0	3.5 to -1	15,5	17,3	26,1		
1	-1 to -2,5	15,5	15,5	15,2		
2	-2.5 to -9.5	16,4	18,2	28,3		
3	-9.5 to -11.5	9,1	9,1	13,0		
4	-11.5 to -17	15,5	15,5	15,2		
5	-17 to -34	17,3	19,3	30,4	1000	∞
6	-34 to -35	11,8	11,8	13,0	30	400
7	-35 to -38	18,2	20	34,8	1500	∞
8	-38 to -43	18,2	18,2	19,6	30	400
9	-43 to -47	18,2	20	34,8	1500	∞
10	-47 to -51	18,2	18,2	19,6	40	400
11	-51 to -53	18,2	20	34,8	1500	∞
12	-53 to -56	19,1	19,1	23,9	50	600
13	-56 to -63	16,4	18,2	28,3	450	∞
14	-63 to -64	18,2	18,2	19,6	60	600
15	-64 to -71	15,5	17,3	26,1	200	∞

Table 5-4 Design values of soil properties

The values in Table 5-3 are standardized values. These values have not been converted to the level of the effective vertical soil stresses $\Delta\sigma'_{v;z}$ of 100 kPa.

Also it should be noted that research on the settlements of the Erasmus Medisch Centrum have shown that the Peat is hardened and has the same c_p c_s values as clay.

According to (Brough, [69]) the load-bearing capacity of a shallow foundation can be determined by equation 17:

$$P_e = b(V_b \cdot p_b + V_g \cdot \gamma_1 \cdot b_q) \qquad (17)$$
$$p_e = (V_b \cdot p_b + V_g \cdot \gamma_1 \cdot b_q)$$

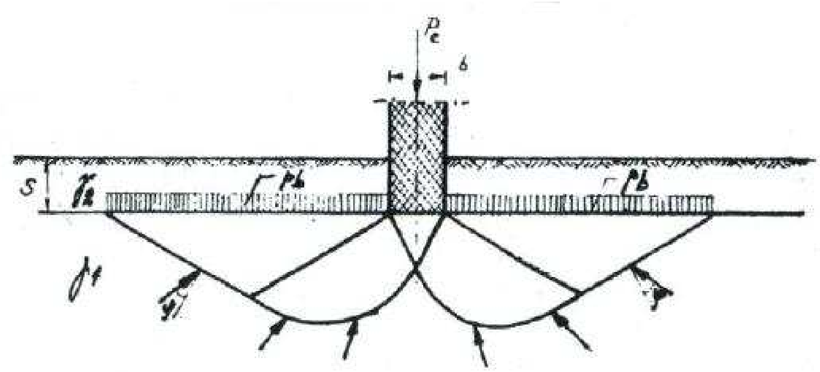


Figure 5-7 Shallow foundation

Where,

$b =$ Width of the foundation [m]

$V_b =$ Coefficient for the (surcharge) depending on φ

$p_b = \gamma_2 \cdot S =$ Surcharge [tf/m^3]

V_g = Coefficient for the (influence of the foundations width depending on φ

γ_1 = Weight of the soil (minus upwards water pressure) [tf/m^3]

γ_2 = Weight of the soil (minus upwards water pressure) [tf/m^3]

This gives a load-bearing capacity of

$$p_e = (1{,}2 \cdot 24{,}6 \cdot 21{,}1 + 0{,}8 * 22{,}5 \cdot 0{,}93 \cdot 140)$$

$$p_e = 2966\_tf / m^2$$

A tonne-force is 1000 kilograms-force or 10 kN which means that the foundations system consisting of a raft has a load-bearing capacity of 29660 kPa or 29660 kN/m^2. (See Appendix J for a more detailed calculation).

5.6 Settlements

Settlements are determined according to NEN-EN 6740 2006 paragraph 13.5.
The total settlement is the sum of:

- The primary settlement (w_1)
- The secondary settlement (w_2)

$$w_1 = \sum_{j=0}^{j=n} \frac{1}{C_{p;j}} \cdot h_j \cdot \ln \frac{\sigma'_{v;z;0} + \Delta\sigma'_{v;z}}{\sigma'_{v;z;0}} \tag{18}$$

$$w_2 = \sum_{j=0}^{j=n} \frac{1}{C'_{s;j}} \cdot h_j \cdot \log \frac{t}{t_0} \cdot \ln \frac{\sigma'_{v;z;0} + \Delta\sigma'_{v;z}}{\sigma'_{v;z;0}} \tag{19}$$

The foundation raft has a width of 140 meters therefore the influence depth of the foundation is 1.8 * 140 =252 meter. The influence width a_e = 4.8*B= 672 meter.

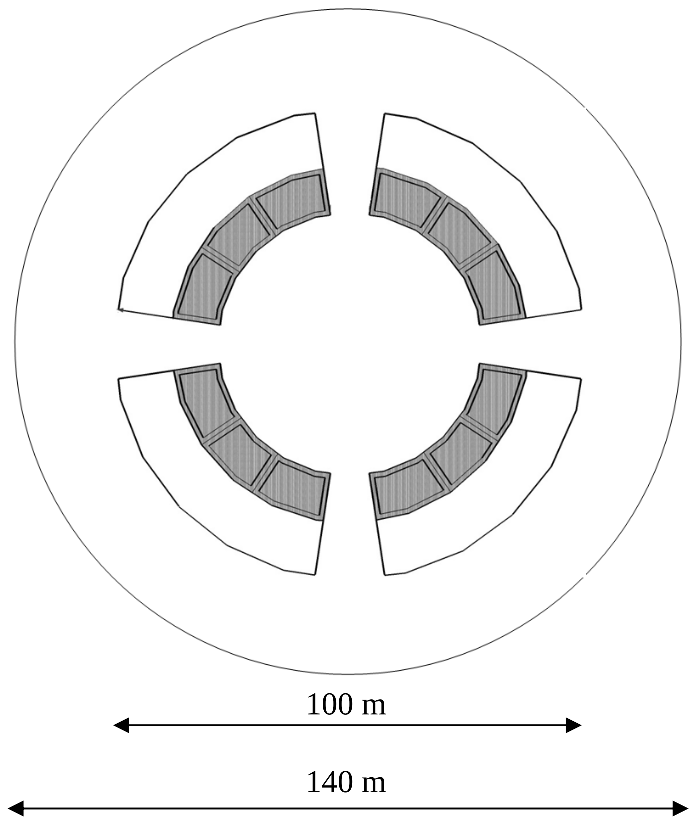


Figure 5-8 Top view tower and basement

5.6.1 Long term settlements

The settlements of the soil layers due to the loads from paragraph 5.4 are given in Table 5-5. (For more details see appendix J).

Layer	C_p	C_s	d (m)	$\sigma'_{v;z;0}$ (kPa)	$\Delta\sigma'_{v;z}$ (kPa)	w1 (m)	w2 (m)	W_{end}(m)
5	1000	∞	17	71.5	1242	0,0495	0,0000	
6	30	400	1	109	1242	0,0839	0,0252	
7	1500	∞	3	121	1242	0,0048	0,0000	
8	30	400	5	157.5	1242	0,3641	0,1092	
9	1500	∞	4	207	1242	0,0052	0,0000	
10	40	500	4	251	1242	0,1783	0,0571	
11	1500	∞	2	294	1192	0,0022	0,0000	
12	50	600	3	340.6	1155	0,0888	0,0296	
13	450	∞	7	397.8	1130	0,0209	0,0000	
14	60	700	1	437.4	1087	0,0208	0,0071	
15	200	∞	7	456.6	1025	0,0412	0,0000	
Total						0,8597	0,2282	1,0879

Table 5-5 Soil layer settlements

The settlements of the soil layers are:

$W_{1,d}$= 0.860 m
$W_{2,d}$= 0.228 m
W_{end}=1.0879m

According to NEN 6740 the settlement needs to be smaller than 150 mm. The Rijnhaven Tower does not satisfy this limit. However this does not automatically mean that the buildings foundation is not good enough. The Dutch building code is not equipped to deal with a building of such a height and special rules and regulations need to be made.

Still a settlement of 1.09 m is very large. Possible measures are reducing the settlement by adding piles (piled (raft) foundation) see paragraph 5.8.
It is also possible to take the settlements into account during the construction and use of the building by increasing the construction level of entrances allowing them to settle and connect to the infrastructure around the tower over time.

It should be noted that the difference in consolidation between the soil before and after the preconsolidation stress has not been taken in to account. Because of this the settlements are overestimated

5.6.2 Differential settlements

In the previous paragraph the total settlements for a uniformly distributed load have been calculated. More important however are the differential settlements since they can cause extra deformation at the top of the building due to a rotation of the foundation. Also a difference in settlements can cause high stresses in structural elements.

We assume that the soil layers are more or less homogeneous and since the footprint of the tower is symmetric the differential settlements are the result of the stresses caused by the bending moment. These stresses have been determined in Appendix J using “ESA scia engineer”. In this chapter we will determine the differential settlements and the effect they have on the total drift of the building.

As mentioned in paragraph 5.4 the differential settlements are the result of wind load which cause a non-uniform stress pattern. In general the wind working on the building consists of short term loads and the soil undergoes an elastic settlement due to this load. The difference between the calculations of the long term settlement is that the layer of Kedichem has a much larger stiffness when subjected to short term loads because the soil is not given time to consolidate and the groundwater is able to participate in the load transfer. Because of this we will assume that the clay layers have an infinite c_p value when subjected to wind loads.

The differential settlements due to the wind load are given in Table 5-6 (for more details see appendix J).

layer	Cp	d	$\sigma'_{v;z;0}$ (kPa)	$\Delta\sigma'_{v;z}$ (kPa)	w1 (m)	w2 (m)
5	2000	17	1313,5	121	0,00075	-0,00082
6	∞	1	1351	121	0,00000	0,00000
7	3000	3	1363	121	0,00009	-0,00009
8	∞	5	1399,5	121	0,00000	0,00000
9	3000	4	1449	121	0,00011	-0,00012
10	∞	4	1493	121	0,00000	0,00000
11	3000	2	1486	163,2	0,00005	-0,00005
12	∞	3	1495,6	158,1	0,00000	0,00000
13	900	7	1527,8	154,7	0,00054	-0,00058
14	∞	1	1524,4	105,875	0,00000	0,00000
15	400	7	1481,6	99,825	0,00114	-0,00122
Total					0,00267	-0,00289

Table 5-6 Soil layer Settlements

$W_{1,d}$ compression = 2,7 mm
$W_{1,d}$ tension = 2,9 mm
$\Delta W_{1,d}$ = 5,6 mm
Rotation = $4 * 10^{-5}$ rad
This gives a deformation of $4 * 10^{-5} * 800000 = 32$ mm at the top of the building.

5.7 Feedback rotation stiffness of the foundation

In chapter 4 several structural alternatives were modelled in ESA. Because the foundation had not yet been designed the tower was assumed to be fully clamped. In reality the foundation should be modelled as rotational spring. The rotation stiffness of the foundation is important because there is no point in adding stiffness to the superstructure when the rotation stiffness of the foundation is insufficient.
In this paragraph we will determine the rotational stiffness of the foundation and determine the final deflection due to the wind load working on the building.

The rotation stiffness of the foundation raft is calculated according to (chapter 2, voorbeelden in de praktijk VSSD) for more details see appendix J.
This rotational stiffness' is added to the ESA models from chapter 4 and gives the following results.

Constraint	**Alternative 1 deformation SLS (mm)**	**Alternative 2 deformation SLS (mm)**	**Alternative 3 deformation SLS (mm)**
Clamped	648	411	339
Rotational stiffness (20000 kN/m^3)	766	476	381

Table 5-7 Influence rotational stiffness foundation

Table 5-7 shows the difference in deformation between a superstructure modelled using a clamped constraint and one with a rotational stiffness.

The deformation of the tower is the sum of the following deformations:

- Deformation due to bending and shear deformation of superstructure (including the second order effect and the rotational stiffness of the foundation.
- Deformation due to rotation as a consequence of differential settlements as a consequence of short term wind loads.

For the chosen alternative, core-outrigger this means a total drift of 508 mm (Table 5-8). This satisfies the criteria of h/500. It can be concluded that the raft has sufficient rotational stiffness and that the deformation due to settlements are not a problem. It should be noted however that squeezing has not been taken into account.

Drift	**Alternative 2 Deformation SLS (mm)**
Drift including rotational stiffness (mm)	476
Drift due to differential settlements (mm)	32
Total Drift (mm)	508

Table 5-8 Total drift at top of the building

5.8 Pile and Raft foundation

As mentioned earlier, a complete design of a pile-and-raft or piled foundation is not possible because of limited time. Earlier it was shown that the raft has enough load-bearing capacity but that the settlements were very large (ca. 1.1 meter).
A high-rise building causes high loads on a relative small area. This causes a difference in settlements within the building and between the high-rise and surrounding structures..
By adding piles the load from the superstructure is partly transferred to deeper and stiffer layers resulting in a reduction of the (differential settlements).

The pile-and-raft foundation should be designed so that the difference in settlements caused by difference in foundation pressure and geometry of the foundation is minimal.
As mentioned in the literature study (Part 1 paragraph 3.1.3) a pile and raft foundation is recommended for high-rise structures with a high slenderness ratio and other structures sensitive to differences in settlements. Because our tower is slender and we expect large concentrated loads a pile and raft foundation can help to control the settlements by transferring a part of the load from the superstructure via the raft and part via the piles.

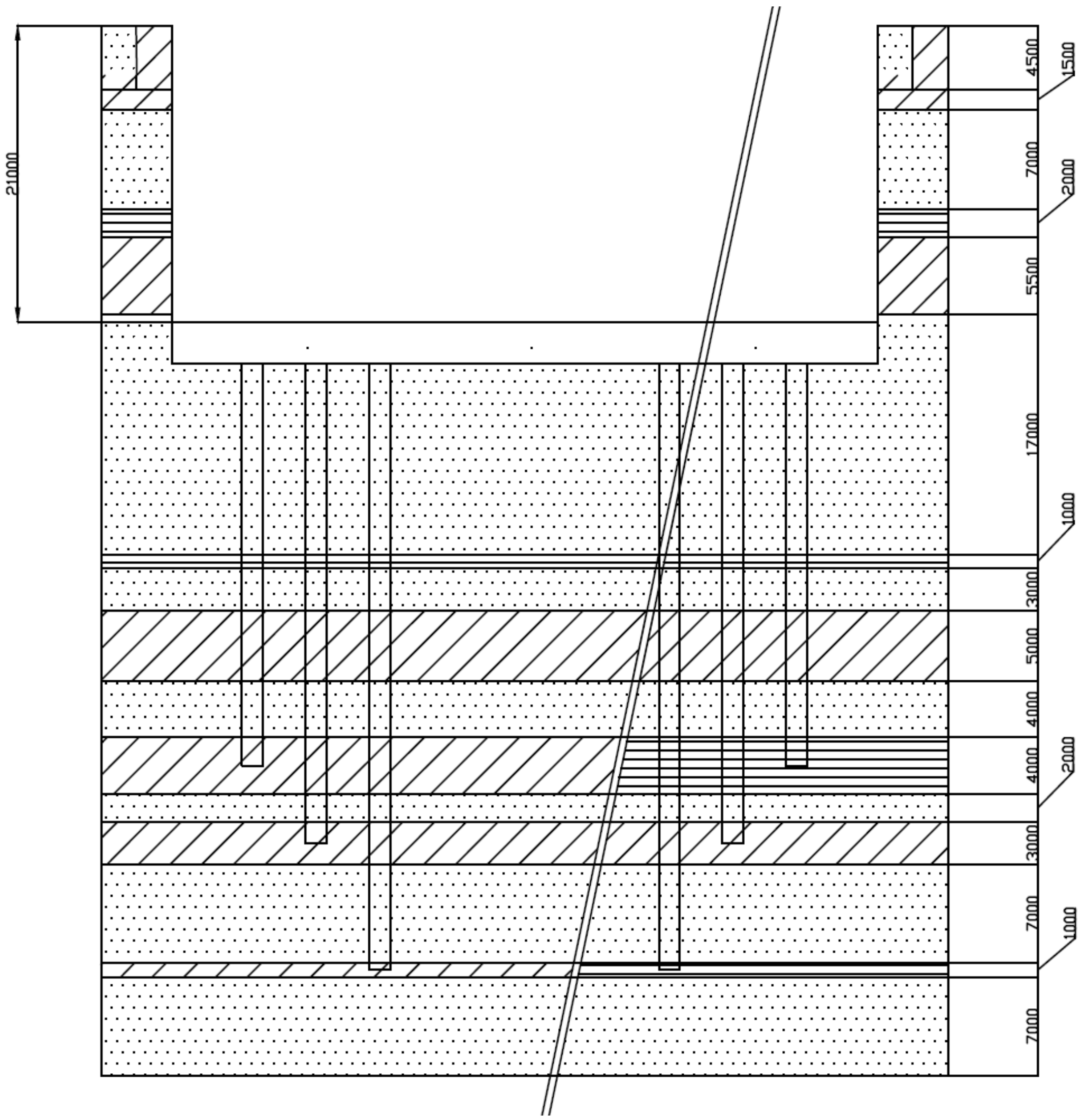


Figure 5-9 Piled raft concept scheme

A seven-layer basement will be constructed until -21 m NAP. The raft is founded on the first load-bearing layer located at -17 m NAP to -34 m NAP. The removed soil will reduce settlements and thus the foundation is able to bear more vertical loads from the superstructure. This will also help reduce the difference in settlements between the new high-rise structure and the existing structures.
The raft distributes the load from the superstructure

Randolph describes 3 design philosophies for piled raft foundation, namely:

- The "conventional approach", in which the piles are designed as a group to carry the major part of the load, while making some allowance for the contribution of the raft, primarily to ultimate load capacity.
- "Creep Piling" in which the piles are designed to operate at a working load at which significant creep starts to occur, typically 70-80% of the ultimate load capacity. Sufficient piles are included to reduce the net contact pressure between the raft and the soil to below the preconsolidation pressure of the soil.
- "Differential settlement control", in which the piles are located strategically in order to reduce the differential settlements, rather than to substantially reduce the overall average settlement.

Earlier it was found that the differential settlements, without taking into account squeezing, were not excessive and its contribution does not cause problem for the criteria of (h/1000 and h/500) for the maximum drift at the top of the building. The settlement due to the long term load however is significantly larger than the limit set by the Dutch building code.
Based on the results found in paragraph the creep piling method is chosen as the design philosophy for the piled raft foundation.

5.9 Conclusion feasibility foundation

In paragraph 5.5 the load-bearing capacity was determined and it was shown that the raft and soil have sufficient load bearing capacity. The settlements however are large and exceed the limit described in the Dutch building code.

The following aspects were not taken into account in the design of the foundation and the calculation for the settlements:

- Differential settlements as a consequence of non-uniform distributed loads.
- Squeezing of weak soil layers.
- The stability of the tower
- The positive effect of upwards water pressure

According to [71] settlements of the ca. 40 year old 114 meter high Erasmus MC are 0.13 meter. If we consider the settlement to be linearly dependant on the height of these buildings we can make a quick comparison between the towers. For our 800 meter high building it would mean we should expect a settlement of (800/114) * 0.13 = 1.15 m.
This is without taking into account that extra mass is necessary to transfer the increased wind load and the increased self-weight of the building due to the extra material necessary to resist vertical en lateral loads.
The Eurocode limits the settlement to ca. 0.15 meters. This value however is meant for buildings of a height to 100-150 meters. The Rijnhaven Tower is a special project which needs special regulation regarding settlement. Since the scale of the building and thus its self-weight and the loads on it is increased a larger settlement is to be expected. Nevertheless reducing the settlements by adding piles is strongly recommended.

Chapter 6 Conclusion and recommendations

6.1 Conclusions

There is large difference in height between high-rise buildings in the Netherlands and high-rise in other continents such as North America and Asia. The tallest building in the Netherlands, the "Maastoren", has a height of 164.75 meter whereas in the rest of the world +300 meter buildings are not uncommon with the Burj Khalifa in Dubai even reaching a height of 828 meter. Figure 6-1 illustrates how high-rise in the Netherlands is still in its infancy compared to the rest of the world.

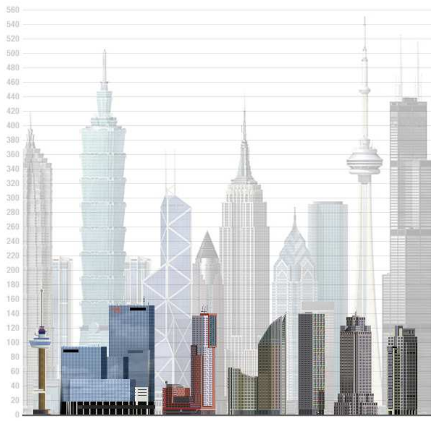


Figure 6-1 Dutch versus foreign high-rise buildings

Each high-rise project is unique and depends on the many conditions which influence the choices made in the design of a tall building. Examples of such conditions are the wind climate, the characteristics of the subsoil and culture. Because of this the following question was asked:

"*Is it technically possible to achieve similar heights (as found in foreign countries) in the Netherlands*?"

In order to answer this question the goal of this thesis was to deliver a structural design for 800 meter high building in the Netherlands.

Firstly, in part 1 a location for the tower was chosen and a literature study on supertall was done. This literature study discusses several aspects which become increasingly important as the height of the building increases. These aspects were used to determine the design of the building and led to the chosen compound structure consisting of several slender towers which are tied together at several points along the height of the building.
This type of superstructure has several advantages over a conventional closed tower such as:

- Reduction of the along-wind forces due to the presence of slots
- Reduction of the across-wind forces and vibrations due to the presence of slots.
- The internal void pushes the building to the perimeter where it has sufficient daylight.

After a footprint for the tower was designed in chapter 3, three structural alternatives (Table 6-1) were compared using the finite element program ESA SCIA Engineer.

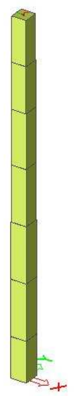
Alternative 1: Core	Alternative 2: Core-outrigger	Alternative 3: (Diagrid)
		

Table 6-1 Structural alternatives

The results in chapter 4 showed that the building acts very stiff when subjected to along-wind forces and that the without the reduction due to slots the across-wind forces would not satisfy the acceleration and deformation limits.

	Core	**Core-outrigger**	**Diagrid**
Along-wind Drift	+-	+	+
Along-wind Forces	+-	+	++
Along-wind accelerations	-	+	+
Across-wind Drift	-	+	+
Across-wind Forces	-	--	--
Across-wind accelerations	+-	+-	+-
Architectural freedom	+	+	--
Erection process	+	+	--
Self-weight	+-	+-	+-

Table 6-2 Comparison structural alternatives.

Alternative 2, the core-outrigger alternative, was chosen as the building superstructure even though the diagrid performed better in certain structural aspects. This choice was made because the alternative 2 has better dynamic behavior due to the use of concrete and the fact that diagrid offered less architectural freedom and the erection process is more difficult.

For the foundation the (differential) settlements of a shallow foundation was calculated. This shallow foundation consists of a raft located at -21 m NAP and a 7 layer basement.
The settlements were found to be very large namely 1.09 meter and it is recommended to reduce the settlement by adding piles and creating a piled (raft) foundation.

The design of the Rijnhaven Tower shows the possibilities for supertalls in the Netherlands. It can be concluded that it is possible to make a superstructure which is able to resist the wind loads and keep the wind-induced vibrations within the limits described in the Building codes. The foundation however needs to be researched further. Due to limited time it was not possible to design and test a piled (raft) foundation. Instead the load-bearing capacity and the settlements of shallow (raft) foundation were calculated in order to get a grasp of the possibilities. The results showed that the differential settlements were not excessive (see Table 6-3) however it should be noted that differential settlements due to squeezing have not been taken into account. The total settlements of the Rijnhaven Tower with a raft foundation were found to be very large and it is therefore recommended to reduce these settlements by adding piles which transfer loads to deeper and stiffer load-bearing soil layers.

alternative	**Alternative 2 Deformation SLS (mm)**
Drift including rotational stiffness	476
Drift due to differential settlements	32
Total drift	**508**

Table 6-3 Total drift at the top of the building

In the design of both the superstructure and the foundation a lot of assumptions and decisions were made see (paragraph 6.2.1). For the superstructure these mostly have to do with aerodynamics and need to be researched further with the help of wind tunnel.
In the design of the foundation the following aspects have not been taken into account.

- Differential settlements due to non-uniformly distributed loads;
- The possibility of “squeezing” of the weak soil layers. It should be noted that this effect does not play a role when piles are added.
- The stability of the tower
- The upwards water pressure

These aspects need to be researched before a conclusion can be made on the geo-technical feasibility of an 800 meter building.

6.1.1 Influence of non-structural considerations on the building's design.

During the design of the Rijnhaven Tower a lot of choices had to be made. It was interesting to see that many of these were not made from a structural point of view but instead were the result of the demands of other important aspects such as erection speed and vertical transport.

Floor-system

A floor-system has a large influence on the self-weight of a building and thus a large impact on the design of other structural elements in the superstructure and foundation. Therefore, from a structural point of view, it makes sense to choose a light floor-system. Also in very tall buildings a reduction in the floor thickness and storey height can even result in more floors for the same building height allowing for more rentable space. However as the building get higher and higher, the self-weight of the floor-system becomes less important and the impact which it has on the erection of the tower becomes the governing aspect. Lighter alternatives such as hollowcore, bubbledeck and slimline floor-systems require many crane movements and as the height of a building increases they become economically unfeasible. Cast-in-situ concrete floors are heavier but much more feasible since concrete can be pumped up to great heights. In the end a composite floor was chosen because of its ability to reduce the dead load and ensure a feasible erection method.

Vertical allocation of the buildings function.

Out of the two possible ways to determine the vertical allocation of the building functions the one which follows from tenant's preference was chosen instead of a structural preference (paragraph). This choice considered the vertical travel habits of the buildings inhabitants such as "how long people are willing to travel" and "how often they leave the building". This once again underlines the importance of vertical transportation in a supertall building.

Number of couplings along the height of the building.

The Rijnhaven Tower uses a compound structure where 4 slender towers are linked along the buildings height through belt-trusses. The stiffness of the tower is therefore dependant on the number of couplings along the height of the building. In literature study ([7]) it was suggested that the improvement when going from 4 to 5 outriggers is minimal for core-outrigger systems. For the structural design of the building a study was done to examine the effect of couplings along the height of the building (see appendix F).
The conclusion of the study was the same as the one found in [7]. A tower with 4 couplings does not gain a significant increase in stiffness when extra couplings are added. In the end a number of 6 couplings was actually determined by the requirements of the vertical transportation system. This transportation system consists of express elevators and local elevators. The building is divided into 25-30 storey-high sky-neighborhoods and skylobbies which are serviced by express lifts. At the skylobbies inhabitants are able to transfer to local lifts which takes them to their final destination.

6.2 Recommendations

6.2.1 Optimization

The results in chapter 4 show that the Rijnhaven Tower performs very well when exposed to along-wind and across-wind loads. Due to its aerodynamic design the wind loads in the along and across-wind direction are greatly reduced. The settlements however are quit large.
Earlier all three alternatives were given the same core dimensions in order to make a fair comparison. It is however, due the larger internal lever arm of alternative 2 and 3, possible to further optimize the structure by using thinner core walls or even omitting them as the building height increases. Optimizing the superstructure in this way can have a large influence on the buildings economic feasibility. Earlier in paragraph 3.3 it was mentioned that the buildings footprint could have a better NFA/GFA ratio if walls were omitted. This means that the building will have a larger total leasable area. The fact that the walls are omitted at the buildings top, which is the most lucrative part of a supertall, makes the building even more economically feasible.

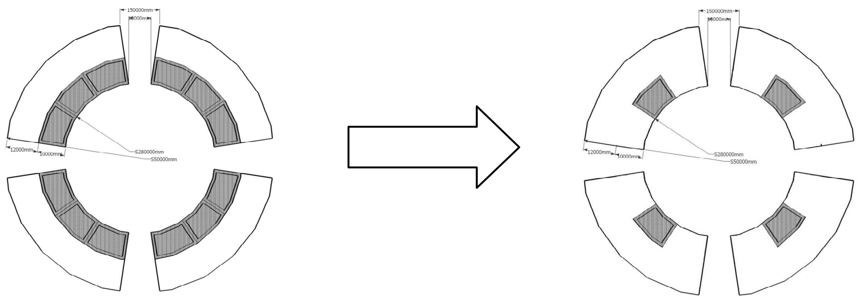


Figure 6-2 Omitting walls results in more economical feasible structure

Another benefit is that the thinner core walls can result in reduced load working on the buildings foundation and thus smaller settlements. However it should be noted that that a reduction in mass can have a negative influence on the dynamic behavior of the tower.

6.2.2 Windtunnel research

Because of limited time and resources wind tunnel research has not been used for the design of the tower. The wind forces were determined by extrapolating existing values and the reduction due to slots was taken into account by using values found in reference projects and experiments.
For a building of great height and slenderness, wind forces and the resulting motions in the upper levels become dominant factors in the structural design. Because of this designing a supertall using a wind tunnel is more rule than exception. [20] For example states that the Burj Khalifa was practically designed using a wind tunnel.

Along-wind and across-wind loads.

It is recommended to use high-frequency-force-balance technique to determine the wind loading on the Rijnhaven Tower's main structure early in the design. The wind tunnel data can then be combined with the dynamic properties of the tower in order to compute the tower's dynamic response and the overall effective wind force distributions at full scale.

Towards the end of the design an aero-elastic model with properly scaled stiffness mass and damping can be used to determine the buildings aerodynamic response. The results of aero-elastic tests can be found to be significantly different to those derived from the force-balance test.

Wind tunnel research on the impact of adding slots and voids to the 800 meter high tower needs to be done. Earlier the along-wind forces were reduced by a factor 3 due to the presence of slots which allow the wind to blow through the building. This factor is based on research done on the Nakheel Tower [27]. This value should be checked by comparing a conventional closed cylinder to a model of the Rijnhaven Tower which has openings.
Interesting questions are:

- How much is the reduction of the wind load compared to a tower without slots.
- What is the effect of the width of the slots on the aerodynamic behaviour of the tower?

Vortex shedding forces have been determined using (simplistic) formulas. The values need to be checked using a wind tunnel.

Neighbouring buildings

The effect which the Rijnhaven Tower has on the neighbouring buildings on the Wilhelminapier should be studied.

Wind effects on cladding

The design of the façade and cladding has not been researched. The following factors play a role in the determination of the local wind loads on a façade:

- the shape of the building
- the influence (neighbouring) buildings have on each other
- the slenderness of the building
- loads during construction
- influence of façade details

For a round shape it is recommended to design the entire façade as a border zone. This is done because the highest local wind load acts at the place where the vortices shed. For a rectangular shape this takes place at the corners but if the building has a round shape the vortices can shed everywhere depending on the wind direction.
Also openings in buildings can strongly increase the local wind loads so wind tunnel research is highly recommended (the only way to determine the local loads).
Two buildings who are close can act as a "funnel". This can cause local pressure differences higher than a single building which cause the wind to accelerate and thus increasing the wind pressure. This should be taken into account for the façade and cladding at the slots.

Whistling caused by wind blowing through the slots

Openings are added to decrease the along-wind forces working on the building and the across-wind induced vibrations. These openings can cause a whistling effect when the wind blows through the building resulting in nuisance for its environment.

6.2.3 Geo-engineering

In the end situation considerable settlements were calculated for the tower with a raft foundation. These settlements do not necessarily have to be a problem if the differential settlements are limited and appropriate measures are taken. For example it is possible to take into account the settlement by increasing the construction level of the entrances which allows them to connect to the infrastructure when the tower settles over time.

A more difficult problem is the consequence which the settlements of the tower have on the surrounding buildings. In part 1 the location in the Rijnhaven was chosen because the tower would not stand on its own resulting in a cluster of high-rise buildings which creates an urban effect. The settlement of the tower can cause settlements and rotation of existing high-rise buildings and therefore the effect of the tower on its surroundings should be studied.

The following was said about the piled-raft foundation in the literature study:

"Pile and raft developed was originally meant as an improvement for shallow foundations. In the Netherlands deep foundations are used for high-rise buildings. However with increasing heights higher loads are leading to denser piling and larger pile diameters. The increasing interest in use of underground space in urban areas and technical developments in the area of building excavation have put relatively stiff and load-bearing soil layers within the reach of foundation slab. Besides a reduction of the upward forces caused by groundwater, the possibility of applying multiple basement layers can also ensure that the foundation slab transfers loads directly to the load-bearing soil layer. This enables the foundation to reduce settlements in deep and weak soil layer compared to the conventional deep foundation".

Now that the load-bearing capacity and the settlements of the foundation, consisting of a raft, are known it is wise to reevaluate the choice of the foundation system by making a thorough comparison between the 3 possible alternatives, namely:

- A raft foundation
- A piled foundation
- A piled raft foundation

The piled raft foundation was chosen because large concentrated loads were expected which could require a high density of piles and large diameters in a deep foundation. The following questions should be answered in order to determine which alternative is the most suitable:

- What is the governing aspect for each alternative? (Total settlement, differential settlement, load-bearing capacity).
- How dense is the piling for a deep foundation and piled raft foundation? How deep do the piles need to go? (The group effect should be taken into account).
- How much parking and storage is necessary for the Rijnhaven Tower and its surrounding building?

When these questions are answered the best alternative can be chosen. However, because the current code and regulations are lacking, the first thing that needs to be done is research to determine suitable rules and regulations which can be applied to a building of 800 meter (see paragraph 6.2.4).

6.2.4 Code and regulations

In a lot of cases the Dutch building code proved to be inadequate and/or lacking. This is not surprising since a building with a height of 800 meter is unheard of in the Netherlands were the highest building is 164.75 meter.

The fire-safety regulations found in the “bouwbesluit” and “eurocode” for example are not meant for an 800 meter building. A rule is given for buildings above a certain height or amount of floors which simply can’t be used because this would mean that an 800 meter high building has the same as a 70 meter high or 15-storey building.

The question is if the regulations should be stricter for supertall buildings, especially for one with a height of 800 meters. A supertall tower has a large population and as seen at the terrorist attacks on World Trade Centre (September 11, 2001) the consequences can be catastrophic. With such a huge population a fire can result in a large number of casualties. Therefore it is important to consider the evacuation time and the ability of the load-bearing structure of the building to remain standing using for example advanced structural fire analysis. It is clear that in this case it is not enough for the building to satisfy the criteria found in the Dutch building code.

The opposite however is also possible. If the building does not satisfy the criteria found in the Dutch building code it does not automatically mean that the building is designed poorly.
A criterion for the settlements of a high-rise building in the Netherlands is ca. 0.15 meters, a value which is meant for buildings with a height of 100-150 meters. This criterion unlike other deformation criteria like the maximum lateral deformation a building (1/500 * h) or the deformation of a floor (0.004l) is not dependent on the height or dimensions of the building.
A building with a height of 800 meter causes a larger concentrated load due to the large number of storeys and the need for larger structural elements to resist lateral loads. Because of this, it is not surprising that the settlements exceed the value of 150 mm.

These examples show that for a special tower there is a need for special regulations in order to accurately judge the buildings structural behavior. The development of criteria for an 800 meter building requires a lot of research and expertise and is beyond the scope of this thesis.

6.2.5 Renewable energy

A supertall has a huge influence on its environment and a proposal to build one can lead to considerable resistance. Opponents often argue that they are too expensive or don’t fit in the townscape. Sustainability can help improve the image of a skyscraper and therefore it is recommended to study possibility of making the building more sustainable by using renewable energy.

Possible ways of achieving this are:

- Wind turbines at openings.
- Wind energy producing spire.
- Solar energy (facade)

Chapter 7 List of figures

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