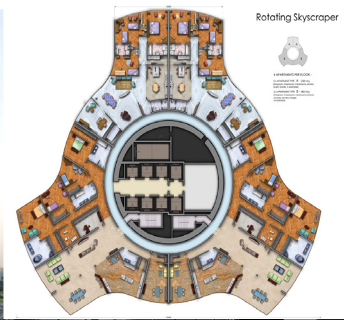
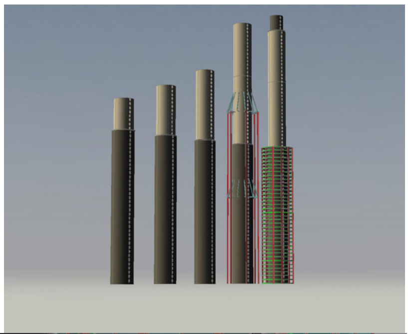
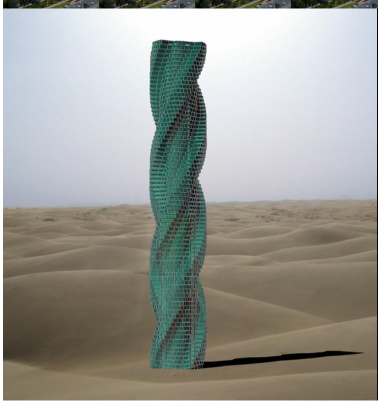
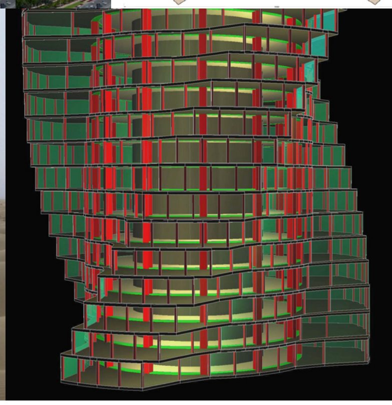
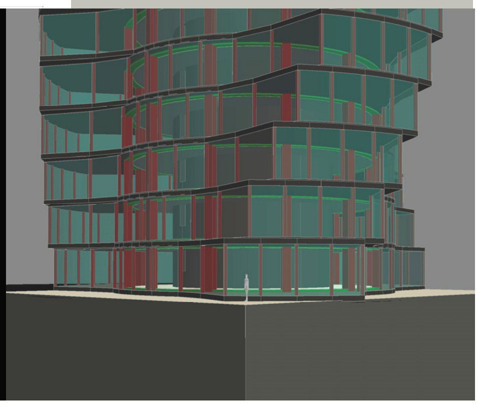
Master thesis report

P. den Besten

“Structural feasibility of the Rotating Tower Dubai”

Graduating committee

Prof. Ir. R. Nijsse	TU Delft
Ir. K.C. Terwel	TU Delft
Dr. Ir. Drs. C.R. Braam	TU Delft
Ing. M.J. Bos	Iv-Consult

TU Delft
Delft University of Technology

Iv-Consult

Preface

This report is the result of my master study project at Delft University of Technology, faculty Civil Engineering (master Building Engineering). During a period of 6 months I've been working on this thesis. I've done this most of the time at Iv-Consult in Papendrecht and partly in Delft at the University.
The subject of this thesis report is the Rotating Tower in Dubai. It is a very ambitious project which triggered my imagination. Making this thesis report was challenging at times, but I enjoyed working on it. I learned of lot from the process and I look back on in with satisfaction.

In this preface I would like to take the time to thank a few people for their help and guidance during the writing of my thesis report. First I would like to thank my graduation committee for their guidance during the process of writing my master thesis report. Thanks to prof. Ir. Nijsse (who despite his busy schedule always finds time for a consult), Dr. Ir. Drs. C.R. Braam (who could help me with his extended knowledge of concrete structures), Ir. K.C. Terwel (for his guidance during my thesis and my complete master study) and Ing. M.J. Bos (for his daily guidance and support).
Further I would like to thank my colleagues at Iv-consult for their help and time. Especially E.S Peltenburg helped me a lot with technical questions and always took the time to help me.

Last but not least I would like to thank my family and friends for their support and encouragement during my study.

Pim den Besten
Papendrecht, May 2011

Table of contents

Abstract

The main objective of this master thesis is designing a feasible load bearing structure for the Rotating Tower, within the set boundary conditions given by the architectural design.

Dynamic architecture
The concept "dynamic architecture" is developed by the Italian architect David Fisher. The main idea behind this concept is: the building of the future. Mr. Fisher translated this idea into the design of the Rotating Tower: a tower with separate rotating floors. The tower consists of multiple (non-circular) floors that can rotate independent around a common axis. Since all floors rotate independent, the building can (theoretically) transform into every shape imaginable.

Different architectural designs were made for the Rotating Tower for different target cities. This thesis report is based on the architectural design for the city of Dubai.

Reference projects
The original design of the Rotating Tower (designed for target city Dubai) has a height of approximately 435 meter. To get a more detailed view of previous designed load bearing structures 3 reference project with comparable height were analysed. These three projects are: Burj Khalifa, Taipei 101 and Shanghai World Financial Centre.

Load cases
Different load cases are analysed and used in the calculation for the structure: dead load, live load, wind load and earthquake load. In this analysis local conditions for Dubai are taken into account.

Current design
With these load cases the original architect's design of the concrete core is checked with a global calculation. It turned out that the original design for the stability structure did not meet any of the requirements given in the codes (deformations are 8 times larger than the maximum allowed value).

Optimization analysis
The current design of the stabilizing core for the Rotating Tower does not meet any of the requirements given in the codes. Therefore different solutions for stiffening and strengthening the structure were investigated. Most striking solutions are: higher concrete grade, thicker core, activating steel structure of the storeys and active systems.

Alternative designs
With the results of the optimization analysis 5 different feasible designs were made for the stability structure of the Rotating Tower. All these designs have one or more adaptations from the architects design. 3 of the 5 alternative designs are considered to be the most relevant for the project and are presented as "final designs". These 3 final designs are worked out to a more detailed level.

Conclusion
The main conclusion of this thesis report covers the structural feasibility of the project. For several designs it is shown that the project is feasible from a structural point of view but not without adapting the architectural design. All alternatives contain one or more adaptations to the architectural design, but keep the main concept of the project unchanged.

1 Introduction

1.1 Concept "dynamic architecture"

The concept "dynamic architecture" is designed by the Italian architect David Fisher. The main idea behind this concept is: the building of the future. Mr. Fisher translated this idea into the design of the Rotating Tower: a tower with separate rotating floors.

David Fisher was inspired by Sheikh Mohammed Bin Rahid Al Maktoum, ruler of Dubai. The Sheikh inspired Mr. Fisher with the saying: "do not wait for the future to come to you …… face the future" [1].

The tower consists of multiple (non-circular) floors that can rotate independent around a common axis. Since all floors rotate independent, the building can (theoretically) transform into every shape imaginable. Each floor has a non-circular shape, so it seems like the building moves constantly.

The Rotating Tower is a concept designed for any location in the world and not for one specific. Different target cities were selected and designs are made for these cities.

1.2 Thesis assignment

1.2.1 Problem definition

Dynamic architecture is a very ambitious concept which has a lot of challenges that need to be solved. Most challenges (like water supply and the driving system) are considered and looked upon (see also section 2.3 and chapter 6). But one important aspect of the tower is not designed yet: the structural system (for overall stability). In the architectural design a conceptual design for a stabilizing core is made. This core is however not checked in accordance with the valid codes (and it is expected that the current design does not meet the requirements for strength and deformation). The design of the main structural system is a governing factor for the feasibility of the tower. When the structural system can't be fitted into the architectural design, the tower can't be built in the way it was intended.

1.2.2 Objective

The main objective of this master thesis report is to create feasible structural designs for the Rotating Tower. These structural designs must be tuned with the architectural design: the main dimensions of the storeys, functions and concept (rotating storeys) of the tower must remain unchanged. The structural design of the storeys itself (which is already made) will be used as input for the design of the structural system.

1.3 Contact information

Student
P. den Besten BSc.

Tel:	+31 (0)6 1317 3137
E-mail:	pimdenbesten@gmail.com
Studentnr:	1295144

Committee

Chairman
Prof. Ir. R Nijsse

Company:	TU Delft Stevinweg 1 Delft
Tel:	+31 (0)15 27 84132
E-mail:	R.Nijsse@tudelft.nl
Chamber:	S2 1.36

Supervisor
Dr. Ir. Drs. C.R. Braam

Company	TU Delft Stevinweg 1 Delft
Tel:	+31 (0)15 27 82779
E-mail:	C.R.Braam@tudelft.nl
Chamber:	S2 2.06

Supervisor
Ir. K.C. Terwel

Company:	TU Delft Stevinweg 1 Delft
Tel:	+31 (0)15 27 81512
E-mail:	K.C.Terwel@tudelft.nl
Chamber:	S2 1.54

Company supervisor
M.J. Bos BBE.

Company:	Iv-Consult Noordhoek 37 Papendrecht
Tel:	+31(0)6 533 82499
E-mail:	m.j.bos@iv-consult.nl

2 Project

The introduction already described the “Rotating Tower” project in short. This chapter will give more information about the project. The different project locations, specifications and challenges of a rotating building are described.

2.1 Project location

The Rotating Tower concept has been developed for several locations. Each design looks different, but the idea stays the same: all floors can rotate separately from the other floors. The different locations which have been chosen and developed are:

- London
- Paris
- New York
- Moscow
- Dubai

The designs for all locations have different specifications. The pictures below give an indication of the different designs:

London

Paris

New York

Moscow

Dubai

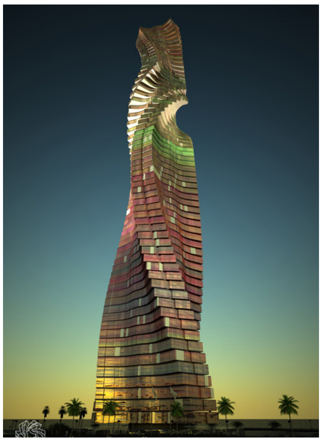


Figure 2-1: different designs Rotating Tower concept [39]

This report is based on the specifications from the Rotating Tower designed for the location Dubai. In the structural design local factors (wind speeds, foundation assumptions, codes etc.) for Dubai will be used.

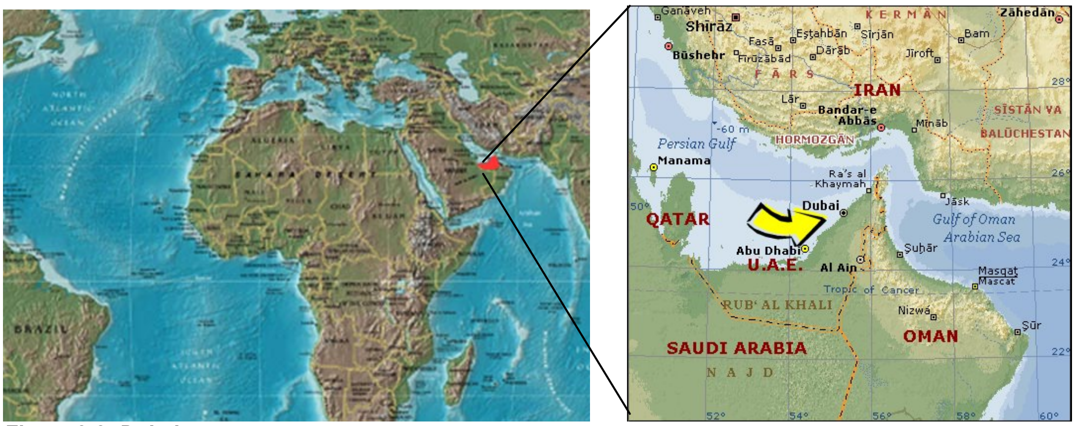


Figure 2-2: Dubai

2.2 Specifications Rotating Tower Dubai (architectural design)

The specifications given in this section are mainly based on the architectural design. Many of these specifications are based on assumptions and need to be engineered before they can considered to be realistic.

Specifications		Magnitude	Unit
Height tower		435.3	m
Average storey height		5.4	m
Number of floors		80	-
Floor area per storey		1142-1826	m^2
Rotation speed		1	rot/h
Diameter core	Floor 0-37	30.5	m
	Floor 38-70	27	m
	Floor 71-80	20	m

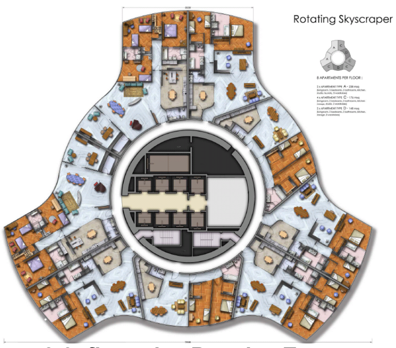


Figure 2-3: floor plan Rotating Tower

Figure 2-4: floor plan Rotating Tower

The total overview of the dimensions and functions of the tower and floors is given in appendix A.

2.3 Challenges

The Rotating Tower project is a completely new concept, therefore some subjects need to be investigated before it will be a feasible project. The main subjects are:

- Water supply
- Human comfort in the tower
- Driving system of the floors (see chapter 5)
- Structural system (for overall stability)

The first three subjects were investigated in an earlier stage. Only the latter is never looked upon in detail. This thesis report is about the structural system.

3 Reference projects

Building a tower with a height of more than 400 meters requires more specific knowledge than for low rise buildings. Different projects in the past are realized with such great height. It is useful to have a good look at these reference projects. In this chapter three reference projects will be analyzed:

- Taipei 101
- Burj Khalifa
- Shanghai World Financial centre

All of these projects have been chosen because they have a similarity to the Rotating Tower. The most important comparisons with the Rotating Tower are:

- Comparable height
- All buildings are at a location where strong (typhoon) winds and earthquakes occur

The projects will be analyzed by looking at the structural design, governing load case, foundation and possible additional interesting information concerning the Rotating Tower.

3.1 Taipei 101

The Taipei 101 tower was finished in 2004. Like the name indicates the tower exists of 101 floors and it also contains a basement of 5 floors. The tower has a maximum height of 508 meter (including the spire) and the top of the 101st floor is located at 457 meter above ground level.


Figure 3-1: Taipei 101

Structural system [4]
The lateral forces (wind and seismic) are resisted through a combination of a braced frame core, outriggers, "super-columns" and moment resisting frames in the perimeter and other locations.
Both the core (22.5 m x 22.5m) and "super-columns" (3m x 2.4m) are composite concrete-steel columns (with concrete up to the 62nd floor to reduce the dead weight). For additional stiffness concrete shear walls are placed between the core columns up to the 8th floor. Above the 8th floor the core is braced by steel V-braces on the outer faces of the core and with moment resisting steel frames on the inner faces of the core.
Every 8 floors the core and "super-columns" are connected by an outrigger truss. The outrigger trusses are either 1 or 2 storeys high, depending on the position in the building. The storeys containing an outrigger are not available for offices, because the structure takes up most of the space

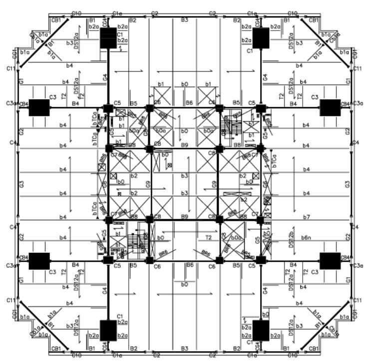


Figure 3-2: Floor plan Taipei 101 [3]

Governing load case [3]
Taipei is an area where both typhoon winds and earthquakes are a returning phenomenon. Therefore, the tower is designed to withstand both load cases.

The governing load case for the Taipei 101 tower is wind load and more specific the wind induced acceleration at the top storeys (7.5 cm/s^2 where 5 cm/s^2 is allowed by the local government). To reduce the accelerations the Taipei 101 has a tuned mass damper at the top levels.
Besides the tuned mass damper another modification was made to the initial design of the tower. Intensive wind tunnel testing showed a decrease of 25% of the base shear force caused by typhoon winds when using chamfered corners (see floor plan – fig. 3-2) instead of straight ones.

Foundation [3,4]
The tower is founded on 380 concrete cast in-place piles driven 30 meter into a layer of bedrock and a concrete slab of 3-5 meters thick. The piles have a diameter of 1.5 meter and a total length of 80 meters.

Tuned mass damper [42]
Typhoon winds and earthquakes often occur in Taiwan. Because very high accelerations at the top are expected in these conditions, a special damping system was developed for the Taipei 101 tower.
A tuned mass damper is installed on the top floors (88th to 92nd floor) of the tower. The TMD has an impressive weight of 730 tons and is completely build up from steel plates.

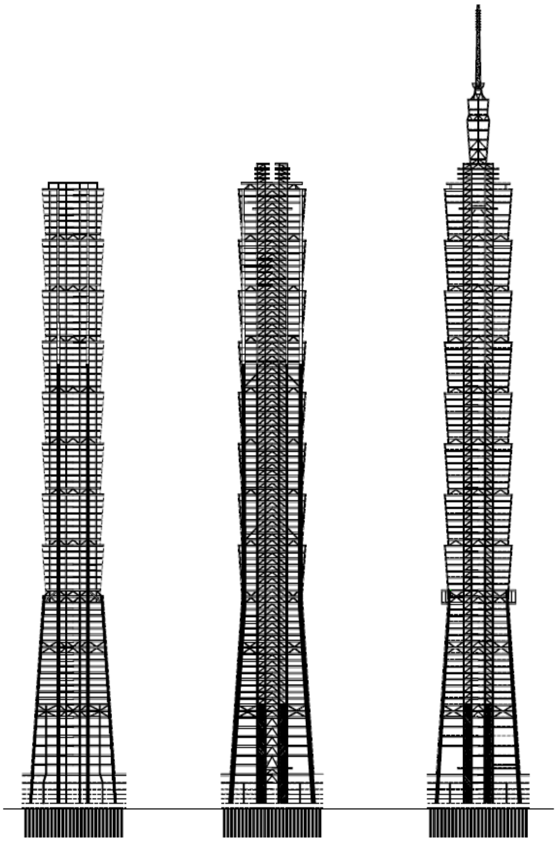


Figure 3-3: Structure Taipei 101 [4]

3.2 Burj Khalifa

The Burj Khalifa (also known as Burj Dubai) is built in the city of Dubai, United Arab Emirates. In 2004 the construction of this enormous project started. The tower reached his highest point on January 2009 and was officially finished on January 2010.

With a total height of 828 meters (tower including spire) the tower is the tallest building in the world. During the construction (in 2007) the Burj Khalifa took this record from the Taipei 101 tower. The tower contains 163 floors and the main functions of the tower are: hotel, office, apartments and observation.

Figure 3-4: Burj Khalifa

Structural system [5]
The lateral and torsional stiffness are provided by a central (hexagonal) core, wing and hammerhead walls, outrigger walls and perimeter columns. The structural system of the Burj Khalifa can be regarded as a core with outrigger system (just like the Taipei 101).

All structural elements are made of (high strength) concrete. Almost every part of the Burj Khalifa is used as a structural element (all internal walls, floors etc.). The only part of the building which is not utilized in the structure is the façade. Because of this clever use of elements, a very stiff structure was created (which made it possible to rise to the enormous height of 828 meter).

Figure 3-5: Floor plan Burj Khalifa

Governing load case [6]

The different load cases on the Burj Khalifa are governing for different parts of the structure. The wind load on the tower is governing for the concrete structure and earthquake loads are governing for the steel spire on top of the tower.
Because wind load is the governing load case for the lateral stability structure, extensive research is done on wind load by wind tunnel testing.
The most important outcome of these tests is the building shape. The floor plans are Y-shaped because this gives the least base shear forces. This is only true when the wind blows in the direction of the wings (just like in figure 3-6). Therefore the tower is situated such that the (historical) strongest winds blow on one of the wings.

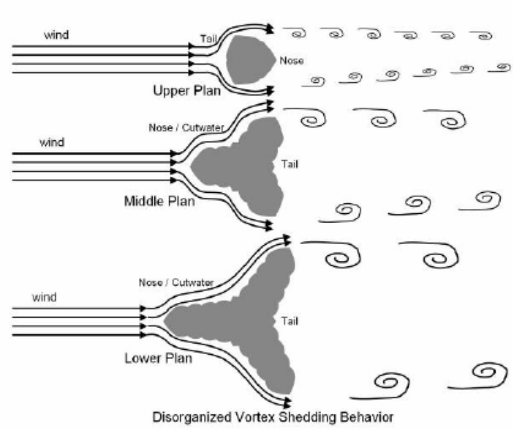


Figure 3-6: wind behaviour Burj Khalifa [6]

Foundation [6]

The Burj Khalifa is founded on 194 piles with a diameter of 1.5 meter and a 3.7 meter thick concrete slab. The piles are 46 meter at a maximum and are drilled concrete cast-in-place piles. The pile with maximum load is loaded up to 35 MN.

Earthquake area [5]

The Burj Khalifa is located in Dubai (just like the Rotating Tower). Dubai is an area where earthquakes can occur. For buildings over 100 meters the local government demands a design which can guarantee that the building stays intact during an earthquake.
The Dubai Municipality specifies Dubai as an UBC97 2a seismic region with a seismic zone factor Z=0.15 and soil-profile SC (soft rock). UBC97 specifies 4 seismic regions (1-4) with 1 being the lowest and 4 the highest.

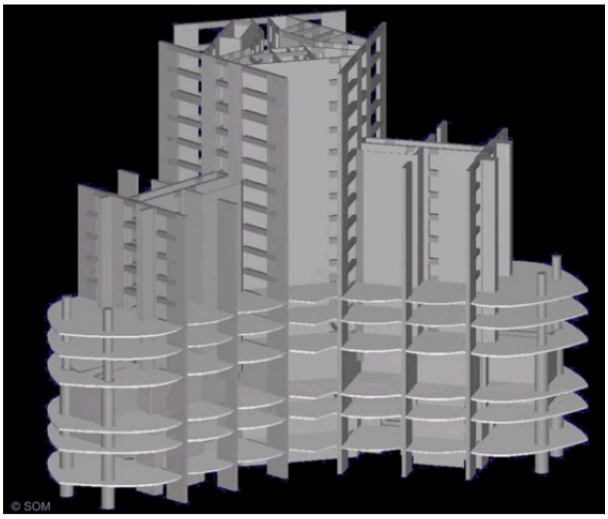


Figure 3-7: concrete structure Burj Khalifa [6]

3.3 Shanghai world financial centre (SWFC)

The Shanghai world financial centre was finished in 2008 and with its height of 492 meters it was the building with the highest roof top and highest occupied floor at that time. The building counts 101 floors and contains a three floor parking garage. The building is used as office space and hotel.

Figure 3-8 : SWFC

Structural system [7]
The Shanghai world financial centre uses a combination of 2 structural systems. Besides a central core with outriggers, a mega structure in the façade is used. The structural system consists of:

Central core
The central core is made of reinforced concrete. The lateral stiffness of this core for resisting wind and earthquake induced loads is decreased, while the stiffness of the perimeter system was increased. By doing so, the dimensions of the concrete core were decreased as well as the self-weight of the total structure.

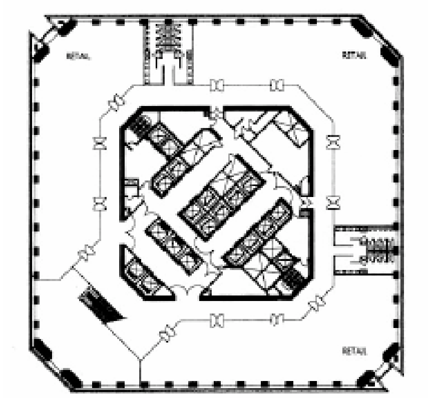


Figure 3-9: Floor plan SWFC

Outrigger trusses
At 7 locations in the building the central core is connected to belt trusses by outrigger trusses.

Mega columns
Four mega columns give the structure more lateral resistance. The mega columns are composite steel-concrete columns. The columns are even bigger than the ones in the Taipei 101 tower: 5.4m x 5.4m.

Belt trusses and mega diagonals in the façade
The perimeter structure is formed by a combination of belt trusses (located at the same elevation as the outrigger trusses), mega diagonals and mega columns. These three components form a truss structure in the perimeter.
The belt truss and diagonals are steel box girders. Because of the smart use of structure the weight was lowered significantly.

Figure 3-10: Structure SWFC [7]

Governing load case [7]
From the literature it is not clear what the results of the wind analyses are. It is expected that wind loads are governing in this project too. This conclusion is based on the earthquake analysis. From this analysis it became clear that the tower behaves elastic during its whole lifetime, while a 4% plasticity ratio is allowed. Based on this fact it is safe to assume that earthquake loads are not governing.

3.4 Rotating buildings

There are no existing projects which can be compared to the Rotating Tower. Although there are examples of buildings with rotating floors, they all have a completely different principle. One of the most famous examples is the Stratosphere Tower in Las Vegas:

***Stratosphere Tower, Las Vegas* [40,43]**

The Stratosphere Tower was finished in 1996. With 356 meter height it is the tallest freestanding structure in Las Vegas and west of the Mississippi.

The most famous part of the tower is the restaurant. The floor of the restaurant can rotate 360 degrees in one hour. Only the floor of the restaurant rotates and not the whole storey. So the façade structure and main load bearing structure stay in the same place during the rotation.

Although the height of the tower and the principle of a rotating floor make it seem like it is a great reference project, the way of rotation is completely different from the Rotating Tower. The storeys of the Rotating Tower rotate as one part around the central core, while in the stratosphere tower only the floor moves. This rotation principle can be a good reference when designing the driving mechanism or investigating the effect of rotation on the human comfort, but not the effect of a rotating storey on the main load bearing structure.

Figure 3-11: Stratosphere tower

Rotating house Everingham [44,45]

Another example of a rotating building is a rotating house in Everingham, Australia. In this building the entire floor rotates 360 degrees.

Just like the Stratosphere Tower the principle of this project is the same as the Rotating Tower, only it is not to be used as a reference project. The house has only one storey and the rotating floor transfers his weight directly to the foundation. This project is useful for designing the driving mechanism, but not for designing a main load bearing structure for a high rise building with rotating floors.

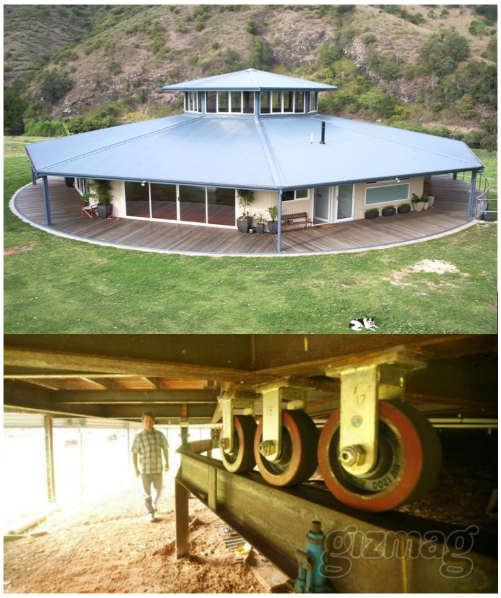


Figure 3-12: Rotating house Everingham

3.5 Conclusions

The most important conclusions which can be drawn from the reference projects are:

Structural design
In projects with great height (over 400 meter), the structural system always contains more than just a central core. In all the reference projects the core was provided with extra strength and stability by an extra structure. This structure was either an outrigger or a perimeter tube structure.

Governing load case
In (most) projects wind was the governing load case for the overall structure and not earthquakes. Wind is most often the governing load case for high rise buildings and more specific maximum acceleration at top levels (human comfort).
This outcome could have been predicted on beforehand. High-rise buildings often have a natural frequency lower than 1 Hz. Figure 3-13 shows that wind load most often acts in those frequencies:

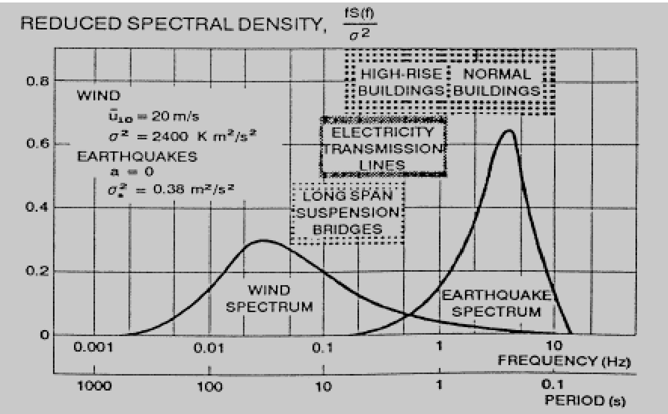


Figure 3-13: wind- and earthquake spectrum [46]

Foundation
The reference projects (almost) all have a pile with slab foundation. Pile depths varying from 46 to 80 meters and pile diameter is about 1.5 m. All pile types were drilled/driven cast-in-place concrete piles. The dimensions of the concrete slabs vary from 3-5 meters.

Local conditions Dubai - earthquakes
Dubai is an area where earthquakes can occur. The Dubai Municipality specifies Dubai as an UBC97 2a seismic region with a seismic zone factor Z=0.15 and soil-profile SC (soft rock).

Building shape
From the Burj Khalifa project an important conclusion can be drawn: the shape of the building (Y-shape) reduces the wind load on the structure. The wind direction also has an influence on the magnitude of the forces and moment. Wind blowing on the tips of the building gives smaller base forces and moments.

Rotating floors
No projects which can be compared to the Rotating Tower are ever built. All projects have rotating floors (and not storeys) or consist of only few storeys. These projects can be used when designing the driving mechanism or investigating the human comfort level due to the rotation, but do not provide any information about the effect of rotating floors on the structure.

4 Basis of design

This chapter contains the basis of design for the stability structure of the Rotating Tower. The entire design of the stability structure is based on this chapter.

4.1 Project information

A more detailed overview of the project information is given in chapter 2 of this report. This section gives an overview of subjects which are important for the design process.

4.1.1 Scope

The scope of this basis of design is the stability structure. A conceptual design of this structure is already made by the architect. This structure will be checked on validity and adapted if necessary.

4.1.2 Project location

Dubai marina, United Arab Emirates.

4.1.3 Building dimensions

See appendix A for the overall building dimensions. The general overview:

Specifications	Magnitude	Unit
Height tower	435.3	m
Average height floors	5.4	m
Number of floors	80	-
Area floors	1142-1826	m^2

4.1.4 Building functions

The building functions are based on the drawings given in the appendix A.

Function	Floors	Elevation
Parking	-3 to -1	-18m to 0m
Office	0 to14	0m to 74.6m
Hotel	15 to 35	74.6m to 188m
Restaurant	36 to 37	188m to 198.8m
Residential	38 to 80	198.8m to 432m

4.2 Design assumptions

The subjects given in this section are the main assumptions used throughout the entire design process.

4.2.1 Codes

All NEN codes are used in combination with the National Annex for the Netherlands (for educational reasons).

Code	Title	Date
NEN 2443	Off street and multi-storey car parks	2000
NEN 6702	Technical principles for building structures: loadings and deformations	2007
NEN 6720	Regulation for concrete, structural requirements and calculation methods	1995
NEN-EN 1990	Basis of structural design	2010
NEN-EN 1991-1-1	General actions: densities, dead weight, imposed loads for buildings	2009

NEN-EN-1991-1-4	General actions: wind actions	2010
NEN-EN-1992	Design of concrete structures	2008
NEN-EN-1994-1-1	Design of composite steel and concrete structures	2005
UBC 97 Volume 1	Administrative, fire- and life-safety and field inspection provisions	1997
UBC 97 Volume 2	Structural engineering design provisions	1997

4.2.2 Structure

The structure of the Rotating Tower consists of a few main parts: a central core, a steel structure and a foundation. This is the structure as mentioned in the architectural design. In chapter 9 a few different alternatives with different structural systems will be given. For completeness of this Basis of Design the original structure is described.

Core
The maximum dimension of the core is given in the architectural design (see appendix A). The dimensions need to be respected, so the stability structure can be fitted into the architectural design.

Steel structure
The steel structure is already designed in an earlier stage. This design will be used as an assumption for the design of the stability structure. When necessary the design of the structural steel can be adapted to the stability structure, since the steel structure is now designed as a free structure hanging from the core.

Foundation
No design for the foundation is made yet. A preliminary design will be made for the foundation in such a way that the rotational stiffness of the foundation is sufficient to resist deformation caused by wind loads.

4.2.3 Limit states

The structure will be designed for both serviceability limit state and ultimate limit state. The load combinations used in both limit states are given in this chapter.

4.3 Preconditions

4.3.1 Architect

The storeys must be able to rotate up to 8 Beaufort wind force (wind speeds up to 20.7 m/s). Above these wind speeds the storeys are allowed to stop rotating. This precondition will be important in further stages of this report.

4.3.2 Occupancy category

The occupancy category is determined according to both Eurocode and UBC97. Both codes demand the determination of an occupancy category and have different notations.

NEN-EN 1990
Class A (residential), B (office) and C (meeting area).

UBC97
Occupancy category 4: standard occupancy structures.

4.3.3 Loads

Dead load
Dead load will be determined with both Eurocode and UBC97. Since both codes will be used (one for wind and one for earthquake loads) al loads need to be determined according to both codes.

Live load
Live load will also be determined with both Eurocode and UBC97.

Wind load
The wind load will be determined according to Eurocode 1 (NEN-EN1991-1-4). When the building shape can't be taken into account properly, additional research will be done to determine a representative wind load.

Earthquake load
The earthquake load will be determined according to UBC97. Dubai is considered a UBC zone 2a with soil type SC (soft rock).

Load from rotating storeys
No loads coming from rotation of the storeys are taken into account.

4.3.4 Load combinations

Load combinations are determined according to Eurocode 0 (NEN-EN1990) and UBC97. The following design assumptions are used with the determination:

- Building had Consequence & Reliability class 3 (K_f=1.1) – Eurocode 1
- Building is designed for strength design – UBC97

	Dead Load	Live load storeys	Wind load	Earthquake load
SLS	1.0	1.0	1.0	0.0
ULS	1.2	0.5	0.0	1.0
	1.3	Ψ_0* 1.65	1.65	0.0
	0.9	Ψ_0* 1.65 / 0	1.65	0.0

4.4 Requirements

4.4.1 Deformation

Total deformation building
The maximum deformation at the top of the tower is: h/500. This deformation is determined according to NEN6702. This is the total deformation consisting of three parts:

- Deformation of the structure (bending)
- Deformation caused by rotation of the foundation
- Second order effect

Deformation of 1 storey
The maximum deformation of one storey is: h/300.

4.4.2 Acceleration

The maximum acceleration is given in figure 4-1. This acceleration is determined according to NEN6702. Line 1 in the figure is valid for office buildings and line 2 for apartment buildings.

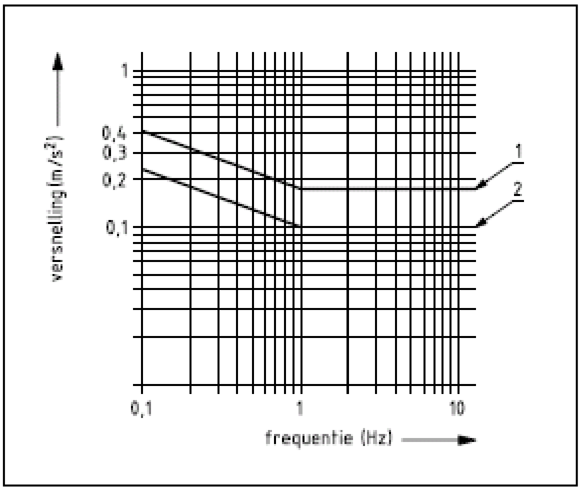


Figure 4-1: maximum accelerations

5 Design input

5.1 Structural design of the floors

The structural (pre-) design of the storeys is already made by Iv-Consult. Two systems have been developed:

1. Box structure

A box structure carries the total floor weight. This structural type is used for the top 10 storeys (the villas), because it is not preferable to have structural elements within the area of the villas. All structural components are situated at the edges and bottom of the floors. The downside of this type of structure is the storey height (5,15 meter and 700 mm space between floors). The weight of the structural steel (figure 5-2) is 5200 kN.

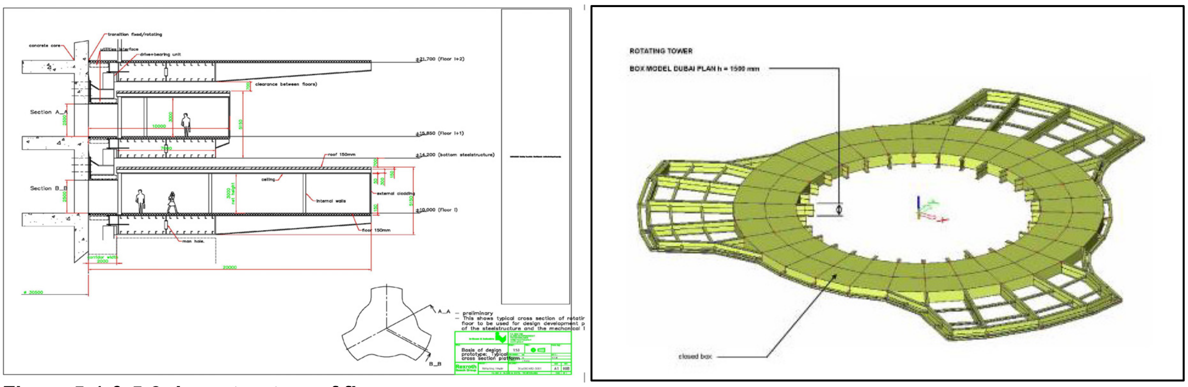


Figure 5-1 & 5-2: box structure of floor

2. Truss structure

The second structure which is designed is the truss structure. Trusses throughout the entire floor carry the loads. This type of structure is used on floors 0-70, since these floors do not require an open floor plan. Because of a more efficient use of structure the storey height can be reduced (4.9 meter and 700 mm space between the floors). The weight of the structural steel (figure 5-4) is 4500 kN.

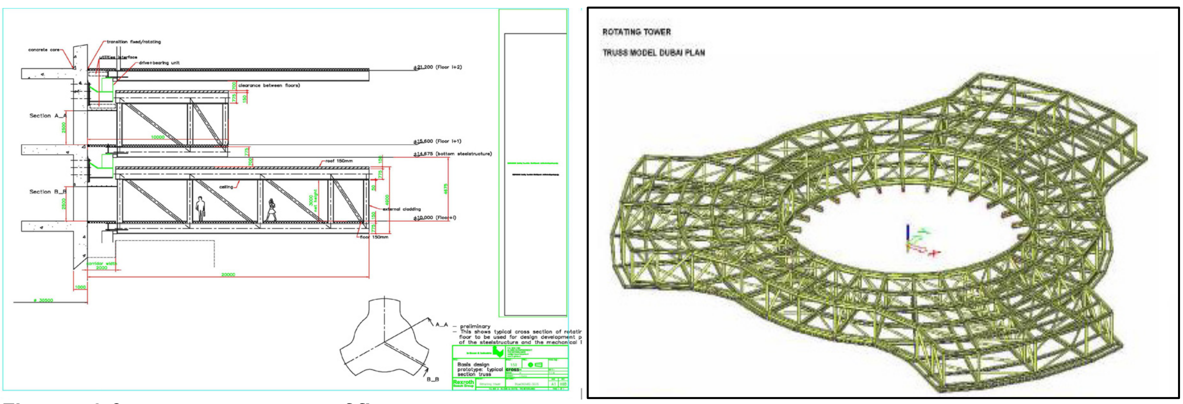


Figure 5-3 & 5-4: truss structure of floors

5.2 Driving system

The most important component in the concept of the Rotating Tower is of course the movement of the separate floors. Therefore a new system had to be developed to rotate the floors. Figure 5-5 and 5-6 show the main principle of this system. A rail is connected to the floors (the upper green part) and this rail rests on wheels which are connected to the core. To move the floors horizontal rotating wheels are connected to the core (black wheels in the figure).The wheels role against the floors and move the entire floor.
The full weight of the floors (approximately 600 to1000 tons per floor) rests on the wheels. Each wheel is designed to carry 50 ton (500kN) and will therefore introduce large local forces to the core.

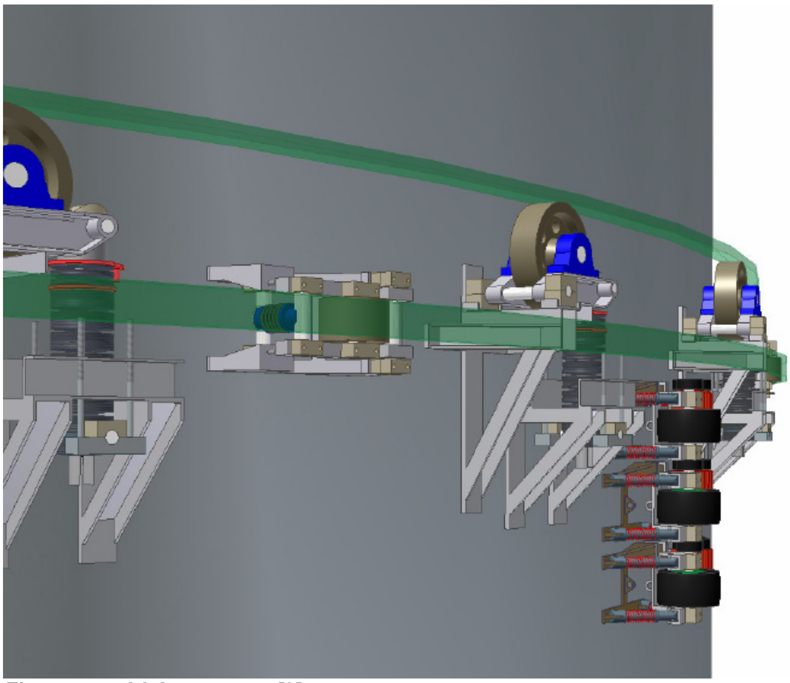


Figure 5-5: driving system [2]

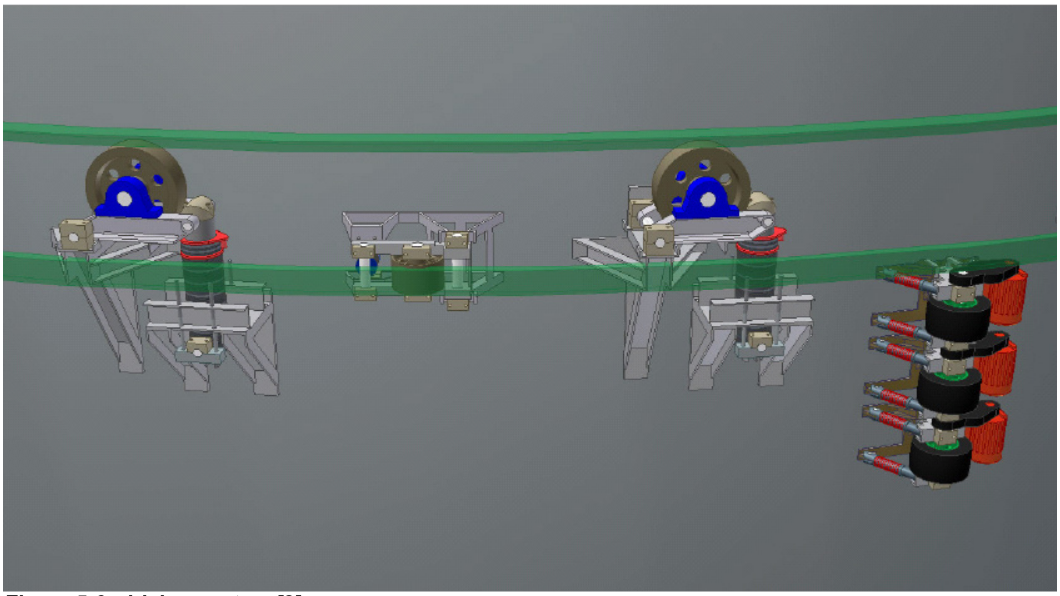


Figure 5-6: driving system [2]

5.3 Parking garage

To be able to make a detailed dead load calculation of the total building, the parking garage has to be considered. In the architectural design the global dimensions of the parking garage are given, but the structural design is lacking.

With general design rules a pre-design is made for the structural elements. It has to be stated that this section only gives an indicative design of the parking garage for the purpose of estimating the dead load of the garage. The design can be optimised in many ways. The reason the design is not worked out to a more detailed level, is because the layout of the parking garage will change in the different alternative designs given in chapter 9. This is just a general concept.

Floors

$$D = \frac{l}{25} = \frac{10.375}{25} = 0.4m$$

Beams

$$H = \frac{l}{15} = \frac{15.25}{15} = 1000mm$$

$$B = 0.5 * H = 500mm$$

Columns

$$\left.\begin{aligned} A_b &= 1.5 * \frac{N_{ed}}{f_{cd}} \\ N_{Ed} &= 2*(weight\ floor + weight\ beam + live\ load) \\ N_{Ed} &= 2*(10.375*15.25*0.4*24 + 1*0.5*15.25 + 10.375*15.25*2) = 3700kN \\ f_{cd} &= \frac{f_{ck}}{1.5} = \frac{25}{1.5} N/mm^2 \end{aligned}\right\} A_b = 154000mm^2$$

A column with a diameter of 450mm satisfies this demand.

To check whether the structural design is useful for the garage also a functional plan is made (also for indicative reasons). This plan is based on rules from the Dutch code NEN 2443. From this plan it became clear that it is possible to give the parking garage a functional use.

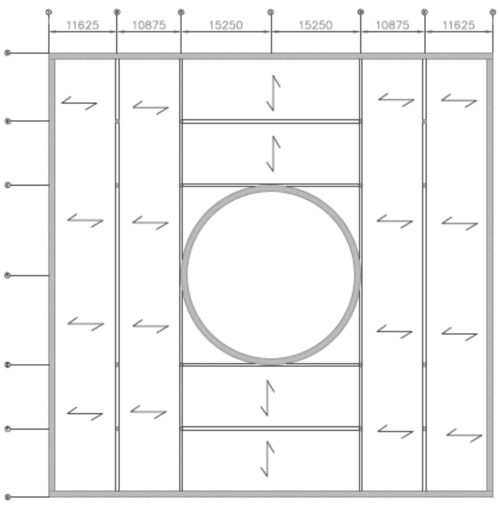


Figure 5-7: Structural design parking garage

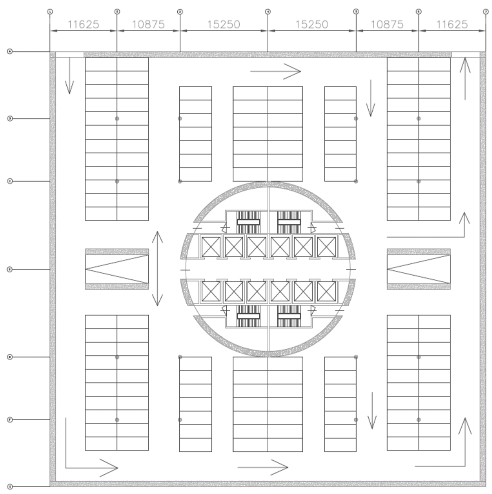


Figure 5-8: Functional plan parking garage

5.4 Foundation

In this section a pre-design is made for the foundation of the Rotating Tower. The most important goal of this design is to explore the feasibility of the foundation and to calculate the rotational stiffness of the foundation. This stiffness is of great importance for the rest of the calculations of the Rotating Tower, because a rotation of the foundation gives a deformation at the top of the tower. This foundation design won't be the final design, because the overall structural design will be changed

5.4.1 Local conditions

The table below shows the results of a cone penetration test (CPT) of another project built in Dubai.[9] The test was only performed for depths up to 12 meter. Because the foundation of the Rotating Tower will most likely be deeper, some additional information is needed.

TABLE 6: Soil Parameters Based on CPT Results

Test Reference	Depth (m)	Average Cone Resistance q_c (MPa)	Average Side Friction fs (MPa)	Friction Ratio Rf (%)	Φ	Unit Weight δ (KN/m^3)	Relative Density D_r (%)	Young Modulus E_s (MPa)	Constrained Modulus M (MPa)	Relative Density
CPT-03	0.0-0.60	7.21	0.136	1.88	31	15	23	21.63	26.03	Loose
	0.60-2.20	17.76	0.227	1.28	37	19	73	53.28	52.40	Dense
	2.20-3.60	6.29	0.152	2.41	30	15	19	18.87	23.73	Loose
	3.60-5.00	1.72	0.041	2.38	28	14	15	5.15	12.29	Very loose
	5.0-6.00	6.35	0.192	3.02	30	15	19	19.05	23.88	Medium dense
	6.0-7.20	2.02	0.026	1.28	28	14	10	6.06	13.05	Very loose
	7.20-8.80	6.69	0.105	1.57	30	15	19	20.07	24.73	Loose
	8.80-10.60	27.07	0.420	1.55	40	18	90	81.21	75.68	Very dense
CPT-09	0.0-0.80	19.64	0.395	2.01	38	18	81	58.92	57.10	Dense
	0.80-2.20	10.87	0.231	2.12	32	16	41	32.61	35.18	Loose
	2.20-4.20	3.84	0.141	3.67	28	14	10	11.52	17.60	Very loose
	4.20-6.40	1.27	0.054	4.25	28	14	10	3.81	11.18	Very loose
	6.40-9.00	7.05	0.128	1.82	31	15	23	21.15	25.63	Medium dense
	9.00-11.80	30.14	0.568	1.87	40	18	95	90.42	83.35	Very dense
	[illegible]	[illegible]	0.080	0.85	32	16	31	28.14	31.45	Loose

No other results concerning a CPT are available for Dubai, because different test methods are used (Standard penetration test –SPT). The table below (results of a SPT test for the Burj Khalifa [10]) shows the results of such a SPT test. It can be concluded that al layers below -12 meters consists of very dense materials, just like the last layer of the CPT test.

Stratum Number	*Description*	*RL Range DMD*	*Undrained Modulus E_u MPa*	*Drained Modulus E' MPa*	*Ultimate Skin Friction kPa*	*Ultimate End Bearing MPa*
1a	Med. Dense silty sand	+2.5 to +1.0	30	25	-	-
1b	Loose-v.loose silty sand	_1.0 to –1.2	12.5	10	-	-
2	Weak-mod. Weak calcarenite	-1.2 to –7.3	400	325	400	4.0
3	V. weak calc. Sandstone	-7.3 to -24	190	150	300	3.0
4	V. weak-weak sandstone/calc. Sandstone	-24 to –28.5	220	175	360	3.6
5A	V. weak-weak-mod. Weak calcisiltite/conglom.	-28.5 to –50	250	200	250	2.5
5B	V. weak-weak-mod. Weak calcisiltite/conglom	-50 to – 70	275	225	275	2.75
6	Calcareous siltstone	-70 & below	500	400	375	3.75

5.4.2 Design foundation

In the design of the foundation different alternatives were made and calculated. Most designs have a concrete slab with piles. The chosen design is shown in figure 5-9. Note that the foundation design given in this chapter is just a concept design used to calculate the rotational stiffness used in the orientation calculation. In chapter 9 different alternative designs for both the lateral load system and the foundation are given.

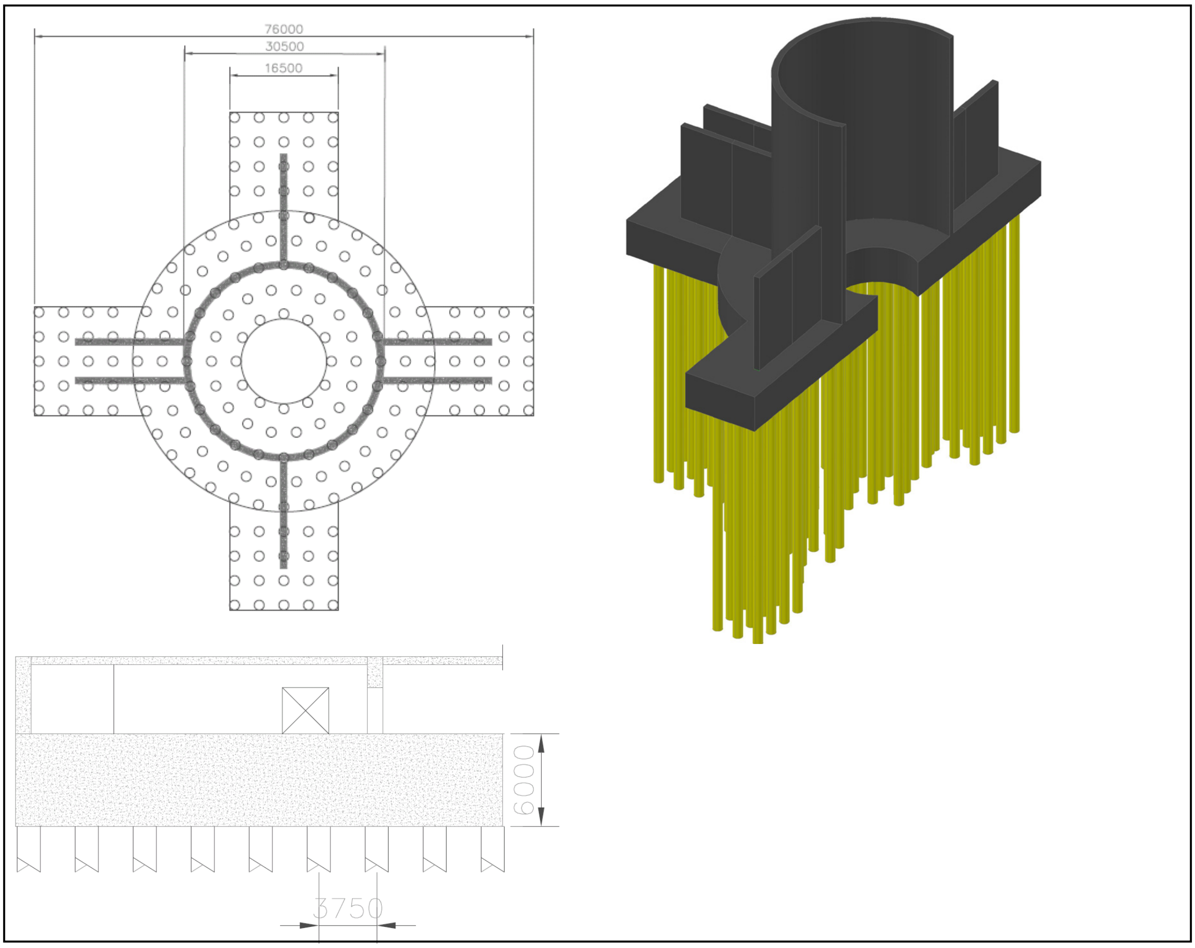


Figure 5-9: foundation design

The foundation is a combination of a concrete slab and piles. The concrete slab is 6 meters thick and the piles are driven cast-in-place concrete piles of 1500 mm diameter. The most important and difficult challenge in the design of the foundation was to spread the forces from the central core to the foundation. This is done by thickening the concrete slab to 6 meter (this thickness is necessary to create a slab with enough stiffness and make sure the forces from the core spread to enough piles) and attaching concrete shear walls to the core inside the parking garage. The shear walls create a wider base to spread the moments to more piles. The length of the pile will be calculated using the pile forces from chapter 7.

6 Load cases

This chapter gives an overview of all load cases used in the calculations of the Rotating Tower. The loads calculated in this chapter are:

- Dead load (with both Eurocode and UBC97)
- Live load (with both Eurocode and UBC97)
- Wind load (Eurocode)
- Earthquake load (UBC97)

6.1 Dead load

This section contains the determination of the total dead load of the building. The input for this calculation is the original dead load calculation performed by the structural engineers who designed the steel structure. The total dead load is the same for both Eurocode and UBC97, because material properties are the same in both codes. The difference between the codes is the safety factors used in the calculations.

6.1.1 Tower

The dead load of the tower is calculated by considering the following parts:

- Steel structure storey
- Architectural part storey (cladding, ceiling etc.)
- Installations
- Water tanks
- Driving system
- Core (incl. internal walls and floors)

The dead load of the tower is **248 [kg/m^3]**, considering an average floor area of 1500 m^2, an average core area of 613 m^2 and a storey height of 5.4 m. This value for the dead load of the tower is calculated as follows:

$$\text{Dead load per m}^3 = \frac{\text{Total dead load}}{\text{Volume building}}$$

$$\text{Volume building} = \sum (A_{floor} * h_{floor}) = A_{average} * h_{building}$$

$$\text{Dead load per m}^3 = \frac{2235869\ [kN]}{(1500+613)\ [m^2]*435\ [m^1]} = 2.43\ [kN/m^3] = 248\ [kg/m^3]$$

A detailed calculation and the values used in this section are given in appendix B.

6.1.2 Parking garage

The dead load of the parking garage is calculated according to the pre-design given in Chapter 5. The different parts which are accounted for are:

- Core (incl. internal walls and floors)
- Floors
- Columns and Beams
- External walls
- Foundation slab

The total dead load of the parking garage is **615 [MN]**.

A detailed calculation is given in appendix B.

6.2 Live load

6.2.1 Eurocode 1

The tower contains different functions: office, hotel, restaurant and housing. The live loads for these functions are based on NEN-EN 1991-1-1. A detailed calculation is given in the appendix.

Function	Live load	Floors
Parking	2 [kN/m^2]	-3 to -1
Office	2.5 [kN/m^2]	0 to14
Hotel	1.75 [kN/m^2]	15 to 35
Restaurant	4 [kN/m^2]	36 to 37
Residential	1.75 [kN/m^2]	38 to 80
Stairs/elevators	2 [kN/m^2]	-3 to 80

Because the building has more than 2 storeys a reduction factor may be used for the live load:

$$\alpha_n = \frac{2+(n-2)*\psi_0}{n}$$
$$n = nr.\ of\ floors$$
$$\psi_0 = 0.4/0.5$$

Formula (6-1)

6.2.2 UBC97

The tower contains different functions: office, hotel, restaurant and housing. The live loads for these functions are based on UB97. A detailed calculation is given in the appendix.

Function	Live load	Floors
Parking	2.4 [kN/m^2]	-3 to -1
Office	2.4 [kN/m^2]	0 to14
Hotel	1.9 [kN/m^2]	15 to 35
Restaurant	4.8 [kN/m^2]	36 to 37
Residential	1.9 [kN/m^2]	38 to 80
Stairs/elevators	2.4 [kN/m^2]	-3 to 80

UBC97 also uses a reduction factor. For floors with an area larger than 14 m^2 a reduction factor may be used, see formula 6-2. The maximum value of the reduction is 60%.

$$R = r(A-13.94)$$
$$with$$
$$R = \text{reduction percentage}$$
$$r = 0.08$$
$$A = \text{area floor } [\text{m}^2]$$

Formula (6-2)

6.3 Wind load

The total calculation of the wind load is given in appendix B. The calculation of the wind load is based on NEN-EN 1991-1-4 (Eurocode 1). The use of this code will be checked with the local conditions of Dubai in this section.

6.3.1 Local conditions Dubai

Dubai is a location where strong wind speeds can occur. Figure 6-1 shows the largest gust wind speeds for different returning periods at a height of 10 meter above ground level.

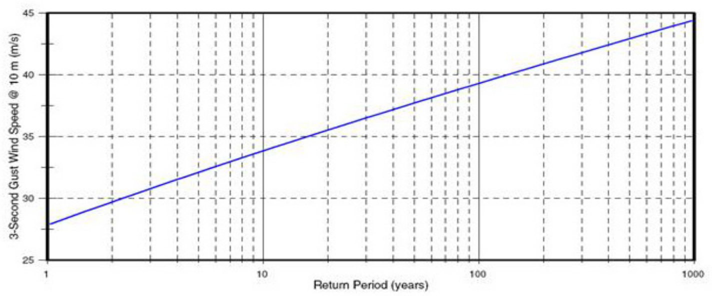


Figure 6-1: 3 second wind gust Dubai

Most of the time high wind speeds in Dubai come from one direction: W-NW. Figures 6-2 and 6-3 show that throughout the entire year strong winds come from 1 direction mainly. This is a fact which can be used in designing the Rotating Tower. Since the tower can rotate in every direction, the tower can be placed in the most favourable position to resist these winds.

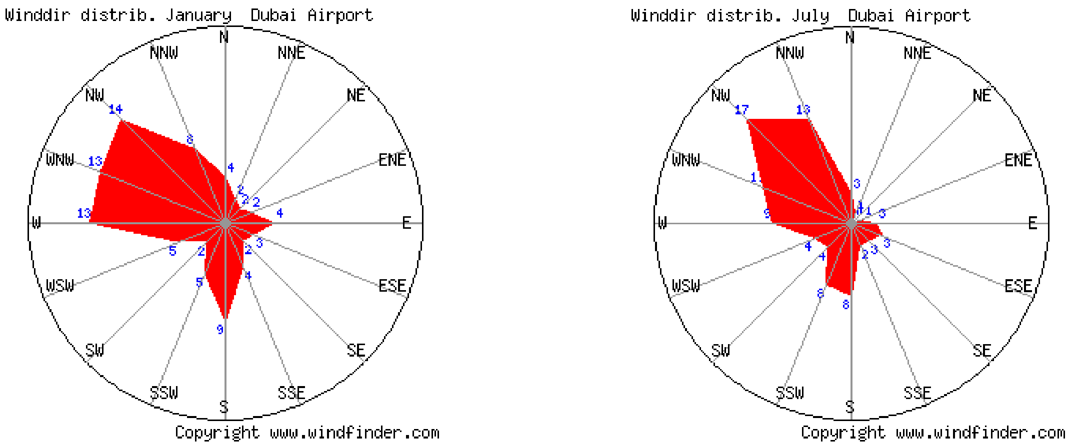


Figure 6-2: wind direction Dubai January and July [47]

6.3.2 Wind pressure

To calculate the wind pressure two factors need to be determined:

- Wind area (I, II or III)
- Terrain category (0, II or III)

Because Dubai is located in a coastal area, wind area I and terrain category 0 is chosen. This corresponds to a coastal area in the Netherlands. The maximum wind speed and extreme wind pressure can be calculated by the following formulas (with factors for wind area I and terrain category 0):

$$Windspeed$$
$$V_m(z) = C_r(z) * C_0(z) * V_b$$
$$Extreme\ windpressure$$
$$q_p(z) = (1 + 7 * I_v(z)) * 0.5 * \rho * V_m{}^2$$

Formula (6-3) & (6-4)

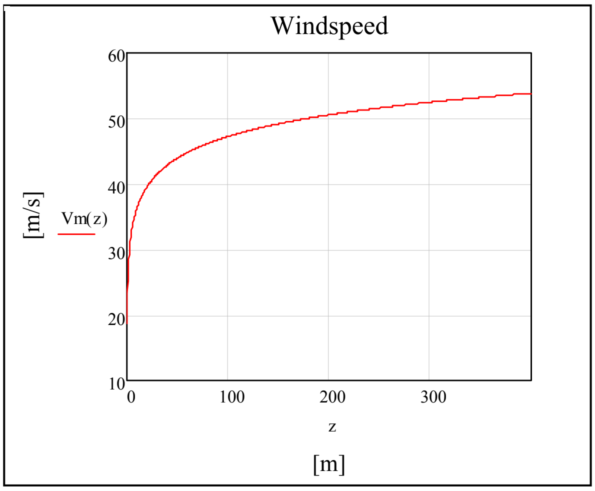


Figure 6-3: wind speed

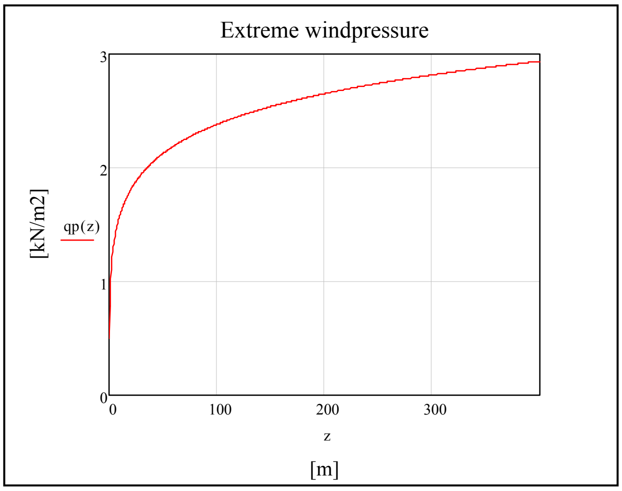


Figure 6-4: extreme wind pressure

6.3.3 Wind load

The wind force according to Eurocode 1 is calculated with the following formula:

$$F_w = c_s c_d * c_{pe} * q_p(z_e) * A_{ref}$$
$$with$$
$$q_p(z_e) = wind\ \ pressure\ at\ reference\ height\ z_e$$

Formula (6-5)

With this formula the wind force for a reference height z_e is calculated. In this formula the wind load is taken uniform over a height of z_e. Because this is a simplification of reality and because it uses a too high value of q, the wind load in this section is calculated differently. Instead of calculating the wind force, the wind load is calculated. By multiplying the wind pressure by the building width instead of the building reference area, the wind load (which is a distributed load and not a point load) is calculated.

$$q_w(z) = c_s c_d * c_{pe} * q_p(z) * b(z)$$
$$with$$
$$c_s = shape\ \ factor \qquad c_d = dynamic\ \ factor$$
$$c_{pe} = pressure\ coefficient \qquad q_p(z) = windpressure$$
$$b(z) = building\ \ width$$

Formula (6-6)

6.3.4 Building shape

With the Eurocode wind loads for different building shapes can be calculated. In this section 3 different building shapes are calculated with Eurocode 1 and the results are compared to choose the most suitable. The Rotating Tower has a floor plan which is not described in the Eurocode, therefore a comparison is also made with results from literature about wind tunnel testing with 4 different building shapes (square, Y-shape, triangle and circle). Finally the effect of the wind direction on the total wind load is studied by looking at a reference project with a comparable shape.

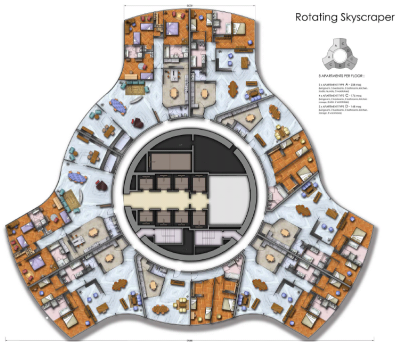


Figure 6-5: Floor plan Rotating Tower

Eurocode

When calculating the wind force with the Eurocode, different building shapes can be used. The effect of the building shape is given by a difference in the pressure coefficient (c_{pe}). The values for three building shapes are:

Shape	Pressure coefficient
Square	1.5
Hexagon	1.15
Circle	0.5-0.84 (depending on Reynolds number)

Comparison building shape

Figure 6-6 gives the results of a study where different building shapes were tested in a wind tunnel (Hayashida & Iwasa, 1990). The figure shows the maximum displacement for the same wind load of different building shapes. The floor plans tested all have the same area (so the widths of the plans are not the same).

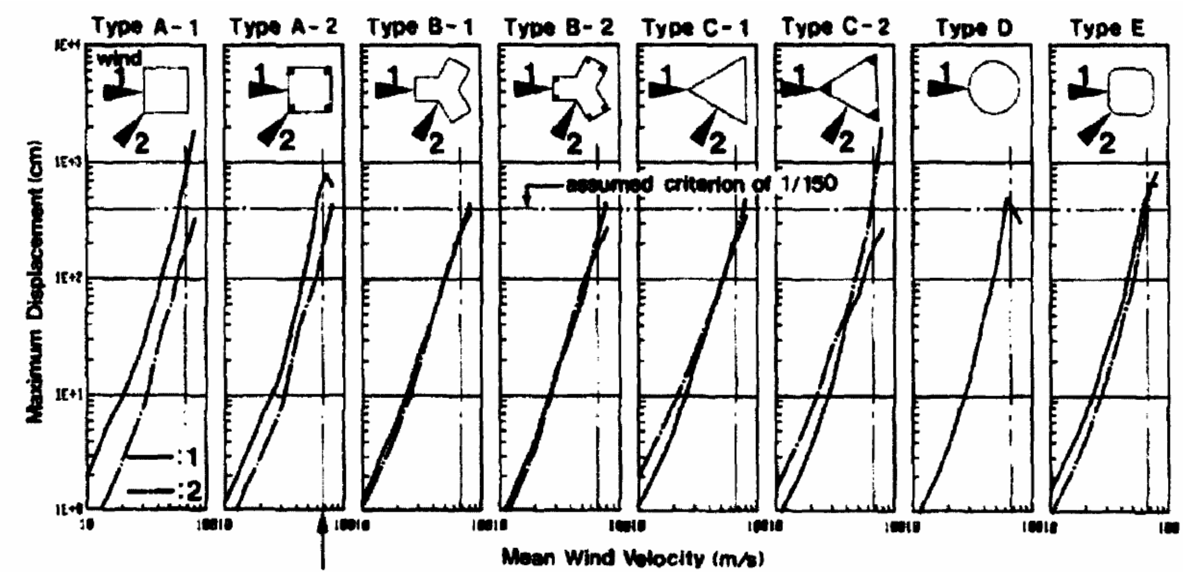


Figure 6-6: test results wind tunnel test (Hayashida & Iwasa,1990) [8]

The results (given in figure 6-6, note the logarithmic-scale of the y-axis) show that a Y-shaped floor plan experiences a displacement which is over 50% smaller than for a square floor plan. The displacement of a tower is directly dependent of the load, so this means the experienced wind load will also be over 50% smaller. It also turns that out a Y-shaped floor plan is comparable to a circular and triangular floor plan regarding the maximum displacements.

Configuration

In the last study to investigate the effect of the building shape on the total wind load, the Burj Khalifa project is investigated. A wind tunnel test was performed on the tower, where the base shear force (in both x and y direction) for different wind angles was measured. The results are given in figures 6-8 and 6-9. When adding the two component together (figure 6-10) the total base shear force is calculated. Figure 6-10 clearly shows 3 peaks and 3 lows. The lows are present when wind blows on the wings of the building and peaks when the wind blows between the wings. The difference in base shear force between the peak and lows is 33%. Positioning the tower in a good orientation reduces the total wind load by 33%.

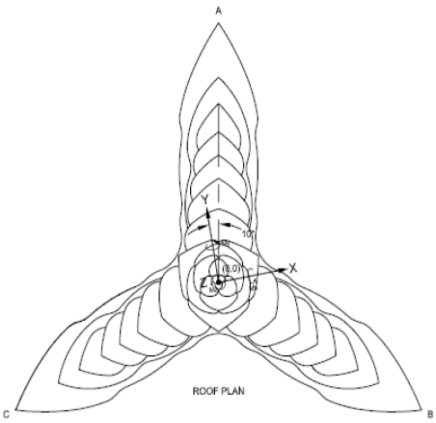


Figure 6-7: Building shape Burj Khalifa[6]

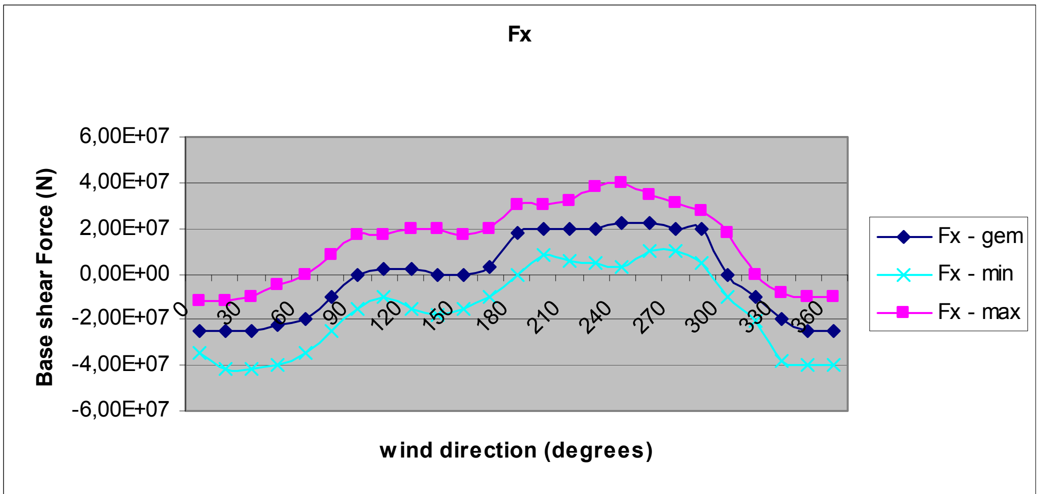


Figure 6-8: Base shear force x-direction

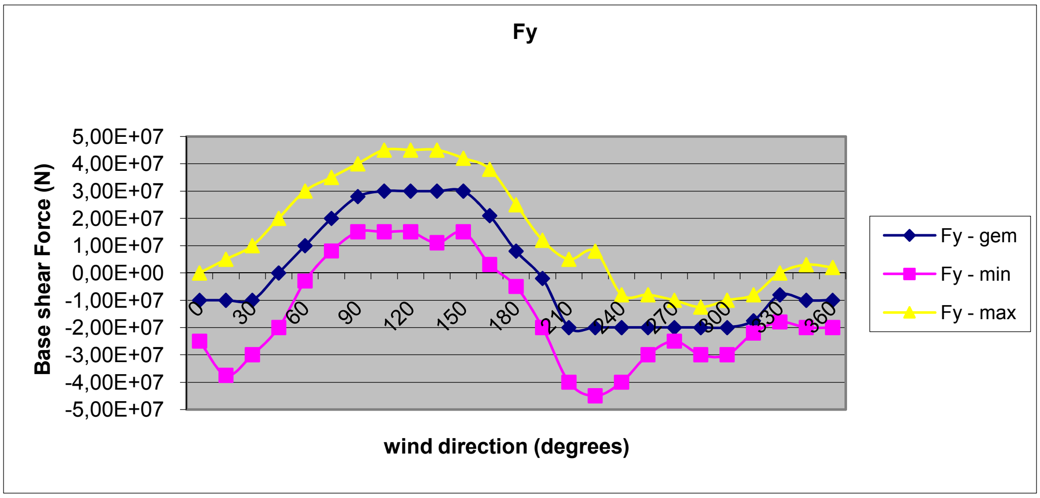


Figure 6-9: Base shear force y-direction

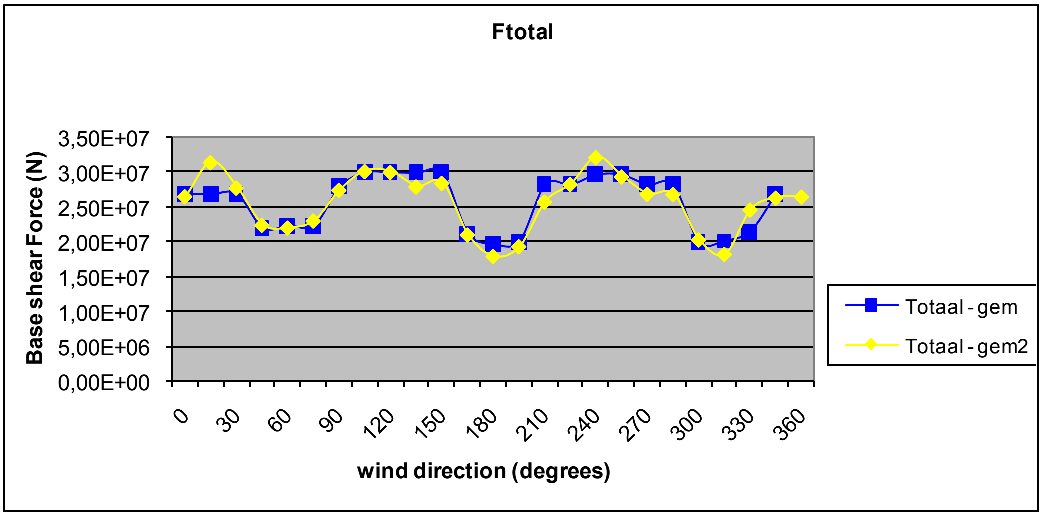


Figure 6-10: Total base shear force (angle with respect to x-axis in figure6-7)

<u>Conclusion</u>
The building shape of the Rotating Tower (comparable with a Y-shaped plan) will have an effect on the total wind load. Based on the results of the literature study, the pressure coefficient C_{pe} will be taken equal to 1.0. This is a 33% reduction in comparison with the pressure coefficient of a square floor plan.
This value is based on the results of the wind tunnel research conducted on the Burj Khalifa. The Rotating Tower can be rotated into every position possible. So when necessary the tower will be positioned with one of the wings straight into the wind (wind coming from the top in figure 6-5). This position is the most aerodynamic position possible. It is expected that the reduction of the wind load (when the tower is rotated in a favourable position instead of the least aerodynamic position) will be comparable to the result found in the wind tunnel research of the Burj Khalifa (which has a comparable shape). It is obvious that the floor plan of the Burj Khalifa is more aerodynamic than the Rotating Tower (the wings of the Burj Khalifa are shaped more aerodynamic than the wings of the Rotating Tower). The value of 33% reduction (in comparison with an square floor plan) will still be used, based on the research of Hayashida and Iwasa. This research shows an even larger reduction in the wind force when comparing an Y-shaped floor plan to a square one. This larger reduction indicates that a Y-shaped building is more aerodynamic than a square shaped floor plan, even when the wind is not blowing in the most favourable direction possible (this is validated by figure 6-6.)
It has to be stated that a wind tunnel test is compulsory for a tower with a height of over 400 meters. When this test gives results higher than the wind load used in this report, there are still a few adaptations which can be used to lower the wind load. An example is chamfering the corners (this is also done in the Taipei 101 tower). In the Taipei 101 tower this adaptation in the design caused a 25% reduction on the overall base shear force (this is also confirmed in figure 6-6).

6.3.5 Check local conditions

Because this calculation is based on local conditions from the Netherlands it must be compared with local conditions in Dubai. To do so another project in Dubai is compared with the outcome of the wind load calculation.
For this check the Burj Khalifa (see reference projects) is chosen. For the design of this tower a maximum wind speed of 36.4 m/s was taken for the height of 10 meter in open field (for a 50 year return period – see figure 6-1). The calculation with the Eurocode gives a wind speed of 36.3 m/s at a height of 10 meter. The difference is so small, that it's safe to use the calculated values.

6.3.6 Result

Because the wind load is a function of the building height, any tower with a different height has its own wind load graphic. Figure 6-11 shows the wind load for the Rotating Tower for a height of 400 meters. The decrease of the wind load above 200 meter is due to the decreasing building width.

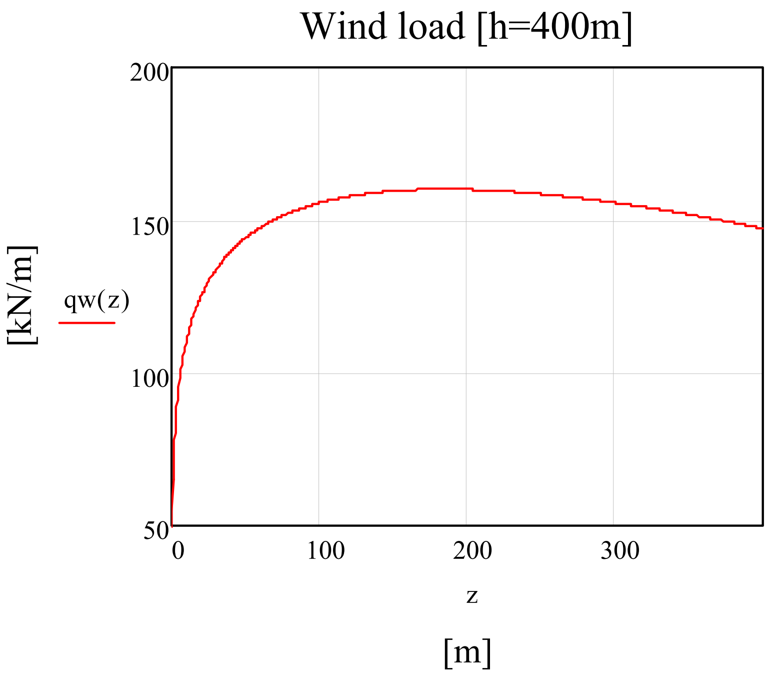


Figure 6-11: wind load for RT up to 400 m

One important issue that needs to be mentioned is the range of the Eurocode. Eurocode 1 is valid for buildings up to 200 meter. In the calculations in this section the formulas are extrapolated up to 400 meter, because no other codes are available for heights over 200 meters.

6.4 Earthquake load

Dubai is an area where earthquakes can occur. Because wind and earthquake loads are mostly governing in the design of a high rise tower, both loads are important to analyze. In this section the earthquake loads are based on local conditions of Dubai. The calculations are based on the Uniform Building Code 1997 (UBC97).

6.4.1 Earthquake

An earthquake occurs when two tectonic plates slip past each other. Normally the boundaries of the plates are smooth and the plates can easily pas each other, but sometimes a fault occurs in the boundary. The boundaries of tectonic plates will get stuck together, while the rest of the plates are still moving. The energy that would normally cause the plates to slide past one another is being stored up.
When the forces moving the plates overcome the friction of the fault, all the stored energy is released causing seismic waves.
Two types of seismic waves can occur:

- P- Waves (primary waves): fast travelling waves causing the earth crust to move horizontally. These waves can travel trough all media.
- S- Waves (secondary waves): slow travelling waves causing a wave pattern in the earth crust. This type of waves can’t travel trough liquid.

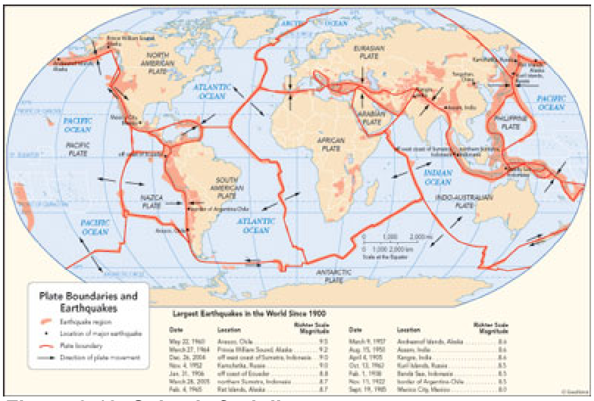


Figure 6-12: Seismic fault lines

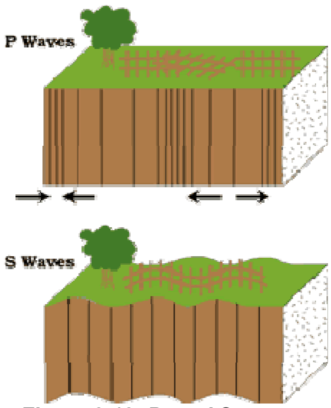


Figure 6-13: P- and S waves

6.4.2 Local conditions

Figures 6-12 and 6-14 show that Dubai lies near the edge of a tectonic plate and is therefore an area sensitive to earthquakes. The Dubai Municipality specifies Dubai as an UBC97 2a seismic region with a seismic zone factor Z=0.15 and soil-profile SC (soft rock). UBC97 specifies 4 seismic regions (1-4) with 1 being the lowest and 4 the highest.

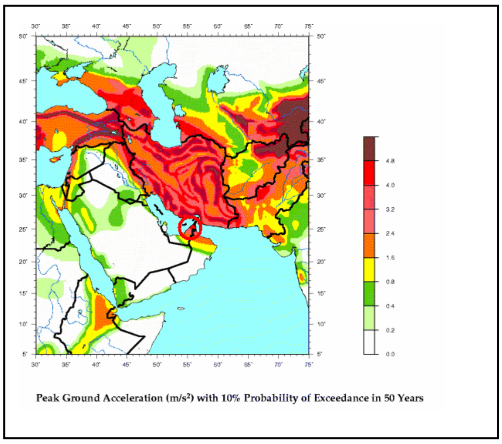


Figure 6-14: seismic hazard map [48]

6.4.3 Earthquake load

Because the building is located in seismic zone 2a and has an occupancy category 4 (standard occupancy structure), it is allowed according to the UBC97 to check the structure statically only.

Design Base shear
The design base shear force is calculated as a percentage of the dead load of the structure. This percentage depends on 3 constants (which are determined according to the structure type and seismic zone) and the elastic foundation period (T).
The choice of the structure type is important in this calculation method, because it influences the Ductility factor. The structure type chosen is: Building Frame system as basic structural system and concrete shear walls as lateral force resisting system.
The formula used to calculate the design base shear force:

$$
\begin{aligned}
&V = \frac{C_v * I}{R * T} * W \\
&\mathit{With} \\
&C_v = 0.25 = \text{Seismic coefficient} \\
&I = 1.00 = \text{Seismic importance factor} \\
&R = 5.5 = \text{Ductility coefficient} \\
&T = C_t (h_n)^{3/4} = \text{Elastic fundamental period of vibration} \\
&W = \text{Seismic Dead Load Structure}
\end{aligned}
$$

Formula (6-7))

Figure 6-15 shows the percentage $\frac{V}{W}$ as a function of the height of the structure.

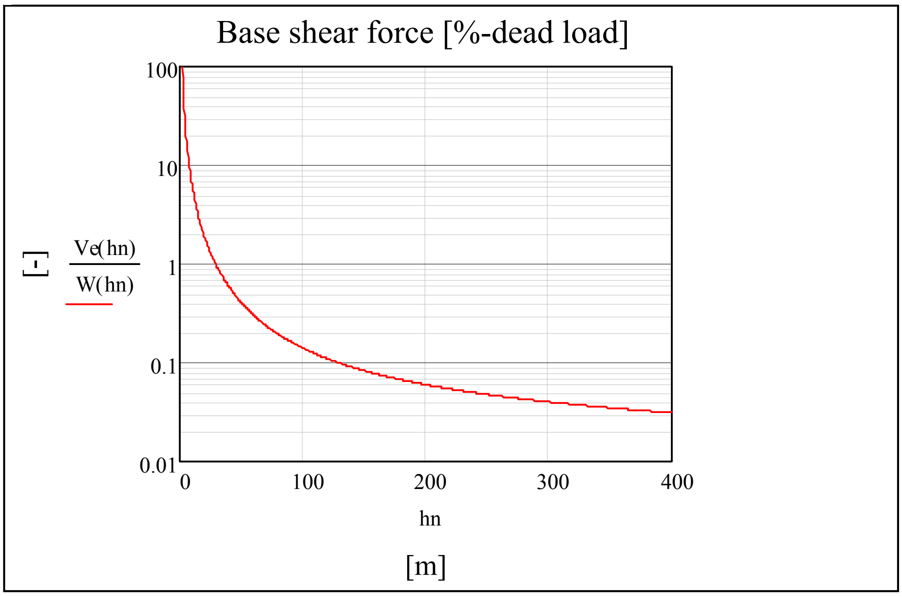
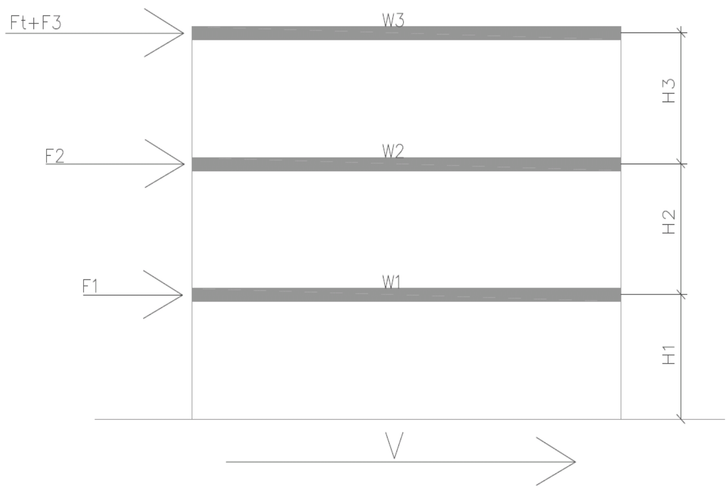


Figure 6-15: Base shear force as a percentage of dead load

Vertical load distribution
The vertical load distribution of earthquake loads is calculated following the individual masses of the storeys. The distribution is calculated as follows:



$$V = F_t + \sum_{i=1}^{n} F_i$$

with

$V = \text{Total base shear force (eq 6.7)}$

$F_t = 0.07 * T * V = \text{Force at the top}$

$$F_i = \frac{(V - F_t) * w_i h_i}{\sum_{i=1}^{n} w_i h_i}$$

$h_i = \text{storey height}$

$w_i = \text{storey weight}$

$n = \text{number of storeys}$

$i = \text{storey number}$

***Figure 6-16: Vertical load distribution EQ load and formula* 6-8**

6.4.4 Load combinations

According to the UBC97 different load cases need to be checked. The different load cases that need to be checked are:

1) $D + 0.5L + E$
2) $0.9D \pm E$

$D = \text{dead load}$

$L = \text{live load}$

$E = \text{earthquake load} = \rho * E_{hor} + E_{ver}$

In the above mentioned earthquake load the horizontal part is equal to the design base shear (V from Formula 6-7) and the vertical part is taken as a percentage of the dead load:

$$E_{ver} = 0.5 * C_a * I * D = 0.09 * D$$

Formula (6-9)

6.4.5 Sanity check

In this section a short check is performed to both wind- and earthquake loads. Both forces are likely to be approximately of the same order of magnitude for the Rotating Tower.

In figure 6-17 the design shear force of both earthquake (red) and wind (blue) loads are compared over the height. It is clear that the forces are of the same order of magnitude. It can also be concluded that the shear force in a smaller tower is bigger for earthquake load than for wind load. For a taller tower the wind load is governing for the base shear force.

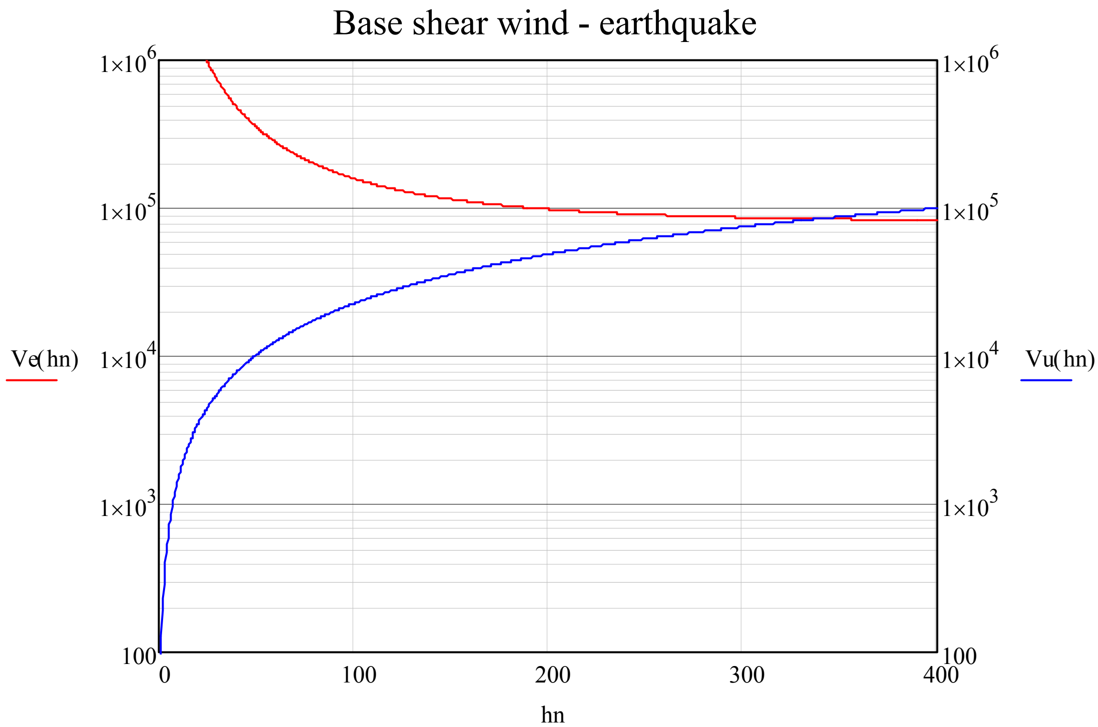


Figure 6-17: Comparison base shear force wind (blue line) and earthquake (red line)

6.5 Rotating floors

All storeys in the Rotating Tower rotate around the central core. In the design of the core it is assumed that this rotation does not have an effect on the core. The storeys are considered as being static structural parts hanging from the core and no forces are induced to the core caused by the rotation (no torsional moments).
Because the storeys are connected to the core by rails, the gravity loads are induced to the core at one location per 5.4 meter (1 storey height). This will create great local forces to the core which will have to be considered in the local design calculations.

7 Current design

In this chapter an orientation calculation is made for the current design. The orientation calculation consists of 3 main parts: rotational stiffness of the foundation, deformation calculation and maximum stress calculation. The summary and results are given in this chapter; the total calculation is given in appendix C. In this orientation calculation the core is assumed to have a constant diameter of 30.5 meter (this makes the calculation easier and still gives a good view of the reaction of the structure to the different load combinations).

7.1 Load combinations

In this orientation calculation the following load combinations are considered:

$\underline{SLS}$	$\underline{ULS}$	$D = dead\ load$
$D + L + W$	$1.2D + 0.5L + 1.0E$	$L = live\ load$
	$1.3D + 1.65L + 1.65W$	$W = wind\ load$
		$E = \rho E_{hor} + E_{ver}$

For deformation only wind load is considered and for strength design wind and earthquake loads are considered. In appendix B the calculation of these loads is given. For both cases wind load was governing.

7.2 Rotational stiffness foundation

The rotational stiffness of the foundation will be calculated following the next steps:

- Calculate maximum pile load (hand calculation)
- Calculate spring stiffness of pile with Mfoundation
- Calculate pile displacement under core
- Calculate rotational stiffness of foundation.

In chapter 9 a more detailed description of the calculation method is given (to keep this chapter conveniently arranged).

Loads

The foundation is designed to withstand the following loads:

- Wind load/earthquake load (overturning moment)
- Dead weight of the core (both above ground level and in the parking garage)
- Dead and live load of the storeys
- Dead weight of the foundation slab

Maximum pile load

The maximum pile load caused by wind load (overturning moment) is calculated with a computer model (for in/output see appendix). The pile force caused by dead+ live load:

Dead load

248 [kg/m^3]* 435 [m]* (1500+613) [m^2] *9.81[m/s^2] *10^{-3} + 44924 [kN] + 220201 [kN] = 2501319 [kN] per 200 piles = (12.5 MN per pile)

Live load

1990 [kN/floor] *45 [floors] = 159200 [kN] per 200 piles (=0.8MN/pile)

Load	SLS	ULS
Wind load (moment)	8.4 [MN/pile]	14.4 [MN/pile]
Wind load incl. second order(moment)	--	25.8 [MN/pile]
Dead + live load	13.3 [MN/pile]	17.6 [MN/pile]

Characteristics piles

Part	Result
Pile Type	Round concrete cast-in-place pile, tube back by driving
Dimension	Diameter 1.5 [m] ; Length 50 [m]
Slip layer	Bentonite
Bearing capacity (limit state 1B)	45.6 [MN]
Deformation due to total load (limit state 2)	0.029 [m]
Deformation due to total load excl. wind load (limit state 2)	0.017 [m]
Spring stiffness	1050 [MN/m]

Rotational stiffness

To calculate the rotational stiffness of the foundation, the found pile stiffness due to wind loading is put in the calculation model of the foundation (which takes the thickness of the concrete slab into account). Two aspects from the output are relevant: the maximum pile forces (which need to be checked with the pile forces taken in the previous calculation) and the maximum displacement of the foundation. Figure 7-1 shows the maximum displacement of the foundation slab. The maximum displacement of the slab underneath the core is equal to 7.3 mm. Using this value, the minimum value of the rotational stiffness will be found.

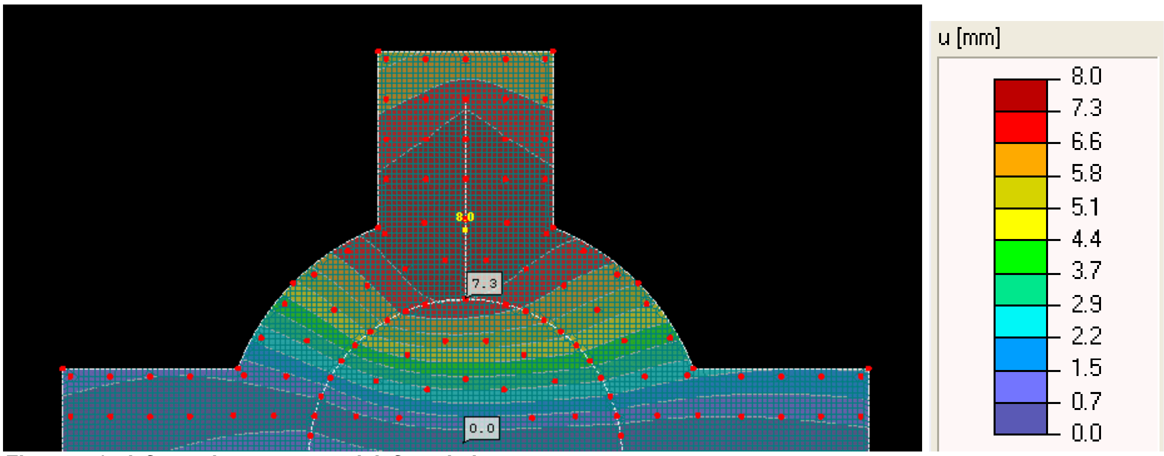


Figure 7-1: deformation concrete slab foundation

$$\left.\begin{array}{l} C = \dfrac{M}{\varphi} \\ with \\ \varphi = \dfrac{\delta}{14.75} = 4.95*10^{-4}\,rad \\ \delta = 0.0073m \\ M = 14510MNm \end{array}\right\} C = 2932*10^{4}\,MNm/rad$$

Formula (7-1)

7.3 Deformation (SLS)

The deformation of the tower exists of 3 components:

1. Deformation caused by rotation of the foundation
2. Deformation caused by bending of the core.
3. Second order effect

This is schematised in the figure and formula below:

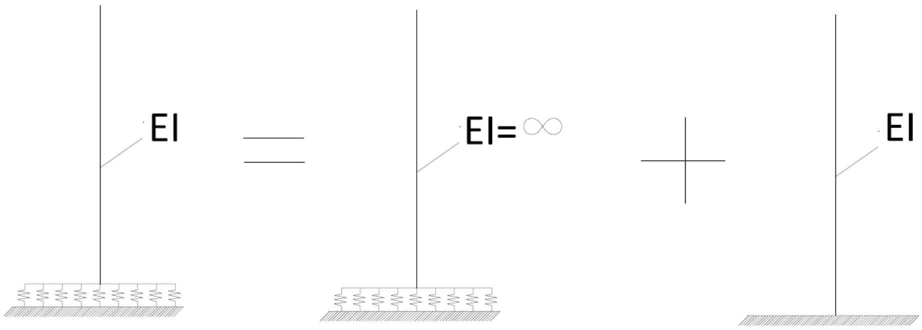


Figure 7-2: mechanics model core + foundation

$$\delta_{total} = (\delta_{rotation} + \delta_{bending}) * \frac{n}{n-1}$$ **Formula (7-2)**

Deformation by rotation of the foundation

The deformation which is caused by the rotation of the foundation is calculated with the results of section 7.2.

$$\begin{aligned}
&\delta_{rotation} = h * \varphi \\
&with \\
&h = height\ tower\ [m] \\
&\varphi = \frac{M}{C}\ [rad] \\
&M = overturning\ moment\ [MNm] \\
&C = rotation\ stiffness\ \ foundation\ [MNm / rad]
\end{aligned}$$ **Formula (7-3)**

With the rotational stiffness derived in section 7.2 the maximum deformation caused by rotation of the foundation is **226 mm.**

Deformation by bending of the core

In this calculation only deformation caused by bending is taken into account. It is assumed that the deformation caused by shear is very small. The derivation of formula 7-4 is given in appendix G.

$$\delta_{bending} = \frac{\int_0^h\int_0^h\int_0^h\int_0^h q\,dx}{E_f I} + \frac{Ax^3}{6E_f I} + \frac{Bx^2}{2E_f I} + \frac{Cx}{E_f I} + \frac{D}{E_f I}$$

$$\begin{aligned}
&with\\
&q = \text{Distrubuted load (wind)}\\
&A,B,C,D = \text{integration constants}\\
&E_f = \tfrac{1}{3} * 10^3 * f'_{ck} = 20000 N/mm^2 \ (C50/60)\\
&I = \text{Moment of inertia core}
\end{aligned}$$

Formula (7-4)

With the given dimensions of the core the deformation caused by bending is **3760 mm.**

Second order effect (n/(n-1))
First step is to check whether a second order effect calculation needs to be made:

$$\begin{aligned}
&\frac{n}{n-1}\\
&with\\
&n = \frac{N_{cr}}{N'_{vd}}\\
&\frac{1}{N_{cr}} = \frac{1}{N_{cr;1}} + \frac{1}{N_{cr;2}}\\
&N'_{vd} = 1.3 * V_g + 1.65 * V_q
\end{aligned}$$

Formula (7-5)

With the given dimensions of the core the second order effect is 1.80. This calculation indicates that the second order effect is larger than 10% and has to be taken into account.

Total deformation
The maximum total deformation is: **7185 mm**. The maximum allowed deformation by Eurocode is 870 mm (1/500 * h). It is clear that the Rotating Tower **does not meet this requirement in the current design**.

7.4 Strength (ULS)

Normal force
Normal force acting on the core consists of dead weight of the structure, dead load and live load from the rotating storeys. All these load are given in chapter 5. The total normal force acting on the governing section of the core (the lowest point):

<u>ULS wind max</u>
1.3 * 248 [kg/m^3] *(1500 +613) [m^2] * 435 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 44924 [kN] + 1.65 * 1990 [kN/floor] * 80 [floors] = **3228MN**

<u>ULS wind min</u>
0.9 * 248 [kg/m^3] *(1500 +613) [m^2] * 435 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 44924 [kN] + 0.0* 1990 [kN/floor] * 80 [floors] = **2053MN**

ULS earthquake max
1.2 * 248 [kg/m^3] *(1500 +613) [m^2] * 243 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 44924 [kN] + 0.5 * 1711 [kN/floor] * 45 [floors] = **2806MN**

ULS earthquake min
0.9 * 248 [kg/m^3] *(1500 +613) [m^2] * 435 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 44924 [kN] + 0.0* 1990 [kN/floor] * 80 [floors] = **2053MN**

Wind load (ULS)
$M_{wind;2th\ order}$ =**4.309*10^7 kNm**

Earthquake load (ULS)
$M_{earthquake}$ = **1.665*10^7 kNm**

Stresses
Governing load case for this height is wind load. The maximum moment acting on the core is larger than for earthquake load. The maximum stresses acting on the structure:

$$\sigma = \sigma_n \pm \sigma_m$$
$$\sigma_n = \frac{N'_{vd}}{A} = \frac{3228*10^6}{0.95*92.7*10^6} = 36.65 N/mm^2$$
$$\sigma_m = \frac{M'_{vd;2thorder}*e}{I} = \frac{43100*10^9*15250}{0.9*1.009*10^{16}} = \pm 72.4 N/mm^2$$

Formula (7-6)

The calculation has a result of a minimum stress of -35.75 N/mm^2 (tension) and a maximum stress of 109 N/mm^2 (compression). Both values are much higher than the allowable values for hardly any concrete grade. This implies that high reinforcement ratios need to be used (probably much more than the maximum allowable reinforcement ratio of 4%).

Indication reinforcement ratio

The reinforcement percentage is calculated with GTB table 10.4 f (see appendix). To use this graph two quantities need to be calculated:

$$\frac{N}{f_{cd}*A_c} = 1.1$$
$$\frac{M}{f_{cd}*A_c*h} = 0.48$$

Formula (7-7)

These values fall outside the range of the graph. This means reinforcement ratios of more than 8%. It can be concluded that this is not feasible.

7.5 Conclusion

The orientation calculation made it clear the architectural design of the lateral load structure does not meet any of the requirements given in the codes. Both deformation and maximum stresses are far outside the range of realistic values for an tower with this height. To make the Rotating Tower a feasible project, the design of the lateral structure has to be adapted. In the next chapter different solution possibilities are investigated. With the results of this optimization analysis, different designs will be made for alternative structures. The main goal of these designs is investigating the feasibility of the Rotating Tower concept.

8 Optimization analysis

The orientation calculation made it clear the Rotating Tower does not meet any of the requirements according to the codes (Eurocode and UBC97). Both deformation and stresses acting on the core are too high. Solutions to lower both values are taken into account in this optimization analysis. Figures 8-1 and 8-2 give an overview of the different solution possibilities for creating higher stiffness and strength.

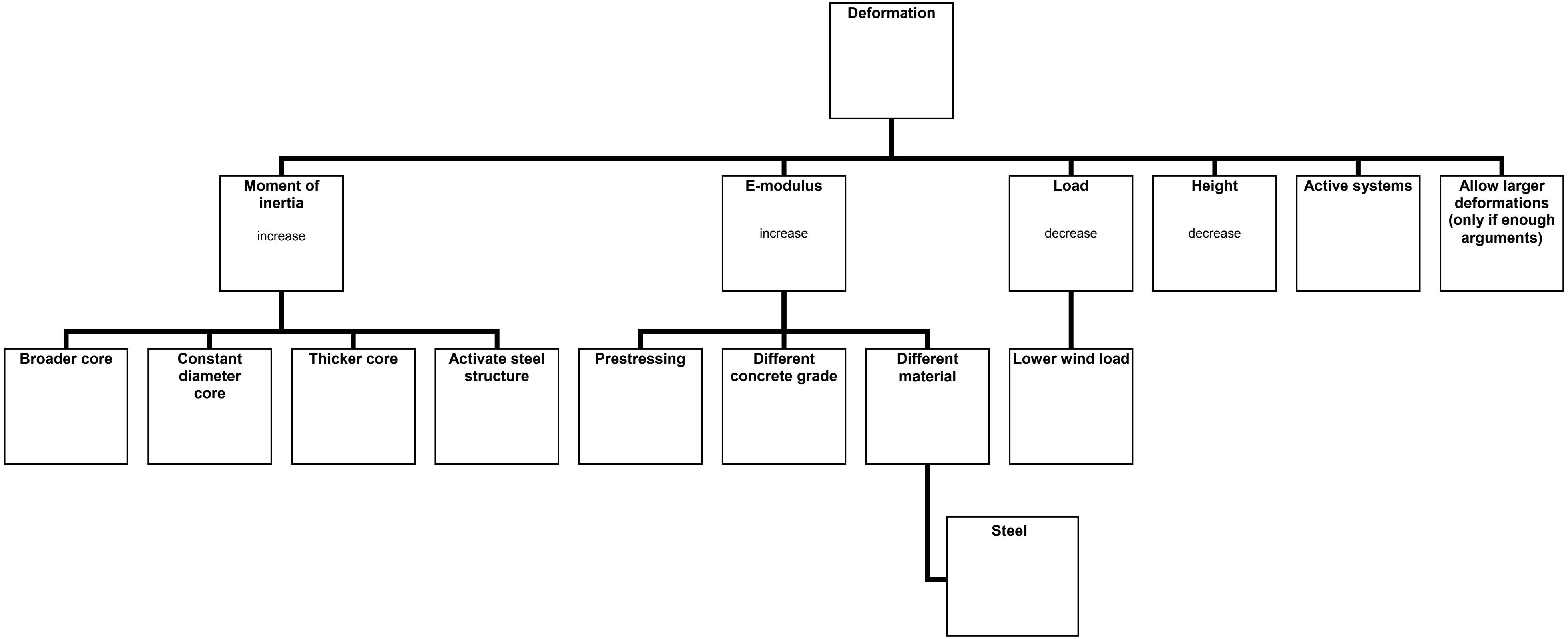


Figure 8-1: optimization analysis deformation

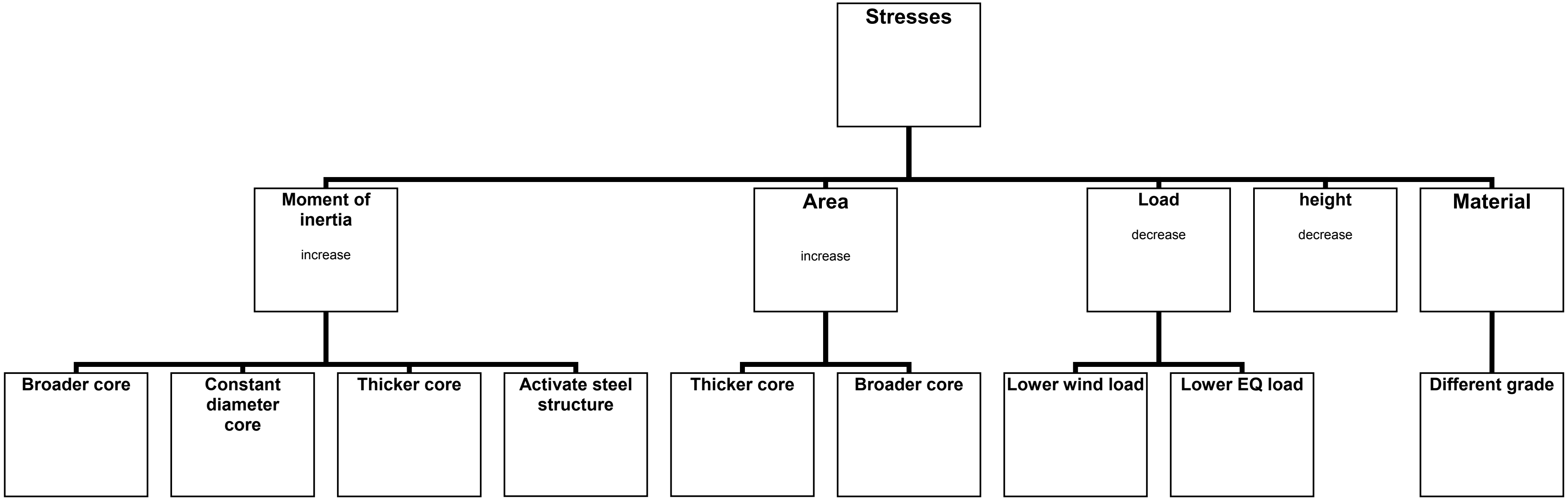


Figure 8-2: optimization analysis stresses

8.1 Deformation

Figure 8-1 gives a scheme of all the different possible solutions. In this section the different solutions will be considered with more detail. The main solution possibilities (explained in this section) are based on the formula for the deformation (formula 7-4).

8.1.1 Moment of inertia

One possibility to reduce the deformation of the tower, is increasing the moment of inertia of the structure. The moment of inertia is one of two quantities determining the stiffness of the structure. Four possible solutions are given in this section.

1. Wider core

A good solution to increase the moment of inertia of the core is making it broader (an increased diameter). By increasing the diameter 2 times the moment of inertia increases 8.4 times. Broadening the core is therefore a very effective solution to stiffen the core (and lateral load structure). But this solution has an unfortunate effect on the design of the tower. Making the core broader and keeping the total area per floor equal, also broadens the overall width of the building. This means that the total building weight and wind load will increase too. Beside this technical effect, also the character of the building changes. The ratio between the height and width will change, making the tower look less slender.

2. Thicker core wall

Another obvious solution is making the core wall thicker. The solution is less effective than broadening the core, but the effect is still considerable. When thickening the core wall from 1 to 2 meters, the moment of inertia increases with a factor 1.8. Increasing the thickness to 3 meters gives an increase of 2.5 times the original moment of inertia.
The main advantage of this solution is the fact that the architectural design does not have to be changed. All dimensions and functions stay the same. A disadvantage of this solution is a higher dead weight of the structure making it sensitive for large second order effects.

3. Constant diameter

The core was calculated with a constant diameter in chapter 7. When using a variable core diameter the deformation will increase even more. Because of economical reasons it will be necessary to consider a variable core diameter in the design alternatives.

4. Steel structure

In the current design, the steel structures of the storeys are merely dead loads hanging from the core. The structure is used only to carry loads coming from the storeys (dead load, live load). The overall lateral forces (wind and earthquake) are resisted by the concrete core. A good solution for decreasing the deformation of the tower, is to use the steel structure also to resist lateral forces. Using the steel structure within the lateral load structure creates a wider base, which means a stiffer structure.

One option in using the steel structure in the overall structure is creating an outrigger structure with perimeter columns.
The effect of an outrigger structure depends on a few important parameters (see also figure 8-3):
1. The area of the perimeter columns (A_c)
2. The moment of inertia (bending stiffness) of the outrigger (I_o)
3. The distance between the perimeter columns (L)
4. E-modulus of the outrigger
5. E-modulus of the columns.
6. Number of outriggers.

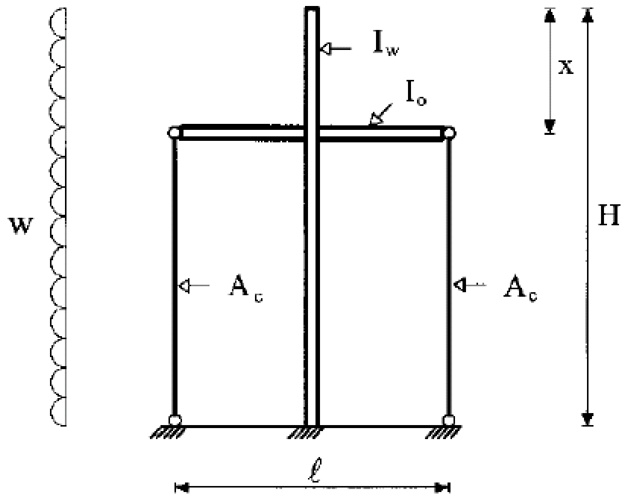


Figure 8-3: outrigger model [21]

When an outrigger system is used the deformation may be reduced:

$$y_{red} = \frac{M_c(H^2 - x^2)}{2EI_w}$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w}\left\{\left(\frac{1}{EI_w} + \frac{2}{EA_c * l^2}\right)(H - x) + \left(\frac{l}{12EI_o}\right)\right\}^{-1}$$

Formula (8-1) [21]

The sensitivity of the structure depends on the design of the outrigger and perimeter columns. Outrigger structures with a clever design are able to decrease the deformation with 40% of the total deformations.

8.1.2 E-modulus

Another possibility to reduce the deformation of the tower is to increase the (design) E-modulus of the core. This can be done in 3 ways:

1. ***Higher concrete grade***

By using a higher concrete grade, the E-modulus can be increased. In the orientation calculation a grade C50/60 was taken. When taking a grade C90/105 (the highest concrete grade according to NEN-EN 1992-1-1), the E-modulus becomes 1.4 times larger. This increase is by far not enough to reduce the deformation of the tower to meet the requirements, but it might be useful when combining this solution with others.

2. ***Other material***

The only other material useful for designing an overall stability structure is steel. The E-modulus of steel is instead 9 times larger than concrete C50/60, so it seems like this is a perfect solution. This is however not the case. Changing from concrete to steel as construction material, will cause a whole different design of the core.
It is not useful and possible to make a steel core with the same dimensions (so also the same moment of inertia) as the concrete core. When using steel for the core, it is most likely that the core will be constructed as steel braced columns. Because the steel columns will have a much smaller moment of inertia than the concrete core, the effect of using steel instead of concrete will be minimal for the deformation of the tower (because the deformation depends on both E and I).

3. Prestressing

Another possibility to increase the E-modulus of concrete is using prestressing. When applying prestressing, the uncracked E-modulus can be used instead of the cracked E-modulus. The uncracked modulus is approximately 1.5 times higher than the cracked modulus.
This is only a possibility in combination with a tower with a decreased height, because pre-stressing means increasing the compression stresses on the tower. Because of the high dead weight of the Rotating Tower it is most likely that in the final designs the core won't experience (high enough) tensile stresses. When this is the case pre-stressing is not useful, because the concrete is already uncracked. This assumption will be checked in the alternative designs.

8.1.3 Load

The last factor determining the deformation of the tower are the load cases acting on the tower. Wind load is the governing (and only) load case concerning deformation of the tower.

1. Wind load

From the formula for deformation of the tower it can be seen that the deformation is directly dependent of the wind load. Although it is not an option for the calculation of the alternative designs to reduce the wind load on the structure, it is useful to investigate the sensitivity to the wind load. In 1 alternative there will be calculations made with two different wind loads (maximum and below 8 Beaufort).
The orientation calculation made it clear that for the maximum wind speed (36.4 m/s at a reference height of 10 meter = 12 Beaufort) the tower does not meet the demands for deformation. Roughly it can be said that the deformation caused by bending (= 3760mm) is 5 times too large. So when the wind load decreases 5 times the tower will meet the demands. Lowering the wind load 5 times means the wind speed will have to be lowered $\sqrt{5}$ =2.24 times. When the wind speed is equal to 36.4/2.24 = 16.4 m/s (= 7 Beaufort) the tower meets the demands given for deformation.

8.1.4 Height

One of the most obvious solutions is lowering the tower. Because the deformation of a clamped in beam is calculated with the length to the power 4, lowering the length has a large effect on the deformation of the tower. For instance when taking half the height of the tower the deformation will become (2^4 =) 16 times smaller.
In this particular case the deformation will become even smaller, because the wind load depends on the height. The higher the tower gets, the higher the distributed load. Because of this effect not the full height reduction might have to be applied.
One more thing needs to be considered when looking to this subject: the deformation requirement of 1/500 * L. When the tower height becomes two times smaller, the maximum allowed deformation will also become 2 times smaller. Just lowering the tower is not enough; also the deformation requirement has to be taken into account.
This solution is the most effective of all. Just a small decrease in height has a large effect on the deformation.

8.2 Stresses

Although the strength (stresses) is not governing in the orientation calculation, the calculated stresses are still too high.
In this section 5 different solution possibilities are given and explained. Most solutions are discussed in the previous section (height, moment of inertia and load), but have a different effect on stresses as they do on deformation.

8.2.1 Moment of inertia

To lower the stress, not only the moment of inertia is important, but the maximum eccentricity of the core (e) as well. The ratio e/I has to decrease approximately by a factor 3 to get allowable stresses (caused by bending moment).
Lowering the stresses caused by bending moments (in other words increasing the moment of inertia) has an effect only when also stresses caused by normal forces are reduced.

1. Wider core

Broadening the core has a direct effect on both e and I. The eccentricity and moment of inertia both increase, only the moment of inertia increases much faster (because it depends on the width to the power 4). Broadening the core is a very effective way of increasing the moment of inertia, only a few important disadvantages are present in this solutions (see section 8.1.1).

2. Thicker core wall

The effect of thickening the core is already explained in section 8.1.1. Another advantage of thickening the core wall is the increase of the cross-sectional area. The effect of this enlargement will be explained further on.

3. Constant diameter

Using a constant diameter does not have any effect on reducing stresses in the core. The maximum stresses will be governing. These stresses will appear at the lowest point of the core. This cross section of the core will still have the same diameter as the cross section used in the orientation calculation.

4. Steel structure

Using the steel structure in the overall lateral load structure creates a situation where the total overturning moment will be divided over the core and the steel structure. This will cause a decrease on the total moment on the core.
For instance when using an outrigger structure the total overturning moment may be reduced with the moment caused by the outrigger. Formula 8-1 gives this moment. In figure 8-4 a visualization of the moment lines in a structure without outriggers, a structure with one outrigger and a structure with 2 outriggers is given.

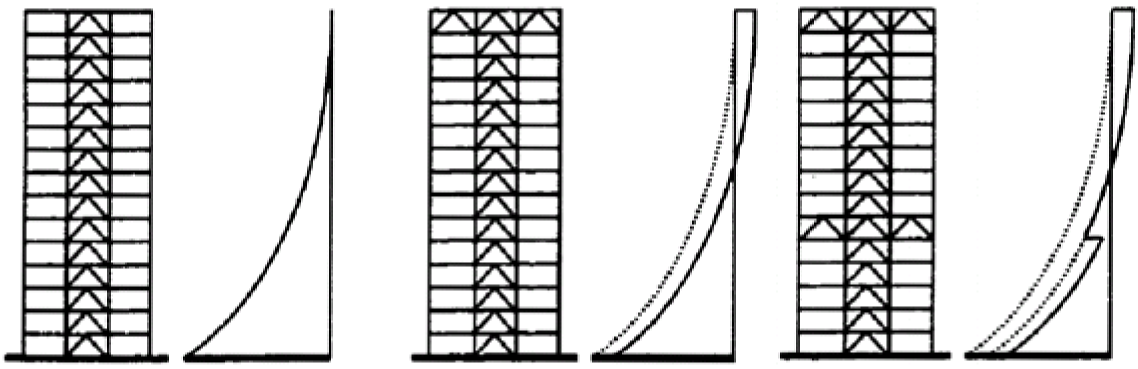


Figure8-4: moment lines outrigger system [24]

8.2.2 Area

To decrease the stress (caused by normal force), the area of the core has to be increased.

1. Wider core

Making the core broader does not change the total area of the core much. For every meter the core gets broader, the area increases 3%. Although it is very effective in increasing the moment of inertia, for increasing the resistance against normal forces broadening the core is not an useful solution.

2. Thicker core

Thickening the core is an effective way to lower the stresses in the core (both caused by normal force and bending moments), because it increases the area and moment of inertia. When increasing the core wall thickness from 1 to 2 meter, the total area will also increase 2 times.

8.2.3 Load

A third possibility to lower the stresses is lowering the moments on the core, or in other words lowering the load acting on the tower. Two load cases are discussed in this section: wind load and earthquake load. In this stage wind load is the governing load case, but when wind loads are lowered earthquake load can become governing.

1. Wind load

As stated before lowering the wind load is not an option used in the design calculation, but the sensitivity of the structure to wind load is useful to know. The wind load acting on the structure causes bending stresses. Only lowering the bending stresses is not sufficient to make sure the structure has enough strength. Also the normal stresses need to be reduced. Therefore it is hard to say how much the stresses need to decrease to make sure the structure meets the requirements.
To give an indication of the sensitivity of the structure the effect of lowering the wind load 3 times is given (when wind load decrease 3 times, the bending stresses do too). When the wind load decreases 3 times the wind speed will have to be $\sqrt{3}$ times smaller. This means a wind speed of 21 m/s (= 9 Beaufort). When bending stresses decrease with a factor 3, the tower meets its requirements (when also the normal stresses are lowered of course). So roughly it can be said the structure is sufficient to withstand loads up to wind force 9 Beaufort.

2. Earthquake load

When normal stresses in the structure are reduced, high tensile stresses might occur in the core. In this case the stresses caused by earthquake loading can become governing. Although it is not possible to decrease the load given by the code for UBC zone 2a, it is however possible to change the UBC zone. Dubai does not have its own code at this time, but is developing one. There is a good possibility that the earthquake zone of Dubai will be one comparable with UBC zone 1 instead of 2a. The current division of zones is given in figure 8-5. It is clear that Dubai is on the edge of zone 2a and 1, and it is therefore possible that Dubai will become a UBC zone 1 in the future.
Changing from zone 2a to 1 means a decrease of 48% in the base shear force (and moments) caused by earthquake load. When this change can be made, the earthquake load is not governing in any situation.

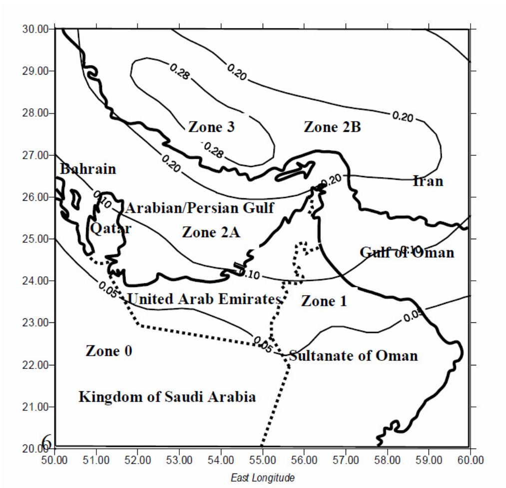


Figure8-5: earthquake zone map [16]

8.2.4 Height

Lowering the height turned out to be a good solution for decreasing the deformation of the tower. Also for strength design it is a good solution to reduce the stresses. By reducing the height of the tower both the normal forces (dead and live load) and overturning moment decrease. By reducing the height by 50%, the normal forces decrease with 50% and the overturning moment decreases with 75%. Reducing the height is very effective in decreasing stresses.

8.2.5 Material characteristics

The last possibility to modify the ability of the structure to withstand forces (stresses) is to use a higher concrete grade. When using a C90/105 grade instead of C50/60, the maximum value for the design compression strength increases 1.8 times and the maximum value for design tensile strength increases 1.24 times.
Although only changing the concrete grade is not enough to make sure the structure does not collapse, it can help a lot in finding the right solution.

8.3 Conclusions

The most promising solutions (only considering sensitivity to lower deformation and stresses) for creating a structure that meets the requirements are given in this section. Not al solutions will be used when designing alternatives for the structure. If a solution does not meet the requirements given by in the architectural design, it will not be used.

Reduced height
A good solution for lowering both stresses and deformation is a decreased height. It turns out that approximately 240m is a feasible height within the requirements for deformation (and maximum stresses). When combining this solution with other solution possibilities a bigger height can be realized. Lowering the height makes it necessary to change the arrangement of the tower. Still this solution will be used in the design of alternatives, because it does not change the floor plans and concept of the Rotating Tower.

Broader core
Broadening the core is a very effective solution to make the core stiffer (and decreasing the maximum deformations). But it is not a useful solution when looking at the architectural design and concept. Broadening the core changes the whole building. Different floor plans, a different slenderness of the tower and a lot of useless space inside the core. Because of these reasons, this solution won't be worked out to a more detailed level.

Thicker core
Although thickening the core is not as effective as broadening the core, it is still an useful solution. Making the core wall thicker reduces both deformation and stresses. Besides these technical advantages, this solution also keeps the architectural design unchanged (which is an important aspect). The only disadvantage of this solution is high dead weight.

Different concrete grade
Changing the concrete grade from C50/60 to C90/105 does not have a big effect on the properties of the core, but it is a solution which does not change any of the other parameters of the tower. Therefore it is a very useful solution.

Lowering wind load
Lowering the governing load on the building, the wind load, also has a great effect on the deformation and stresses. Although it is not possible to actually lower the wind load, it is a good tool to see to which wind speeds the building can be operable in the current situation (without taking extra measurements).

Activating steel structure
The last promising solution is using the steel structure as a structural component in the overall structure. When activating the steel structure as tension/compression components, a wider and stiffer structure appears. Using the steel structure in the overall stability structure will be a big challenge (considering the dynamic nature of the storeys), but is the only option for making a structure stiff enough to reach the total height of the tower (80 storeys). This solution will be used in designing alternatives.

9 Alternative designs

The results of the optimization analysis are translated into 5 alternative designs for the lateral load bearing structure:

Alternative 1: Architect's design
Alternative 2: Higher concrete grade
Alternative 3: Increased wall thickness
Alternative 4: Outrigger braced concrete core
Alternative 5: Perimeter columns with stiff floors

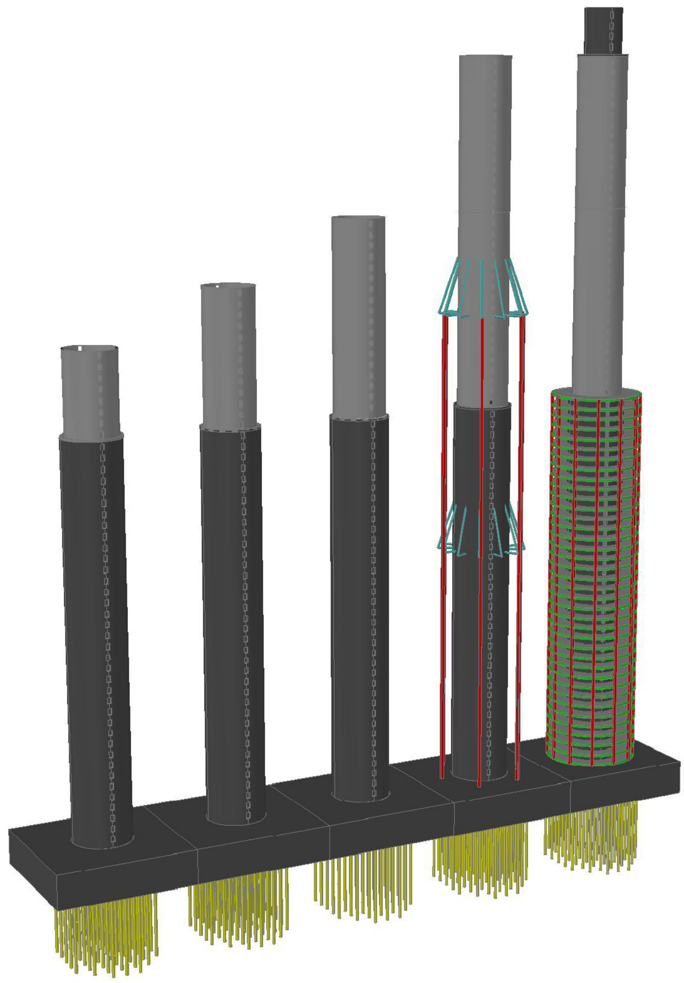


Figure 9-1: 5 alternative designs - (1) on the left, (5) on the right

Many concept designs were drawn in order to make the final alternatives. This chapter shows the final alternatives and some concept designs. The main focus in designing the alternatives was creating a stiffer structure (because deformation of the tower incl. a large second order effect turned out to be governing in the orientation calculation). This is mainly done by making the core itself stiffer (higher concrete grade and an increased wall thickness) and creating a wider base to withstand the lateral forces (outrigger braced core and perimeter columns with stiff floors).

All alternatives are described and calculated in this chapter. The first section gives al the used formulas and calculation methods. The following section gives a brief overview of al non feasible concept designs (in order to give insight in the different solution possibilities which are considered). The next sections describe the alternatives and give the calculation results. Detailed calculations are given in the appendix E of this report. Finally the alternative designs are compared and a general conclusion is drawn about the relevance of the alternatives to the project.

9.1 Formulas and calculation

This first section gives an overview of the formulas used in this chapter and explains the calculation method. The dead weight of the structure per alternative is also given. The formulas given in this section are used in the calculation of all alternatives. If specific formulas are used just for one alternative, the formulas are given in the section covering the alternative.

9.1.1 Rotational stiffness foundation

The calculation of the rotational stiffness is performed in a few consecutive steps:

- Determination of the maximum pile forces

The maximum overturning moment caused by the governing load case is put into a 3D calculation program (the program input/output per alternative is given in the appendix) to determine the maximum loaded pile. The normal force is assumed to be divided over the piles equally in the simple foundation layouts (alternative 1, 2 and 3). In the complex layouts (alternative 4 and 5) the division of the normal force is determined with the same computer model. The use of this model is an iterative method, because the spring stiffness of the piles is used in the model and need to be calculated with the result of the model (the force on the piles).

- Determination of the spring stiffness of the piles

With the program Mfoundation the maximum loaded pile is analysed. The desired output of the program is: the bearing capacity of the pile and the pile deformation (both with and without the load caused by wind load). With this pile deformation and the pile load, the spring stiffness is calculated with formula 9-1.

$$k_{pile;wind} = 1.5 * \frac{F_{wind}}{w_{total} - w_{total;perm}}$$

$$F_{wind} = \text{pile force due to wind load}$$

$$w_{total} = \text{ pile deformation due to total load (wind and dead/live)}$$

$$w_{total;perm} = \text{ pile deformation due to load excl. wind load}$$

Formula (9-1) [13]

- Determination of the rotational stiffness of the foundation

To calculate the rotational stiffness of the foundation, the pile stiffness derived for wind loading is put into the 3D calculation model of the foundation. The desired output of the model is: the maximum pile forces (which need to be checked with the pile force taken in the previous calculation) and the maximum displacement of the foundation. The maximum displacement of the foundation underneath the core is used to determine the rotational stiffness:

$$C = \frac{M}{\varphi}$$

$$with$$

$$\varphi = \frac{\delta}{\text{distance to centrepoint}}$$

$$\delta = \text{max displacement}$$

$$M = \text{ Max overturning moment}$$

Formula (9-2)

9.1.2 Normal forces

To calculate the normal forces used in different calculations, the tower is divided into three parts:

1. Tower above ground level
2. Core in the parking garage (0 to -16 meter under ground level)
3. Foundation slab

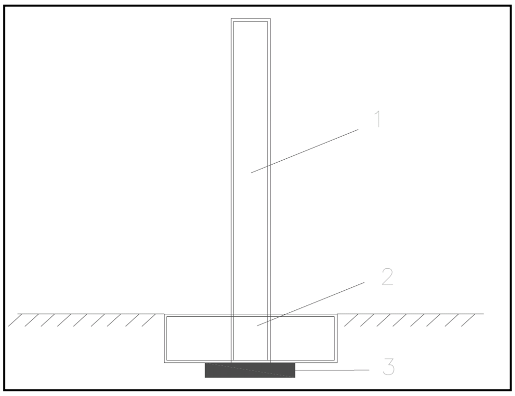


Figure 9-2: dead weight division

The full dead weight (incl. foundation slab) does not have to be taken into account for all calculations. The following table gives the calculations which have dead weight as input and state which part of the dead weight needs to be accounted for.

Calculation	Part of dead weight taken into account
Earthquake load	1
Wind load	1
2nd order	1
Maximum stresses core	1 + 2
Maximum pile load	1 + 2 + 3

The dead weight for the alternatives is different than calculated in chapter 6. Chapter 6 gives the dead weight of the core (both above and beneath ground level) for the original architectural design. Alternatives 1 and 2 have the same dead weight calculated in chapter 6. The other alternatives have a different (higher) dead weight. The dead weight of the different alternatives is given in the following table:

Alternative	Dead weight core (part 1)	Dead weight core inside parking (part 2)	Dead weight foundation slab (part 3)	Cause of higher dead weight in comparison with Chapter 4
Alternative 1	248 [kg/m^3]	44924 [kN]	183500 [kN]	--
Alternative 2	248 [kg/m^3]	44924 [kN]	183500 [kN]	--
Alternative 3	289 [kg/m^3]	62025 [kN]	220201 [kN]	- Increased wall thickness - Thicker foundation slab
Alternative 4	315 [kg/m^3]	62025 [kN]	358441 [kN]	- Increased wall thickness - 6 perimeter columns - Thicker foundation slab
Alternative 5	316 [kg/m^3]	62025 [kN]	358441 [kN]	- Increased wall thickness - 12 perimeter columns - Thicker foundation slab

9.1.3 Deformation

The total deformation of the structure has 3 components:

- Rotation of the foundation

$$\delta_{rotation} = h * \varphi$$
$$with$$
$$h = height\ tower\ [m]$$
$$\varphi = \frac{M}{C}\ [rad]$$
$$M = overturning\ moment\ [MNm]$$
$$C = rotational\ stiffness\ \ foundation\ [MNm/rad]$$

Formula (9-3)

- Bending

The formula used for deformation caused by bending of the core is derived in appendix G.

$$v(x) = \frac{\iiiint q(x)}{EI} - \frac{\int_0^l q(x)dx * x^3}{6EI} + \frac{\int_0^l q(x)dx * l * x^2}{2EI} - \frac{\int_0^l\int_0^l q(x)dx * x^2}{2EI}$$
$$q = \text{wind load}$$
$$\text{EI= bendig stiffness core}$$
$$\text{L= heigth tower}$$

Formula (9-4)

In all alternatives the EI of the core changes over the height of the tower. The deformation of an element with different (in this case 2) bending stiffness's is calculated with the following formula:

$$v_{total} = \frac{q*L_1^4}{8*EI_1} + \frac{q*L_2^4}{8*EI_2} + \frac{q*L_1^3L_2}{6*EI_1} + \frac{q*L_1^3L_2}{3*EI_1} + \frac{q*L_1^2L_2^2}{2*EI_1} + \frac{q*L_1^2L_2^2}{4*EI_1} + \frac{q*L_1L_2^3}{2*EI_1}$$

Formula (9-5)

This formula is valid for an element containing two parts loaded with an uniform load q:
- Part 1 with length L_1 and stiffness EI_1
- Part 2 with length L_2 and stiffness EI_2

This formula shows that the difference between the deformation of a core/column with a constant EI or with two different EI only depends on the bending of the second part of the core/columns (with EI_2). The deformation of the core with 2 different EI, is calculated by adding the "extra" bending of the upper part of the core to the calculated deformation with one constant EI. This extra bending is given in the results for deformation in every section.

- Second order effect

Also the second order effect might have to be taken into account. When the second order effect is below 10% (n<11), it does not have to be taken into account.

In this calculation the EI used is the "relative" EI. This EI is calculated by first calculating the total bending deformation (also taking into account the extra bending caused by the smaller bending stiffness of the core at the top storeys) and then calculate the "relative" EI of the total core (with different EI). In the calculations given in appendix E this "relative" EI is already used as input.

$$\delta_{bending} = \frac{w * H^4}{8EI_{relative}}$$
$$EI_{relative} = \frac{w * H^4}{8 * \delta_{bending;total}}$$

Formula (9-6)

$$\frac{n}{n-1}$$
$$with$$
$$n = \frac{N_{cr}}{N'_{vd}}$$
$$\frac{1}{N_{cr}} = \frac{1}{N_{cr;1}} + \frac{1}{N_{cr;2}}$$
$$N_{cr;1} = \frac{\pi^2 * EI_{relative}}{(1.12 l^2)}$$
$$N_{cr;2} = \frac{C}{0.5 * l}$$
$$N'_{vd} = 1.3 * V_g + 1.65 * V_q$$

Formula (9-7)

9.1.4 Acceleration

The tower will experience an acceleration due to a fluctuating part of the wind force. Wind force is not a constant value, but fluctuates over time. This effect creates considerable accelerations at the top of the tower. The acceleration is calculated according to NEN6702. The formula used is :

$$a = 1.6 * \left(\frac{\phi_2 * p_{w1} * C_t * b_m}{\rho_1} \right) \leq a_{\max}$$
$$\phi_2 = \sqrt{\frac{0.0344 * f_e^{-\frac{2}{3}}}{D * (1 + 0.12 * f_e * h) * (1 + 0.20 * f_e * b_m)}}$$
$$D = 0.015 = \text{damping coefficient}$$
$$\mathrm{f}_e = \frac{17.73}{H^2} * \sqrt{\frac{EI}{\rho_1}} = \text{natural frequency}$$
$$p_{w1} = 100 * \ln(\frac{h}{0.2}) = \text{fluctuating part of the wind load}$$
$$C_t = 1.2 \ [-] = \text{force coefficient wind}$$
$$b_m = 65 \ [m] = \text{average building width}$$
$$\rho_1 = [kg / m^1] = \text{building mass per meter}$$

Formula (9-8)

The force coefficient for wind load is taken 1.2 instead of 1.0 (like stated in chapter 6). This factor is used because the calculation is performed in accordance with the NEN code instead of Eurocode 1. NEN prescribes a value of 1.2 for coefficient C_t.

9.1.5 Stresses

The maximum stresses are calculated with formula 9-9

$$\sigma_{n;\max} = \frac{N_{\max}}{A}$$
$$\mathrm{N}_{\max} = 1.3 * V_g + 1.65 * V_q$$
$$\sigma_{n;\min} = \frac{N_{\min}}{A}$$
$$\mathrm{N}_{\min} = 0.9 * V_g + 1.65 * V_q$$
$$\sigma_m = \frac{M * e}{I}$$

Formula (9-9)

9.1.6 E-modulus

The E-modulus used in formulae 9-4 and 9-5 are calculated in the following way:

Core [36,37]

$$\underline{\mathit{Concrete}\ \mathrm{C50/60}}$$
$$E_f = 1/3 * 1000 * f'_{ck}$$
$$\underline{\mathit{Concrete}\ \mathrm{C90/105}}$$
$$\mathrm{E}_f = 2685 * \sqrt{f'_{ck}}$$

Formula (9-10)

These values are save assumptions for the E-modulus for concrete, which take long term effects (creep) into account.

Columns [27]

The E-modulus for the concrete in the columns is calculated with formula 9-11:

$$E_c = \frac{E_{cm}}{1 + \varphi * \frac{N_{G,Ed}}{N_{Ed}}}$$
$$\mathit{with}$$
$$\varphi = \text{ creep factor}$$
$$\frac{N_{G,Ed}}{N_{Ed}} = \text{ratio between permanent and total load}$$

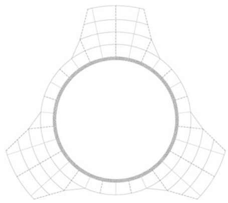
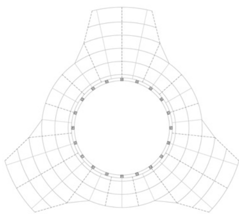
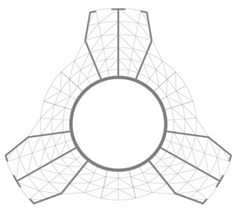
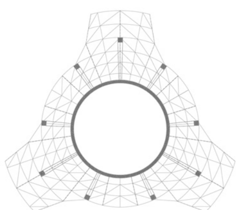
Formula (9-11)

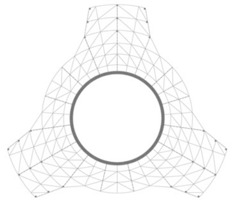
9.2 Non feasible designs and active systems

In order to give insight into the different solution directions which were considered, this section gives an overview of all non feasible designs and a (brief) description of active systems. This section is just a summary of the results. A more detailed description is given in appendix D of this report. This appendix describes the concepts in more detail and gives an overview of reference projects using active systems (hydraulic jacks).

9.2.1 Non feasible designs

This section gives a brief overview of al designs that are made in order to create the alternative designs of this chapter. The designs given in this section are all concept designs which are not used as alternative for the lateral load bearing structure. The concepts are described with more detail in appendix D of this report. The overview of the non feasible designs:

Nr.	Concept design	Layout	Reason of non feasibility
1	Broader core		Less (valuable) space in the storeys
2	Steel/concrete core		A steel concrete core turned out to be less stiff than the concrete alternatives used
3	Constant core diameter	The core diameter of 30.5 meter is used throughout the entire building	Less (valuable) space in the storeys
4	Hammerhead walls		Using hammerhead walls makes it impossible to rotate the storeys
5	Façade mega truss	An outrigger structure in combination with a façade mega truss.	Too much limitations for the concept of rotating storeys and too small advantages in comparison with other comparable solutions
6	Improved outrigger structure		Too much limitations on the concept

7	Combination perimeter columns and outriggers	A combination of perimeter columns with stiff floors and an outrigger	Too much limitations for the concept of rotating storeys and too small advantages in comparison with other comparable solutions
8	Small steel columns in the wings		Connection between the columns will be (almost) impossible to create
9	Extreme stiff foundation	Stiffer foundation	Too small effect on the deformation considering the extra material and work demanded.

9.2.2 Active systems

Another solution presented in the optimization analysis (chapter 8) are active systems. With active systems a solution is meant which “adapts” itself to the deformations caused by wind force. Figure 9-3 illustrates this principle. The active system (a spring-damper system) will generate a force which is in opposite direction to the (moment caused by) wind force. Because the forces of the wind load will be matched by the active system, the deformation of the system will be reduced drastically.

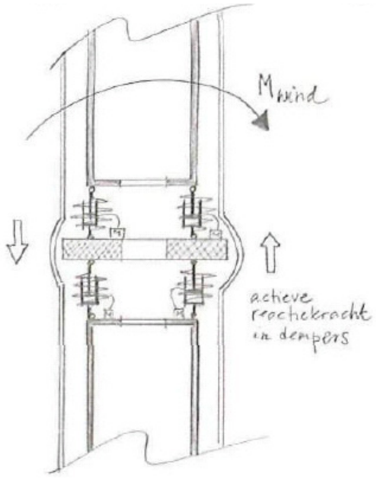


Figure 9-3: active system **[17]**

One of the most striking examples of an active solution is the use of hydraulic jacks within the solution. Two reference project using hydraulic jacks are described in the appendix of this report. The main reasons not to use active systems are:

Experience
There is not much experience (both general and personal) in using active systems (like hydraulic jacks) in buildings in the way it would by necessary in the Rotating Tower. Because of this lack of experience it is considered too much risk to use it in a project like the Rotating Tower.

Risks
When using an active system a mechanical failure will have major consequences. When one of the jacks used in the structure will loose pressure, it will not only not be able to reduce the deformation, but it will also not be able to transfer the forces. This will cause (in the worst possible case) collapse of the tower.

9.3 Architect's design (alternative 1)

Alternative 1 has the same dimensions and contains the same materials as used in the architectural design. Chapter 7 made clear the tower does not meet any of the requirements given in the codes in the current design. Therefore one part of the tower dimensions is adjusted: the height. This alternative is mainly made to show the possibilities of the current (architect's) design.
Alternative 1 keeps the concept of the rotating storeys unchanged from the original design. The driving system, the size and steel structure of the storeys are exactly the same as given in appendix A.
The height of the tower of this alternative is 243 meter. With an average storey height of 5.4 meter, the tower can contain 45 storeys. The main functions of the storeys (hotel, office or residential) will have to be put on different floors than stated in appendix A, to give the tower an economical functional layout.

9.3.1 Structural system

The core dimensions are kept equal to the original architectural design. The core has a diameter of 30.5 meter up to 200 meter and 27 meter above 200 meter (indicated with different colours in figure 9-4, the concrete grade is equal at all heights). The thickness of the core wall is 1 meter and is continuous over the whole height.
The concrete grade used in this alternative is C50/60, which is the highest normal strength concrete grade.
Like figure 9-4 indicates the core has several openings. These openings are needed to make the core (containing staircases and elevators) accessible for users. These opening have an effect on the overall stiffness of the core. The stiffness of the core is decreased to 90% of a continuous core, to take this effect into account.

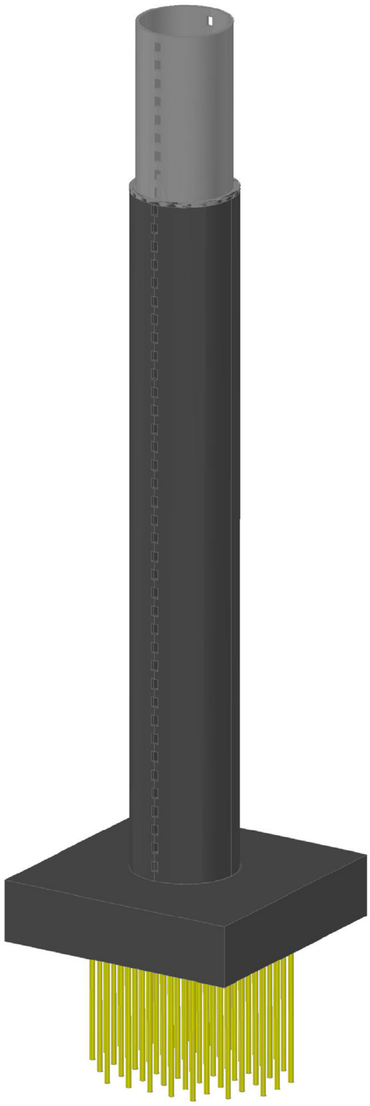


Figure 9-4: Alternative 1

9.3.2 Foundation

Lowering the tower means a smaller and lighter foundation can be made. A foundation slab with round drilled piles is used.
The slab is made from concrete C40/50 with a thickness of 5 meter. The slab follows the round shape of the core and is designed as a circle (for the best force distribution for forces coming from the core). Figures 9-4/5/6 show a more detailed view of the foundation.
The concrete slab is supported by 120 piles. The piles are round cast-in-place concrete piles with a length of 25 meter
The bearing capacity of the chosen piles is 16% higher than strictly necessary for the maximum loaded pile. By using a foundation with a larger bearing capacity, the rotational stiffness of the foundation can be increased.

Layout

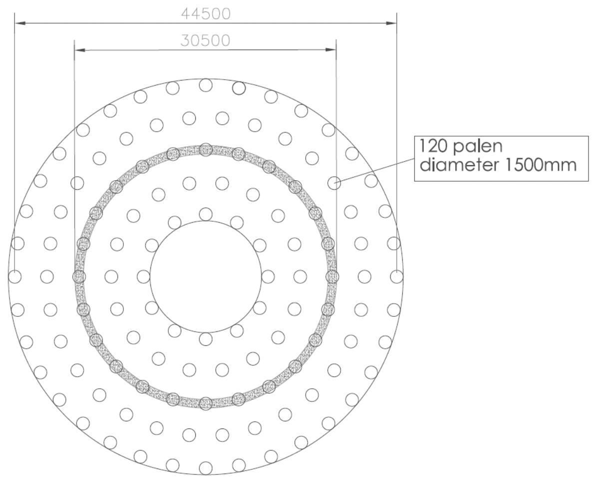


Figure 9-5: Top view foundation

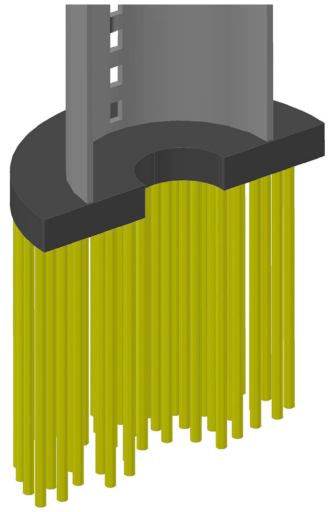


Figure 9-6: 3D figure of foundation

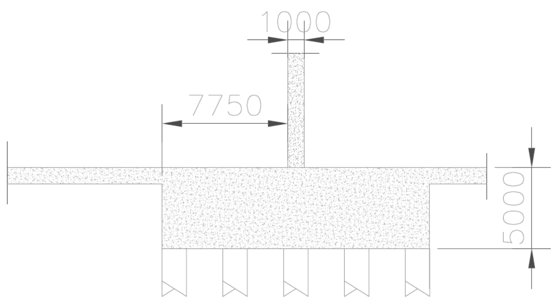


Figure 9-7: Cross section foundation

Loads

The foundation is designed to withstand the following loads (see also section 9.1):

- Wind load/earthquake load (overturning moment)
- Dead weight of the core (both above ground level and in the parking garage)
- Dead and live load of the storeys
- Dead weight of the foundation slab

Maximum pile load

The maximum pile load caused by wind load (overturning moment) is calculated with a computer model (for in/output see appendix). The pile force caused by dead+ live load:

Dead load

248 [kg/m^3]* 243 [m]* (1500+613) [m^2] *9.81[m/s^2] *10^{-3} + 44924 [kN] + 183500 [kN] = 1477608 [kN] per 120 piles = (12.3 MN per pile)

Live load

1990 [kN/floor] *45 [floors]= 89550 [kN] per 120 piles (=0.75MN/pile)

Load	SLS	ULS
Wind load (moment)	5.0 [MN/pile]	8.25 [MN/pile]
Dead + live load	13.1 [MN/pile]	17.2 [MN/pile]

Characteristics piles

Part	Result
Pile Type	Round concrete cast-in-place pile, tube back by driving
Dimension	Diameter 1.5 [m] ; Length 25 [m]
Slip layer	Bentonite
Bearing capacity (limit state 1B)	30.7 [MN]
Deformation due to total load (limit state 2)	0.020 [m]
Deformation due to total load excl. wind load (limit state 2)	0.013 [m]
Spring stiffness	1070 [MN/m]

Rotational stiffness

To calculate the rotational stiffness of the foundation, the calculated pile stiffness due to wind loading is put in the calculation model of the foundation. Two aspects from the output are relevant: the maximum pile forces (which need to be checked with the pile forces taken in the previous calculation) and the maximum displacement of the foundation. Figure 9-8 shows the maximum displacement of the foundation slab. The maximum displacement of the slab underneath the core is equal to 4.6mm. Using this value, the minimum value of the rotational stiffness will be found.

$$\left.\begin{array}{l} C = \dfrac{M}{\varphi} \\ with \\ \varphi = \dfrac{\delta}{14.75} = 3.12 * 10^{-4}\, rad \\ \delta = 0.0046m \\ M = 4739 MNm \end{array}\right\} C = 1520 * 10^{4}\, MNm / rad$$

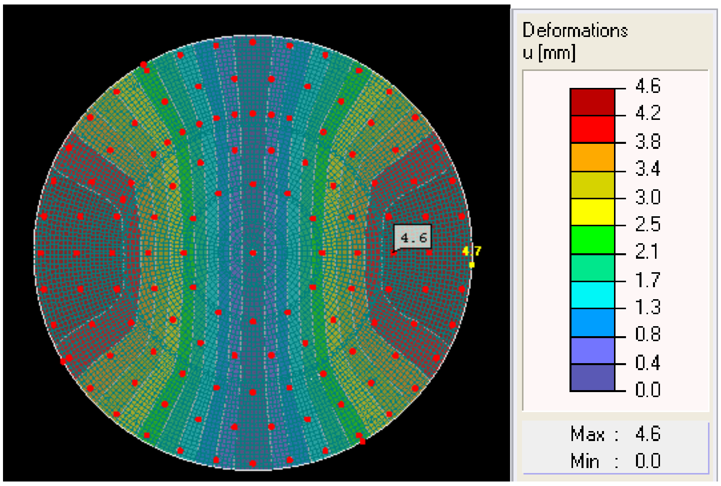


Figure 9-8: Foundation deformations

9.3.3 Deformation

The total deformation of the tower can be calculated with formula 9-4. The total calculation is given in appendix E.

Deformation	Magnitude	Unit
Rotation foundation	81	mm
Bending core	388	mm
Second order effect	9.7	%

The total deformation is 469 mm (were 486 mm is allowed). The structures deformation is designed to be just below the maximum value. Increasing the tower' s height, creates a situation where the second order effect becomes larger than 10% (and has to be taken into account). In this situation the deformation of the tower becomes larger than the maximum allowed value.

9.3.4 Accelerations at the top

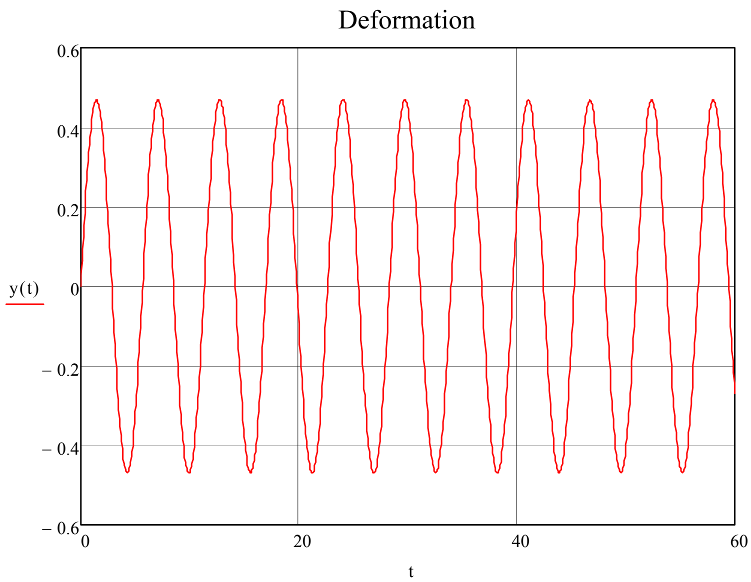


Figure 9-9: deformation (without damping) of the core

The maximum calculated value of the acceleration is 0.101 m/s^2. This is (well) below the maximum allowed value given in figure 4-1 . The structure is not very sensitive to large acceleration at the top. This is due to the relative large dead weight of the tower and low natural frequency.

9.3.5 Stresses governing section

Normal force

Normal force acting on the core consists of dead weight of the structure, dead load and live load from the rotating storeys. All these loads are given in chapter 6 and the first section of this chapter. The total normal force acting on the governing section of the core (the lowest point):

<u>SLS</u>
1.0 * 248 [kg/m^3] *(1500 + 613) [m^2] * 243 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 44924 [kN] + 1.0 * 1990 [kN/floor] * 45 [floors] = **1384MN**

<u>ULS wind max</u>
1.3 * 248 [kg/m^3] *(1500 + 613) [m^2] * 243 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 44924 [kN] + 1.65 * 1990 [kN/floor] * 45 [floors] = **1830MN**

<u>ULS wind min</u>
0.9 * 248 [kg/m^3] *(1500 +613) [m^2] * 243 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 44924 [kN] + 0.0* 1990 [kN/floor] * 45 [floors] = **1165MN**

<u>ULS earthquake max</u>
1.2 * 248 [kg/m^3] *(1500 + 613) [m^2] * 243 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 44924 [kN] + 0.5 * 1711 [kN/floor] * 45 [floors] = **1591MN**

<u>ULS earthquake min</u>
0.9 * 248 [kg/m^3] *(1500 + 613) [m^2] * 243 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 44924 [kN] + 0.0* 1711 [kN/floor] * 45 [floors] = **1165MN**

Wind load
M_{wind} = **4.739*10^6 kNm (SLS)**
M_{wind} = **7.819*10^6 kNm (ULS)**

Earthquake load
$M_{earthquake}$ = **7.639*10^6 kNm (ULS)**

<u>Stresses</u>
Governing load case for this height is wind load. The maximum overturning moment caused by wind force exceeds the maximum overturning moment caused by earthquake loads. The maximum stresses acting on the structure:

Stresses SLS

$$\sigma_n = \frac{N}{A} = \frac{1384*10^6}{0.95*92.7*10^6} = 15.68N/mm^2$$

$$\sigma_m = \frac{M*e}{I} = \frac{4.739*10^{12}*15250}{0.9*1.009*10^{16}} = 7.96N/mm^2$$

Because no tensile stresses occur in the core, the concrete stays uncracked (important aspect because the E-modulus used in the deformation calculation is calculated assuming the concrete would stay uncracked).

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{1830*10^6}{0.95*92.7*10^6} = 20.8N/mm^2$$

$$\sigma_{n;\min} = \frac{N}{A} = \frac{1165*10^6}{0.95*92.7*10^6} = 13.22N/mm^2$$

$$\sigma_m = \frac{M*e}{I} = \frac{7.819*10^{12}*15250}{0.9*1.009*10^{16}} = 13.1N/mm^2$$

No tensile stresses occur in the governing cross-section of the core. Therefore no reinforcement is needed to resist tensile forces (for this check). A concrete core must contain a minimum reinforcement ratio according to Eurocode 2 (see appendix F).

9.3.6 Conclusion

The design meets all the requirements (strength, deformation and acceleration). This alternative was designed using the same dimensions and materials as the architect's design. The total feasible height of the tower within this concept is 243 meter (45 storeys). Alternative 1 keeps the concept of the tower intact, only the height of the tower had to be reduced. With a few adaptations (for instance the division of the different functions over the storeys), the Rotating Tower can be built within this concept.

9.4 Higher concrete grade (alternative 2)

Alternative 1 was exactly the same as the architectural design, only with a decreased height. Alternative 2 is comparable with alternative 1, only now a higher concrete grade is used (with the highest concrete strength available). In an attempt to increase the stiffness of the core, concrete grade C90/105 is used in the calculations. The E-modulus of this grade is 1.4 times bigger than of grade C50/60.
Beside the change of concrete grade all dimensions (except the height) are kept equal to the architectural design. The height of the tower in this alternative is 270 meter. With an average storey height of 5.4 meter, the tower can contain 50 storeys. The main functions of the storeys (hotel, office or residential) will have to be put on different floors than stated in appendix A, to give the tower an economical functional layout.
This alternative shows the tallest tower possible, when keeping the dimensions of the tower exactly the same as given in the architectural design. A concrete grade with higher stiffness is not available. When the tower is designed like stated by the architect (same dimensions only with a different concrete grade) the tower can be designed with a height of 270 meter.

9.4.1 Structural system

Alternative 2 is a tower with a height of 270 meter. The lateral load structure is made of a concrete core with a wall thickness of 1 meter. The core has a changing diameter over the height. From 0 - 200 m the core has a diameter of 30.5 meter and from 200 - 270 meter the core has a diameter of 27 meter.
The core has several openings (for access to the staircases and elevators). These openings in the core lower the stiffness significantly. Because of this effect, the moment of inertia of the core is taken 10% smaller.

9.4.2 Foundation

The foundation for this alternative is unchanged in comparison with alternative 1 . This means the foundation is built up from 120 cast-in-place concrete piles with a diameter of 1500 mm and a 5 meter thick concrete slab (with concrete grade C40/50).
The forces acting on the foundation are larger in this alternative in comparison with alternative 1. The foundation of alternative 1 had a larger bearing capacity then necessary for the maximum loads. Therefore an "over" capacity was reached and the foundation turned out to be suitable for this alternative too.

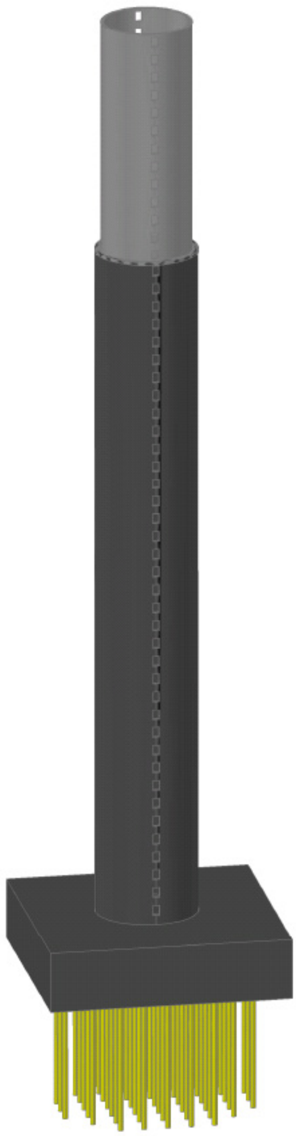


Figure 9-10: alternative 2

Layout

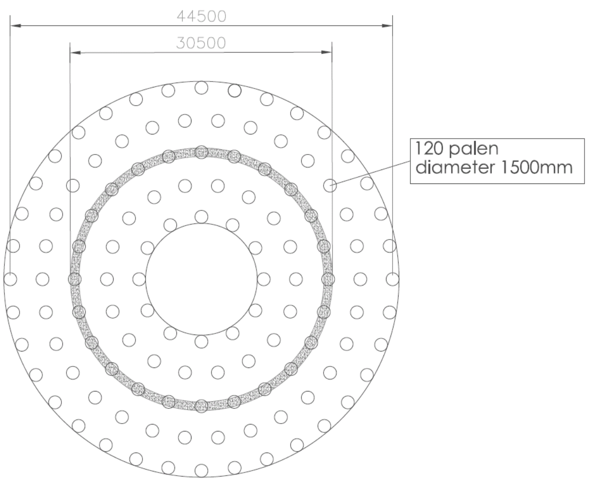


Figure 9-11: Top view foundation

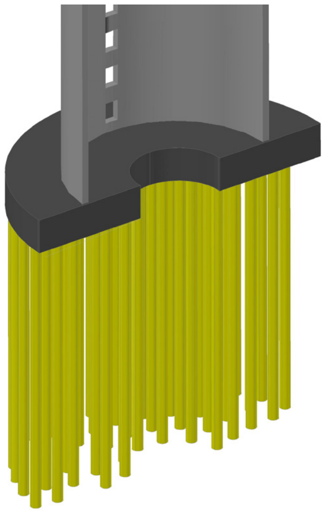


Figure 9-12: 3D figure of foundation

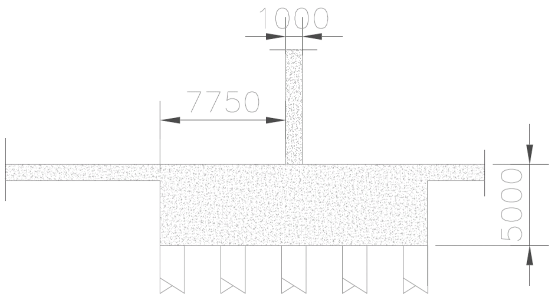


Figure 9-13: Cross section foundation

Loads

The foundation is designed to resist the following loads (see also section 9.1):

- Wind load/earthquake load (overturning moment)
- Dead weight of the core (both above ground level and in the parking garage)
- Dead and live load of the storeys
- Dead weight of the foundation slab

Maximum pile load

The maximum pile load caused by wind load (overturning moment) is calculated with a computer model (for in/output see appendix). The pile force caused by dead+ live load:

Dead load

248 [kg/m^3] * 270 [m]* (1500+613) [m^2] *9.81[m/s^2] *10^{-3} + 44924 [kN] + 183500 [kN] = 1616406 [kN] per 120 piles = (13.5 MN per pile)

Live load

1990 [kN/floor] *50 [floors]=99500 [kN] per 120 piles (=0.8MN/pile)

Load	SLS	ULS
Wind load (moment)	6.2 [MN/pile]	10.3 [MN/pile]
Dead + live load	14.3 [MN/pile]	18.8 [MN/pile]

Part	Result
Pile Type	Round concrete cast-in-place pile, tube back by driving
Dimension	Diameter 1.5 [m] ; Length 25 [m]
Slip layer	Bentonite
Bearing capacity (limit state 1B)	30.7 [MN]
Deformation due to total load (limit state 2)	0.023 [m]
Deformation due to total load excl. wind load (limit state 2)	0.015 [m]
Spring stiffness	1150 [MN/m]

Rotational stiffness
To calculate the rotational stiffness of the foundation, the calculate pile stiffness due to wind loading is put in the calculation model of the foundation. Two aspects from the output are relevant: the maximum pile forces (which need to be checked with the pile forces taken in the previous calculation) and the maximum displacement of the foundation. Figure 9-14 shows the maximum displacement of the foundation slab. The maximum displacement of the slab underneath the core is equal to 5.4 mm. Using this value, the minimum value of the rotational stiffness will be found.

$$\left.\begin{array}{l} C = \dfrac{M}{\varphi} \\ with \\ \varphi = \dfrac{\delta}{14.75} = 3.7 * 10^{-4}\, rad \\ \delta = 0.0054m \\ M = 5845MNm \end{array}\right\} C = 1600 * 10^{4}\, MNm/rad$$

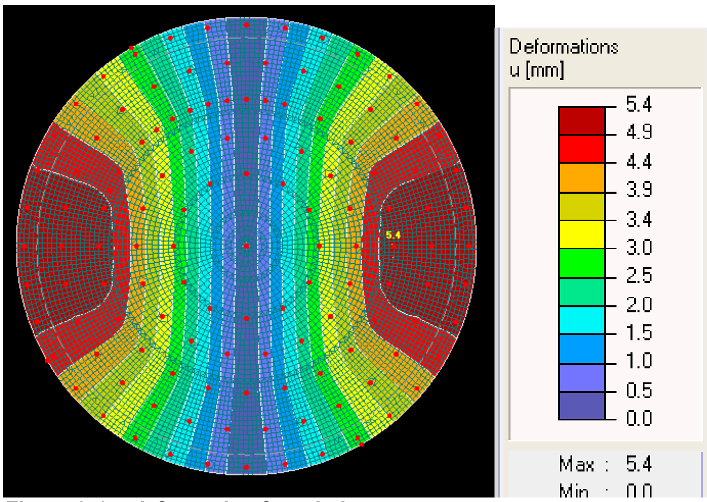


Figure 9-14: deformation foundation

9.4.3 Deformation

The total deformation of the tower can be calculated with formula 9-4. The total calculation is given in the appendix.

Deformation	Magnitude	Unit
Rotation foundation	104	mm
Bending core	423 (extra bending = 1 mm)	mm
Second order effect	9.8	%

The total deformation is 527 mm (were 540 mm is allowed). The deformation of the structure in alternative 2 is well within the limits stated by the Eurocode. Increasing the height of the tower instantly creates a situation where all three parts of the deformation (rotation, bending and 2nd order effect) together give a too large value.

9.4.4 Accelerations at the top

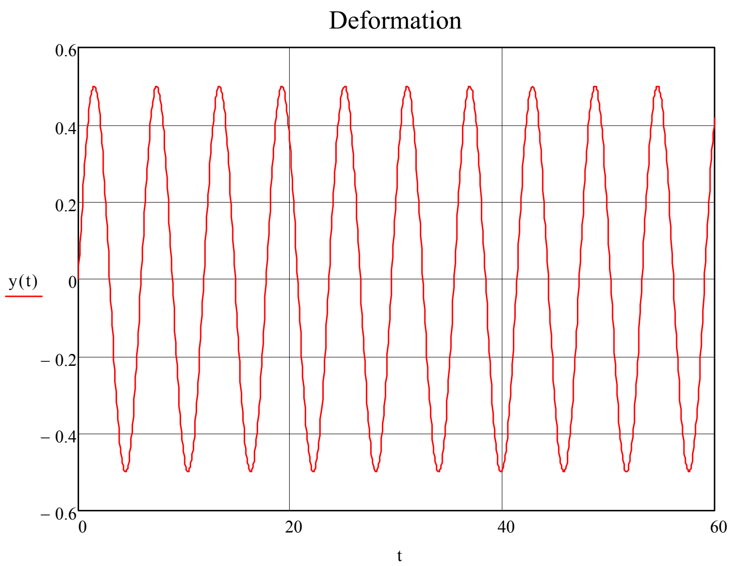


Figure 9-15: acceleration tower

The maximum calculated value of the acceleration is 0.103 m/s^2. This is (well) below the maximum allowed value given in figure 4-1. Just like alternative 1 this structure is not very sensitive for maximum accelerations at the top (for the same reasons).

9.4.5 Stresses

Normal force

SLS

1.0 * 248 [kg/m^3] *(1500 +613) [m^2] * 270 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 44924 [kN] + 1.0 * 1990 [kN/floor] * 50 [floors] = **1532MN**

ULS wind max

1.3 * 248 [kg/m^3] *(1500 + 613) [m^2] * 270 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 44924 [kN] + 1.65 * 1990 [kN/floor] * 50 [floors] = **2027MN**

ULS wind min

0.9 * 248 [kg/m^3] *(1500 + 613) [m^2] * 270 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 44924 [kN] + 0.0* 1990 [kN/floor] * 50 [floors] = **1290MN**

<u>ULS earthquake max</u>
1.2 * 248 [kg/m^3] *(1500 + 613) [m^2] * 270 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 44924 [kN] + 0.5 * 1711 [kN/floor] * 50 [floors] = **1762MN**

<u>ULS earthquake min</u>
0.9 * 248 [kg/m^3] *(1500 +613) [m^2] * 270 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 44924 [kN] + 0.0* 1711 [kN/floor] * 50 [floors] = **1290MN**

Wind load
M_{wind} **= 5.845*10^6 kNm**
M_{wind} **= 9.644*10^6 kNm**

Earthquake load
$M_{earthquake}$ **=8.538*10^6 kNm (ULS)**

Stresses SLS

$$\sigma_n = \frac{N}{A} = \frac{1532*10^6}{0.95*92.7*10^6} = 17.4N/mm^2$$

$$\sigma_m = \frac{M_{2th;order}*e}{I} = \frac{5.845*10^{12}*15250}{0.9*1.009*10^{16}} = 9.8N/mm^2$$

Because no tensile tresses occur in the core, the concrete stays uncracked (important aspect because the E-modulus used in the deformation calculation is calculated assuming the concrete would stay uncracked).

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{2027*10^6}{0.95*92.7*10^6} = 23.0N/mm^2$$

$$\sigma_{n;\min} = \frac{N}{A} = \frac{1290*10^6}{0.95*92.7*10^6} = 14.6N/mm^2$$

$$\sigma_m = \frac{M_{2th;order}*e}{I} = \frac{9.644*10^{12}*15250}{0.9*1.009*10^{16}} = 16.2N/mm^2$$

The stresses stay within the limits for concrete C90/105: maximum compressive stress is 39.2 N/mm^2 and maximum tensile stress is -1.6 N/mm^2.

9.4.6 Conclusion

By using the concrete grade with the highest strength available a tower with 50 storeys (270 meter) can be designed within the requirements of the codes. This height represents the maximum possible height which can be created when keeping the dimensions of the architectural design. The design keeps the building concept and dimensions intact.

9.5 Increased wall thickness (alternative 3)

Alternative 2 was designed to keep the dimensions exactly the same as in the architectural design. The only characteristic changed is the concrete grade. Instead of C50/60 a grade C90/105 was used. Alternative 3 is almost the same as alternative 2, only has a core with increased wall thickness (so also concrete grade C90/105). Thickening the walls of the core was one of the solution possibilities stated in the optimization analysis which had an effect on increasing the stiffness of the structure and also has a large effect on resisting the stresses on the structure.
The tower has a height of 286 meter (53 storeys). The core wall is made thicker to the inside of the core, so the storey sizes can be kept the same as in the architectural design. Because the core is made just 0.5 meter thicker, all original functions of the core can still be placed inside without compromising.
Making the core thicker creates an increase in dead weight. Because of the higher dead weight the tower is more sensitive to second order effects and the foundation need to be heavier. These are the main two reasons why the core wall was not made thicker than 1.5 meter.

9.5.1 Structural system

The height of the tower of alternative 3 is 286 meter (54 storeys). The lateral load structure is made of a concrete core with a wall thickness of 1.5 meter. The core has a changing diameter over the height. From 0 – 200 m the core has a diameter of 30.5 meter and from 200- 286 meter the core has a diameter of 27 meter.
The core has several openings (for access to the staircases and lifts). These openings in the core lower the stiffness significantly. Because of this effect, the moment of inertia of the core is decreased with 10%.

9.5.2 Foundation

Because of the increased height of the tower the total loads (coming from dead, live and wind load) are considerably higher. Because of this increase the foundation had to be made heavier. The overall layout is kept similar: 120 concrete cast-in-place piles (length 40 meter) with a circular shaped concrete slab. The only difference is the thickness of the slab and the concrete grade used. The slab is made 1 meter thicker to 6 meter and the concrete grade used is C70/85 (to create a stiffer slab which can distribute the forces better to al the piles underneath the slab).
Because of the increased thickness of the slab the dead weight is a lot bigger than the slab used in the previous alternatives. This higher dead weight gives larger forces on the foundation piles. Making the raft thicker, creates a stiffer foundation but gives an even higher dead weight. Therefore the slab is not made any thicker than 6 meter (in all the alternatives).

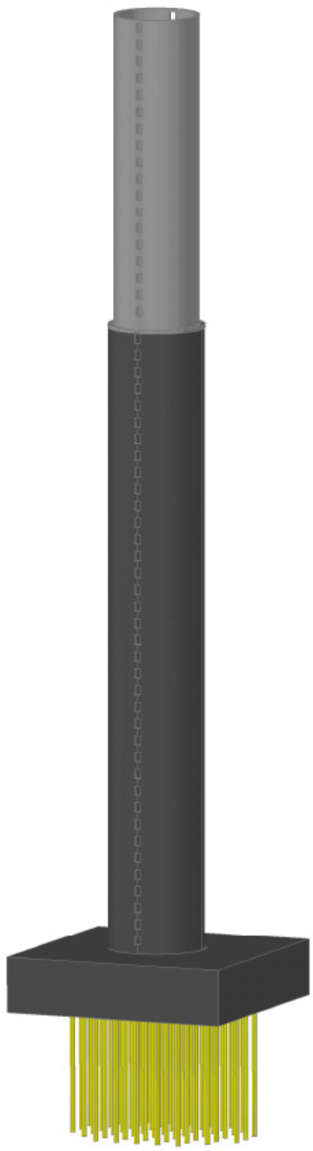


Figure 9-16: alternative 3

Layout

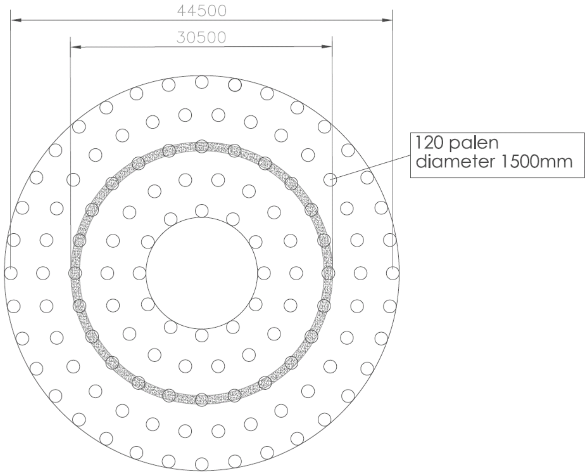


Figure 9-17: Top view foundation

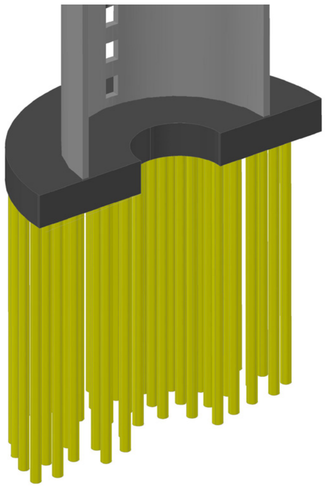


Figure 9-18: 3D figure of foundation

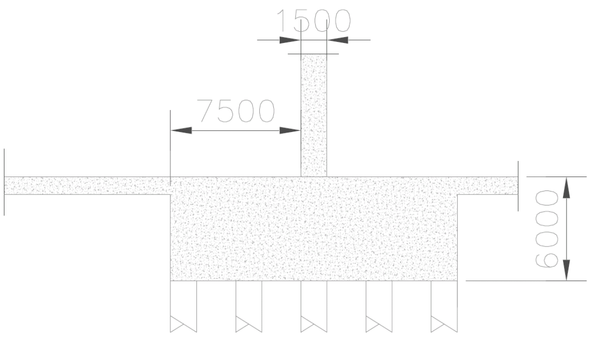


Figure 9-19: Cross section foundation

Loads

The foundation is designed to withstand the following loads (see also section 9.1):

- Wind load/earthquake load (overturning moment)
- Dead weight of the core (both above ground level and in the parking garage) with 1.5 meter wall thickness
- Dead and live load of the storeys
- Dead weight of the foundation slab

Maximum pile load

The maximum pile load caused by wind load (overturning moment) is calculated with an computer model (for in/output see appendix). The pile force caused by dead+ live load:

Dead load

289 [kg/m^3] * 292 [m]* (1500+613) [m^2] *9.81[m/s^2] *10^{-3} + 62025 [kN] + 220201 [kN] = 1996720 [kN] per 120 piles = (16.6 MN per pile)

Live load

1990 [kN/floor] *53 [floors]=105470 [kN] per 120 piles (=0.9MN/pile)

Load	SLS	ULS
Wind load (moment)	7.3 [MN/pile]	12.0 [MN/pile]
Dead + live load	17.5 [MN/pile]	23.1 [MN/pile]

Characteristics piles

Part	Result
Pile Type	Round concrete cast-in-place pile, tube back by driving
Dimension	Diameter 1.5 [m] ; Length 40 [m]
Slip layer	Bentonite
Bearing capacity (limit state 1B)	39.7 [MN]
Deformation due to total load (limit state 2)	0.031 [m]
Deformation due to total load excl. wind load (limit state 2)	0.021 [m]
Spring stiffness	1100 [MN/m]

Rotational stiffness

To calculate the rotational stiffness of the foundation, the calculated pile stiffness due to wind loading is put in the calculation model of the foundation. Two aspects from the output are relevant: the maximum pile forces (which need to be checked with the pile forces taken in the previous calculation) and the maximum displacement of the foundation. Figure 9-20 shows the maximum displacement of the foundation slab. The maximum displacement of the slab underneath the core is equal to 5.9 mm. Using this value, the minimum value of the rotational stiffness will be found.

$$\left.\begin{array}{l} C = \dfrac{M}{\varphi} \\ with \\ \varphi = \dfrac{\delta}{14750} = 4.0*10^{-4}\,rad \\ \delta = 5.9mm \\ M = 6599MNm \end{array}\right\} C = 1650*10^{4}\,MNm/rad$$

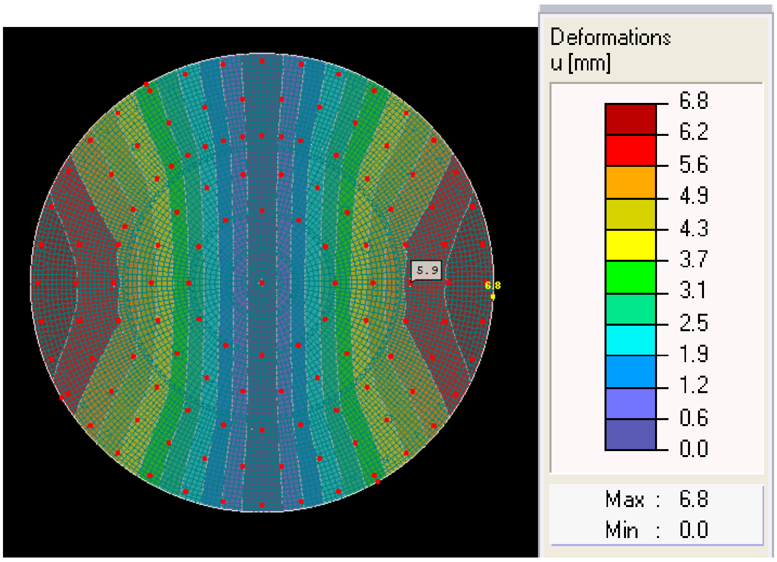


Figure 9-20: deformation foundation

9.5.3 Deformation

The total deformation of the tower can be calculated with formula 9-4. The total calculation is given in the appendix.

Deformation	Magnitude	Unit
Rotation foundation	121	mm
Bending core	377 (extra bending = 2)	mm
Second order effect	9.9	%

The total deformation is **498 mm** (were 572 mm is allowed). It is obvious the deformation is far below the maximum allowed value. The reason for this low value is because the second order effect is kept below 10% (and it does not have to be accounted for). By increasing the height of the tower, the second order effect will have to be taken into account (making the deformation and forces a lot bigger). In that situation the tower does not meet the requirements. This sensitivity to second order effects is triggered by the larger dead weight of this alternative. Because of this extra sensitivity to the second order effect, the core wall wasn't made any thicker than it is.

9.5.4 Accelerations at the top

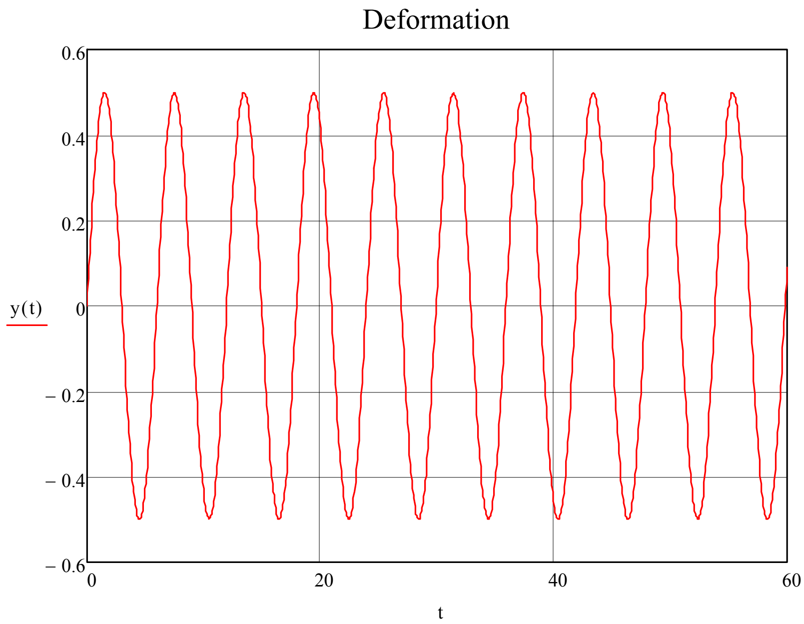


Figure 9-21: accelerations tower

The maximum calculated value of the acceleration is 0.088 m/s^2. This is (well) below the maximum allowed value given in figure 4-1. The bigger dead weight of the structure is an advantage in the case of accelerations at the top. The bigger dead weight of the structure lowers the acceleration considerably.

9.5.5 Stresses

Normal force

SLS
1.0 * 289 [kg/m^3] *(1500 +613) [m^2] * 286 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 62025 [kN] + 1.0 * 1990 [kN/floor] * 53 [floors] = **1880MN**

ULS wind max
1.3 * 289 [kg/m^3] *(1500 + 613) [m^2] * 286 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 62025 [kN] + 1.65 * 1990 [kN/floor] * 53 [floors] = **2482 MN**

ULS wind min
0.9 * 289 [kg/m^3] *(1500 + 613) [m^2] * 286 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0* 1990 [kN/floor] * 53 [floors] = **1598MN**

ULS earthquake max
1.2 * 289 [kg/m^3] *(1500 + 613) [m^2] * 286 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 62025 [kN] + 0.5 * 1711 [kN/floor] * 53 [floors] = **2183MN**

ULS earthquake min
0.9 * 289 [kg/m^3] *(1500 + 613) [m^2] * 286 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0* 1711 [kN/floor] * 53 [floors] = **1598MN**

Wind load
M_{wind} **= 6.599 *10^6 kNm (SLS)**
M_{wind} **= 1.089 *10^7 kNm (ULS)**

Earthquake load
$M_{earthquake}$ **= 1.058 *10^6 kNm (ULS)**

Stresses SLS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{1880*10^6}{0.95*136.7*10^6} = 14.5 N/mm^2$$

$$\sigma_m = \frac{M*e}{I} = \frac{6.599*10^{12}*15250}{0.9*1.44*10^{16}} = 7.8 N/mm^2$$

Because no tensile stresses occur in the core, the concrete stays uncracked (important aspect because the E-modulus used in the deformation calculation is calculated assuming the concrete would stay uncracked).

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{2482*10^6}{0.95*136.7*10^6} = 19.1 N/mm^2$$

$$\sigma_{n;\min} = \frac{N}{A} = \frac{1598*10^6}{0.95*136.7*10^6} = 12.3 N/mm^2$$

$$\sigma_m = \frac{M*e}{I} = \frac{1.089*10^{13}*15250}{0.9*1.44*10^{16}} = 12.8 N/mm^2$$

The stresses stay within the limits for concrete C90/105: maximum compressive stress is 31.9 N/mm^2 and a maximum tensile stress of -0.5 N/mm^2.

9.5.6 Conclusion

Making the core wall 0.5 meter thicker creates a stiffer structure. The structure has sufficient stiffness to resist lateral forces up to a height of 286 meter (53 storeys). Because of the increased dead weight, the second order effect (n/(n-1)) is a governing factor in determining the height of the tower. By increasing the height just a few meter, the second order effect would be larger than 10%. Although the core itself is less sensitive to second order effect (because of an increased stiffness), the high normal force (dead weight) on the core makes it sensitive to second order effects.

9.6 *Outrigger system (alternative 4)*

In order to create a lateral load bearing structure which can resist wind- and earthquake load up to greater heights, it is important to create a structure with a wider base. Making the concrete core wider does meet this demand, only makes it necessary to decrease the storey area (when keeping the same outside dimensions).

The solution used in alternative 4 to create a wider base, are outrigger braced columns. The outrigger braced structure is partly present in the rotating part of the building. The perimeter columns and outrigger trusses will be placed inside the steel structure of the storeys. The perimeter columns will not be connected to each other when the building is rotating (this is when the wind speed is below 20.7 m/s). In this mode the concrete core is the only lateral structural part.

The storeys stop rotating when the wind reaches a speed of 20.7 m/s (8 Beaufort) at a reference height of 10 meter above ground level. The columns will then be connected to each other and to the outrigger trusses. When the columns are connected, the stiffer outrigger braced structure will arise.

9.6.1 Structural system

The concrete core has the same dimension as in alternative 3. The core has a varying diameter:

- From 0 - 200 meter: 30.5 meter
- From 200 - 376 meter: 27 meter

The wall thickness of the core is 1.5 meter and a concrete grade C90/105 is used in the calculations.

The most important additions are the outriggers and perimeter columns. 6 perimeter columns are added to the structure to create more stiffness. The columns are divided equally over 360^0 (with an angle of 60^0 between them). All columns are steel concrete columns of 2.5 x 2.5 m^2. Figure 9-32 shows the positioning of the columns within the steel structure. The columns are placed within the circular part of the steel structure on purpose. When placing the columns in the “wings” of the storeys, the wings containing outrigger trusses won’t be able to rotate. By placing the columns closer to the core, the “wings” of the structure containing outrigger trusses will be able to keep rotating although the rest of the storey stays static.

The columns are connected to the core at 2 levels with outrigger trusses. The outrigger trusses are 5 storeys high, in order to create enough stiffness to the trusses. The choice for 2 outrigger levels with a height of 5 storeys is made conscious. Because the connection of the outrigger to the core is complicated, it is not possible to rotate the storeys containing an outrigger truss. The configuration with 2 outrigger levels turned out to be the most efficient.

The storeys between and above the outrigger trusses are able to rotate completely. The perimeter columns will be in a disconnected mode when the storeys rotate and are connected when the storeys do not rotate. The connection of the columns is described in more detail in one of the following sections.

Figures 9-23 and 9-24 show two different modes the building can occur in (static and rotating). In figure 9-24 it is clearly visible that the columns are not connected to each other when the storeys are rotating.

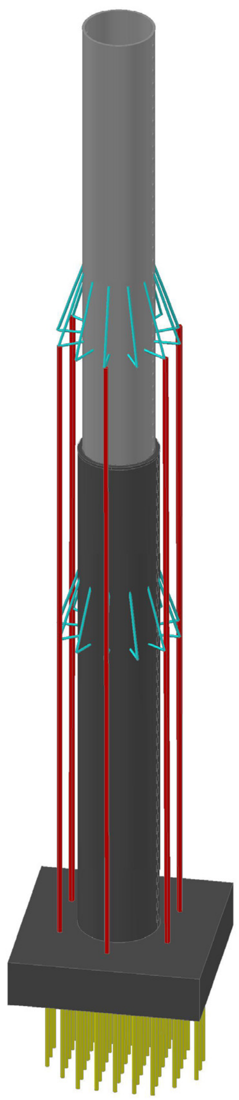


Figure 9-22: structural system alternative 4

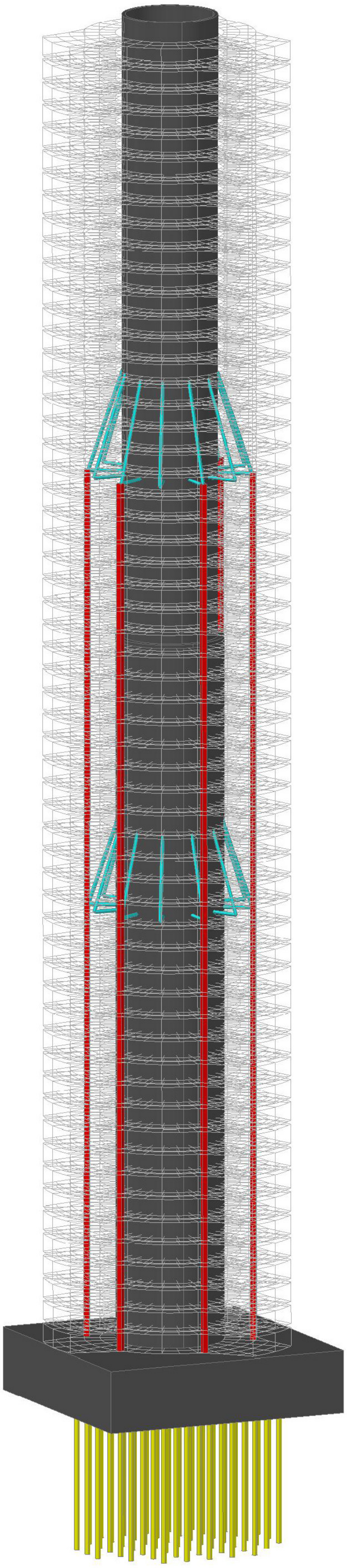


Figure 9-23: structural system static position

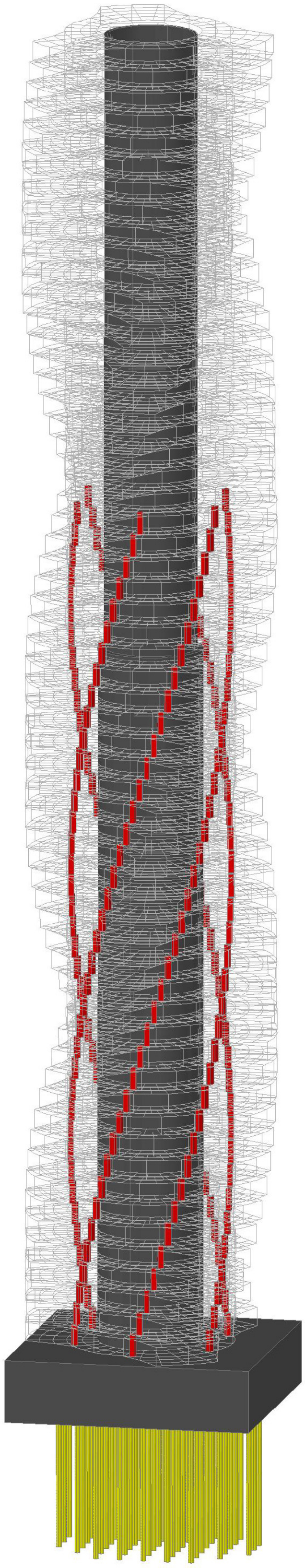


Figure 9-24: structural system when rotating

9.6.2 Foundation

The foundation of alternative 4 is different from the previous ones. The layout is changed because the perimeter columns need to be founded. Therefore the original round foundation slab is extended with 12 “extra” square slabs. These square slabs are used as foundation for the perimeter columns. Although there are only 6 perimeter columns, the 12 slabs are necessary. Because the columns rotate with the storeys, the columns can rotate to any position possible. When the tower stops rotating (because of high wind forces) the columns will be connected to the foundation. The storeys will be positioned in such a way the tower experiences the least wind load (see also chapter 6). To make sure the tower can be placed in (almost) every position possible, the perimeter columns must be able to be connected to the foundation at more than one location. Beside this important reason, the extra 6 slabs also create a stiffer foundation.
The foundation consists of 192 piles (cast-in-place) with a diameter of 1500mm. The total foundation slab has a thickness of 6 meter and is made of concrete grade C70/85.

<u>Layout</u>

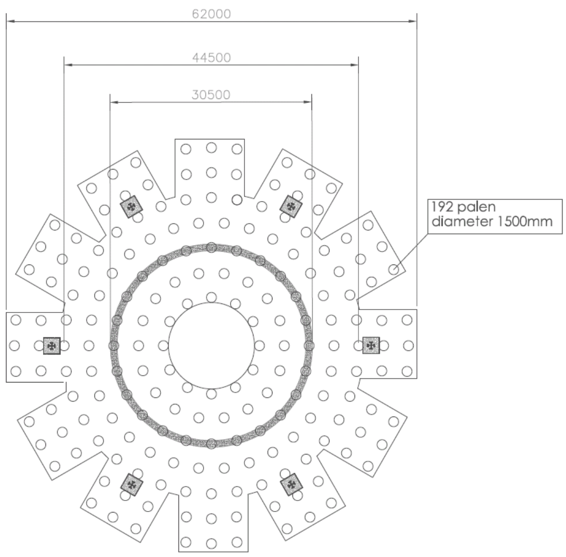


Figure 9-25: Top view foundation

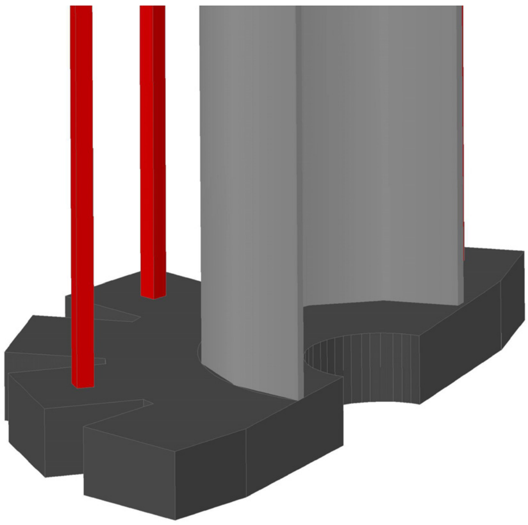


Figure 9-26: 3D figure of foundation

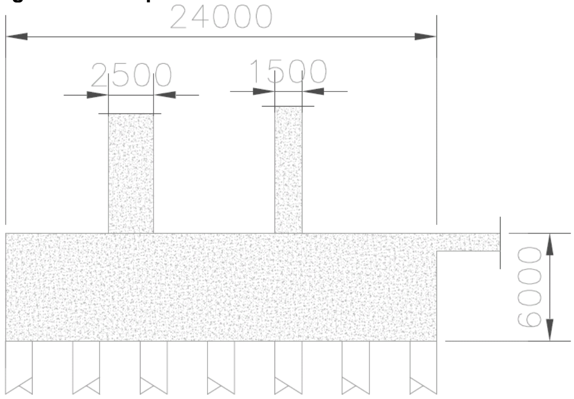


Figure 9-27: Cross section foundation

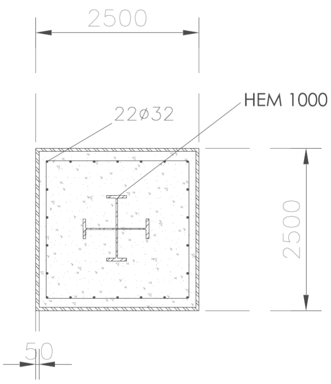


Figure 9-28: Cross section perimeter column

<u>Loads maximum loaded pile</u>

Al pile load are calculated with an 3D computer analysis (for in/output see appendix)

Load case	SLS	ULS
Max wind load (moment)	5.5 [MN/pile]	9.6 [MN/pile]
Max wind load (moment) Incl. second order	- [MN/pile]	10.7 [MN/pile]
Dead + live load	20.6 [MN/pile]	27 [MN/pile]

Characteristics piles

Part	Result
Pile Type	Round concrete cast-in-place pile, tube back by driving
Dimension	Diameter 1.5 [m] ; Length 40 [m]
Slip layer	Bentonite
Bearing capacity (limit state 1B)	39.6 [MN]
Deformation due to total load (limit state 2)	0.033 [m]
Deformation due to total load excl. wind load (limit state 2)	0.025 [m]
Spring stiffness	1050 [MN/m]

Rotational stiffness foundation
Figure 9-29 shows the deformations of the foundation slab. The deformation is linear distributed along the slab underneath the core. Therefore the rotational stiffness can be calculated by taking the maximum deformation and calculating the angle of rotation with that deformation.

$$\left.\begin{aligned} & C = \frac{M}{\varphi} \\ & \varphi = \frac{5.2}{14750} = 3.5*10^{-4}\,rad \\ & M = 11210\,MNm \end{aligned}\right\} C = 3180*10^{4}\,MNm/rad$$

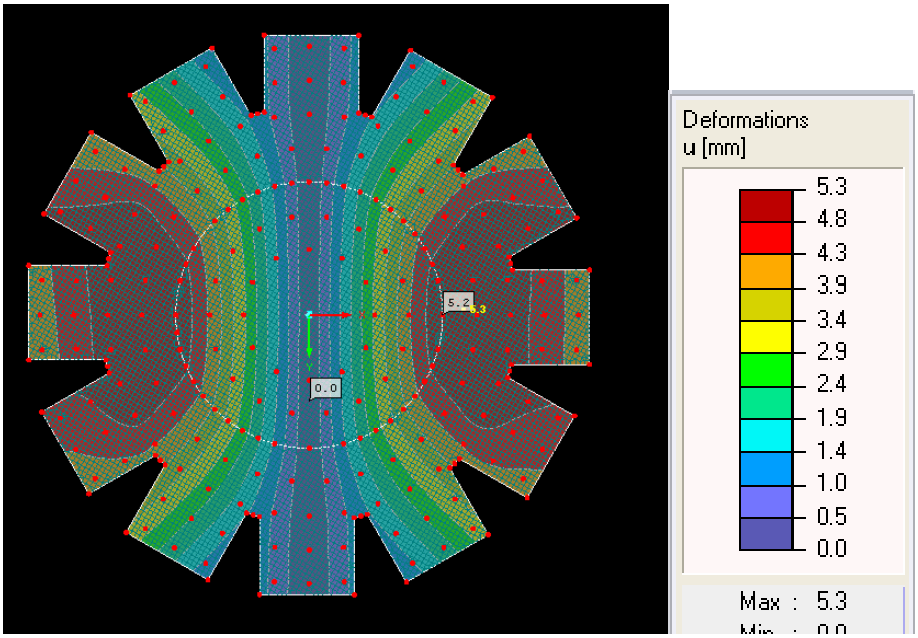


Figure 9-29: Core foundation deformations

9.6.3 Perimeter columns

An important aspect of this alternative is the connection between the columns. Figures 9-30 and 9-31 give a global impression of the column to column connection. A more detailed description of the connection is given in the next section

In the rotating mode the columns are not connected to each other. Because the columns are placed with an angle of 60^0 with respect to each other, the tower can take every shape imaginable without the columns passing each other. This makes it possible to create a simple connection between the columns. When the wind force reaches 8 Beaufort, the storeys will be rotated in such a way the columns will be connected to each other creating 6 ongoing columns. The columns are composite steel/concrete with a dimension of 2.5 x 2.5 m^2 (a size indication is given in figure 9-33). The cross section of one of the columns is given in figure 9-28.

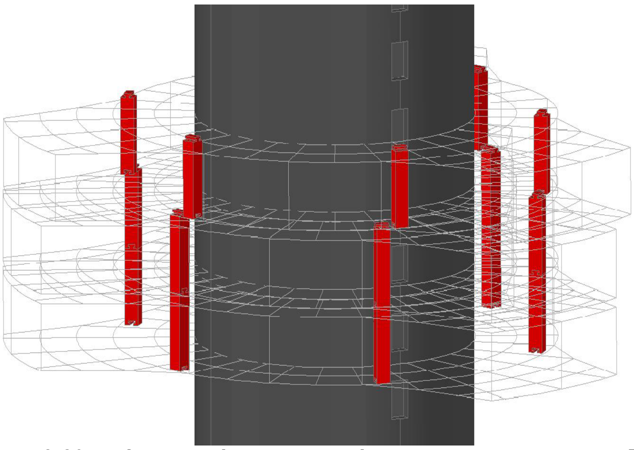


Figure 9-30:perimeter column connection

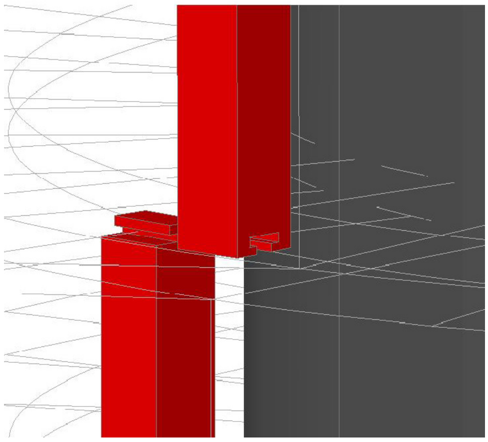


Figure 9-31: perimeter column connection

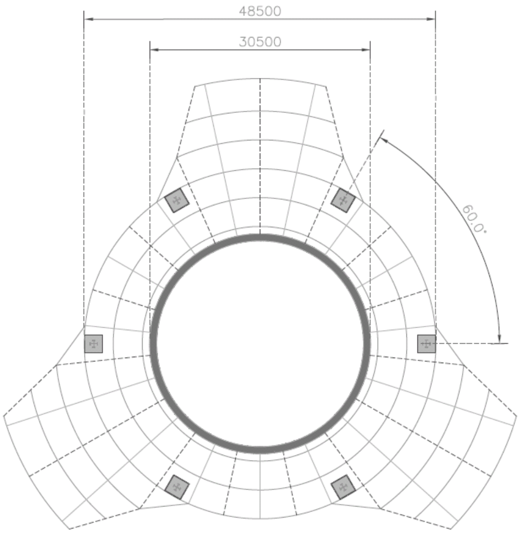


Figure 9-32:perimeter column positions

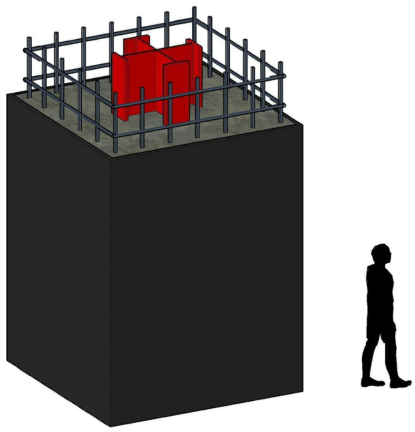


Figure 9-33:perimeter column 3d view

9.6.4 Connection perimeter columns

An important aspect which determines the feasibility of alternative 4 is the connection between the perimeter columns. This section describes an overview of the connection. Note that this design is still a concept and the connection needs to be engineered to a more detailed level. This section shows sketches of the general idea of the connection and gives an explanation of the preconditions used in the calculation and their impact on the design of the connection.

Concept

Figure 9-34 shows the concept design of the connection between the perimeter columns. The main concept of the connection was already described in the previous section: a connection which can (dis)connect to both directions. During the rotation of the storeys, the columns will not pass each other, so the connection can be (and is) a tight fit. The red parts in the figure are added to make sure that the columns will be "guided" into the right direction and the columns will connect correctly. This part of the connection also contains a "slope" to push the upper column up. This is necessary to make sure the columns will also carry a part of the self weight of the storey structures (and create a compression force in the columns instead of tensile forces due to only wind load). The "guiding" structure will be covered with a slick material (for instance Teflon), to create as less friction as possible between the structure and the column.
To make sure the columns will connect correctly, it is necessary that the columns are connected to each other, starting with the lowest columns. When the columns will be connected random, it can happen that a lower storey has a greater deflection (due to higher live load) and the columns won't connect (in other words the upper storey is located to high for the columns to connect). Another advantage of this way of connecting is the small vertical force acting on the column during the connecting. Only the weight of the storey itself will have to be carried by the connection during connection, so the horizontal force which is present (due to the slope in the connection) will be small. When connection from top to bottom or random the bottom connection will experience all the dead weight of the upper storeys during connecting and a very large horizontal force will be present (making it impossible to connect the columns).

Figure 9-34: Concept connection

Force (re)distribution

The column will push the storeys up, to create a situation where the columns also carry dead weight. Figure 9-35 show the scheme of the situation where the columns are not connected: the storeys are loaded with a uniform load q (dead and live load). The storeys have a vertical displacement u due to the q-load (at the location of the columns the value for u has a maximum value: 0.004 * L = 0.004 * 9000 = 36mm). When the columns are connected to each other a force F is added to the structure. The force F will also give a vertical displacement (see figure 9-36). The connection is designed in such a way the force F will create a displacement u which is equal to the displacement u created by the q-load (in other words, the storey will be pushed up until they hang horizontally). In this situation the force F is equal to 3/8*q*l (see figure 9-37.).

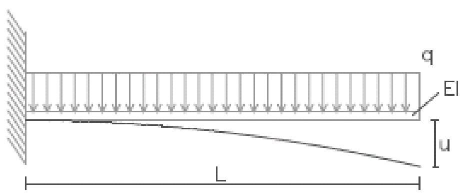


Figure 9-35: scheme with unconnected columns

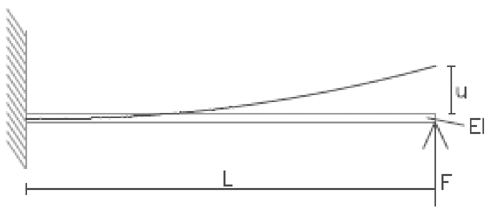
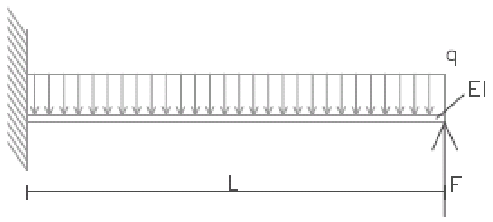


Figure 9-36: Deformation caused by force F



$$\left.\begin{aligned} u &= \frac{q * l^4}{8EI} \\ u &= \frac{F * l^3}{3EI} \end{aligned}\right\} F = \frac{3}{8} ql$$

Figure 9-37: scheme with connected columns

Force F (calculated in figure 9-37) is necessary for the stress calculation of the perimeter column and the core. The distribution of the forces between the core and columns is given is figure 9-37. The shaded part of figure 9-38 is the part of the storey weight which will be carried by the core. The rest of the weight (the white part of the figure) will be distributed between the columns. The part of the storey carried by the core is equal to:

$$A = \pi * (\left(r_{core} + \frac{5}{8} * l \right)^2 - r_{core}^2) = \pi * (\left(15.25 + \frac{5}{8} * 7.75 \right)^2 - 15.25^2) = 540 m^2$$

$with$

$l =$ distance between core and force F in figure 9-37

The floor area carried by one column is now equal to:

$$A_{column} = \frac{A_{floor} - A_{core}}{6} = \frac{1500 - 540}{6} = 160 m^2$$

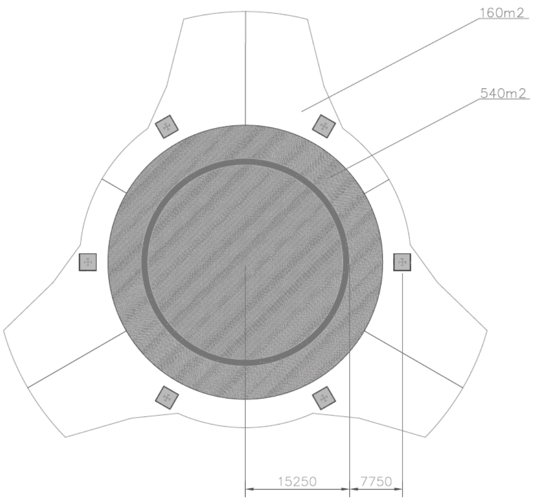


Figure 9-38: force distribution between core and columns

Global dimensions

Figure 9-39 shows the connection in more detail. The total height of the connection is 700 mm, just like the space between the storeys. The slope in the "guidance" structure has a height of 120 mm, so it will be able to equal the deflection of the storey above. The maximum deflection (located at the columns) will be equal to 0.004 * 9000= 36 mm. The connection is made in such a way that the storey above will be pushed up so the storey will be horizontal. The "guidance" structure has a length of approximately 7 meters (creating a slope with an angle of 1^0) and has a circular shape following the shape of the storeys (this was not indicated in the concept in figure 9-34).

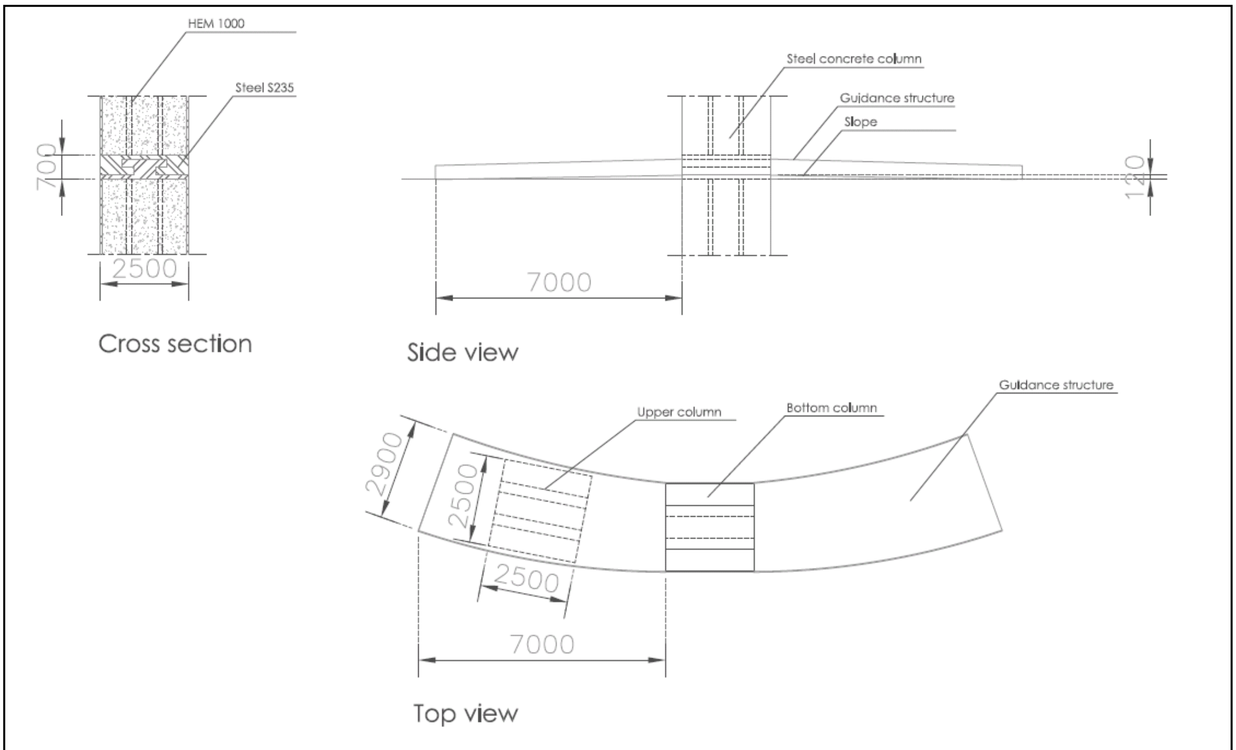


Figure 9-39: Concept connection

Connection

The connection given in this section is just a rough concept of how it could be made. Its main purpose is to give an indication of the possible solution for the connection used in alternative 4. The way the connection is drawn in figure 9-39, makes it very likely that noise disturbance occurs. This is one of the essential points that need to be considered when designing this connection into more detail.

9.6.5 Outrigger (calculation)

The structure contains 2 outrigger levels. A part of the levels containing outrigger trusses is not able to rotate. The outrigger trusses are placed in the circular part of the floor plan. This part is not able to rotate. The "wings" of the storeys will be able to rotate in this alternative to make sure that the exterior character will be unchanged.
Each level contains 12 outrigger trusses. When the outrigger braced structure is in use (with high wind speeds), only 6 trusses are in use per level (because there are only 6 perimeter columns). The storeys will be positioned in such a way that the tower experiences the least wind load (see also chapter 6). To make sure the tower can be placed in (almost) every position possible, the perimeter columns must be able to be connected to the outrigger trusses at more than one location.

Calculation [20, 21,22 & 23]
The calculation of an outrigger braced structural system is more complicated than for a regular core (like the previous sections). To calculate the deformation, the total system needs to be simplified tot the scheme in figure 9-40.

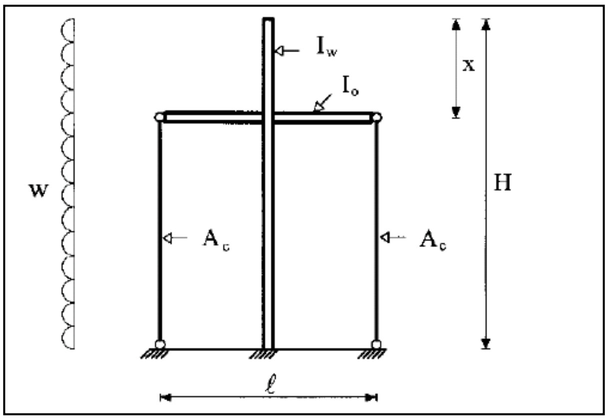


Figure 9-40: outrigger model

Where w is the wind load (uniform distributed), I_w the moment of inertia of the core, I_o the moment of inertia of the outrigger truss and A_c the sectional area of the façade columns. In such a system the total deformation is calculated by formula 9-12.

$$\delta_{bending} = \frac{w * H^4}{8EI_w} - \frac{M_c(H^2 - x^2)}{2EI_w}$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w}\left\{\left(\frac{1}{EI_w} + \frac{2}{EA_c * l^2}\right)(H - x) + \left(\frac{l}{12EI_o}\right)\right\}^{-1}$$

Formula (9-12)

In the formula for calculating the deformation of an outrigger system the wind load is taken as an uniform distributed load. In practice the wind load is not uniformly distributed. Therefore an equivalent load is calculated the same way the relative bending stiffness is calculated (this is explained in section 9.1)

Like the formula indicates the additional stiffness of the outrigger structure depends on 3 variables:

- The cross sectional area and stiffness of the columns
- The bending stiffness of the outrigger
- The distance between the outriggers.

Beside these three quantities, also the number of outrigger levels used is an important quantity. Formula 9-12 is only valid when applying one level of outrigger trusses in the structure. To create more stiffness, more outrigger levels can be applied in the structure. The calculation changes to a more complex one:

$$\delta_{bending} = \frac{w*H^4}{8EI_w} - \frac{H^2}{2EI_w}\sum_{i=1}^{n} M_i(1-\xi_i^2)$$

$$\begin{pmatrix} M_1 \\ M_2 \\ . \\ M_i \\ . \\ M_n \end{pmatrix} = \frac{wH^2}{EI_w S}[A][B]^T$$

$$S = \frac{1}{EI_w} + \frac{2}{EA_c l^2}$$

$$[A] = \begin{bmatrix} \omega+(1-\xi_1) & 1-\xi_2 & 1-\xi_3 & .. & 1-\xi_i & .. & 1-\xi_n \\ 1-\xi_2 & \omega+(1-\xi_2) & 1-\xi_3 & .. & 1-\xi_i & .. & 1-\xi_n \\ 1-\xi_3 & 1-\xi_3 & \omega+(1-\xi_3) & .. & 1-\xi_i & .. & 1-\xi_n \\ .. & .. & .. & .. & .. & .. & .. \\ 1-\xi_i & 1-\xi_i & 1-\xi_i & .. & \omega+(1-\xi_i) & .. & 1-\xi_n \\ .. & .. & .. & .. & .. & .. & .. \\ 1-\xi_n & 1-\xi_n & 1-\xi_n & .. & 1-\xi_n & .. & \omega+(1-\xi_n) \end{bmatrix}^{-1}$$

$$[B] = \begin{bmatrix} 1-\xi_1^3 & 1-\xi_2^3 & 1-\xi_3^3 & .. & 1-\xi_i^3 & .. & 1-\xi_n^3 \end{bmatrix}$$

$$\omega = \frac{S_1}{SH}$$

$$\xi_i = \frac{x_i}{H}$$

$$S_1 = \frac{l}{12EI_o}$$

Formula (9-13)

When using more truss levels, the structure becomes stiffer. To have a high stiffness, it would be the best solution to use an infinite number of outrigger truss levels. When doing this the perimeter columns will be connected to the core continuously. In practice this is not possible, so an finite number of outrigger trusses will be used.
Because a part of the storeys containing an outrigger won't be able to rotate, as few outrigger levels as possible will be used. The table below gives an comparison of the reduction on the deformation for a different number of outrigger levels (with different storey height, but with the same number of storeys out of use):

Nr of truss levels	Number of storeys out of use	Deformation reduction
2	10	629 [mm]
3	9	617 [mm]
4	8	600 [mm]
5	10	606 [mm]

9.6.6 Deformation

The deformation at the top of the tower is calculated for the 2 different modes the tower can appear in:

- Rotating: only core to resist lateral loads (wind force up to 8 Beaufort)
- Static: core and outrigger braced columns to resist lateral loads (max. wind force)

Deformation when rotating

The total deformation of the tower can be calculated with formula 9-4. The total calculation is given in the appendix.

Deformation	Magnitude	Unit
Rotation foundation	41	mm
Bending core	333 (extra bending = 6)	mm
Second order effect	24	%

Combining these values gives a total deformation of **464 mm**. According to the Eurocode a deformation of 752mm is allowed with at height.

Deformation when in static position

When the wind force exceeds 8 Beaufort (20.7 m/s wind speed at a reference height of 10 meter above ground level) the storeys will stop rotating and the outrigger – perimeter column structure will be activated. A stiffer structure will appear in this way. The deformation of the system will be calculated with formula 9-4 and 9-13.

Deformation	Magnitude	Unit
Rotation foundation	138	[mm]
Bending core	1117 (extra bending =25)	[mm]
Reduction outrigger	-629	[mm]
Second order effect	10.8	[%]

Combining these values gives a total deformation of **700 mm**. According to the Eurocode a deformation of 752mm is allowed for this height. Because of the high dead weight the structure is sensitive to the second order effect. The effect exceeds 10% and needs to be taken into account.

9.6.7 Accelerations at the top

The maximum calculated value of the acceleration is 0.092m/s^2. This is below the maximum allowed value given in figure 4-1. Because of the high dead weight, the structure is not sensitive to large accelerations.

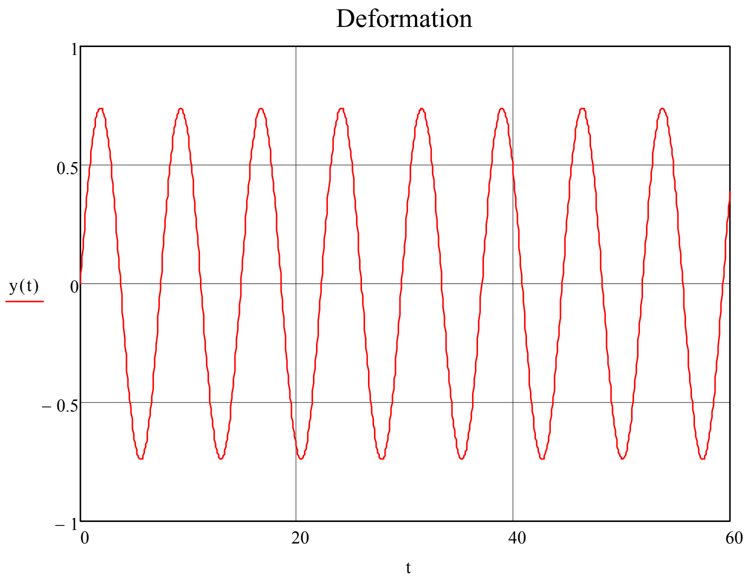


Figure 9-41: accelerations tower

9.6.8 Stresses core

The stresses in this section will be determined for the governing cross-section for three different events:

1. Dynamic structure

The storeys are rotating and the outrigger structure is not activated. The core is the only lateral load bearing structure in the tower. In this situation a maximum wind force of 8 Beaufort is allowed.

2. Static structure

When wind forces above 8 Beaufort are expected/measured, the tower will stop rotating. The (stiffer) outrigger-perimeter structure is activated. The design wind force in this situation is the maximum wind load expected (see chapter 6).

3. Static structure with mechanical failure

The availability of the outrigger-perimeter structure depends on the rotation of the storeys. When mechanical failure occurs, the structure won't be able to connect the perimeter columns. In that situation the core will be the only available structural element. Because the chance of occurrence of both maximum wind force and mechanical failure is very small, it will be allowed that the tower does not meet the requirements for maximum deformation. It will however not be allowed that the tower does not meet the requirements for strength design, or in other words that the tower will collapse when this situation occurs. Therefore this situation is calculated in this chapter.

1. Dynamic structure

Normal force

<u>SLS</u>
1.0 * 315 [kg/m^3] *(1500 +613) [m^2] * 376 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 62025 [kN] + 1.0 * 1990 [kN/floor] * 70 [floors] = **2656MN**

<u>ULS wind max</u>
1.3 * 315 [kg/m^3] *(1500 + 613) [m^2] * 376 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 62025 [kN] + 1.65 * 1990 [kN/floor] * 70 [floors] = **3502MN**

<u>ULS wind min</u>
0.9 * 315 [kg/m^3] *(1500 + 613) [m^2] * 376 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0* 1990 [kN/floor] * 70 [floors] = **2265MN**

<u>ULS earthquake max</u>
1.2 * 315 [kg/m^3] *(1500 + 613) [m^2] * 376 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 62025 [kN] + 0.5 * 1711 [kN/floor] * 70 [floors] = **3080MN**

<u>ULS earthquake min</u>
0.9 * 315 [kg/m^3] *(1500 + 613) [m^2] * 376 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0* 1711 [kN/floor] * 70 [floors] = **2265MN**

Wind load (ULS)
$M_{wind;2th\ order}$ = **6.68 *10^6 kNm**

Earthquake load (ULS)
$M_{earthquake}$ = **1.564 *10^7 kNm**

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{3080*10^6}{0.95*136.7*10^6} = 23.7N/mm^2$$
$$\sigma_{n;\min} = \frac{N}{A} = \frac{2265*10^6}{0.95*136.7*10^6} = 17.4N/mm^2$$
$$\sigma_m = \frac{M*e}{I} = \frac{1.564*10^{13}*15250}{0.9*1.44*10^{16}} = 18.4N/mm^2$$

The stresses stay within the limits for concrete C90/105: maximum compressive stress is 42.1 N/mm^2 and a maximum tensile stress of -1 N/mm^2.

2. Static structure- outrigger structure activated
Normal force
SLS
1.0 * 315 [kg/m^3] *(1500 + 613) [m^2] * 126 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 315 [kg/m^3] *(540 + 613) [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 62025 [kN] + 1.0 * 1990 [kN/floor] * 70 [floors] = **1915MN**

ULS wind max
1.3 * 315 [kg/m^3] *(1500 + 613) [m^2] * 126 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 315 [kg/m^3] *(540 + 613) [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 62025 [kN] + 1.65 * 1990 [kN/floor] * 70 [floors] = **2538MN**

ULS wind min
0.9 * 315 [kg/m^3] *(1500 + 613) [m^2] * 126 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 315 [kg/m^3] *(540 + 613) [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0 * 1990 [kN/floor] * 70 [floors] = **1598N**

ULS earthquake max
1.2 * 315 [kg/m^3] *(1500 + 613) [m^2] * 126 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 315 [kg/m^3] *(540 + 613) [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 62025 [kN] + 0.5 * 1711 [kN/floor] * 70 [floors] = **2200MN**

ULS earthquake min
0.9 * 315 [kg/m^3] *(1500 + 613) [m^2] * 126 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 315 [kg/m^3] *(540 + 613) [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0 * 1711 [kN/floor] * 70 [floors] = **1598N**

Wind load
$M_{wind;2th\ order}$ = **0.703*10^7 kNm (SLS)**
$M_{wind;2th\ order}$ = **1.286*10^7 kNm (ULS)**

Earthquake load
$M_{earthquake}$ = **0.98 *10^6 kNm (ULS)**

Stresses SLS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{1915*10^6}{0.95*136.7*10^6} = 14.7N/mm^2$$
$$\sigma_m = \frac{M*e}{I} = \frac{0.70*10^{13}*15250}{0.9*1.44*10^{16}} = 8.3N/mm^2$$

Because no tensile tresses occur in the core, the concrete stays uncracked (important aspect because the E-modulus used in the deformation calculation is calculated assuming the concrete would stay uncracked).

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{2538*10^6}{0.95*136.7*10^6} = 19.5 N/mm^2$$
$$\sigma_{n;\min} = \frac{N}{A} = \frac{1598*10^6}{0.95*136.7*10^6} = 12.3 N/mm^2$$
$$\sigma_m = \frac{M*e}{I} = \frac{1.265*10^{13}*15250}{0.9*1.44*10^{16}} = 14.9 N/mm^2$$

The stresses stay within the limits for concrete C90/105: maximum compressive stress is 34.4 N/mm^2 and a maximum tensile stress of -2.6 N/mm^2.

3. Static structure – mechanical failure
Normal force of this event is equal to the normal stresses of situation one:

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{3502*10^6}{0.95*136.7*10^6} = 27 N/mm^2$$
$$\sigma_{n;\min} = \frac{N}{A} = \frac{2265*10^6}{0.95*136.7*10^6} = 17.4 N/mm^2$$
$$\sigma_m = \frac{M*e}{I} = \frac{2.294*10^{13}*15250}{0.9*1.44*10^{16}} = 26.9 N/mm^2$$

The stresses stay within the limits for concrete C90/105: maximum compressive stress is 53.9 N/mm^2 and a maximum tensile stress of -9.5 N/mm^2. This situation is governing for the core. Because of the tensile stress of -9.5 N/mm^2, reinforcement need to be applied. The reinforcement ratio is determined with GTB-table 10.4-g (see appendix H) : 2.0% (this value is lower than the reinforcement ratio determined in appendix F for partial instability of the core).

9.6.9 Stresses perimeter columns

<u>**Stresses**</u>
The columns will we loaded to maximum caused by wind load (see also stresses for the core). The columns will only experience normal forces and no bending moments:

Dead – live load

<u>Max</u>
1.3 * 315 [kg/m^3] *(160) [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} +1.65 * 1990 [kN/floor] * 46 [floors] * 1/6 = **186MN**

<u>Min</u>
0.9 * 315 [kg/m^3] *200 [m^2] * 250 [m] * 9.81 [m/s^2] *10^{-3} +0.0 * 1990 [kN/floor] * 46 [floors] * 1/6 = **111MN**

Overturning moments
$M_{wind;2th\ order}$ = 8.33*10^6 kNm

The total moment in distributed over 2 sets of columns:

$$N_{column} = \frac{8.33*10^6}{2} * \frac{1}{41} = 10.2*10^4 kN$$

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{(186+102)*10^6}{2500^2} = 46.1 N/mm^2$$

$$\sigma_{n;\min} = \frac{N}{A} = \frac{(111-102)*10^6}{2500^2} = 1.4 N/mm^2$$

9.6.10 Conclusion

An outrigger braced system with rotating storeys is, because of the pre-conditions, not able to resist lateral forces up to 435 meter. A height of 376 meter (70 storeys) is the maximum height which can be realised within this concept. Both deformation and maximum stresses were governing for this alternative.
This alternative is a very ambitious concept with a lot of risks. The most important one is the chance of mechanical failure and the effect of that event. When the storeys won't be able to rotate, the outrigger structure can't be activated and the tower is not able to resist maximum wind forces. This risk is taken into account in the design calculation.

9.7 Perimeter columns with stiff floors (alternative 6)

Alternative 5 contains a small adjustment of the concept. The first 37 storeys (containing office and hotel space) are partly non rotatable. The circular part of the steel structure (a ring of 9 meter around the core at any height) is kept static. The "wings" of the storeys are still able to rotate around this static part. The concept is therefore a little different, because not all storeys are able to rotate completely around the core.
The most important aspect of the tower, the exterior character, is kept intact. The building can still change into every shape imaginable. Also the first 37 floors do still contain rotating parts: the wings. The size of the wings is of considerable size to use it as office/hotel space making these rooms more high end.

9.7.1 Structural system

The concrete core has the same dimension as in alternatives 3 and 4. The core has a varying diameter:

- From 0 - 200 meter: 30.5 meter
- From 200 - 376 meter: 27 meter
- From 375 - 405 meter: 20 meter

The wall thickness of the core is 1.5 meter and a concrete grade C90/105 is used in the calculations.
From 0 to 200 meter the core is surrounded by 12 steel/concrete columns to create more lateral stiffness. The core and columns are connected to each other with stiff diaphragms (concrete floors). The connection between the columns and the core creates a cohesive structure and the connection can be considered continuous.
Purposely columns are used for the outer ring of the structure and not a second concrete core, because this would make the room inside the second structure useless. By using columns there is still sufficient light inside the floors to keep them in use as office/hotel space.
The columns are steel/concrete columns with a size of 2 x 2 m^2. The columns consist of a steel square tube (with wall thickness 75mm), two HEM1000 profiles and are filled with concrete. Figure 9-48 shows the main layout of one of the columns.
The columns are only available up to the 37^{th} floor. Above that floors the storeys contain apartments (up to 12 per floor) and it is not practical to have a static and rotating part within these floors. It is not possible to make a promising floor plan for apartments on these floors.

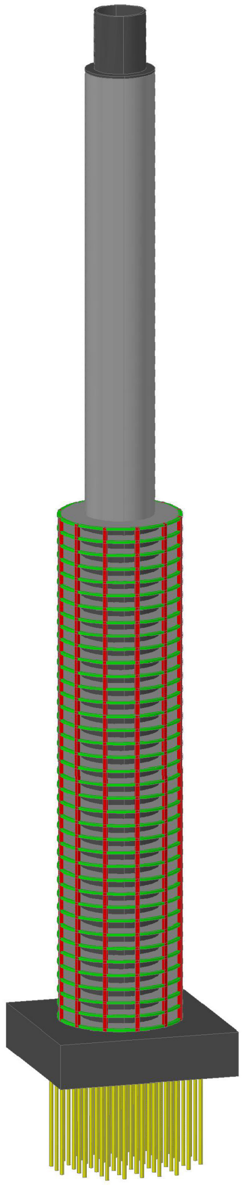


Figure 9-42: alternative 5

9.7.2 Foundation

The foundation of alternative 5 is similar to the foundation of alternative 4. This means a foundation consisting of 192 cast-in-place piles with a length of 45 meter and a diameter of 1.5 meter. The foundation slab is made of 6 meter thick C70/85 concrete. The only difference from the foundation of alternative 4 is the connection of the columns. In this case the 12 perimeter columns are permanently connected to the foundation. Because of the wide base of the foundation, a very stiff foundation is created making sure the deformation caused by rotation of the foundation stayed within the limits.

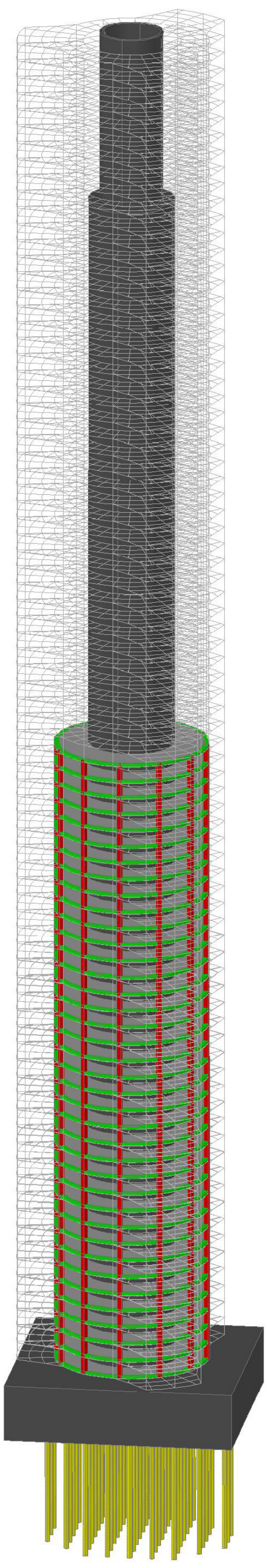


Figure 9-43: structural system static position

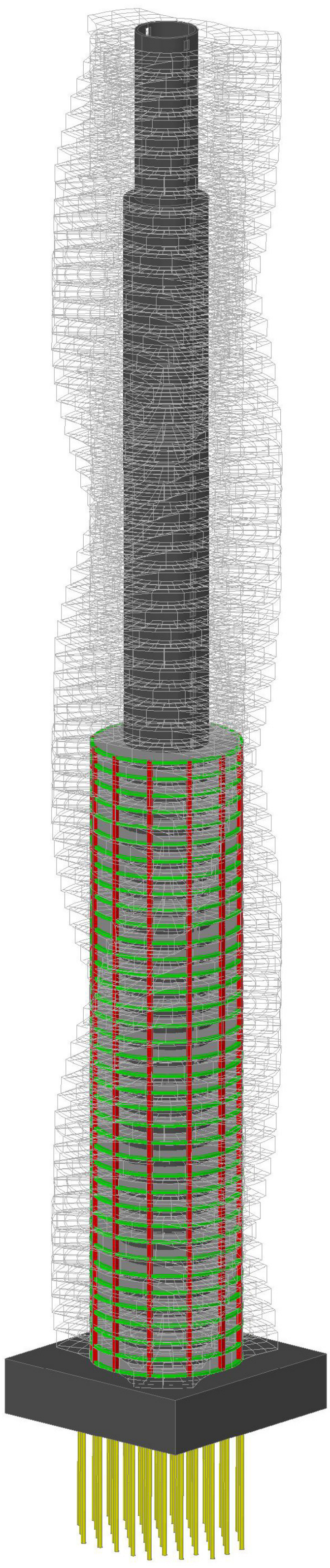


Figure 9-44: structural system when rotating

Layout

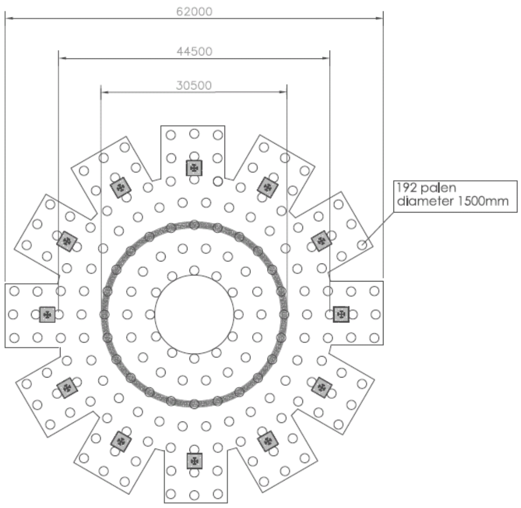


Figure 9-45: layout foundation

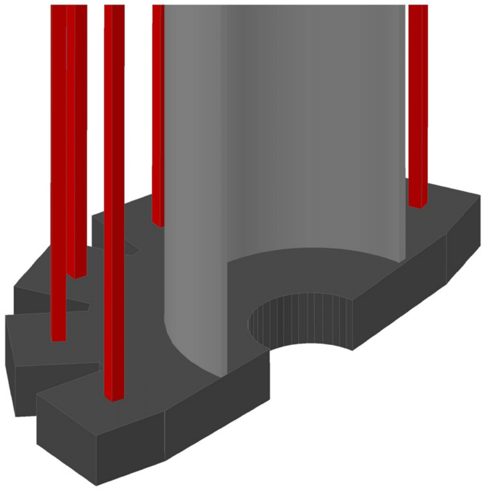


Figure 9-46: 3d model foundation

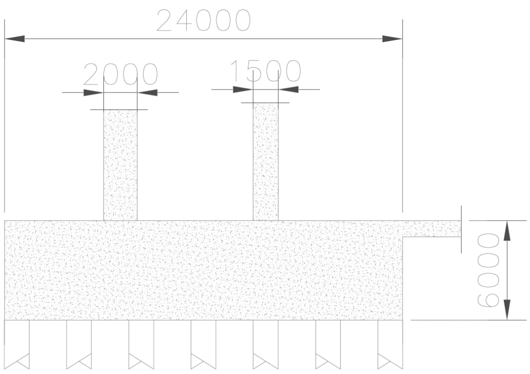


Figure 9-47: cross section foundation

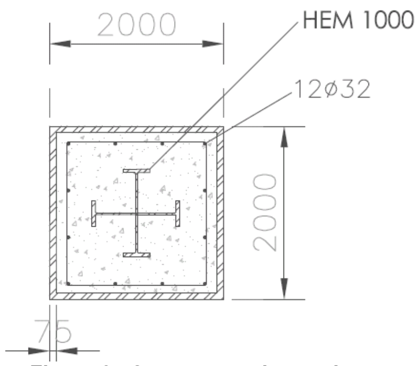


Figure 9-48: cross section perimeter column

Loads maximum loaded pile static situation

Al pile load are calculated with an 3D computer analysis (for in/output see appendix)

Load case	SLS	ULS
Max wind load (moment)	5.6 [MN/pile]	9.2 [MN/pile]
Max wind load (moment) Inc. second order	- [MN/pile]	10.3[MN/pile]
Dead + live load	21.2 [MN/pile]	27.5 [MN/pile]

Characteristics piles

Part	Result
Pile Type	Round concrete cast-in-place pile, tube back by driving
Dimension	Diameter 1.5 [m] ; Length 45 [m]
Slip layer	Bentonite
Bearing capacity (limit state 1B)	42.6 [MN]
Deformation due to total load (limit state 2)	0.034 [m]
Deformation due to total load excl. wind load (limit state 2)	0.027[m]
Spring stiffness	1200 [MN/m]

Rotational stiffness foundation

Figure 9-49 shows the deformations of the foundation slab. The deformation is linear distributed along the slab underneath the core. Therefore the rotational stiffness can be calculated by taking the maximum deformation and calculating the angle of rotation with that deformation.

$$\left.\begin{aligned} C &= \frac{M}{\varphi} \\ \varphi &= \frac{4.7}{14750} = 3.186 * 10^{-4}\, rad \\ M &= 12880\, MNm \end{aligned}\right\} C = 4000 * 10^{4}\, MNm/rad$$

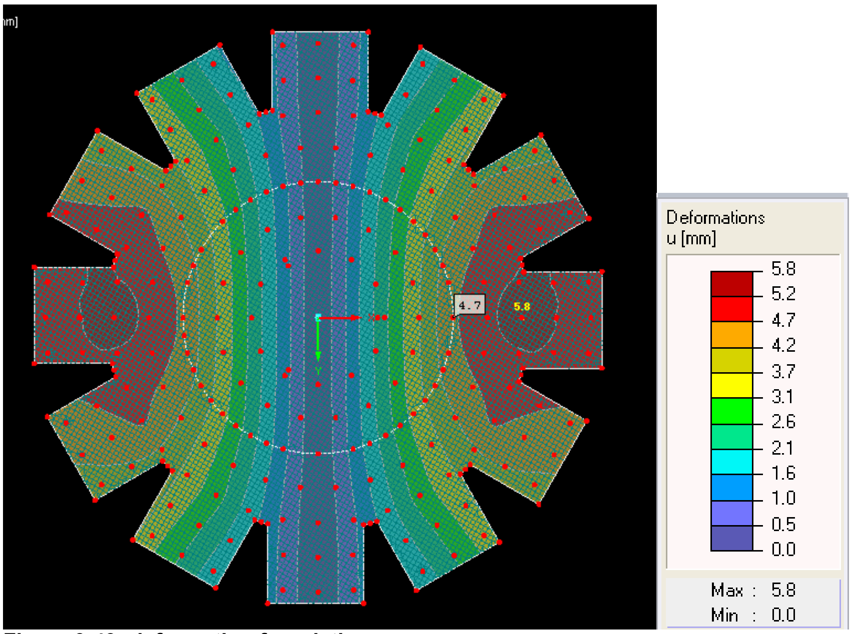


Figure 9-49: deformation foundation

9.7.3 Deformation

The total deformation of the tower can be calculated with formula 9-4. The total calculation is given in the appendix.

Deformation	Magnitude	Unit
Rotation foundation	136	mm
Bending core	564 (extra bending = 110mm)	mm
Second order effect	11.3	%

The total deformation of the tower **779mm** (where 810mm is allowed). This is well within the limits from the Eurocode.

9.7.4 Accelerations at the top

The maximum calculated value of the acceleration is 0.097m/s^2. This is below the maximum allowed value given in figure 4-1. Because of the high dead weight, the structure is not sensitive to large accelerations.

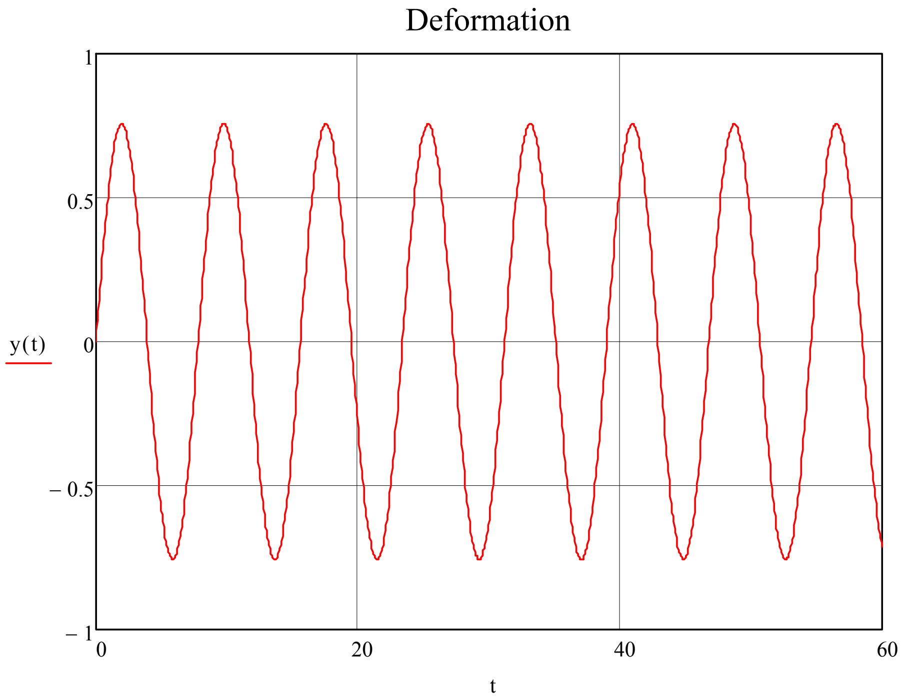


Figure 9-50: accelerations tower

9.7.5 Stresses

9.7.5.1 Core

Normal force

SLS
1.0 * 316 [kg/m^3] *(1500 +613) [m^2] * 205 [m] * 9.81 [m/s^2] *10^{-3} +1.0 * 316 [kg/m^3] *(540 +613) [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 1.0 * 62025 [kN] + 1.0 * 1990 [kN/floor] * 75 [floors] = **2269MN**

ULS wind max
1.3 * 316 [kg/m^3] *(1500 +613) [m^2] * 205 [m] * 9.81 [m/s^2] *10^{-3} +1.3 * 316 [kg/m^3] *(540 +613) [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 1.3 * 62025 [kN] + 1.65 * 1990 [kN/floor] * 75 [floors] = **3002MN**

ULS wind min
0.9 * 315 [kg/m^3] *(1500 +613) [m^2] * 205 [m] * 9.81 [m/s^2] *10^{-3} +0.9 * 315 [kg/m^3] *(540 +613) [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0 * 1990 [kN/floor] * 75 [floors] = **1908MN**

ULS earthquake max
1.2 * 316 [kg/m^3] *(1500 +613) [m^2] * 205 [m] * 9.81 [m/s^2] *10^{-3} +1.2 * 316 [kg/m^3] *(540 +613) [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 1.2 * 62025 [kN] + 0.5 * 1711 [kN/floor] * 75 [floors] = **2618MN**

ULS earthquake min
0.9 * 315 [kg/m^3] *(1500 +613) [m^2] * 205 [m] * 9.81 [m/s^2] *10^{-3} +0.9 * 315 [kg/m^3] *(540 +613) [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 0.9 * 62025 [kN] + 0.0 * 1990 [kN/floor] * 75 [floors] = **1908MN**

Wind load
M_{wind} = **0.40 * 10^7 kNm (SLS)**
$M_{wind;2th\ order}$ = **0.74 * 10^7 kNm (ULS)**

Earthquake load
$M_{earthquake}$ = **0.537 * 10^7 kNm (ULS)**

Stresses SLS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{2269*10^6}{0.95*136.7*10^6} = 17.36 N/mm^2$$

$$\sigma_m = \frac{M*e}{I} = \frac{0.40*10^{13}*15250}{0.9*1.44*10^{16}} = 4.7 N/mm^2$$

Because no tensile stresses occur in the core, the concrete stays uncracked (important aspect because the E-modulus used in the deformation calculation is calculated assuming the concrete would stay uncracked).

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{3002*10^6}{0.95*136.7*10^6} = 23.1 N/mm^2$$

$$\sigma_{n;\min} = \frac{N}{A} = \frac{1908*10^6}{0.95*136.7*10^6} = 14.7 N/mm^2$$

$$\sigma_m = \frac{M*e}{I} = \frac{0.74*10^{13}*15250}{0.9*1.44*10^{16}} = 8.7 N/mm^2$$

The stresses stay within the limits for concrete C90/105: maximum compressive stress is 31.8 N/mm^2 and no tensile stresses.

9.7.5.2 Columns

The columns will we loaded to a maximum caused by wind load (see also stresses for the core). The columns will only experience normal forces and no bending moments:

Dead – live load
The governing load case turned out to be wind load. The forces acting on the columns due to wind load are:

<u>Max</u>
1.3 * 316 [kg/m^3] *80 [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 1.65 * 1990 [kN/floor] * 37 [floors] * 1/12 = **74.6MN**

<u>Min</u>
0.9 * 316 [kg/m^3] *80 [m^2] * 200 [m] * 9.81 [m/s^2] *10^{-3} + 0.0 * 1990 [kN/floor] * 37 [floors] * 1/12 = **44.6MN**

Moments
$M_{wind;2th\ order}$ = 1.59*10^4 kNm

The total moment in distributed over 6 sets of columns. The maximum loaded column is the one with the biggest centre to centre distance (=z):

$$N_{column;\max} = \frac{M_{total}}{EI_{columns;total}} * \frac{EI_{column;\max load}}{z}$$

$$N_{column;\max} = \frac{1.59*10^{13}[Nmm]}{62000[N/mm^2]*1.29*10^{16}[mm^4]} * \frac{2000^2[mm^4]*\frac{46500^2}{2}[mm^2]*62000[N/mm^2]}{46500[mm]} = 114.7[MN]$$

Stresses ULS

$$\sigma_{n;\max} = \frac{N}{A} = \frac{(74.6+114.7)*10^6}{2000^2} = 47.3/mm^2$$

$$\sigma_{n;\min} = \frac{N}{A} = \frac{(44.6-114.7)*10^6}{2000^2} = -17.5N/mm^2$$

The stresses stay well within the limits (see appendix F).

9.7.6 Conclusion

Alternative 5 shows an alternative for the lateral load structure which honours the architectural exterior design and requires minor adjustments to the interior design to create a structure stiff enough to resist all forces. The structure can withstand loads up to a height of 405 meter. A tower with this height is exposed to very high wind forces and is also very sensitive to second order effects because of the high dead weight.

9.8 Comparison

This chapter presented 5 alternative designs for the main load bearing structure of the Rotating Tower. Al these alternatives are feasible (the non feasible alternatives are presented in section 9.2). This section will therefore give a comparison of the alternatives on two aspects: pros and cons and risks. Based on those two aspects, the best alternatives are chosen. These alternatives will be presented as final designs and worked out to a more detailed level in appendix F.

9.8.1 Pros and cons

First aspect where the alternatives are compared on are the pros and cons. The most important aspects where attention will be paid to are: is the design feasible within the concept of the Rotating Tower, which changes need to be made to the architect's design and how much commercial area is available (economic value)?

Alternative	Pros	Cons
Alternative 1 **"Architects Design"**	Architectural design unchanged	Height of just 45 storeys (loss of valuable space)
	Concept of rotating storeys unchanged	
Alternative 2 **"Higher concrete grade"**	Architectural design unchanged	Height of just 50 storeys (loss of valuable space)
	Concept of rotating storeys unchanged	Very high concrete grade used
Alternative 3 **"Increased wall thickness"**	Architectural design unchanged	Height of just 54 storeys (loss of valuable space)
	Concept of rotating storeys unchanged	Very high concrete grade used
		High self weight: increased sensitivity for second order effects. The increased stiffness can't be used to full capacity.
Alternative 4 **"Outrigger system"**	Height of 70 storeys (valuable space): economic value	Static (parts of) floors: change of concept
	Dynamic concept also present in structure	Perimeter columns take up valuable space
	Good representation of the original exterior design(height/broad proportion)	All floors must rotate in the same direction
Alternative 5 **"Perimeter columns with stiff floors"**	Height of 75 storeys (valuable space): economic value	First 37 storeys contain a static part: (small) adaptation architectural design)
	Small adaptation to the architectural design	Perimeter columns take up valuable space
	Possibility to decrease storey height: tower containing 80 storeys can be made with decreased height	
	Good representation of the original exterior design(height/broad proportion)	

9.8.2 Risks

In this section the risks are described for all alternatives. Risks considering al alternatives are described and alternative specific risks. The most important risks are described and will be linked to an alternative if necessary. The risks can be divided in four categories: wind, mechanical, foundation and concept.

Wind

1. Wind direction

The wind load is determined by assuming that the tower will be positioned with one of the wings in the same direction as the wind. This configuration of the tower gives the lowest wind load (see section 6.3.4). This condition is based on the assumption that the wind direction is constant. If the wind would change direction during a storm, the wind load will increase.
This risk is considered to be low, because of two main reasons. The first reason is the historical wind direction in Dubai (see figures 6-2 and 6-3). Great wind speeds in Dubai are reported to come from one main direction (the Burj Khalifa's orientation is even based on this main direction). Therefore it is expected that the wind direction will not change sufficiently. Beside this, figure 6-10 shows reason number 2. The minimum wind load is constant for wind varying with an angle of ± 15^0, so the wind can vary 15^0 without an effect on the wind load.

2. Rapidly increasing wind speeds

When wind speeds reach 8 Beaufort, the storeys of the Rotating Tower will be put on hold and rotated in the most efficient orientation. The storeys will stop rotating when storm conditions are forecasted or if large wind speed are measured. But (theoretical) it can occur that the wind speed rapidly increases to speeds above 8 Beaufort, while the tower is not rotated in the best orientation. This is especially valid for alternative 4, because the outrigger structure must be activated in this situation.
This risk is not of great importance, because the boundary of a wind speed of 8 Beaufort is not determined from a constructive point of view. This boundary is necessary for local effects on the storeys. For the main load bearing structure a wind speed greater than 8 Beaufort is no problem. Therefore there is a buffer between stop rotating and the critical wind speed.
Because of the above mentioned reason this risk is assumed to be low.

3. Exceeding wind load

The wind load is determined with Eurocode 1 and different literature. A wind tunnel test is necessary to determine the real value of the wind load. There is a chance that the wind load will be larger than the value used.
Because the assumptions are based on reliable literature, this risk is expected to be low. This can only be validated by wind tunnel testing.

Mechanical

4. Mechanical failure storeys

This risk is particularly valid for alternative 4. When there's a mechanical failure, the outrigger structure won't be able to be activated. When this happens the load bearing structure only consists of the core. This risk is taken into account in the design of alternative 4. When the outrigger structure is not activated and maximum wind load occur, the tower must still meet the requirement for strength design. Increased deformation will be accepted, but the tower must not collapse.

5. Noise disturbance due to rotating storeys

This risk is particularly valid for alternative 4. When the perimeter columns connect to each other, a noise disturbance can occur. This risk can be managed by a proper design of the connection between the columns.

Foundation

6. Local conditions foundation

The conditions used in the calculations of the foundation are based on the conditions of a project nearby. Although there is a risk that the local conditions for the Rotating Tower will be different, it is expected that the conditions will be comparable. And if the conditions are different, the foundation design can be changed to create one with a comparable stiffness as used in the calculation.
Therefore this risk is assumed to be low.

Concept

7. Change of concept

All alternatives have one or more adaptations from the original architectural design (height, static parts, perimeter columns etc.). Because the designs are not presented to the architect, there is a chance none of the designs will be approved. To minimize this risks, the designs are al made considering the architect's point of view.

9.9 Conclusion

This chapter described 5 (feasible) designs for the Rotating Tower. At the end all alternatives are compared with each other concerning risks and advantages and disadvantages. Based on this comparison a conclusion can be drawn about the relevance of the different alternatives to the project. The alternatives which are the most promising:

Alternative 1 : architect's design
Alternative 1 is a direct translation of the architect's design of the main load bearing structure. With this alternative no changes are made in the concept, except for the height. The choice for the final design of the structure will be made by the architect, and therefore it is very useful to have a complete view of the possibilities of the original design. In appendix F a more detailed calculation is made for the core, in order to create a complete view of the design.

Alternative 4: Dynamic outrigger structure
Alternative 4 is probable the most ambitious design made in this chapter. It contains the most risks of all designs, but also has the most similarity with the concept of the project (even the structure is dynamic). The main risk of this structure (mechanical failure) is anticipated in the design: the structure is design to withstand maximum wind loads in the situation of mechanical failure (only the deflection will exceed the maximum allowed value). The design contains (together with alternative 5) the most similarity with the original exterior design. The exterior design determines the charisma of the tower. Because of the relevance of this design, a more detailed calculation is made for this alternative in appendix F. This appendix contains a detailed calculation of the core, perimeter columns and outrigger truss.

Alternative 5: Core-perimeter column structure
Alternative 5 is (just like alternative 4) a design with the most similarity to the original exterior design. With a small adaptation to the original design, a structure with sufficient stiffness is designed. The bottom 37 floors contain a static part which connects the core to the perimeter columns. A big advantage of this structure is the lack of "extra" risks in comparison with the original design. In appendix F a more detailed calculation is given of the core and the perimeter columns.

These three design are the final designs for the lateral load bearing structure of the Rotating Tower, and will be presented to the architect. All three final designs are calculated to a more detailed level (in appendix F).

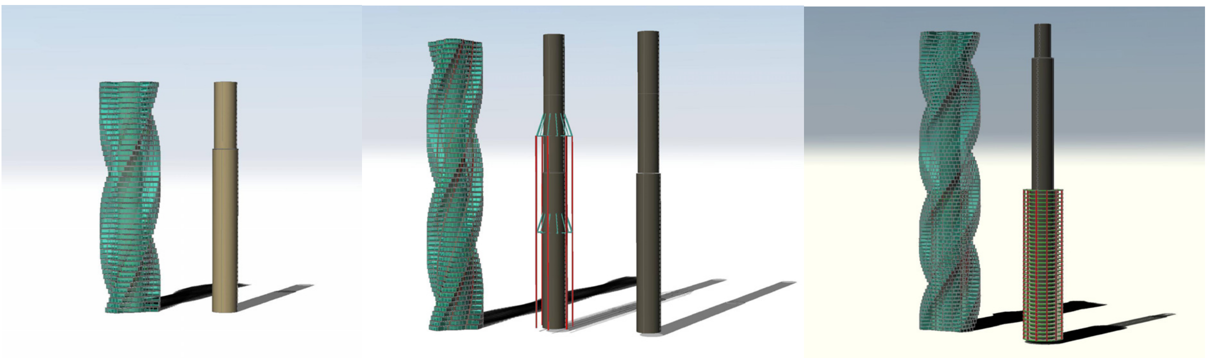


Figure 9-51: Final designs : alternatives 1, 4 and 5

10 Conclusion and recommendations

The main objective of this master study project was designing feasible load bearing structures for the Rotating Tower, within the set boundary conditions given by the architectural design. This objective is achieved by creating 5 feasible designs. From these 5 alternative designs, 3 final designs were chosen and worked out to a more detailed level. These 3 designs do however all contain one or more adaptations from the architectural design (varying from non rotatable parts to decreased height), but keep the main concept, functions and the storey dimensions of the project unchanged.

10.1 Conclusions

Feasibility
From a structural point of view the concept “Rotating Tower” is feasible, but not without concessions. To create a design which rises to the intended height of approximately 400 meter (80 storeys), just a structural core is not sufficient. The structural system must be extended with either an outrigger structure or perimeter columns.

Governing load case
The conclusion drawn from the reference projects indicated that the wind load was the governing load case for all projects. This is also true for the Rotating Tower (this was already expected). Figure 3-13 shows why this holds for all tall buildings: the frequency of earthquakes is that high, that the tall tower (which has a low natural frequency) does not experience large forces.

Final designs
To investigate the structural feasibility of the “Rotating Tower”, three designs were worked out to a more detailed level (see appendix F). All designs have one or more adaptations to the original architect’s design, but keep the main concept of individually rotating storeys intact. The adaptations vary from a decreased height to non rotatable parts of the tower.

Architect
The main objective of this thesis report is creating a feasible structural design. Eventually three final designs were made, to show the possibilities for the structural design within the concept of the “Rotating Tower”. It must be stated in this conclusion that the decision for the final design will be made by the architect of the project.

Personal preference
Alternative 5 (core – perimeter columns) is considered the personal preference. In my opinion this design comes closest to the original idea for the “Rotating Tower”. With this design the greatest height can be realized, with only a small adaptation of the original concept (the first 37 floors must contain a static part). The commercially most valuable space (apartments and villas at the top storeys) is still available and the status of the building remains unchanged. But as stated before, the final choice will be made by the architect.

Risks
All alternative designs face some risks. The most important risks are: different wind load (either because of changing wind direction or wrong assumptions), different local foundation conditions and acceptance of the designs by the architect. Alternative 4 has a few more risks: noise disturbance and mechanical failure of the storeys (so the outrigger structure can’t be activated).

Acceptance dynamic load bearing structure
The “dynamic outrigger structure” is very ambitious, but fits perfectly in the concept “dynamic architecture”. However it must be realized that the risks of this alternative design are many times larger than for all other designs and there’s a chance that the alternative will not be accepted by the local authority.

Dynamic architecture
The concept "dynamic architecture" is very ambitious and still faces a few important challenges. From a structural point of view it is possible to make the concept reality. Therefore it is expected that the concept will be realized in the future.

10.2 Recommendations

Foundation
The foundation conditions are based on another project in Dubai. It is advisable that a CPT (cone penetration test) is performed on the final project location to determine the exact conditions. These conditions are important input for the design (and rotational stiffness) of the foundation.

Wind tunnel testing
In chapter 6 the wind load is determined by using codes and literature. It must be stated in this conclusion that a wind tunnel test is obligated in Dubai. This wind tunnel test can be used to optimize the design of the tower and it is advised to do so.

Building shape
Changing the building shape to a more aerodynamic one, can have a great effect on the wind load. Because wind is the governing load case, this would have a direct effect on the forces acting on the tower. Therefore it is advised that the building shape is changed during the wind tunnel tests to create a shape with the best aerodynamic quality (but with the same characteristics as the architectural design). This can be done by chamfering the corners for instance.

Active systems
Because the risks of using active systems were considered to be too high, this option was not used. Lack of experience (both personal and in previous projects) and the enormous consequences caused by mechanical failure, were both arguments not to use this solution.
It is however an option which might be the perfect solution for the "Rotating Tower". In this project the deformations are governing for all alternatives. Active systems (like hydraulic jacks) counteract these deformations, so the structure can be made very slender (probably without additional structures like outriggers). Therefore it can be useful to investigate this solution possibility into more detail for future use.

E-modulus concrete
The E-modulus used in the calculation of the designs is based on a save assumption (which also takes long term effects into account, see formula 9-10). Because the designs are worked out to a concept level, this assumption was used. A more detailed calculation of the E-modulus can result in a higher value. Therefore it is advised to do so in a further stage of the project.

Creep in steel-concrete columns
The cross section of the steel concrete columns are determined to create columns with enough stiffness to reduce the deformation. In the calculation of the stiffness of the columns the creep effect is taken into account (see chapter 9). Further investigation on the effect of concrete creep on the concrete steel column is however necessary, because steel is not sensitive to creep and the effect of this is unknown in this stage.

Connection perimeter columns (alternative 4)
One important aspect of alternative 4 has to be designed and engineered to a more detailed level: the connection of the perimeter columns. A global concept is given in chapter 9, but this design needs to be elaborated. The connection of the perimeter columns is an important aspect of alternative 4 and also determines the feasibility of this concept.

Transition core diameter
The design of the core contains different diameters over the height. In the zone where the diameter changes (for instance from 30.5 to 27 meter), a complex transition arises. This transition zone must be considered with more detail. The most important aspects which need to be considered are:

- Force transfer from upper part to bottom part.
- Horizontal forces in the core due to transition.

Connection storeys and core

The connection of the core and the storeys (see section 5.2) is worked out to a concept level. According to this concept design (and concept calculation) the connection is feasible in the current situation. It is advised to engineer this connection to a more detailed level, because it is an important aspect in the feasibility of the project.

References

Articles and books

[1] Fisher; D; *Dynamic architecture: the Rotating Tower Dubai;* RTTI; 2008

[2] Van der Schans, J.; *Rotating Tower: Vooronderzoek, menselijke perceptie van de rotatie en het voorontwerp van de ophanging en de aandrijving van de verdiepingen;* 2008

[3] Chang, C: *structural design of Taipei 101*; evergreen consulting

[4] Shieh, S, Chang, C & Jong, J; *structural design of composite super-columns for the Taipei 101 tower; evergreen consulting;*

[5] Baker, W, Korista, D & Novak, L: *Engineering the world's Tallest-Burj Dubai*; CTBUH 8th World Congress; 2008

[6] Maertens, L ; *The Burj Dubai Tower;* Besix, November 2008

[7] Katz, P, Robertson, L & See, S; *Shanghai world financial centre: without compromise…;* CTBUH 8th World Congress; 2008

[8] Hayashida, Iwasa; *Aerodynamic shape effects of tall building of vortex induced vibration;* Journal of wind engineering and industrial aerodynamics, 1990

[9] *Geotechnical investigation report for Hamriya maritime city part A;* Buynunah laboratories, April 2008

[10] *Poulos, H & Bunce, G; Foundation design for the Burj Dubai*; 6th international conference on geotechnical engineering, august 2008

[11] Simone, A*; Analysis of slender structures*; TU Delft, 2010

[12] Everts,H; *Veerstijfheid paalfunderingen;* Cement en Beton; 1998

[13] Van Tol, A*; Fundamenteel 2;* Cement en Beton; 1995

[14] Verruijt, A; Grondmechanica; TU Delft; 2001

[15] Van der Marel, A; *CT3051A Beton- deel 2*; TU Delft; September 2006

[16] Abdalla, J. A & AL-Homoud, A.; Earthquake hazard zonation of eastern Arabia; 13th World Conference on Earthquake Engineering 2004

[17] Nijsse, R; *intreerede: "over strekte, stijfheid en beweging. De latente koppeling tussen voelbare beweging en veiligheid";* TU Delft februari 2008

[18] Font Freide, J.J.M & Snijders D.P; *Verticale stad aan de Maas;* Cement online 2010

[19] De Boo, A.J & Mans, D.G; *Delftse Poort, Hoogste kantoorgebouw van Nederland;* Cement online 1990

[20] Hoenderkamp, J.C.D; *High rise structures: preliminary design for lateral load;* TU Eindhoven 2007

[21] Hoenderkamp, J.C.D & Snijder, H; *Simplified analysis of façade rigger braced high-rise structures;* TU Eindhoven 2000

[22] Hoenderkamp, J.C.D; *Shear wall with outrigger trusses of wall and column Foundations;* TU Eindhoven 2003

[23] Wu, J.R & Li, Q.S; *Structural performance of multi-outrigger braced tall buildings;* City University of Hong Kong 2002

[24] Bayati, z, Mahdikhani, M & Rahaei, A; *Optimized use of multi-outriggers system to stiffen tall buildings;* Marikabir University of Technology 2008

[25] Braam, C.R & Lagendijk, R; *Constructieleer gewapend beton;* Cement en Beton 2009

[26] Sagel, R & van Dongen, A.J; *Constructieleer gewapend beton;* Cement en Beton 2006

[27] Maatschap Stark; *Staal-beton constructies;* Vereniging bouwen met staal / betonvereniging 2009 / 2010

Codes

[28] NEN-EN 1990: Basis of structural design

[29] NEN-EN 1991-1-1: General actions - Densities, self-weight, imposed loads for buildings

[30] NEN-EN 1991-1-4: general actions: wind actions

[31] NEN-EN 1992: Design of concrete structures

[32] NEN-EN 1994-1-1: Design of composite steel and concrete structures

[33] UBC97 Volume 1 & 2

[34] NEN 2443: off street and multi-storey car parks

[35] NEN 6702: Technical principles of building structures: loading and deformation

[36] NEN 6720: Regulation for concrete, structural requirements and calculation methods

[37] CUR aanbeveling 37: Hoge sterkte beton

[38] GTB 2010

Internet

[39] www.dynamicarchitecture.net

[40] http://en.wikipedia.org/wiki/

[41] http://www.rwdi.com/project/taipei_101#

[42] http://www.popularmechanics.com/technology/gadgets/news/1612252

[43] www.worldtravelguide2010.com

[44] www.gizmag.com

[45] www.everinghamrotatinghouse.com

[46] www.nd.edu

[47] www.windfinder.com

[48] http://earthquake.usgs.gov

[49] www.iitk.ac.in

Computer programs

[50] RFEM; Dlubal engineering software

[51] Mfoundation; GeoDelft

[52] Mathcad 14; PTC

[53] Autocad 2010; Autodesk

[54] Mathtype 6.7; Dessci

[55] Microsoft office 2007; Microsoft

[56] PDF creator; Adobe