Feasibility study on extended high-rise buildings

Elaboration phase

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Preface

In this report the elaboration phase of a thesis can be found. In combination with a literature study, this report describes a master thesis about the feasibility of extended high-rise buildings. In the introduction of this report an extensive summary is given about the literature study. In this summary, most important findings have been described. However, to get a profound idea about what has been done in the literature study, it is recommended to take the entire report into account.

The thesis is made at the structural company Zonneveld Ingenieurs. For the thesis, information which is available at the company has been used. From here I would like to thank Zonneveld Ingenieurs for giving me the opportunity to work on my thesis in an environment with skillful engineers, for providing me useful information with regard to various projects and for their valuable invested time.

The master thesis is guided by a committee. The committee exists out of four members, an overview of the committee is given below.

Prof. Ir. R. Nijsse (chairman)
Professor at the Department of Building Engineering, Civil Engineering and at the Department of Building Technology , Architecture, TU Delft

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Professor at the Department of Concrete Structures, Civil Engineering, TU Delft

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Consultative Engineer, Zonneveld Ingenieurs

From here I would like to thank the graduation committee for their useful help and provided knowledge during my research project.

Furthermore my gratitude goes out to Sven ten Hagen, as a fellow graduating student at Zonneveld Ingenieurs often we had useful discussions about both our thesis's. Also I would like to thank Bas, Calvin, Earmy, Huub, Pieter and Richard for considering me as one of their colleagues during the entire graduating process.

Next I would like to thank Mathis Chlosta and Jurjen Verploegh for checking important sections in the report on the English language, my parents for their financial support during my study. Last but not least my gratitude goes out to my girlfriend Merel for supporting me, during the process of my master thesis.

Hildo Herfst, January 2013

Abstract

Which factors determine the feasibility of an extension on top of a high-rise building? This question and its corresponding answer is the main topic of this master's thesis. A diagram was composed in which the most important factors are shown and explained. This diagram is provided at the end of this abstract.

Using the diagram as a starting point, a case study for the Oval Tower was conducted. The intention of this case study is to answer the question whether it is structurally feasible to extend the Oval Tower with two floors and a crown on top of the existing structure.

Previously, during a design competition held in 2010, a preliminary design was already made for an extension on top of the Oval tower. The Oval Tower is an existing building with a height of about 100m. However, following that competition, this design was never put into practice. According to experts in the field, this project still has a great potential. It was therefore chosen to further investigate the feasibility of this extension project.

General incentives for building on top of existing high-rise projects have to do with urban accents, improve on the image of a building and the increase of their financial value. In the general high-rise building process, one of the first steps is to analyse whether the project is feasible, concerning not only the before-mentioned initial incentives, but also the structural feasibility. The objective of this master's thesis is to give insight in the multitude of factors which determine this feasibility and make-ability of an extension on top of an existing high-rise project.

For the determination of these factors several reference projects have been elaborated. Both existing research projects as well as existing extended buildings have been taken into account. Subsequently a Quick Scan of three buildings has been made. In this Quick Scan the structural behaviour of three high-rise projects with an extension was analysed.

Using the results of the Quick Scan for all three buildings several key factors were derived, of which an initial diagram was composed. In this diagram a division was made between a technical, a functional and a financial inventory. It was chosen to mainly focus on the technical feasibility factors in the further research. The most important technical factors were found to be: use of materials, loads, level of safety, load bearing capacity of structural elements and construction methodology. Further investigation has been done throughout these factors. In the following subsections these factors will be described.

The first factor which determines the technical inventory is the use of materials. The research on building materials that has been conducted during this research shows that steel and timber are favorable materials for making a construction on top of an existing building. Next to that the choice for a load bearing- and floor-system are dependent on the architectural design of the extension.

For the loads which have to be applied to the existing building and its extension, the differences between the Eurocodes and older building standards are of importance. For this cause the current Eurocode has been compared to the older Dutch national code NEN6702. This comparison showed that differences are generally very small. A minor difference can be found in the design values of load due to wind thrust, which are slightly higher in case of application of the Eurocode. The shape of the extension also has an impact on the values of the design wind loads.

The third technical factor is the level of safety which will be prescribed by the classification procedure of the building. The following systematic judgement is recommended: For an existing building the level of safety for new buildings may be taken into account, unless the costs to fulfill the requirements of new buildings are disproportionately high. In this case the safety level for existing buildings is sufficient. The way the load factors have to be applied are ambiguous and are also related to additional building costs.

To get a profound insight in the load bearing capacity of structural elements which is the fourth technical factor, calculations have to be made on strength, stability and stiffness. The most important structural elements are the foundation piles, the foundation beams, the vertical load bearing elements, the roof floor, the structure of the extension and the connections between the new and the existing structure. This research resulted in three methods of strengthening an existing structure.

The final factor which determines the technical inventory is the construction methodology. An extension can be built with help of various construction methods. To compare these methods a trade off matrix was used. Besides standard criteria, additional criteria caused by the application of an extension can be used. Furthermore it is advised to also make a comparison on time, costs and risk for each construction method.

Having analyzed all inventories, several inter-inventory relationships and dependencies were also found. All findings were used to adapt the initial diagram. It is concluded that the inventories cannot be treated separately since they are highly related. Next to the diagram, recommendations have been made with regard to the working method. It is advised to split up the existing and new situation for the technical inventory. Using the diagram the before mentioned Oval Tower case study was conducted.

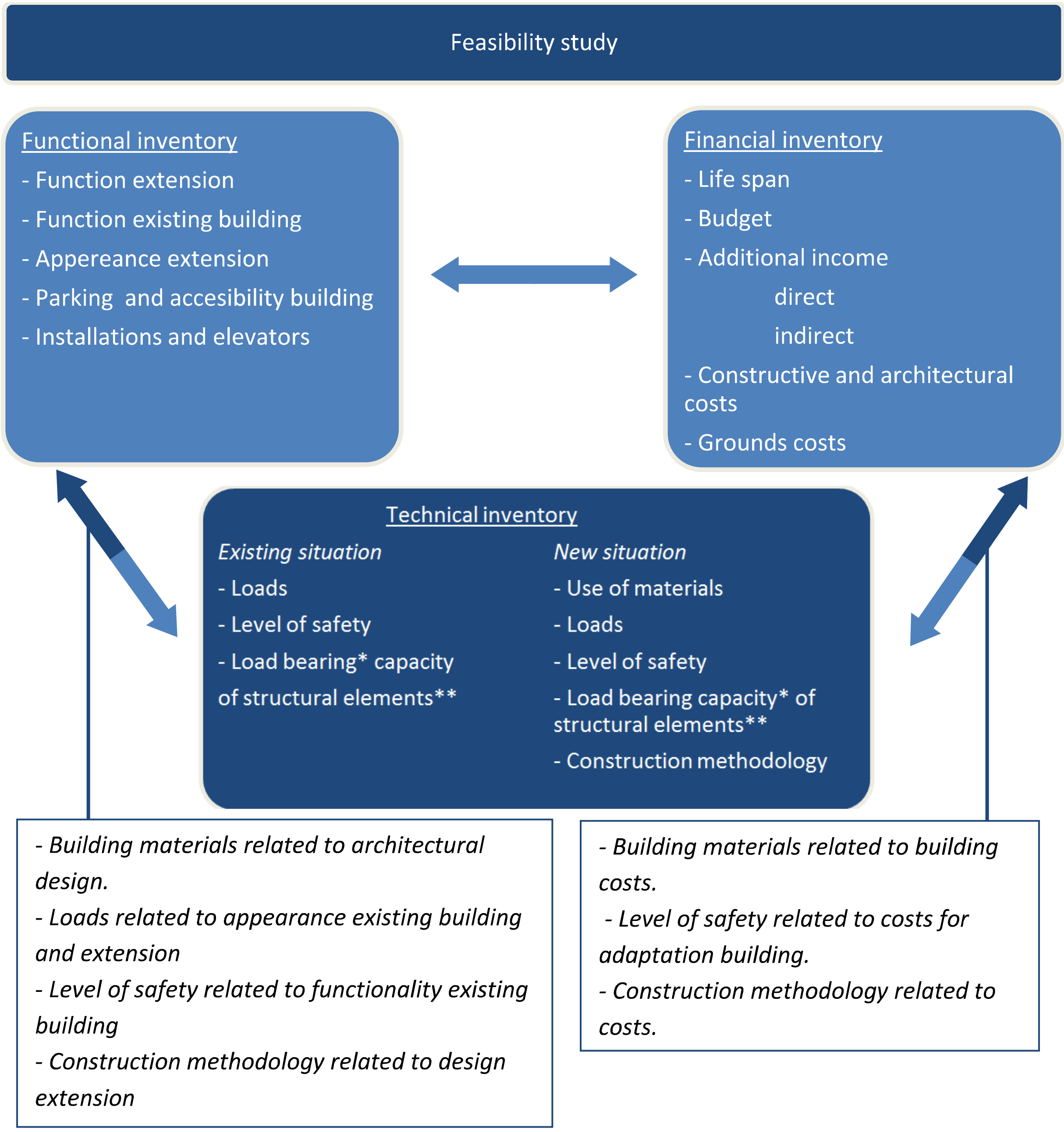
As a starting point for the calculations on the Oval Tower case study the existing situation was described, the available calculations and models, dimensions of the building, materials and loads were determined. Using this data calculations were made for the existing Oval Tower structure. It was shown that the core and the columns subjected to wind loads are the leading structural elements. Next to that it turned out that various other elements contain overcapacity.

Using the findings on the present overcapacity in the Oval Tower structure and the conclusions on the functional design, three alternatives for the extension were derived. The design alternative that posed minimal structural adaptations was eventually chosen to be most suitable. For this design alternative three construction methods were compared with the help of a trade off matrix. Investigating the three construction methods through the trade off matrix led to a choice on the most appropriate method. A detailed plan for this construction method was composed.

Combining all research findings as described before, several final conclusions and recommendations can be drawn. First, the main research question of this master’s thesis will be recalled: Which factors determine the feasibility of an extension on top of a high-rise building? This research showed that the feasibility of a high-rise extension project is determined by a set of technical- financial- and functional factors. Particularly the group of technical factors was thoroughly analysed, thereby clearly defining the technical inventory. Furthermore it is concluded that a strong relation between the various inventories is present. When investigating the overall feasibility of a high-rise extension project, the three inventories therefore always have to be considered in a combined fashion.

Recommendations were made on the way the diagram has to be completed. After having analysed the technical inventory in detail, it is recommended to further analyse the functional and financial inventories and their corresponding factors in detail. Subsequently the relationship, interaction and possible cohesion between the functional and financial inventory have to be further determined.

Considering the Oval Tower extension case study it can be concluded that it is structurally possible to extend the tower with two additional floors as well as a crown structure on top. However the research also showed that structural strengthening of several building elements is necessary. During the case study the diagram was successfully applied, making several assumptions on the functional and financial inventory. These could be successfully linked to the technical inventory, but as stated before additional research on these inventories is needed.



* the load bearing capacity of the structure has to be investigated for strength, stability and stiffness

** the most important structural elements are the foundation piles, foundation beams, vertical load bearing elements, stabilizing elements, roof floor, structure extension, connections

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1 Introduction

1.1 Oval Tower

The executed literature study provides an introduction to the Oval Tower and its expansion. Because of the importance of the project throughout this elaboration phase a short introduction will be provided again. The literature study showed the Oval Tower was going to be renovated and during the composition of this master thesis the renovation as described in there was elaborated.

First a brief history on the Oval Tower will be provided. Afterwards the current renovation will be explained, to conclude with the boundary conditions which are going to be taken into account in this elaboration phase.

In 2001 in the south east of Amsterdam the Oval Tower was delivered. The building has a height of about 95m from ground level to the roof level, the floor area of the building is 28.500 m^2, and is divided over 24 floors. The owner of the building is Deka Immobilien Investment GmbH and the tenant of the building was the ABN AMRO Bank.[1]

figure 1.1 Oval Tower, current situation

In 2001 the ABN AMRO Bank had signed an agreement to rent the Oval Tower for 10 years which came to an end during 2011, resulting in vacancy of the tower which was a direct threat for the building owner. The building owner therefore decided to modernise the building and make it ready for a more modern way of working. The main entrance is replaced to the side of the building where the ArenA Boulevard is situated. This part of the renovation is performed by demolishing three floors in the lowest part of the building and creating a more open entrance. Sustainability is an important aspect for the current renovation. Various building installations are added and replaced, therefore the use in energy in the tower is expected to be reduced by 40%. All these changes to the building resulted in a LEED certification to be granted together with an A^{+++}-label.[2,3]

The building was renovated in phases. Because of this phasing the remaining floors could stay operational. Noisy activities often have been performed while the building was outside of office hours (evenings and Saturdays).

[1] (BBN_Adviseurs)
[2] (DTZ_Zadelhoff, 2011)
[3] (Stedenbouw, 2012)

The current renovation can be seen as a success. The degree of occupation at time of writing is about 80%, with negotiations on-going for other floors it is expected the degree of occupation will be 100% by the end of 2012.[4]

figure 1.2 Renovation Oval Tower in 2012, entrance (left) after conctruction (right)

The elaboration phase of this master thesis takes the situation of the Oval Tower into account prior to the renovation. Reason setting this boundary is that otherwise the work within this thesis may also be subjected to continuous change which is not desirable.

However for the design of the extension, the initial design has been taken into account which is originated out of a design competition held in 2010 and won by the Dutch architectural company OPL Architects. In this design the relocation of the entrance and the change in climate installations as described before were designed. In an early design phase a crown on top of the building was situated which had both an architectural and an ecological function. Together with the tenant, the developer made the choice for the relocation of the entrance and the upgrade to a more environmentally friendly building. The initially designed crown on top of the building eventually has not been build.[5]

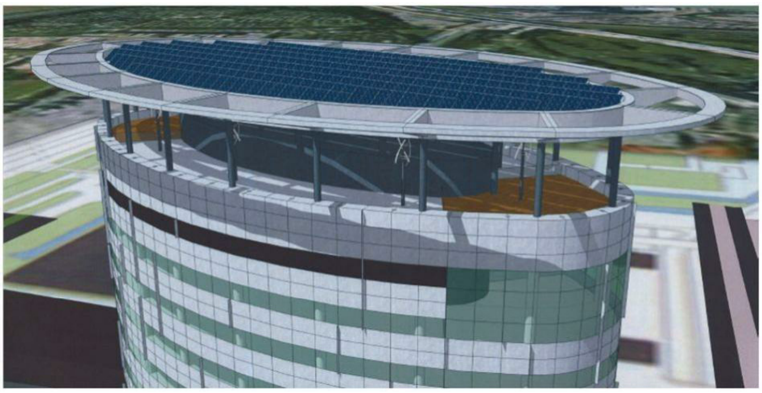


figure 1.3 Oval Tower, impression crown, design OPL Architects

It can be concluded the current renovation of the Oval Tower is a success. However it is still interesting to see whether the extension on top of the building was a feasible idea. As a result this master thesis provides this feasibility study as discussed in following sections.

[4] (Vastgoedmarkt, 2012)
[5] (OPL_Architecten)

1.2 Summary Literature study

This section provides the conclusions and recommendations as found in the literature study. In the introduction of the literature study the main research question has been determined, this question is given below:

- *Which factors determine the feasibility of an extension on top of a high-rise building?*

The diagram shows the approach taken within the literature study to answer the above main research question.

- Start of point diagram (H1) → - Reference projects - Existing research - Quick scan (H2 &H3) → - Determing of variables diagram (H4) → - Investigation into variables (H5) → - Final diagram + conclusions investigation variables (H6)

diagram 1.1 approach literature study

A first answer to the main research question has been given with the help of a diagram. The diagram has been made with the investigation into research projects in which the central and shared topic was extended buildings. Furthermore also existing building projects have been investigated. Another investigation which provided input into the diagram is a Quick Scan in which three buildings with a various load bearing structure but with a similar height have been investigated. The diagram developed as a result of the literature (see chapter 4 within the study) can be found below.

Feasibility study

Functional inventory	Technical inventory	Financial inventory
- Function extension - Function existing building - Appereance extension - Parking and accesibility building - Installations and elevators	- Use of materials - Loads old situation new situation - Level of safety old situation new situation - Load bearing capacity* of structural elements** old situation new situation - Construction methodology	- Life span - Budget - Additional income direct indirect - Constructive and architectural costs - Grounds costs

* the load bearing capacity of the structure has to be investigated for strength, stability and stiffness

** the most important structural elements are the foundation piles, foundation beams, vertical load bearing elements, stabilizing elements, roof floor, structure extension, connections

diagram 1.2 design diagram after H4 literature study

After the development of the first diagram, the next step upon which has been elaborated is the investigation of the various variables of the technical inventory. Below an overview of the most important conclusions can be found.

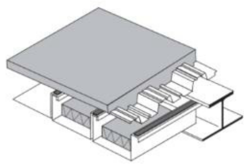
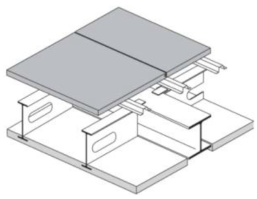
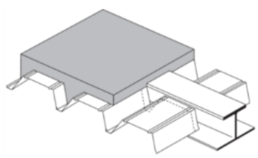
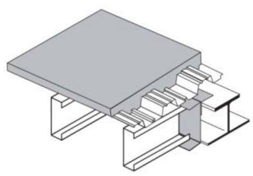
1.2.1 Use of materials

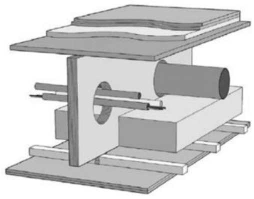
- Steel and timber are the most favorable materials for making a construction on top of an existing building.
- The choice for a load bearing system depends on the architectural design of the extension. A table which can help with making a choice for the most suitable load bearing structure is given below.

	Steel framing	Light steel framing	Timber framing
Self-weight	+	++	+
Flexibility	+	0	0
Use of glass façade elements	+	0	0

table 1.1 Advantages and disadvantages material main load bearing structure extension

- The choice for a floor system depends on the architectural design of the extension. With help of the table below in an initial design phase a choice can be made for a floor system.

Floor type	Description	Impression	Properties*	Field of application
IDES floor	Girders in one direction, cold formed steel open profiles in other direction, finishing with anhydrite or triplex		80 – 180 kg/m2; 200 – 340 mm; 5,0 – 7,0 m; 60 – 120 min	Buildings in which a thin and very light floor is desired
Slimline floor	Prefabricated concrete lower flange in which steel profiles are poured, openings in secondary girders,		250 – 320 kg/m2; 300 – 500 mm; 5,5 – 11,0 m; 120 min	Buildings in which integration of ducts is desired
Composite floor deck	Floor exists out of steel plates which constructively work together with the concrete layer. Steel profiles can be stacked during crane movements		260 kg/m2; 330 mm; 4,5 – 9,0m; 60 minutes**	Buildings with a non-uniform floor plan. Buildings in which a high building speed is desired
Light steel framing floor	Secondary girders are cold formed C-profiles, bottom layer will be finished with gypsum board, top layer with an anhydrite layer.		130 – 160 kg/m2; 260 – 400 mm; 3,6 – 7,2 m; 60 minutes**	Buildings in which a thin and very light floor is desired

Flexfloor	The flexfloor is a floor in which plates are glued to vertical girders. In the girders openings are adopted in which ducts can be placed.		150 kg/m2; 385 mm; 7,0 – 8,0 m; 30 minutes**	Buildings with a timber load bearing structure

* an estimation is made for floors on which a live load of 2,5 kN/m2 is working, the following properties are given: mass; thickness; span; fire safety level
** the fire safety capacity can be adjusted to 120 minutes or more with adjustments

table 1.2 comparison floor systems

1.2.2 Loads

- The main conclusions which can be drawn from the investigations with regard to the building standards, is that in most cases there will be no difference between the Eurocode and the NEN6702. When there is a difference thrust according to the Eurocode will be slightly higher.
- Furthermore it can be concluded that the shape of the extension impacts the wind loads which must be taken into account. Opportunities to adjust the shape of the extension are given. A sculptured building top, varying the shape of the building, openings in the top of the building, corner modification and orientation of the building in relation to the leading wind direction are given. These ideas can be used for the architectural design of the extension.

1.2.3 Level of safety

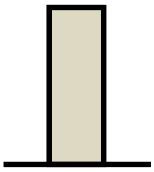
Main conclusions about building standards:

- The Bouwbesluit 2003 for new buildings as well as existing buildings refers to NEN6700 and NEN6702.
- According to the Bouwbesluit 2012 a division can be made in two main group of buildings: new buildings and existing buildings.
- For new buildings NEN-EN1990 and NEN-EN1991 are applicable. For existing buildings NEN8700 has to be applied.
- For existing buildings the following systematic judgment is recommended: For an existing building the level of safety for new buildings has to be taken into account, unless the costs to fulfill the requirements of new buildings are disproportionately high. In this situation the safety level for existing buildings are sufficient.

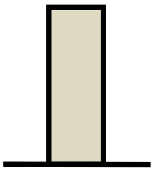
Conclusions from comparison standards:

- The load factors for new buildings according to NEN-EN1990 always have a bigger value in comparison to the values given in NEN6700.
- The load factors for rebuilding’s according to NEN8700 always have a bigger value in comparison to NEN6700 unless the building may be taken into account with help of the values determined in NEN6700 or before.
- The way the load factors have to be applied are not unambiguous and are sub ordinary to the costs which have to be made to let the building fulfill the requirements of new buildings.

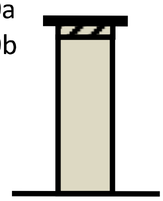
- Situation I: Interpret total building (including extension) as rebuilding's.

Interpret as		Dead load Disadvantageous	Dead load Advantageous	Leading variable load	Remaining variable load
Rebuilding	6.10a 6.10b 	1,4 1,25	0,9 0,9	1,6 ψ_0 1,6	1,5 ψ_0 1,5 ψ_0
Rebuilding (values according to NEN6700)		1,2	0,9	1,5	1,5 ψ_0

- Situation II: Interpret extension as new building, interpret existing building as rebuilding.

Interpret as		Dead load Disadvantageous	Dead load Advantageous	Leading variable load	Remaining variable load
New building	6.10a 6.10b 	1,5 1,3	0,9 0,9	1,65 ψ_0 1,65	1,65 ψ_0 1,65 ψ_0
Rebuilding (values according to NEN6700)		1,2	0,9	1,5	1,5 ψ_0

- Situation III: Interpret total building (including extension) as new building.

Interpret as		Dead load Disadvantageous	Dead load Advantageous	Leading variable load	Remaining variable load
New building	6.10a 6.10b 	1,5 1,3	0,9 0,9	1,65 ψ_0 1,65	1,65 ψ_0 1,65 ψ_0

For an existing high-rise project, the building can be interpreted in various ways. Above three situations are given in which way an existing high-rise project, on which an extension is going to be build, can be interpreted.

The situation which has to be taken into account depends on the overcapacity in the building and the costs which have to be made to satisfy the requirements for new buildings. Situation III is the desired level, situation I is the absolute minimum which would have to be applied.

1.2.4 Load bearing capacity of structural elements

In the table below three methods have been described which can be used to strengthen an existing building. Additional elements are often placed near the existing structural elements. The reason for placing the additional structure near the existing ones is that no decrease of floor space will occur. When additional elements are placed into open floor space, this has been done for reasons of stability. Another method is the use of external glued reinforcement around an existing structure. The table below provides an overview of the three methods.

	Additional elements inside building	Adding elements around existing structure	Glued reinforcement
Stability building	++	0	-
Stiffness building	++	+	+
Strength building	++	+	+
Losses in floor area	-	+	++
Dimension elements	-	+	++
Transport elements	-	0	+
Costs	+	+	-
Erection speed	0	-	+

table 1.3 Comparison methods to strengthen existing building elements

The table shows that all three methods are suitable for strengthening of the main load bearing structure. Depending on the design of the existing load bearing structure and the new extension a decision can be made which method can be applied best.

1.2.5 Construction methodology

An extension can be built with help of various construction methodologies. To provide an answer to the question which building methodology is most suitable a procedure has been provided wherein building methods can be compared.

With help of reference projects various requirements have been determined. With help of this combined list of requirements the building methodologies can be compared. The requirements are admitted in the first design of the trade off matrix which is given in diagram 1.3.

A division is made between standard and additional criteria with regard to the extension. The requirements can be rated for the various building alternatives. The items can be put in three categories: poor (-), average (0) and good (+). Together with the criteria a rating can be made, providing a quick but thorough decision within the design phase for a particular building method.

In case an extension will be situated on top of an existing high-rise project the trade-off matrix will look like the one below. According to the boundary conditions and local conditions complementary requirements can be added to the diagram.

Construction Methodology	I, conventional building method	II, crane on top of the building	III, lifting extension as one piece
Standard criteria construction methodology			
Access to site			
Storage at site			
Maneuvering space at site			
Subsoil capacity at site			
Lifting operation, sensitivity to wind			
Tolerances (ability to absorb)			
Environmental impact: noise/transport			
Additional criteria construction methodology with regard to extension			
Operability building during construction			
Nuisance on ground floor to users surrounding buildings			
Construction time / costs / risks			
Construction time			
Costs			
Accuracy cost estimate			
Risks			
Final choice			

diagram 1.3 Trade off matrix alternative building methods

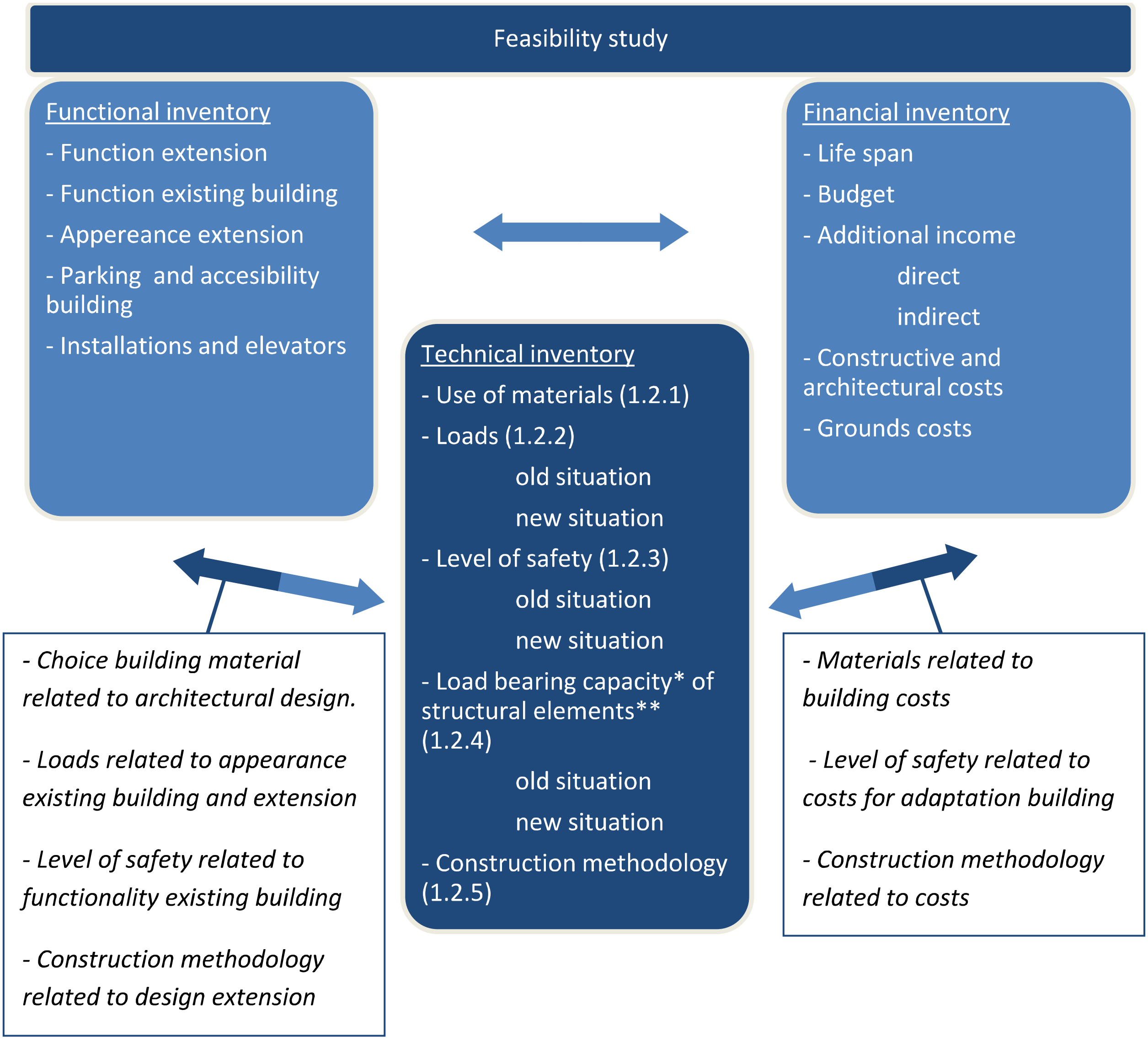
When the location, the existing building and further extension requirements are known, numerical amounts can be determined and the matrix can be filled in. In the elaboration phase the requirements can be further specified according to the design. Numerical values can be added to the matrix. The methods can then be compared and the most suitable building method can be chosen for the elaboration of the project.

1.2.6 Final diagram after literature study

The final diagram which can be used in case a technical inventory is going to be made for an extension on top of an existing high-rise project can be seen in the diagram below.

In the diagram the variables which are required for the technical inventory are shown. In the various chapters (which are admitted between brackets), conclusions and findings are provided.

Furthermore various conclusions are drawn within the investigation into the variables with regard to the relation between the various inventories. Several variables in the technical inventory cannot be considered without taking the variables in the adjacent inventories into account. The variables in the technical inventory which are highly related to the other two inventories are adopted in the diagram below.



* the load bearing capacity of the structure has to be investigated for strength, stability and stiffness

** the most important structural elements are the foundation piles, foundation beams, vertical load bearing elements, stabilizing elements, roof floor, structure extension, connections

diagram 1.4 final design diagram following the literature study

When an extension is to be built on top of an existing building the scheme which can be seen in

diagram 1.4 should be used to understand the complexities and requirements. From the variables which are shown in the diagram various conclusions and findings have been made can be used as a start point in case an extension on top of a high-rise project is going to be investigated. The technical inventory has been researched in detail, in this investigation it becomes clear various variables from both inventories are highly related and are shown in the diagram.

1.2.7 Recommendations with regard to working method

From investigations made in the literature study, recommendations have been made about the procedure which can be followed when making calculations to an extended high-rise project. These recommendations can be seen below:

- An elaborated structural investigation of the existing structure in the beginning of the process helps in determining the structural attractiveness of the extension.
- Where possible, make use of the overcapacity which is present in the existing load bearing elements.
- Where possible, adapt the structural design of the extension in a way the reaction forces will be distributed over the elements which contain overcapacity.

The above recommendations are advised to be part of the calculations required for any high-rise project to allow for this extension to be built efficiently.

1.3 Research objective

In the previous sections an introduction to the Oval Tower and a summary of the literature study are given. In the literature study a design has been made for a diagram which can be used in case an existing high-rise project is going to be extended.

The research objective of this elaboration phase is to use the diagram for the case study. The project which will be elaborated upon during the case study is the Oval Tower. After the technical inventory of the Oval Tower has been made, conclusions can be made with how the diagram is usable. Furthermore the diagram possibly has to be adopted.

With help of the elaboration phase an answer will be provided to whether it is feasible from a structural perspective to extend the Oval Tower with two floors and a crown.

1.4 Research question

The main research question of this master thesis is given below:

- *Which factors determine the feasibility of an extension on top of a high-rise building?*

Answer to this question has partly been provided in the literature study. A diagram has been made in which the variables for the technical inventory are adopted. Conclusions and findings with regards to these variables have been shown and recommendations have been made with regard to the working method.

In the elaboration phase it is intended to use the diagram for a case study. After the case study has been made, a review will follow and the usability of the diagram will be evaluated.

After the case study has been elaborated upon, an answer will be given to the following secondary research question:

- *Is it structurally feasible to extend the Oval Tower with two floors and a crown?*

The literature study shows that an investigation into the structural behaviour of the existing building is of great importance. Therefore this will be the first step in the elaboration phase which will also assist in answering the questions below, which are secondary research questions:

- *What is/are the most leading structural aspect(s) in the current design of the Oval Tower?*
- *Where does the current Oval Tower contain overcapacity?*

After this first phase and answers have been provided to the questions above, various alternatives for the extension will be elaborated upon. One alternative will be chosen and will be further investigated. The construction methodology will also be taken into account. Research questions which belong to this particular phase are the ones below:

- *Does the current building has to be strengthened because of the extension?*
- *Which construction method can best be used to extend the Oval Tower?*
- *Is it structurally possible to extend the Oval Tower with two floors and a crown?*

Answer to these secondary research questions can help in evaluating the diagram which has been designed in the literature study.

1.5 Approach

With help of the diagram and the recommendations with regard to the working method an approach, which is going to be followed in this case study, has been determined. The following recommendations have been made in the literature study:

- An elaborated structural investigation of the existing structure in the beginning of the process helps in determining the structural attractiveness of the extension.

Because of this recommendation and because of the distinction which has been made in the diagram, the existing situation and new situation are separated. Below this new approach is given.

<u>Technical inventory</u>

Existing situation	*New situation*
- Loads (1.2.2)	- Use of materials (1.2.1)
- Level of safety (1.2.3)	- Loads (1.2.2)
- Load bearing* capacity of structural elements** (1.2.4)	- Level of safety (1.2.3)
	- Load bearing capacity* of structural elements** (1.2.4)
	- Construction methodology (1.2.5)

* the load bearing capacity of the structure has to be investigated for strength, stability and stiffness

** the most important structural elements are the foundation piles, foundation beams, vertical load bearing elements, stabilizing elements, roof floor, structure extension, connections

The separation between the existing and new situation has led to the division in chapters as given in the table below.

Overview elaboration phase	Situation	Chapter
Starting point calculations existing situation	Existing	2
Calculations existing situation		3
Conclusions existing situation		4
Starting point design extension	New	5
Three alternatives extension		6
Structural design final alternative		7
Construction methodology		8

table 1.4 plan elaboration phase

2 Starting point calculations existing situation

2.1 Available calculations and models

For the calculations which are going to be made in the elaboration phase of this master thesis several existing documents will be used. Below an overview of these documents. Behind the documents abbreviations are admitted, in this report reference will be made to this abbreviations.

Literature study [LS]
As described in the introduction, preceding to this master thesis a literature study has been made. In this literature study a diagram is made which can be used when a structural design on top of an existing high-rise building will be made. This diagram will be used as a starting point for the calculations which will be elaborated in this thesis.

Weight and stability calculations Oval Tower [WSC OT]
During the first phases of the Oval Tower, Zonneveld Ingenieurs is involved in the structural design of the tower. In the initial phase of the Oval Tower a weight and stability calculation is made for the Oval Tower. In this calculations the main dimensions of the building are determined. The weight and stability calculations of the Oval Tower are approved in 1999 after which the Oval Tower is constructed. Parts of the calculations are admitted in this master thesis and can be recognized as the hand written parts.

Structural drawings Oval Tower [SD]
Besides the weight and stability calculations also structural drawings are made of the Oval Tower. In the structural drawings several building properties are enlisted. Furthermore on these drawings the load bearing capacity of the foundation is given. The amount of structural drawings is big, the most important ones used in this thesis are admitted in appendix B.

Report TNO dynamical calculations [TNO]
When the Oval Tower was in use nuisance because of dynamical behavior was encountered by the users of the building. Several measurements were done by TNO. These measurements were discussed by TNO and Zonneveld Ingenieurs. From these meetings reports are made. The reports are admitted in appendix C.

ESA Prima Win model [ESA PW]
For the dynamical calculations as described above the Oval Tower is modelled in a calculation program called ESA Prima Win. The model was used to determine the dynamical behaviour of the tower. In this master thesis this model is used as a start of point and is further expanded.

Booklet OPL Architects [OPL]
A booklet in which drawings of the existing situation and drawings of the renovation are admitted is made by OPL Architects. As described in the introduction, the designed crown of OPL Architects eventually has not been build. However this crown will be a starting point for the architectural design in this master thesis. Several drawings out of this booklet are admitted in appendix D.

2.2 Dimensions

To get an idea of the shape and the dimensions of the Oval Tower in this subchapter the most important aspects with regard to the structural system and the dimension of the tower are given. In appendix B several drawings of the existing situation are admitted.

The table below provides an overview of the arrangement in floors of the Oval Tower. The main load bearing structure of the tower exists out of a concrete core and concrete columns. The façade is attached to these columns. In the top floor of the building the concrete core is connected to the columns with help of an outrigger system. Furthermore in the bottom floor of the building, abutments are present. The abutments connect the columns to the core on the lower ground floor. Above ground level the building has a total of 26 floors which are divided as can be seen in table 2.1. Furthermore in Appendix B it can be seen a small extension in which technical functions are present, is built on top of the Oval Tower, the total height therefore becomes 98,5m.

	Height	Function
Ground floor	3,6m	Entrance
24 typical floors	24 x 3,6m = 86,4m	Offices
Top floor	5,2m	Structural + technical
Technical extension	3,3m	Technical
Total building height	98,5m	

table 2.1 arrangement floors oval tower

A typical floor plan with the most important dimensions in which the core and the columns are drawn can be seen in the figure below. Furthermore the two leading directions which can be distinguished are given. The letters in the figure below are the axes on which the columns are situated. These axes will be used and will refer to the same location throughout the entire report.

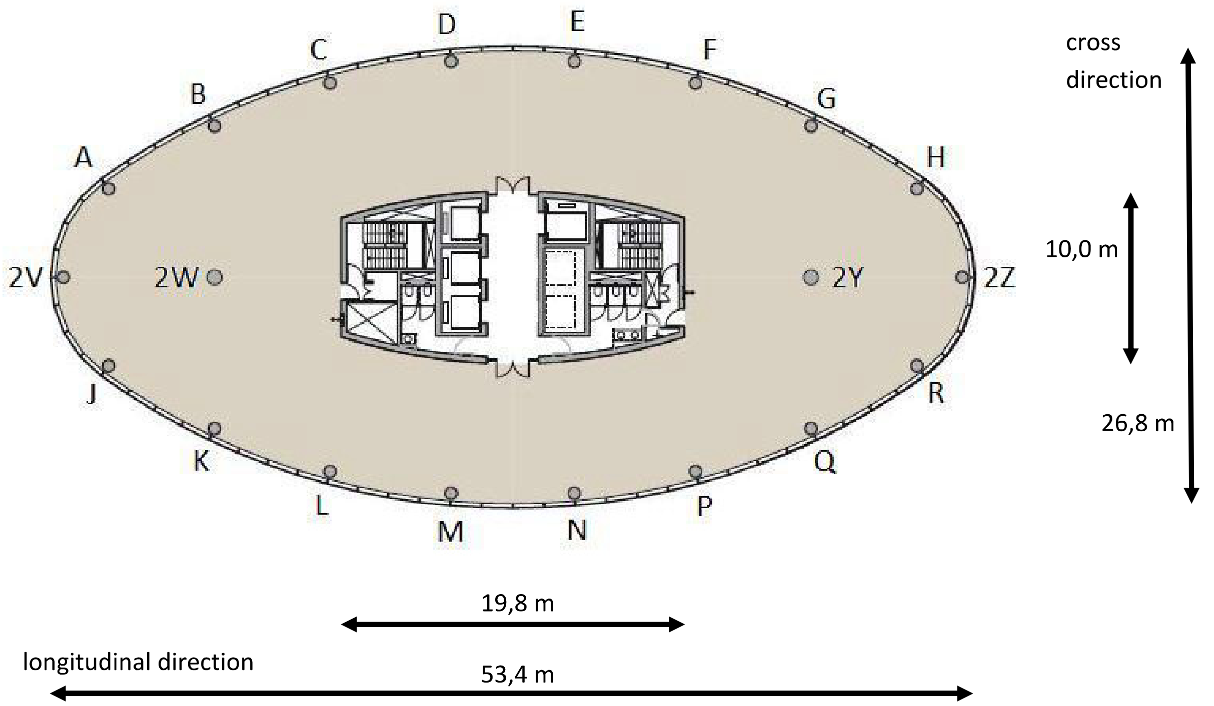


figure 2.1 dimensions cross section Oval Tower

2.3 Materials

In the Oval Tower several building materials are used. The two main materials are concrete and steel for the upper outrigger floors. In the table below an overview of concrete and steel which is used for the various elements is given. The materials follow out of [SD], which is admitted in Appendix B.

Element	Material	Concrete class / steel quality (notation in [SD])	Concrete class / steel quality (current notation)
Core	Concrete	B45	C35/45
Columns (both prefab and in situ)	Concrete	B65	C53/65
Abutments	Concrete	B45	C35/45
Floors outside core (polystyrene plate floor)	Concrete	B35	C28/35
Strengthened strip outside core (composite plank floor)	Concrete	B35	C28/35
Floors inside core (composite plank floor)	Concrete	B45	C35/45
Structure upper floors (according to profile)	Steel	S235 / S275	S235 / S275
Outriggers	Steel	S355	S355
Foundation beam	Concrete	B45	C35/45

table 2.2 materials elements concrete class / steel quality Oval Tower

2.4 Calculation methods and software

The regulatory requirements which are used for the calculations of the existing Oval tower in [WSC OT] are: NEN6702 and NEN6720. When taking the existing situation into account calculations are made with help of these requirements. In the [LS] it can be read new buildings have to fulfil the requirements of the Bouwbesluit 2012 which refers to the newer Eurocodes. When taking the extension into account calculations will be made with help of the Eurocodes.

In [WSC OT] most of the calculations are made by hand. In a few cases calculations are made with help of a framework program called Technosoft.

In [ESAPW] the Oval Tower is imported in a finite element program called ESA Prima Win. For the elaboration of the extension on top of the Oval Tower the basis of the ESA Prima Win model will be used.

Because [ESAPW] is made for determining the dynamical behaviour of the Oval Tower, first it has to be checked whether the model is in consensus with [WSC OT]. This will be done in chapter 3.1.

Currently a common structural calculation program which is present at Zonneveld Ingenieurs is SCIA engineer. SCIA engineer is a successor of ESA prima win. The structural behaviour of the new extension will be made with help of SCIA engineer. Therefore the [ESAPW] model first has to be converted to a SCIA engineer model.

2.5 Overview loads

As described in chapter 2.1 the Oval Tower is constructed according to the results of the [WSC OT]. In this master thesis it is intended to take the structural design into account with help of a finite element model. The [ESAPW] model is made to take the dynamical behaviour of the Oval Tower into account. This means not all the loads determined in the [WSC OT] are taken into account in this model. To continue working with the [ESAPW] first the behaviour of the model has to be the same as calculated in the [WSCOT]. In chapter 3.1 a comparison between the two documents is made and the [ESAPW] is adopted where needed.

To make this comparison first the loads, described in both models, have to be taken into account. For an overview of the loads a distinction will be made throughout vertical loads and horizontal loads. Furthermore the load combinations which will be taken into account will be determined.

2.5.1 Vertical loads

According to [WSC OT] several loads are taken into account. A distinction is made into two various load cases. Permanent load (G) and live load (Q) are taken into account. In the table below the permanent load and the live load are admitted. The table is a summary of the load cases determined in [WSC OT].

	Permanent load (G)	Live load (Q)
Typical intermediate floor		4,0 (kN/m2); ψ=0,5
polyplate floor (d=170mm) + compression slab + ceiling	4,37 (kN/m2)	
Concrete Floor (d=450mm) + ceiling	11,3 (kN/m2)	
Core Floor (d=250mm) + finishing layer + tubes	7,5 (kN/m2)	
Stairs + altars	7,5 (kN/m2)	
Basement floor		5,0 (kN/m2); ψ=1,0
Concrete Floor (d=400mm) + finishing layer	10,6 (kN/m2)	
Ground floor		4,0 (kN/m2); ψ=0,5
Concrete floor (d=280mm) + finishing layer + tubes	9,2 (kN/m2)	
25th floor		7,0 (kN/m2); ψ=1,0
Steel plate concrete floor (d= 320mm) + outrigger structure + ceiling	8,6 (kN/m2)	
Roof floor		1,0 (kN/m2); ψ=0
Steel plate concrete floor + finishing layer + gravel	6,1 (kN/m2)	
Remaining significant elements		
Columns ø650	8,3 (kN/m)	
Columns ø790	12,3 (kN/m)	
Columns ø850	14,2 (kN/m)	
Prefabricated concrete façade elements	8,9 (kN/m)	
Façade elements	0,50 (kN/m2)	
Beams core	9,1 (kN/m)	

table 2.3 summary of permanent and live loads according to [WSC OT]

In the [ESAPW] model elements are chosen. The dead load (G) will be determined automatically by the program. The live load (Q) has to be imported by hand. In the figure below an overview of the live loads, which have been taken into account, is given.

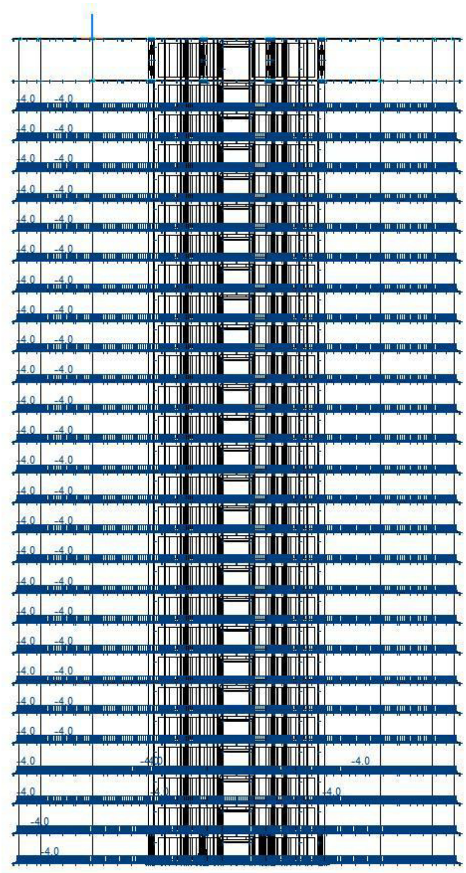


figure 2.2 live load [ESAPW], only typical floor load taken into account

It can be seen that only the live load on the typical floors is taken into account. To get a more detailed impression of the behaviour of the Oval Tower the live loads in the [ESAPW] model have to be adopted.

2.5.2 Horizontal loads

As described in chapter 2.1 two main directions can be distinguished. The length in the cross direction is smaller compared to the longitudinal direction. To decrease the top deflection in the cross direction outriggers are adopted at the top floor and abutments are adopted at the lower floor of the building. Therefore it cannot be directly seen which direction is the leading one and both situations have to be taken into account.

In the [WSC OT] wind out of both directions has been taken into account. Wind in the cross direction is calculated for half the building. Wind out of the longitudinal direction is calculated for the total width of the building. Furthermore in both situations the building is simulated as a single vertical beam.

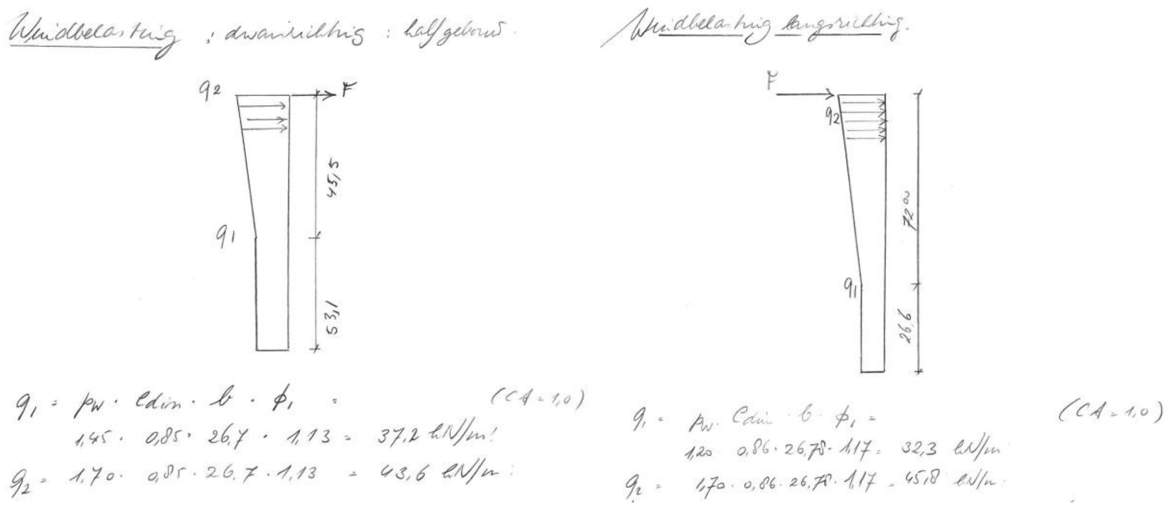


figure 2.3 wind loads according to [WSC OT] cross direction (left), longitudinal direction (right)

The wind load which has been calculated according to the [WSC OT] is taken over the total length of the building 53,1 + 45,5 = 98,6m. 98,6m is the building length including the technical extension. The height of the building without the technical extension is 95,2m, this means additional wind load is taken into account because of the small technical extension on top of the Oval Tower.

It can be concluded that because of the technical extension additional wind load is taken into account which did not had to be taken into account. The area on which additional wind load is taken into account is drawn as a red area in the figure below.

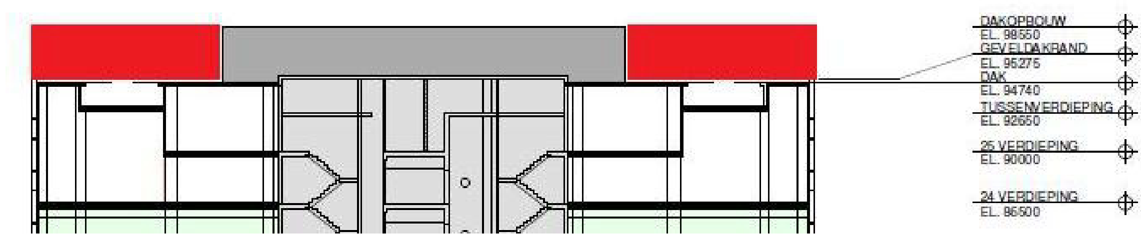


figure 2.4 additional wind load taken into account in [WSC OT] (red area)

The wind load which is taken into account in the [ESAPW] model can be seen in the figure below.

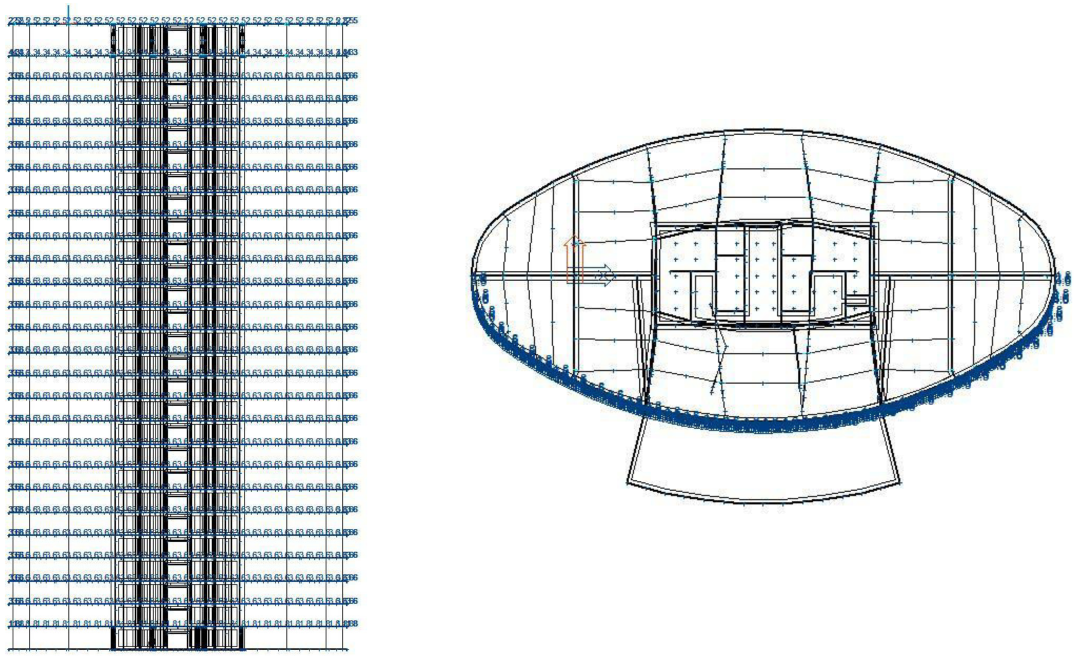


figure 2.5 statically wind load according to [ESAPW], front view (left), top view (right)

What can be seen is that the wind load is taken into account for every single floor with a height of 3,6m. This means the value of the wind which is taken into account is 1,0 kN/m2. This value is a statically wind load (this can be read in appendix C) and is taken into account to determine the dynamical behaviour. When the stresses in the core and the loads in the columns are going to be calculated the wind loads have to be adopted to the actual wind values. This will be done in chapter 3.1.

2.5.3 Load cases and load combinations

The load cases which are taken into account for the calculation of the existing situation are given in the table below.

Load case	
BG1	Self-weight
BG2	Variable floor load
BG3	Wind cross direction
BG4	Wind longitudinal direction

table 2.4 load cases

The load cases have been taken into account in various combinations which are given in table 2.5.

Load Combination	SLS/ULS	BG1	BG2	BG3	BG4
LC1	SLS	1,0	1,0 * ψ	1,0	
LC2	SLS	1,0	1,0 * ψ		1,0
LC3	ULS	1,2	1,5 * ψ	1,5	
LC4	ULS	1,2	1,5 * ψ		1,5
LC5	ULS	0,9		1,5	
LC6	ULS	0,9			1,5

table 2.5 load combinations

3 Calculations existing situation

As described in the [LS] the following has been recommended with regard to the procedure which can be followed when making calculations to an extended high-rise project:

- *An elaborated structural investigation of the existing structure in the beginning of the process helps in determining the structural attractiveness of the extension.*

Therefore in this chapter a start has been made with such a structural investigation of the Oval Tower. The Oval Tower is built according to the [WSC OT]. In these set of calculations mainly hand calculations are made to describe the behaviour of the Oval Tower. Later on the [ESA PW] model is made in order to determine the dynamical behaviour of the building. In this chapter it will be investigated for which purposes both sets of calculations will be used in the further elaboration of this master thesis.

The investigations and the models which are used in this chapter (chapter 3) are all situations of the Oval Tower <u>*without*</u> extension. In chapter 4 conclusions about this current situation will be drawn. In chapter 5 and further the new situation of the building <u>*with*</u> extension will be taken into account.

3.1 Comparison calculations according to [WSC OT] and [ESA PW]

3.1.1 Dead load

As described in chapter 2.5.1 the dead load in the [ESAPW] model will be determined automatically. The dead load in the [WSCOT] is determined by an elaborated set of calculations. In the figure below the reaction forces on the various axis and below the core according to the [ESAPW] model are given.

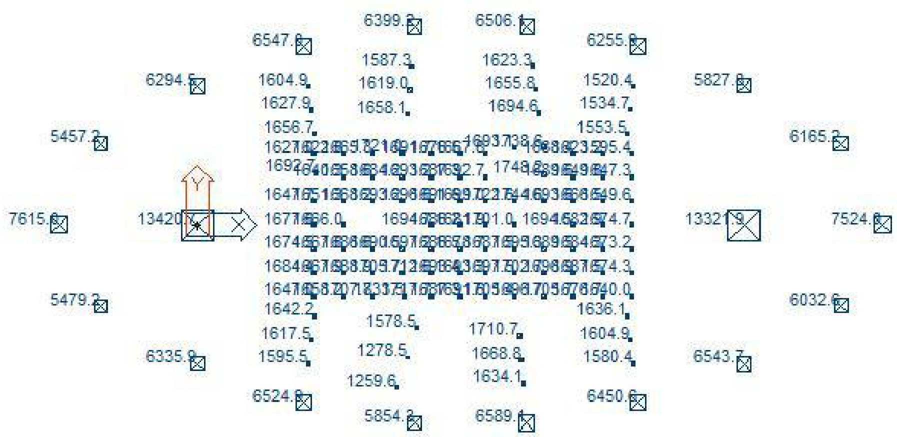


figure 3.1 reaction forces Oval Tower due to dead load according to [ESA PW]

Furthermore the program is able to calculate the resultant reaction forces below the total building or below a part of the building. The resultant forces have been calculated for the total building and for the left and right side of the core.

Resultante (selectie).
Lineair statisch - extreme van alle combinaties
Groep van knopen:1/5282
Groep van belastinggevallen:1, Eigen gewicht

BG	Rz [kN]
1	313951.18

Centraal punt: 117.315 -0.009 -10.800 m

table 3.1 reaction forces dead load total building

Resultante (selectie).
Lineair statisch - extreme van alle combinaties
Groep van

3751/3754,3758/3760,3764,3766/3767,3770/3771,3803,4199/4200,4202/4204
Groep van belastinggevallen:1, Eigen gewicht

BG	Rz [kN]
1	85967.13

Centraal punt: 111.550 0.015 -10.800 m

Resultante (selectie).
Lineair statisch - extreme van alle combinaties
Groep van

3755/3757,3761/3763,3765,3768/3769,3772/3773,4208/4214,4563
Groep van belastinggevallen:1, Eigen gewicht

BG	Rz [kN]
1	86839.05

Centraal punt: 123.118 -0.038 -10.800 m

table 3.2 reaction forces dead load core, left side and right side

The calculations for the dead load in [WSC OT] are summarized in the table below. Furthermore the reaction forces in the figure above are also admitted in this table.

	WSC OT	ESA PW
Column on axis	G (kN)	G (kN)
A	4779	5457
B	6824	6294
C	6984	6547
D	6798	6399
E	6798	6506
F	7062	6255
G	6934	5827
H	4779	6165
J	4800	5479
K	6740	6336
L	6959	6525
M	6895	5854
N	6948	6589
P	7406	6541
Q	6839	6544
R	4832	6033
2V	6891	7615
2W	18431	13421
2Y	18649	13322
2Z	6968	7524
Core		
core left	74630	85967
core right	77257	86839
Total	305203	313951

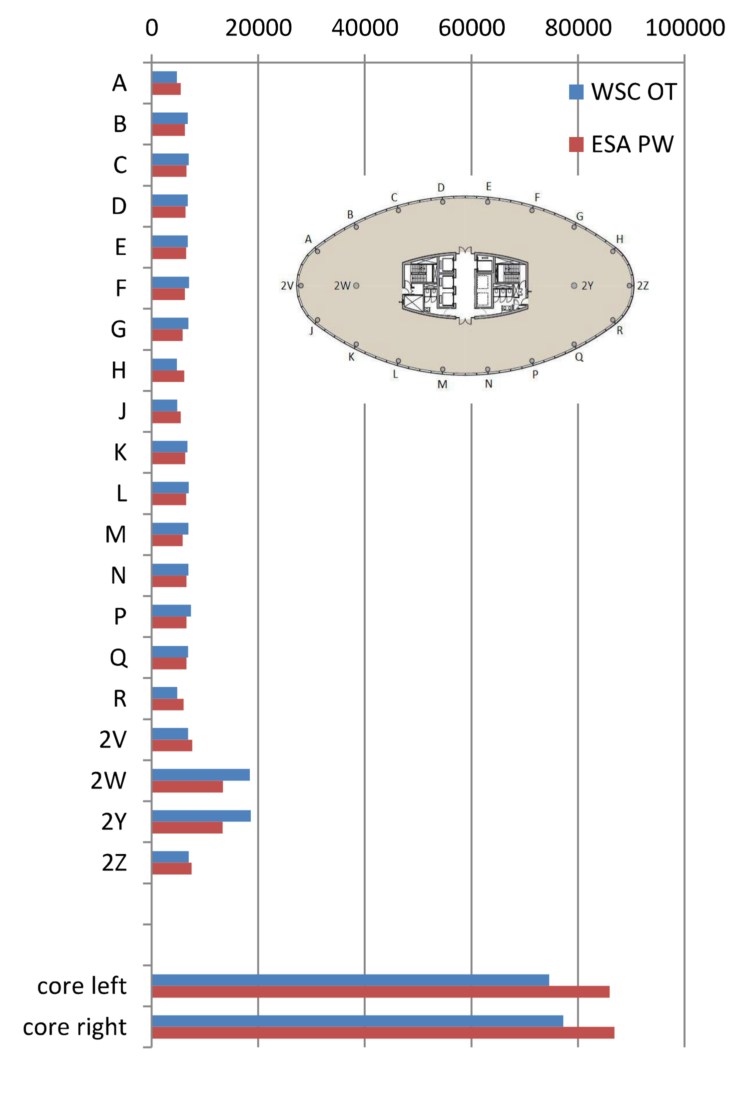


table 3.3 reaction forces dead load according to [WSC OT] and [ESAPW]

What can be learned from the table above is that the force distribution according to both models is slightly different. The biggest differences occur in the columns 2W and 2Y and in the core. Furthermore the difference of the total reaction forces between the two models is 313951/305203 → 2,9%. In the next section this difference will be explained.

3.1.2 Live load

In chapter 2.5.1 it is stated that in the [ESA PW] model only the live floor load on the typical floors is imported. A more detailed input is needed and therefore the remaining floor loads as stated in table 2.3 are also imported in the [ESA PW] model. In the [WSC OT] the reaction forces of the live loads are calculated including the ψ-factor. Therefore for the input in the [ESA PW] model also this ψ-factor will be taken into account. This leads to the distribution of live loads as can be seen in the figure below.

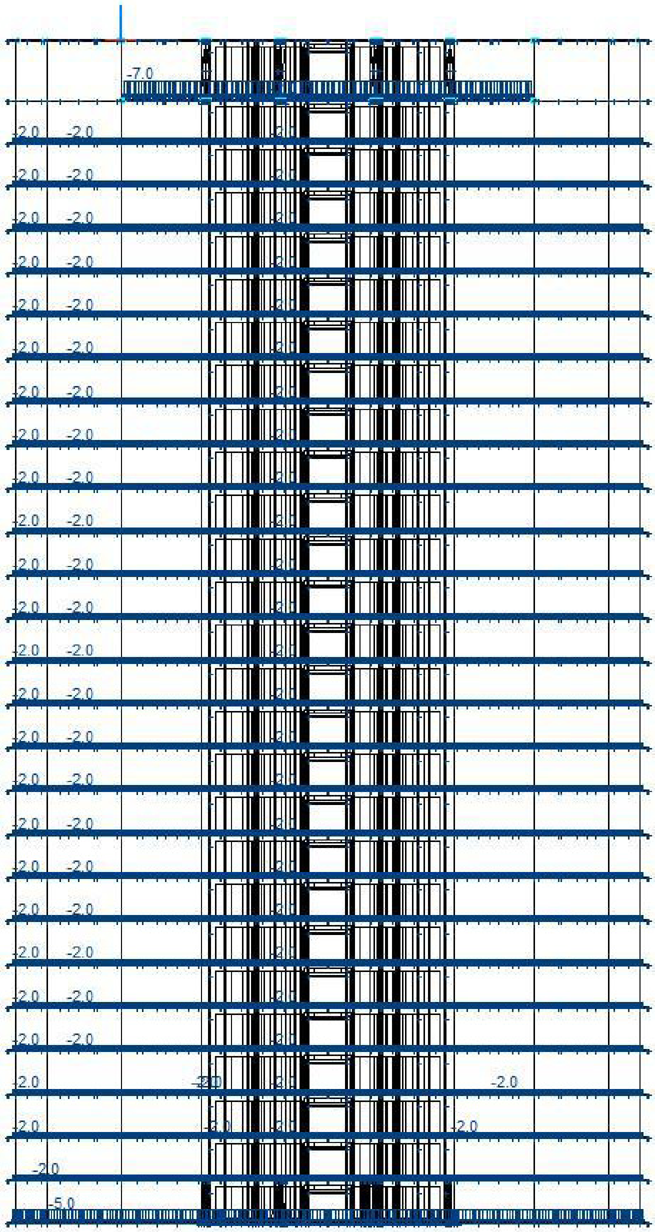


figure 3.2 Variable floor load, including ψ -factor

Now the same procedure can be followed comparable to the procedure made in the last subchapter. First the reaction forces according to the [ESA PW] model will be determined.

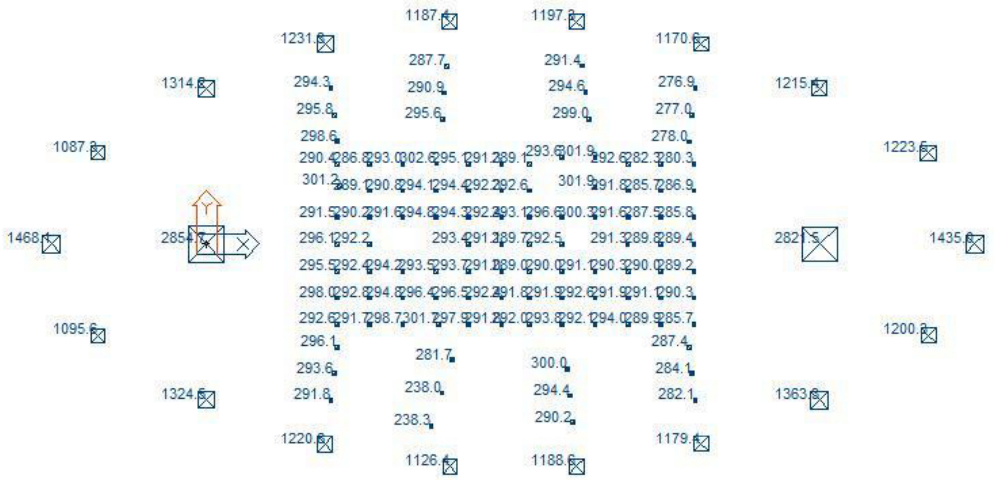


figure 3.3 reaction forces Oval Tower due to live load according to [ESA PW]

Resultante (selectie).
Lineair statisch - extreme van alle combinaties
Groep van
knopen:805/841,920/923,941/958,3593/3773,3803,4199/4200,4202/4204,4208/4214,4563
Groep van belastinggevallen:2, Veranderlijk vloeren

BG	Rz [kN]
2	58159.71

Centraal punt: 117.315 -0.009 -10.800 m

table 3.4 reaction forces live load total building

Resultante (selectie).
Lineair statisch - extreme van alle combinaties
Groep van

3755/3757,3761/3763,3765,3768/3769,3772/3773,4208/4214,4563
Groep van belastinggevallen:2, Veranderlijk vloeren

BG	Rz [kN]
2	15098.02

Centraal punt: 123.118 -0.038 -10.800 m

Resultante (selectie).
Lineair statisch - extreme van alle combinaties
Groep van

3751/3754,3758/3760,3764,3766/3767,3770/3771,3803,4199/4200,4202/4204
Groep van belastinggevallen:2, Veranderlijk vloeren

BG	Rz [kN]
2	15156.51

Centraal punt: 111.550 0.015 -10.800 m

table 3.5 reaction forces live load core, left side and right side

The next step is to summarize the reaction forces which are determined in the [WSC OT]. This values are compared with the values calculated above. In the table below the comparison can be seen.

	WSC OT	ESA PW
Column on axis	Q (kN)	Q (kN)
A	636	1087
B	1397	1314
C	1580	1231
D	1465	1187
E	1465	1197
F	1606	1171
G	1434	1215
H	636	1224
J	657	1096
K	1412	1325
L	1463	1221
M	1414	1126
N	1429	1189
P	1521	1180
Q	2037	1363
R	1227	1200
2V	998	1468
2W	5347	2855
2Y	6235	2822
2Z	1438	1436
Core		
core left	12582	15157
core right	12784	15098
Total	60763	58160

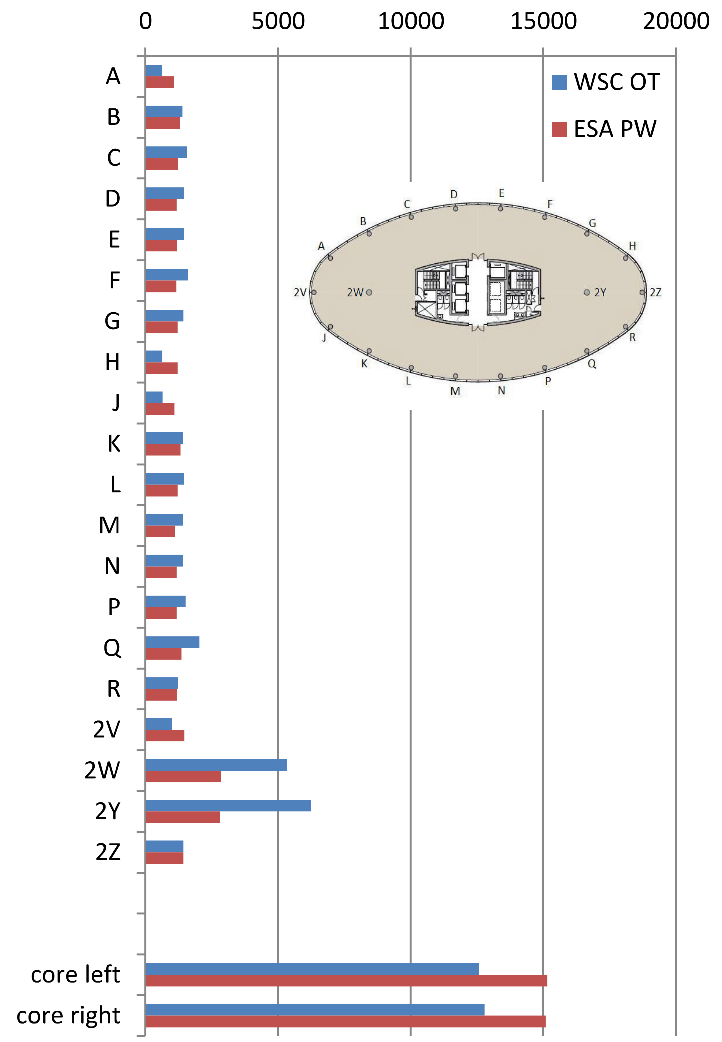


table 3.6 reaction forces live load according to [WSC OT] and [ESA PW]

Again equality exists between the two models. The total difference between the reaction forces is 60763 / 58160 → 4,3%. Again the biggest difference between the two models occurs in the columns 2W, 2Y and the core.

The differences between the distribution of reaction forces because of dead load and live load in the previous sections has two causes. These causes will be explained below.

The first cause is the degree of detail in which the loads are imported. In the [ESA PW] model the dead load will be determined automatically and depends on the elements which are imported. The elements which are imported are restricted to the main load bearing structure of the building (floors, columns, façade elements, steel structure). In the [WSC OT], besides the elements stated before, additional (local) non load bearing elements are imported (for example partition walls, brickwork, local timber floors). The values in the [WSC OT] therefore have a higher accuracy.

The second cause is the way the floors are imported in both models. The current existing floor consists out of a polyplate floor. The plates transfer the forces to an in situ strip which is present between the core and the columns on axis 2V till 2Y on one side and to the supporting facade elements on the other side. The way the floors span is defined with help of the blue arrow in the figure below.

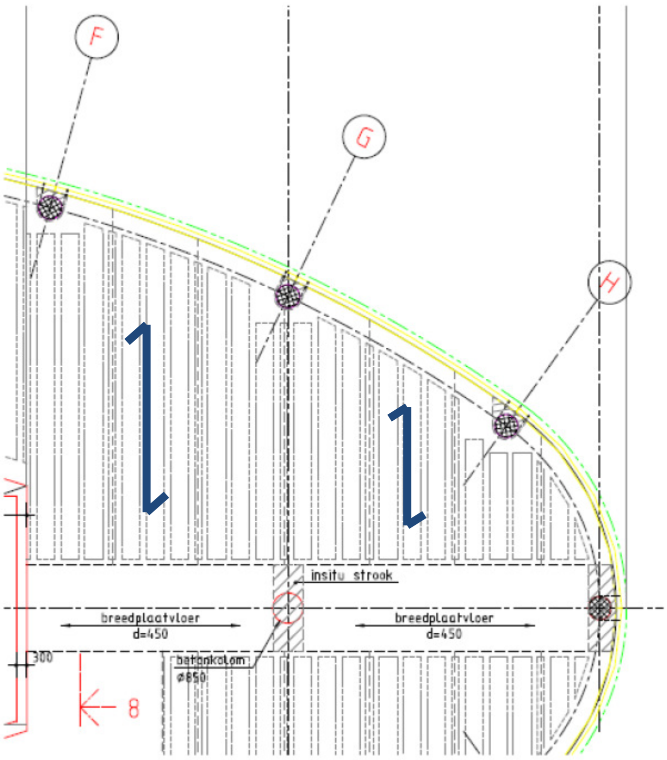


figure 3.4 floor system Oval Tower, polyplate floor combined with an in situ strip

In the [WSC OT] the distribution of forces is adopted to this arrangement. On the other hand in the [ESA PW] model the floors are imported as a single slab. The input of the floor as a slab causes another distribution of forces to the core and columns.

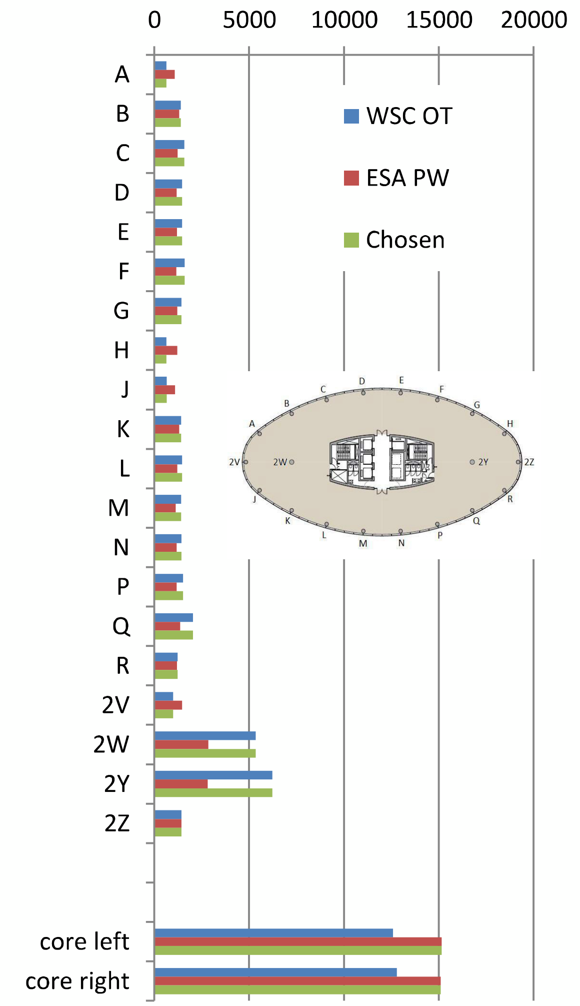
The distribution of forces according to the [WSC OT] is the most accurate one. Therefore the calculations according to the [WSC OT] are more accurate.

According to the [ESA PW] model the loads which are present in the core are higher compared to the values calculated in the [WSC OT]. However the values are too high, it is a safe assumption to further calculate the stresses with help of the values found in the [ESA PW] model.

In conclusion it can be stated for the distribution of forces the [WSC OT] is a more accurate model. Therefore the force distribution according to the [WSC OT] in further calculations will be taken into account. However according to the [ESA PW] model the loads in the core are higher, for the core this higher value will be taken into account.

For clarity reasons the values which have been chosen to work further with for live load are given in the table below.

	WSC OT	ESA PW	Chosen
Column on axis	Q (kN)	Q (kN)	Q (kN)
A	636	1087	636
B	1397	1314	1397
C	1580	1231	1580
D	1465	1187	1465
E	1465	1197	1465
F	1606	1171	1606
G	1434	1215	1434
H	636	1224	636
J	657	1096	657
K	1412	1325	1412
L	1463	1221	1463
M	1414	1126	1414
N	1429	1189	1429
P	1521	1180	1521
Q	2037	1363	2037
R	1227	1200	1227
2V	998	1468	998
2W	5347	2855	5347
2Y	6235	2822	6235
2Z	1438	1436	1438
core left	12582	15157	15157
core right	12784	15098	15098
Total	60763	58162	65652



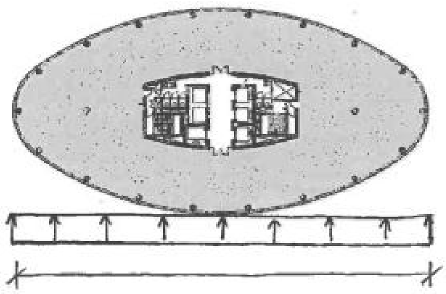
The chosen column values are calculated in the WSC OT, the chosen core values are calculated with help of the [ESA PW] model. The choice for these values is a safe assumption.

3.1.3 Wind load

As described in chapter 2.5.2 in the [ESA PW] model the wind load is imported as a statically wind load. To determine the additional forces in the columns and the stresses which are working in the core because of wind load, first the real wind load has to be adopted in the [ESA PW] model.

In the [WSC OT] the Oval Tower is simulated as a singular beam. The wind load is calculated as a line load which is working on this vertical beam. In the [ESA PW] model the wind load will be imported as a line load which is working on a horizontal floor. Therefore the line load which is working on the singular beam has to be converted to a line load which is working on every single floor.

Furthermore a reduction has to be made because the calculated load is working over the width of the building and not over half the circumference of the building as imported in the model. The reduction factor is given by the building width divided by the length of half the circumference. For the cross direction this factor becomes 53,4 / 63,4 = 0,84. For the longitudinal direction this becomes 26,8 / 63,4 = 0,42. (The half outline of the building is measured with help of [SD] and has a value of 2 x 31,7 =63,4m). The reduction has been made to the wind load which will be imported in the model, the reduction factors are admitted in table 3.7.



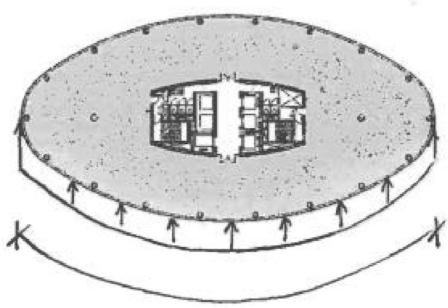
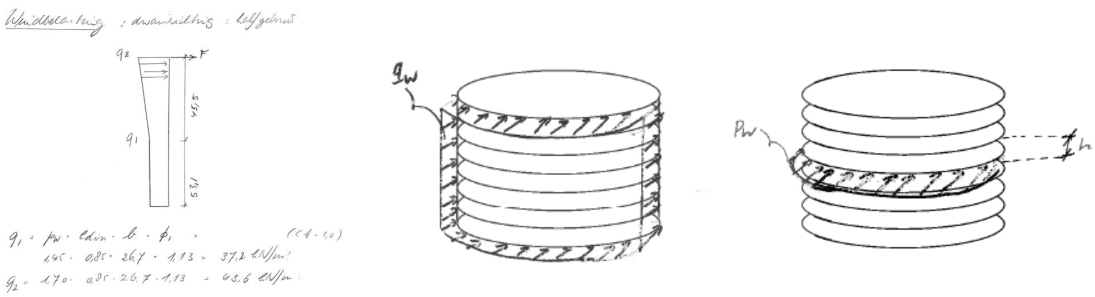


figure 3.5 working wind load (left), imported wind load (right)

In the cross direction the wind load will be converted as can be seen in the table below.

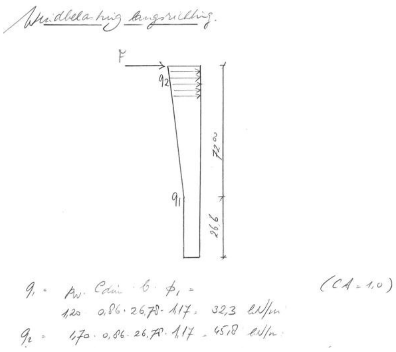


	pw (kN/m2)	cdim (-)	phi1 (-)	qw (kN/m2)
q1 (kN/m2)	1,45	0,85	1,13	1,39
q2 (kN/m2)	1,7	0,85	1,13	1,63

floor	start (m)	h floor (m)	h area floor (m)	qw (kN/m2)	qw add (kN/m2)	pw floor (kN/m)	reduction (-)	pw floor reduced (kN/m)
roof	95,2		2,6	1,64	0,02	4,27	0,84	3,58
25th	90	5,2	4,4	1,63	0,03	7,15	0,84	6,01
24th	86,4	3,6	3,6	1,60	0,02	5,76	0,84	4,84
23rd	82,8	3,6	3,6	1,58	0,02	5,68	0,84	4,77
22nd	79,2	3,6	3,6	1,56	0,02	5,61	0,84	4,71
21st	75,6	3,6	3,6	1,54	0,02	5,53	0,84	4,65
20th	72	3,6	3,6	1,52	0,02	5,46	0,84	4,58
19th	68,4	3,6	3,6	1,49	0,02	5,38	0,84	4,52
18th	64,8	3,6	3,6	1,47	0,02	5,31	0,84	4,46
17th	61,2	3,6	3,6	1,45	0,02	5,23	0,84	4,39
16th	57,6	3,6	3,6	1,43	0,02	5,16	0,84	4,33
15th	54	3,6	3,6	1,41	0,02	5,08	0,84	4,27
14th	50,4	3,6	3,6	1,39		5,00	0,84	4,20
13th	46,8	3,6	3,6	1,39		5,00	0,84	4,20
12th	43,2	3,6	3,6	1,39		5,00	0,84	4,20
11th	39,6	3,6	3,6	1,39		5,00	0,84	4,20
10th	36	3,6	3,6	1,39		5,00	0,84	4,20
9th	32,4	3,6	3,6	1,39		5,00	0,84	4,20
8th	28,8	3,6	3,6	1,39		5,00	0,84	4,20
7th	25,2	3,6	3,6	1,39		5,00	0,84	4,20
6th	21,6	3,6	3,6	1,39		5,00	0,84	4,20
5th	18	3,6	3,6	1,39		5,00	0,84	4,20
4th	14,4	3,6	3,6	1,39		5,00	0,84	4,20
3rd	10,8	3,6	3,6	1,39		5,00	0,84	4,20
2nd	7,2	3,6	3,6	1,39		5,00	0,84	4,20
1st	3,6	3,6	3,6	1,39		5,00	0,84	4,20
bg	0	3,6	1,8	1,39		2,50	0,84	2,10

table 3.7 distrubution over the height wind load cross direction

In the longitudinal direction the wind load is converted in a similar way, which leads to the values in the table below.



	pw (kN/m2)	cdim (-)	phi1 (-)	qw (kN/m2)
q1 (kN/m2)	1,2	0,86	1,17	1,21
q2 (kN/m2)	1,7	0,86	1,17	1,71

floor	start (m)	h floor (m)	h area floor (m)	qw (kN/m2)	qw add (kN/m2)	pw floor (kN/m)	reduction (-)	pw floor reduced (kN/m)
roof	95,2		2,6	1,73	0,02	4,49	0,42	1,89
25th	90	5,2	4,4	1,71	0,03	7,51	0,42	3,15
24th	86,4	3,6	3,6	1,67	0,03	6,03	0,42	2,53
23rd	82,8	3,6	3,6	1,65	0,03	5,93	0,42	2,49
22nd	79,2	3,6	3,6	1,62	0,03	5,83	0,42	2,45
21st	75,6	3,6	3,6	1,59	0,03	5,73	0,42	2,41
20th	72	3,6	3,6	1,56	0,03	5,63	0,42	2,37
19th	68,4	3,6	3,6	1,54	0,03	5,53	0,42	2,32
18th	64,8	3,6	3,6	1,51	0,03	5,44	0,42	2,28
17th	61,2	3,6	3,6	1,48	0,03	5,34	0,42	2,24
16th	57,6	3,6	3,6	1,46	0,03	5,24	0,42	2,20
15th	54	3,6	3,6	1,43	0,03	5,14	0,42	2,16
14th	50,4	3,6	3,6	1,40	0,03	5,04	0,42	2,12
13th	46,8	3,6	3,6	1,37	0,03	4,95	0,42	2,08
12th	43,2	3,6	3,6	1,35	0,03	4,85	0,42	2,04
11th	39,6	3,6	3,6	1,32	0,03	4,75	0,42	1,99
10th	36	3,6	3,6	1,29	0,03	4,65	0,42	1,95
9th	32,4	3,6	3,6	1,26	0,03	4,55	0,42	1,91
8th	28,8	3,6	3,6	1,24	0,03	4,45	0,42	1,87
7th	25,2	3,6	3,6	1,21		4,36	0,42	1,83
6th	21,6	3,6	3,6	1,21		4,36	0,42	1,83
5th	18	3,6	3,6	1,21		4,36	0,42	1,83
4th	14,4	3,6	3,6	1,21		4,36	0,42	1,83
3rd	10,8	3,6	3,6	1,21		4,36	0,42	1,83
2nd	7,2	3,6	3,6	1,21		4,36	0,42	1,83
1st	3,6	3,6	3,6	1,21		4,36	0,42	1,83
bg	0	3,6	1,8	1,21		2,18	0,42	0,91

table 3.8 distribution over the height wind load longitudinal direction

The way the wind in cross direction is imported on the Oval Tower can be seen in the figures below. Both figures give a top view of the Oval Tower.

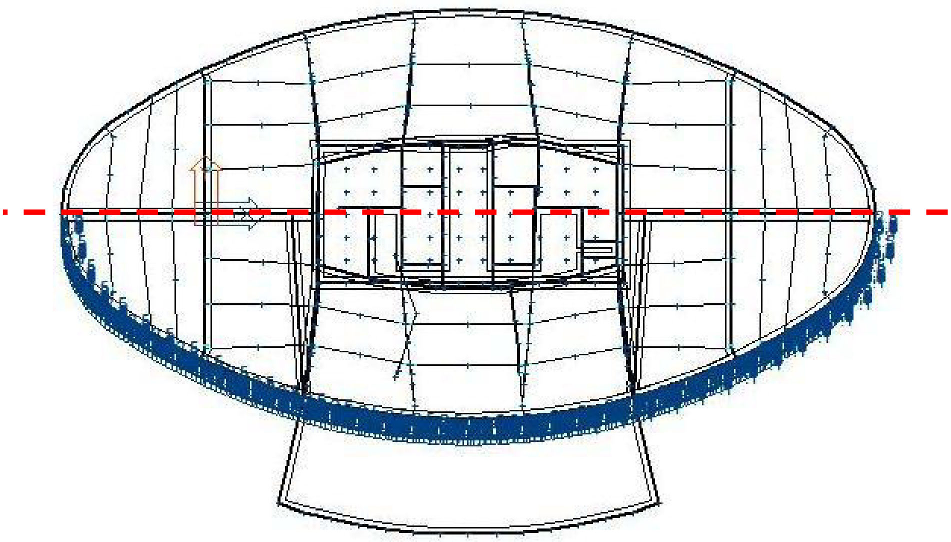


figure 3.6 wind in cross direction, axis of symmetry (dotted red line)

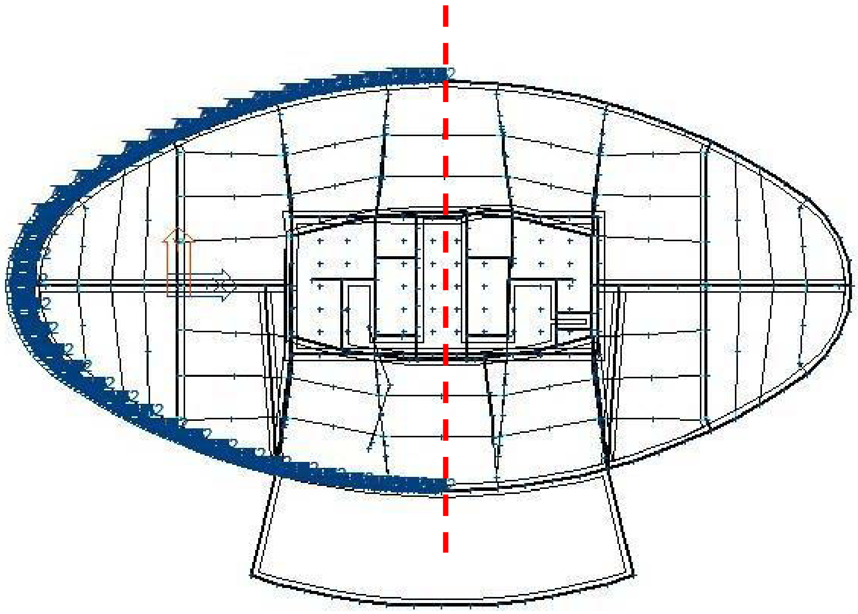
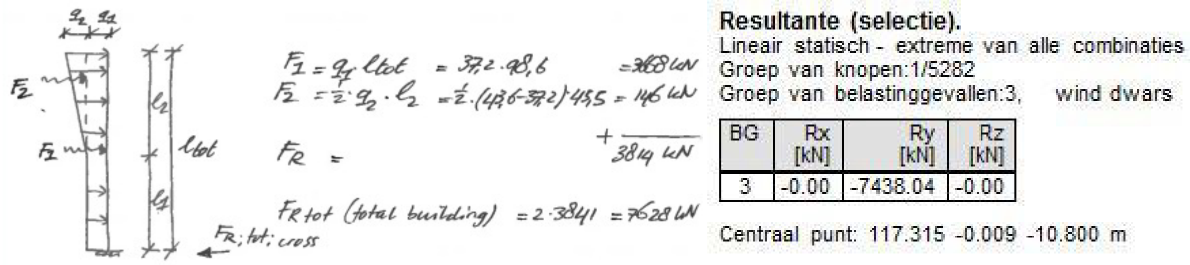


figure 3.7 wind in longitudinal direction, axis of symmetry (dotted red line)

Furthermore symmetry exists between the two half's. In both cases also wind can occur in the opposite direction. When making calculations of the project wind out of the opposite direction in both situations above also have to be taken into account. This can be done by multiplying the wind loads with a value of -1.

To see whether the wind load in both models is equal the total resultant horizontal forces will be compared.

First the resultant of the wind load in cross direction will be compared. Below the resultant of the [WSC OT] is calculated. The total resultant force because of wind load in cross direction on the total building becomes 7628 kN.



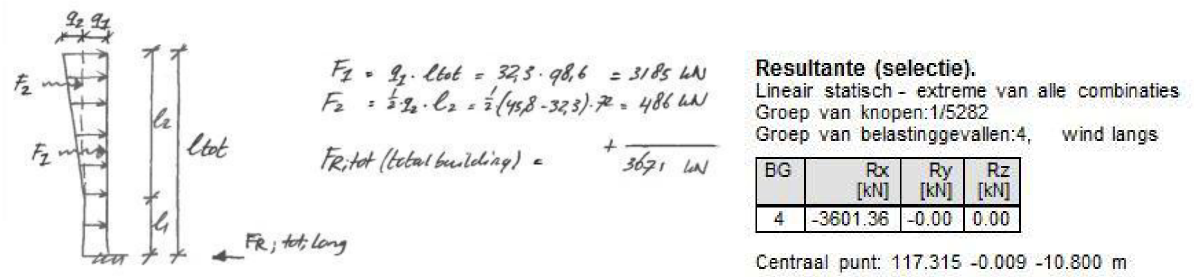
BG	Rx [kN]	Ry [kN]	Rz [kN]
3	-0.00	-7438.04	-0.00

Centraal punt: 117.315 -0.009 -10.800 m

figure 3.8 resultant horizontal force cross direction according to [WSC OT] (left), according to [ESA PW] (right)

The resultant in the [ESA PW] file can be automatically distributed by the program. The result can be seen in the figure below. Both resultants are about equal.

The same can be done for wind in longitudinal direction.



BG	Rx [kN]	Ry [kN]	Rz [kN]
4	-3601.36	-0.00	0.00

Centraal punt: 117.315 -0.009 -10.800 m

figure 3.9 resultant horizontal force longitudinal direction according to [WSC OT] (left), according to [ESA PW] (right)

It can be seen the resultant wind loads calculated according to the [WSC OT] are about equal to the resultant adjusted wind loads imported in the [ESA PW] model.

From here it can be concluded that the adjusted wind load which is taken into account in the [ESA PW] model is equal to the wind load which is taken into account in the [WSC OT] model and therefore representative for the wind load which is working on the Oval Tower.

3.1.4 Conclusion comparison between [WSC OT] and [ESA PW] model

In the previous sections a comparison has been made between two models. The first model [WSC OT] is an original set of equations which have been made during the design of the Oval Tower. Eventually with these equations the Oval Tower has been build. The second model is a finite element model [ESA PW] which have been used afterwards to determine the dynamical behaviour of the Oval Tower.

In this master thesis an extension on top of the Oval Tower will be calculated. It is intended to make use of a finite element model for this calculations. The comparison between the original calculations and the finite element model has been made to see whether the [ESA PW] model can be used for calculating the extension on top of the Oval Tower.

First the dead load is taken into account. It can be concluded that equality exists between both models. The biggest differences will occur in the columns on axis 2W and 2Y and in the left and right side of the core.

Afterwards the live load is taken into account. Again equality exists between both models. Again the biggest difference between the two models occurs in the columns on axis 2W, 2Y and the core.

The differences in dead load and live load has two causes. The first cause has to do with the detail in the loads which are imported in the program and which are used for the calculations in the original set of calculations. The second cause has to do with the way the floors are simulated in both set of calculations.

From the investigations it is concluded the column behaviour further will be taken into account with help of the values found in the [WSC OT]. For the stress behaviour in the core the values calculated with the [ESA PW] model will be taken into account. However the values which are calculated with help of the [ESA PW] model are slightly to high it can be assumed calculating with these values gives a safe assumption for the actual behaviour.

The third aspect which has been taken into account is the wind load. The wind load which is used in the existing [ESA PW] model was a statically wind load in order to calculate the dynamical behaviour. The values have been adopted. Both models have been compared by the horizontal resultant force in both directions. Equality exists between these values.

Overall it can be concluded that for further column calculations the values found in [WSC OT] can be used. When calculations of the core will be made the values determined in the [ESA PW] will be taken into account. This assumption gives values which are slightly to high, however the assumption is a safe one.

3.2 Stresses core

In this subchapter the stresses in the core will be taken into account. As described in the previous chapters four load cases will be taken care of, dead load, live load and wind load in two directions. Furthermore the resultant of this four load cases will be taken into account. When the resultant stress will be compared with the allowable stresses, it can be concluded whether the core contains overcapacity and possibly can be additionally loaded. Furthermore it is checked in which part of the core tension occurs. From this investigation it can be concluded whether cracks occur in the lower part of the core.

It is assumed the maximum stresses will occur in the outer region of the core. The stress in the partition walls therefore is neglected. Furthermore the lower abutments will be neglected.

The core is split up in two half's. From a structural point of view both half's work together. The reason for the separation only has to do with the visibility of the stresses for the reader of this report.

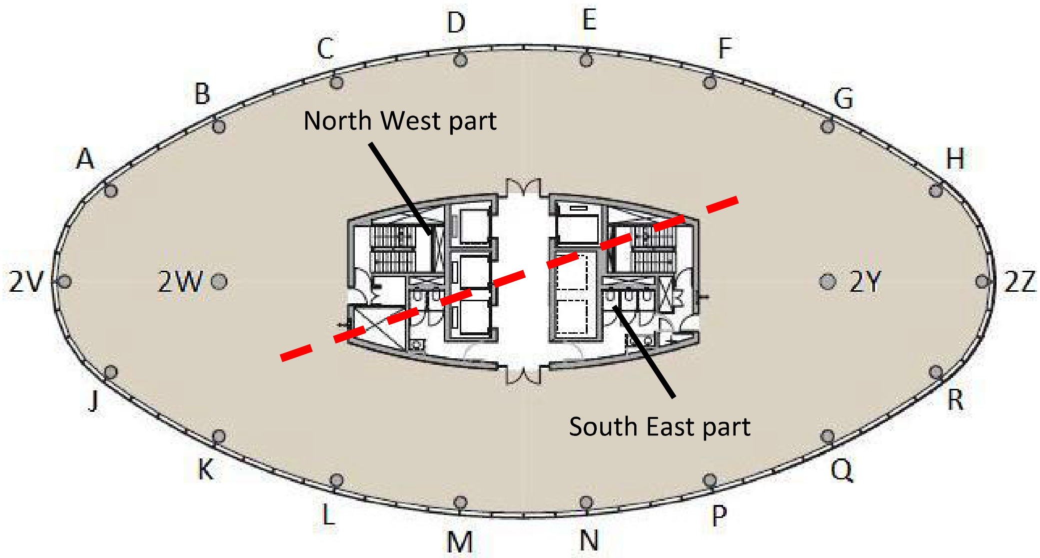


figure 3.10 Core split up in two half's

In table 2.2 the material of the concrete walls is defined. The concrete class of the walls is B45. The properties of this material can be calculated with the equations below.[6]

Design value compressive strength: $f'b = f'brep / \gamma m = 0{,}72 * f'ck / 1{,}2$
Design value tensile strength: $fb = fbrep / \gamma m = 0{,}7 * (1{,}05 + 0{,}05 * f'ck) / 1{,}4$
Average tensile strength: $fbm = 1{,}4 * fbrep = 1{,}4 * (0{,}7 * (1{,}05 + 0{,}05 * f'ck))$
Modulus of elasticity: $E'b =(22250 + 250 * f'ck)$

[6] (NEN6720, 1995)

	According to old drawings	Current notation	Characteristic compressive strength f'ck (N/mm^2)	Design value compressive strength f'b (N/mm^2)	Design value tensile strength fb (N/mm^2)	Average tensile strength fbm (N/mm^2)	Modulus of elasticity E'b (N/mm^2)
Concrete walls typical floor plan	B45	C35/45	45	27	1,65	3,3	33500

table 3.9 material properties Porthos Woensel

In the next subchapters the stresses in the core because of dead load, live load and wind load will be calculated. First the live loads will be calculated separately after which the resultant of the stresses will be calculated. The stresses will be measured with help of figures which are made with help of the [ESA PW] model. In this investigation the main stresses will be taken into account, local stresses will be neglected.

Note: In the various figures an uniform scale will be used. The boundaries of the scale are chosen just below the design tensile strength and just above the design compressive strength. The values are in MPa which is equal to N/mm2.

3.2.1 Dead load

In figure 3.11 the tension in the core because of dead load is drawn. It can be seen the division of the stresses is nearly symmetrical. No tension occurs and the maximum compressive value because of dead load will lie between 4 and 8 N/mm2.

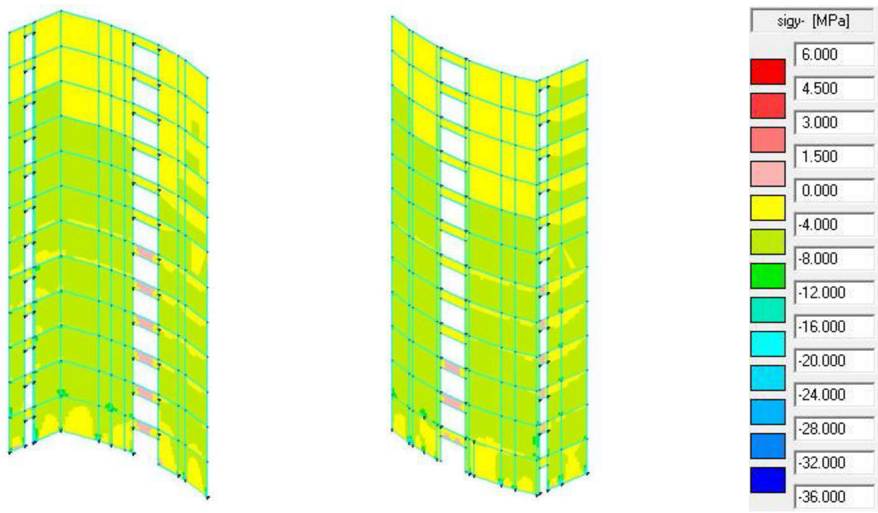


figure 3.11 stresses core dead load

3.2.2 Live load

The stresses because of live load are drawn in figure 3.12. It can be seen because of live load compression occurs with a maximum value of 4 N/mm2.

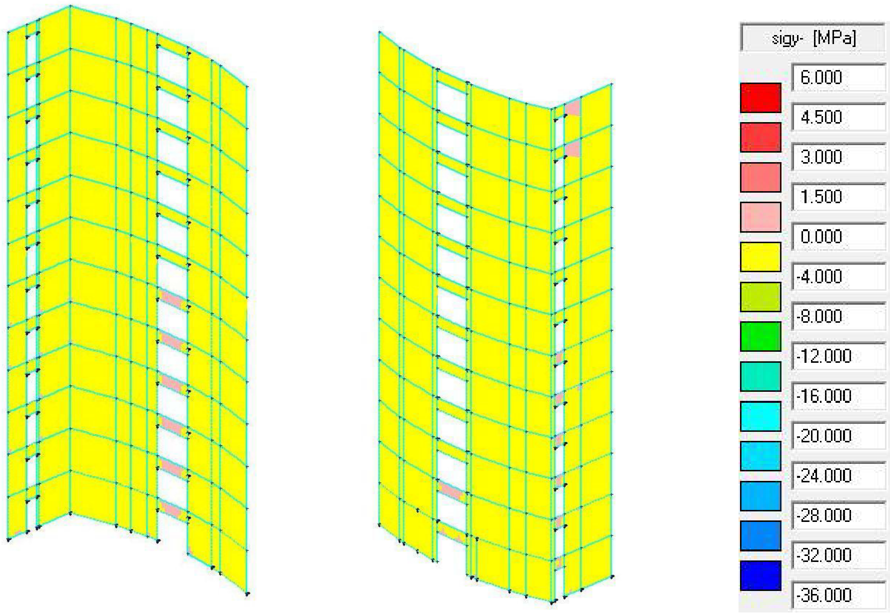


figure 3.12 stresses core live load

3.2.3 Wind load cross direction

Stresses because of wind load in the cross direction can be seen in figure 3.13. It is clearly visible tension occurs on one side of the core and compression on the opposite side. As explained before wind out of the opposite direction also can occur which means the tension and compression can occur at both sides.

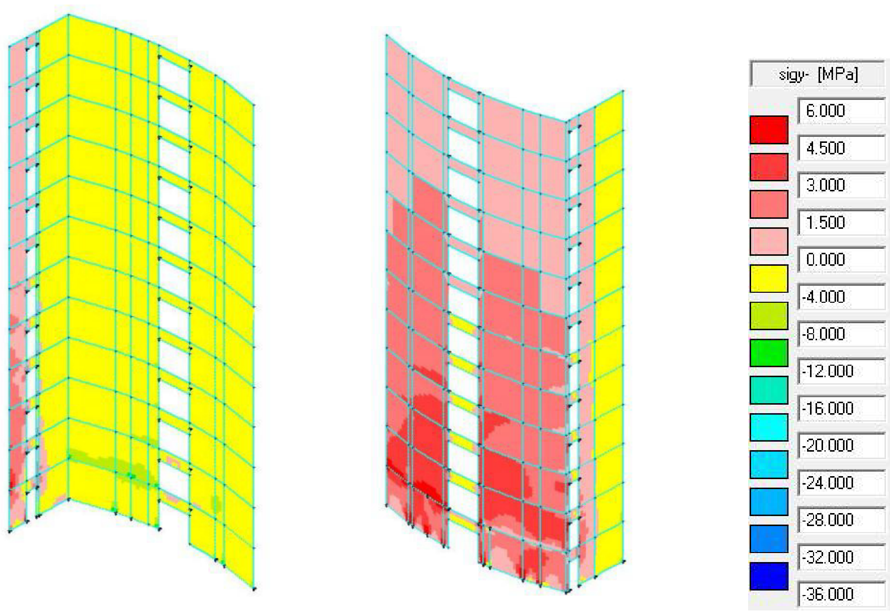


figure 3.13 stresses core wind cross direction

3.2.4 Wind load longitudinal direction

Stresses because of wind load in the longitudinal direction can be seen in the figure below. Stresses will occur at one side of the core, while on the opposite site compression occurs.

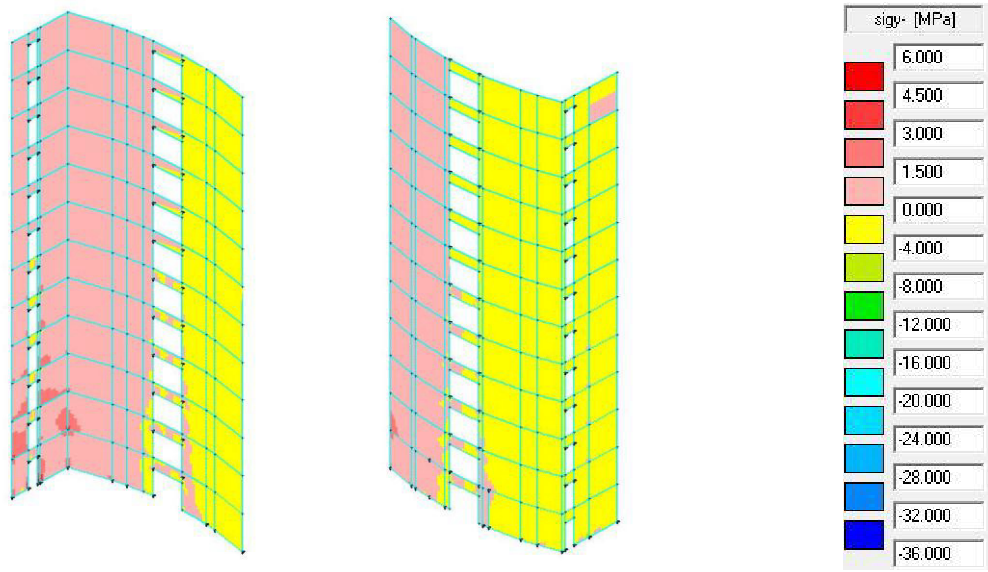


figure 3.14 stresses core wind longitudinal direction

What can be learned from the investigation above is that tensile stresses in the core can occur due to wind load. However this stresses are calculated for the wind separately which means no dead load is present. To give a valid answer to the question whether tension occurs in the final situation the resultant of the stresses has to be taken into account which is given by the envelope. Two situations will be taken into account. First the maximum envelope, in this figure the maximum stress will be given. The second envelope is the minimum envelope. The minimum envelope will give an idea whether tension is present in the final situation.

3.2.5 Envelope

The stresses calculated in the previous sections can be combined till an envelope. The envelope in the SLS and ULS will be taken into account.

The envelope in the SLS will be taken into account to see whether tension occurs in the core. The envelope in the ULS will be taken into account to check the maximum compression and tension. Because the existing situation of the Oval Tower will be taken into account it can be expected these values will not exceed the allowed values.

3.2.5.1 SLS

The envelope in SLS is given by the combinations in the table below.

Load Combination	SLS/ULS	BG1; DL	BG2; LL	BG3; WLC	BG4; WLL
LC1	SLS	1,0	1,0 * ψ	1,0	
LC2	SLS	1,0	1,0 * ψ		1,0

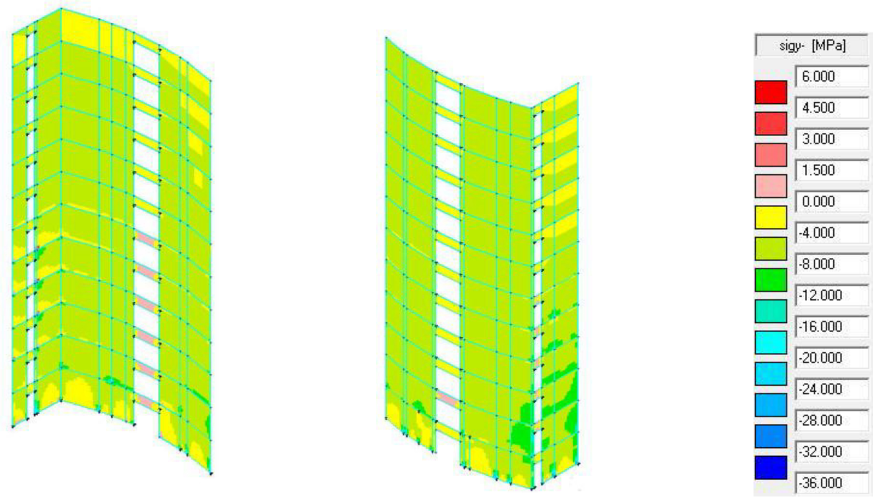


figure 3.15 maximum envelope SLS

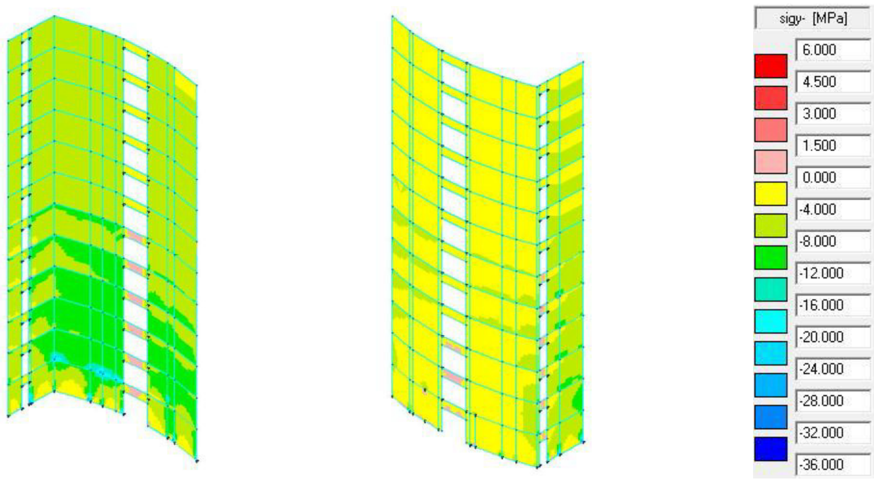


figure 3.16 minimum envelope SLS

In the figures above it can be seen no tension occurs in the SLS. This means no reduction has to be made to the modulus of elasticity.

3.2.5.2 ULS

The envelope in ULS is given by the combinations in table 2.5, the ULS value of this table again are given below.

Load Combination	SLS/ULS	BG1; DL	BG2; LL	BG3; WLC	BG4; WLL
LC3	ULS	1,2	1,5 * ψ	1,5	
LC4	ULS	1,2	1,5 * ψ		1,5
LC5	ULS	0,9		1,5	
LC6	ULS	0,9			1,5

table 3.10 load combinations

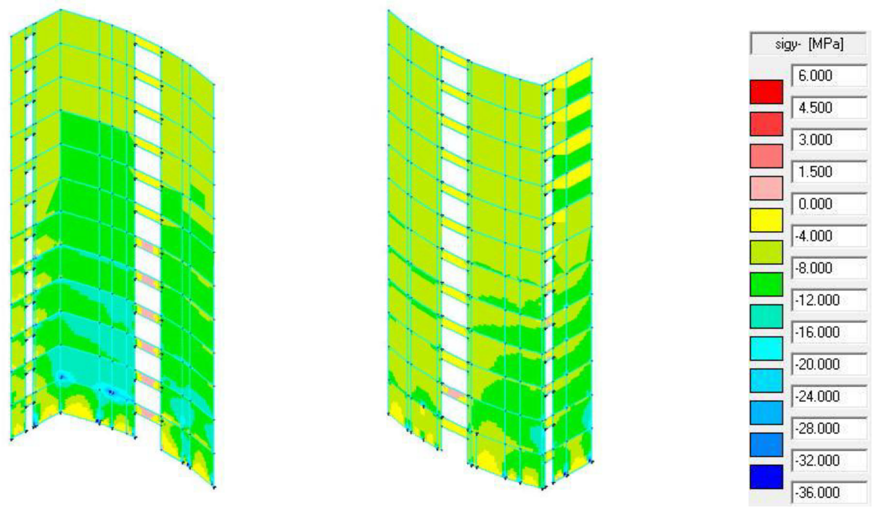


figure 3.17 minimum envelope ULS

The minimum stresses (which means a maximum compression) in the lower part of the core will lie between 16 and 20 N/mm2. In figure 3.18 the maximum stresses are drawn. According to this figure the maximum value of the tensile stress will lie between 1,5 and 3,0 N/mm2.

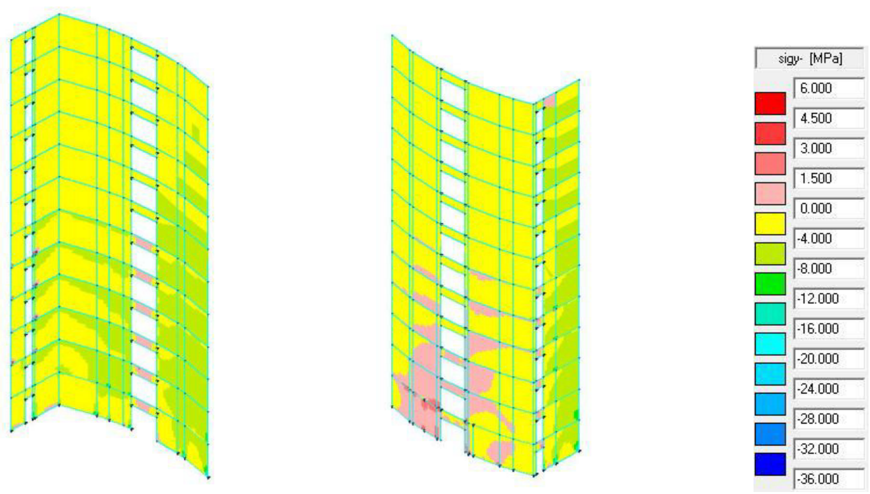


figure 3.18 maximum envelope combined stresses

In the investigations above, the maximum compression and tension forces are calculated. The values occur because of wind out of a singular direction. In figure 3.6 and figure 3.7 the wind load only will be taken into account in one direction. Furthermore it can be seen there is symmetry. This means the minimum and maximum value can occur at the calculated site as well as the opposite site of the core.

3.2.6 Conclusion

In the investigation above local values on a small area occur. These values in this investigation have been neglected. For this investigation the main areas in which tension or compression occurs have been taken into account.

From the investigation in the previous section it can be concluded no tension will occur in the SLS. This means the modulus of Elasticity does not have to be reduced for calculations in which the top deflection will be elaborated.

From the investigations in the ULS it can be concluded that the maximum compression force in the core will not exceed the value of 20 N/mm2. The allowable value for the compression force in the core is 27 N/mm2. This means overcapacity is available in the core and the core can be additionally loaded.

In the lower six floors of the core tension will occur. The allowable tension in the core is 1,65 N/mm2. In the lower three floors tension with a value close to 1,65 N/mm2 will occur. For this floors a detailed investigation is needed when building an extension.

The core can deal with additional compression forces. However additional tensile forces are not allowed. Because no additional tension in the core is allowed, tensile forces in the lower part of the core is a critical aspect in this investigation.

When an extension will be built on top of the Oval Tower the same checks can be elaborated. With these checks it can be seen whether adjustments have to be made to the existing core.

3.3 Columns

3.3.1 Design values

In chapter 3.1 the forces because of dead load and live load have been described. In this subchapter the design load of the columns will be determined. The values will be determined by the forces because of dead load, live load and additional wind loads because. To determine the forces first it has to be clear in which way the forces will be transferred to the foundation. For the dead load and live load this already has been described, for the additional wind load the transfer of loads will be described below.

First the wind out of the cross direction will be taken into account. In this direction outriggers are present. The location of the outriggers can be seen in the figure below. The outriggers are present in the upper floor and therefore cause additional forces in the columns on axis C, D, E, F and L, M, N, P and the core.

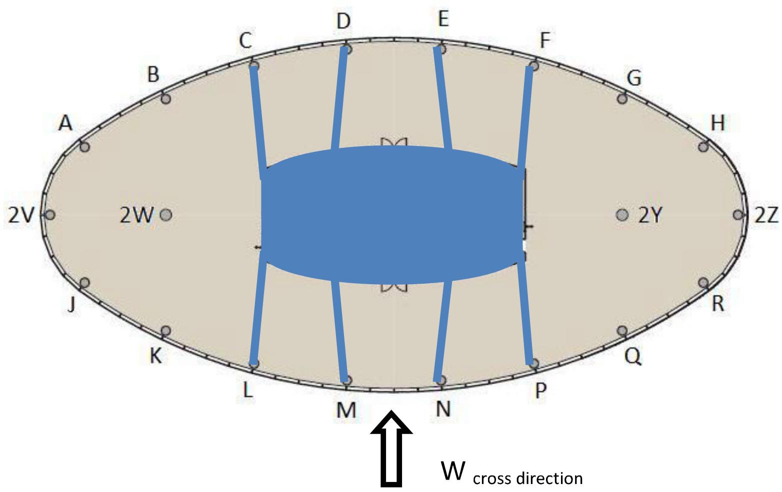


figure 3.19 wind load cross direction, location outriggers top floor

In figure 3.20 again a horizontal cross section of the building has been drawn. The wind load in the longitudinal direction will be transferred via the core to the foundation. Subsequently the loads will be transferred to the foundation. The additional loads can be expected below the core and therefore no additional forces in the columns occur because of wind in this direction.

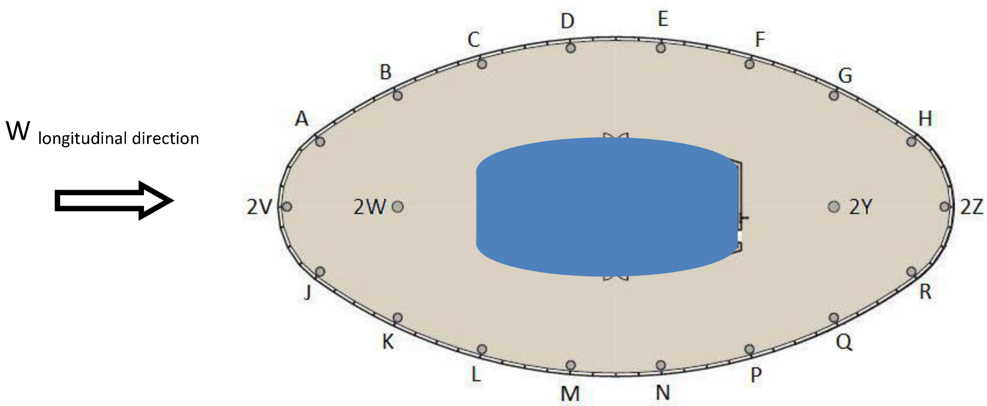
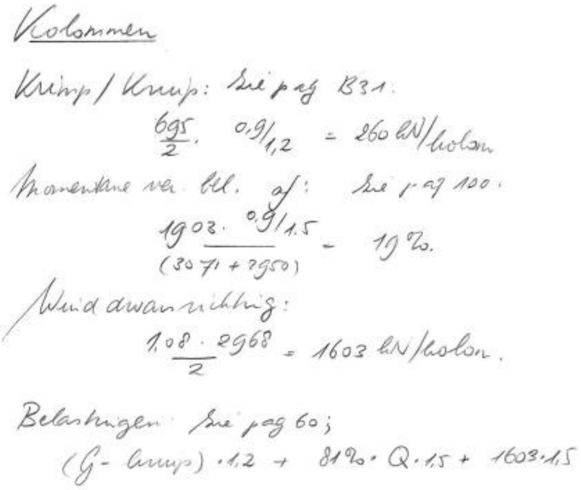


figure 3.20 wind load longidutional direction, location core

As described in section 3.1.4 for the column check the [WSC OT] can be used. The total additional load on the axes C, D, E, F and L, M, N, P because of wind load is calculated in the [WSC OT] and has a maximum value of 1603 kN per column. In these calculations furthermore the shrinkage and creep behaviour and the redistribution because of live load has been taken into account. In the figure below a part out of the calculations can be seen.



N'd = (G–creep)*1,2 + 0,81*Q*1,5 + W*1,5 (1)

N'd = G * 1,2 + Q * 1,5 (2)

figure 3.21 Column calculation according to [WSC OT]

The calculations above have been made to determine the total reaction force below the column. Therefore the reaction force because of wind load (2968kN) has been taken into account. In the figure below the reaction forces below the column can be seen. Furthermore it can be seen that the force which is working in the column is not equal to the reaction force below the column. The reaction force which is working in the column is 2022kN.

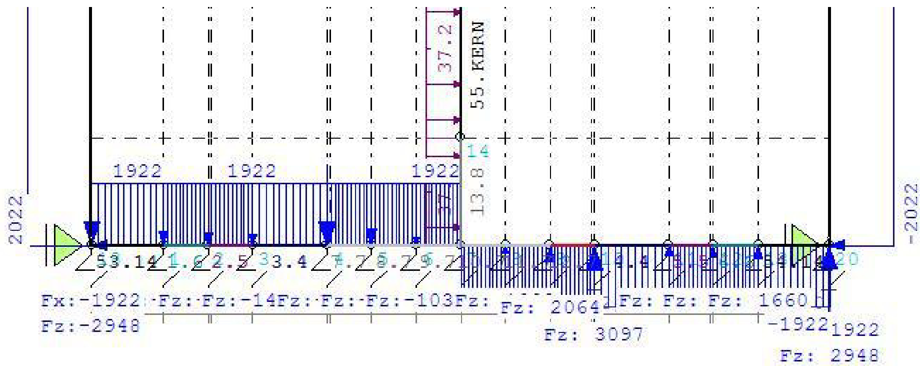


figure 3.22 normal forces and reaction forces because of wind

The model which is taken into account is the same model which is used in the [LS]. Because of symmetry a half building is modelled in which two columns are modelled as a single column. With help of the equation in figure 3.21 the reaction force in the column can be calculated and becomes: (1,08*2022) / 2 = 1092 kN.

Furthermore it can be seen the columns C, D, E, F and L, M, N, P can be calculated with formula (1), the rest of the columns can be calculated with formula (2). These calculations have been made in the table on the next page.

	WSC OT			
Column on axis	G (kN)	Q (kN)	W (kN)	N'd (kN)
A	4779	636		6689
B	6824	1397		10284
C	6984	1580	1092	11627
D	6798	1465	1092	11264
E	6798	1465	1092	11264
F	7062	1606	1092	11752
G	6934	1434		10472
H	4779	636		6689
J	4800	657		6746
K	6740	1412		10206
L	6959	1463	1092	11454
M	6895	1414	1092	11318
N	6948	1429	1092	11400
P	7406	1521	1092	12061
Q	6839	2037		11262
R	4832	1227		7639
2V	6891	998		9766
2W	18431	5347		30138
2Y	18649	6235		31731
2Z	6968	1438		10519

table 3.11 Design load columns according to [WSC OT]

3.3.2 Column capacity

Now that the design value is know the capacity of the columns has to be calculated. The capacity can be determined with help of various equations out of NEN6720. To fill in these equations first the dimension of the columns have to be determined. The prefabricated columns are reinforced. In [SD] the dimension of the columns including reinforcement is given.

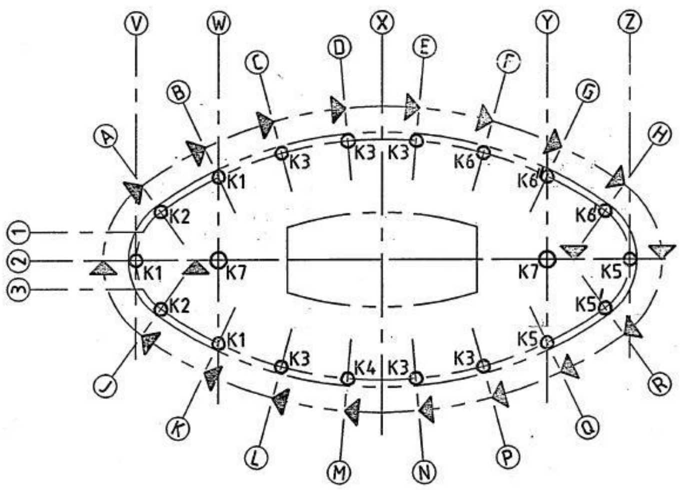


figure 3.23 overview columns Oval Tower

As an example the reinforcement as drawn in column K1 is given in the figure below. The remaining columns K2 till K6 are reinforced in a similar way, the amount of rebar's and the diameter of the rebar's is different. The two columns K7 on axis W2 and Y2 differ in dimension compared to the remaining columns. Furthermore the reinforcement used in these columns is executed in two rings.

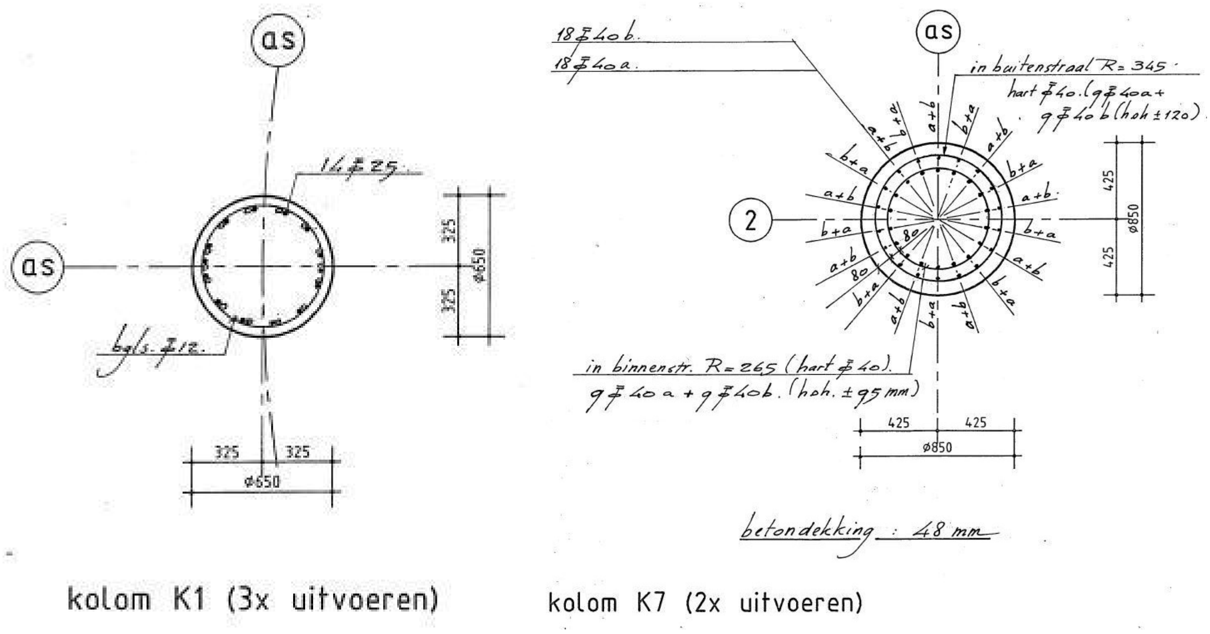


figure 3.24 reinforcement column K1 (left), K7 (right)

In the table below an overview of the properties of the reinforcement used in the various columns is given. With the formula $A = n * \pi * r^2$ the area of the reinforcement is determined. The area of the various columns can be seen in the table below.

	First ring		Second ring		
Column	Number of reinforcement bars (-)	Diameter reinforcement bar (mm)	Number of reinforcement bars (-)	Diameter reinforcement bar (mm)	A (mm2)
K1	14	25			6872
K2	14	20			4398
K3	18	25			8835
K4	18	25			8835
K5	18	25			8835
K6	14	25			6872
K7	18	40	18	40	45238

table 3.12 area columns Oval Tower

In most cases when a structural engineer is calculating a project the reinforcement in a column will be determined with help of the loads which are working on the column. First the column dimension will be estimated. According to the loads working on the column the reinforcement in the column will be determined. A column is economically reinforced when the reinforcement percentage is below 8%. A column calculation has to be made for various projects, therefore at Zonneveld Ingenieurs a sheet is available which can be used to determine the reinforcement in the column. An overview of the calculation sheet is given in the figure below.

3.3.3 Column check

The next step is to compare the design load of the columns with the capacity of the column. This check also is known as a unity check. In the table below the values are given in the diagram below an overview of the various unity checks.

UC = N'd / N'u

	WSC OT					Capacity	
Column on axis	G (kN)	Q (kN)	W (kN)	N'd (kN)	column	N'u (kN)	UC [WSC OT]
A	4779	636		6689	k2	10900	0,61
B	6824	1397		10284	k1	11700	0,88
C	6984	1580	1092	11627	k3	12300	0,95
D	6798	1465	1092	11264	k3	12300	0,92
E	6798	1465	1092	11264	k3	12300	0,92
F	7062	1606	1092	11752	k6	11700	1,00
G	6934	1434		10472	k6	11700	0,90
H	4779	636		6689	k6	11700	0,57
J	4800	657		6746	k2	10900	0,62
K	6740	1412		10206	k1	11700	0,87
L	6959	1463	1092	11454	k3	12300	0,93
M	6895	1414	1092	11318	k4	12300	0,92
N	6948	1429	1092	11400	k3	12300	0,93
P	7406	1521	1092	12061	k3	12300	0,98
Q	6839	2037		11262	k5	12300	0,92
R	4832	1227		7639	k5	12300	0,62
2V	6891	998		9766	k1	11700	0,83
2W	18431	5347		30138	k7	31100	0,97
2Y	18649	6235		31731	k7	31100	1,02
2Z	6968	1438		10519	k5	12300	0,86

table 3.14 Unity check existing columns Oval Tower

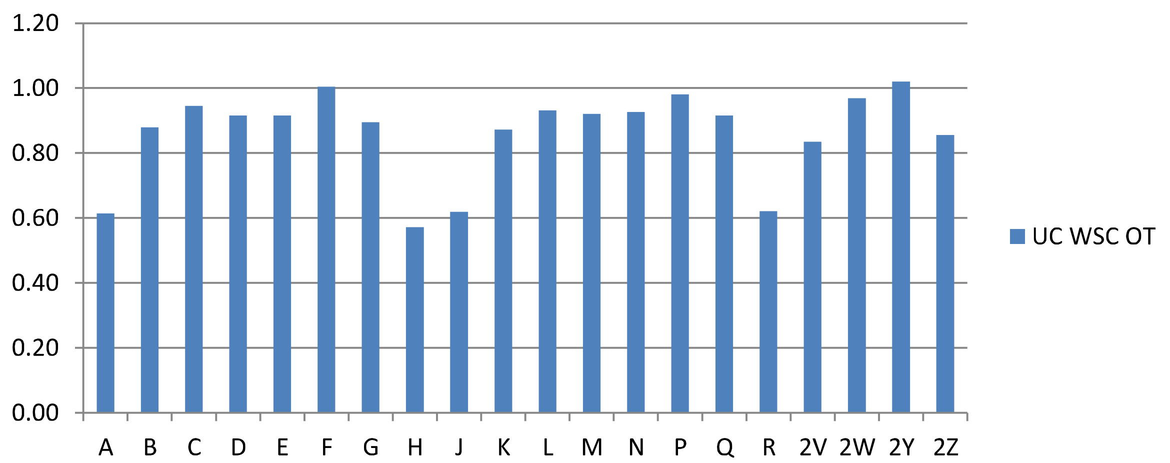


diagram 3.1 UC columns according to WSC OT

From this investigation it follows that several unity checks are close to one. It can be concluded that the columns with an unity check close to cannot be additional loaded. The columns A, H, J and R are low in unity check and therefore overcapacity is present in these columns. Furthermore the columns B, G, K, Q, 2V and 2Z also can be slightly additional loaded. An overview of the location of these columns is given in the figure below.

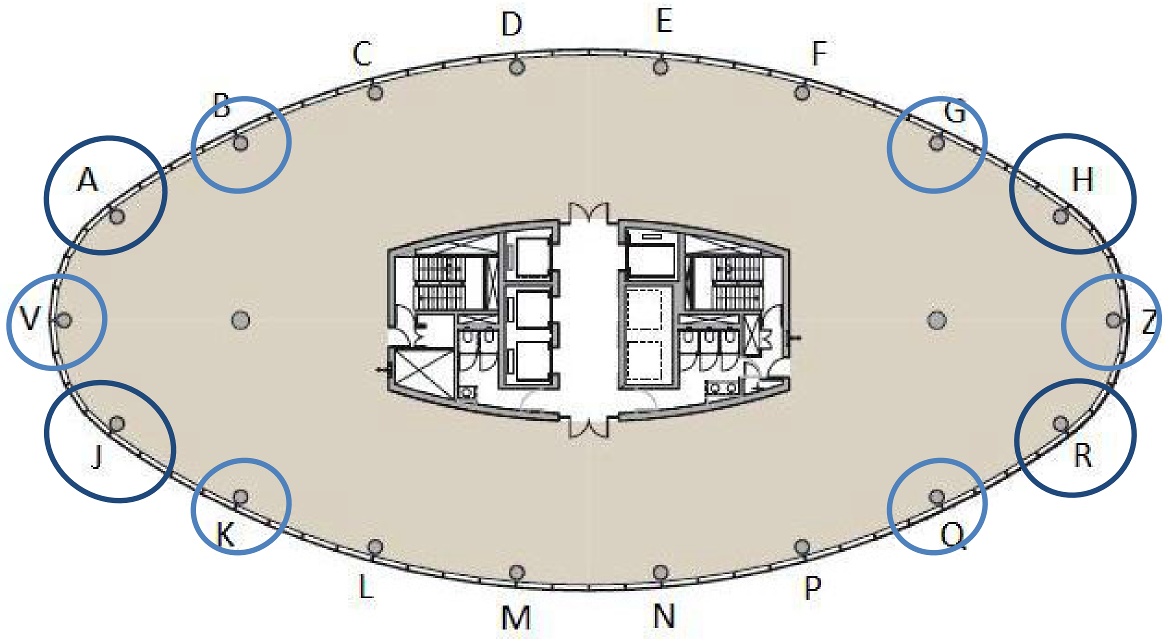


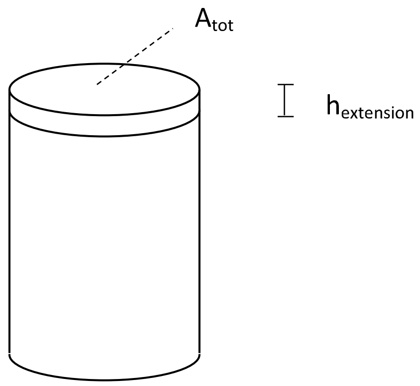
figure 3.25 Columns which contain overcapacity, (red circle UC ≈ 0,6; blue circle UC ≈ 0,85)

From this investigation it will be recommended the columns subjected to wind load and the columns 2W and 2Y cannot be additional loaded. The other columns contain overcapacity. It can be concluded these columns can be used to transfer additional loads to the foundation. The available overcapacity is given in the table below.

Column	Overcapacity (N'u - N'd) (kN)
A	4211
B	1416
G	1228
H	5011
J	4155
K	1494
Q	1038
R	4661
V	1934
Z	1781
Total	26929

figure 3.26 Overcapacity column not subjected to wind load

The total load of the extension can be estimated with help of the calculation below, which includes the total dead load of the structure and the live load on the floors:



$F_{total} = \gamma * q_{dl} * A_{tot} * h_{extension} + \gamma * q_{vl} * A_{tot}$ * number of floors = 1,2 * 2,4 * 1000 * 10 + 1,5 * 2,5 * 1000 * 2 = 36300 kN.

36300 > 26929

With help of the investigation above the following conclusion can be drawn: The columns which contain overcapacity can help in the vertical transfer of forces but are unable to deal with the total load of the extension. The columns can transfer approximately 70% of the total load of the extension.

3.4 Foundation

To make an investigation of the foundation first the pile plan will be investigated. The pile plan is given in appendix B. It can be clearly seen piles are present below the core and the columns. Furthermore it also can be seen the pile plan is symmetric. Because of symmetry of the pile plan and of the tower, the project can be divided into four parts with an equal area.

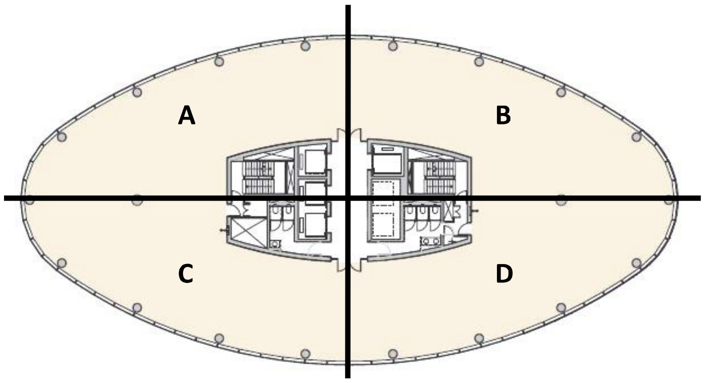


figure 3.27 symmetry Oval Tower

The load bearing capacity of the piles in the pile plan differ in the various areas. On the drawing of the pile plan in appendix B it can be seen area D is the area with the lowest load bearing capacity. In this chapter the maximum column and core loads will be compared with the load bearing capacity of the foundation piles. A distinction will be made into piles below the core and piles below the columns.

3.4.1 Foundation below core

The foundation below the core in area D is drawn in the figure below, this is a part out of the drawing in appendix B. The piles which belong to the area below the core in part D are numbered from 1 till 23 and 41 till 46, the numbering is taken over from [WSC OT].

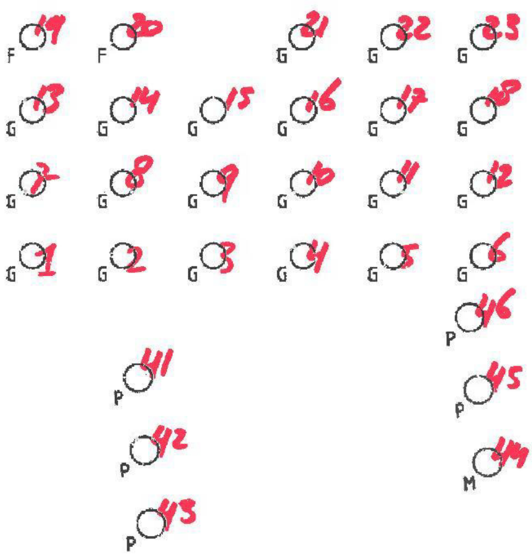


figure 3.28 piles below core area D

The maximum forces are determined and are the result of a combination of dead load, live load and wind load in both directions. In the figures below the reaction forces according to dead load and live load can be seen. In chapter 3.1.2 it is already stated these loads have a higher value compared to the loads which are taken into account in the [WSC OT] and therefore this estimation can be seen as a safe assumption.

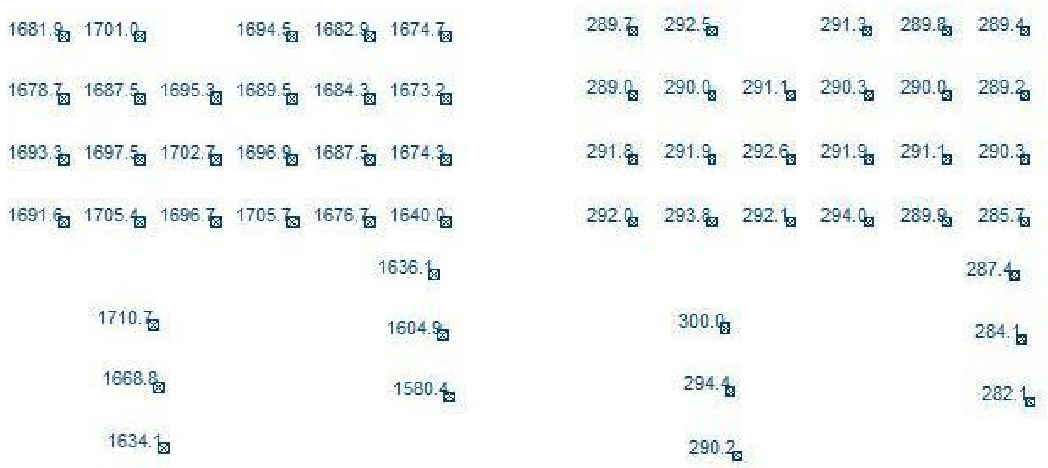


figure 3.29 reaction forces core area D dead load (left) live load (right)

The maximum compression forces because of wind load will occur in area B. The compression forces in this area are drawn in the figures below.

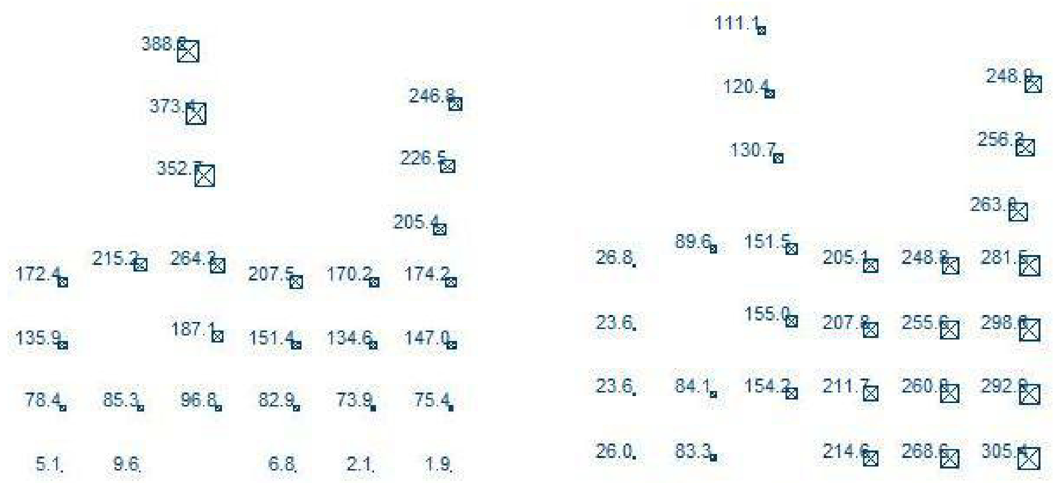


figure 3.30 reaction forces core area B wind load cross direction (left) longitudinal direction (right)

Because of symmetry reasons mentioned before, these values can also be applied to part D. This can be done by mirroring the values around the symmetry axis between area B and D.

In the table on the next page the design loads are elaborated. Furthermore the capacity of the piles is given. An unity check will be elaborated in which UC = N'd / N'u.

pileplan right									
number	G (kN)	Q (kN)	Wcross (kN)	Wlong (kN)	Wmax (kN)	N'd (kN)	N'u (kN)	UC (-)	Overcapacity (kN)
1	1692	292	172	27	172	2726	3224	0,85	498
2	1705	294	215	90	215	2810	3224	0,87	415
3	1697	292	264	152	264	2870	3224	0,89	354
4	1706	294	208	205	208	2800	3224	0,87	424
5	1677	290	170	249	249	2821	3224	0,87	403
6	1640	286	174	282	282	2820	3224	0,87	404
7	1693	292	136	24	136	2674	3224	0,83	550
8	1698	292	0	0	0	2476	3224	0,77	748
9	1703	293	187	155	187	2764	3224	0,86	460
10	1697	292	151	207	207	2785	3224	0,86	439
11	1688	291	135	256	256	2846	3224	0,88	378
12	1674	290	147	299	299	2892	3224	0,90	332
13	1679	289	78	24	78	2565	3224	0,80	659
14	1688	290	85	84	85	2588	3224	0,80	636
15	1695	291	97	154	154	2702	3224	0,84	523
16	1690	290	83	212	212	2781	3224	0,86	443
17	1684	290	74	261	261	2847	3224	0,88	377
18	1673	289	75	293	293	2881	3224	0,89	343
19	1682	290	5	26	26	2492	4642	0,54	1075
20	1701	293	10	83	83	2605	4642	0,56	1018
21	1695	291	7	215	215	2793	3224	0,87	216
22	1683	290	2	269	269	2858	3224	0,89	183
23	1675	289	2	305	305	2901	3224	0,90	162
41	1711	300	353	131	353	3033	3713	0,82	680
42	1669	294	373	120	373	3003	3713	0,81	710
43	1634	290	388	111	388	2978	3713	0,80	735
44	1580	282	247	249	249	2693	3652	0,74	960
45	1605	284	227	256	256	2736	3713	0,74	977
46	1636	287	205	263	263	2788	3713	0,75	925
									16024

table 3.15 loads and overcapacity below piles area D core (values in kN)

Furthermore in the table the available overcapacity is determined. Here a note has to be made that the piles 19 till 23 are exactly in the middle of the line of symmetry. A part of the loads which are working to area B also will be transferred to these piles. The values for overcapacity therefore are divided by two. The sum of this individual loads is the total overcapacity which is available below the core.

The total overcapacity below the core at least is 4 * 16024 = 64096 kN. Because the pile capacity below the areas A, C and D is higher, the overcapacity will also be higher. In the last chapter an estimation for the weight of the extension was made. The weight was 36300 kN. According to this investigation the foundation below the core is able to deal with the weight of the extension.

3.4.2 Foundation below columns

The same procedure as in the last subchapter can be followed with regard to the piles below the columns. The symmetry as described before does not count for the piles below the columns. Therefore every single column is taken into account.

The maximum design loads are the loads which are determined in the [WSC OT] as described in the chapter in which the columns are checked. Here a note has to be made that the forces below the columns have a bigger value compared to the loads in the columns, this is due to the fact the lower abutments are connected with the outer columns. Furthermore the foundation piles are modelled as springs. In the table below the design load compared to the capacity of the foundation.

Axes	N'd (kN)	Piles	Cap pile	Cap piles	UC	OC (kN)	OC (kN)
A	6689	2	4118	8236	0,81	1547	
B	10284	3	4158	12474	0,82	2190	
C	12393	4	4158	16632	0,75		4239
D	12030	4	3756	15024	0,80		2994
E	12030	4	3566	14264	0,84		2234
F	12518	4	3566	14264	0,88		1746
G	10472	3	4160	12480	0,84	2008	
H	6689	3	2600	7800	0,86	1111	
J	6746	2	4228	8456	0,80	1710	
K	10206	3	4228	12684	0,80	2478	
L	12221	4	4338	17352	0,70		5131
M	12085	4	4338	17352	0,70		5267
N	12166	4	3713	14852	0,82		2686
P	12828	4	3652	14608	0,88		1780
Q	11262	4	2842	11368	0,99	106	
R	7639	3	2842	8526	0,90	887	
2V	9766	3	4228	12684	0,77	2918	
2W	30138	8	4228	33824	0,89		3686
2Y	31731	9	3713	33417	0,95		1686
2Z	10519	4	2842	11368	0,93	849	

table 3.16 unity check foundation piles below columns

It can be concluded all piles meet the requirements. In section 3.3.3 it is recommended to not additionally load the columns which are subjected to wind load and the columns 2W and 2Y. Therefore a distinction is made between the piles which can be additional loaded and the piles which are recommended to not additionally load.

3.5 Top deflection

Horizontal forces will cause top deflection in a building. In the [QS] this top deflection already has been calculated. When the [WSC OT] will be taken into account in detail, two various calculations have been made.

The first situation describes the behaviour of the tower in case the core is uncracked. The second situation describes the behaviour of the tower when tension occurs in the lower six meter of the core. The difference in behaviour has been modelled by varying the modulus of elasticity. In the table below the values which have been used in the existing calculations can be seen.

	Lower six floors	Upper floors
Situation 1: entire core imported as uncracked concrete	33500 N/mm2	33500 N/mm2
Situation 2: lower six floors imported as cracked concrete	15000 N/mm2	33500 N/mm2

table 3.17 modulus of elasticity according to [WSC OT] for both situations

figure 3.31 top deflection situation 1 (left), situation 2 (right)

The allowed value for top deflection is h/500 which is equal to 98600/500 = 197mm. Both 117<197 and 107<197; which means the requirements for top deflection will be met according to the [WSC OT].

In the previous calculation the modulus of elasticity has been chosen as 33500 N/mm^2 for uncracked concrete and 15000 N/mm^2 for cracked concrete. When the building will be calculated according to the Eurocode the modulus of elasticity has to be reduced. With help of the Eurocode[7] and with help of a textbook about concrete structures in the Eurocode[8] the modulus of elasticity for cracked and uncracked concrete has been determined. Below the equations which have been used. Furthermore the corresponding values have been given. As described in chapter 2.3 the concrete class of the core is C35/45, for E_{cm} a value of 33500 N/mm^2 can be used.

	Equation	Value for E_{cd} (N/mm2)
$E_{cd,cracked}$	$E_{cd,cracked} = 0{,}4 * E_{cm} / 1{,}2$	11667 (N/mm2)
$E_{cd,uncracked}$	$E_{cd,uncracked} = 0{,}8 * E_{cm} / 1{,}2$	22333 (N/mm2)

table 3.18 calculation Ecd according to Eurocode

Again the Oval Tower without extension will be taken into account.

	Lower six floors	Upper floors
Situation 3: entire core imported as uncracked concrete	22333 N/mm2	22333 N/mm2
Situation 4: lower six floors imported as cracked concrete	11667 N/mm2	22333 N/mm2

table 3.19 modulus of elasticity according to [WSC OT] for both situations

figure 3.32 top deflection situation 3 (left) and situation 4 (right)

The maximum top deflection in this situation becomes 135 mm, this value is still below the value of 197mm.

---

[7] (NEN-EN 1992-1-1)
[8] (Braam, et al., juli 2011)

3.6 Dynamical behaviour

In the [QS] the acceleration of the Oval Tower is taken into account. It is concluded the value which is calculated in the [WSC OT] is calculated with help of NEN6702 without modifications. After the calculations of the Oval Tower where made modifications have been made to NEN6702. In the [QS] the acceleration is calculated with help of NEN6702 with modification. In the table below an overview of this calculation.

		NEN6702	NEN6702 adopted values TNO
a_{max}	Acceleration (m/s2)	0,095	0,073
$\emptyset_2$	Factor dependent on the structure (-)	0,56	0,43
$p_{w;1}$	Fluctuating wind pressure (N/m2)	620	620
C_t	Wind factors for pressure and suction (-)	1,2	1,2
b_m	Building length perpendicular to wind direction (m)	53,4	53,4
ρ_1	Volumetric mass per meter height (kg/m)	371150	371150
D	Measure of damping (-)	0,010	0,007
f_e	First natural frequency (hz)	0,28	0,41
$f_{e;new}$	f_e multiplied by (1 + 20/h)	0,34	0,49
h	Height (m)	98,6	98,6

table 3.20 calculated value for the acceleration with help of NEN6702

After the Oval Tower has been build nuisance because of dynamical behaviour appeared. Because of this nuisance, TNO investigated the dynamical behaviour of the Oval Tower. A report of these measurements is admitted in appendix C. With help of these measured values again the acceleration is calculated according to NEN6702. The outcome is adopted in table 3.20 and can be seen in figure 3.33.

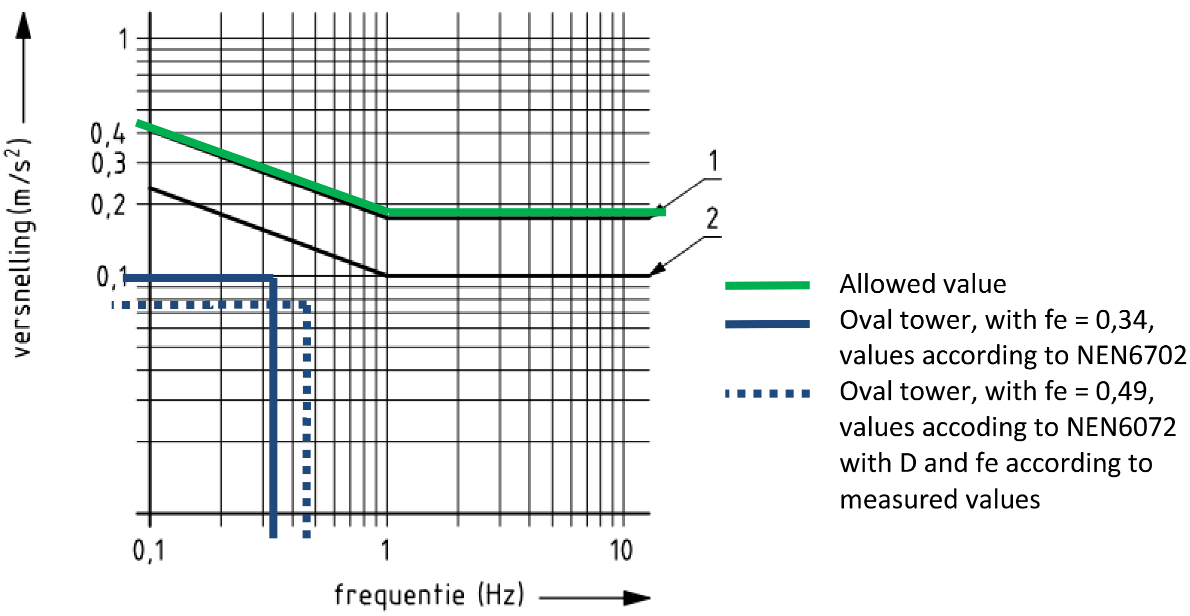


figure 3.33 acceleration Oval Tower

From here it can be concluded that requirements for dynamical behaviour according to NEN6702 will be met. The damping and first natural frequency are calculated by TNO. The value for acceleration is calculated with help of these values.

4 Conclusions existing situation

Chapter 1.4 presented the main research question together with the following two secondary questions aiming to provide insights with regards to the existing Oval Tower.

- *What is/are the most leading structural aspect(s) in the current design of the Oval Tower?*
- *Where does the current Oval Tower contain overcapacity?*

Chapter 3 provided the calculations based on the existing situation of the Oval Tower and are briefly discussed per section below. The summarisation of these sections will assist in delivering an answer to the secondary research questions as presented above.

Furthermore the literature study provides several recommendations of which the below two are key to this research:

- *Where possible, make use of the overcapacity as present in the existing load bearing elements.*
- *Where possible, adapt the structural design of the extension in such a way that reaction forces will be distributed over the elements containing overcapacity.*

Therefore recommendations will be made with regard to the possible shape of the extension and the way the extension has to deal with the transfer of loads to the existing building.

In <u>section 3.1</u> a comparison has been made between two existing models of the Oval Tower. The first model is a set of elaborated hand calculations, the weight and stability calculations, which were used in the actual delivery and build of the Oval Tower. The second model is a finite element model developed to check the dynamical behaviour of the tower after the construction was finished. In the further elaboration phase it is intended to make use of the values found in the hand calculations as well as the values found with help of the finite element model; therefore both models have to be in consensus.

After taking both models into account, it can be concluded that for further column calculations the values found in the weight and stability calculations can be used for further calculations. When additional calculations of the core will be made the values found with help of the finite element model can be used.

<u>Section 3.2</u> presents the results from research into the stresses occurred in the core of the Oval Tower. Conclusion made is that no tension will occur in the core in serviceability limit state, meaning that the modulus of elasticity does not has to be reduced for calculations of top deflection.

Investigations made in the ultimate limit state showed compression stress will not exceed the maximum value. This means overcapacity is available and the core can be additionally loaded. Furthermore in the lower six floors in the ultimate limit state tension will occur and the allowable value will only be overdrawn locally.

Section 3.3 presents the investigations into the columns of the Oval Tower. Conclusion is that the columns which are not subjected to wind load with exception of the inner columns contain overcapacity. The available overcapacity is sufficient to deal with approximately 70% of the value of the total intended extension. It is recommended to not additionally load the columns subjected to wind load.

Section 3.4 presents the investigations with regards to the foundation of the structure. Conclusion can be made that all piles meet the requirements, some of them can be additionally loaded and especially underneath the core there is a high number of overcapacity available. The core provides sufficient capacity for the entire extension to be built. The piles below the columns provide some overcapacity but this is significantly smaller compared to the piles below the core.

Section 3.5 provides insight into the top deflection of the Oval Tower. As presented in this chapter and in section 3.2, no cracks will occur in the concrete of the core, therefore top deflection is calculated without modifications to the modulus of elasticity of the concrete. However top deflection is also calculated with a reduced modulus of elasticity. In both situations the requirements for top deflection will be met and additional top deflection is allowed.

Section 3.6 discusses the dynamical behaviour of the Oval Tower. A value for acceleration is determined with the help of calculated values for the first natural frequency and damping number. The value for the maximum acceleration has been determined with help of measured values for the first natural frequency and damping. For both calculations the determined value for maximum acceleration stayed below the allowed value.

From the conclusions as presented above the research questions can be answered as following:

- A leading structural element in the current design of the Oval Tower is the core. Additional compression is allowed, however tension in the lower part of the core becomes a critical aspect.

- The columns not subjected to wind load with exception of the inner columns contain overcapacity.
- It is not recommended to additionally load the columns subjected to wind load.
- The foundation piles below the core contain enough overcapacity to deal with the extension.
- The foundation piles below the existing columns contain overcapacity.
- For top deflection and dynamical behaviour no problems are expected to occur in case an extension will be build on top of the Oval Tower.

5 Starting point design extension

In the previous chapters the existing situation of the Oval Tower has been taken into account. From this existing situation several conclusions are drawn. Recommendations have been made which elements can be additionally loaded.

In chapter 6 three alternatives for the extension on top of the Oval Tower will be elaborated. First an architectural impression will be given. Afterwards a load bearing structure of the various extensions is made. The concepts are designed with help of various starting points. In this chapter these starting points will be described.

5.1 Capacity existing structure

The first starting point is the current situation of the top of the Oval Tower. An impression of the existing top can be seen in the figure below.

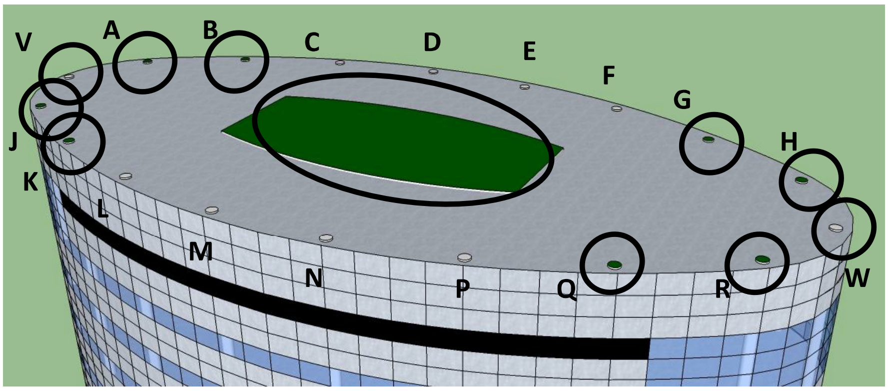


figure 5.1 current situation top floor Oval Tower, circled elements contain overcapacity

In the previous chapter it is stated the core can be additional loaded and the columns A, B, G, H, J, K, Q, R, V and W contain overcapacity. In the designs in the next chapter this overcapacity can be used as a starting point for the extensions.

5.2 Design OPL architects

The second starting point for the architectural design of the extension is the design of OPL architects as described in the introduction of this master thesis. An impression of the crown can be seen in the figure below. Only the shape of the extension has been used as a starting point the functionality windmills and solar cells have been neglected because a boundary condition was a functional extension.

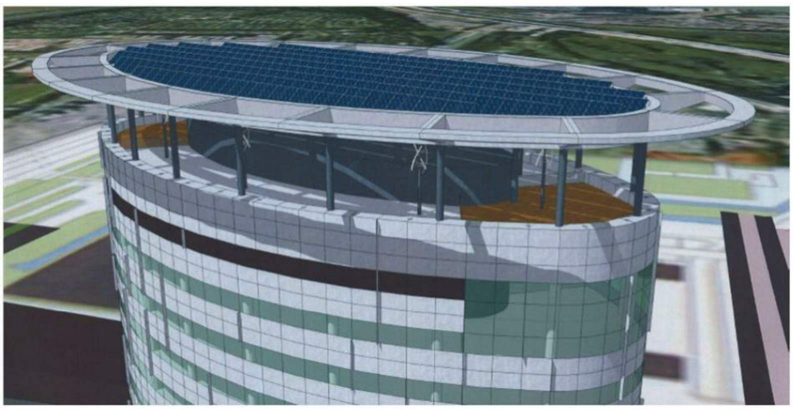


figure 5.2 crown Oval Tower, design OPL architects

5.3 Dimensions extension

The next starting point are the dimensions to which the extension is limited. In the introduction the scope of the thesis has been described. It is stated the top of the Oval Tower will be about 10% of the height of the current building. The extension will exist out of two additional floors and a crown. The total building height is 95,2m. This means the extension will have a height of 9,5m. The total building height is divided as can be seen in the table below.

	Height	Function
Current building		
Ground Floor	3,6m	Entrance
24 typical floors	24 x 3,6m = 86,4m	Offices
Top Floor	5,2m	Structural + technical
Extension		
2 additional floors	2 x 4,0m = 8,0m	Restaurant
Crown	1,5m	Architectural and technical
Total building height	104,7m	

table 5.1 arrangement floors oval tower with extension

5.4 Functionality and environment

On top of the current building of the Oval Tower a small technical extension with a height of 3,3m is present. The function of this extension is a technical one. In the figure below an air picture of the Oval Tower can be seen. It is clearly visible the existing technical extension is present around several air units. In appendix B a technical drawing of the roof of the current Oval Tower is adopted.

figure 5.3 air picture Oval Tower

When the roof floor will be investigated it can be concluded the main ducts and air handling units are present above the core of the existing building. The air handling units has to adjoin the air. Because of the planned extension the air handling units have to be replaced. In figure 5.2 a sketch of the crown is given, the solar cells will be removed. Air handling units now can be placed in these areas. Furthermore free space has to be available above the core to connect the existing ducts with the new air handling units. In figure 5.4 and 5.5 the existing and new situation are given.

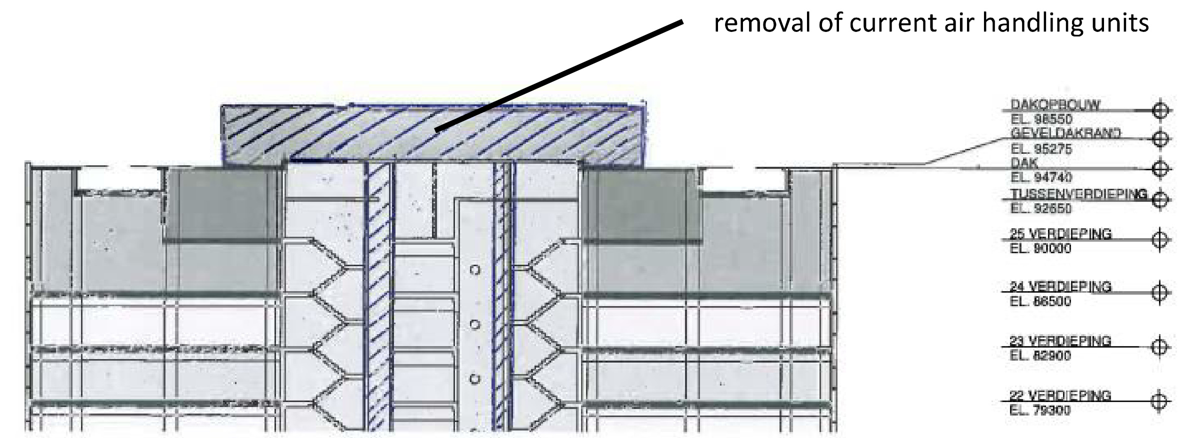


figure 5.4 existing situation ducts and air handling units

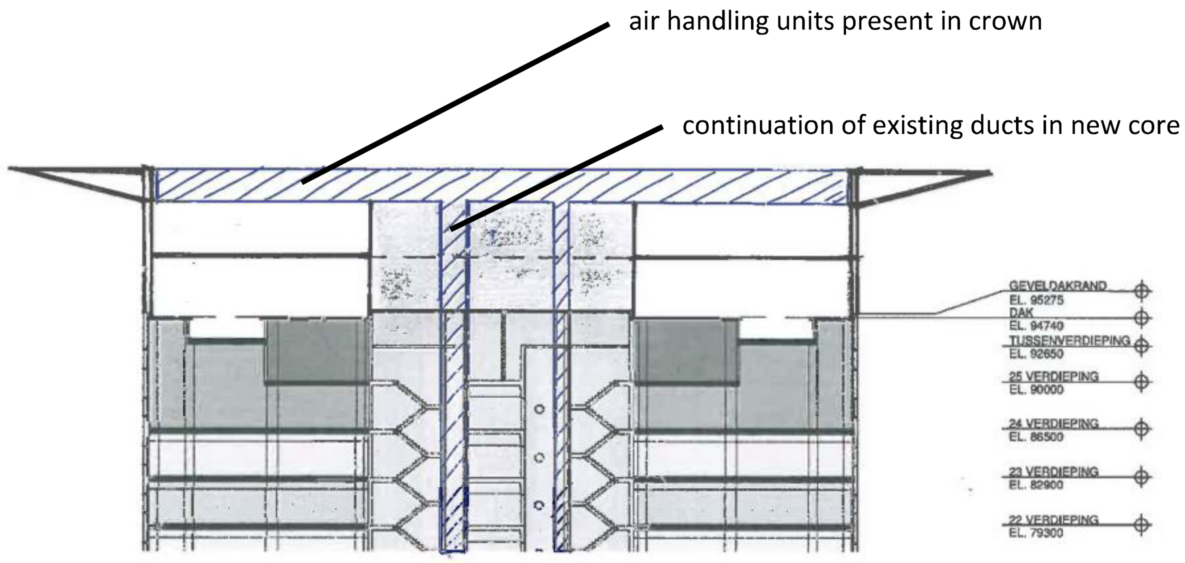


figure 5.5 schematic view new situation ducts and air handling units

Furthermore the Oval Tower is situated in the South-East part of Amsterdam. Below an overview of this area. The Oval Tower is the highest building in this part of Amsterdam.

figure 5.6 environment Oval Tower (left), transfer of restaurant to extension (right)

The view on top of the building will be used as the main basis for the function in the extension. In the current situations a restaurant is situated on the 12^{th} and 13^{th} floor of the Oval Tower. To use the extension for most users of the building, the restaurant will be transferred to the top of the building. The two additional floors on the 12^{th} and 13^{th} floor will be classified as typical office floors. The profit for the building owner is the additional 2000m^2 rentable floor space on the 12^{th} and 13^{th} floor which will be caused by the transfer of the restaurant.

5.5 Link to diagram

The functional factors which determine the feasibility are adopted in the scheme which was composed after the literature study. The part in which the functional factors are determined is admitted to the right.

Functional inventory
- Function extension
- Function existing building
- Appereance extension
- Parking and accesibility building
- Installations and elevators

In this master thesis a choice is made to elaborate the technical factors in detail and to investigate the functional and financial factors more restricted. The factors to the right in the previous sections have been investigated in general. Therefore from here the following conclusion can be drawn.

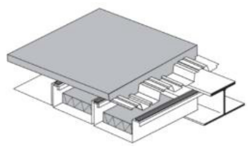
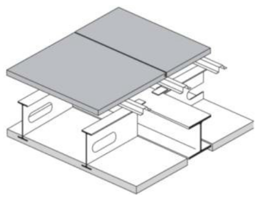
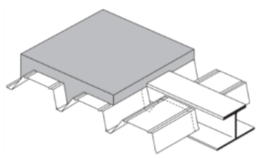
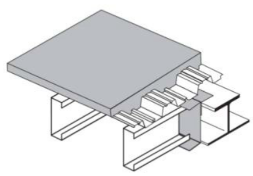
- Additional research throughout the functional inventory is needed to give a thorough answer to the functional feasibility of the extension.

5.6 Materials extension

In [LS] it is recommended steel and timber are suitable building materials to construct an extension. Because the top floor of the existing Oval Tower exists out of steel profiles, the extension will also be build up out of steel profiles. Furthermore the bases of the crown also will be elaborated in steel. The finishing of the crown is non-structural. The weight of the extension is important. The crown therefore will be completed out of a light weight structure, aluminium will be chosen for the finishing of the crown.

In the [LS] a table is made which main load bearing structure can be used best. It is intended to make use of large glass elements for the extension, therefore a steel framing structure will be chosen. According to the [LS] steel framing can be used best for the main load bearing structure.

Another aspect which is determined in the [LS] is the floor system. Five various different floor types are taken into account. It is not desirable to make use of a timber floor system, therefore from this investigation four floor types can still be used for the floors of the extension. These four floor types are the IDES floor, slimline floor, light steel framing floor and composite floor deck. Below a comparison which follows from [LS] is given.

Floor type	Description	Impression	Properties*	Field of application
IDES floor	Girders in one direction, cold formed steel open profiles in other direction, finishing with anhydrite or triplex		80 – 180 kg/m2; 200 – 340 mm; 5,0 – 7,0 m; 60 – 120 min	Buildings in which a thin and very light floor is desired
Slimline floor	Prefabricated concrete lower flange in which steel profiles are poured, openings in secondary girders,		250 – 320 kg/m2; 300 – 500 mm; 5,5 – 11,0 m; 120 min	Buildings in which integration of ducts is desired
Composite floor deck	Floor exists out of steel plates which work constructively works together with the concrete layer. Steel profiles can be stacked during crane movements		260 kg/m2; 330 mm; 4,5 – 9,0m; 60 minutes**	Buildings with a non-uniform floor plan. Buildings in which a high building speed is desired
Light steel framing floor	Secondary girders are cold formed C-profiles, bottom layer will be finished with gypsum board, top layer with an anhydrite layer.		130 – 160 kg/m2; 260 – 400 mm; 3,6 – 7,2 m; 60 minutes**	Buildings in which a thin and very light floor is desired

* an estimation is made for floors on which a live load of 2,5 kN/m2 is working, the following properties are given: mass; thickness; span; fire safety level

** the fire safety capacity can be adjusted to 120 minutes or more with adjustments

table 5.2 floor types usable for extension Oval Tower

Furthermore in the literature study it is concluded the choice in floor type also depends on the architectural design. The choice for the floor system therefore will be made according to the design and with help of the specifications given in the table above.

As described before for the façade, glass elements will be used. In the current Oval Tower glass elements of the firm Alcoa are used. These elements consist out of glass in an aluminium frame. In the figures below two impressions of these elements are given. It can be seen the amount of glass compared to the frame is big. Furthermore the elements can be easily placed under an angle which is desirable for the design. The elements can be pre-assembled and only have to be attached at the building site.[9] These advantages make the glass elements a suitable option for the façade of the extension.

figure 5.7 impressions glass elements

In the [WSC OT] a weight of 0,50 kN/m2 is used for the glass panels. In figure 5.8 left the main spans of the design of the extension are given. The height between the floors is 4m, the distance between the elements is 1,2m. In the existing situation the biggest vertical span (as can be seen in figure 5.8 left) is 4,8m and the span between the elements is also 1,2m. From here it can be concluded the same elements can be used as in the existing situation. The weight of these elements is 0,50 kN/m2.

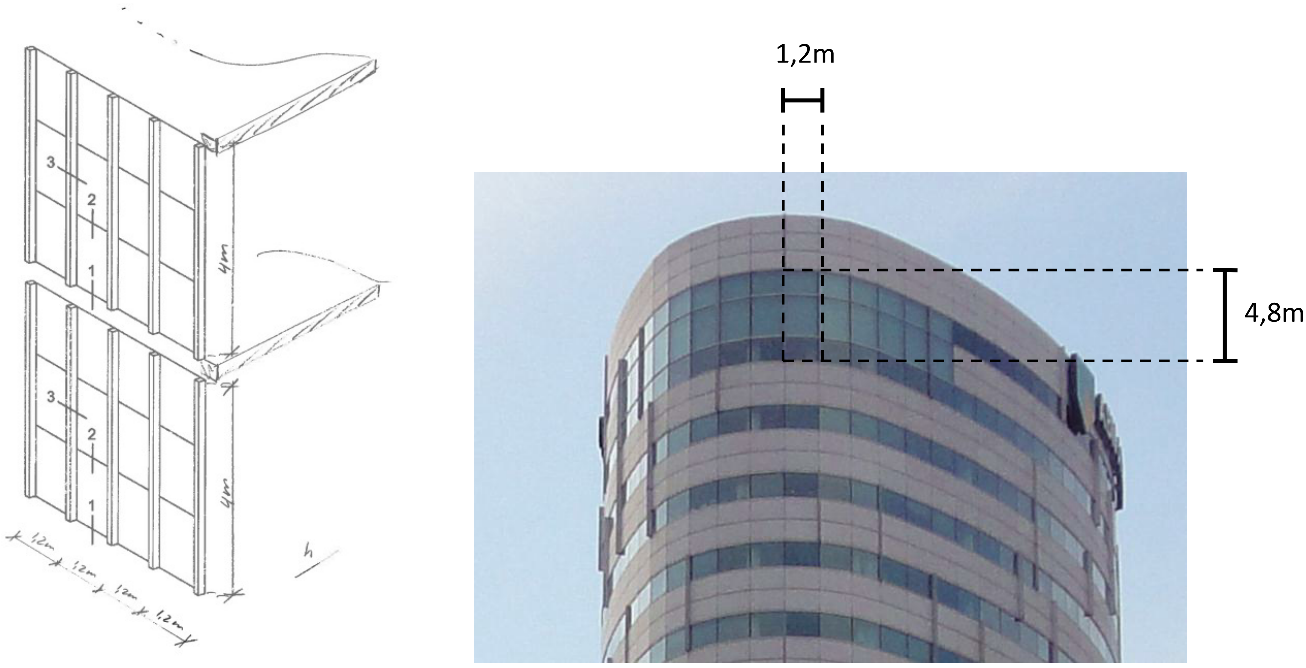


figure 5.8 main dimensions extension (left) existing top floors (right)

[9] (Alcoa, Gevelsysteem AA100Q, 2012)

For the walls which are present in the building a light weight system is preferred. A company which is specialized in light weight wall systems is Gyproc.[10] The partition walls which are fabricated by this company are build up from gypsum board combined with mineral wool. Light weight metal steel frames form the base for these structures. On the website of Gyproc with help of various demands a choice for a type of wall can be made. First the fire safety will be asked. Because the extension is situated on top of a high-rise structure the fire safety will be chosen as 120min. The next demand is the impact resistance. A restaurant is situated in the extension therefore a maximum impact resistance will be chosen. The height will be chosen as 4m (as determined in the previous section). Because of the restaurant function the moisture content will be chosen as raised. This leads to a Gyproc Soundbloc element. Technical specifications of these partition walls can be seen in the figure below.

Systeemcode	AS 150 dB RD/3.75.3.A
Geluidsisolatie R_w	62 dB
Brandwerendheid	120 minuten
Wandhoogte toepassingsgebied I	6000 mm
Wandhoogte toepassingsgebied II	5500 mm
Stootvastheid	Klasse 6
Wanddikte	150 mm
Beplating	1x Rigidur 12,5 2x Gyproc dB 12,5
Profielen	1x AcouStud 75
Minerale wol	1x 60 mm Isover glaswol
Gewicht	70 kg/m^2

figure 5.9 partition walls (left) technical specification (right)

An overview of building materials which will be used in the extension is given below.

Element	*Building material*
Crown	Steel structure with aluminum finishing
Load bearing structure	Steel structure
Floor system	According to design extension
Façade	Alcoa glass elements
Partition walls	Gyproc soundbloc elements

table 5.3 building materials elements crown

[10] (Gyproc, 2012)

5.7 Loads extension

5.7.1 Dead load

The dead load of the main load bearing structure and the structural elements of the crown will be determined by the amount of steel which will be used. The finishing of the crown will be accomplished with aluminium profiles. The weight of steel is 7850 kg/m^3 and for aluminium 2768 kg/m^3.

The dead load for the floor system depends on the choice, which will be made according to the design. The mass of the various floor systems is given in table 5.2. The dead load for the roof floor also depends on the design. However various assumptions can be made with help of the [WSC OT]. As stated in this calculations for finishing and ducts a value of 1,5 kN/m^2 can be assumed. This also counts for the roof floor. For the installations an additional load of 1,65 kN/m^2 can be assumed, for the existing Oval Tower this load was only present around the core. In the new situation this loads will be present over the entire roof floor.

The dead load of the façade elements will be chosen according to the initial calculations made in the [WSC OT] and has a value of 0,50 kN/m^2.

Partition walls: The partition walls in the various designs will be chosen according to the data given in figure 5.9. The dead load of the walls becomes (70 * 9,81) / 1000 = 0,69 kN/m^2.

5.7.2 Live load

In the table below an overview of the live loads for the existing floors and the planned additional floors. In the new situation it can be seen no adjustments have to be made to the structure on the 24th and 25th floor. However the load on the 26th floor is increased. Adjustments or an additional new structure has to be placed on the 26th floor.

Floor number	Existing situation		New situation	
	Function	Load	Function	Load
Top floor	-	-	Roof	1,0 kN/m^2 (ψ=0)
27th	-	-	Restaurant	4,0 kN/m^2 (ψ=0,5)
26th	Roof	1,0 kN/m^2 (ψ=0)	Restaurant	4,0 kN/m^2 (ψ=0,5)
25th	Office	2,5 kN/m^2 (ψ=0,5)	Office	2,5 kN/m^2 (ψ=0,5)
24th	Office	2,5 kN/m^2 (ψ=0,5)	Office	2,5 kN/m^2 (ψ=0,5)

table 5.4 live loads

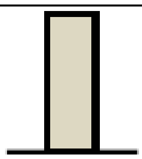
5.7.3 Wind load

In the [LS] research has been done throughout the horizontal loads which have to be applied to an extended high-rise structure. The wind load depends on the design of the extension and will be calculated for the various alternatives in the next chapter. Furthermore in the [LS] methods are given in which the shape of the extension can be adjusted to reduce the wind loads. A sculptured building top, varying the shape of the building, openings in the top of the building, corner modification and orientation of the building in relation to the leading wind direction are possibilities.

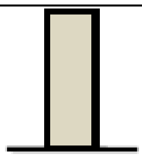
5.7.4 Load combinations

In the [LS] an elaborated study has been made in which three situation are described how the load combinations can be taken into account. In the literature study the tables below are provided.

- Situation I: Interpret total building (including extension) as rebuilding's.

Interpret as		Dead load Disadvantuous	Dead load Advantageous	Leading variable load	Remaining variable load
Rebuilding	6.10a 6.10b 	1,4 1,25	0,9 0,9	1,6 ψ_0 1,6	1,5 ψ_0 1,5 ψ_0
Rebuilding (values according to NEN6700)		1,2	0,9	1,5	1,5 ψ_0

- Situation II: Interpret extension as new building, interpret existing building as rebuilding.

Interpret as		Dead load Disadvantuous	Dead load Advantageous	Leading variable load	Remaining variable load
New building	6.10a 6.10b 	1,5 1,3	0,9 0,9	1,65 ψ_0 1,65	1,65 ψ_0 1,65 ψ_0
Rebuilding (values according to NEN6700)		1,2	0,9	1,5	1,5 ψ_0

- Situation III: Interpret total building (including extension) as new building.

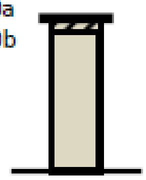
Interpret as		Dead load Disadvantuous	Dead load Advantageous	Leading variable load	Remaining variable load
New building	6.10a 6.10b 	1,5 1,3	0,9 0,9	1,65 ψ_0 1,65	1,65 ψ_0 1,65 ψ_0

figure 5.10 load factors three situations

The desired safety level is the third option. However in case this notion leads to disproportionally high costs the options as presented for situation I and II may be fulfilled. To define whether interpretation as presented in situation III leads to high costs the columns and foundation below the columns is checked. This has been done for the building without extension.

In appendix E these calculations are provided. It can be concluded that interpreting the Oval Tower with demands for new buildings, leads to the transgression of allowed values in at least five various building columns and three foundation groups below columns. Furthermore eleven columns and three foundation groups become critical. Because calculations for the Oval Tower are made without extension it can be assumed with extension more building elements will have to be strengthened which leads to additional costs.

From here it will be assumed these costs are disproportionally high and it is chosen to take the second situation into account. In the second situation the existing building will be interpreted as rebuilding, the load factors are equal to the calculations which have been made of the current building. The extension will be interpreted as a new building.

The combinations in the scheme above are combined. The load combinations in the table on the next page have to be taken into account. The ψ0 factor for variable floor load on the 26th and 27th floor is 0,5. The ψ0 factor for wind load is 0. With this factors combined several load cases will expire. In table 5.6 the load cases which do have to be taken into account are given. In total there are 20 load combinations. Furthermore it will be checked whether reduction can be made to the load factors. In the [LS] for reduction of to load factors the following has been concluded.

ψ is de factor voor de bepaling van de momentane belastingswaarde, zoals gespecificeerd in hoofdstuk 8;

t is de referentieperiode voor de bepaling van de reductiefactor voor de extreme waarde van de gelijkmatig verdeelde belasting, in jaren, volgens 5.5.1 met een minimum van:

— veiligheidsklasse 1: $t \geq 1$ jaar;

— veiligheidsklassen 2 en 3: $t \geq 15$ jaar.

$$\psi_t = 1 + \left(\frac{1-\psi}{9}\right) \times \ln\left(\frac{t}{t_{50}}\right), \text{ met } t_{50} = 50 \text{ jaar}$$

figure 5.11 reduction factor according to NEN6702

With help of the equation above a reduction factor can be calculated. It is stated that highrise projects can be classified in safety class 3. For safety class 3 a minimum of t ≥ 15 years is valid. The year of delivery of the Oval Tower as presented in the introduction was 2001. At moment of writing it is 2012 which means in this particular case t = 2012-2001=11 < 15 years. From here it can be concluded no reduction to the load factors may be applied to the Oval Tower.

BC		Dead load		Live load		Wind load cross direction		Wind load long direction	
		OT	ext	OT	ext	OT	ext	OT	ext
	SLS								
BC1		1,0		1,0*ψ0		1,0			
BC2		1,0		1,0*ψ0				1,0	
BC3		1,0		1,0		1,0*ψ0			
BC4		1,0		1,0				1,0*ψ0	
BC5		1,0				1,0			
BC6		1,0						1,0	
	ULS								
BC7	str/geo (6.10a)	1,2	1,5	1,5*ψ0	1,65*ψ0	1,5*ψ0	1,65*ψ0		
BC8	str/geo (6.10a)	1,2	1,5	1,5*ψ0	1,65*ψ0			1,5*ψ0	1,65*ψ0
BC9	str/geo (6.10a)	0,9		1,5*ψ0	1,65*ψ0	1,5*ψ0	1,65*ψ0		
BC10	str/geo (6.10a)	0,9		1,5*ψ0	1,65*ψ0			1,5*ψ0	1,65*ψ0
BC11	str/geo (6.10b)	1,2	1,3	1,5	1,65	1,5*ψ0	1,65*ψ0		
BC12	str/geo (6.10b)	1,2	1,3	1,5	1,65			1,5*ψ0	1,65*ψ0
BC13	str/geo (6.10b)	0,9		1,5	1,65	1,5*ψ0	1,65*ψ0		
BC14	str/geo (6.10b)	0,9		1,5	1,65			1,5*ψ0	1,65*ψ0
BC15	str/geo (6.10b)	1,2	1,3	1,5*ψ0	1,65*ψ0	1,5	1,65		
BC16	str/geo (6.10b)	1,2	1,3	1,5*ψ0	1,65*ψ0			1,5	1,65
BC17	str/geo (6.10b)	0,9		1,5*ψ0	1,65*ψ0	1,5	1,65		
BC18	str/geo (6.10b)	0,9		1,5*ψ0	1,65*ψ0			1,5	1,65
BC19	str/geo (6.10b)	0,9				1,5	1,65		
BC20	str/geo (6.10b)	0,9						1,5	1,65

table 5.5 load combinations existing building and extension; (OT = Oval Tower; ext = extension)

BC		Dead load		Live load		Wind load cross direction		Wind load long direction	
		OT	ext	OT	ext	OT	ext	OT	ext
	SLS								
BC1		1,0		0,5		1,0			
BC2		1,0		0,5				1,0	
BC3		1,0		1,0		0			
BC4		1,0		1,0				0	
BC5		1,0				1,0			
BC6		1,0						1,0	
	ULS								
BC7	str/geo (6.10a)	1,2	1,5	0,75	0,83	0	0		
BC8	str/geo (6.10a)	1,2	1,5	0,75	0,83			0	0
BC9	str/geo (6.10a)	0,9		0,75	0,83	0	0		
BC10	str/geo (6.10a)	0,9		0,75	0,83			0	0
BC11	str/geo (6.10b)	1,2	1,3	1,5	1,65	0	0		
BC12	str/geo (6.10b)	1,2	1,3	1,5	1,65			0	0
BC13	str/geo (6.10b)	0,9		1,5	1,65	0	0		
BC14	str/geo (6.10b)	0,9		1,5	1,65			0	0
BC15	str/geo (6.10b)	1,2	1,3	0,75	0,83	1,5	1,65		
BC16	str/geo (6.10b)	1,2	1,3	0,75	0,83			1,5	1,65
BC17	str/geo (6.10b)	0,9		0,75	0,83	1,5	1,65		
BC18	str/geo (6.10b)	0,9		0,75	0,83			1,5	1,65
BC19	str/geo (6.10b)	0,9				1,5	1,65		
BC20	str/geo (6.10b)	0,9						1,5	1,65

table 5.6 load combinations existing building and extension including ψ0 factors; (OT = Oval Tower; ext = extension)

BC4 will expire because of equality with BC3. BC8, BC9 and BC10 will expire because the result in these combinations will always be lower compared to the results of BC7. BC12 will expire because of equality with BC11. BC13 and BC14 will expire because these combinations will always give lower values compared to BC11. This also counts for BC17 and BC18 in combination with BC15 and BC16. This leads to the load combinations which are given in the table below.

BC		Dead load		Live load		Wind load cross direction		Wind load long direction	
		OT	ext	OT	ext	OT	ext	OT	ext
	SLS								
BC1		1,0		0,5		1,0			
BC2		1,0		0,5				1,0	
BC3		1,0		1,0					
BC5		1,0				1,0			
BC6		1,0						1,0	
	ULS								
BC7	str/geo (6.10a)	1,2	1,5	0,75	0,83				
BC11	str/geo (6.10b)	1,2	1,3	1,5	1,65				
BC15	str/geo (6.10b)	1,2	1,3	0,75	0,83	1,5	1,65		
BC16	str/geo (6.10b)	1,2	1,3	0,75	0,83			1,5	1,65
BC19	str/geo (6.10b)	0,9				1,5	1,65		
BC20	str/geo (6.10b)	0,9						1,5	1,65

table 5.7 final load combinations existing building and extension including ψ0 factor; (OT = Oval Tower; ext = extension)

Furthermore wind from the opposite direction also has to be taken into account. This can be done by multiplying the load combinations in which wind load is present with -1,0. Eight new load combinations arise which gives a total of 17 load combinations.

5.7.5 Overview loads extension

In the table below an overview of the loads which will be applied in the design on top of the Oval Tower. The * in the table means the dead load of the floor will be determined by the design of the extension.

Element	Self-weight (kN/m3)	Dead load (kN/m2)	Live load (kN/m2)
Main load bearing structure (steel)	78,5		
Structural elements crown (steel)	78,5		
Finishing elements crown (aluminium)	27,7		
Restaurant floors			
26th and 27th		*	4,0 (ψ=0,5)
Finishing layer and ducts		1,50	
Top Floor		*	1,0 (ψ=0)
Finishing layer and ducts		1,50	
Installations		1,65	
Partition walls		0,69	
Façade elements		0,50	

table 5.8 overview loads extension

Furthermore the wind load will be determined by the design. To reduce the wind load the design can be adopted in several ways.

When the extension is situated on top of the Oval Tower, the existing building can be interpreted as rebuilding. The extension on top of the building can be interpreted as a new building. A total of 17 load combinations will arise which are given in table 5.7.

6 Three alternatives extension

6.1 Introduction

In this chapter three alternatives for the extension will be taken into account. First the architectural design will be explained. The three alternatives have several identical structural components , these components will be calculated. Afterwards the structural design of the alternatives will be taken into account separately. With help of these findings the vertical and horizontal reaction forces will be determined and compared with the overcapacity in the current building. With help of these findings conclusions, recommendations and a choice for the final alternative will be made. This alternative will be elaborated in detail in the next chapter.

6.2 Architectural design

6.2.1 Alternative A

For alternative A the two additional floors are extended as typical floors. The two floors will be built in the same shape the typical floors of the existing building are build. The only difference is that instead of the concrete façade elements, a glass façade will be used over the full height of the extension, this has been done to catch the view of the environment in the most optimal way. An architectural impression is given in the figure below.

figure 6.1 impression alternative A

Inside the extension the floors will differ from the typical floors. On the axis C till F and L till P a void will be designed. The void has been designed to not additional load the columns on these axis. The void can be seen in the figure below. The top floor of the existing Oval Tower is designed on a live load of 1,0 kN/m2. The new variable floor load will be 4,0 kN/m2 which means the top floor does not meet the various requirements. Therefore the first step is the design of a beam grid which will transfer the forces to the core and partly to the columns A, B, G, H, J, K, Q, R, V and W. Later on the sizes of the beams which will be applied in the grid will be determined. The place in which the beam grid will be situated can be seen in the left figure below.

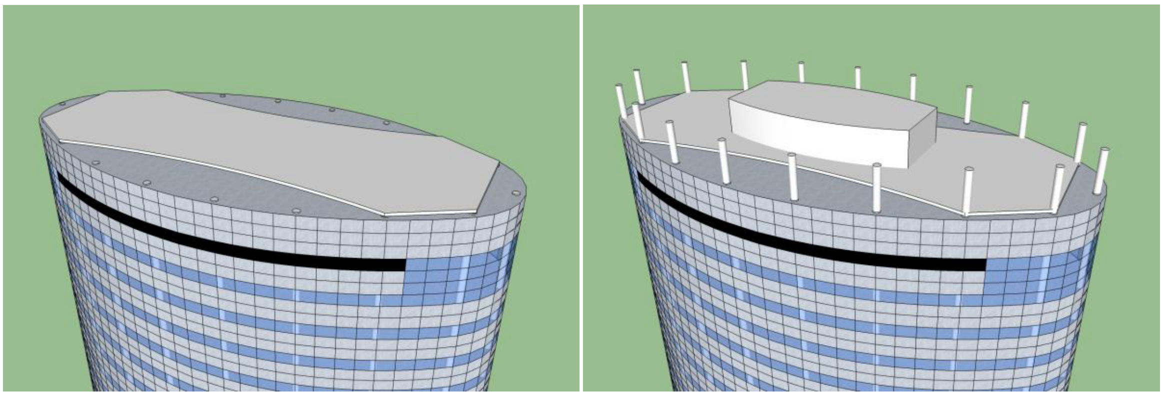


figure 6.2 area beam grid (left) structural elements (right) alternative A

The second floor of the extension will continue in the same way as the first floor. The core will continue and so are the columns. The columns on the axis C till F and L till P will only be additional loaded because of the dead weight of the glass façade elements. These forces are small in comparison with the forces because of additional live load.

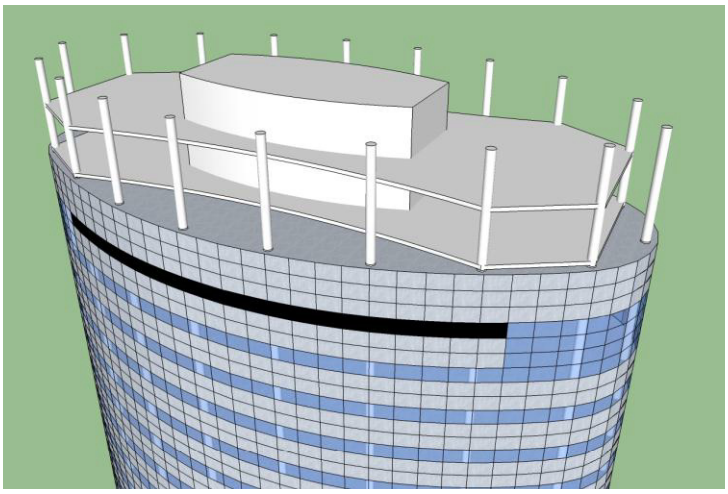


figure 6.3 second floor alternative A

On top of the extension the crown will be situated. Inside the crown the ducts and technical installation which are connected with the core will be situated.

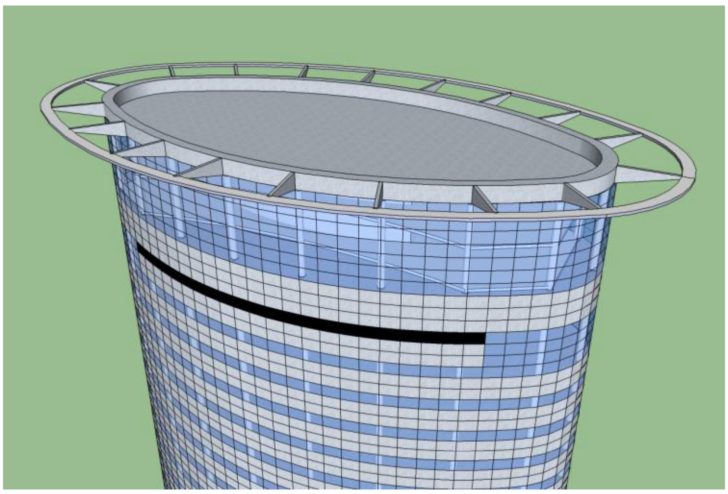


figure 6.4 extension alternative A

6.2.2 Alternative B

The second concept will be based on the reduction of wind forces. The building has a sculptured building top which reduced the wind loads working on the building. Before it has already been concluded the cross direction is the leading direction. In concept B the design will be made in a way the wind load working in the cross direction will be decreased. This will be done by slightly turning the two top floors and the crown. An impression of this concept in the figure below.

figure 6.5 impression alternative B

Because of the live load which cannot be increased and because of the cantilever which will occur, here also a steel beam grid will be designed. The first floor will be slightly twisted. The columns on axis A, B, Q and R can be additional loaded because of the design. Furthermore a steel structure will be placed above the concrete core which has to transfer the rest of the loads.

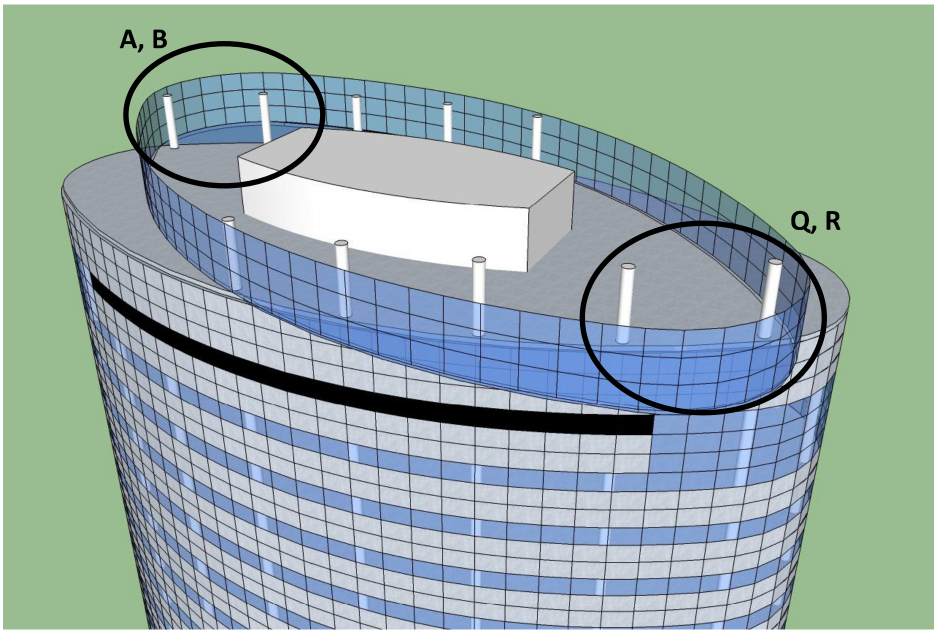


figure 6.6 alternative B, 1st floor

The second floor again will be twisted. The columns on axis B and Q contain overcapacity. For the structural design this has to be taken into account. The structure has to be made in a way additional forces will be transferred via those columns.

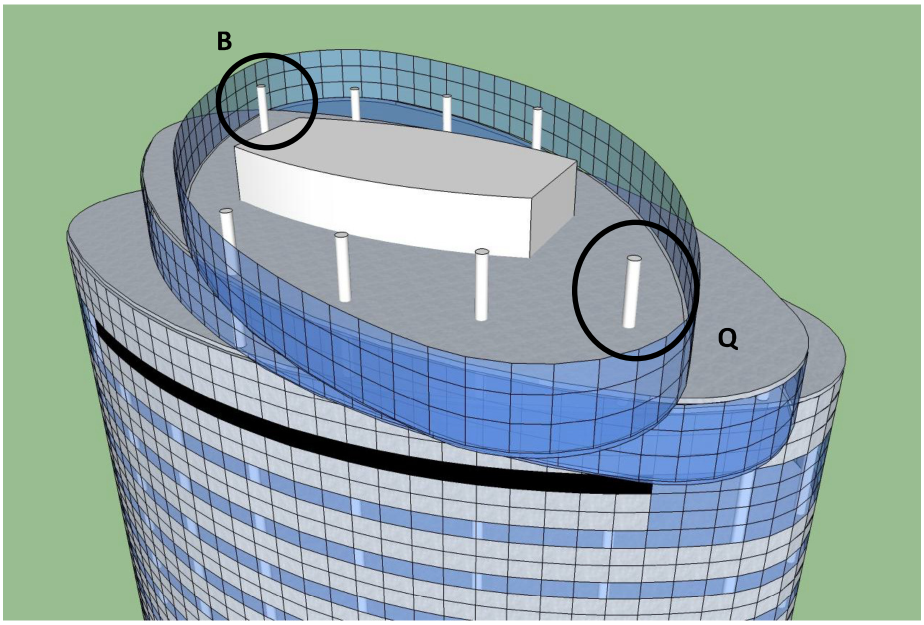


figure 6.7 alternative B, 2nd floor

With the crown the building will look like the figure below. Again the crown can be used for the technical installations.

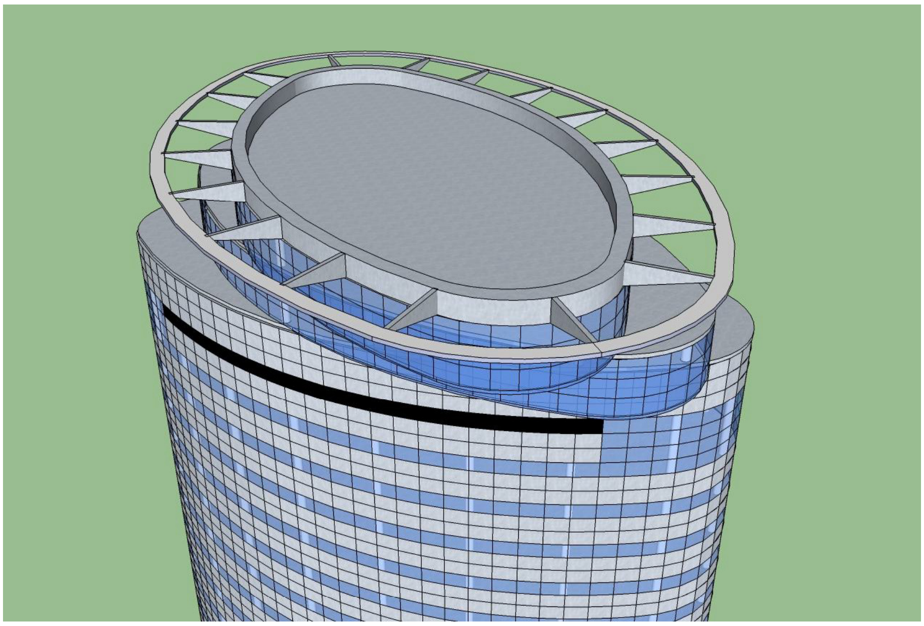


figure 6.8 alterantive B final extension

6.2.3 Alternative C

In the third concept the design is based on the reduction of wind load by making an opening at the top of the building. Because of the opening two individual extensions can be distinguished which are connected by the crown. An impression of this concept can be seen in the figure below.

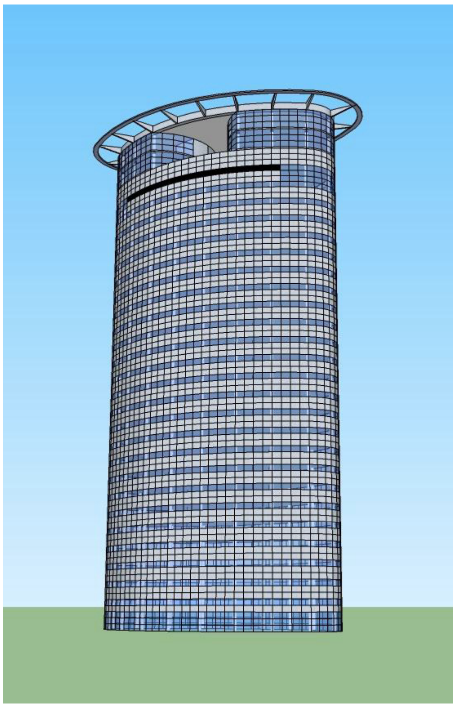


figure 6.9 impression alternative C

The opening is directly situated above the current core. Most of the loads have to be transferred via the core. Again for this configuration a beam grid will be situated on top of the top floor of the current building. The area in which the beam grid can be situated is drawn in the figure below.

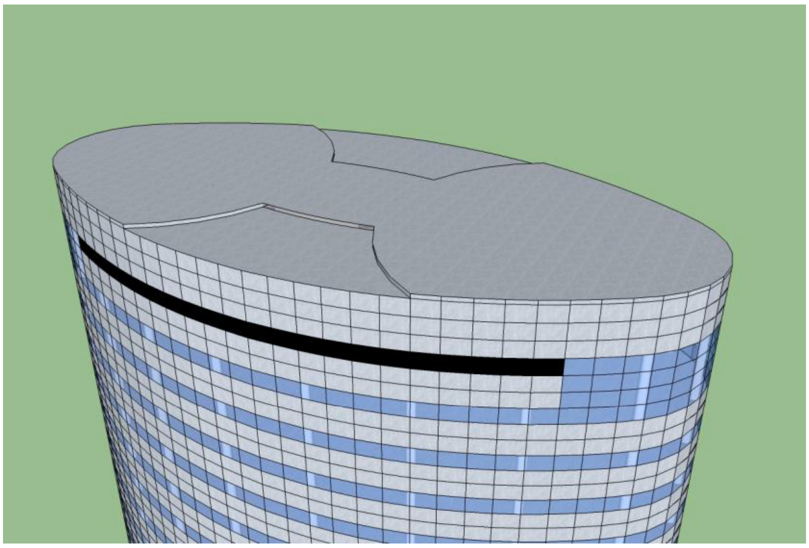


figure 6.10 area beam grid alternative C

The first floor is build up out of two steel structures. Both structures are placed directly above the core. The loads because of the extension has to be transferred to the core via this structure and partly via the façade columns which contain overcapacity. The second floor is build up the same as the first floor. Both floors can be seen in the figure below.

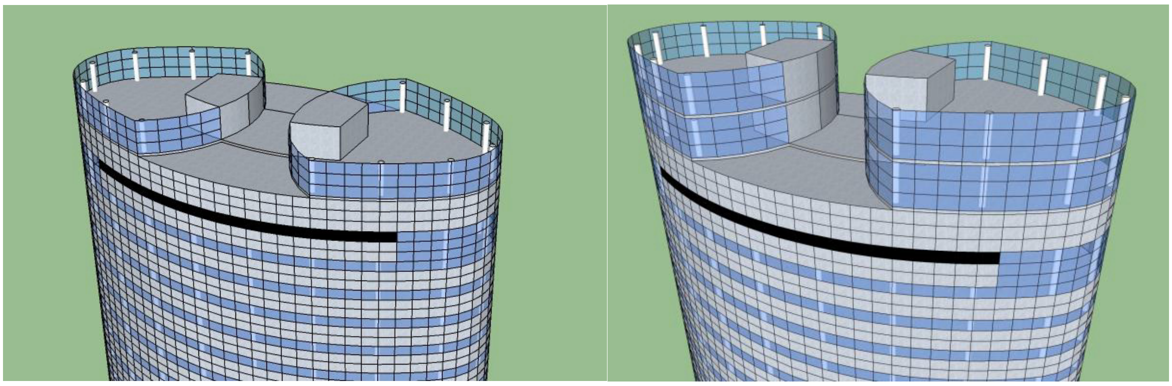


figure 6.11 first floor (left) second floor (right) alternative C

The final crown can be used for technical installations and is drawn in the figure below.

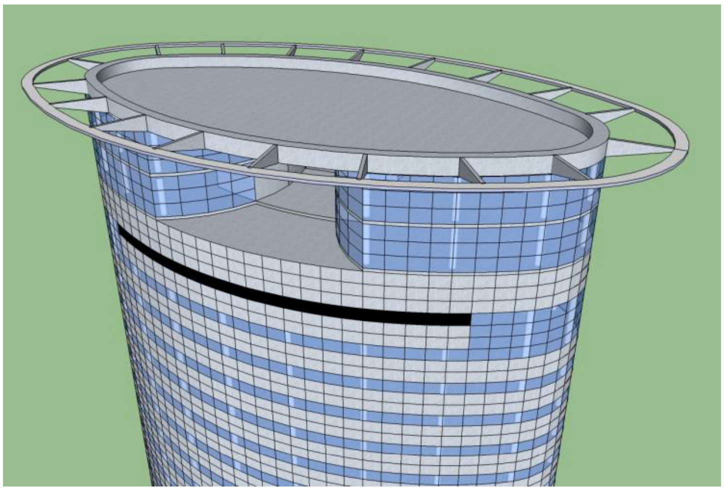


figure 6.12 crown alternative C

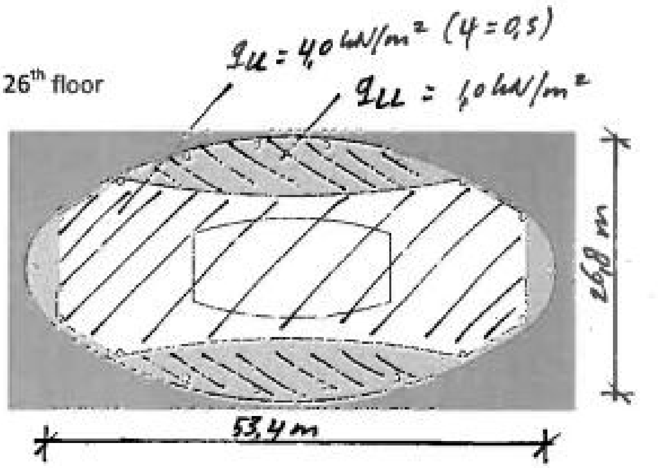
6.3 Dimensions and loads

In this chapter the main dimensions of the three alternatives are given. Furthermore the live load which have to be applied to the various alternatives is adopted to the drawings.

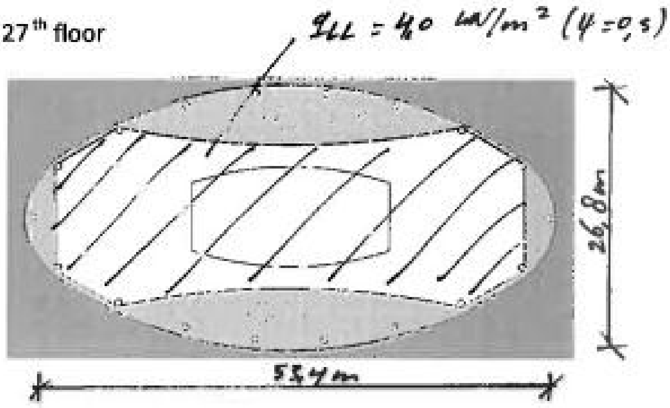
6.3.1 Alternative A

In the figures below a top view of the 26th, 27th and top floor can be seen. Furthermore a front and side view of the alternative is admitted. In the drawings of the top views the various live loads which have to be applied are given and the main dimensions are given.

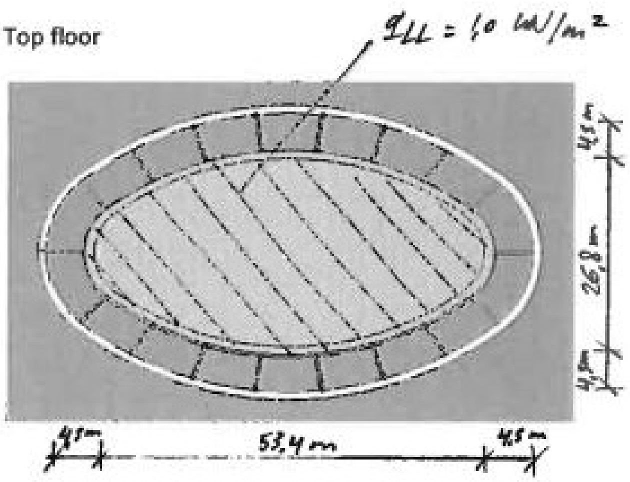
26th floor

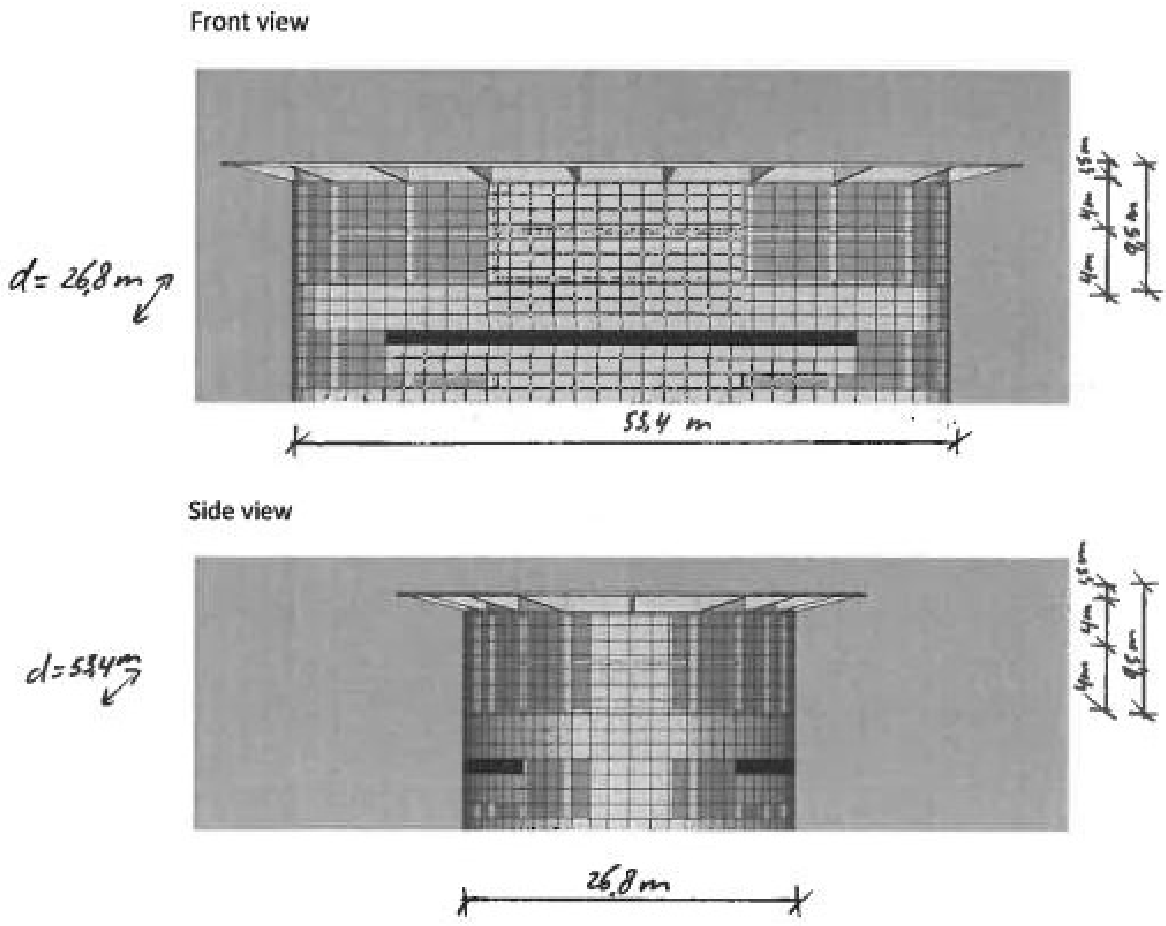


27th floor

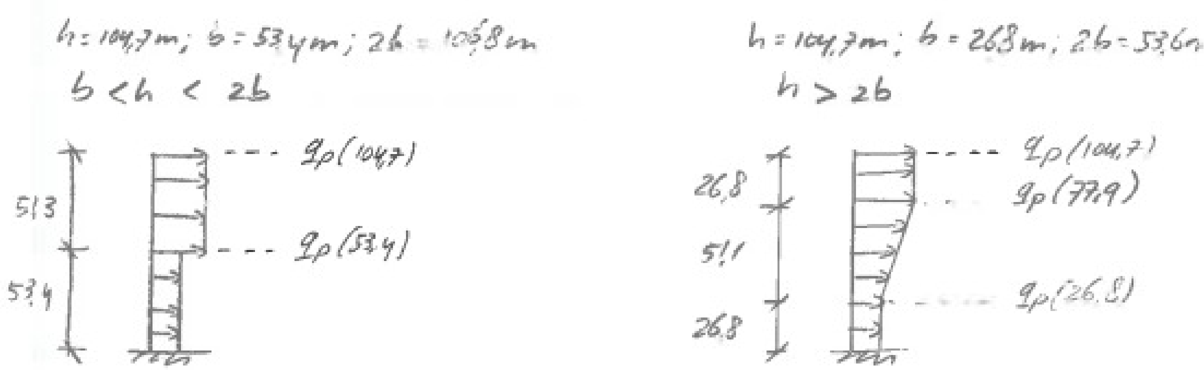


Top floor





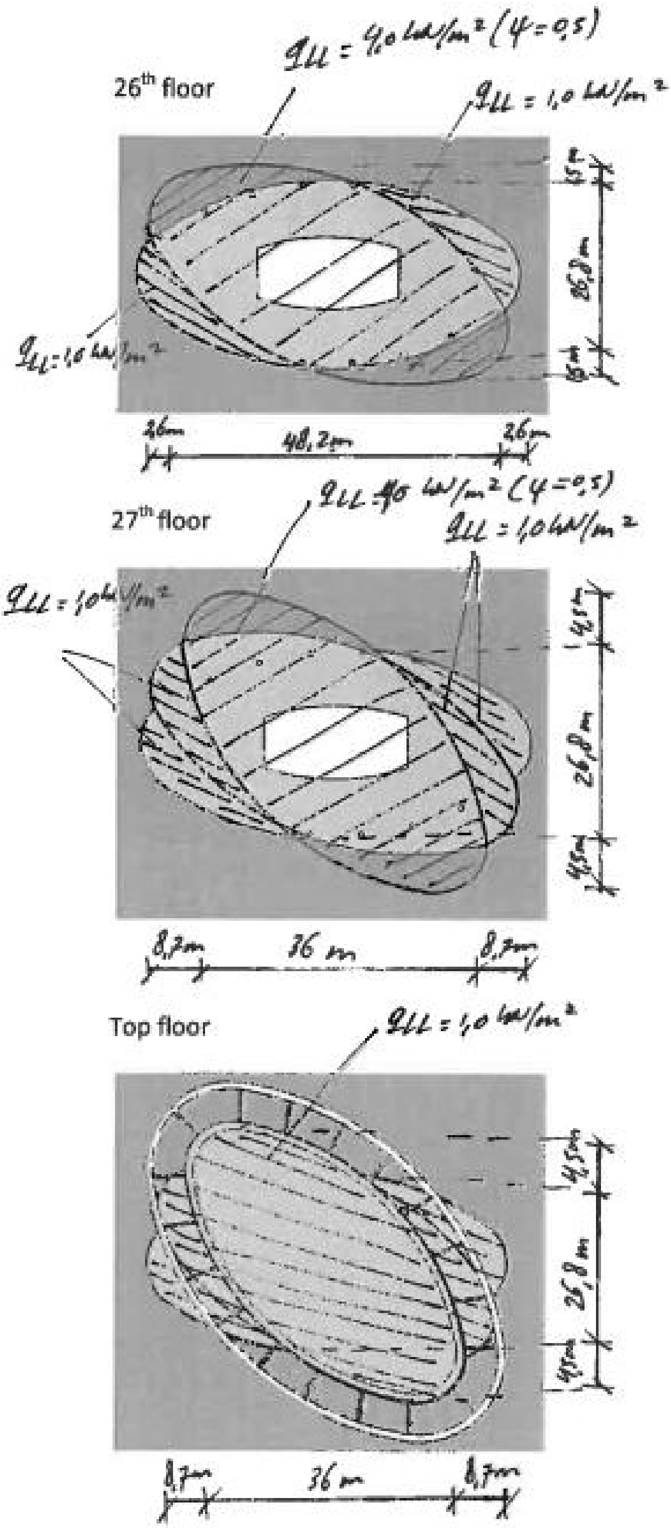
In appendix F the wind load is calculated. The final wind loads can be seen in the figure below.

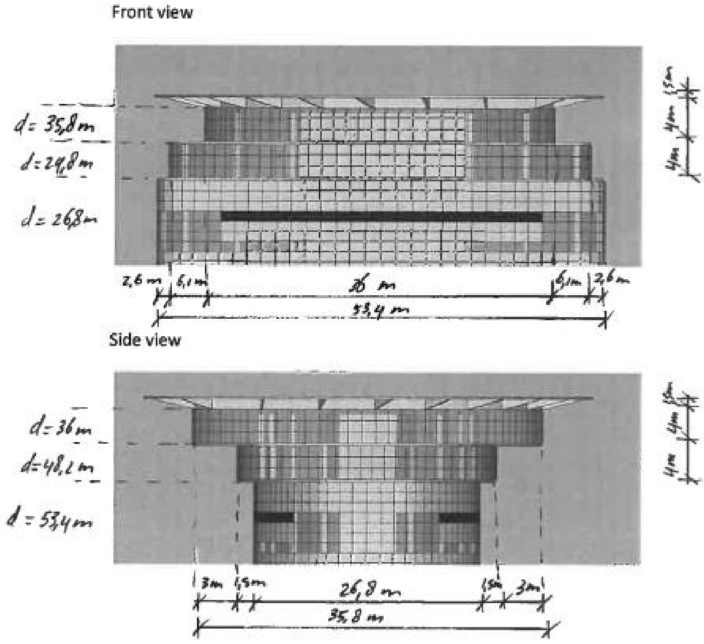


h (m)	wind cross (kN/m2)	h (m)	Wind long (kN/m2)
104,7	2,14	104,7	1,95
53,4	1,76	26,8	1,29

6.3.2 Alternative B

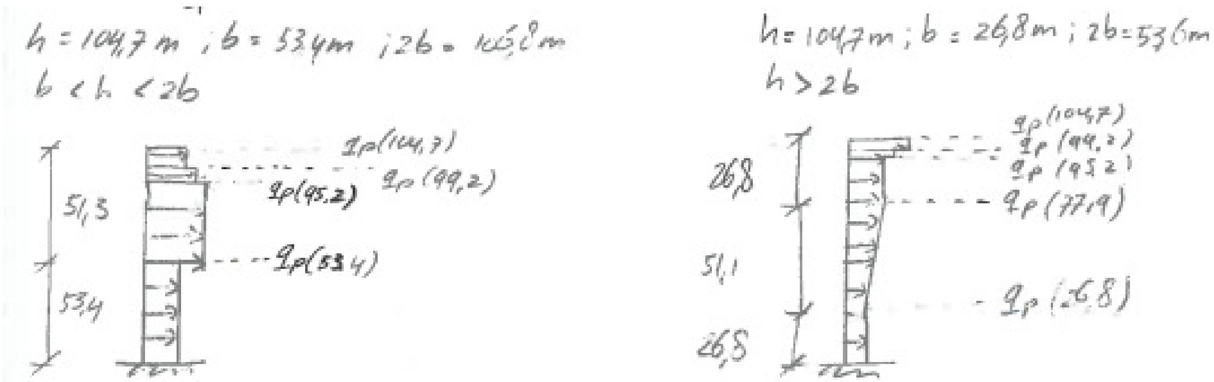
In the figures below a top view of the 26th, 27th and top floor of alternative B can be seen. Furthermore a front and side view of the alternative is adopted. In the drawings the live loads can be seen.





Because of the twisted extension a cantilever will arise. The maximum dimension of the cantilevered floor span is 4,5m. In cross direction the width of the building becomes 8,7m less on both sides.

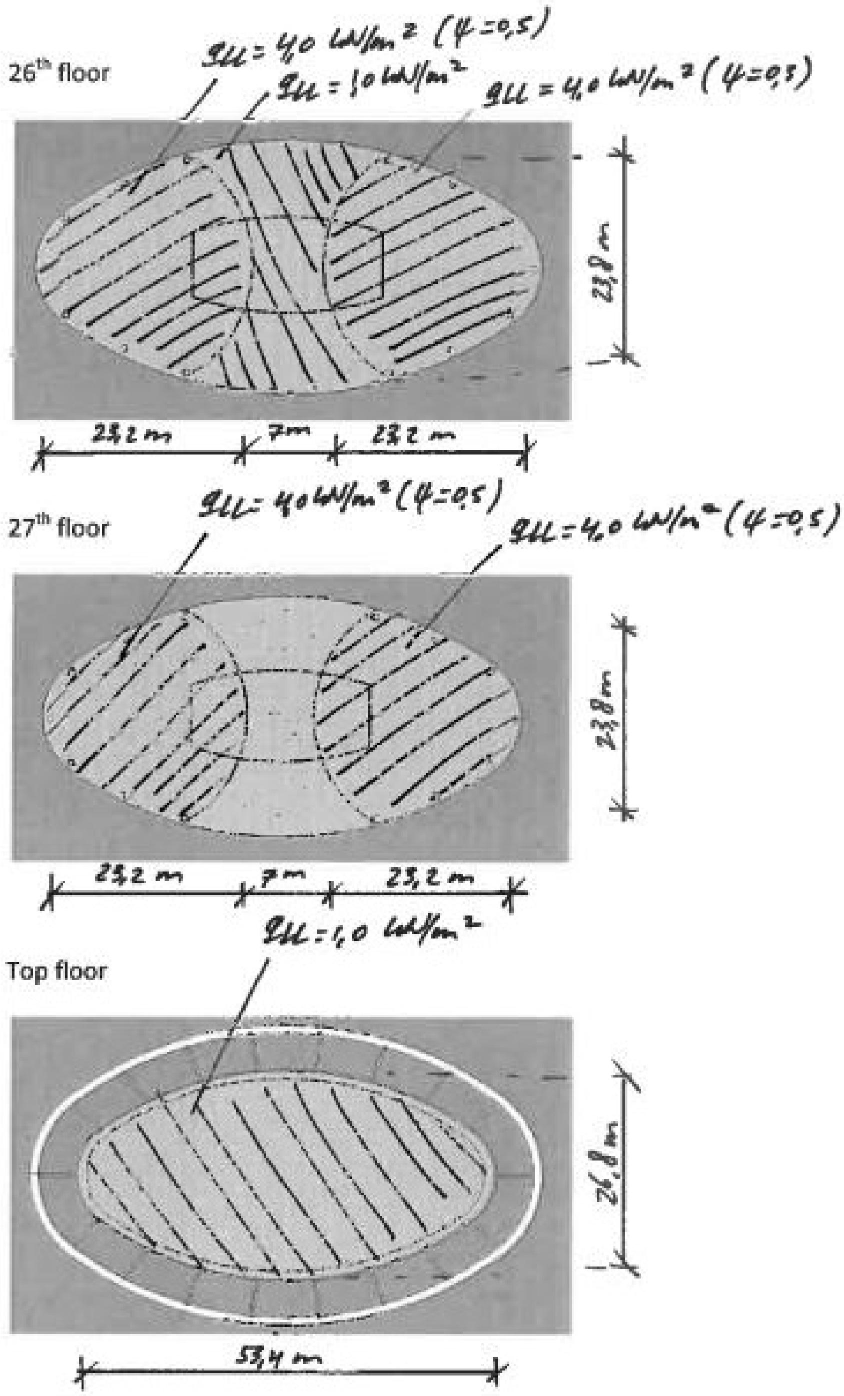
The wind load is calculated in appendix E, the results are summarized in the figure and table below. It is clearly visible the wind load in cross direction will be reduced because of the design. In longitudinal direction additional wind load has to be applied.

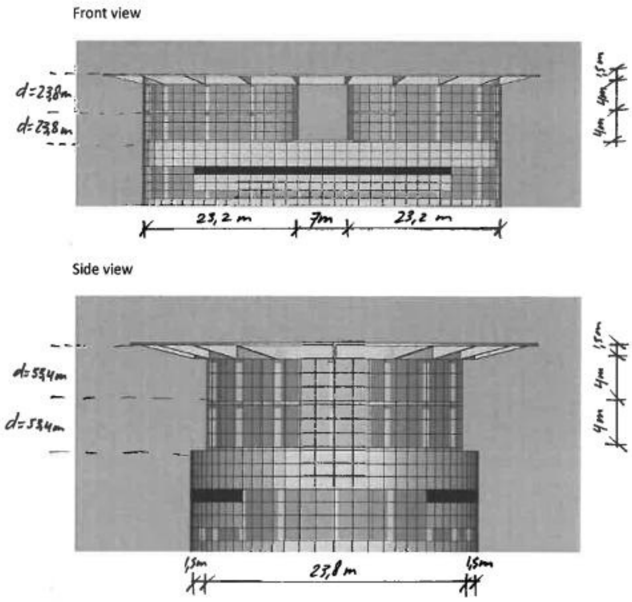


h (m)	wind cross (kN/m2)	h (m)	Wind long (kN/m2)
104,7	2,04	104,7	2,03
99,2	2,10	99,2	1,97
95,2	2,14	95,2	1,95
53,4	1,76	26,8	1,29

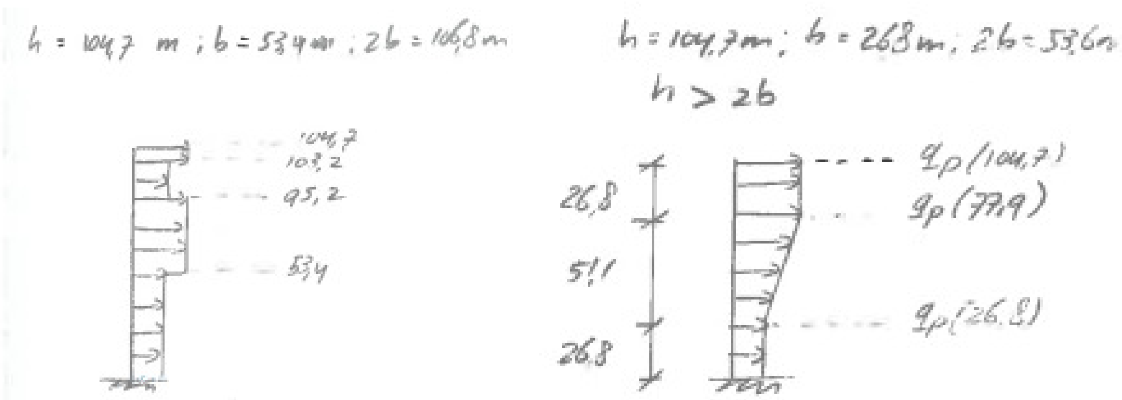
6.3.3 Alternative C

In the figures below the horizontal cross section and top view of the crown from alternative C can be seen. Furthermore a side view and a front view are adopted.





For the wind load will be referred to appendix F. A summary can be seen in the figure and table below.



h (m)	wind cross (kN/m2)	h (m)	Wind long (kN/m2)
104,7	2,14	104,7	1,95
103,2	2,29	26,8	1,29
95,2	2,09		
53,4	1,76		

The wind load which have to be applied to the part from 95,2 till 103,2m (the part with the opening) has a value of 2,29 kN/m2 which is higher compared to the value above and below this part. However the width of the building is smaller (2*23,2m < 53,4m) which means the wind load for this part of the building has a smaller value compared to the part below and above the opening.

6.4 General structural components

In the section before three architectural designs are described. The three alternatives are based on the existing Oval Tower. Therefore for several components of the extension the dimensions can be chosen equal. In this section an estimation will be made for various elements. The calculations are a first estimation and are used for the following components: crown, core, façade columns and floor structure.

The calculation of the elements are adopted in appendix F. A summary of the calculations can be seen in the following sections.

6.4.1 Crown

In appendix F calculations are made for the crown. It is stated the structure is based on the crown of the Strijkijzer in the Hague. In the figure below an impression and a structural drawing of this element of the building is given.

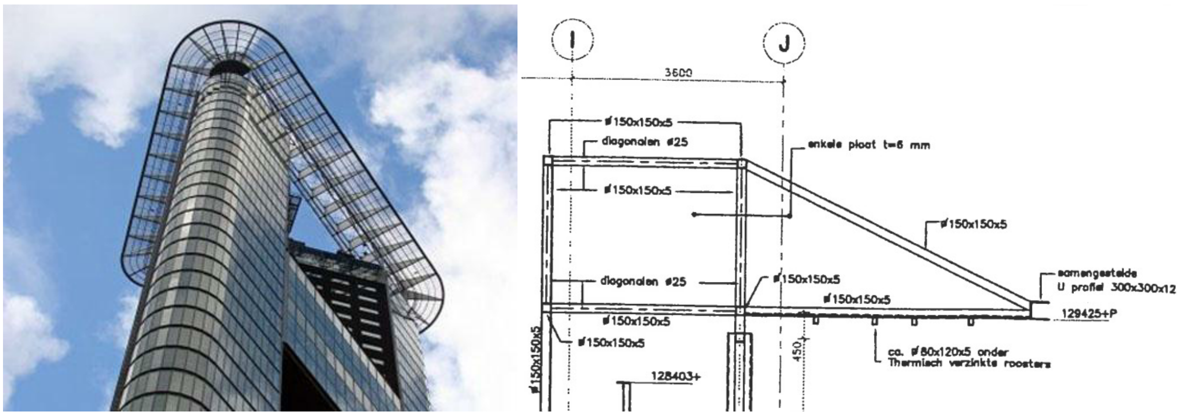


figure 6.13 impression crown strijkijzer the Hague (left) structural drawing (right)

Because of the bigger center to center distance and the restricted length of the crown, for the design of the Oval Tower the following elements will be used.

Element	Dimension
Main profiles	150 x 150 x 10
Diagonals	Ø 25mm
Aluminum plates	t = 6mm

table 6.1 dimension elements crown

The crown is a cantilevered structure. To reduce the moment above the column, on the opposite side of the cantilever, the structure is connected with the beam elements of the top floor. The cantilevered structure is calculated with help of Technosoft. In the figure below the structure can be seen including the reaction forces because of dead load and live load. The loads which are applied to the crown can be read in appendix F.

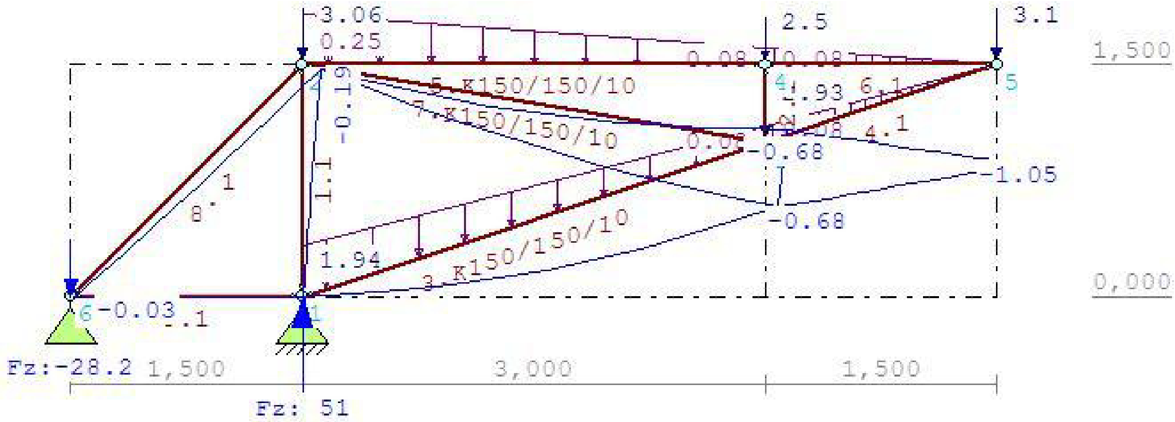


figure 6.14 reaction forces and deflection DL

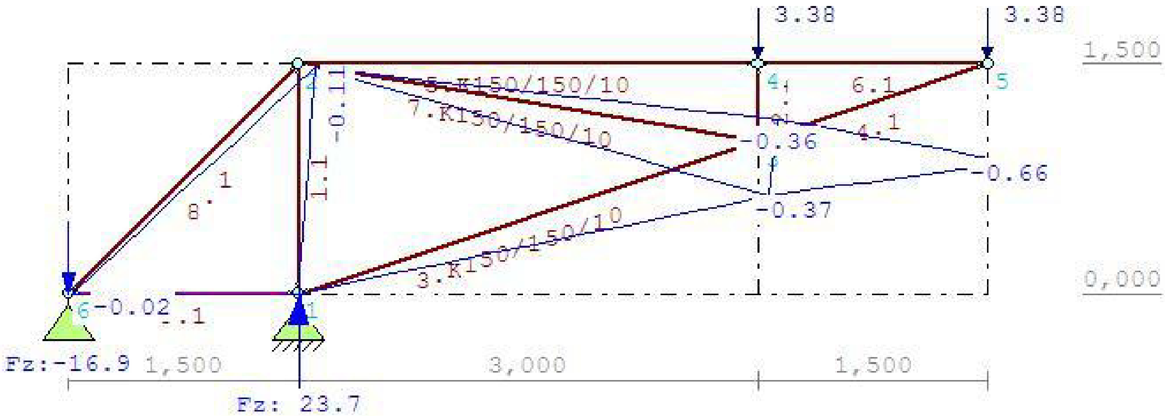


figure 6.15 reaction forces and deflection LL

It can be concluded the structure meets the requirements of deflection. In a later stage the elements can be optimized. From here the dimension of elements as chosen in table 6.1 can be adopted for the structure of the crown. For further calculations the total structure will be imported in a FEM program called SCIA engineer. In the figure below an impression of the structure.

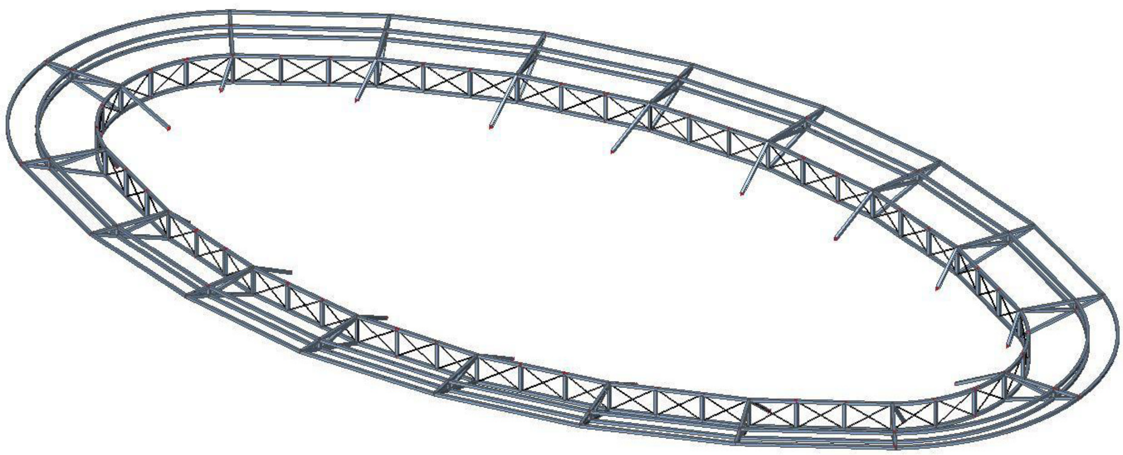


figure 6.16 impression structural elements crown

6.4.2 Façade columns

For the division of the vertical forces between the core and the façade columns it will be assumed, half of the loads will be transferred to the core. The other half will be transferred via the façade columns. A remark has to be made this assumption is very rough and only will be used for a first idea of the dimension of the columns.

With the vertical live loads and the horizontal wind loads working on the columns, a first choice for the dimension of the façade column will be made. The dimension becomes Ø273 with t=10mm. This is the dimension for a façade column with a height of 4m. The column is loaded with maximum dead load, live load and wind load. In alternative A, a column with a height of 8m is present. This column however is not subjected to loads from the 27^{th} floor. The dimension of this columns therefore becomes Ø273 with t=16mm.

6.4.3 Core

The core in the extension will be build up out of steel elements. The core which is present in the existing building can be seen in the figure below. On top of the existing core (in blue) the location of the new steel columns is given. The location of the columns has been chosen in a way the vertical transport of elements (elevators and stairs) can continue in the new extension. Furthermore the dotted lines give the locations in which diagonals are present. The diagonal elements are present above walls in which no or few openings are situated.

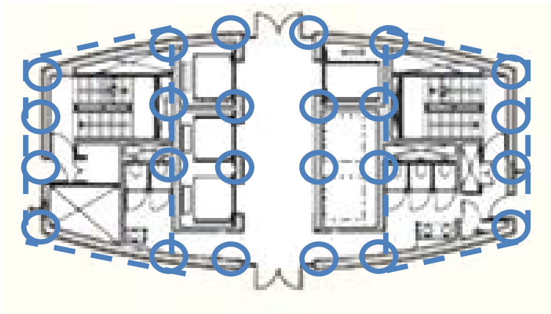


figure 6.17 structural steel elements core extension

In appendix F the dimensions of the profiles are calculated. Below the maximum forces in the profiles because of wind in cross and longitudinal direction are given. Wind in one direction has been taken into account. In the model therefore only the profiles in which tension occurs have been modelled. The diagonals in which compression occurs in reality are present but in the model the behaviour will be neglected. In reality the forces in the tensile diagonals therefore will be slightly lower. For explicitness the location of the diagonals which are subjected to compression in figure 6.18 are given with a dotted grey line.

For wind in the opposite direction the dimension of the diagonals will be chosen equal to the ones in the direction as described above. This modelling leads to a distribution of forces which will be slightly lower in reality and therefore the assumption is a safe one.

The diagonals have been chosen in a way the angle with the horizontal beam is smaller than 60 degrees and close to 45 degrees. This angle is the most cost efficient one.[11]

[11] (Verburg, 2004)

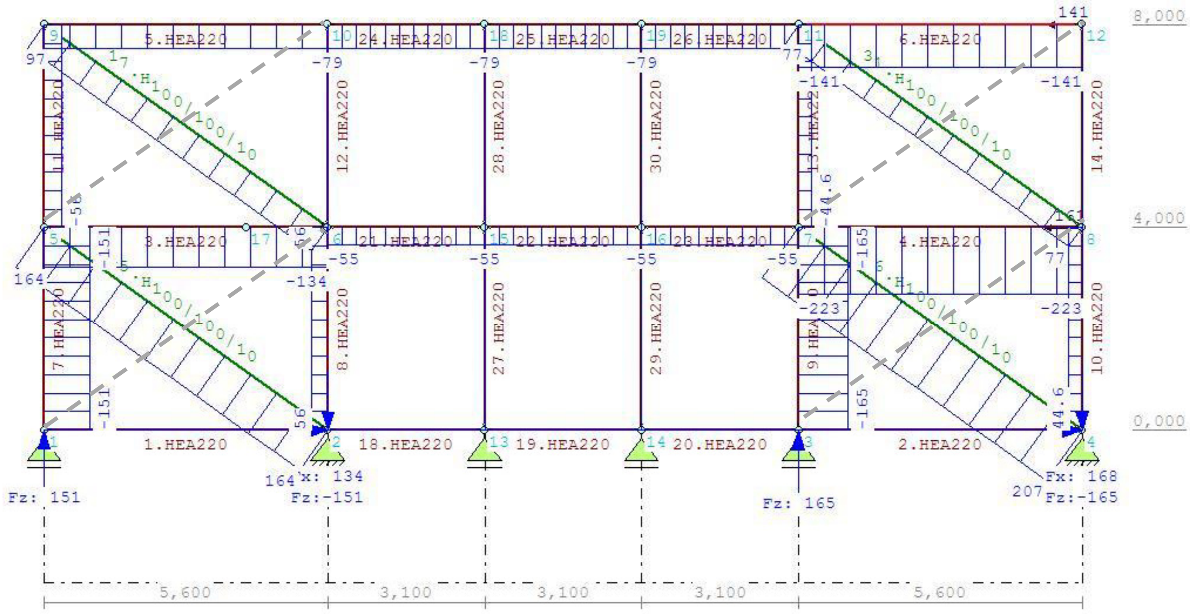


figure 6.18 maximum forces because of wind in longitudinal direction

figure 6.19 maximum forces because of wind in cross direction

	Wind in longitudinal direction	Wind in cross direction
Column	165 kN (compression)	243 kN (compression)
Diagonal	207 kN (tension)	259 kN (tension)
Beam	223 kN (compression)	274 kN (compression)

table 6.2 maximum forces because of wind in both directions

The calculations in appendix F leads to the following profiles: Diagonals L100x10, columns and beam core HE220A.

The façade columns and the elements for the core of the extension can be imported in the SCIA model. An impression of the structure can be seen in figure 6.20.

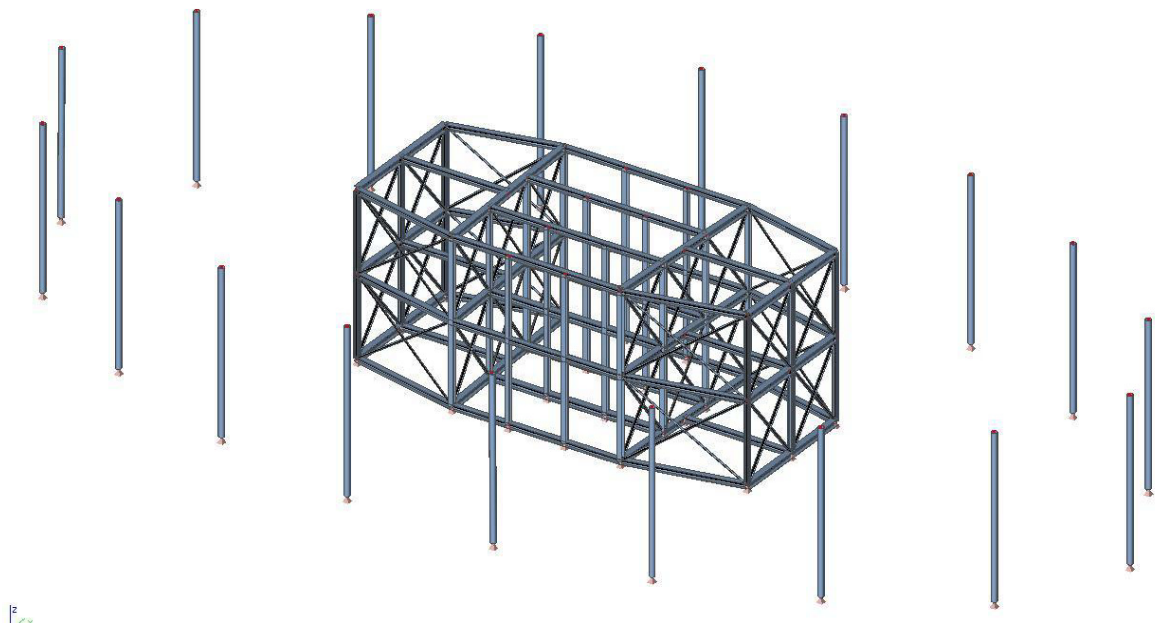


figure 6.20 impression structure facade columns core

6.4.4 Floor system

Now that the dimensions of the crown, core and columns are determined, the floors can be designed. The main shape of the floors of the various alternatives is oval shaped. The span between the core and the columns is 7,8m. Beams have to span this distance. Because of the place of columns and core the beams has to be placed under an angle compared to the other beams. For this irregular placing of beams a floor system with a high flexibility is needed. From table 5.2 and the design as described above it becomes clear a steel plate concrete floor is the most suitable option.

In the design of the floor two direction will be distinguished. A primary direction and a secondary direction. First the beams in primary direction will be determined. To make use of repetition of beams a division has been made in spans with about the same value. In the figure below, this spans can be seen. The maximum length of a beam is 10m, however this beam only transfers a small amount of the corner loads. Therefore the leading beam has a span of 8,7m with a center to center to distance of 7m. The primary beams will be designed on this span.

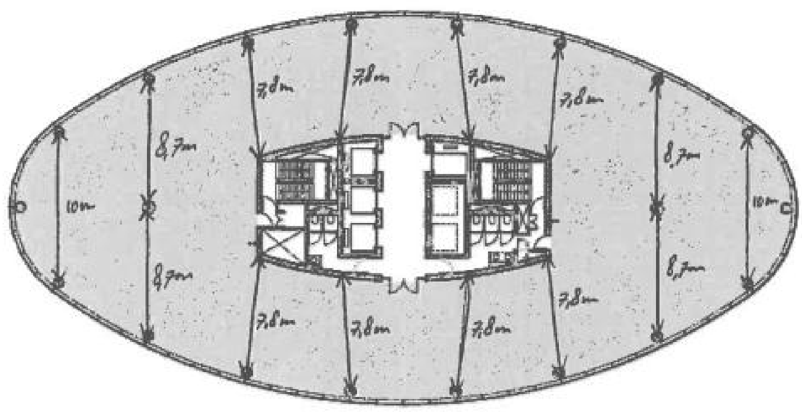


figure 6.21 span primary girders

The secondary beams have a main span of 7m. It is intended to keep the maximum centre to centre distance of these beams 3m. The way the secondary beams span can be seen in the figure below. Furthermore the direction in which the steel elements span is adopted. The floors spans are chosen with help of the existing floor span of the top floor as can be seen in Appendix B.

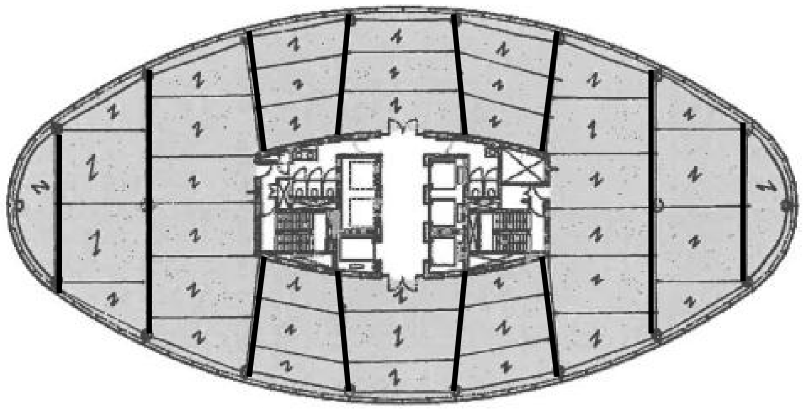


figure 6.22 primary, secondary girders and span steelplates

An overview of the maximum spans on which the girders will be designed can be seen in the table below. With help of these properties and the possible live loads a choice will be made for a Comflor 75 steel plate concrete floor.

	Span	Center to center distance
Primary girder	8,7m	7m
Secondary girder	7m	3m

table 6.3 spans primary and secondary girders

The dead load for this Comflor75 floor is 2,2 kN/m2. The live load working on the top floor is 1,0 kN/m2. The live load working on the 26^{th} and 27^{th} floor is 4,0 kN/m2. With help of the dead load (own weight floor, finishing layer/ducts, installations on top floor) and live load the dimensions of the primary and secondary girders can be determined. An overview of this various girders can be seen in the table below. Calculations are adopted in appendix F.

	Primary girder	Secondary girder
Top floor	IPE500	IPE330
27^{th} floor	HE360B	IPE360
26^{th} floor	HE360B	IPE360

table 6.4 primary and secondary girders floors extension

The outer border girders on the top floor have to be combined with the girders in the crown. For the design of the crown square 150x150x10 elements combined with diagonals are chosen. It has to be checked whether these girders are able to deal with the additional variable floor load. For the inner border girders the core elements have to be combined with the top floor. In the design of the core HE220A elements are chosen, the secondary floor girders are IPE330 elements. It has to be checked whether the HE220A elements are able to deal with the additional live loads.

6.5 Structural design

In this chapter the structural design of the various alternatives will be further elaborated. As a start of point for the alternatives the general structural components as determined in the last section can be used.

6.5.1 Alternative A

6.5.1.1 Crown, core, columns

The crown, core and façade columns as designed in chapter 6.3.1 can be applied to alternative A. A start of point for this design is the introduction of a frame above the columns which are attached to the existing outrigger system. Due to this design the supports below the columns can be removed. The forces will be transferred to the columns which contain overcapacity. Furthermore on the axis W and Y two additional columns have been placed. Because these columns are not subjected to wind load the same dimension as the façade columns can be used. The structure with the crown, columns and core now will look like the figure below.

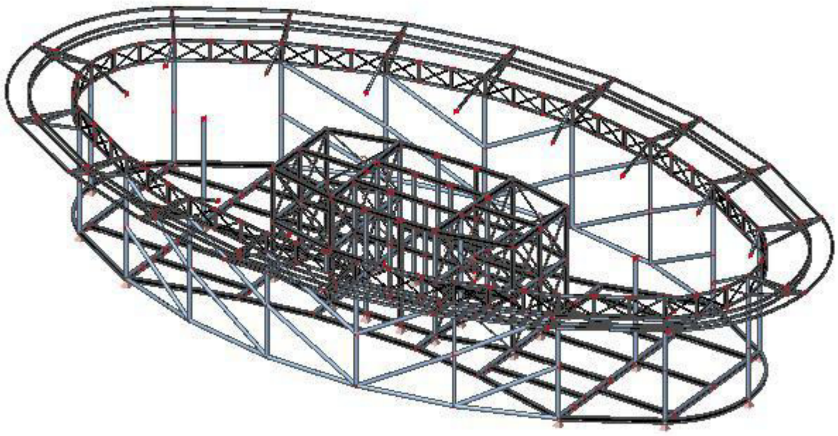


figure 6.23 crown, core and columns alternative A

6.5.1.2 Floor systems

The dimensions for the girders in the top floor can be taken over from 6.4.4. The girders will be placed as can be seen in the figure below.

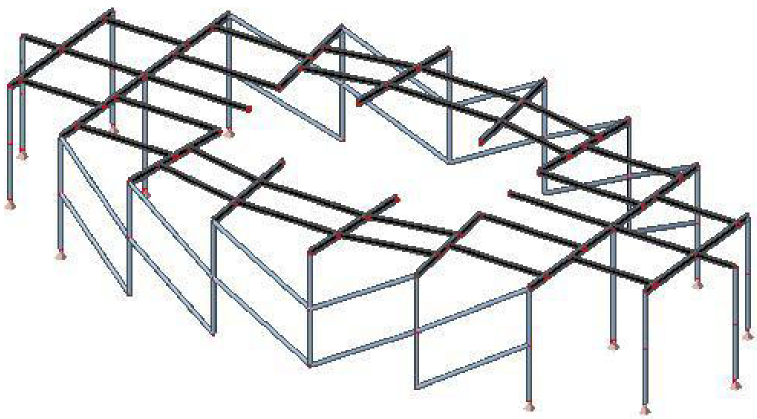
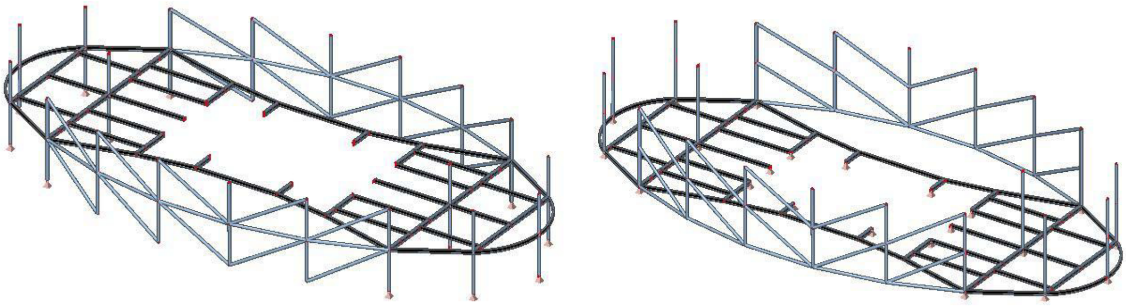


figure 6.24 top floor alternative A with columns

For the 27th and 26th floor the maximum spans can be taken over from section 6.4.4. A start of point for the design in alternative A is the reduction of forces above the existing columns which are subjected to wind loads because of the outrigger structure. The grids of both floors can be seen in the figure below.



6.25 girders alternative A, 27th and 26th floor

An impression of the entire structure of alternative A can be seen in the figure below.

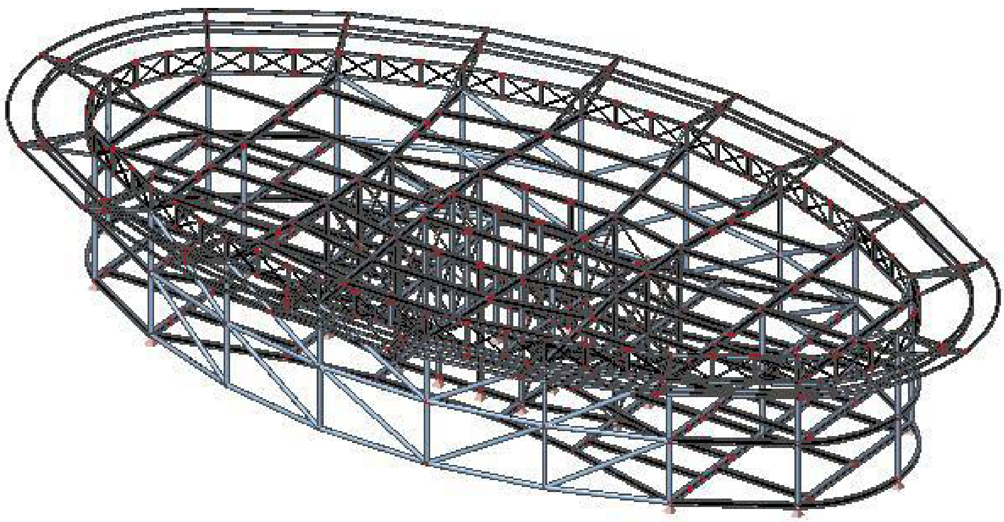


figure 6.26 structural elements alternative A

6.5.2 Alternative B

6.5.2.1 Cantilevered structure

In alternative B, cantilevered structures are present. In the figures below it can be seen where the cantilevered structures are present. The leading cantilevered structure is the one with the biggest length. This structure will be elaborated, the dimensions of the smaller cantilevered structures will be chosen according to these calculations.

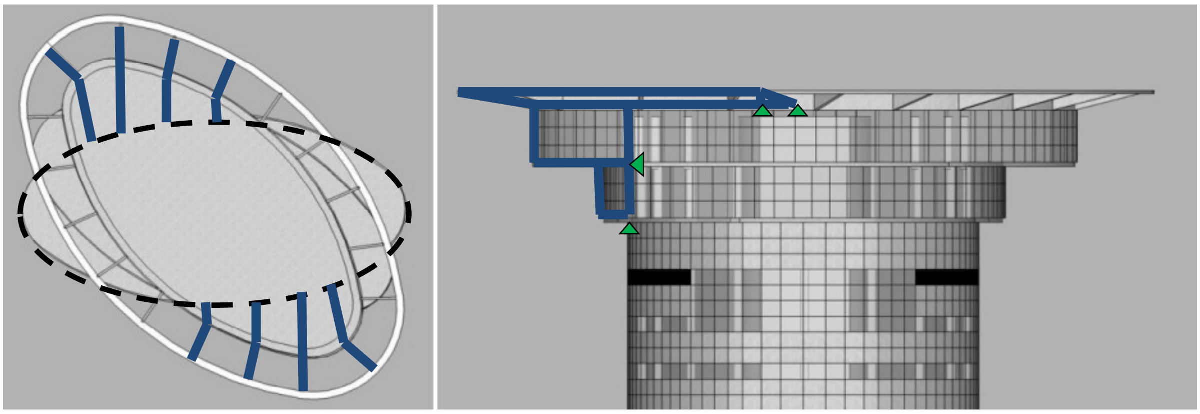


figure 6.27 place cantilevered structure

The cantilevered structures exists out of glass and steel elements. The columns of the cantilever are subjected to the same horizontal wind loads as calculated for the façade columns. Therefore the columns will be chosen as Ø273; t=10mm. It is intended to hang the structural elements, of the 26th and 27th floor, to the crown. In the crown therefore additional structural elements will be placed. To deal with the reaction forces the elements will be extended till the core. In the figure below the structural elements of the maximum cantilevered structure can be seen. The centre to centre distance will be chosen as 7m. The beam elements which are present in the cantilevered structure have the same dimension as the floor system. The girders are calculated with a length of 8,7m. It can be concluded the girders with a smaller span meet the requirements.

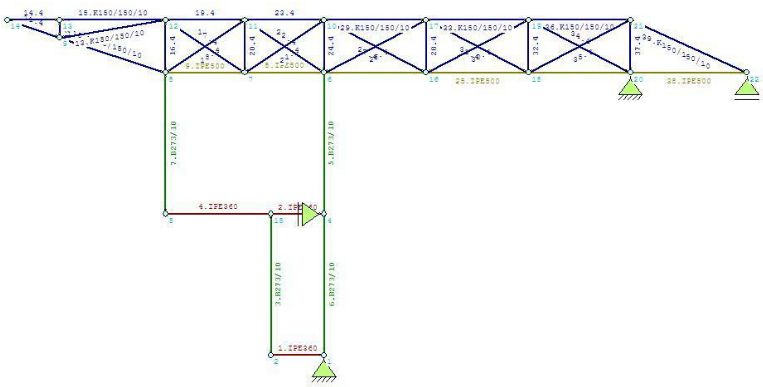


figure 6.28 structural elements leading cantilevered structure

For this preliminary design several aspects will be checked. First the deflection of the total structure, this calculation will be made in SLS. In figure 6.33 the deformed structure can be seen. The maximum deflection becomes 22,0mm. This deflection occurs when the top floor will be fully loaded with live load and the 26th and 27th floor will be instantaneous loaded. The allowed deflection is 9000/250=36mm.

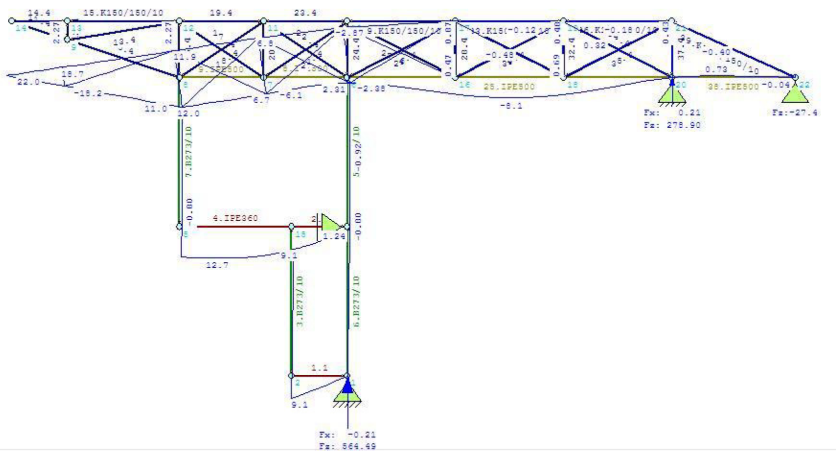


figure 6.29 deflection cantilevered structure

Furthermore the additional structural elements will be checked. The ultimate limit state will be checked, axial forces are acting on these elements. In the figure below it can be seen the maximum tensile force which is present in the rectangular 150x150x10 elements is 530kN. The area of these elements is 54,9e2 mm2. This means a tension of F/A = 530e3/54,9e2 = 97N/mm2 will occur in this element, which is below the allowed value of 355 N/mm2.

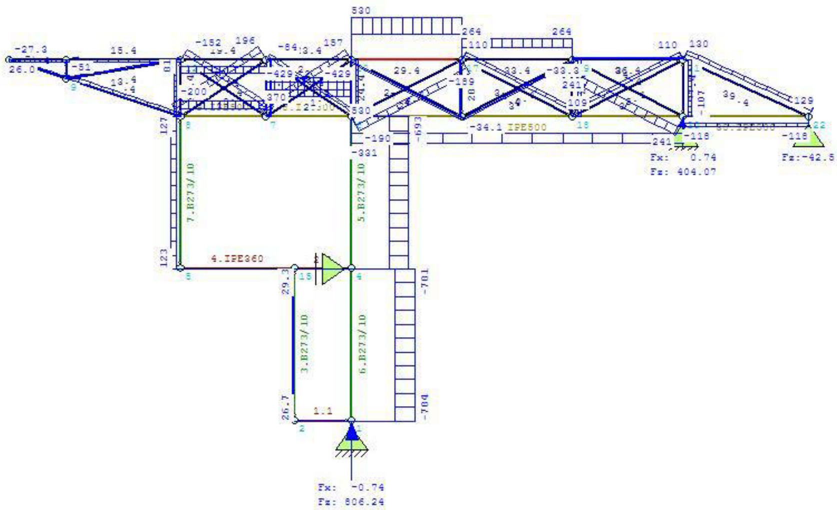


figure 6.30 normal forces cantilevered structure

Furthermore the normal force in the column will be checked. The maximum normal force is 781kN. This value is above the calculated value of 650kN, however no additional wind moments are working on these columns because the columns are not attached to the façade. This means the tension in these columns will be lower than the previous calculated façade column and therefore below the allowed value of 355 N/mm2.

From here it can be concluded for the preliminary design of the cantilevered structure the most important elements as designed above meet the requirements.

6.5.2.2 Crown, core, top floor

The structural elements which will be used in the crown will be taken over from section 6.4.1. Furthermore because of the cantilevered floors, additional structural elements will be used in the crown. As can be seen in the architectural design the floors are slightly twisted. To make a design for the crown which is in cooperation with the floors below, the shape of the crown will slightly differ from the oval shape.

The core will be designed as described before in the section about general components. The top floor will be designed in a way the maximum length of the primary girders will not override the length of 8,7m with a center to center distance of 7m and the secondary girders will not override the span of 7m with a center to center distance of 3m. An impression of the crown, core and top floor can be seen in the figure below.

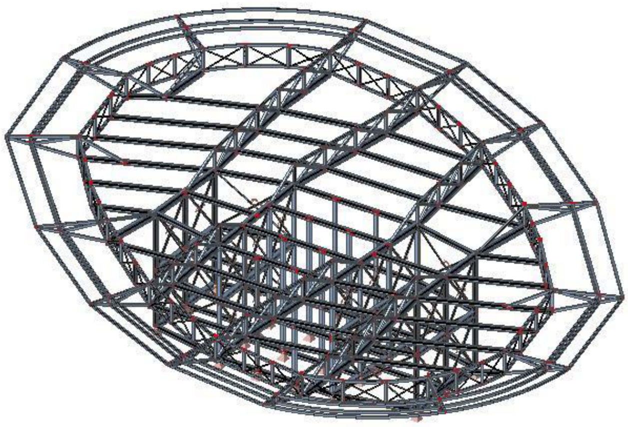


figure 6.31 alternative B crown, core, 26th floor

6.5.2.3 Columns, 27th floor

The dimension for the façade columns has been determined in the section about general components. Furthermore additional diagonals are placed to make the transfer to the columns in which overcapacity is available possible. An impression of the columns and the 27th floor can be seen in the figure below.

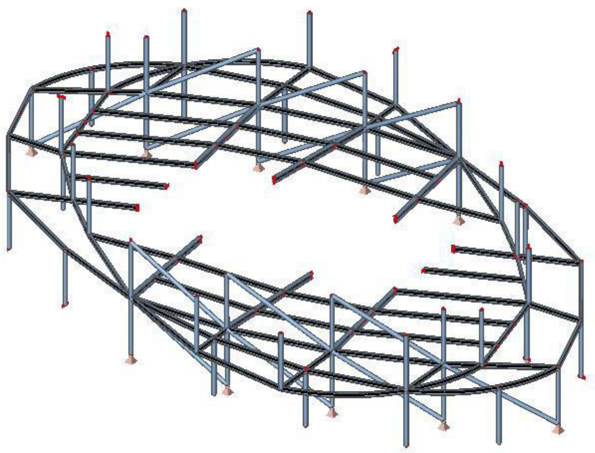


figure 6.32 alternative B columns, 27th floor

6.5.2.4 26th floor

The structural elements of the 26th floor of alternative B can be seen in the figure below. The design of the floor is made in a way the primary and secondary girders do not exceed the maximum beam spans as determined in section 6.4.4. In the figure below an impression of the 26th floor is given. For clarity of the structure again the core is given adopted.

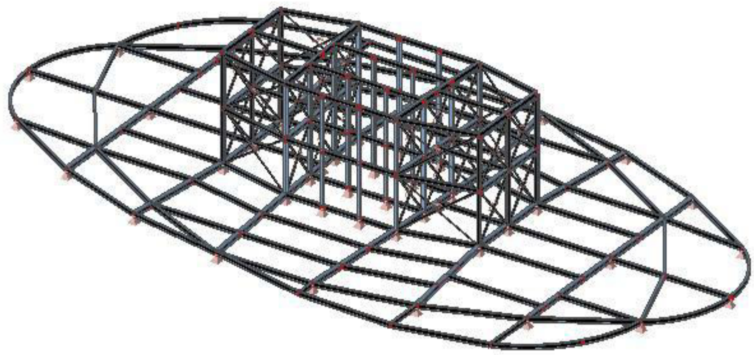


figure 6.33 structural elements 26th floor and core

An impression of all the structural elements used in alternative B can be seen in the figure below.

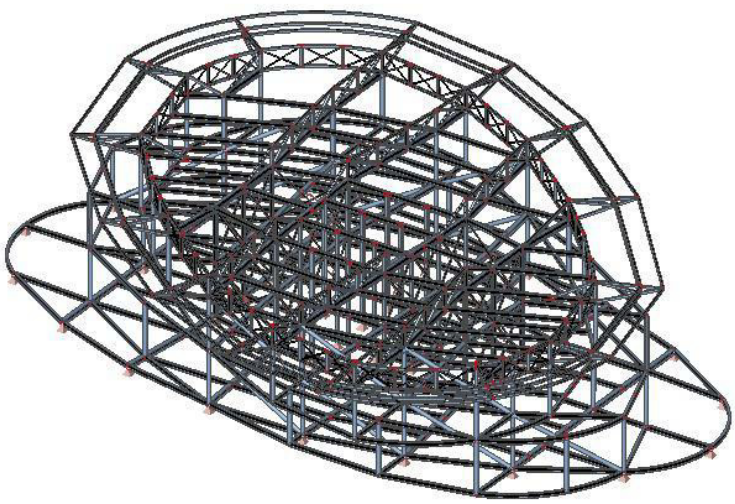


figure 6.34 structural elements alternative B

6.5.3 Alternative C

6.5.3.1 Crown, core, columns

The main architectural starting point of alternative C is the opening which is present in the middle of the building. To realize this opening the core has to be split up in two half's. Therefore only horizontal beams and columns have to be removed. The diagonals can stay at their place. In cross direction for the wind therefore nothing changes and the structural design of the core can be kept the same. For wind in longitudinal direction however the structural design will change. The change can be seen in the figure below.

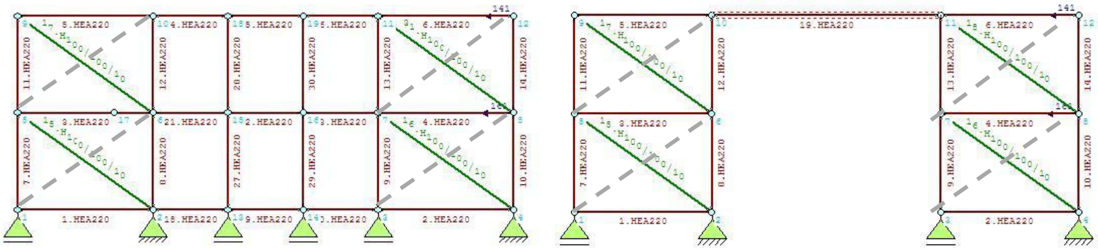


figure 6.35 structure core alternative A and B (left) structure core alternative C (right)

When wind load will be applied the force distribution will change from the general design of the crown. The load distribution can be seen in the figure below.

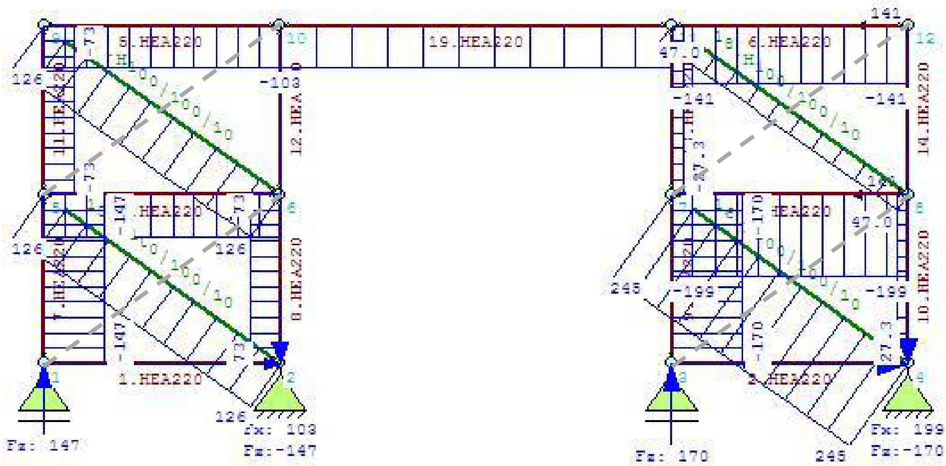


figure 6.36 load distribution core

The maximum force in the diagonal will be raised to 245kN, this value is still below the leading value of 259kN on which the diagonals in longitudinal direction are designed. The maximum value of compression in the column will be 170kN, this value also is below the leading value on which the columns are dimensioned. The value in the upper beam will be raised to a value of 103kN. This value is below the leading value in the beam which is caused by wind from the cross direction. It can be concluded the dimensions of the beams chosen in the general design of the crown are sufficient.

Because of the hole in the core the maximum span of the crown will be increased. To deal with this span additional structure will be placed in the crown. The elements will be fixed to the core. The dimension of the columns can be taken over from the section in which the general elements are determined and have a dimension of Ø273 t=10mm. An impression of the crown (including additional structure), the core and the columns can be seen in the figure below.

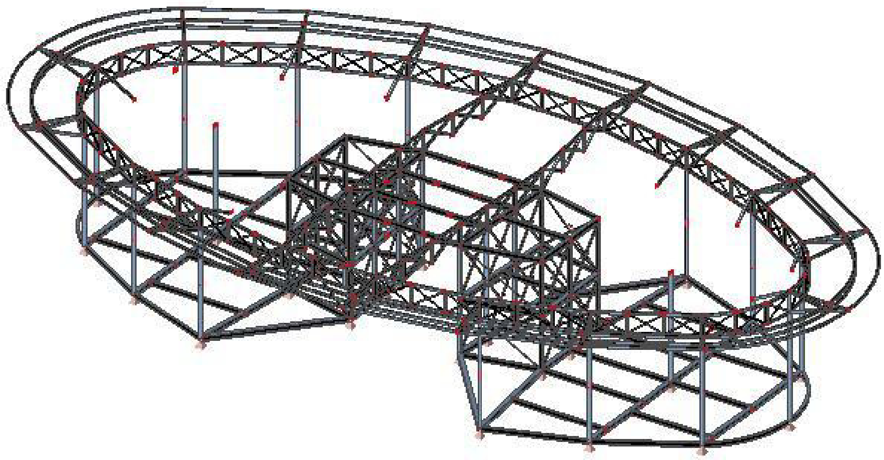


figure 6.37 impression columns, core and crown

6.5.3.2 Top floor, 27th and 26th floor

The design of the beam grid again will be made with help of the dimensions determined in the section about general elements. The primary girders in the top floor are IPE500 elements, the secondary girders are IPE330 elements. Below an overview of the beam grid.

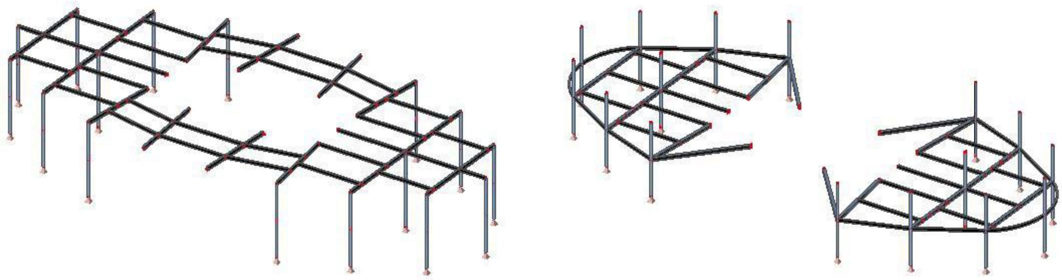


figure 6.38 structural elements top floor (left), 27th floor (right)

For the 27th and 26th floor the beam grid will exists out of HE360B elements in leading direction and IPE360 beams for the opposite direction.

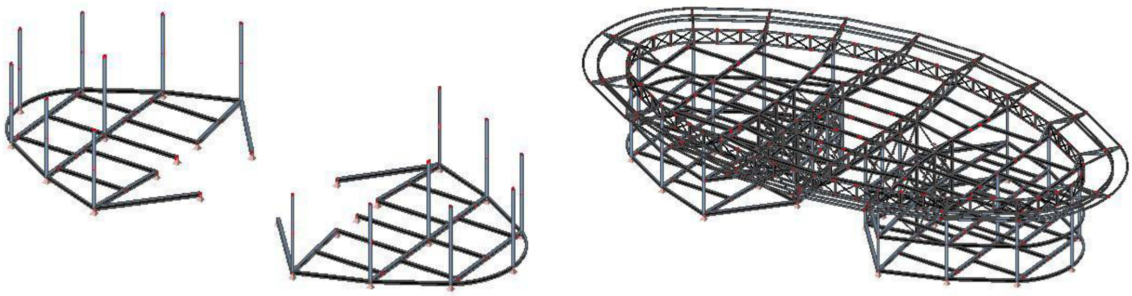


figure 6.39 structural design 26th floor (left) entire structure (right)

6.6 Calculations of alternatives

6.6.1 Loads and load combinations

In the previous sections a first structural design has been made for the three various alternatives. In this chapter the loads in the table below will be applied to the design.

Element	Self-weight (kN/m3)	Dead load (kN/m2)	Live load (kN/m2)
Main load bearing structure (steel)	78,5		
Structural elements crown (steel)	78,5		
Finishing elements crown (aluminium)	27,7		
Restaurant floors			
26th and 27th		2,20	4,0 (ψ=0,5)
Finishing layer and ducts		1,50	
Top Floor		2,20	1,0 (ψ=0)
Finishing layer and ducts		1,50	
Installations		1,65	
Partition walls		0,69	
Façade elements		0,50	

table 6.5 overview loads extension alternative A

The load combinations in the table below have been taken into account. Furthermore for every combination in which wind is applied also wind in the opposite direction has been taken into account. Because in this chapter the extension will be calculated, the factors which have to be taken into account for the extension are highlighted in the table below.

BC		Dead load		Live load		Wind load cross direction		Wind load long direction	
		OT	ext	OT	ext	OT	ext	OT	ext
SLS									
BC1		**1,0**		**0,5**		**1,0**			
BC2		**1,0**		**0,5**				**1,0**	
BC3		**1,0**		**1,0**					
BC5		**1,0**				**1,0**			
BC6		**1,0**						**1,0**	
ULS									
BC7	str/geo (6.10a)	1,2	**1,5**	0,75	**0,83**				
BC11	str/geo (6.10b)	1,2	**1,3**	1,5	**1,65**				
BC15	str/geo (6.10b)	1,2	**1,3**	0,75	**0,83**	1,5	**1,65**		
BC16	str/geo (6.10b)	1,2	**1,3**	0,75	**0,83**			1,5	**1,65**
BC19	str/geo (6.10b)	**0,9**				1,5	**1,65**		
BC20	str/geo (6.10b)	**0,9**						1,5	**1,65**

table 6.6 final load combinations extension Oval Tower according to NEN8700

The wind loads which are determined in appendix F will be imported in the model as line loads working on the façade. The dead load and live loads which are present on a floor can be imported with help of load panels. In the figure below an example of a load panel. The partition walls and façade elements are imported as a distributed load working on the beams. Therefore the dead load as given in the table above is multiplied with the associated height.

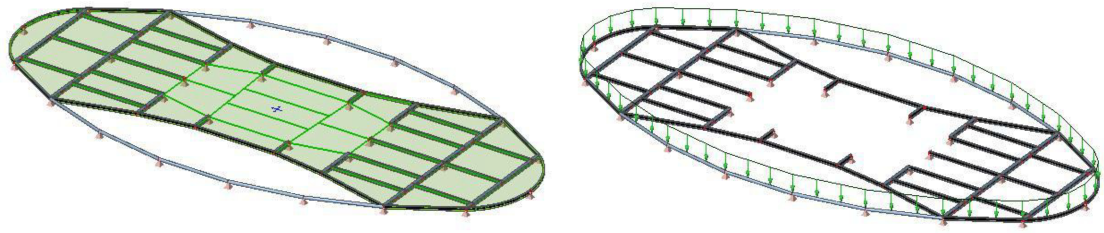


figure 6.40 import of load per square meter (left) import of load per meter (right)

6.6.2 Reaction forces

With help of the loads and load combinations several reactions forces working on the existing building will be taken into account. First the maximum additional force on the columns (ULS). Afterwards the maximum additional vertical load which is working on the core (ULS). Finally the resultant of the wind load in both direction (SLS). In the figure below the various reaction forces can be seen.

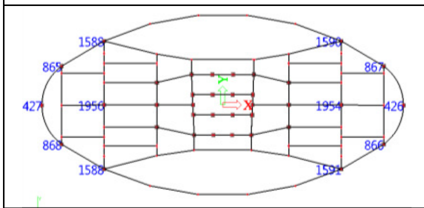
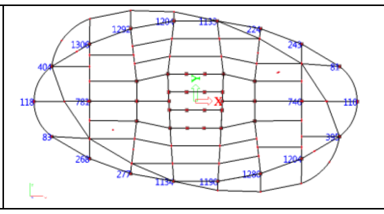
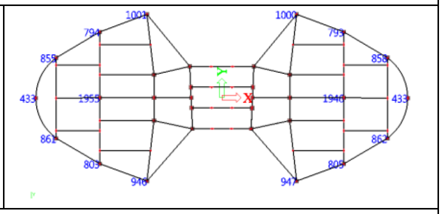
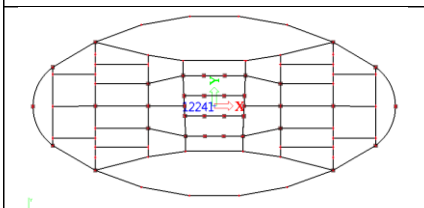
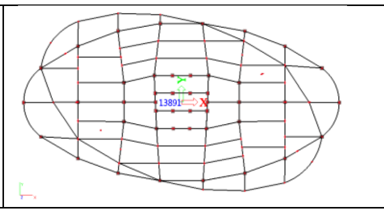
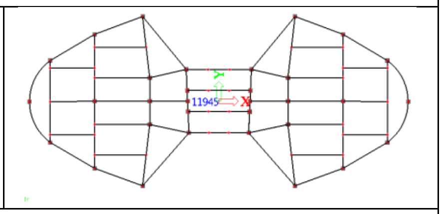
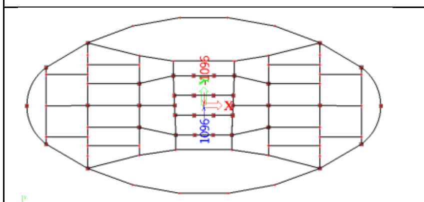
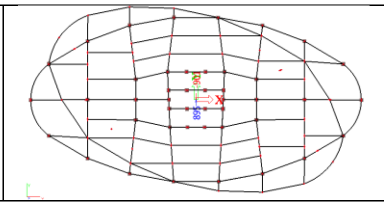
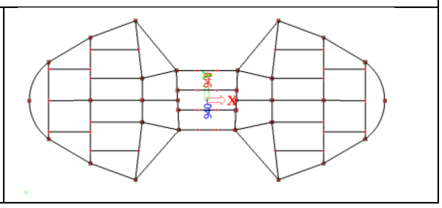
Alternative A	Alternative B	Alternative C
Maximum additional force columns		
		
Maximum additional force core		
		
Resultant force wind cross direction		
		

figure 6.41 reaction forces various alternatives

6.7 Comparison and choice alternative

In this chapter a comparison and a choice for an alternative will be made. In chapter 4 it is concluded the core and the columns are the critical aspects, therefore the additional forces on these elements are taken into account.

6.7.1 Core + foundation

The additional forces which are working to the core for the three alternatives are adopted in the table below.

	N'd extension UGT (kN)		
Alternative	Alt A	Alt B	Alt C
Core	12241	13891	11945

table 6.7 additional forces below core

The maximum additional forces which is working on the core is 13891 kN. The area of the core in the upper part of the building is 25,9m2. The additional compression on the core therefore will be 13891e3 / 25,9e6 = 0,54 N/mm2. This additional tension is small in comparison with the available overcapacity for compression in the core.

Furthermore the overcapacity which is available below the core is 64096 kN. The additional force which will be working on the foundation because of the extension can be transferred via the existing foundation below the core.

Tension in the core has to be taken into account in detail in the final model, after the choice for an alternative is made in the next chapter the tension on the existing core will be elaborated in detail.

From here it can be concluded that an extension on top of an existing building causes no problem with regard to the additional compression which occurs because of this extension. Furthermore the foundation below the core is able to deal with the additional loads.

6.7.2 Columns + foundation

The maximum additional force which is working on the columns can be compared with the overcapacity which is present in the columns and the foundation.

The existing columns below the extension will be loaded by the maximum force which occurs because of dead load, live load and wind load which is working onto the extension. These maximum additional forces are given figure 6.41.

Besides the forces described above, wind load will cause additional forces in the outrigger columns. A scheme of this process is adopted in the figure below. It can be expected when the extension will be placed on the current building additional vertical forces will be present in the columns on axes C to F and L to P.

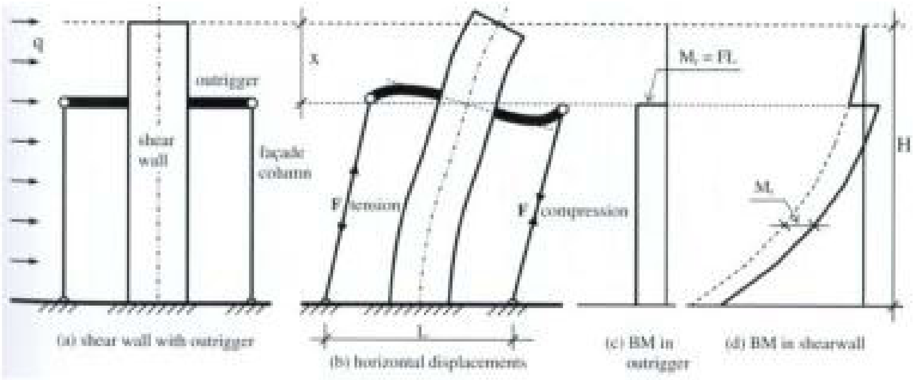


figure 6.42 outrigger structure

In the columns on axes C to F and L to P small overcapacity is present. It is favourable that vertical forces due to dead load and live load of the extension will not be transferred via those columns. To check whether these columns are able to deal with the additional force the maximum wind load from figure 6.41 is taken into account. This load is applied to the current structure.

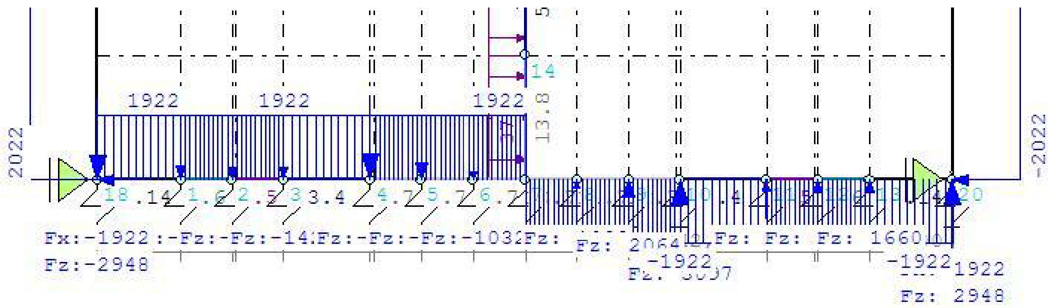


figure 6.43 force in outrigger column Oval Tower without extension

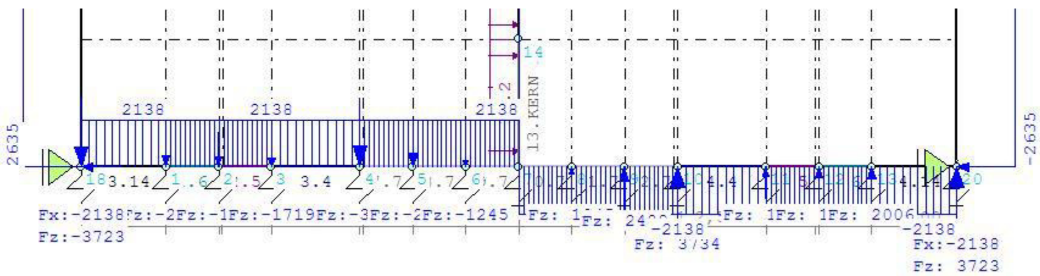


figure 6.44 maximum force in outrigger column Oval Tower with extension

In the figures above the situation without and with extension is taken into account. The force of 2022 kN is the force which is working in two columns as described in chapter 4. The maximal additional load which was calculated including creep behaviour became: (1,08*2022 / 2) = 1092 kN. The new maximum force because of wind in the outrigger column becomes (1,08*2635 / 2) = 1423 kN. The difference between those two values is 1423-1092 = 331kN. It can be concluded that because of the outrigger structure the columns will be loaded with an additional 331 kN.

Now that the maximum force is know it can be checked whether the columns on the axis C to F and L to P are able to deal with the additional force.

In appendix G tables are admitted in which (for the three alternatives) various checks are made. The first check is to see whether the columns of the existing building can deal with the existing overcapacity. The second check is to see whether the foundation below the columns can deal with the overcapacity because of the forces from the extension. Below the two checks are described. These checks have been made for all columns and all foundation piles below the columns.

$$Check\ 1: \frac{N'd\ max; additional}{Overcapacity\ column}$$

$$Check\ 2: \frac{N'd\ max; additional}{Overcapacity\ foundation\ piles}$$

When the maximum design value is bigger compared to the overcapacity available in the column, the column or foundation does not meet the requirements. The value can be subdivided in three classes. These classes are given in the table below.

Classes	Description	Value in table	Color in table
	Column does not meet requirements		
1	Overcapacity < design value	difference < 500 kN	
2	Overcapacity << design value	500 kN ≤ difference < 1000 kN	
3	Overcapacity <<< design value	1000 ≤ difference	

table 6.8 division in classes

A way to deal with this loads is to strengthened the columns or foundation. A column or foundation pile in class 3 is least desirable, subsequently class 2 and afterwards class 1. With help of this division the table below can be made.

Piles or foundation in class	Alternative A	Alternative B	Alternative C
1	7	6	2
2	1	2	3
3	2	2	3

table 6.9 classes compared per alternative

With regard to additional loads on the columns it can be concluded alternative A is the most favourable option. The second most favourable option is alternative B and the least favourable option is alternative C.

6.7.3 Conclusion

With help of the comparison made in this chapter it can be concluded for additional compression in the core no problems will occur tension has to be investigated in detail, which will be done in the next chapter.

Furthermore an investigation has been made into the additional loads which have to be transferred to the columns and foundation. From a first comparison it can be said alternative A seems most attractive to work further with. Therefore from here a choice will be made to elaborated alternative A in detail in the next chapter.

7 Structural design final alternative

7.1 Structural adjustments

In this investigation it will be checked whether it is feasible to extend the Oval Tower. Therefore the investigation from appendix G again will be taken into account. The table which belongs to this investigation is given below.

	Alt A						
Axes	N'd DL LL WL	N'd WL outrig	N'd max	Overcapacity column	check	Overcapacity foundation	check
A	865		865	4211	ok	1547	ok
B	1588		1588	1416	172	2190	ok
C	0	331	331	674	ok	4239	ok
D	0	331	331	1036	ok	2994	ok
E	0	331	331	1036	ok	2234	ok
F	0	331	331	0	331	1746	ok
G	1598		1598	1228	370	2008	ok
H	867		867	5011	ok	1111	ok
J	868		868	4155	ok	1710	ok
K	1588		1588	1494	94	2478	ok
L	0	331	331	846	ok	5131	ok
M	0	331	331	982	ok	5267	ok
N	0	331	331	900	ok	2686	ok
P	0	331	331	239	92	1780	ok
Q	1592		1592	1038	554	106	1486
R	866		866	4661	ok	887	ok
2V	427		427	1934	ok	2918	ok
2W	1950		1950	962	988	3686	ok
2Y	1954		1954	0	1954	1686	268
2Z	426		426	1781	ok	849	ok

table 7.1 column and foundation check Alternative A

In the table it can be seen the columns on axes B, G, K and Q will not meet the requirements. An option to deal with these forces is to strengthening the columns. However the columns which are nearby on respectively the axes A, H, J, R still contain large overcapacity.

Therefore adjustments to the structure will be made. With the adjustments it is intended to lower the number of columns which have to be strengthened. The adjustments will be made to the 27th floor. The structural elements of the 27th floor will look like figure below.

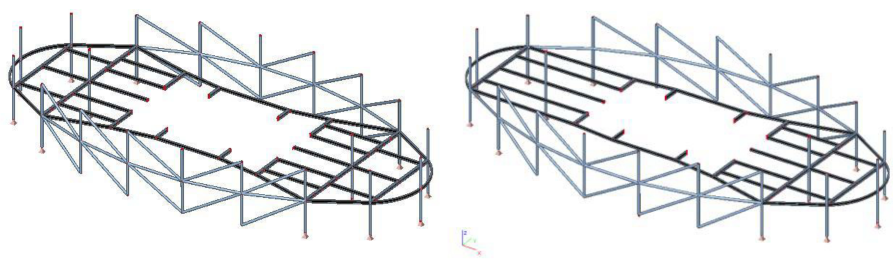


figure 7.1 current structural design elements alternative A (left) adopted structural design elements alternative A (right)

It can be seen that the corner of the floor transfers the loads to the columns on the axes B, G, K and Q. The structural design therefore will be changed in a way the loads will be transferred to the columns which contain overcapacity. This leads to the structure which can be seen in figure 7.1.

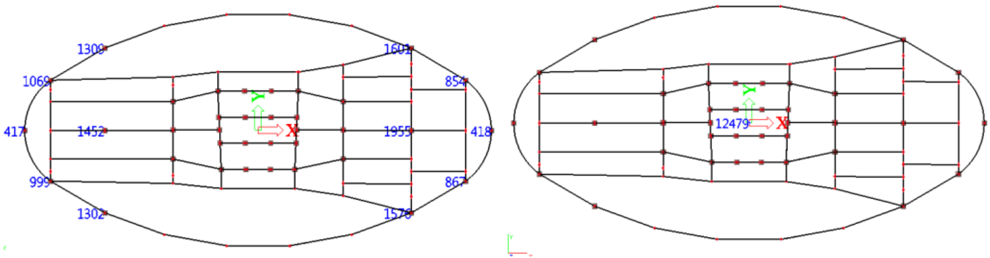


figure 7.2 maximum loads after adjusting floor structure columns (left), core (right)

A note has to be made, the investigation only has been made to see whether it is possible to adjust the structure in a way existing columns do not have to be strengthened.

Axes	Alt A N'd DL LL WL	N'd WL outrig	N'd max	Overcapcity column	check	Overcapacity foundation	check
A	1069		1069	4211	ok	1547	ok
B	1309		1309	1416	OK	2190	ok
C	0	331	331	674	ok	4239	ok
D	0	331	331	1036	ok	2994	ok
E	0	331	331	1036	ok	2234	ok
F	0	331	331	0	331	1746	ok
G	1598		1598	1228	370	2008	ok
H	867		867	5011	ok	1111	ok
J	999		999	4155	ok	1710	ok
K	1302		1302	1494	OK	2478	ok
L	0	331	331	846	ok	5131	ok
M	0	331	331	982	ok	5267	ok
N	0	331	331	900	ok	2686	ok
P	0	331	331	239	92	1780	ok
Q	1592		1592	1038	554	106	1486
R	866		866	4661	ok	887	ok
2V	417		417	1934	ok	2918	ok
2W	1452		1452	962	490	3686	ok
2Y	1954		1954	0	1954	1686	268
2Z	426		426	1781	ok	849	ok

In the investigation above it can be seen that it is possible to make slight adjustments to the design in which the various checks will be OK.

Therefore it is recommended to first investigate whether it is possible to adjust the design before concluding a column or foundation has to be strengthened.

7.2 Final structure

In this part of the chapter the final design of the Oval Tower with extension will be taken into account. The core including the foundation below the core will be checked. Furthermore the columns and the foundation below the columns will be checked. The top deflection and finally the dynamical behaviour will be taken into account.

7.2.1 Core building including foundation

In this section, first stresses in the core will be checked. All load cases in table 5.7 will be taken into account. From these load cases the envelopes will be checked. Minimum and maximum envelopes are checked. It has to be taken into account that a minimum envelope gives a maximum compression value and a maximum envelope gives a maximum tensile value.

First the SLS is taken into account to see whether tension occurs in the core. In the figures below the maximum envelope in SLS is given without and with extension.

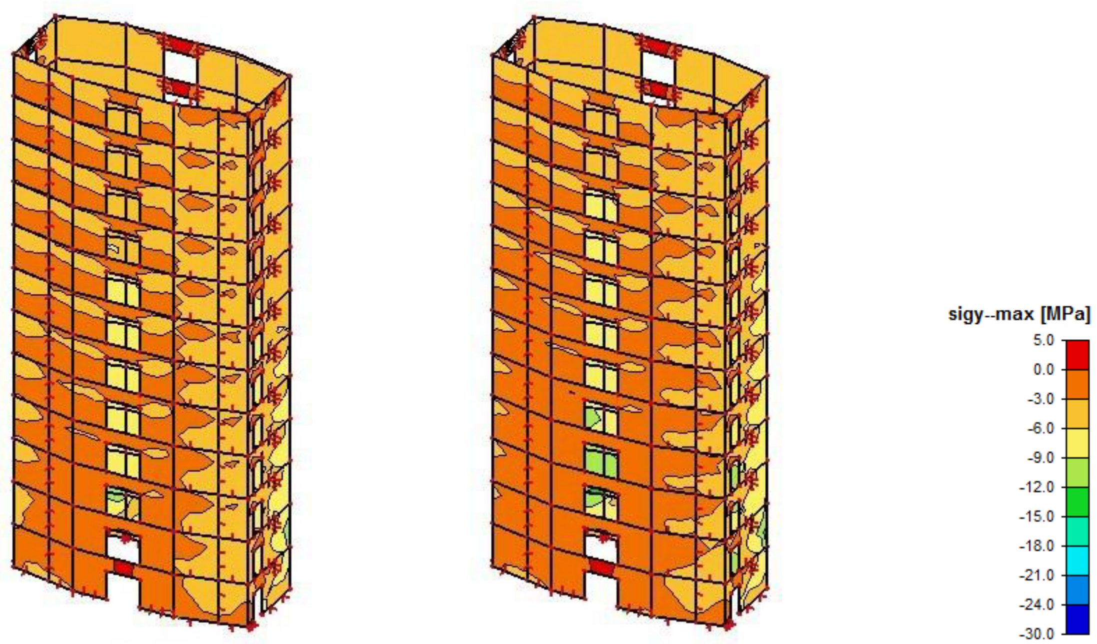


figure 7.3 maximum envelope SLS without extension (left), with extension (right)

From here it can be concluded no tensile forces occur in the SLS. This means no reduction has to be made to the modulus of elasticity in case top deflection will be taken into account.

The next step is to see whether in ULS the compression forces stay below the allowed value of 27 N/mm2. Therefore the minimum envelope in ULS has been checked. In the figures below, the values without and with extension.

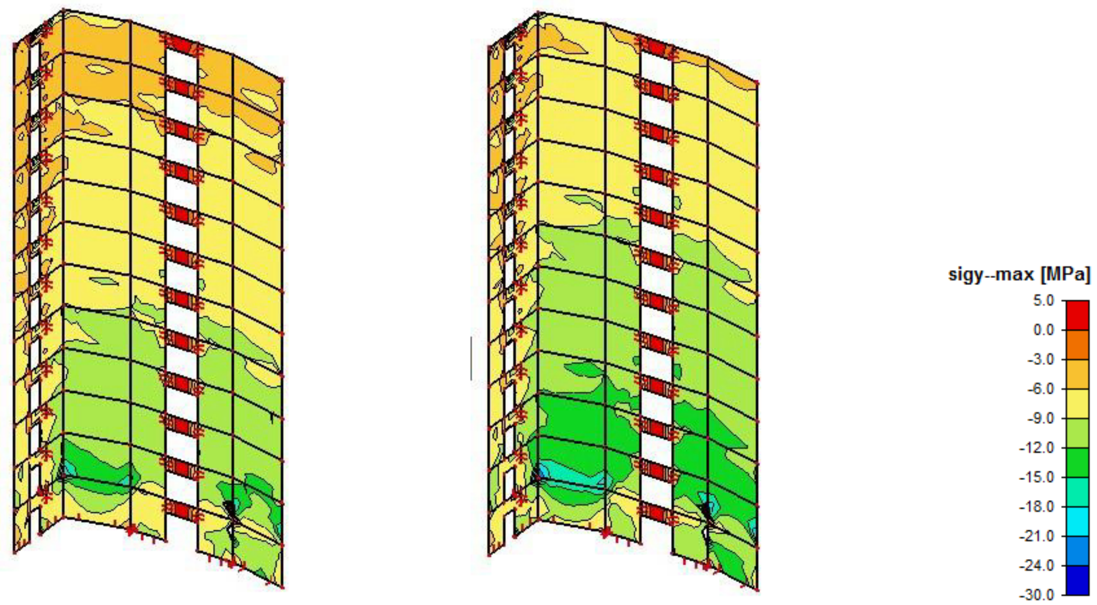


figure 7.4 minimum envelope ULS without extension (left), with extension (right)

From here it can be concluded the values for compression stay below the allowed value of 27 N/mm2 and the core does not have to be strengthened.

The next situation which has been taken into account is the maximum envelope in ULS. This has been done to see whether tension occurs in ULS.

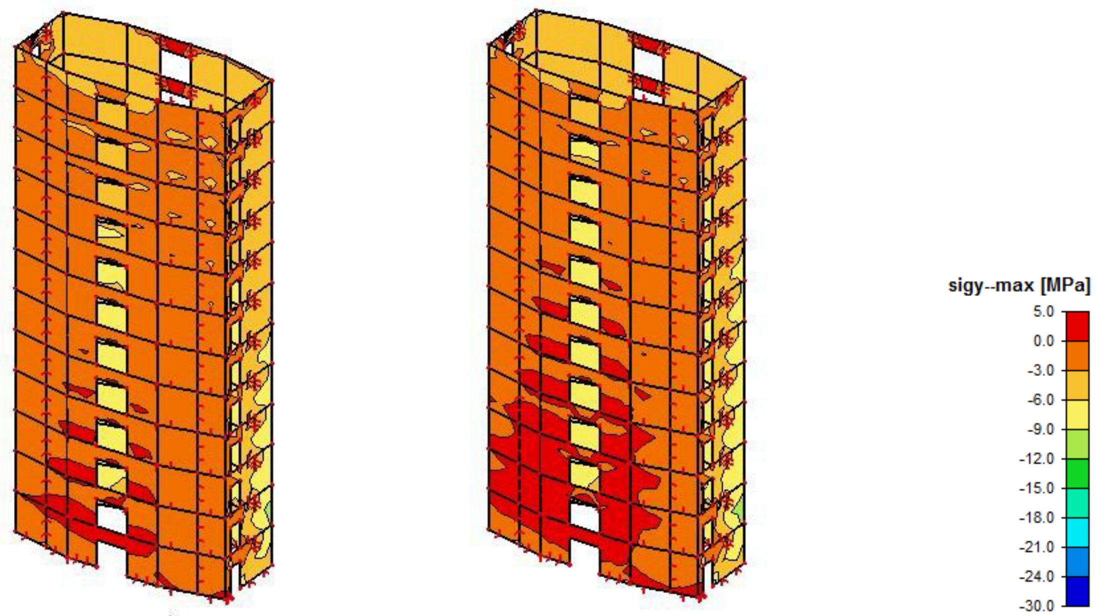


figure 7.5 maximum envelope ULS without extension (left), with extension (right)

It can be seen in the ULS tension occurs in the core. In the situation without extension also tensile forces occur. To see whether adaptations to the core have to be made first it has to be checked whether the reinforcement in the core is able to deal with the tensile forces. When there is insufficient reinforcement present to deal with the additional forces adaptations to the core have to be implemented.

There are various ways in which the core can be strengthened. In the literature study an investigation has been made throughout the ways in which building elements can be strengthened. Three methods will be distinguished. These methods can be seen in the table below.

	Additional elements inside building	Adding elements around existing structure	Glued reinforcement
Stability building	++	0	-
Stiffness building	++	+	+
Strength building	++	+	+
Losses in floor area	-	+	++
Dimension elements	-	+	++
Transport elements	-	0	+
Costs	+	+	-
Erection speed	0	-	+

table 7.2 Comparison methods to strengthen existing columns

The first method is to add additional elements inside the building. The main structural system of the Oval Tower is an outrigger system. An outrigger system is used to reduce the moment in the lower core. What can be done to again reduce the moment is addition of an extra outrigger system. For example at half the height of the building. In the figure below a scheme is admitted in which way the system works. However disadvantages of this model is additional forces in the column. Most likely because of these additional forces the columns have to be strengthened.

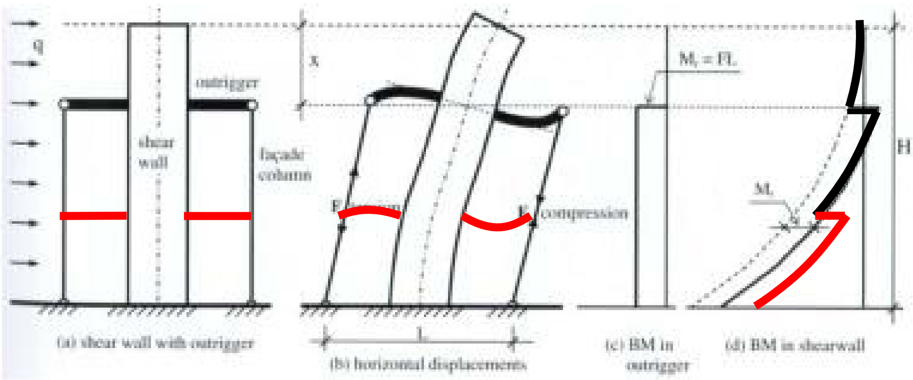


figure 7.6 additional outrigger system

The second method is adding elements around existing structure. This method is most applicable when smaller elements like columns have to be strengthened. In case of the core it is not needed to add elements around the entire structure because the additional tension is only locally present. Therefore this method will not be taken further into account.

The third method to deal with the additional tension in the core is making use of glued reinforcement. Because the floors on which additional tension will appear is limited the amount of glued reinforcement will be limited.

It can be concluded that because of the additional tension in the core first it has to be checked whether the reinforcement which is present in the core is able to deal with the additional stress. If the core is unable to deal with this additional stress two options are given, namely making use of glued reinforcement or adding of additional outrigger elements. A detailed investigations has to be made which option is most suitable. Costs here will be a leading element.

Furthermore the foundation below the core will be taken into account. In chapter 3.4 it is already concluded because of symmetry reasons the core can be divided in four equal parts. The division again is given below.

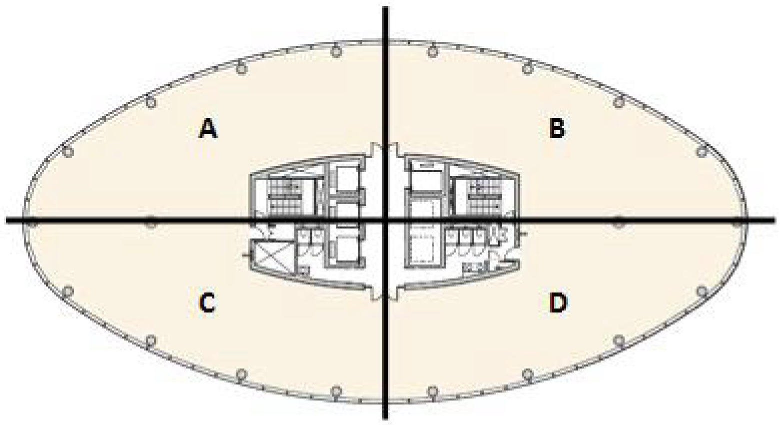


figure 7.7 distribution in zones concerning leading foundation

The capacity of the piles in zone D is lower compared to the other zones, therefore zone D is leading and will be taken into account. The numbering of the piles is given in figure 7.8. Furthermore the maximum load which is working on the piles is also given in the figure below.

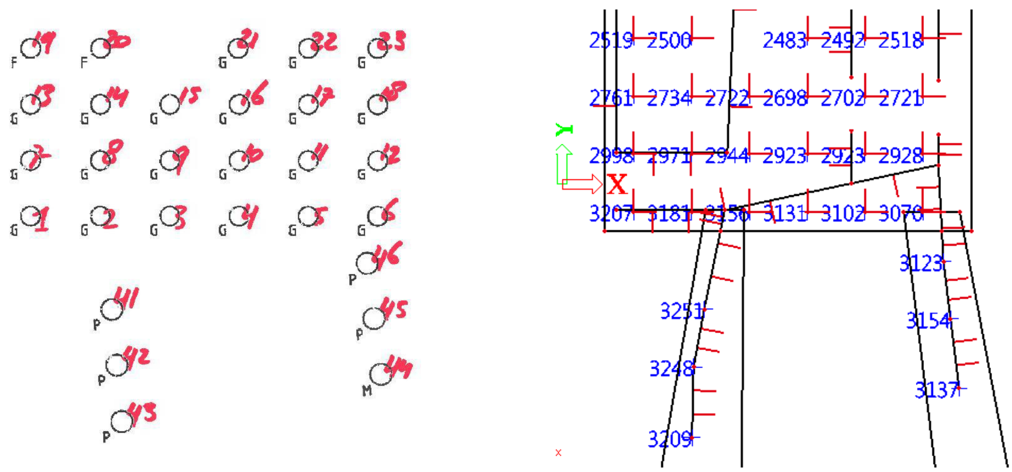


figure 7.8 numbering piles zone D (left), maximum reaction forces building with extension (right)

The capacity of the piles has been determined before. When the maximum forces in the piles will be compared with the capacity of the piles a unity check can be executed. In the table below the various unity checks have been given.

Pile	N'd with extension (kN)	N'u (kN)	UC (-)	Pile	N'd with extension (kN)	N'u (kN)	UC (-)
1	3207	3224	0,99	16	2698	3224	0,84
2	3181	3224	0,99	17	2702	3224	0,84
3	3156	3224	0,98	18	2721	3224	0,84
4	3131	3224	0,97	19	2519	4642	0,54
5	3102	3224	0,96	20	2500	4642	0,54
6	3070	3224	0,95	21	2483	3224	0,77
7	2998	3224	0,93	22	2492	3224	0,77
8	2971	3224	0,92	23	2518	3224	0,78
9	2944	3224	0,91	41	3251	3713	0,88
10	2923	3224	0,91	42	3248	3713	0,87
11	2923	3224	0,91	43	3209	3713	0,86
12	2928	3224	0,91	44	3123	3652	0,86
13	2761	3224	0,86	45	3154	3713	0,85
14	2734	3224	0,85	46	3137	3713	0,84
15	2722	3224	0,84				

table 7.3 unity check foundation piles below core

From here it can be concluded that in the leading zone D all piles meet the strength requirements. This means the entire foundation below the core and the abutments meet the requirements for strength, no adaptations to the foundation below the core have to be made because of the extension.

7.2.2 Columns including foundation

The final structure of the extension will be chosen according to alternative A.

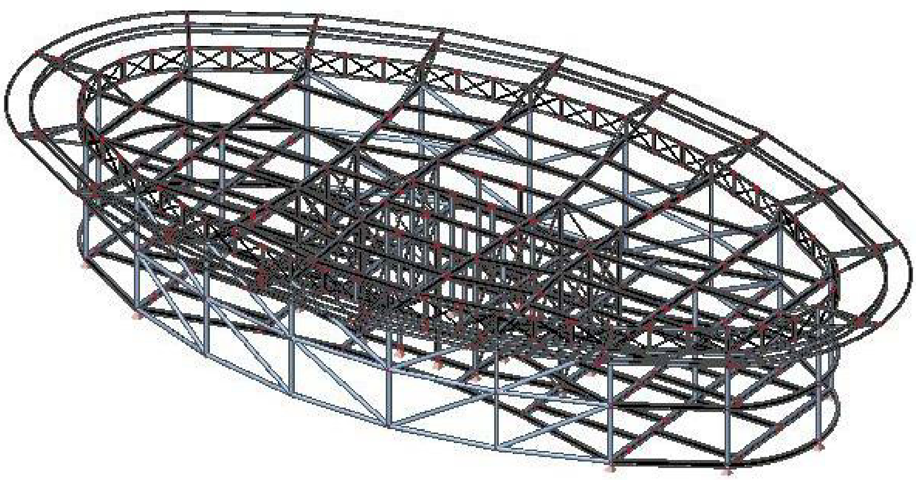


figure 7.9 structural design Alternative A

When the structural design as can be seen in the figure above will be adopted to the current structure. Various columns and foundation piles have to be strengthened. It is recommended to first investigate whether it is possible to adjust the design before concluding a column or foundation has to be strengthened. In the table below it can be seen which columns have to be strengthened when no adaptations will be made.

	Alt A						
Axes	N'd DL LL WL	N'd WL outrig	N'd max	OC column	check	OC found	check
B	1588		1588	1416	172	2190	ok
F	0	331	331	0	331	1746	ok
G	1598		1598	1228	370	2008	ok
K	1588		1588	1494	94	2478	ok
P	0	331	331	239	92	1780	ok
Q	1592		1592	1038	554	106	1486
2W	1950		1950	962	988	3686	ok
2Y	1954		1954	0	1954	1686	268

figure 7.10 columns and foundations which have to be strengthened

From the investigation in chapter 7.1 it is concluded that with slight structural adjustments to the 27th floor the loads can be transferred in a way the columns below the axes B and G does not have to be strengthened. For symmetry reasons this also can be applied to the columns on axes K and Q. However for column Q after these adaptations still the foundation does not met the requirements.

	Alt A						
Axes	N'd DL LL WL	N'd WL outrig	N'd max	OC column	check	OC found	Check
F	0	331	331	0	331	1746	ok
P	0	331	331	239	92	1780	ok
Q	1592		1592			106	1486
2W	1950		1950	962	988	3686	ok
2Y	1954		1954	0	1954	1686	268

figure 7.11 columns and foundation which have to be strengthened after adaptations

When the elements above will be strengthened, the structural feasibility of the extension on top of the Oval Tower will be obtained for the columns and the foundation below the columns.

7.2.3 Top deflection

For calculation of the top deflection the existing model which is used in the [WSC OT] has been used. Before it is already calculated the modulus of elasticity has to be reduced for cracked concrete. The extension will be modelled as a singular concrete core with the same properties as the existing core. For wind load the same values which are working to the top of the existing building are used. The situations described in the table below have been taken into account.

	Lower six floors	Upper floors
Situation 1: entire core imported as uncracked concrete	22333 N/mm2	22333 N/mm2
Situation 2: lower six floors imported as cracked concrete	11667 N/mm2	22333 N/mm2

table 7.4 modulus of elasticity according to [WSC OT] for both situations

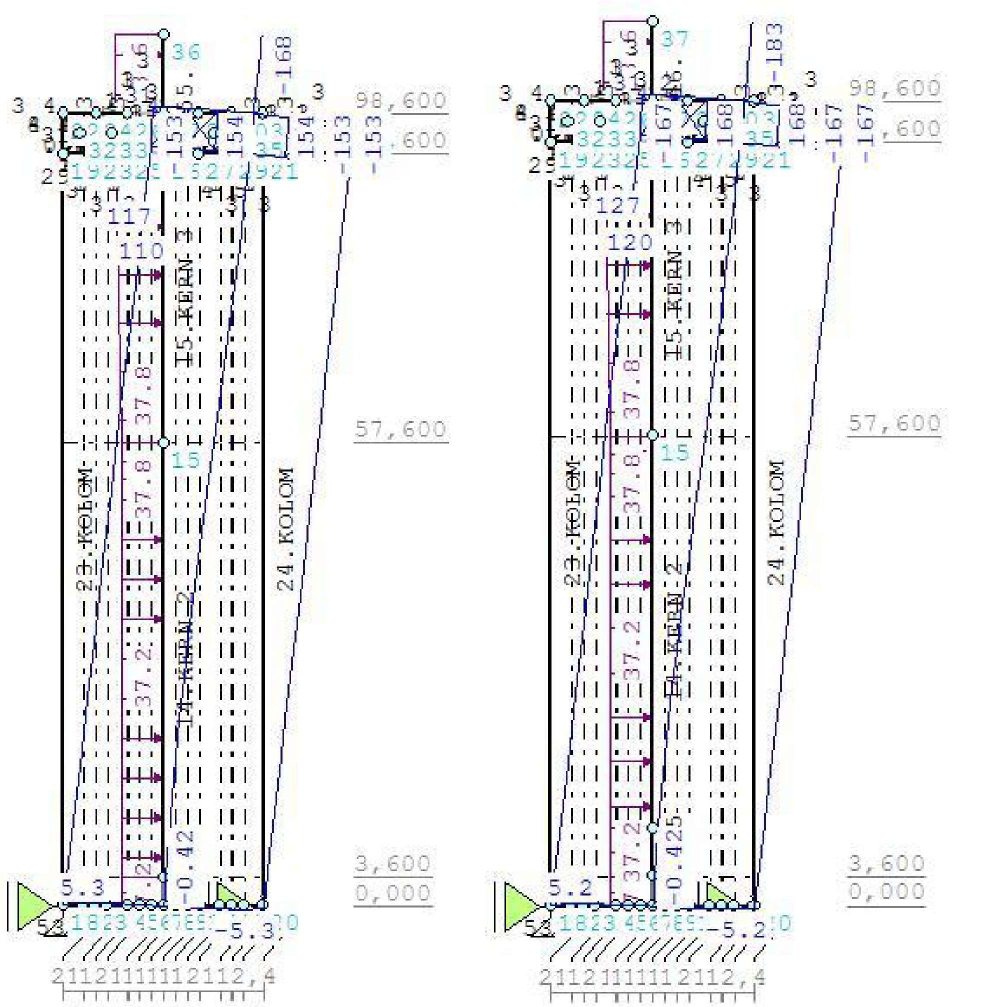


figure 7.12 top deflection situation 5 (left), situation 6 (right)

The allowed value becomes 108500/500 = 217mm. In both situations the requirements for top deflection will be met.

From here it can be concluded the requirements for top deflection will be met in case an extension will be built on top of the Oval Tower.

7.2.4 Dynamical behaviour

In the [WSC OT], literature study and chapter 3.6 various calculations have been made with regard to the dynamical behaviour of the Oval Tower. Below an overview of the various calculations.

- In the [WSC OT], the dynamical behaviour has been determined with help of NEN6702.
- After delivery of the building modifications have been made to the building standards. The calculations in the [WSC OT] give lower values compared to calculations which are made with the modified NEN6702.
- Shortly after delivering of the Oval Tower nuisance was experienced by the users of the building. Because of this nuisance TNO did measurement to the dynamical properties of the building. A report of these measurements can be seen in Appendix C. Calculations have been made with help of the measured values.

	a;cross (m/s^2)	a;long (m/s^2)	a;torsion (m/s^2)	a;total (m/s^2)
Values according to NEN6702 calculated in [WSC OT]	0,037	0,030		
Values according to NEN6702 after modifications building codes	0,095	0,059		
Values according to NEN6702 with help of measured values TNO	0,073	0,032	0,037	0,082

table 7.5 dynamical behaviour, values according to previous calculation

Calculation of the own frequency in NEN6702 will be done with help of a formula in which the frequency will be approached. Furthermore the damping ratio of the building according to NEN6702 also will be estimated with help of a formula in which the behaviour will be approached. The measured values for damping and own frequency are calculated in special for the Oval Tower and therefore these values will be considered as accurate. Further calculations will be elaborated with help of the measured values for own frequency and damping.

Besides the values calculated in table 7.5, another calculation has been made. The current building standards which have to be adopted for dynamical behaviour is NEN-EN1991-4, from the Eurocode series.

	a; cross (m/s^2)	a; long (m/s^2)	a; torsion (m/s^2)	A; total (m/s^2)
Values according to NEN-EN1991-4	0,074	0,034	0,050	0,089

table 7.6 dynamical behaviour, values according to NEN-EN1991-4

The values calculated according to NEN-EN1991-4 are slightly higher compared to the calculated values in NEN6702. For further dynamical calculations the values calculated with help of the eurocode will be taken into account.

For the dynamical calculations of the Oval Tower with extension, calculations will be made according to NEN-EN1991-4. Various values will differ in case an extension will be situated on top of the Oval Tower. Below an overview of these values and the way the values are adopted.

- h (m): The height which is taken into account in the original calculations is 98,6m; the height of the building without extension is 95,2m; the new height of the building will be 95,2 * 1,1 = 104,7m. In chapter 5.4 drawings of the various height can be seen.
- b (m): This value stays equal for calculations without and with extension
- fe, own (hz): fe depends on various building properties. Below a formula to determine the own frequency of a building is given.[12] To determine the own frequency with and without the extension it will be assumed the height is the only varying value and the new height is an additional 10% of the current height.

$$f_{e;without} = \frac{17{,}73}{h_{wihout}{}^2} * \sqrt{\frac{EI}{\rho_1}}$$

$$f_{e;with} = \frac{17{,}73}{h_{with}{}^2} * \sqrt{\frac{EI}{\rho_1}} = \frac{17{,}73}{(1{,}1 * h_{without})^2} * \sqrt{\frac{EI}{\rho_1}} = 0{,}83 * \frac{17{,}73}{h_{without}{}^2} * \sqrt{\frac{EI}{\rho_1}}$$

$$f_{e;with} = 0{,}83 * f_{e;without}$$

- D (-): for the damping ratio the measured values will be used. In both situation these values will be kept equal.
- Total mass (kg): In previous chapters for the weight of the existing tower a value of 371150 kg/m has been calculated. This value also will be taken in account for the extension. The total mass in kg can be determined by multiplying this value with the height of the building.

The maximum value of the total acceleration in cross or longitudinal direction depends on the direction in which the maximum acceleration is working.

Here the value in cross direction is leading and therefore the total acceleration can be calculated with: $a_{total} = \sqrt{a_{cross}{}^2 + a_{torsion}{}^2}$

	cross direction		Longitudinal direction		Torsion		Total	
Without / with extension	Without	With	Without	With	Without	With	Without	With
h (m)	98,6	104,7	98,6	104,7	98,6	104,7		
b (m)	53,4	53,4	26,7	26,7	53,4	53,4		
fe, own (hz)	0,41	0,34	0,58	0,48	0,82	0,68		
D (-)	0,007	0,007	0,007	0,007	0,005	0,005		
Total mass (kg)	36,6e6	38,9e6	36,6e6	38,9e6	36,6e6	38,9e6		
a (m/s²)	**0,074**	**0,091**	**0,034**	**0,041**	**0,050**	**0,063**	**0,089**	**0,111**

table 7.7 calculated value for accelaration Oval Tower

[12] (Hoenderkamp, 2007)

The maximum value for the Oval Tower without extension is 0,089 m/s^2. With extension the maximum value becomes 0,111 m/s^2. Both values are adopted in the figure below.

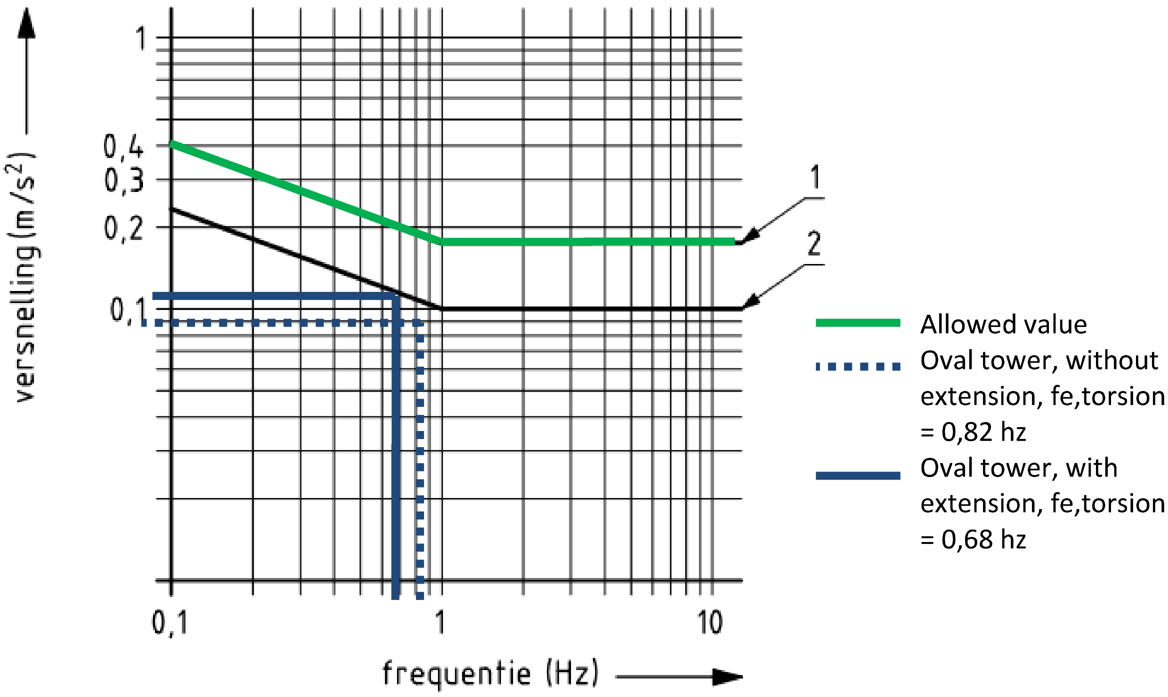


figure 7.13 maximum acceleration Oval Tower without and with extension

From the figure above it can be concluded that for the Oval Tower with extension, the values for dynamical behaviour stay below the allowed values. Because of the complexity of these calculations the values have been checked at the structural company, the assumed values are plausible values.[13]

[13] (van der Windt, 2012)

7.2.5 Conclusion

In this chapter the structural feasibility of the Oval Tower with extension has been taken into account. With help of various investigations the following conclusions have been drawn.

- In the ultimate limit state the lower six floors of the core will be subjected to tensile forces. In the existing situation three floors are subjected to tensile forces.

A recommendation has been given in which way a solution can be found to deal with the additional stresses.

- First it has to be checked whether the reinforcement which is present in the core is able to deal with the additional stress. If the core is unable to deal with this additional stress two options are given. First the addition of extra elements, secondly the use of glued reinforcement. A detailed investigation has to be made which option is most suitable. Costs here will be an important factor.

Furthermore the foundation piles below the core has been taken into account. It can be concluded that:

- In the leading zone all piles meet the strength requirements. This means the foundation piles below the core and the abutments meet the requirements for strength, no adaptations to the foundation piles below the core have to be made because of the extension.

Various columns and foundation piles below the columns will not meet the strength requirements. With slight changes to the lay out of the floor plan positive unity checks will be obtained. From here the following will be recommended:

- If a building element from the existing building does not meet the requirements after addition of the extension, first try to adopt the design in a way the requirements will be met.

Furthermore the top deflection and dynamical behavior have been taken into account, the following conclusions have been drawn:

- The requirements for top deflection will be met in case an extension will be built on top of the Oval Tower.
- For the Oval Tower with extension, the values for dynamical behaviour stay below the allowed values.

8 Construction methodology

In the previous chapters the structural behaviour of the Oval Tower without and with extension has been taken into account. One of the variables which determines the feasibility of the extension is the construction methodology. In this chapter the construction methodology will be taken into account. In the literature study three methods in which the extension can be built are taken into account. Furthermore a trade off matrix has been composed. In this chapter the trade off matrix will be filled. With help of the matrix, answer will be given to the research question below:

- *Which construction method can best be used to extend the Oval Tower?*

After a final choice has been made for the construction method, an explanation will be given in which way this method will be adopted to the Oval Tower.

8.1 Trade off matrix

In the literature study the trade off matrix has been made. The three building methods are determined with help of reference projects and the requirements are defined in the text. In the previous chapters the structural behaviour of the Oval Tower has been investigated and the design of the extension have been made. With help of this investigation various requirements have been added. These requirements will be explained below.

Possibility to pour concrete: Concrete in the main load bearing structure of the extension is not desirable. However for various floor systems which have advantageous properties for an extension, a concrete top layer is used. To be able to pour concrete to a big height adaptations have to be made. The impact of these adaptations can be compared for the various construction methodologies.

Positioning of the crown elements: The crown which is going to be situated on top of the Oval Tower will cantilever in relation to the existing building. The attachment of the crown to the existing structure is a criteria which can influence the building process.

Attachment installation: The installations will be placed in the crown as can be seen in figure 5.5. During the elaboration of the Oval Tower the new installations and ducts has to be attached to the current installations. The way the construction methodology is suitable for this operation is another criteria.

Dismantling of equipment: After the extension has been built, dismantling of the equipment has to be executed. The dismantling may not cause any nuisance to the users of the building.

With these additional requirements the final trade off matrix will look like the one in diagram 8.1. In the upcoming sections the criteria will be taken into account for the individual construction methodologies. The criteria will be rated in three classes: poor (-), average (0) and good (+). Furthermore for the costs and time a comparison will be made with help of estimated values.

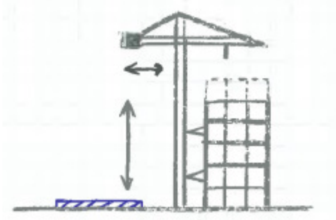
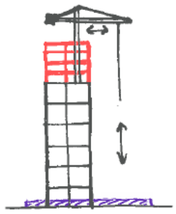
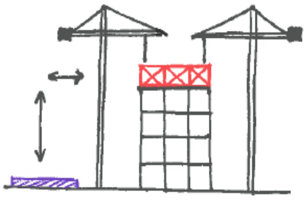
Alternative I, conventional building method	Alternative II, crane on top of the building	Alternative III, lifting extension as one piece
		

figure 8.1 three alternatives building method

Construction Methodology	I, conventional building method	II, crane on top of the building	III, lifting extension as one piece
Standard criteria construction methodology			
Access to site			
Storage at site			
Maneuvering space at site			
Subsoil capacity at site			
Lifting operation, sensitivity to wind			
Tolerances (ability to absorb)			
Environmental impact: noise/transport			
Additional criteria construction methodology with regard to extension			
Operability building during construction			
Nuisance to users surrounding buildings			
Possibility to pour concrete			
Positioning of the crown elements			
Attachment installations			
Dismantling of equipment			
Construction time / costs / risks			
Construction time			
Costs			
Accuracy cost estimate			
Risks			
Final choice			

diagram 8.1 Final trade off matrix

8.1.1 Standard criteria

In this subchapter the criteria for the construction methodology will be evaluated for every individual method.

Access to site: The building site in alternative I and II can be reached easily due to transport via the road. The building site on ground floor is restricted due to the area of the crane. However the top of the existing tower in alternative I can also be used as a building area. For alternative II this is the other way around, on top of the building, the area is diminished by the crane and on ground floor additional space is available. For the third alternative access to the building site is good, the building site is not restricted by placing of a building crane and is accessible in every direction.

Storage at site: For storage at site alternative I and II are limited because of surrounding buildings. Furthermore the same situation counts which was described in access to site. For alternative I, additional storage is available at the top of the building, for alternative II, additional space is present at ground floor. For alternative III the storage at site is good, no restriction because of encircled buildings.

Manoeuvring space at site: For the construction method in which the extension will be build up on ground floor, construction workers and equipment can move freely around the building site. For method I and II the manoeuvring space at the top of the building once more is restricted by the surrounding buildings. Furthermore more attention has to be paid to manoeuvring of equipment and construction workers on top of the building compared to the ground floor.

Subsoil capacity at site: The building site around the building for alternative I already has been used as a building site during construction of the existing building. The foundation for a building crane is still present next to the building and can be reached without major modification to existing buildings. The building crane which is used for alternative II has to be attached to the existing core of the building. The loads which have to be lifted to the top of the building can cause additional tensile forces which are working on the core. When a choice will be made for this method extensive calculations are needed to deal with these additional forces.

Lifting operation, sensitivity to wind: Lifting operations are always sensitive to wind loads. In alternative I and II elements will be lifted to the top. In alternative III the extension will be lifted as one piece. Because of the bigger load and the bigger area subjected to wind load alternative III is more sensitive to wind loads.

Tolerances (ability to absorb): This criteria has to do with the robustness of the construction plan. The robustness can be measured by taking the consequences of a failure into account. Here the failure during a lifting operation will be taken into account. It will be stated the consequences of a failure in the lifting of the extension as one piece, has a bigger consequence compared to the other two methods.

Environmental impact; noise/transport: For construction methodology III a part of a public space will be shut down. This close down has a negative influence on the environmental aspect. On the other hand when a building crane will be placed in front of a building, the view will be disturbed and therefore this has negative influences to the users of the building. When the building crane is situated on top of the building the only nuisance is of elements which will be hoisted.

8.1.2 Additional criteria

Operability building during construction: In this project it is intended that the users of the building always are able to make use of the current building structure. However a building site on top of the building and at the bottom will always cause nuisance for the users. When alternative III will be used for the construction methodology, the operability will be disturbed least.

Nuisance to users surrounding buildings: For users of the surrounding buildings the third alternative is least advantageous. A part of a public space will be offered up for a building site. For alternative I a building crane will be placed in the surrounding area, which can disturb the view. For alternative II the crane will be placed on the roof and therefore least nuisance will be found for this alternative.

Possibility to pour concrete: In the section about floor systems, a floor system is chosen in which concrete have to be poured on top of the steel plates which are used. The concrete of this composite floor system only can be casted when the steel plates are in place. In alternative III the extension will be built up on ground floor, this means the concrete also can be poured on ground floor and the extension as a whole can be lifted up. In alternative I a pump can be placed at ground floor and ducts can be attached to the building crane. With help of this system the concrete can be poured on a height. In the second alternative it is not possible to attach ducts, therefore additional measures have to be taken to be able to pour the concrete.

Positioning of the crown elements: The elements of the crown on top of the extension are cantilevering elements. Therefore for construction workers it is more difficult to attach the elements on a big height in comparison to the same process on ground floor. Additional attention has to be paid to the attachment of the crown.

Attachment installations: On top of the current building above the existing core installations and ducts are present. The technical installations will be placed directly above the core and in the additional crown of the extension. When the first building method will be used the crane can be used to detach and install the old and new installations. For the second building method the crane is placed on top of the core. A conflict occurs between the building crane and the installations. When the third method is used all installations can be placed on ground floor. The old installation can be demolished and the new installation can be attached when placing the extension.

Dismantling of equipment: Eventually when the extension has been build the equipment has to be dismantled. For the first alternative the dismantling can be easily done. It has to be kept in mind conflicts can occur with cantilevering parts. For the second building method the dismantling of the equipment is less straightforward. The crane can be demolished and transported to the ground floor via for the cargo elevator in which the elements have been brought up. The dismantling of the cranes for the third alternative is significantly however not leading for the building process because the dismantling can be done separately from the existing building.

8.1.3 Costs

In this chapter it is intended to give an answer to the question which construction methodology is most feasible to extend the Oval Tower. An answer to this question can be given when an appraisal will be made of the building costs including the costs for the construction methodology. After these costs are determined the amount of money can be compared with the budget which is available for the extension. The equation which belongs to this text is given below. For a first estimation of a positive feasibility, with regard to the costs, the following equation has to be satisfied.

Budget > total building costs

In the trade off matrix a distinction will be made between three various construction methods. The costs for the method will be determined by the price of the building material. Therefore the equation which will be investigated in this section is the one below.

Budget > equipment costs + building costs

Below values will be determined for the budget, equipment and remaining building costs. Costs are subjected to fluctuations in time and therefore the used values are mean values throughout time as much as possible. The investigation is only made to give a first appraisal about the cost feasibility of the building methods for an extended high-rise project. The stated values can differ for various projects.

For the budget of the project the BAR-method will be used. The BAR-method is an often used method in the master of real estate and housing at the faculty of architecture of the TU Delft[14] to determine the value and quality of a building object. BAR is an abbreviation of 'bruto aanvangsrendement' which is translated as gross initial efficiency. The value is determined by the net rental profit divided by the total investment. The equation therefore looks like the one below

$$BAR = \frac{net\ rental\ profit\ (€)}{total\ investment\ (€)}$$

The total investment for this investigation will be presumed equal to the budget which is available for the extension on top of the Oval Tower. With help of a document[15] in which the BAR number will be determined for the individual regions in the Netherlands a percentage will be found. The percentage for Amsterdam is between 6,2% and 8,0%. Because of the attractiveness of the location a percentage of 6,5% will be taken into account.

Furthermore the net rental profit can be determined by multiplying the rental price with the total amount of square meters. The rental price will be chosen as €200,-/m^2/year[13]. The total amount of additional square meters is 2000m^2.

The budget of the extension now can be estimated and is equal to:

$$total\ investment = \frac{net\ rental\ profit}{BAR} = \frac{200 * 2000}{6{,}5\%} \approx €6.000.000,-$$

[14] (Overbosch, 2012)
[15] (DTZ Zadelhoff, 2012)

The total building costs can be calculated with help of estimated values. According to various sites[16,17] the costs for high-rise projects with more than 20 building levels is about €1500,-/m^2 BVO.

Peildatum: januari 2012		
Kantoorblokken		
Kantoorvilla	**€ 1.276**	/ m² BVO
Kleinschalig kantoor	**€ 1.131**	/ m² BVO
Kantoorblok [< 8 lagen]	**€ 1.106**	/ m² BVO
Kantoorblok [< 12 lagen]	**€ 1.137**	/ m² BVO
Hoogbouw [< 20 lagen]	**€ 1.336**	/ m² BVO
Hoogbouw [> 20 lagen]	**€ 1.426**	/ m² BVO
Bijzondere vorm [< 15 lagen]	**€ 1.435**	/ m² BVO
Bijzondere vorm [> 15 lagen]	**€ 1.491**	/ m² BVO
Brug-gebouw (< 8 lagen)	**€ 1.444**	/ m² BVO
Brug-gebouw (>8 lagen)	**€ 1.543**	/ m² BVO

figure 8.2 cost estimation office buildings januari 2012

Because of the complexity of the structure, the additional crown and the demolishing of various elements on top of the building, for a first costs estimation €2000,-/m^2 BVO will be estimated. For the extension on top of the Oval Tower this means a building price of €4.000.000,-

In this price also the general costs for the building site are admitted. In alternative I and II the building cranes which will be used are common building cranes for high-rise projects. For these two building methods it can be said the budget (€6.000.000,-) > total building costs (€4.000.000) and a positive feasibility will be obtained.

In the price which is determined above, the general building site costs are implemented as well. For the third building alternative which is adopted in the trade off matrix, these costs differ from the first two alternatives. According to Bouwend Nederland[18] the general building site costs are about 12% of the total building costs.

Tabel:
Algemene BouwplaatsKosten (ABK) van woningbouwprojecten als percentage van de totale directe projectkosten[1)], exclusief het aandeel van de ABK daarin, naar projectaard.

Projectgrootte	Woningbouw		Totaal woningbouw
Aanneemsom	Nieuwbouw	Herstel, verbouw, (groot) onderhoud, renovatie	
x € 1.000	%ABK	%ABK	%ABK
250 – < 1.000	17,8	12,3	16,0
1.000 – < 5.000	14,4	13,4	14,3
5.000 – < 10.000	12,7	-	12,7[2]
Totaal	13,6	13,1	13,5

[1)] Gewogen op basis van de directe bouwkosten.
[2)] Het betreft hier uitsluitend nieuwbouwprojecten.

figure 8.3 general building costs

[16] (Bouwkostenkompas)
[17] (Bouwkosten online)
[18] (Bouwend Nederland)

In the third alternative the extension will be build up on ground floor which reduces the complexity of the building process. For the total building costs of the third alternative the value of €1500,-/m^2 BVO will be taken into account. The total building costs will be €3.000.000,-. A reduction of 12% will be made because of the general building site costs will be calculated for the big crawler cranes which have to be used to lift the extension. The total costs without equipment therefore will be estimated as €2.684.000,-

For the total costs of the third alternative a cost estimation has been made by the company which rents the Demag CC12600 cranes.[19] In consultation with Mammoet the price has been determined to lift the extension with help of crawler cranes as one piece to the top of the building. The price of the material and the equipment for this building method exceeds the budget. Therefore the third building method from a first cost estimation is not feasible.

From here it can be concluded that from a first cost estimation the first two alternatives seem feasible. For the third building method it can be said from a costs perspective, the alternative is not feasible because the building costs combined with the costs for the equipment are higher compared to the budget.

The budget is determined with decent values, the cost estimation for the first two alternatives is not accurate. Building costs are determined according to prices which are applicable to general high-rise projects. However with this investigation it is not intended to give exact costs. The costs are used to give a first idea of the feasibility. The accuracy of the equipment costs for the third alternative are discussed with a cost expert, the budget and the price for the crawler crane are determined with accurate values.

8.1.3.1 Link to diagram

In the previous section costs have been determined. Various aspects of the financial inventory made in the main diagram of this thesis were taken into account. The budget was determined with a BAR-method, furthermore the additional income, constructive and architectural costs have been roughly estimated.

With the investigation it was not intended to give exact costs. The costs are used to give a first idea of the feasibility. The determined values can only be adopted to a single project, the Oval Tower. Furthermore the indirect income has not been taken into account. For a general feasibility therefore it can be concluded:

- Additional research throughout the financial inventory is needed to give a thorough answer to the feasibility of an extension on top of a high-rise project.

Financial inventory
- Life span
- Budget
- Additional income
    - direct
    - indirect
- Constructive and architectural costs
- Grounds costs

[19] (de Jong, 2012)

8.1.4 Time

Alternative III will be constructed on ground floor. Because the extension is build up on ground floor, the new structure can be reached directly from the ground on the inside as well as the outside of the extension. Furthermore less delay because of wind caused by individual lift movements will occur for alternative III. Besides the structure of the extension also the crawler cranes have to be built up. However this can be done separately from the extension. From here it can be concluded the shortest building time will be established when alternative III will be taken into account.

8.1.5 Risks

For lifting the extension as one piece, bigger risks occur. As can be read in the [LS] the lifting movement is a very complex process in which failures are highly undesirable. Furthermore contractors often will have more experience with the other methods. Building an extension with a method in which the risks are high only can be accounted for in exchange of high advantages.

8.2 Final choice construction methodology

With help of the previous chapters the trade off matrix now can be filled in and is given below.

Construction Methodology	I, conventional building method	II, crane on top of the building	III, lifting extension as one piece
Standard criteria construction methodology			
Access to site	0	0	+
Storage at site	0	0	+
Maneuvering space at site	0	0	+
Subsoil capacity at site	+	0	-
Lifting operation, sensitivity to wind	0	0	-
Tolerances (ability to absorb)	0	0	-
Environmental impact: noise/transport	0	+	-
Additional criteria construction methodology with regard to extension			
Operability building during construction	0	0	+
Nuisance on ground floor to users surrounding buildings	0	+	-
Possibility to pour concrete	+	0	+
Attachment installations	+	-	0
Attachment crown to current structure	0	0	+
Dismantling of equipment	+	-	0
Construction time / costs / risks			
Costs	+	+	-
Accuracy cost estimate	0	0	+
Construction time	0	0	+
Risks	+	+	0
Final judgement	++++++	++	++ however not attractive due tocosts

For the final construction methodology a choice will be made for the conventional building method.

8.3 Elaboration plan

In the previous section it has been determined for the final construction methodology a conventional building method will be chosen. In this chapter a description of the elaboration plan will be given. The description will be given to investigate most critical points in the process. With help of this investigation conclusions and recommendations will be made with regard to the construction of extensions on top of existing high-rise projects.

For this elaboration plan in the previous section various assumptions have been made. Below a list with assumptions which are important for the elaboration plan.

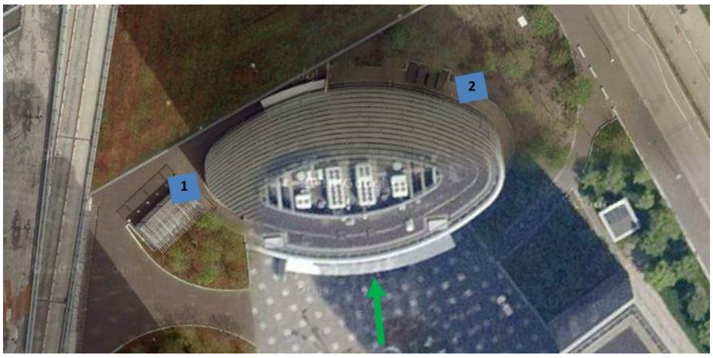
- A conventional building method will be used in which a building crane will be placed alongside the building.
- For the current building two building cranes are used. The two locations on which these building cranes where situated can be used for the building cranes in the situation in which an extension is going to be build.
- The building has to stay operational during the elaboration phase, the time the building will be closed because of the elaboration has to be minimized.

Furthermore in this chapter assumptions will be made with regard to the hoisting movements along and above the building.

- Hoisting alongside the building is allowed, additional attention has to be paid that rotation of elements does not lead to collision with the existing building.
- Fall protection has to be installed for small falling objects.
- For vertical transport the construction workers have to be separated from the users of the existing building.

With help of the assumptions above, in this chapter a process in which the Oval Tower can be extended will be described. The investigations made in the [LS] and the previous sections will be used as a start of point for this investigation.

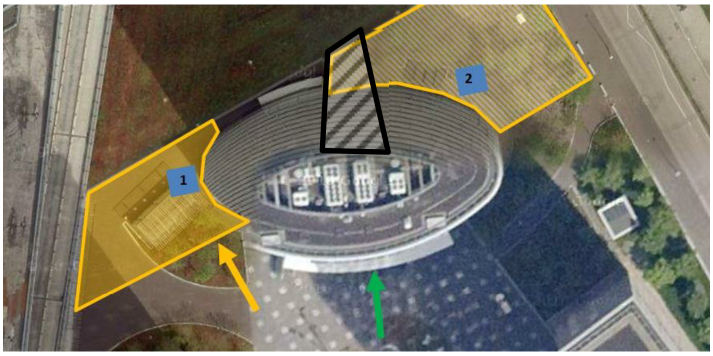
In the figure below an existing top view of the Oval Tower is depicted. The entrance for current users of the building is depicted. The two rectangular blue surfaces are locations on which building cranes where installed during the elaboration of the existing building.



entrance users current building

location foundation building cranes

With help of the current situation, the building plot has been determined. For vertical transport separation of construction workers and existing users of the building is desired. An external building lift therefore will be situated on location 1, this building lift can be used for construction workers and light weight building materials. The location around the building crane is only accessible for construction workers and can be used for storage. On the second location a building crane will be placed. The location of the crane is chosen close to the road because of the elements which have to be transported via the road.



entrance users current building

entrance construction workers

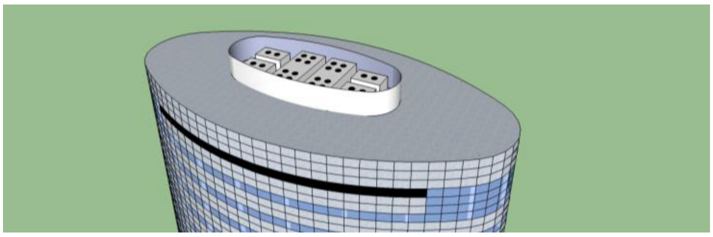
location foundation building cranes

building site, stacking of large elements

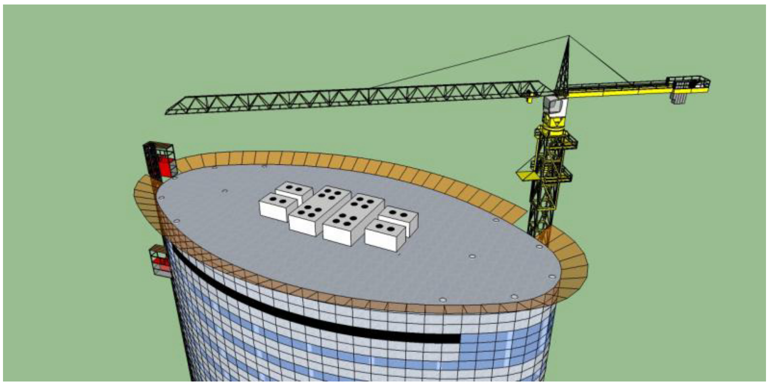
building site , stacking of small elements

hoisting zone

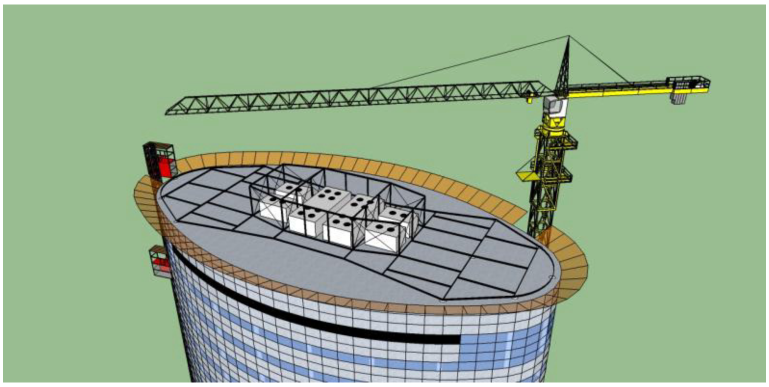
An impression of the current Oval Tower can be seen in the figure below. On top of the existing roof installations are present. Small protection is present around the existing installations. In the steps which will be described afterwards the building stays operational.



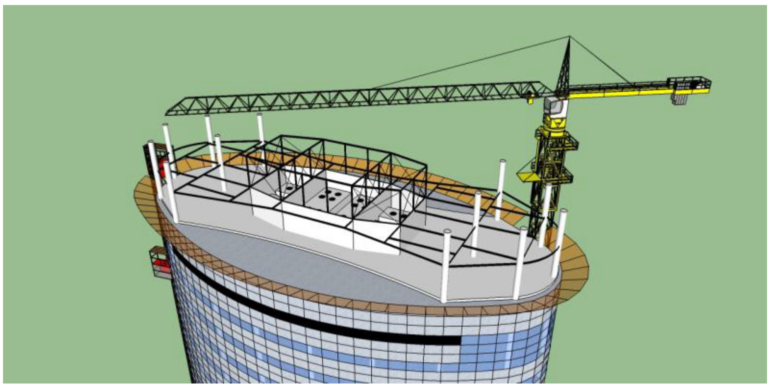
First the building lift and building crane will be placed. The existing protection around the installations will be removed. Falling protection will be attached to the edges of the tower. Furthermore elements are connected to the existing columns because of the new beam grid which will be placed.



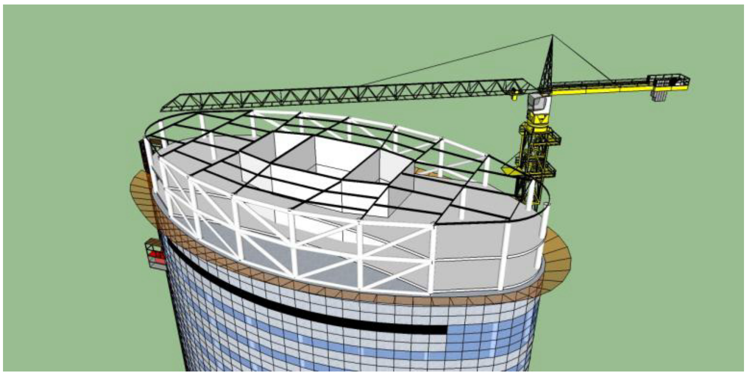
The steel floor of the extension will be placed. The steel elements of the core will be build up, the installations on top of the existing building have to stay operational and therefore will be removed in a later phase. The core first will be build up around the existing installations.



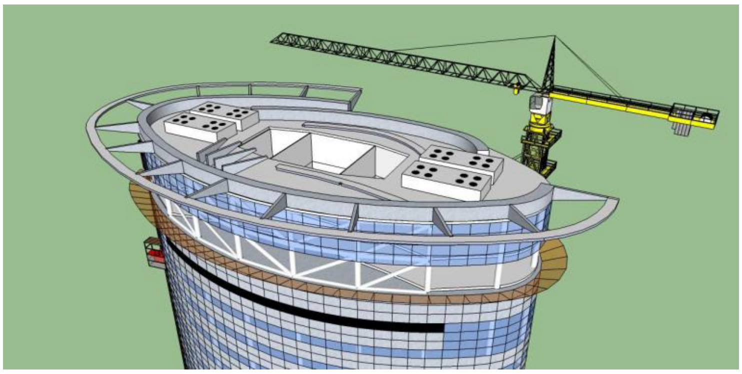
The columns will be placed and, with help of steel girders of the 1st floor of the extension, attached to the core. Furthermore the steel plates can be placed and the concrete can be poured to finish the composite floor system. The concrete can be casted with help of pipes which are attached to the building crane. At ground floor the pipes are attached to an installation which is able to pump the concrete to the top of the building. The walls of the core will be finished. Furthermore the steel elements of the core of the second floor can be build up.



The steel frameworks can be hoisted up and attached to the existing columns. Furthermore the concrete of the second floor can be casted and the steel frame of the top floor will be attached. The wall elements of the second floor will be accomplished.



The next step is finishing of the top floor. Furthermore the glass facade elements of the second floor can be placed. Also new installations will be placed on top of the roof floor. The installations have to be tested. The crown can be partly build up. The remaining elements of the crown can be lifted up to the roof floor. The crown can only be finished when the building crane is dismantled otherwise conflicts will occur with the location of the crown and building crane.

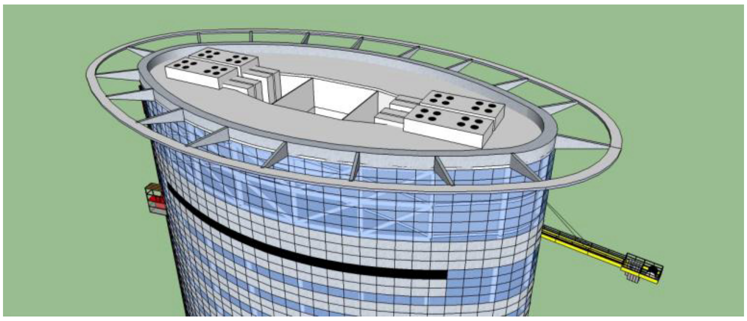


When all steps above are elaborated, the building has to be closed down. When the building is closed down, the following actions have to be elaborated:

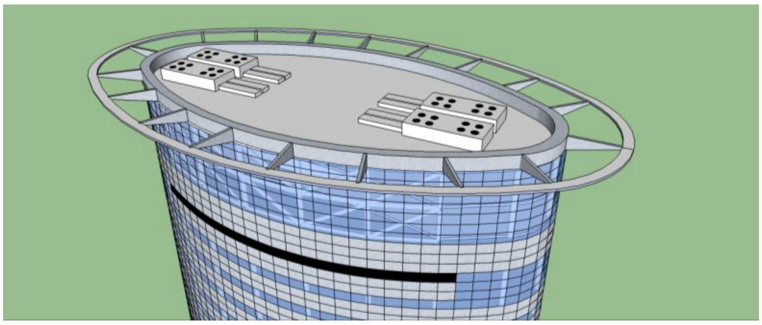
- Disconnection and removal of old building installations through core
- Attaching of existing ducts to new building installations

At the same time the following actions can be elaborated:

- Finishing of the remaining crown elements
- Dismantling of fall protection
- Placing the facade elements of the 1st floor.



When these actions are elaborated the building can be operational again and will be reopened. The building crane can be removed. After the reopening the core including the new ducts can be finished. However the external building lift will stay operational. After finishing of the interior and the remaining elements, the building lift also can be removed. An impression of the final extension on top of the Oval Tower is given in the figure below.



8.4 Conclusions

In the previous sections an investigation to the construction methodology has been made. It is intended to give answer to the research question below:

- *Which construction method can best be used to extend the Oval Tower?*

An investigation has been made with help of a trade off matrix. From this trade off matrix it is concluded a conventional building method is most suitable to build a construction on top of the Oval Tower.

A remark has to be made the criteria are subjected to environmental influences and influences with regard to the design. For a different building project, it is possible another building method is more suitable.

Furthermore in this chapter a plan for the elaboration of the extension has been made. In this plan it is intended that during the elaboration phase the building has to stay operational or the closure of the building has to be minimized. From the investigation it follows that it is not possible to build the extension without closure of the building. The operations stated below are leading for closing down of the building. It is recommended to avoid or minimize the time in which the operations below will be elaborated.

- Replacing / dismantling of old building installations
- Finishing of the crown (because of conflicts building crane)
- Dismantling of fall protection
- Placing of final façade elements

9 Conclusions & recommendations

This chapter provides the conclusions and recommendations of this master thesis. First general conclusions with regard to the main research question and final diagram will be provided. Secondly conclusions about the way the diagram was used for a case-study of an existing high-rise project on which an extension was situated are made. Finally following from the conclusions, recommendations for further research will be presented at the end of this chapter.

9.1 General conclusions

The main research question of this master thesis is as following:

- *Which factors determine the feasibility of an extension on top of a high-rise building?*

To answer this question a diagram was developed which shows all the factors that determine the feasibility of an extension. The factors can be divided in three categories; functional, financial and technical inventory. The factors which determine the technical inventory have been described in detail throughout the master thesis whilst the functional and financial were not discussed to the same extent within this thesis..

The first factor which determines the technical inventory is the use of materials. Steel and timber are favorable materials for making a construction on top of an existing building. The choice for a load bearing system and floor system are related to the architectural design of the extension.

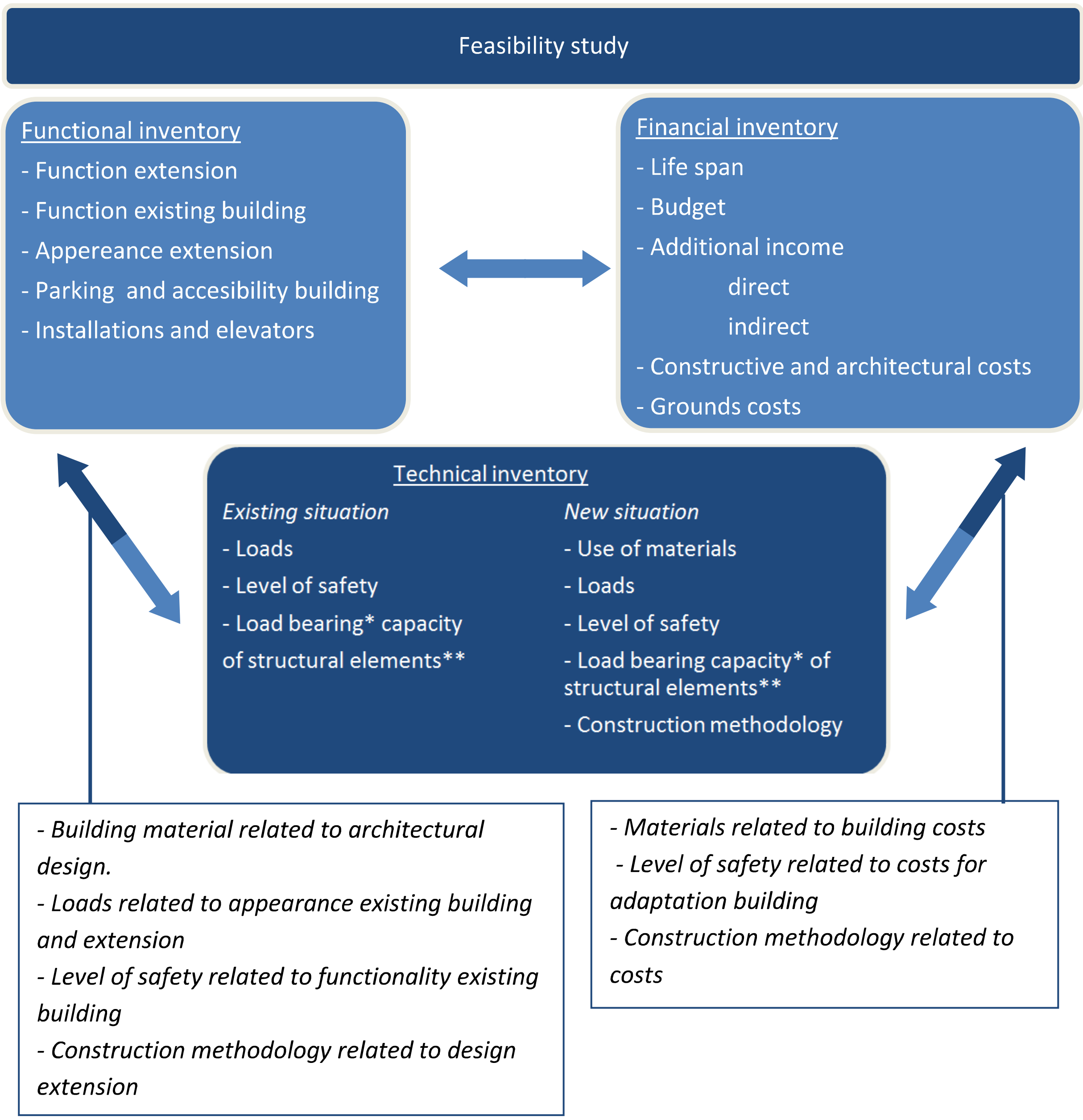
Second factor to consider are the loads which have to be applied to the existing building and its extension, the differences between the Eurocodes and older building standards are of importance. For this cause the current Eurocode has been compared to the older Dutch national code NEN6702. This comparison showed that differences are generally very small. A minor difference can be found in the design values of load due to wind thrust, which are slightly higher in case of application of the Eurocode. Naturally, the shape of the extension also has an impact on the values of the design wind loads. A sculptured building top, varying the shape of the building, openings in the top of the building, corner modification and orientation of the building in relation to the leading wind direction are opportunities to reduce the wind loads.

The third factor is the level of safety which will be prescribed by the way a building can be interpreted. For existing buildings the following systematic judgment is recommended: for existing buildings the level of safety for new buildings should be taken into account, unless the costs to fulfill the requirements of new buildings are disproportionately high. In this case the safety level for existing buildings is sufficient. The way the load factors have to be applied are not unambiguous and are related to additional building costs.

To get a profound insight in the load bearing capacity of structural elements, calculations of the building have to be made for strength, stability and stiffness. Most structurally important elements are foundation piles, foundation beams, vertical load bearing elements, roof floor, structure extension and connections between new and existing structure. Three methods are shown in which ways an existing building can be strengthened. These methods are adding additional elements inside the building, adding elements around existing structure and making use of glued reinforcement are options which according to the design of the current structure can be used.

The final factor which determines the technical inventory is the construction methodology. An extension can be built with help of various construction methods, to compare these methods a trade-off matrix can be used. Besides standard criteria (access to site, storage at site, maneuvering space at site, subsoil capacity at site, sensitivity to wind, tolerances and environmental impact) additional criteria because of an extension can be adopted. Additional criteria used are operability building, nuisance on ground floor, possibility to pour concrete, attachment installations, attachment new structure and dismantling equipment. Furthermore a comparison on time, costs and risks can be made.

During the investigation which lead to the above factors, which determine the technical inventory it has been found, various variables in this inventory are highly related to variables in the functional and financial inventories. Therefore it can be concluded it is only possible to give a thorough answer to the question whether a project is feasible if all three inventories are investigated in detail.



* the load bearing capacity of the structure has to be investigated for strength, stability and stiffness

** the most important structural elements are the foundation piles, foundation beams, vertical load bearing elements, stabilizing elements, roof floor, structure extension, connections

9.2 Conclusions Oval Tower

The Oval Tower is a high-rise building in Amsterdam with a height of about 100m for which a preliminary design was available. With help of the diagram provided in the previous section a case study has been made in which an extension on top of the Oval Tower was situated. In this case study it is intended to give answer to the following question:

- *Is it structurally feasible to extend the Oval Tower with two floors and a crown on top of the existing building?*

In the previous section it is concluded that because of the strong relation between the three inventories a thorough answer to the question whether a project is feasible can be given if all three inventories are investigated in detail. In this thesis the technical inventory is taken into account in detail. However the functional and financial inventories are briefly taken into account. An unambiguous answer the question above can therefore not be given. The conclusions of the technical inventory of the Oval Tower are shown in the next sections.

9.2.1 Existing building without extension

To make a technical inventory of the Oval Tower first the existing situation must be elaborated. Drawn conclusions are summarized below.

- The core contains overcapacity, additional compression is allowed. However additional tension in the lower part of the core becomes a critical aspect.
- Various columns which are not subjected to wind load contain overcapacity.
- The columns which are subjected to wind load are critical elements, therefore it is recommended to not additionally load the columns which are subjected to wind load.
- The foundation piles below the core contain overcapacity.
- The foundation piles below the existing columns contain overcapacity.
- For top deflection and dynamical behaviour no problems are expected to occur.

The above conclusions provide an answer to the secondary research question as provided below:

- *What is/are the most leading structural aspect(s) in the current design of the Oval Tower?*

A leading structural element in the current design of the Oval Tower is the core. Additional tension in the core becomes a critical aspect. Furthermore the columns subjected to wind load are leading structural elements.

- *Where does the current Oval Tower contain overcapacity?*

The columns not subjected to wind load contain overcapacity. All foundation piles below the core and columns contain overcapacity. Furthermore additional compression is allowed in the core.

9.2.2 Existing building with extension

Conclusions related to the extension of two floors and a crown as discussed earlier are the following ones:

- In the existing situation in ultimate limit state the lower three floors are subjected to tensile forces. With an extension the lower six floors of the core will be subjected to tensile forces.

To deal with the additional stresses the following recommendation is provided:

- First it must be checked whether the reinforcement which is present in the core is able to deal with the additional stress. If the core is unable to deal with this additional stress two solutions can be used to mitigate for this. First the adjustments of the structure with help of additional elements, secondly the use of glued reinforcement. A detailed investigation has to be made to which option is most suitable. Costs most likely will be a leading decision factor.

Furthermore the foundation below the core has been taken into account for the building with extension, leading to the following conclusions:

- In the leading zone, in which the load bearing capacity of the soil is the lowest, all foundation piles meet the strength requirements. This means all foundation piles below the core and the abutments meet the requirements for strength, no adaptations to the foundation piles below the core have to be made because of the extension.

Various columns and foundation piles below the columns will not meet the strength requirements. With slight changes to the layout of the floor plan positive unity checks will be obtained. From here the following recommendation can be made:

- If a building element from the existing building does not meet the requirements after addition of the extension, first try to adopt the design in a way the requirements will be met.

Furthermore the top deflection and dynamical behavior have been taken into account; the following conclusions have been drawn:

- The requirements for top deflection will be met in case an extension will be built on top of the Oval Tower.
- For the Oval Tower with extension, the values for dynamical behaviour stay below the allowed values.

The above leads to answering the secondary research question below:

- *<u>Does the current building has to be strengthened because of the extension?</u>*

If the reinforcement in the core is unable to deal with the additional tensile forces, the core has to be strengthened. Furthermore a few columns have to be strengthened. The foundation below a single column has to be strengthened.

The technical inventory has been finalized with an investigation into the construction methodology. The investigation is executed with help of a trade off matrix. From this trade off matrix it is concluded a conventional building method is most suitable to build a construction on top of the Oval Tower.

A comment has to be made as the criteria are subjected to environmental influences and influences with regard to the design. For a different building project, it is possible another building method is most suitable.

Furthermore a plan for the execution of the extension has been made. In this plan a starting point is to keep the building operational as much as possible during construction of the extension. However, research showed it is not possible to build the extension without closure of the building. The building operations stated below are key reasons for the closing down of the building. It is recommended to avoid or minimize the time in which the operations below will be elaborated, especially during normal office hours.

- Replacing / dismantling of old building installations
- Finishing of the crown (because of conflicts building crane)
- Dismantling of fall protection
- Placing of final façade elements

The secondary research question which belongs to this chapter is the one below:

- *Which construction method can best be used to extend the Oval Tower?*

To extend the Oval Tower with two floors and a crown a conventional building method is most suitable. The building method will be characterized by a building crane which will be placed alongside the building.

After finalizing of the technical inventory an answer can be given to the following research question:

- *Is it structurally possible to extend the Oval Tower with two floors and a crown?*

With help of the technical inventory it can be concluded that it is structurally possible to extend the Oval Tower with two floors and a crown, however various building elements have to be strengthened.

9.2.3 Adjustments diagram after case study

After the case study has been made it can be concluded that the diagram as presented before can help in determining the structural possibilities to extend the Oval Tower. During the case study the diagram was successfully applied, making several assumptions on the functional and financial inventory.

To give a profound answer to the question whether it is feasible to extend a high-rise building project additional research in the functional and financial inventory is needed. This will be further explained in the following section in which the recommendations will be discussed. With help of this additional research the diagram can be extended and finalized.

9.3 Recommendations

In the previous sections the main conclusions of this thesis have been presented together with answers to the research questions. Main conclusion is that it is structurally possible to extend the Oval Tower with two floors and a crown when various building elements have to been strengthened.

The feasibility of the extension on top of the building besides the technical inventory also will be determined by the functional and financial attractiveness of the extension. The technical inventory has been taken into account in detail in the thesis whilst it was made clear a strong relation between the three factors is present. A final answer to the question whether it is feasible to extend the Oval Tower therefore cannot be given as these factors have not been researched on a similar level.

To fully understand whether it is feasible to extend a high-rise project with two floors and a crown, the following recommendations for further research are made:

- Investigate the variables which determine the functional and financial inventory in detail.
- Investigate the relations between the functional and financial inventory.

When the above recommendations have been answered the diagram as presented earlier can be completed. To determine the feasibility of an extension on top of a high-rise building the entire diagram must be taken into account.

A technical inventory to the variables which determine the feasibility of the Oval Tower have been elaborated. In this investigation the core and columns subjected to wind load are leading elements. For the construction method a conventional building method can be used best to build the extension. These two conclusions only count for one specific situation, the Oval Tower. It is interesting to see whether leading elements in the investigation count for high-rise projects in general. Therefore the following is recommended:

- Apply the diagram to various high-rise projects. With help of the results, general conclusions about the leading aspects of high-rise projects can be drawn.

For the investigation into materials three common building materials namely; steel, timber and concrete have been compared. A disadvantage of concrete is the big self-weight, however when ultra high performance concrete will be used a higher strength can be obtained which causes less self-weight. Furthermore other building materials which are very low in self-weight might also be useable to build an extension. This leads to the following recommendation.

- Investigate whether building materials other than steel, timber or regular concrete which are low in self-weight can be applied as building material for an extension.

The level of safety will be prescribed by the costs which are needed to adjust the existing building structure to the requirements imposed in the current building codes. When the costs are disproportionally high it is allowed to calculate with older building standards. In the case study calculations of the old structure with the current building codes, lead to strengthening of plenty of columns and existing foundation piles. It is assumed this leads to disproportionally high costs. Disproportionally high still is a notion which can be interrupted in various ways. Therefore the following recommendation will be made.

- Investigate whether it is possible to quantify the notion ‘disproportionally’ and express this statement in terms of total building costs.

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