The Zalmhaven tower

An investigation on the feasibility of prefab concrete in a high rise building

Literature study

Sven ten Hagen

Student number: 1364049

Date: 29-11-2012

Author:

Sven ten Hagen
Sventenhagen@gmail.com

Graduation committee:

Prof.ir. R. Nijsse	ABT/ Delft University of Technology, chairman
Prof.ir. A.Q.C. van der Horst	BAM Infraconsult/ Delft University of Technology
Dr.ir.drs. C.R. Braam	Delft University of Technology
Ing. H.J Hoorn	Zonneveld ingenieurs
Ir. D.C. van Keulen	Ingenieursstudio DCK/ Delft University of Technology

Preface

This literature study is the first part of my master thesis research at the faculty of Civil Engineering and Geosciences at Delft University of Technology. With this thesis I will conclude my study Structural Design at the section Building and Structural Engineering.

The literature study contains the most important aspects that are required to design a 200 meter high prefabricated tower and the accompanying construction methodology. In report 2 of the literature study, the floor plans, shear deformation calculation, calculations made by Zonneveld ingenieurs and the Fault Tree Analysis are combined. The research report contains the actual design of the Zalmhaven tower and construction methodology.

I would like to thank the graduation committee for their time, guidance, knowledge and enthusiasm. Without their effort I would not be able to present the reader this report. I would also like to thank Zonneveld ingenieurs for providing me the opportunity to graduate at a first class engineering company. I sincerely thank all the employees for their support and guidance.

Sven ten Hagen,

Rotterdam, November 2012

Table of contents

1 Introduction

1.1 General

High rise buildings nowadays determine the skyline of a city. Started as a symbol of status in the growing economy of America, the Home Insurance Building in Chicago from 1885 can be considered as the first skyscraper in the world. Today, the Middle East and Far East house the tallest buildings in the world.
Because of the growing world population and economic pressure, the urban areas increase in their size. At the end of the twentieth century around 46% of the world population lived in urban areas. It is predicted that this number will grow [Hayden, 2009:10-29]. High rise buildings are the current solution for the lack of space: large areas with a small footprint. The Netherlands is not considered as a player on the high rise market, but the Dutch skyscrapers are slender compared to buildings in other countries. This high slenderness is the result of the Dutch building code on the amount of light entering the building and the lack of extraordinary loads, such as earthquakes and hurricanes.

But what is the most suitable material for a high rise building? Steel and concrete are the two most applied options. Steel made high rise possible but nowadays most high rise buildings are constructed from concrete. Concrete is very suitable for these buildings because of its high mass, large damping factor and lateral stiffness. Concrete buildings are more stable and its occupants are less able to perceive building motion. When one chooses for concrete, there is a second question to be answered: prefab or cast in situ? Cast in situ is very common in the Netherlands, but prefab is becoming more and more important. The Prinsenhof[1], Het Strijkijzer, Waterstadtoren and the Erasmus MC tower are a few examples of this trend.

1.2 Problem description

The demand for high rise buildings is growing. Furthermore, they become more and more slender with growing heights. Building costs and time are under large pressure and experienced construction workers are hard to find. In the recent years, most industries have optimized there process. Research of ING [ING 2010] has shown that this optimisation has not occurred in the construction sector (there is optimisation to a certain level, but this is not radical enough). To survive the financial crisis a change of mind is needed. Prefab in combination with an integrated transport process could be part of this change of mind.

Zonneveld ingenieurs is interested in the possibilities of prefab concrete at one of their projects: the Zalmhaven tower in Rotterdam. The Zalmhaven tower was originally designed with cast in situ concrete. To cast the building on site, a tunnel system would be used. But this tower has potential to be built in prefab concrete. The following aspects explain why:

- The building is rectangular, has sixty-two floors and consists out of regular floor plans. These regular floor plans and the amount of floors are beneficial for the repetition factor.
- The building site is located in the centre of Rotterdam. The site is rather small and it is surrounded by dwellings.

[1] The Prinsenhof consists out of 8 towers. Only the 4 office towers are made out of prefab concrete.

- The construction time is reduced, because at the buildings site the construction workers only have to assemble the building. By using less different elements there will be more repetition in the factory and on the building site. After the elements are constructed they pass a quality check at factory and this results in a swift and efficient assembly process. Simple and fast connections reduce the time needed for the assembling even more.
- The market for dwellings is currently under large pressure. The financial crisis is responsible for the fact that this building isn't constructed yet. Reducing the construction time results in apartments that are easier to sell. For example, a dwelling that is finished within eighteen months is more attractive than a dwelling that is finished within thirty months.
- Reducing the construction time could also result in a higher profit. Looking at the current market for dwellings, this will probably be the main argument to use prefab concrete instead of cast in situ concrete.
- The construction process is shifted from building site to factory: construction becomes assembling. The conditions in the factory are better and this part of the building process becomes independent of the weather. Furthermore, at the factory the building process is more centralized: the transport of material and equipment is reduced. The material arriving at the building site mainly consists out of finished prefabricated elements and this results in a cleaner building site where less area and personnel are required.

But a 200 meter prefab building has never been build before. And current transport systems for prefab have been used for a maximum height of 136 meter. Why is the largest prefab building only 136 meter high (approximately 1/6 of the tallest building: the Burj Khalifa) and are tower cranes used at het Strijkijzer or a hoisting shed of the Erasmus MC tower sufficient or is a new system required? These questions result in the following research question:

Is it structurally and logistically feasible to construct a 200 meter tower in Rotterdam?

This report contains the literature study were the available literature is studied to gather essential information and knowledge for the research report. The research question for this report is as following:

What do I have to know in order to design the structure and construction methodology of a 200 meter tower in Rotterdam?

1.3 Goals

To solve the problem description several goals are formulated.

Main goal:

- The main goal of this thesis is to investigate the structural and logistical possibilities of a 200 meter tower with prefab concrete.

Sub goals:

- Design a prefab stability structure that is capable to support a high rise tower of 200 meter.
- Design an optimal floor system with fast construction time.
- Design a floor-wall and a wall-wall connection that ensures a fast construction time and a second load bearing system.
- Design a transport system that supports the building method with the least amount of limitations.

Sub-sub goals:

- Create a time schedule and calculate the costs.
- Design a structure that is durable and sustainable.
- Design a sustainable transport system.

1.4 Boundary conditions

This master thesis is based on the Zalmhaven tower project and this results in a custom design. The next aspect that is predefined is the use of prefab concrete. Zonneveld ingenieurs already made several different concepts and they are interested in a prefab concept. The results of this research will continually be compared with those concepts. Due to these applied assumptions and principles, it's difficult (but not impossible) to provide generally applicable quantitative conclusions for all high rise buildings. The European building codes are used in combination with the Dutch National Annex and local requirements.
The master thesis is based on the main and sub goals. The durability and economical aspect are included to create a complete report, but they are not the main goal of this thesis.

1.5 Outline of the report

The literature study report is divided into two parts: "the structural design" and "the construction methodology". During the integral design both aspects are considered simultaneously, but in this report they are divided to increase the readability.

The literature study will start with the research description in chapter 1. In chapter 2 a project description is given of the current design of Zonneveld ingenieurs. Chapter 3 continues with several reference projects and chapter 4 includes interviews with experts. Chapter 5 ends with the building vision that was made before the literature study.

The structural part of the literature study starts with the criteria for a good structural design in chapter 6. In Chapter 7 the loads acting on the structure are determined. The foundation is described in chapter 8. Stability systems in general and for precast buildings are discussed in chapter 9 and 10. In chapter 11 several material properties are discussed. Chapter 12 describes the phenomena progressive collapse and concludes the structural part of the literature study.

The second part of the literature study, devoted to the construction methodology, starts with a composition of criteria in chapter 13. Chapter 14 continues with a reconsideration of the building method. Chapter 15 includes the influence factors of the transport system and chapter 16 continues with a preliminary design of the transport system. The tolerances and cycle time are examined respectively in chapter 17 and 18. Chapter 18 is also the last chapter of the second part of the literature study.

In chapter 19 a short abstract is provided of the entire literature study and chapter 20 concludes with several recommendations for the research phase.

2 Project description

Chapter 2 describes the project and the calculations made by Zonneveld. This chapter starts with an architectural design in section 2.1 and the structural design is next in section 2.2. Section 2.3 continues with the soil conditions and section 2.4 and 2.5 conclude this chapter with two alternative designs.

2.1 Architectural design

In 2004 Zonneveld ingenieurs started with a structural design for the Zalmhaven complex commissioned by Zalmhaven C.V.. Because of financial problems, the building isn't constructed yet. The Zalmhaven tower would be located in the centre of Rotterdam near the Erasmus bridge (see Figure 1).

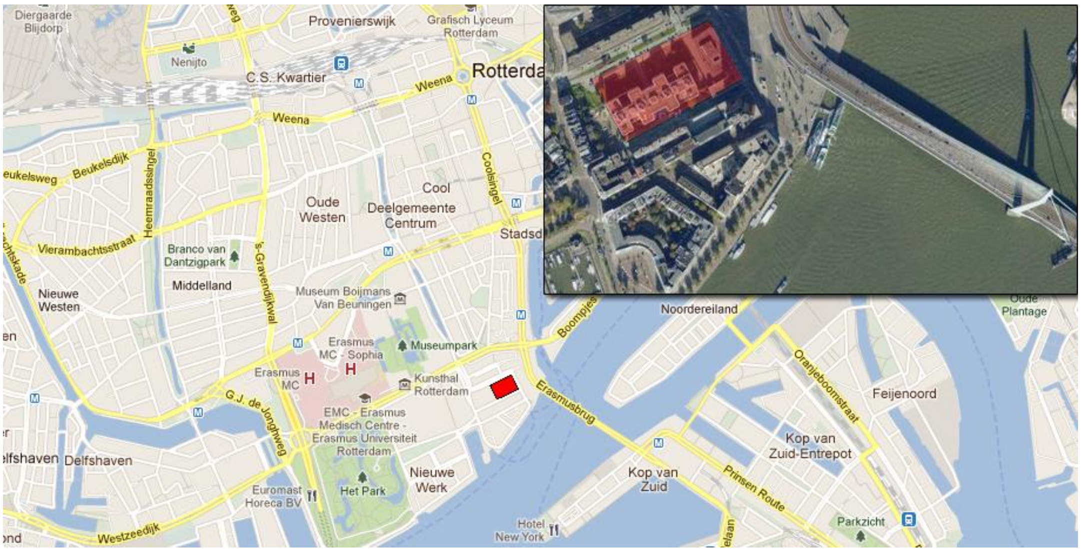


Figure 1 Location of the building plot [Google Maps 2011]

The tower is designed by Diederick Dam from Dam & Partners Architecten. With a height of 190m (62 floors), excluding a steel pole of 29m, this would be the tallest building of the Netherlands. The tower houses dwellings and the offices are located in the Willemsplein building designed by Han van den Bom from KCAP Architects and planners. The low rise buildings surrounding the tower contain more dwellings and a commercial area is added to increase the liveliness. These buildings are designed by Kees Kaan from Claus en Kaan Architecten. Several artist impressions are depicted in Figure 3 and Figure 4.

Figure 2 Artist impression from the Zalmhaven tower and low rise 1 [Top100.nl]

Figure 3 Artist impression from the Zalmhaven tower and low rise 2 [Zalmhaven]

Figure 4, Figure 5 and Figure 6 show a bird's eye view of the building plot. The Hoge Heren with a height of 103m (the two towers on the north side) and the Hoge Erasmus with a height of 93m are clearly visible.

Figure 4 Aerial view from the west [Bing maps]

Figure 5 Aerial view from the south [Bing maps]

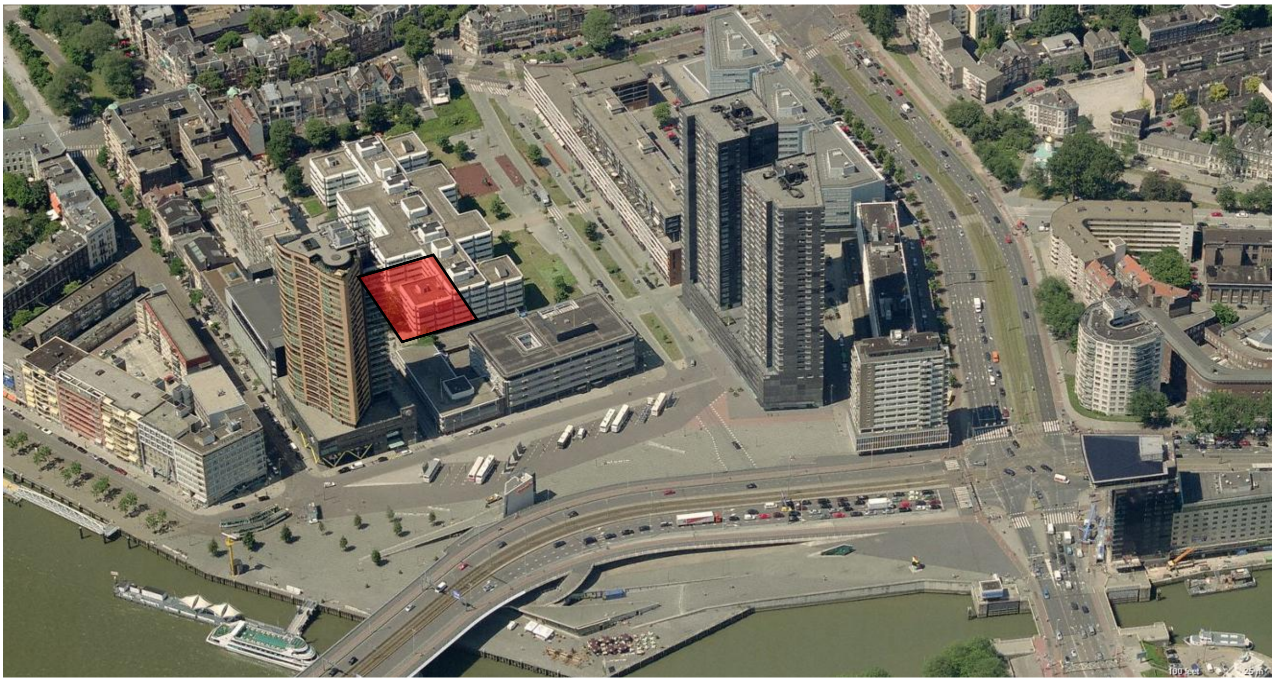


Figure 6 Aerial view from the east [Bing maps]

2.2 Structural design by Zonneveld

The main load bearing system is composed out of cast in situ concrete and the tower is constructed with a tunnel system. Solid floors of 290mm are used in the apartments and 320mm in the core. The floor thickness in the core is enlarged, because it contains a large amount of openings. The maximum floor span is 7800mm. For the stability two 500mm thick walls are necessary in x-direction and three 400mm walls in the y-direction (see Figure 7 and Figure 8). The walls in the x-direction will be reduced from 500mm to 400mm from level 27 till the roof. The stiffness of the load bearing facade is negligible for the stability (facade in the x-direction of Figure 8) and the non-structural facade is supported by the floor edge. The eight prefab balconies per floor level are connected to the floors and the concrete elements are insulated to prevent heat loss (thermal bridge).

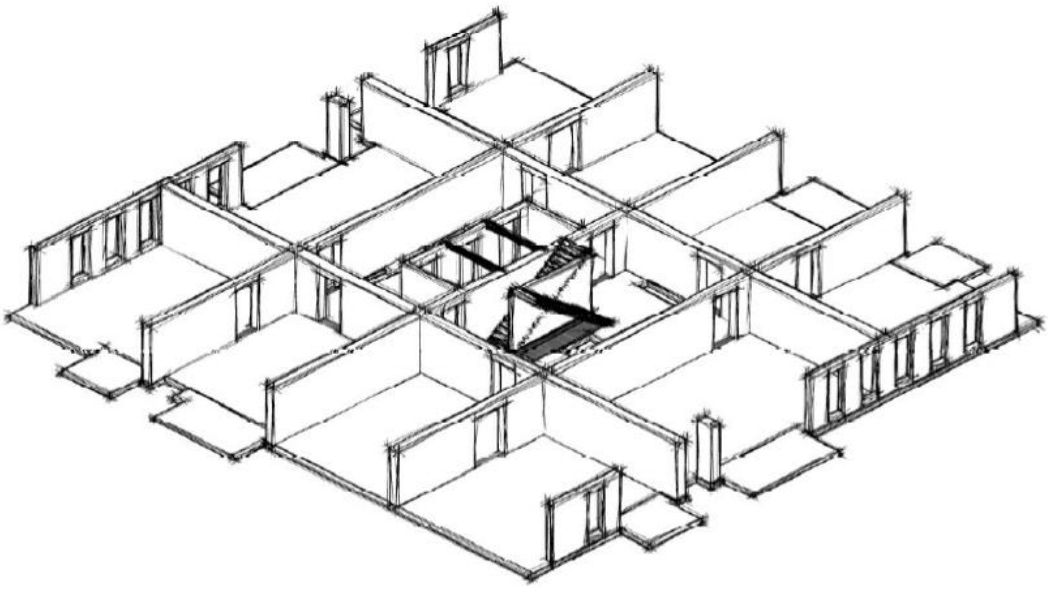


Figure 7 Building layout [Zonneveld ingenieurs]

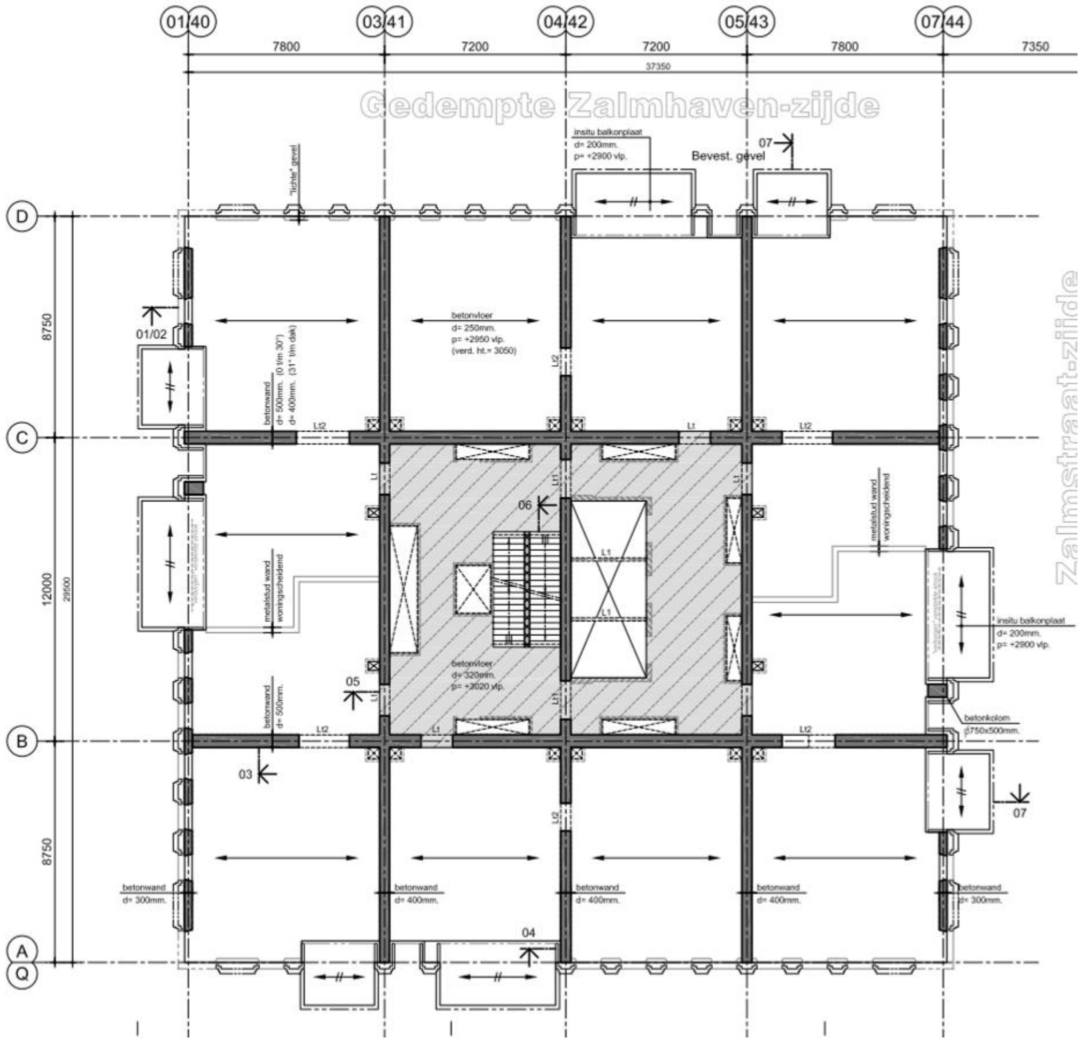


Figure 8 Average floor plan [Zonneveld ingenieurs]

The entire main load bearing system is entered in the 3D finite element program EsaPrimaWin. With this model it was possible to analyze the structure and determine the displacements and the support reactions.

Several concrete strength classes are used, to optimize the design (see Table 1). It's likely that a part of the concrete walls will crack (for example the lintels above the doors). To prevent a Finite Element Method analysis with multiple concrete sections with a different Young's-modulus in one wall, all the walls are calculated with a smeared Young's-modulus.

Table 1 Concrete strength classes in the building

Element	Concrete class	Young's modulus [N/mm^2]
Floors	C28/35	12000
Balconies	C28/35	12000
Walls x-direction d=500mm GF till 25	C53/65	20000
Walls x-direction d=400mm 26 till 35	C53/65	20000
Walls x-direction d=400mm 36 till roof	C45/55	20000
Walls y-direction d=400mm GF till roof	C53/65	20000
Foundation slab	C35/45	20000

The Diaphragm walls in the foundation are modelled as springs. The stiffness of these springs is calculated by MOS grondmechanica in Rhoon and the results are displayed in chapter 2.3.

For the calculations several starting points are used:

Safety class:	3 (dwellings),
Reference period:	50 years,
Fire resistance:	120 minutes.

The following loads are used in the calculations:

Dwelling floor	G	Q
Floor slab (d=250mm)	6.25kN/m^2	
Distributed wall load	0.60kN/m^2	
Finishing	1.50kN/m^2	
Live load		1.75kN/m^2
Total	8.35kN/m^2	1.75kN/m^2

Core floor	G	Q
Floor slab (d=320mm)	8.00kN/m^2	
Distributed wall load	0.60kN/m^2	
Finishing	0.60kN/m^2	
Live load		3.00kN/m^2
Total	9.20kN/m^2	3.00kN/m^2

Wind load
P_w=2.02kN/m^2 (area 2, built-up (bebouwd in Dutch)),
C_{dim}=0.87,
C_{d+z}=1.2,
C_{ww}=0.04,
Phi 1=1.25,
Second order=1.07.

The wind load is reduced to 20% of the normal load in case of a calamity.

The results of the EsaPrimaWin model can be found in Figure 9. The largest displacement occurs in the y-direction (3 walls of 400mm) and they are within the limit of w_{max}=h/500=200/500=0.4m. An overview of all the floor sections can be found in report 2 of the literature study.

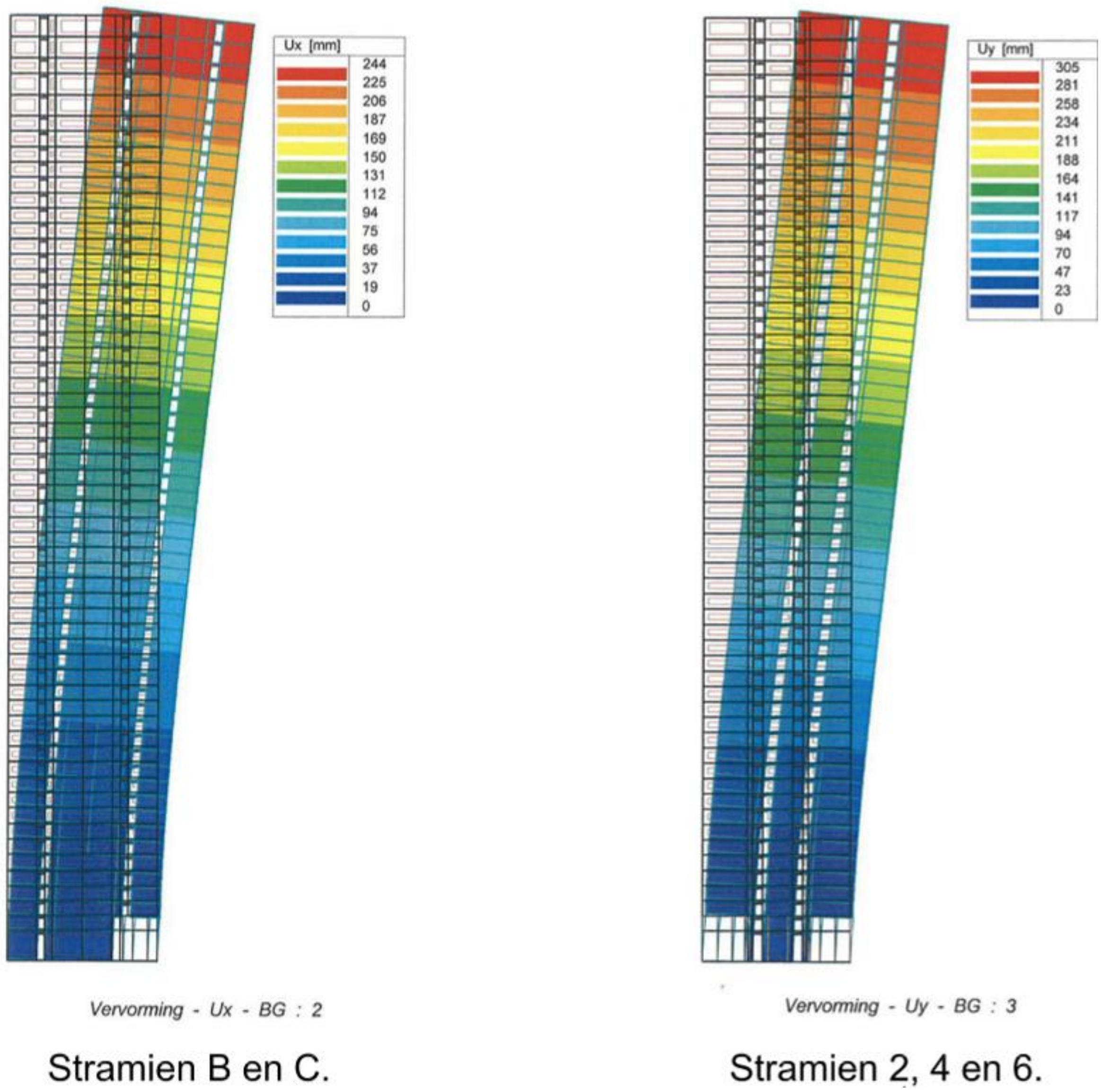


Figure 9 Horizontal displacements in EsaPrimaWin [Zonneveld ingenieurs]

2.3 Foundation and soil condition

The surface level of the soil is located at 3.20m+NAP. From the surface level till 17.5m-NAP, the soil mainly consists of poorly water-permeable sand, clay and peat. The first dense and highly water-permeable sand layer is located at 17.5m-NAP till 40m-NAP. The layer of Kedichem extends from 40m-NAP till 55m-NAP. This layer consists of thick and solid clay. Below the layer of Kedichem a thick layer of sand is located.

Table 2 Soil layer description

Layer	Depth NAP [m]	Description
1	+3.2 to -17.5	Sand, clay and peat
2	-17.5 to -40	Sand
3	-40 to -55	Clay
4	-55 to 60	Sand

Diaphragm walls are used for the foundation of the tower. They are approximately 62.3m long and protrude trough the layer of Kedichem. The first sand layer can't be used for the foundation because the bearing capacity is too low and it would result in large settlements for the surrounding buildings.

By using the second sand layer for the foundation, a stiffer foundation is obtained. This is beneficial, because on average one-third of the top displacements are created by the rotation of the foundation. To connect the Diaphragm walls with the building, a 1m thick foundation slab is used. The surrounding buildings are dilatated from the high rise tower, to prevent problems with settlements. For the foundation 25m long vibro(combinatie)piles are used.

MOS grondmechanica in Rhoon has executed several (deep) cone penetration tests (CPT) up to 60m-NAP. To confirm the test, several holes are bored and analysed in a laboratory. Based on this information, calculations are made for the load capacity and the stiffness of the diaphragm walls.

The load capacity calculations are made with the method of Van Weele and they are executed for two levels: 50m-NAP and 60m-NAP. Table 3 shows the representative values for the shaft friction of the outline of the panels ($F_{r,max,Schacht,rep}$) and the load bearing capacity at two different levels ($F_{r,net,d}$).
A safety factor of 2.5 is applied on the load bearing capacity: $F_{r,net,d}= F_{r,net,rep}/2.5$. The panels are calculated with B30 concrete with $E_{concrete}$=20000MPa. The panel point resistance is limited to $p_{r,max,punt}$=2.5MPa.

Table 3 Results load bearing capacity

	Panel level = 50m-NAP		Panel level = 60m-NAP	
Panel size	$F_{r,max,Schacht,rep}$	$F_{r,net,d}$	$F_{r,max,Schacht,rep}$	$F_{r,net,d}$
[m^2]	[kN/m]	[kN]	[kN/m]	[kN]
0.8x3.3	4403	17080	6096	22635
1.0x3.3	4403	18445	6096	24270
1.2x3.3	4403	19810	6096	25905
1.5x3.3	4403	21860	6096	28360

The settlement and stiffness are also calculated for every panel size. The results are presented in Table 3. During the calculation, the following assumption is made: maximum load per Diaphragm wall element: $F_{s,rep}=F_{r,net,d}/1.3$. Therefore a safety factor of 1.3x2.5=3.25 is applied at the stiffness calculation. As a result, the calculation is made in the stiff part of the load settlement diagram.

Table 4 Results of the settlements and stiffness

Panel level	Panel size	Point settlement	Elastic shortening	Top settlement	Stiffness
	[m^2]	[mm]	[mm]	[mm]	[kN/m]
50m-NAP	0.8x3.3	12	13	25	525650
	1.0x3.3	13	11	24	581165
	1.2x3.3	14	10	24	633555
	1.5x3.3	15	9	24	711230
60m-NAP	0.8x3.3	13	20	34	517355
	1.0x3.3	15	17	32	575555
	1.2x3.3	16	16	32	627225
	1.5x3.3	18	14	31	697190

The settlements of Table 4 are without the compression of lower soil layers which will occur with time. This additional settlement is estimated at a maximum of 10mm.

2.4 Alternative concrete design

Ballast Nedam originally proposed a 2 bay design in the two directions instead of the current 3 bay design with 3 bays in the y-direction and 2 bays in the x-direction. Eventually they proposed the 3 bay design, because this reduced the floor span and increased the stiffness of the entire structure.

2.5 Alternative steel design

Besides the two concrete designs, Zonneveld also made a steel design. With this steel design it was possible to reach a higher construction speed.
The starting point of this design is a concrete core of $12x14.4m^2$ (same dimensions as the original design). Because the core is reduced from a 3 bay system to a tube section, it's possible to use a sliding formwork or a self climbing formwork. A benefit of these systems is that the crane capacity can be reduced. At the bottom the core thickness is 500mm and at halfway the core thickness is reduced by 100mm. Concrete with a strength class of C53/65 is used.

Several models were calculated in the program EsaPrimaWin, to find out what kind of steel structure would be needed. The first model only contained the concrete core. The results showed that this model didn't meet the requirements by far. The second model contained a four level outrigger at the top. This model didn't meet the requirements either. The third model contained an outrigger at the top and halfway. This was still not sufficient and the acceleration due to cyclic wind loading of the building was a point of concern. In order to meet the requirements, a mega structure would be required.

With the "light" steel structure it is possible to use the first sand layer for the foundation. This will reduce the construction cost and the building time. Approximately 600 vibro(combinative)piles with a length of 35m are required. MOS grondmechanica has made settlement calculations and the first sand layer is able to bear a maximum load of $500kN/m^2$. The original concrete design had a foundation load of $765kN/m^2$. By applying "light" floors, walls and facades it's possible to stay within the limit of $500kN/m^2$. Further calculations concluded that no problems will arise with settlements of the surrounding buildings because the high rise building now uses the same sand layer for the foundation.

It's expected that a building cycle of 2.5 to 3 weeks is needed for 3 floors. At the start the vertical transport is relatively small compared to higher levels and the construction time is relatively short. With an increasing building height, the construction workers will be able to work faster because of the learning effect. The construction time is estimated on 80 weeks: a 50% time reduction compared with the original concrete design (approximately 3 years or 156 weeks).

3 Reference projects

In this chapter preceding developments of high rise buildings are described that consist out of a prefabricated concrete load bearing structure. Two projects are extensively researched: Het Strijkijzer (section 3.1) and The Erasmus MC tower (section 3.2). In section 3.3, the Prinsenhof project is described shortly. Section 3.4 continues with a short description of the Delftse Poort and section 3.5 concludes with two reference projects from abroad.

3.1 Het Strijkijzer in The Hague [Font Freide]

The 131.5m high dwelling tower (see Figure 10) was originally designed with a cast in situ building method. This was a logical choice because of the integral stability system that uses the internal walls and the facade. Furthermore, the contractor had a preference for a tunnel system. B65 was used as concrete quality for the columns and walls, B35 for the floors and B45 for the foundation. There were two reasons why they chose B65. The first reason was the possibility to construct thinner elements which could bear a high load. This resulted in a reinforcement percentage up to 8% in the bottom part of the building (600kg/m^3). The second reason was the high Young's modulus of B65: more stiffness with less material. Steel beams are concealed within the columns on the lower levels to maintain the stiffness when loaded with tension.

Figure 10 Het Strijkijzer [Corsmit PowerPoint]

In a later stage it was decided to construct the building from level 5 till the roof in prefab concrete. This enormous decision was made because it would reduce the construction time with 1 year (from 3 to 2 years). The first four levels were not made out of prefab, because there are too many interruptions. The creation of these levels on site also resulted in extra time for the fabrication of the prefab elements. The stresses at the intersection of the two building methods were a point of interest. The original design had a large amount of reinforcement, but with prefab the amount of connecting rebar is limited. Therefore the concrete quality of the third, fourth, fifth and sixth floor are enlarged to B85. This higher concrete quality also overcomes the stiffness reduction because of this connection. According to the Finite Element Method (FEM) models the total stiffness would only be reduced with 5 to 10% by applying a prefab main load bearing system.

The logistics and safety were a big challenge, because of the limited free building space. The building site is located at a large traffic junction and on one side of the building the traffic was temporarily diverted to supply the building material. A temporary structure was used to protect the street and tramway on the other side. To get the material at the desired level, a tower crane was used. Just in Time (JiT) delivery was applied, because there was almost no storage area.

The main stability structure consists out of a tube structure. By using the internal walls, shafts and all other vertical elements the tube structure becomes stiffer and the shear lag is reduced. The entire structure is utilised and this is beneficial for lettable area. To increase the flexibility, large openings are created in the internal separation walls (see Figure 12). It's possible to combine two small apartments into one big apartment, if that is required.
In the early design phase, simple 1D hand calculations are used to calculate the minimal required dimensions of the stability system. The reduction because of the foundation and the holes in the structure are estimated. The hand calculation resulted in a wall thickness of 250mm.
The final design is calculated with a FEM program (see Figure 11). The model consists out of beam and plate elements and it took 3 days to calculate. Recently a few calculations were checked and it took only 30 minutes to calculate. For the strength calculations they used an average Young's modulus of 15000N/mm^2 and in the SLS (Serviceability Limit State) they used an Young's modulus of 30000N/mm^2. The Young's modulus is reduced in the ULS (Ultimate Limit State), because cracks are formed under the tension loading. A different Young's modulus results in a different stiffness and therefore the dynamic magnification factors (Φ_1) are not the same for the two situations. After the tender a new model was made of the prefab structure including the joints. With this model it was possible to analyse the interaction of the elements.

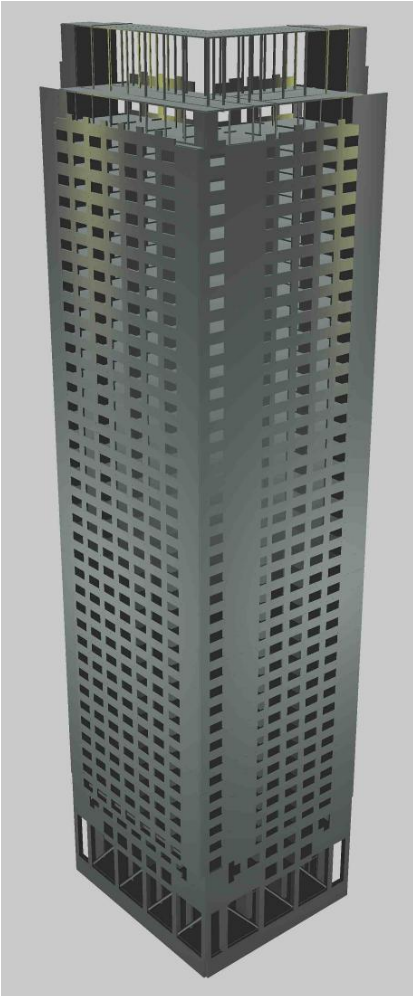


Figure 11 Cast in situ FEM model of Het Strijkijzer [Corsmit PowerPoint]

Tension ties en key connections ensure a second load bearing system. In Figure 12 a special 3D interlock connection is visible.

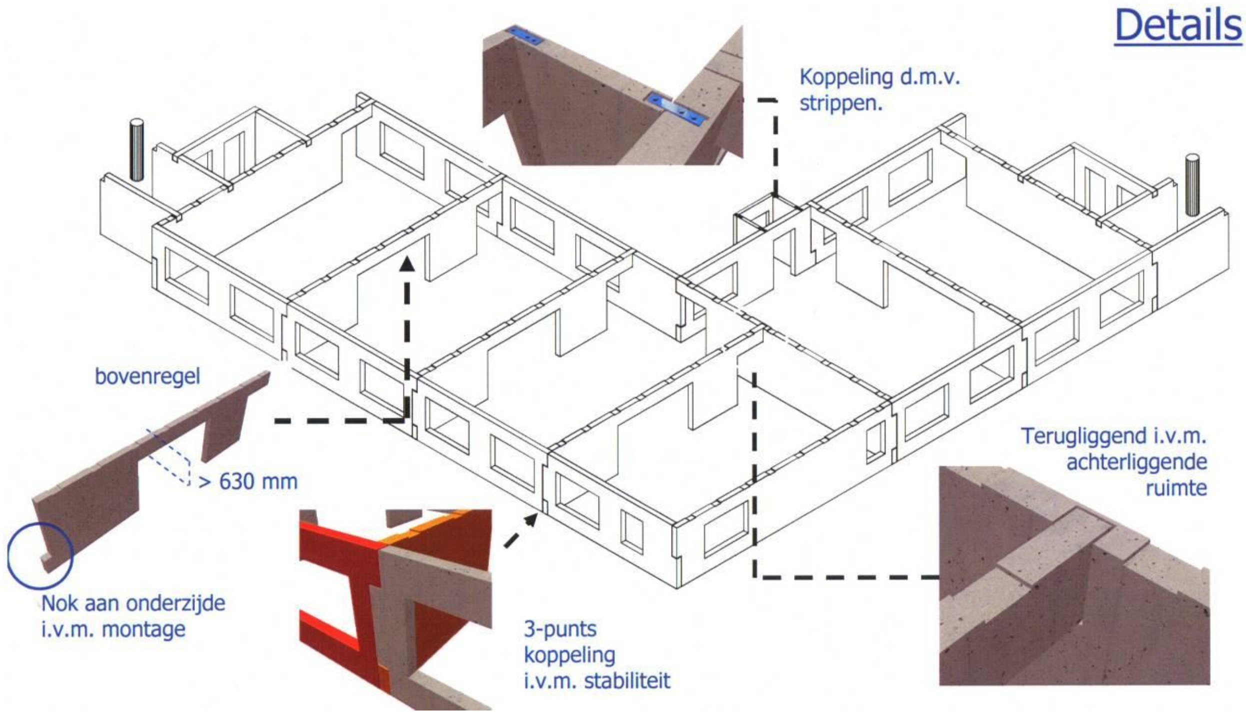


Figure 12 Connections of the elements [Corsmit PowerPoint]

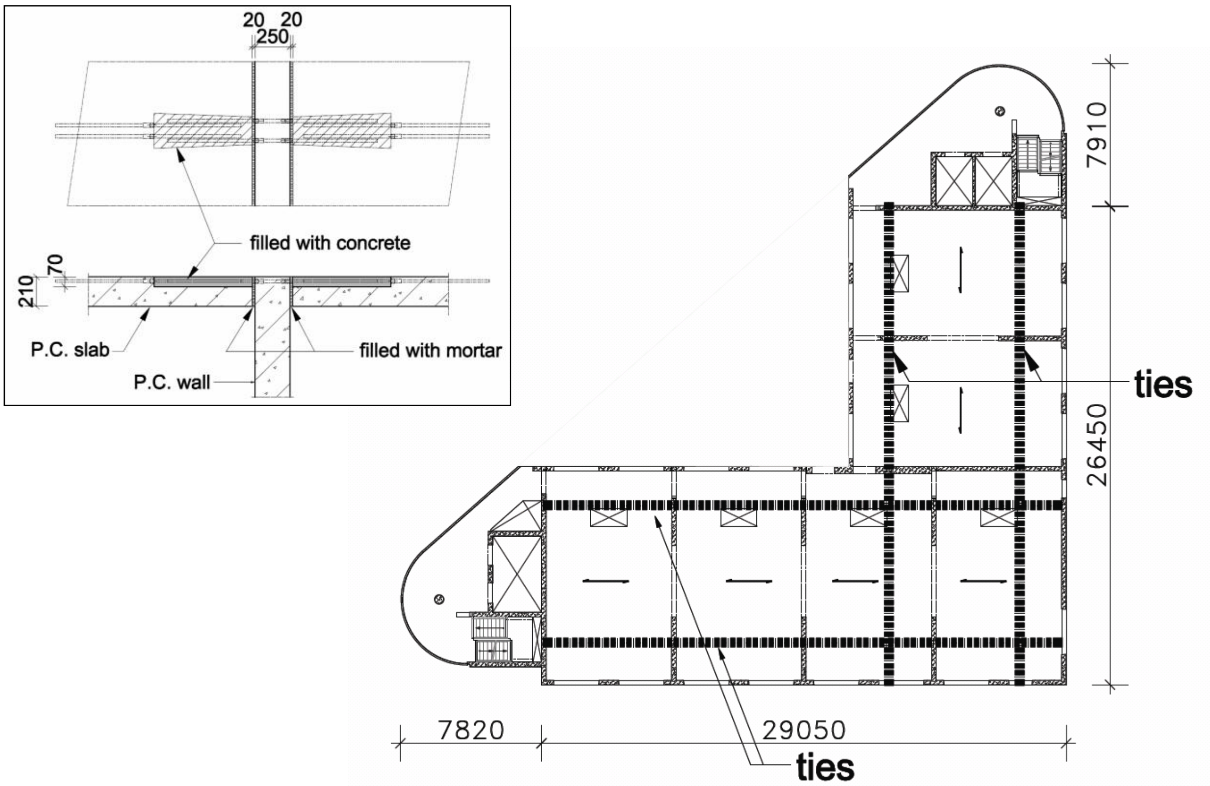


Figure 13 Tension ties layout [Corsmit PowerPoint]

To achieve a fast construction time, no structural pressure layer is applied at the floors. The diaphragm action is achieved with tension ties that are installed in the prefabricated floor elements (see Figure 13). The Waterstadtoren in Rotterdam used a similar system, but at this project the tension were placed in the joint. Figure 14 shows a detail of this connection.

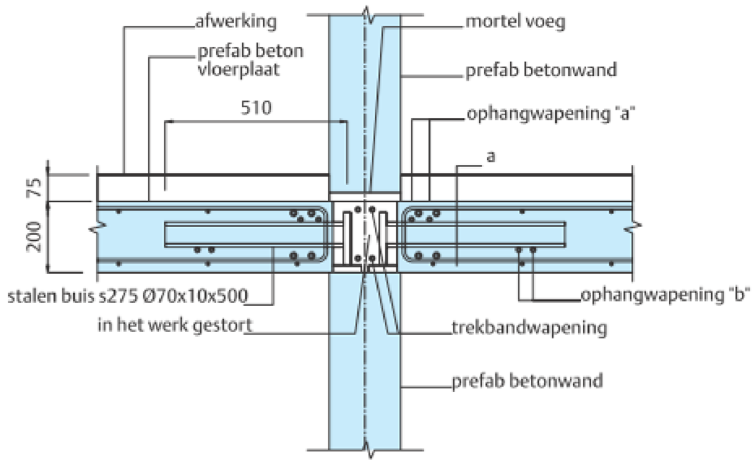


Figure 14 Floor support Waterstadtoren [Alphen 2005]

The floors of Het Strijkijzer are supported by steel angles on the top of the floor element instead of the steel tubes from the Waterstadtoren. This eliminates the thick joint and a cycle in the process. Because of the innovations that are applied in this project, it was possible to reach a spectacular construction speed of 2 new floors every week. Compared to cycle time of 5 days per new floor of the Erasmus MC tower, this looks quite fast, but at Het Strijkijzer they used double shifts instead of single shifts.

The foundation of the tower is an important aspect of the main load bearing system. All the walls are supported by 2 to 3m thick blocks (poeren in Dutch). The structure is supported by 250 pre-tensioned prefab piles. This system is used because of the favourable ratio between cost and load capacity. Furthermore, pre-tensioned piles are very stiff when loaded in tension. In an early design stage it was estimated that 50% of the top displacement would be caused by the foundation. During the finite element analysis this was reduced to 30%.

Possible benefits for the Zalmhaven tower
Het Strijkijzer is an interesting project with many innovations. The use of 3D interlocking connections and the floor supports proved to be very smart and could be applied at the Zalmhaven tower. This project also demonstrated that it's possible to construct a prefab high rise building on a postage stamp. At the vertical transport, the projects might differ. For Het Strijkijzer a tower crane was used. Unfortunately Jan Font Freide did not know if there was enough crane capacity and how many days where lost due to the weather (verletdagen in Dutch). With the increased demands for better working circumstances and quality, it's beneficial to use a hoisting shed. At the Erasmus MC tower in Rotterdam they used a hoisting shed (see section 3.2) and no days were lost due to the weather.

3.2 The Erasmus MC tower in Rotterdam [Henkens 2010]

The Erasmus Medical Centre will undergo major building activities in the current decade. This enormous project will be divided in three tranches to reduce the complexity. During Tranche 0 a part of the building site was cleared (building entries and sky bridges were demolished) and several pipes were moved. During Tranche 1, several buildings are constructed on Trance 0 (Bouwdeel Oost and Bouwdeel West in Figure 15). In 2017 Tranche 2 will start and several old buildings will be demolished.

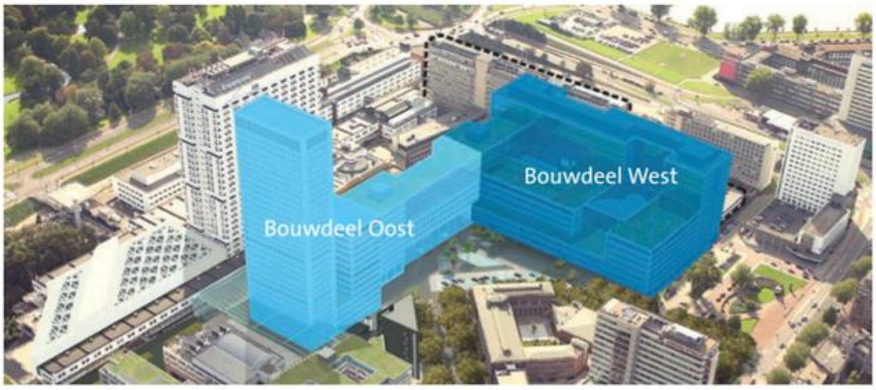


Figure 15 Constructed buildings at Tranche 1 [Nieuwbouw Erasmus MC]

To achieve the minimal required floor area on the building plot, it was clear that there was only one solution: increase the building height. An elaborated analysis was made on the location of the high rise tower. They eventually decided to construct a slender tower at Bouwdeel Oost. The tower houses the staff sections and laboratories. To maximize the floor area, it was advised from the beginning to use a facade tube. They first thought of a diagrid structure with skewed steel columns because the structure was not prismatic. Eventually they used a prismatic tower witch concrete elements because of the architecture, the function and the costs. The facade tube has a thickness of 320mm and uses a concrete quality of C53/65. The element weight ranges from 15 ton to nearly 36 ton with a maximum dimension of 7.2x3.6m^2 (see Figure 16).


Figure 16 Large sandwich elements

Sandwich elements are used with architectural concrete on the outside. The load bearing inner leaf (binnenspouwblad in Dutch) has nearly the same weight as the architectural outer leaf (buitenspouwblad in Dutch). The black part in Figure 17 is where the insulation and outer leaf will be placed from the next element: a vertical overlap joint is created. The elements are placed in a stretcher bond or masonry configuration (halfsteensverband in Dutch) to increase the coherency. The corner elements are connected by a wet connection (kelkvoeg in Dutch). Hurks precom+ suggested using a dry corner connection, but this was never used. All the elements that were used are unique. In case an element is damaged, a problem arises: the construction has to wait until the element is fixed on site or when a new element is constructed. The probability that the construction process would come to a hold was so small that they chose to use unique elements. K70 mortar was used for the wet connections.

Figure 17 Connection joint of the elements

The first four floors are cast in situ because of the large amount of interruptions (the architect wanted an open structure at the public plinth). This also gave extra time for the construction of the prefab elements. To overcome the problem regarding the lack of stiffness, the core was used in the bottom. The core provides the extra stiffness and it is connected at two levels with a very thick floor to the facade tube. Because of this connection, the core will take up the horizontal force and the bending moment is resisted by the elements that are cast in situ. From level 5, the core is also prefabricated. To ensure the consistency of the core, Interlocking Above Ceiling Connections (IACC) are used. At the JuBi towers in The Hague they also applied a facade tube in combination with a core because there were also too many interruptions at the ground level. The only difference at this project was that the in situ core was used over the total height and a tube in tube structure was created.

The floors of the tower consist of a self bearing concrete slab of 160mm and a structural screed layer of 60mm. Furthermore, the floors have a maximum span of 7.2m. The tower has no basement and for the foundation a 2m thick concrete slab with 333 foundation piles is used. The foundation was quite complicated because the layer of Kedichem is not confined in the bottom by a second sand layer. Settlements up to 200mm are expected. The low rise buildings next to the tower have an average height of 60m and settlements of 80mm are expected. Because of the influence of the tower, the settlements will increase to 135mm in the direction of the tower: the low rise buildings will become skewed if no actions are taken. The complexity of the problem was increased because a 40m glass facade would be constructed between the tower and the low rise buildings. A detailed analysis was made on how this connection should be made without any large deformations and stresses. Eventually double dilatations were applied to solve the problems. The settlements of the tower and the low rise buildings also result in settlements of nearby buildings. On several locations damage is expected after several years. This damage is reviewed and accepted.

With the use of a hoisting shed, the tower could reach a high construction speed: it was possible to construct a new layer every four days. Eventually they used a building cycle of five days. The building team Nieuwbouw Erasmus MC (consisting of BAM Utiliteitsbouw Grote projecten and Ballast Nedam Bouw Speciale Projecten) decided to use a hoisting shed because of prior experiences at the Nationale Nederlanden and several other important reasons. When a tower crane is used, a drop safe zone has to be created. The new tower is constructed nearby the current hospital and several walkways and operating rooms are within that zone. Furthermore, elements with a maximum weight of 36 ton would be used. This resulted is two very heavy Potain MD1100 cranes that are very difficult to rent (there are only a few MD1100 cranes in the world). This will creates a large risk when the project is delayed: the crane may be rented out to another project before the delayed project is finished. A hoisting shed is not affected by this risk because it’s especially made for the project. Aside from the risk, a hoisting shed also reduces the weather dependency and increases the safety for the construction workers.

The hoisting shed has a dead load of 450 ton and provides room for two gantry cranes. One crane is used for the vertical transport and the other for the horizontal transport. The cranes have a maximum capacity of 40 ton and the hoisting shed is supported by two large truss beams. Because of the large element mass and a very wide stabiliser (dubbele evenaar in Dutch), no problems with wind were encountered (see Figure 18). During the entire project the construction process only stopped for one hour, because light installation tubes had to be transported in heavy winds. At that moment the construction workers had their lunch time and no time was lost. At the end of the project the construction workers already transported elements on Friday that had to be transported on Monday, so that they could go home early on Monday too. This had consequences for the transport and factory because they had to produce and transport elements earlier. For the transport a Just in Time delivery was used. This resulted in a very efficient transport cycle from Veldhoven (factory location) to a nearby waiting place and eventually to the building site.

Figure 18 Gantry crane with large stabiliser

Possible benefits for the Zalmhaven tower
The Erasmus MC tower in Rotterdam is the second project in the Netherlands that used a hoisting shed (Delftse Poort was the first, see section 3.4). With this hoisting shed it was possible to transport heavy elements very fast. This resulted in a very fast cycle time of 4 days. It also reduced the dropping zone, financial risk, weather dependency and increased the safety. Using a hoisting shed at the Zalmhave tower could be beneficial. The previous mentioned benefits could result with some adjustments in a highly efficient building process with a cycle time of 3 days. The biggest adjustment should be avoiding concrete casting on site. A fully prefabricated floor is recommended.
The very large elements are another innovation applied at this project. The largest elements measure 7.2x3.6m^2 and weigh up to 36 tons. By using very large elements, the amount of weak joints are reduced and this has a positive influence on the stiffness of the building. Furthermore, heavy elements result in less sway during the vertical transport. The concrete factory could deliver elements up to 80 tons, but the transport and the gantry crane are the limiting factor. The building site of the Erasmus MC tower is near the Zalmhaven plot: in the centre of Rotterdam. At the Erasmus MC tower they used JiT delivery and this resulted in an efficient transport system. Because of the small building sites at both projects, JiT delivery has a lot of potential to be used at the Zalmhaven tower.
At the Erasmus MC tower, the stability was provided by a façade tube. Near ground level, the core in combination with columns were used because of the large interruptions in the façade tube. At the Zalmhaven tower there are also several interruptions at ground level and level 1. Furthermore, The Zalmhaven tower is 80m higher and therefore the bottom part of the structure becomes more critical. During the analysis, rough calculations will have to determine if an interrupted façade tube or a tube in a tube structure is necessary.

The Erasmus MC tower is a very interesting project with a lot of innovations that could be beneficial for the Zalmhaven tower. By using an optimized system for the dry finishing, a fast cycle time will also result in a short completion time.

3.3 Prinsenhof in The Hague [Lindhoud 2003]

The Prinsenhof project consists out of four office towers, three residential towers and one hotel tower. A low rise plinth connects all the towers at the bottom. The highest tower of the Prinsenhof is 95m tall (one of the office towers) and when the steel top is included, it even reaches till 130m.

Figure 19 The Prinsenhof in The Hague [Kock 2006]

The residential towers are constructed with a tunnel system. This building method was preferred over a prefab method because less crane capacity and storage area would be required. This system also provided a fast building cycle of three days and the system could be reused in the other two residential towers. For the four office towers a prefab building method was used. This is because these towers have a larger floor span and building height. The stability of the four prefab towers is provided by the facade and the facade elements are 400mm thick at the base and 200mm at the top. The element configuration was a point of interest. When the elements were placed regularly above each other with a simple connection, the continuous joint would create a relative weak stability wall. If the vertical connections are not able to transfer large shear forces, the elements will behave as single walls (see Figure 20).

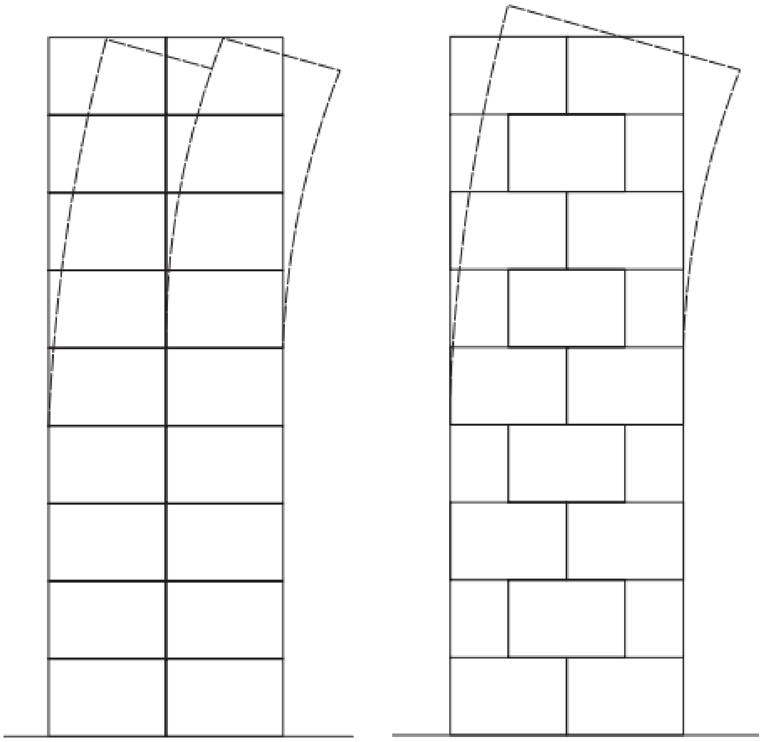


Figure 20 Element configuration at the Prinsenhof [Falger 2004]

To achieve a nearly monolithic behaviour with the normal configuration, the joints should contain a tooth profile and have to be filled with mortar. Instead of using this time consuming and expensive solution, the elements are placed in a staggered or masonry configuration (see Figure 20). This was the first time that this system was applied in the Netherlands and it turned out to be very efficient. For the corners they used a staggered connection (the front and side facade alternate per floor). With this simple connection expensive and labour intensive corner connections are prevented. Because of this connection, only a tenth of the building height is used as effective width for the flanges.

The small building area was characteristic for this project. The available space was used as construction area and a small amount of storage was possible. Therefore all the building materials were immediately transported to their final destination by one of the eight tower cranes. A cycle time of six days per floor (approximately 1000m^2) was reached.

Possible benefits for the Zalmhaven tower
The Prinsenhof project showed that it's beneficial to use prefab elements in a staggered configuration. By applying this system, time and money is saved without any negative side effects. A building production of 166,7m^2 per day per shift on a very small plot is also an interesting aspect.

3.4 Delftse Poort in Rotterdam [Köhne 1991]

Before the erection of the Delftse Poort, high rise projects were always constructed with modified low rise construction methods. But with a height of 150m these methods didn't satisfy the requirements of the Delftse Poort. The building team Delftse Poort proposed to use sliding formwork for the cores and a hoisting shed for the office wings. Knowledge on slide formwork was available and a high level of quality could be guaranteed. The thin and long office wings required more research. Because of the large repetition factor, prefabricated elements were the first choice. But at these heights the process is very sensitive for weather conditions. From this consideration, a factory environment at the building site was preferred. A self climbing hoisting shed was an attractive solution and because of the project scale, the investment was profitable. In Figure 21 an interior view is given. The two gantry cranes placing the facade elements are clearly visible.

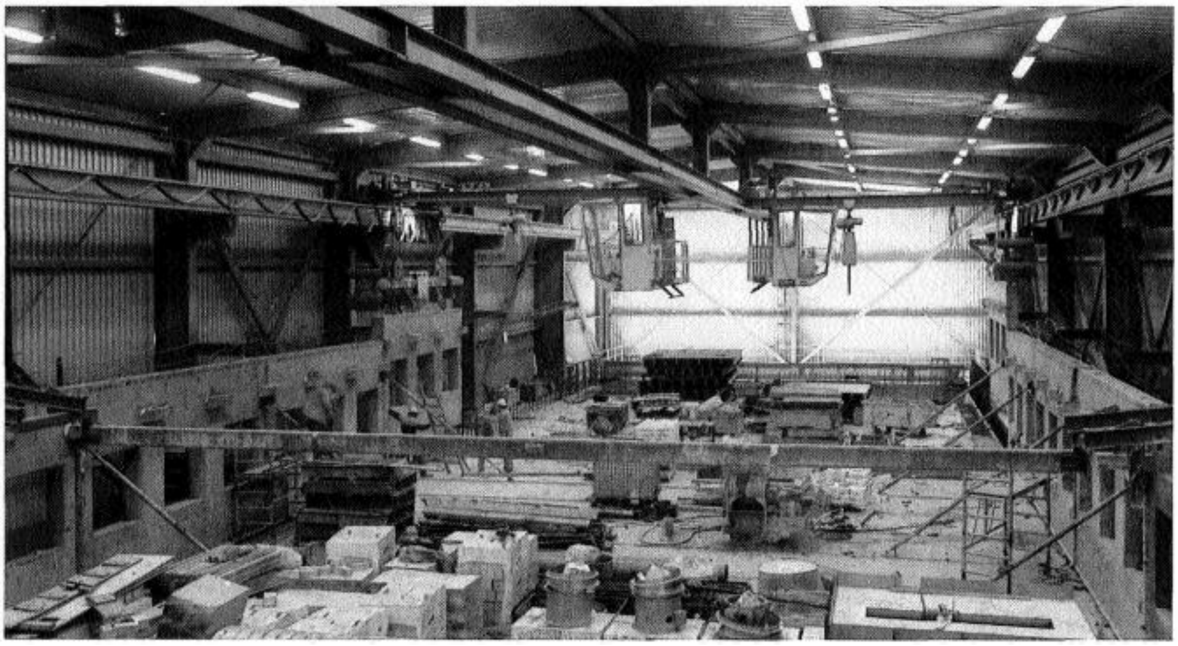


Figure 21 Hoisting shed of the Delftse Poort [Köhne 1991]

The hoisting shed was supported by four hydraulic jacks and had a total weight of approximately 2000kN. Because of the wind pressure, every support was designed to be able to resist 1000kN. The gantry cranes have a capacity of 150kN (approximately 15 ton) each and a guidance rail prevents the elements from swaying (see Figure 22). The glass facade was applied afterwards.

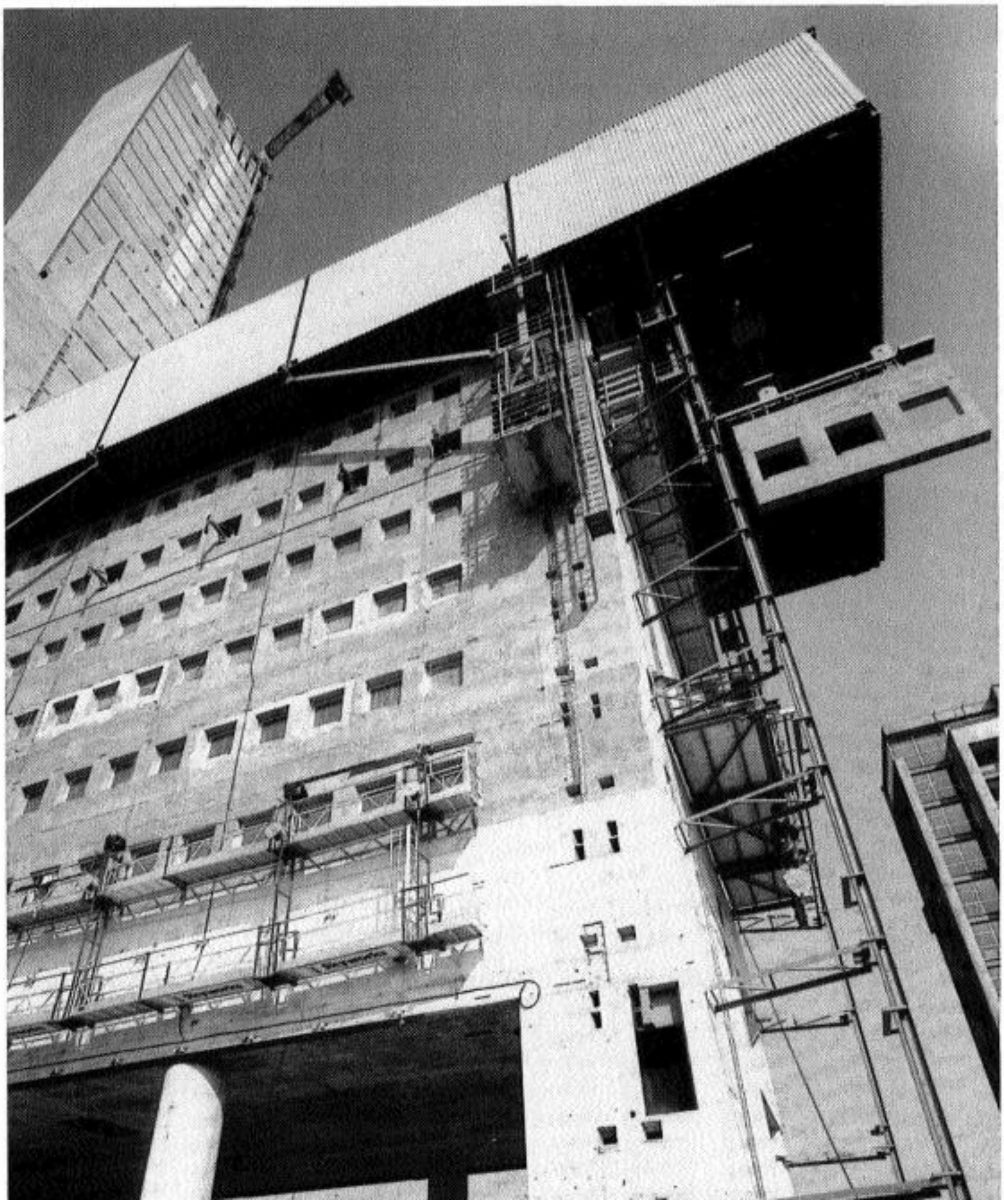


Figure 22 Hoisting shed with guided transport [Köhne 1991]

The lack of space resulted in a prefab element delivery that was done per hour. Every element had its own tracking number and the floor elements were hoisted from the truck. Two shifts of 16 persons worked 8 hours per day and a building cycle of 5 days per floor was achieved. This resulted in a building production of $900m^2/5$ day=$180m^2$ per day for two shifts. Per shift this is $90m^2$. For 1991, this is a quite astonishing value.

Possible benefits for the Zalmhaven tower

The hoisting shed was an important innovation that resulted in a fast construction time and excellent execution conditions. A new phase in the Dutch execution of high rise buildings was created. The requirements that led to this design are also applicable for the Zalmhaven tower. Applying a hoisting shed with Just in Time delivery could be an interesting solution.

3.5 Reference projects abroad

The scale at which prefab concrete is applied in the Netherlands in unique. For example, approximately 50% of concrete production is utilised for prefabricated elements. For comparison, in Germany this value is only 26%, in France 18% and in the United States of America only 12% [Vambersky 2007]. This division is also shown by the number of fully prefabricated projects around the world, only two projects were found: the Lindhagsskrapan in Sweden and the Xiwang tower in China.

The Lindhagsskrapan, with a height of 75m, was constructed in 2009 in Stockholm and houses dwellings. The first two layers were cast on site and the remaining 22 floors are constructed with a prefabricated load bearing facade (sandwich elements). Within 18 weeks, the 22 prefabricated floors were constructed. The total construction time was 2 years. In Figure 23 the tower is shown.

Figure 23 Lindhagsskrapan tower in Sweden [Svensk Betong]

The Xiwang tower in China has a total height of 170.6m and was constructed in 1999. This office tower has 43 stories and one story was completed every 3 days. The building is located in a high seismic zone on the northeast coast of China (city of Dalian). The building uses prefabricated concrete beams, slabs and exterior architectural cladding. The vertical load bearing system is created on site. In Figure 24 the tower is shown.


Figure 24 Dalian Xiwang tower [YPDGL]

4 Interviews with experts

This chapter contains four interviews with experts of the building industry on prefab concrete.

4.1 Jan Font Freide from Corsmit (Het Strijkijzer)

Jan Font Freide was responsible for the structural design of Het Strijkijzer. He made the (computer) calculations for the original cast in situ concept. In the end Aveco de Bondt reengineered the design in prefab, commissioned by Boele & van Eesteren (the contractor of the project), but Corsmit was still responsible for the overall design. Level 5 until the roof were made with prefab concrete and engineered by Aveco de Bondt. The basement until level 4 were made with cast in situ and engineered by Corsmit.

Het Strijkijzer was an innovative project awarded with the golden Emporis Skyscraper Award 2007. Jan Font Freide was very pleased with the outcome and the only encountered problem was the steel grid on top of the roof. At a specific wind direction this grid would make a whistling noise. The connection between prefab and cast in situ was a point of interest, but no problems were encountered.

4.2 Ron Vonken and Gerard Baggermans from Hurks Beton (Erasmus MC tower)

Gerard Baggermans (Chief calculation/Technical advisor) and Ron Vonken (Advisor) were responsible for the Erasmus MC tower at Hurks precom+. Aronsohn Constructies raadgevende ingenieurs delivered the structural design and Hurks precom+ made sure the elements could resist the calculated loads. Hurks precom+ did no global calculations; they only designed and constructed the elements. The cooperation between all the parties was achieved with a building team organisation.

Gerard Baggermans and Ron Vonken were very pleased with the outcome of the project. The hoisting shed was beneficial and resulted in a short building cycle. Constructing this tower with tower cranes would have resulted in more problems. The very small amount of time lost due to the weather was not really a surprise for them. They believe that it’s a widespread misunderstanding that prefab structures have more weather delays (verletdagen in Dutch) than cast in situ structures. The average amount of days lost due to the weather is around one or two days per project at Hurks precom+. This is because of the large weight of the elements and the lack of pouring concrete on site. Besides the advantages of the project, they also encountered several problems at the project. By using partly prefabricated floors, a lot of time was lost. Using entirely prefabricated floors would save time and reduce the amount of different disciplines on site. The construction process also lost a lot of time during the finishing phase. If the tower is finished but the interior isn’t, you still can’t use the building. Hurks precom+ designed a system that makes it possible to finish the interior 3 layers behind the construction phase. This means that after the buildings is finished, it only takes 3 weeks (3 layers x 1 week) to finish the entire building! By using this process, the design and finishing stage will be altered. Just like the prefabrication of the structure, the finishing is partly prefabricated. More decisions have to be made in an earlier stage and the system should contain more flexibility. By using a special duct system, the most difficult element in a dwelling (the bathroom) can be placed as a black box within a day at any location in the apartment.

When Gerard Baggermans and Ron Vonken look to the structure, it would be possible to reach beyond the current height of 120m. There are no problems with the stability of the façade tube. The only limiting factor is the vertical force at the transparent plinth: because of the public function the amount of concrete is drastically reduced. The problem is enlarged because of the enormous dead weight of the structure. If the façade tube would continue to the foundation, larger heights could be reached.

If Hurks precom+ would have had the chance to redo this project, they would change several things. With the current design, a large amount restrictions and requirements where placed upon the elements and Aronsohn Constructies raadgevende ingenieurs already did a lot of the design work. Therefore, Hurks precom+ could not apply all of their own knowledge and expertise. If they were contacted in an earlier stage, the structure would have been altered.

When Gerard Baggermans and Ron Vonken look to the future of high rise structures and prefabrication in the Netherlands, they see a lot of opportunities. Gerard Baggermans expects that the building heights will not exceed the 200m level because of the soil conditions. It would become too expensive to pass this level. Prefabrication in high rise will only increase. Currently all buildings are designed as monolithic structures and in a later stage prefabrication is considered. They would like to see a change of mind and let people realise that prefabrication is not a second option. A lot of contractors think that a prefabricated wall is more expensive than a cast in situ wall. This is because of the cast in situ design and the cast in situ cost calculation.

4.3 George Henkens from Aronsohn (Erasmus MC tower)

George Henkens (director) from Aronsohn Constructies raadgevende ingenieurs oversaw the structural design of the Erasmus MC tower. He worked on this project, but did no calculations. Aronsohn delivered the structural design, calculations, specifications, detailed drawings and monitored the construction process.

George Henkens was very pleased with the design. Aronsohn was founded more than 80 years ago and they did already two more or less identical projects: Medische Faculteit in Rotterdam (1966) and Delftse Poort (1990). Aronsohn applied this knowledge in the Erasmus MC tower. During the second final design, the window sizes were changed to one size because of aesthetics (the first final design contained small windows in the bottom, resulting in a larger stiffness). Aronsohn always applies a structural screed layer at their projects. At several projects they didn’t apply this layer and they still regret those decisions. By using a structural screed layer there are less inaccuracies, the structure becomes more monolithic and better fit for the future. Therefore they also applied a structural screed layer of 60mm at the Erasmus MC project.

The use of a hoisting shed had no disadvantages for Aronsohn and the increased load on the corner elements was no problem.

The current design of the tower was made for 120m. There is no additional capacity for extra layers on top in the future. The design is determined by the bottom elements. No reduction of thickness has been applied, only the reinforcement is decreased at higher levels. George Henkens thinks it should be possible to go even higher with this structure at this location (up to 150m), the only problem is the vertical load. Large settlements of 200mm are expected and damage will occur in the surrounding buildings. At the moment the building has settled less than expected and this creates problems at the connections: the tower was constructed higher than the connecting buildings and with the settlement this difference would disappear. Unfortunately this is not the case at this moment.

George Henkens thinks there are possibilities for prefab in the future. The possibilities of façade tubes on the other hand are limited. This is because of the high mass, reduced flexibility and very small windows. With a façade tube it’s possible to have column free spans, but with a core and steel façade columns it’s also possible to achieve free spans. By using steel façade columns, the façade becomes very flexible and the structure will be more durable (fit for purpose). According to George Henkens fully prefabricated buildings are not the future. Prefabrication (steel or concrete) will only be used in a part of the building. A concrete core by sliding formwork with an outrigger, steel columns and beams in combination with a steel and concrete composite floor is currently the best solution for office buildings.

4.4 Willem van Dijk from Ballast Nedam (Erasmus MC tower)

Willem van Dijk is deputy staff director at Ballast Nedam Bouw & Ontwikkeling Speciale Projecten and he is responsible for the Erasmus MC tower and the Zalmhaven project. Ballast Nedam is one of the largest construction firms in the Netherlands and they deliver a wide range of products and services related to construction. One of their expertise products is the DBFM-contract, where they design, build, finance and maintain the project.

A construction firm like Ballast Nedam is mainly focused on the triangle Technology-Costs-Processes. When they enter a project, they are responsible for the realization. Costs, manufacturability (maakbaarheid in Dutch), feasibility, building method, design, connections, settlements and many more elements should be checked. Problems they might miss will become their responsibility. An early cooperation is advised, because of their expertise. Difficulties could be diverted in a very early stage and a lower risk project is created. When the entry point is moved to the specification phase (bestekfase Dutch), many decisions are already taken and the freedom and inventiveness of a contractor is limited. This was the case with the Erasmus MC project were Ballast Nedam and BAM Utiliteitsbouw subscribed for the tender phase. They were not allowed to hand in an alternative design and the execution should be done as pointed out in the documents. This resulted in two extremely large tower cranes which are very difficult to rent (there are only a few of these cranes available in the world). To overcome this problem the heavy facade sandwich elements were taken apart. This would reduce the load capacity of the crane (only 1 crane was necessary in this option) and the facade elements could be placed by a crane independent system. This was not allowed and a different solution was needed. Eventually a plan was made to use a hoisting shed and the client accepted the alternative design. If a contractor was consulted from the beginning, costs and time for design and execution could have been reduced significantly.

When Ballast Nedam tenders for a project, they start with specification documents. These documents are studied in detail and the project is divided in sections. For example Bouwdeel Oost and Bouwdeel West at the Erasmus MC project (see Figure 15). Then every section is divided in sub sections, for example basement, floor 1, floor 2, and so on. By dividing the project in sections and sub section a complex project is broken down in manageable portions: the complexity is reduced. Furthermore, everybody knows which location is meant with Bouwdeel Oost and problems are prevented. It should be noted that when a complex project is divided in sections, the connections become more important. After the division, the building process is designed. With this building process it's possible to make an estimation for the required material, equipment, personal and time. For a small amount of aspects a calculation tool is used, but the estimations are mainly based on expertise. The building processes are also used for the planning schedule. By studying every process and relation in detail, it's possible to determine the cycle times and the occupancy factor. The design should be reevaluated if these values are not realistic (crane load of 120% or a large amount of construction workers per m^2). When this is finished, the logistical process is designed and all these steps are combined in an approach plan.

The combination of Ballast Nedam and BAM Utiliteitsbouw is not very uncommon today. Besides working together on the Erasmus MC project, they also joined forces on the Prinsenhof and JUBI project in The Hague. This combination is mainly based on spreading risks and because of good experiences from the past. Ballast Nedam could bear all the risks on its own, but the current market and the consistency of work (employing a steady number of personnel over the year) make it more attractive to collaborate with BAM Utiliteitsbouw.

When Willem van Dijk looks back on the construction of the Erasmus MC tower, there were two important influence factors for the execution process: settlements and vibrations because of pile driving. The client forgot to thoroughly investigate the geotechnical properties. Eventually more research was combined with a very expensive insurance, paid by the client. Vibration requirements ware necessary because part of the hospital was not closed.

At the Zalmhaven project, Ballast Nedam was involved from the start. Because of their experiences, they changed the two by two bay design to a three by two bay design. This resulted in smaller floor spans and a stiffer stability system. Whether it's possible to apply a hoisting shed depends on the structural design, the building layout, steering indicators (time, money or quality) and the preparation process. The structural design should be based on prefabrication and not a sliced monolithic design. When prefab is applied, the preparation process should be radically changed. Prefabrication requires more information in an early stage compared to casting on site. Therefore the internal finishing should be designed in such a way that the buyer has still a large amount of freedom to modify his (expensive) apartment.

5 Vision for the construction methodology

Preliminary research was done in order to make the work plan for this thesis. During this research, a vision for the construction methodology (building method and transport system) was made. This was necessary, because it's impossible to make a good integrated prefab design without the construction methodology. During the literature study more information is gathered on this subject. The construction methodology will be reconsidered after the structural design is finished, because these two aspects influence each other.

5.1 Vision for building method

There are many aspects on which prefab concrete distinguishes itself from cast in situ structures. The following three aspects are considered to be the most important:

- The construction time is reduced because at the building site the construction workers only have to assemble the building. By using less different elements there will be more repetition in the factory and on the building site. After the elements are constructed they pass a quality check at factory and this results in a swift and efficient assembly process. Simple and fast connections reduce the time needed for the assembling even more.
- With the reduced construction time, the financial risk is minimized. This is because the building can be sold or rented out in an earlier stage and the interest costs are reduced. The high quality of the elements also ensures a larger lettable area. Outsourcing work to a controlled environment reduces the financial risks even more. On the other hand, a concrete prefab building has a higher initial cost.
- The construction process is shifted from building site to factory: construction becomes assembling. The conditions in the factory are better and this part of the building process becomes independent of the weather. Furthermore, at the factory the building process is more centralized: the transport of material and equipment is reduced. This results in a cleaner building site where less area and personnel are needed.

Prefab concrete has the opportunity to become the new building standard, where cast in situ still struggles with the 3D building syndrome (Dirty, Dangerous and Difficult).
To become the new standard, an efficient building process is essential. This process depends on several key elements:

- A transport system is necessary for personal, material, equipment and waste. The vertical transport is usually the limiting factor.
- An overview of the storage, transfer and transport operations is needed to control the flow of material. To decrease the amount of storage on the building site, Just in Time delivery (JiT) is preferred. A small amount of storage is necessary to prevent any disruptions in the building process.
- The construction of a prefab high rise building is a serial process and it is very sensitive for disruptions. Without the third floor it is impossible to work on the fourth floor (using a bottom up construction). Disruptions will lead to storage and a longer construction time.
  Wind, temperature and rain are the most well known factors for disruptions. The lack of correct information on the right moment is often underestimated.
- High rise buildings are often constructed on small building plots in very dens areas. The plot size determines the storage and transfer operations. The location also plays an important role. Just in Time delivery is the main solution for small building plots in high populated areas.
- The design of a tall building is often made by multi disciplinary teams and traditionally there is a separation between the design and the construction.

Problems arise because of different liabilities and responsibilities. With a Design and Construct organization most of these problems are overcome and the possibilities for control and information exchange are enlarged.
The relation between the main contractor and the subcontractors is of utmost importance too. The subcontractors are obligated to deliver their product at a premium quality within a certain time span. Cooperation results in more knowledge in an earlier stage and a reduction of failure.

The benefits of cast in situ should not be underestimated. The following four aspects are considered to be the most important benefits of cast in situ structures:

- Cast in situ buildings are jointless and a monolithic structure is created. This has a positive effect on the interaction and the flow of forces.
- The designer has a large amount of freedom. The formwork is the only limiting factor. Floor spans in two directions are common and the integration of ducts and pipes are no problem.
- Because there is no prefabrication, the final designs can be made at a later stage. This reduces initial cost and interest loss.
- Liquid concrete has a small transport volume compared to prefab elements and the amount of trucks can be reduced.

The choice between prefabrication and cast in situ is a regularly recurring issue. Sometimes a project is made in cast in situ, while a comparable project is made with prefab concrete. The choice whether prefab or not depends on a comparison of both methods and the preference of the designer, contractor and client.

The benefits of prefab concrete in combination with an efficient and integrated transport system are discussed above. But is it possible to use prefab for the Zalmhaven tower? The Zalmhaven tower was originally designed with cast in situ concrete. To cast the building on site, a tunnel system would be used. But this tower has potential to be build in prefab concrete. The following aspects explain why:

- The building is rectangular, has 65 floors and consists out of regular floor plans. These regular floor plans and the amount of floors are beneficial for the repetition factor. It’s possible to reduce the element thickness over the height, but this will reduce the repetition factor.
- The building site is located in the centre of Rotterdam. The site is rather small and it is surrounded by dwellings. Two hundred meters on the south west side of the plot, the river Nieuwe Maas is situated. This gives an excellent opportunity for transport by water. Most concrete factories are located near a river and transport over water will reduce the nuisance for centre of Rotterdam.
- The market for dwellings is currently under large pressure. The financial crisis is responsible for the fact that this building isn’t constructed yet. Reducing the construction time results in apartments that are easier to sell. For example, a dwelling that is finished within 15 months is more attractive than a dwelling that is finished within 30 months.
- Reducing the construction time also results in a higher profit. Looking at the current market for dwellings, this will probably be the main argument to use prefab concrete instead of cast in situ concrete.

When a prefab building method is used, the connections become very important. For the horizontal joint wet reinforced connections with a smooth surface are normally used. For the vertical joint it’s possible to use several wet or dry connections:

Wet:

- concrete unreinforced joints with a smooth surface,
- concrete reinforced joints with a smooth surface,
- concrete reinforced joints with teeth,
- concrete joints reinforced by loops.

Dry:

- welded joints with cast in steel plates,
- welded joints with cast in profiles.

It’s also possible to use an open non structural vertical joint. The Prinsenhof in The Hague was the first building where they applied this connection. This joint contains no structural facilities and therefore is unable to transfer any loads between the elements. By placing the elements in a masonry configuration it’s still possible to transfer vertical shear force and the stiffness is comparable to the concrete reinforced connections with teeth (without the extra work).

In this early stage, prefab elements in masonry configuration with non structural vertical joints are considered to be the best solution.

5.2 Vision for the transport system

The transport system can be divided into two parts: from the factory to the building side (horizontal transport) and from the building site to the final location (vertical transport). For the first phase, there are four options: transport by road with storage on the building site, transport by road with Just in Time delivery, transport by water with storage on the building site and transport by water with Just in Time delivery. Transport by water with the JiT principle is considered the best solution for this project at this phase because of the following benefits:

- The ship has a very large capacity with almost no restrictions for the prefab elements. Without the horizontal transport restrictions, the elements will be limited by the factory and the building site (mainly by the vertical transport system).
- The building site area is very small and with the JiT principle less storage is required.
- By using water instead of the road, the busy centre of Rotterdam is relieved of extra transport.
- Compared to trucks, the transport time will be longer over water. With a correctly managed process this doesn’t have to be disadvantage.

Because of the large capacity of a ship, some storage will be created inside the ship. Therefore the term JiT is not 100% applicable.

In order to apply JiT over water, the prefab elements have to be lifted out of the ship. A temporary or ship mounted crane is necessary with a high load capacity and a short reach. Next, the elements have to be transported by road for 200m. There are no height and width limitations. The waterbus has to be temporarily diverted to a nearby dock, to make room for the transport ships (see Figure 25).

Figure 25 Building location and loading dock [Google Maps 2011]

For the second stage there are two solutions: a separated system or a non separated system. With the separated system, the vertical transport is separated from the horizontal transport at the site. A good modern example of this system is the hoisting shed for the Erasmus MC tower. To determine the final system, four aspects have to be taken into account [Meij 2012]:

- The costs. For example purchase cost, running cost, disassembly cost, interest cost and residual value.
- The transport time. The transport time is composed out of the horizontal and vertical transport including the return, picking up, placing, adjusting and stabilising of the elements. The time that is needed before the following action can take place (the lead time) is also important.
- The capacity of the system. This depends on the amount of elements, the maximum weight, the size and the amount of other material that have to be transported.
- Limitations. The limitations depend on the visibility on the transport and the sensitivity for wind, rain and frost. There are also limitations for noise, nearby plots and safety because of the building standards.

A hoisting shed is used for the second phase instead of tower cranes because of following benefits:

- The transport time is reduced because vertical transport is separated from the horizontal transport (the critical path is reduced). When the vertical transport is leading, which is common for high rise towers, the time reduction will become smaller.
- By using prefab as building method, a large amount of elements have to be transported. JiT over water has les restrictions and the size and weight of the elements will increase compared to transport by road. Therefore the total amount of elements will slightly decrease. A hoisting shed is able to transport heavier elements faster than a tower crane[2].

[2] In [Meij 2012] the crane of the Erasmus MC hoisting shed is compared with a Liebherr 420 EC-H20 crane. At a load of $20*10^3$kg (maximum load of the Liebherr crane), the Erasmus MC crane is 4.5m/min faster.

- A hoisting shed has a good visibility on the transport and the workers are protected from wind, rain and frost. The vertical transport is still sensitive for wind and it's advised to place it at the wake site of the building. Because the wake site has more turbulence, a guided transport system could be used to avoid collisions and a large hoisting zone. At the Erasmus MC tower they used a broad stabiliser (dubbele evenaar in Dutch) and this system does the same without a guidance rail (see Figure 26). This saves time and money. The hoisting shed also reduces construction noise and the level of safety is increased.
- There is one disadvantage: the costs. The purchase, running, disassembly and interest cost of a hoisting shed are larger than for a tower crane. The residual costs will not be very high either because it's tailor made for the building. Elements can be reused, but it's unlikely that the entire structure will be reused. These costs have to be lower than the extra revenues of a faster building time in order to be economically attractive.

Figure 26 Broad stribiliser with two gantry cranes [Nieubouw EMC]

To reduce the construction time even more, the cycle time can be optimised. A cycle time contains all the stages that are necessary to complete a building layer and it depends on the following six aspects [Meij 2012]:

- Critical path. Reducing the critical path by removing actions will result in a shorter building time and less interaction problems. Casting floors and joints should be avoided as much as possible.
- Separated transport flows. The hoisting shed is a good example of the separated transport system. Because the flows are separated, the critical path will be reduced.

- Multiple transport flows. When the building height increases, the vertical transport will become more important. Using a second transport system (another crane or a different system) can avoid delays.
- Assembly sequence and lead time. An optimized assembly sequence is beneficial for the cycle time. The use of fast drying mortars in combination with a protected climate will minimize the lead time.
- Number of actions. Reducing the number of actions decreases the building time. Mounting the facade on the load bearing structure is normally not included in the critical path, but the amount of actions are larger compared to an integrated load bearing facade (sandwich element). Using an integrated facade will also reduce the flow of material.
- Building method. The building method is of large importance for the cycle time. Using a high level of prefabrication will reduce the amount of disciplines on the building site and the risk for delays. A good example is the pressure layer on the floors.

To reduce the cycle time at the Zalmhaven tower, the cycle time should be optimised. The following three aspects are used to achieve this goal:

- A hoisting shed. Using a hoisting shed ensures a separated transport flow and a reduced cycle time. At a low building height, the crane will have an over capacity and the horizontal transport will be governing. At the top of the building, the hoisting shed will have an under capacity and the vertical transport might be governing. A second crane in the hoisting shed may resolve the problem (multiple transport flows). Further research will indicate if this is necessary. Small material and personnel will be transported by an internal elevator.
  The hoisting shed will also reduce the lead time, because the cycle time is less dependent on the weather.
- Prefab floors. For the floors a massive concrete slab with integrated ducts will be used. A structural pressure layer is avoided and the reinforcement is included in the elements. This will reduce the amount of actions, the lead time, the flow of material and the amount of different disciplines. Further research will indicate if this system can be used for a second load bearing system.
- Integrated load bearing facade. Similar to the prefab floor, applying an integrated facade will reduce the amount of actions, the lead time, the flow of material and the amount of different disciplines.

With the vision for the building method and transport system, it’s possible to make a structural design. During the design phase it’s recommended to keep the transport system in mind. This will result in a better integrated design. When the structural design is finished, the transport system will be revised and redesigned. The new transport design will also be compared with a traditional system and recommendations are made.

Part 1: Structural Design

In the previous chapters a short introduction is provided into the Zalmhaven tower project. In Part 1: Structural design, all the aspects required to design a 200m prefabricated tower will be examined.

6 Criteria for structural design

A large high rise project like the Zalmhaven tower needs an integral and multidisciplinary design. Only the structure and construction methodology will be highlighted in this thesis, but at an engineering firm several different disciplines will be working on the same project.
According to the reader CT2061 Intergraal Ontwerpen in de Civiel Techniek [Ridder 2006] an integral design is a means to increase the value of the end result and reduce unnecessary costs.
Life Cycle Management and Systems Engineering are the current techniques to achieve an integrated design (information transfer in all directions) and these techniques are becoming more and more important in the building industry.
In this chapter different criteria are discussed that are essential for an integrated structural design. The reader CIE4170 Construction Technology of Civil Engineering Projects [Horst 2011] is used to create this chapter.

6.1 Primary criteria

The criteria are divided into two groups: primary and secondary criteria. The following primary criteria are essential to achieve a good and integrated design. They also provide guidance for decisions between different concepts.

<u>Cost</u>
Cost is one of the most important factors. If the project is too expensive, it will never be constructed. With (nearly) unlimited funds (for example the Middle East), the sky is the limit. The largest costs are produced during the execution process and they consist out of personnel, equipment and materials. Little changes in the design can have a large impact on the costs. Therefore, it's very important to know what the cost drivers are and to optimize them. In other words, the execution process should be taken into account during the structural design. The design itself only takes a small portion of the overall costs. The distribution of cost and influence over time are schematized in Figure 27.

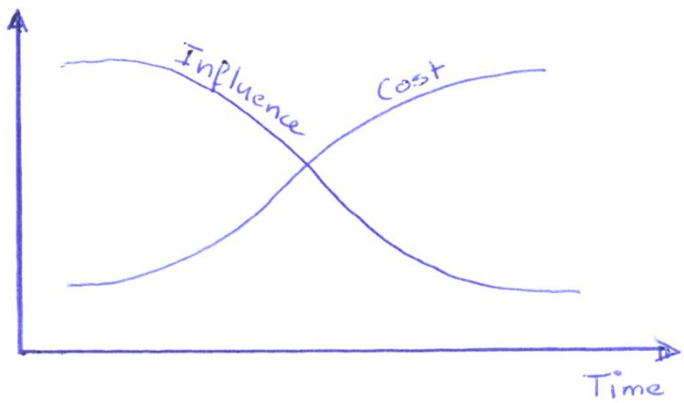


Figure 27 Distribution of cost and influence over time

<u>Risk</u>
A risk is a function of a chance multiplied by a consequence. By combining risk analysis and risk management, the chance and consequence can be determined and reduced. Reducing this will result in less cost and potentially more profit. Profit margins are a special topic in civil engineering: risks are very high and profits are very low compared to other industries. Therefore risks should be managed carefully and this starts with awareness: what are the risks and who bears these risks? After this, the risks should be assessed with a risk analysis. After the assessment, the following actions can be taken: prevention, limit consequences or acceptance. During the design, the influence on the

results is very large. Therefore, the most important risk (for example design errors and delays) should be prevented in this phase.

Capacity
The capacity of the structure contains the stability (Serviceability Limit State and Ultimate Limit State), stiffness (SLS) and strength (SLS and ULS). In order to be constructed, the structure must meet a certain level of these requirements. This level can be found in the standards and every structure must oblige. A good design distinguishes itself from other designs by satisfying these rules in a smart and effective way. For example: a concrete core and facade tube can both fulfil the strength, stiffness and stability requirements, but the core structure needs more material because it's less effective.
During the design, all the phases of the construction should be checked regarding stability, strength and stiffness. Furthermore, all the design choices should be checked on their impact on the time schedule.

6.2 Secondary criteria

Constructability
A good design is easy to construct. By using prefab concrete with simple connections, the constructability increases compared to structure casted on site.
The tolerances of prefabricated structure are an important aspect of the constructability. Compared to cast in situ structure, tolerances are more important in prefabricated structures because they are less easy to adapt. To prevent problems, design aspects that require a small tolerance should be avoided. For example: Figure 28 shows a load bearing facade element from the JuBi tower in The Hague. The elements are divided by a joint and this joint is located at a window opening. This provided large problems with the windows because of inaccuracies during the placement of the elements. Beside the inaccuracies, they also encountered problems with the stone windowsills (vensterbank in Dutch). They forgot to place expansion joints and the windows were placed on a cold day. When the protection foil was removed, they discovered that all the windowsills in the entire building were cracked.

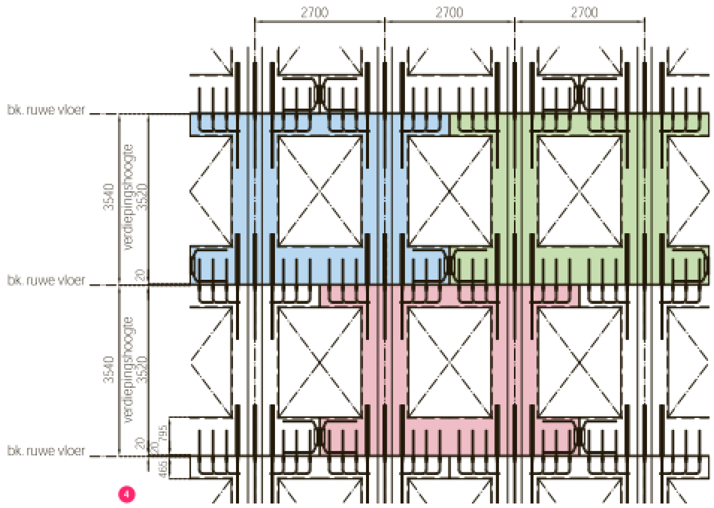


Figure 28 Load bearing facade element with small tolerance [Robbemont 2011]

Durability
An important design aspect is the durability during the lifetime of the structure. The durability performance is governed by two key influence factors: design and execution. During the design, the concrete cover and mix are determined. In combination with special details they have a large impact on the lifespan of a building. During the execution, the design has to be realised. Because of complications and inaccuracies

problems may arise. A very dense reinforcement net could reduce the cover and result in segregated concrete. A poorly execution process could also lead to less denser concrete (air pockets in the concrete) and several other problems. Taking the durability into account during the design will result in an structure with higher value.

Sustainability
A sustainable building is a smart building. The designer should consider the impact of the materials, the amount of materials, the required maintenance during service life, the construction method and the dismantling of the building. Legislation requires a minimal level of sustainability, but occasionally a higher level is required by the owner.

Flexibility
Flexibility is an important aspect for the lifespan of a building. Functional changes, different space requirements and future extensions could be simplified when flexibility is taken into account during the design. A flexible structure could facilitate different functions and this increases the value.

Reliability
Reliability is one of the basic requirements, in combination with safety and serviceability. The reliability of the design is influenced by the correctness of the input parameters. Therefore statistical data is preferred (for example dead and live loads). Experienced judgement and knowledge is necessary when there are uncertainties. A risk analysis in combination with risk management can assist the engineer in order to make correct decisions.

Redundancy
Redundancy is the ability to absorb the unforeseen without disproportional damage. Solutions to prevent damage are: prevent the event, prevent damage due to the event or prevent collapse due to the damage. To elaborate the solutions, an example of a column subjected a car crash is provided: to prevent the event, it’s possible to forbid any traffic around the column. If that’s not possible, an impact structure could be placed around the column to prevent any damage. If that solution is undesired, it’s possible to increase the column diameter to prevent a collapse due to the collision.

Maintenance
Maintenance is an important design criteria and occurs in several other criteria. The required amount of maintenance should be reduced as much as possible because it provides extra costs and nuisance for the occupants. Furthermore, maintenance should simple and accessible. Optimizing maintenance in relation to cost is preferred. For example, nowadays it’s possible to buy glass with a special self cleaning coating. The price of this glass is higher, but the amount of maintenance is reduced.

Quality
The demand for quality is increasing compared to the past and there is an interaction between Design and QHSE (quality, health, safety and environmental care). Material choice, construction method, construction equipment, details and temporary structures can have an influence on the quality. Quality Control and Quality Assurance is used to meet the specified requirements and boundary conditions.

An overview of the design process is illustrated in Figure 29. The structural criteria should be considered in all stages.

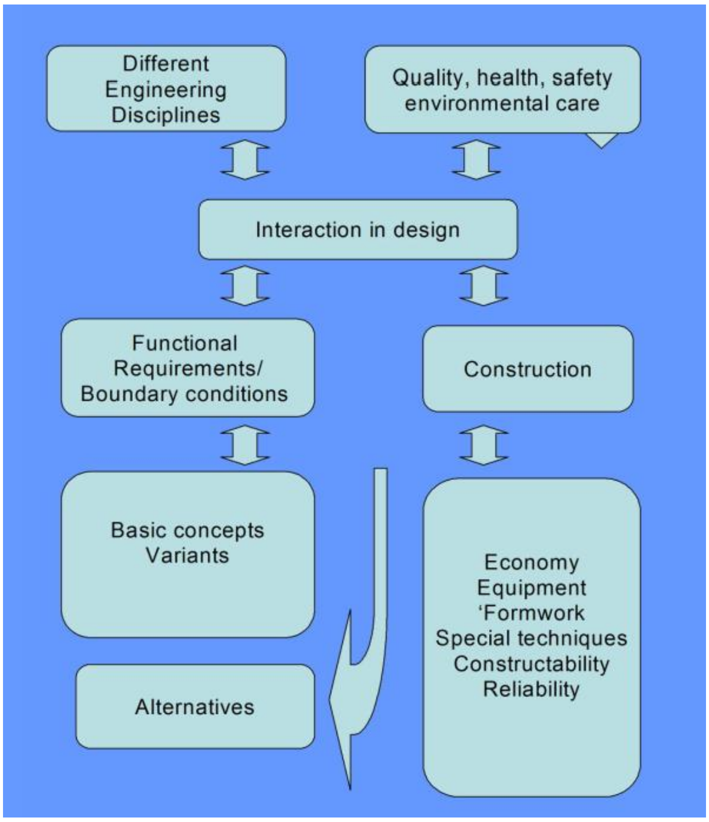


Figure 29 Design process of civil structures [Horst 2011]

The question: "which of these criteria should be used as main control criteria" isn't answered easily. Due to the relations between the criteria, a small change might have large consequences with regard to another criteria. Therefore considering one aspect (for example costs) will result in problems and it's advised to consider all the aspects.

7 Loads

In 2010, most of the Dutch national codes ware replaced by the Eurocode (in 2012 the Eurocode became mandatory). This has several consequences for the calculations. This chapter will start with the definitions and assumptions of the design in section 7.1. Than the differences in wind load between the NEN 6702 and the Eurocode EN 1991-1-4 will be explained in section 7.2. In section 7.3 the Eurocode EN 1991-1-4 will be elaborated and values for the wind load will be calculated. Section 7.4 follows with the acceleration of the building. The wind interference and influence of the surrounding buildings will be explained in section 7.4 and 7.5. In section 7.7 the snow load will be determined. The chapter will end with a conclusion in section 7.8.

7.1 Definition of loads and assumptions of the design

The following assumptions and loads will be used in the design:

Assumptions

Consequence class:	CC3 (large consequences regarding the loss of life, and/or significant economic or social effects on the environment). More information on CC3 can be found in chapter 12.
Reference period class:	3 (50 years for buildings and normal structures).
Fire resistance:	120 minutes.

Dead load

Most of the structure will be made out of concrete. A characteristic self-weight of concrete of $25kN/m^3$ is used in calculations.

For separation walls and other small elements placed on the floors a uniform distributed load of $0.5kN/m^2$ is taken into account.

Live loads

The following live loads and combination factors φ_0 (based on NEN-EN 1990/NB) will be taken into account in the design:

- Dwellings (cat. A) — φ_0=0.4
  - Floors — $2.0kN/m^2$
  - Traffic area — $2.5kN/m^2$
  - Stairs — $2.0kN/m^2$
- Public function (cat. C) — φ_0=0.4or 0.6[3]
  - Entrance hall/sky lobby — $5.0kN/m^2$
- Parking spaces (cat. F) — φ_0=0.7
  - Floors — $2.5kN/m^2$

Maximum 1 floor is fully loaded when combined with another load. All the remaining floors loads are multiplied with φ_0.

The wind and snow load are determined in chapter 7.3 and 7.7.

[3] φ_0=0.6 when the area is also used as escape route

7.2 Differences between the NEN EN 1991-1-4 and NEN 6702

With the introduction of the Eurocode EN 1991-1-4, students and professionals have to alter their calculations. The differences between the codes are rather small and this section will elaborate on the reasons for these differences in the calculations [Geurts 2011].

To calculate the wind load on buildings, a wind loading chain is used: wind – area – building – structure – criteria. The first deviation occurs at the wind section:

- The wind speed in the Eurocode is calculated with a repetition time of 50 years. The NEN 6702 uses a repetition time of 12.5 years[4].
- The Eurocode uses a 10 minutes average for the wind speed, the NEN 6702 assumes a 1 hour average.

A 10 minutes average with a repetition time of 50 years gives higher values for the wind speed ($v_{b,0}$) than a 1 hour average with a repetition time of 12.5 years.

At the area section also some changes have been applied. In the Eurocode it is possible to choose from 5 area categories. In the NEN6702 there were only 2 categories (non-built and built-up: onbebouwd and bebouwd in Dutch).

Large changes have taken place at the force coefficient of buildings. The load on the main load bearing structure is determined by the pressure distribution on the outer shell. This pressure distribution consists out of a wind trust multiplied by a force coefficient. In the NEN 6702 a force coefficient of 0.8-(-0.4)=1.2 was applied regardless of the building seize. In the EN 1991-1-4, the force coefficient depends on the slenderness of the building. Table 5 shows the force coefficients according to the Eurocode.

Table 5 Form coefficient according to EN 1991-1-4 [Geurts 2011]

zone	*D*	*E*	*reductiefactor*	*totale vormfactor*	*% verschil met NEN 6702*
h/d	$c_{pe,10}$	$c_{pe,10}$			
5	+0,8	-0,7	1,0	1,5	+25%
1	+0,8	-0,5	0,85	1,1	-8%
≤0,25	+0,7	-0,3	0,85	0,85	-30%

The Dutch National annex is more moderate and does not follow the Eurocode entirely. The large reduction factor for the non-slender buildings (≤0.25) is not applied and very slender buildings are allowed to use a reduction factor. This results in a new table of force coefficients in the Dutch National annex (see Table 6). Between the values linear interpolation is allowed.

[4] The repetition time of 50 years was divided by four, because a rectangular building in east-west direction would only be fully loaded when the wind was blowing from the north or the south. The other remaining 6 directions are less critical and it was considered that a repetition time of 12.5 years would be sufficient. It should be noted that this does not hold for all building shapes (for example round buildings). See section 8.6.2 of NEN 6702.

Table 6 Form coefficient according to EN 1991-1-4 with National annex [Geurts 2011]

zone	D	E	reductiefactor	totale vormfactor	% verschil met NEN 6702
h/d	$c_{pe,10}$	$c_{pe,10}$			
5	+0,8	-0,7	0,85	1,3	+8%
≤ 1	+0,8	-0,5	0,85	1,1	-8%

Both tables also show the difference between EN 1991-1-4 and NEN 6702. Besides the force coefficient, also the reference height and the friction have changed. These changes result in higher loads for high rise structures and lower loads for low rise structures.

At the structure and criteria several changes have been applied. The factor $c_{dim}f_1$ from NEN 6702 is now replaced by c_sc_d. c_s is a reduction factor because the non simultaneous wind gusts aren't present at the entire structure. c_d is the dynamic magnification factor. A new elaborate formula is applied to calculate c_sc_d.

It can be concluded that the Eurocode has more parameters to determine the wind load (for example the slope of the area, influence of surrounding buildings and wind directions). This results in more accurate calculations, especially for high rise structures, but they also are more time consuming than the NEN 6702.

7.3 Wind loads according to NEN EN 1991-1-4 and Dutch National annex

The NEN EN 1991-1-4 gives guidelines to calculate the wind load. Because of to the National Annex, the calculation deviates slightly from the original Eurocode code. The calculation is based on the scheme of Table 7 and can be divided in three phases:

- phase 1: peak velocity pressure,
- phase 2: wind pressure on surfaces,
- phase 3: wind loads on the structure.

Table 7 Calculation scheme according to NEN EN 1991-1-4 and Dutch National annex [NEN-EN 2005]

Parameter
extreme stuwdruk q_p
basiswindsnelheid v_b
referentiehoogte z_e
terreincategorie
karakteristieke extreme stuwdruk q_p
turbulentie-intensiteit I_v
gemiddelde windsnelheid v_m
orografiefactor $c_o(z)$
ruwheidcoëfficiënt $c_r(z)$
Winddruk, bijvoorbeeld voor bekleding, bevestigingen en constructiedelen
uitwendige drukcoëfficiënt c_{pe}
inwendige drukcoëfficiënt c_{pi}
nettodrukcoefficient $c_{p,net}$
uitwendige winddruk $w_e = q_p\, c_{pe}$
inwendige winddruk: $w_i = q_p\, c_{pi}$
Windkrachten op constructies, bijvoorbeeld voor algemene windeffecten
bouwwerkfactor $c_s c_d$
windkracht F_w berekend met krachtcoëfficiënten
windkracht F_w berekend met drukcoëfficiënten

7.3.1 Phase 1: peak velocity pressure

In this section the Peak velocity pressure will be calculated, starting with the basic wind velocity and ending with the roughness factor.

7.3.1.1 Basic wind velocity

The basic wind velocity is defined as a function of the wind direction and the time of year at 10m above the ground. The formula is given by:

$$v_b = c_{dir} * c_{season} * v_{b,0}$$

in which:

c_{dir} is the wind direction factor (recommended value: c_{dir}=1.0),
c_{season} is the wind season factor (recommended value: c_{season}=1.0),
$v_{b,0}$ is the fundamental value for the basic wind velocity, see Figure 30).

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

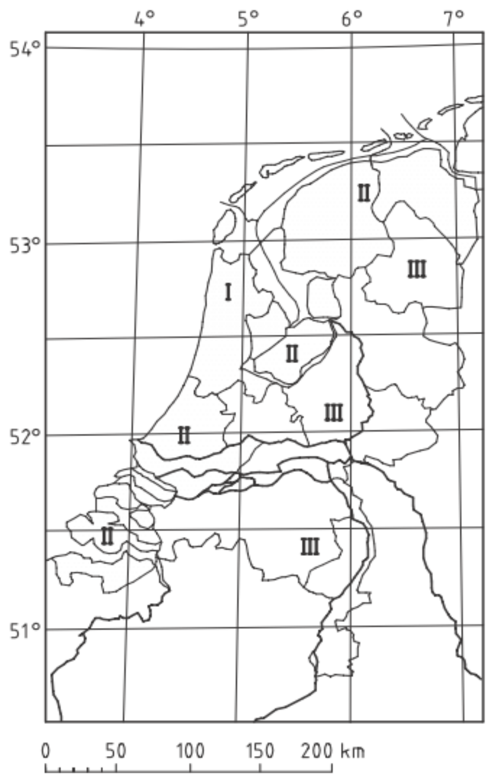


Figure 30 Values for $v_{b,0}$ (ULS) [NEN-EN 2005]

An important remark has to be made: the fundamental value for the basic wind velocity shown in Figure 30 is used for the calculation of the wind load F_w and the structural factor c_sc_d. When the acceleration is calculated, the basic wind velocity may be reduced to 19.4m/s in area II according to the NTA Hoogbouw (03-A Wind) report. This is because the accelerations are calculated with a repetition time of once every year while the wind load and structural factor are calculated with a repetition time of once every 50 years (estimated lifetime of the building).

7.3.1.2 Reference height

The reference height z_e for rectangular buildings depend on the slenderness. The reference height is located at the top of each level. To reduce the amount of different sections with a different reference height, the following methods may be applied:

- If the height is smaller than the width, the building may be considered as one section.
- If the height is larger than the width, but smaller than twice the width, the building may be divided in two sections.
- If the height is larger than twice the width, the building may considered to exist out of multiple sections

In Figure 31 this is visualised.

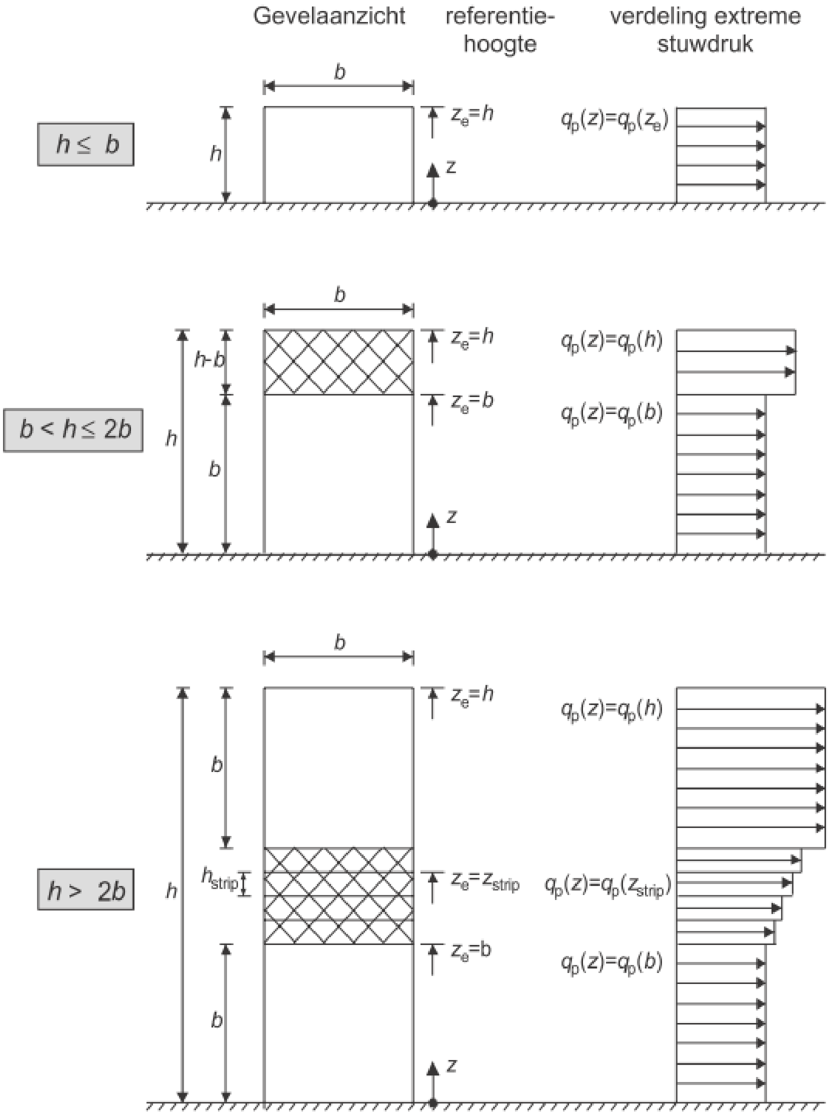


Figure 31 Reference height z_e [NEN-EN 2005]

7.3.1.3 Terrain category

Several terrain categories are specified in the Eurocode (see Table 8). The Dutch National annex deviates from this list and provides a new list (see Table 9). For high rise projects only category 0 or II may be used.

Table 8 Terrain categories [NEN-EN 2005]

Terreincategorie	z_0 m	z_{min} m
0 Zee of kustgebied met wind aanstromend over open zee	0,003	1
I Meren of vlak en horizontaal gebied met verwaarloosbare vegetatie en zonder obstakels	0,01	1
II Gebied met lage begroeiing als gras en vrijstaande obstakels (bomen, gebouwen) met een tussenruimte van ten minste 20 obstakelhoogtes	0,05	2
III Gebied met regelmatige begroeiing of gebouwen of vrijstaande obstakels met een tussenruimte van ten hoogste 20 obstakelhoogtes (zoals dorpen, voorstedelijk terrein, blijvend bos)	0,3	5
IV Gebied waar ten minste 15 % van de oppervlakte is bedekt met gebouwen met een gemiddelde hoogte boven 15 m	1,0	10
De terreincategorieën zijn geïllustreerd in A.1.		

Table 9 Terrain categories according to National annex [NEN-EN 2005]

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

7.3.1.4 Peak velocity pressure

The peak velocity pressure $q_p(z)$ at height z, which includes mean and short term velocity fluctuations, is given by:

$$q_p(z)=(1+7*l_v(z))*0.5*\rho*v_m^2(z)$$

in which:

l_v is the wind turbulence intensity, see section 7.3.1.5,
ρ is the air density, ρ=1.25kg/m^3,
$v_m(z)$ is the mean wind velocity at a height z above the terrain, see 0.

7.3.1.5 Wind turbulence intensity

The wind turbulence will be taken into account with the use of the turbulence intensity l_v, given by:

$$l_v(z)=k_1/(c_0(z)*\ln(z/z_0)) \quad \text{for } z_{min}\leq z\leq z_{max}$$
$$l_v(z)=l_v(z_{min}) \quad \text{for } z<z_{min}$$

in which:

k_1 is the turbulence factor, k_1=1.0 (recommended, but conservative),
c_0 is the orography factor, see section 0,
z_0 is the roughness length, see Table 9.

7.3.1.6 Mean wind velocity, roughness- and orography factor

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness, orography and on the basic wind velocity v_b:

$$V_m(z)=c_r(z)*c_0(z)*v_b$$

in which:

$c_r(z)$ is the roughness factor. It accounts for the mean wind velocity at the site due to the ground roughness of the terrain upwind of the structure in the considered wind direction,

$C_0(z)$ is the orography factor. Effects of orography may be neglected when the average slope of the upwind terrain is less than 3° (recommended value: $c_0(z)=1.0$),

v_b is the basic wind velocity. See 7.3.1.1.

The roughness factor can be calculated with the following formula:

$$c_r(z)=k_r*\ln(z/z_0) \qquad \text{for } z_{min}\leq z\leq z_{max}$$
$$c_r(z)=c_r(z_{min}) \qquad \text{for } z<z_{min}$$

in which:

z_0 is the roughness length, depending on the terrain category. See 7.3.1.3,

z_{min} is the minimum height. See 7.3.1.3,

z_{max} is the maximum height, z_{max}=200m,

k_r is the terrain factor depending on the roughness length z_0 given by:

$$k_r=0.19*(z_0/z_{0,II})^{0.07}$$

in which:

z_o is the roughness length, depending on the terrain category. See 7.3.1.3,

$z_{o,II}$ is the reference value (category II, see Table 8): $z_{o,II}$=0.05.

When all the values are calculated, the peak velocity pressure $q_p(z)$ (see 7.3.1.4) can be obtained. This has already been done in the Dutch National annex and the values can be found in Table 10.

Table 10 Peak velocity pressures in kN/m^2 as a function of the height in the NEN-EN 1991-1-4 [NEN-EN 2005]

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

When these values are compared to the values of NEN 6702 (see Table 11), it can be concluded that the Eurocode gives more moderate values for the peak velocity pressure in area II and III. In area I this is reversed and the Eurocode give in most cases higher values. This is because of the new roughness values in the Eurocode.

Table 11 Peak velocity pressures in kN/m^2 as a function of the height in the NEN 6702 [NEN]

h [m]	P_w [kN/m^2] Gebied 1 Onbebouwd	Bebouwd	Gebied 2 Onbebouwd	Bebouwd	Gebied 3 Onbebouwd	Bebouwd
≤ 2	0.64	0.64	0.54	0.54	0.46	0.46
3	0.70	0.64	0.54	0.54	0.46	0.46
4	0.78	0.64	0.62	0.54	0.49	0.46
5	0.84	0.64	0.68	0.54	0.55	0.46
6	0.90	0.64	0.73	0.54	0.59	0.46
7	0.95	0.64	0.78	0.54	0.63	0.46
8	0.99	0.64	0.81	0.54	0.67	0.46
9	1.02	0.64	0.85	0.54	0.70	0.46
10	1.06	0.70	0.88	0.59	0.73	0.50
11	1.09	0.76	0.91	0.64	0.76	0.54
12	1.12	0.81	0.94	0.68	0.78	0.58
13	1.14	0.86	0.96	0.72	0.80	0.61
14	1.17	0.90	0.99	0.76	0.82	0.64
15	1.19	0.94	1.01	0.79	0.84	0.67
16	1.21	0.98	1.03	0.82	0.86	0.70
17	1.23	1.02	1.05	0.85	0.88	0.72
18	1.25	1.05	1.07	0.88	0.90	0.75
19	1.27	1.08	1.09	0.90	0.91	0.77
20	1.29	1.11	1.10	0.93	0.93	0.79
25	1.37	1.23	1.18	1.03	1.00	0.88
30	1.43	1.34	1.24	1.12	1.06	0.95
35	1.49	1.43	1.30	1.20	1.11	1.02
40	1.54	1.50	1.35	1.26	1.15	1.07
45	1.58	1.57	1.39	1.32	1.19	1.12
50	1.62	1.62	1.43	1.37	1.23	1.16
55	1.66	1.66	1.46	1.42	1.26	1.20
60	1.69	1.69	1.50	1.46	1.29	1.24
65	1.73	1.73	1.53	1.50	1.32	1.27
70	1.76	1.76	1.56	1.54	1.34	1.31
75	1.78	1.78	1.58	1.57	1.37	1.33
80	1.81	1.81	1.61	1.60	1.39	1.36
85	1.83	1.83	1.63	1.63	1.41	1.39
90	1.86	1.86	1.65	1.65	1.43	1.41
95	1.88	1.88	1.68	1.68	1.45	1.44
100	1.90	1.90	1.70	1.70	1.47	1.46
110	1.94	1.94	1.74	1.74	1.51	1.50
120	1.98	1.98	1.77	1.77	1.54	1.54
130	2.01	2.01	1.80	1.80	1.57	1.57
140	2.04	2.04	1.83	1.83	1.60	1.60
150	2.07	2.07	1.86	1.86	1.62	1.62

7.3.2 Phase 2: wind pressure on surfaces

Wind actions on structures and structural elements should be determined taking into account both external and internal wind pressures. The wind pressure acting on external surfaces is given by:

$$w_e = q_p(z_e) * c_{pe}$$

The wind pressure acting on internal surfaces of a structure is given by:

$$w_i = q_p(z_i) * c_{pi}$$

in which:

$q_p(z_e)$ and $q_p(z_i)$	are the peak velocity pressures, see Table 10,
c_{pe} and c_{pi}	are the pressure coefficients for either external or internal pressure.

The total pressure on a wall, roof or element is given by the difference between the internal and external pressures (see Figure 32). The most adverse situation needs to be taken into account.

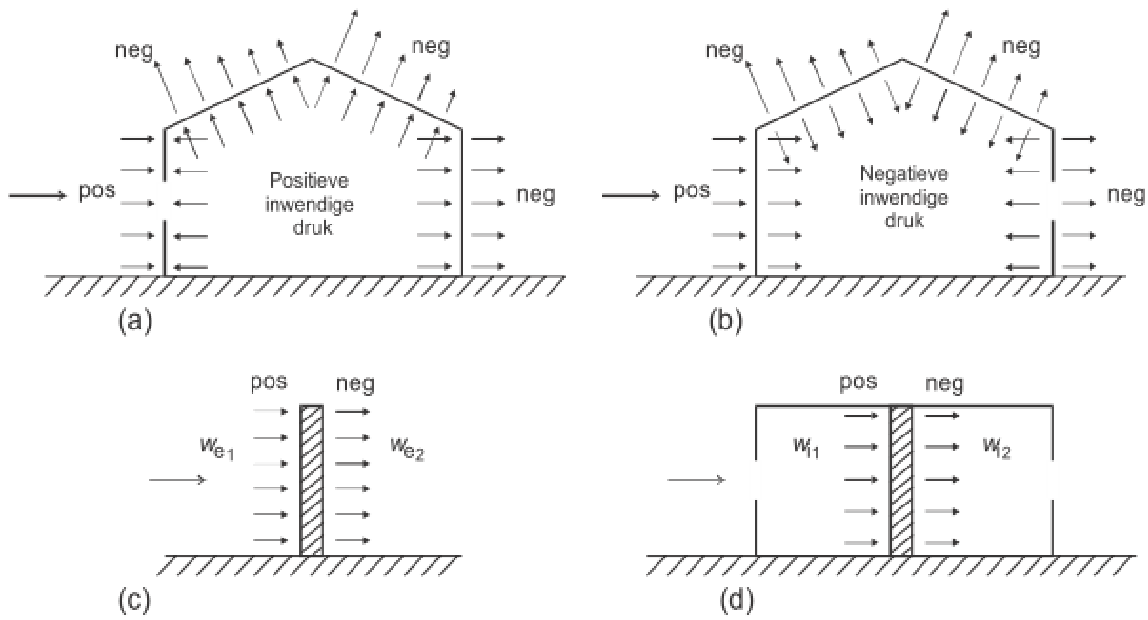


Figure 32 wind pressure on surfaces [NEN-EN 2005]

7.3.2.1 External pressure coefficients

The external pressure coefficient c_{pe} for buildings and sections of buildings depends on the loaded area A. The Eurocode has coefficients for a large amount of different building configurations. For square buildings, the coefficients are displayed in Table 6.

7.3.2.2 Internal pressure coefficients

The internal pressure coefficient c_{pi} depends on the size and distribution of the openings in the building envelope. The rules in the code are very specific and depend strictly on the design of the structure. For design purposes, an internal pressure coefficient of:

$$c_{pi} = +/- \ 0.3$$

is used. Interaction between the wind direction and the estimated internal pressure will not be taken into account.

7.3.3 Phase 3: wind loads on the structure

The wind force F_w for the structure or a structural component should be determined by using either the force coefficient method which include all effects:

$$F_w=c_sc_d*c_f*q_p(z_e)*A_{ref}$$

or by a vectorial summation of the forces: $F_{w,e}$, $F_{w,i}$ and F_{fr}, representing respectively the forces due to external pressure, internal pressure and friction:

$$F_{w,e}=c_sc_d \sum_{surfaces} w_e*A_{ref}$$

$$F_{w,i}=c_sc_d \sum_{surfaces} w_i*A_{ref}$$

$$F_{fr}=c_{fr}*q_p(z_e)*A_{fr}$$

in which:

c_sc_d is the structural factor for taking into account the effect of wind actions from the non-simultaneous occurrence of peak wind pressures on the surface (c_s) together with the effect of the vibrations of the structure due to turbulence (c_d). See 7.3.3.2 for the calculation of c_sc_d,

c_f is the force coefficient for the structure or a structural element (see Table 6),

w_e is the external pressure on a surface at height z_e (see 7.3.2.1),

w_i is the internal pressure on a surface at height z_i (see 7.3.2.2),

A_{ref} is the reference area of the surface,

A_{fr} is the friction area of the surface.

The first method is preferred for the design of a stability structure. For local situations as facade elements or columns, method 2 with the vectorial summation of forces on surfaces should be used.

7.3.3.1 Calculation of the structural factor c_sc_d

The structural factor c_sc_d takes into account the effect of wind actions from the non-simultaneous occurrence of peak wind pressures on the surface (c_s) together with the effect of the vibrations of the structure due to turbulence (c_d). The factor c_sc_d may be assumed equal to 1 if:

- the building is smaller than 15m,
- the structure contains a framework and several stability walls in combination with a maximum height of 100m, whereby the height should be smaller than four times the building depth.

For all the buildings that fall outside the limits given above, the structural factor has to be calculated. For general shapes, such as vertical structures, this can be done with the following equation:

$$c_sc_d=\frac{1+2k_pl_v(z_s)\sqrt[2]{B^2+R^2}}{1+7l_v(z_s)}$$

in which:

z_s is the reference height for the structural factor. For vertical structures this is equal to (see Figure 33 and for z_{min} see 7.3.1.3):

$$z_s=0.6*h\geq z_{min}$$

k_p is the peak factor defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation (see 7.3.3.2),
I_v is the wind turbulence factor (see 7.3.1.5),
B^2 is the background factor (see 7.3.3.3),
R^2 is the resonance response factor (see 7.3.3.4).

In the Eurocode, it's recommended to use annex B to calculate the structural factor. Annex C gives an alternative calculation and the difference between the two calculations is less than 5%. Because annex B gives systematically lower values than annex C (small "mistakes" are made in annex B), it's advised to use annex C. In this calculation annex C will be used.

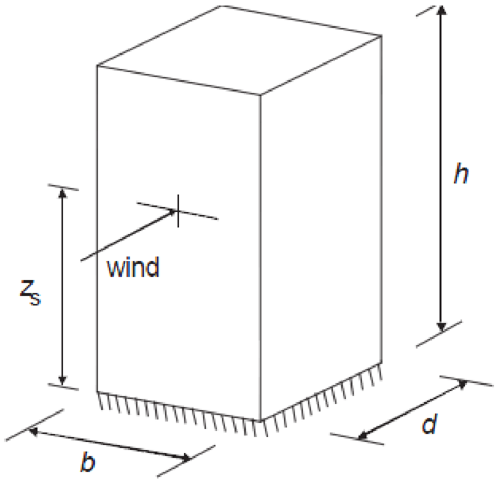


Figure 33 Calculation of z_s for vertical structures [NEN-EN 2005]

7.3.3.2 Peak factor k_p

The peak factor k_p is the largest value of the following equations:

$$k_p=\sqrt[2]{2\ln(v*T)}+\frac{0.6}{\sqrt[2]{2\ln(v*T)}}$$

or

$$k_p=3$$

in which:
v is the frequency of a gust, calculated with the following formula:

$$v=n_{1,x}\sqrt[2]{\frac{R^2}{B^2+R^2}}\geq 0.08hz$$

in which:
$n_{1,x}$ is the natural frequency of the structure,
B^2 is the background factor (see 7.3.3.3),
R^2 is the resonance response factor (see 7.3.3.4).

T is the average time of the reference wind speed. T=600s is recommended.

7.3.3.3 Background factor B^2

The background factor allows for the lack of full correlation of the pressure on the structure surface, given by:

$$B^2 = \frac{1}{1 + 1.5 * \sqrt[2]{(\frac{b}{L(z_s)})^2 + (\frac{h}{L(z_s)})^2 + (\frac{b}{L(z_s)} + \frac{h}{L(z_s)})^2}}$$

in which:

h and b — are the width and height of the structure,

$L(z_s)$ — is the turbulent length scale at reference height z_s given by:

$$L(z_s)=L_t(z_s/z_t)^a \text{ for } z_s \geq z_{min}$$
$$L(z_s)=L(z_{min}) \text{ for } z_s < z_{min}$$

in which:

z_t — is the reference height: z_t=200m,

L_t — is the reference length scale: L_t=300m,

a — =0.67+0.05ln(z_0). For z_0 see Table 8.

The estimation B^2=1 is on the safe side.

7.3.3.4 Resonance response factor R^2

The resonance response factor allows for turbulence in resonance with the considered vibration mode of the structure, given by:

$$R^2=\pi^2 * S_L(z_s,n_{1,x}) * K_s(n_{1,x})/2\delta$$

in which:

δ — is the total logarithmic decrement of the damping given by:

$$\delta=\delta_s+\delta_a+\delta_d$$

in which:

δ_s — is the logarithmic decrement of the structural damping. For reinforced concrete buildings δ_s=0.10 may be used (see table F.2 of annex F.5, NEN-EN 1991-1-4),

δ_a — is the logarithmic decrement of the aerodynamic damping. For constant modal deflections the along wind vibrations may be estimated by:

$$\delta_a = \frac{c_f \rho b v_m(z_s)}{2 n_1 m_e}$$

in which:

c_f — is the force coefficient for the structure or a structural element (see Table 6),

ρ — is the air density: ρ=1.25kg/m^3,

m_e — is the equivalent mass per unit length given by:

$$m_e = \frac{\int_0^1 m(s)\Phi_1^2(s)ds}{\int_0^1 \Phi_1^2(s)ds}$$

since the fundamental mode is given by:

$$\Phi_1(z) = (\frac{z}{h})^{\xi}$$

which is linear over the height for buildings with a central core plus peripheral columns ($\xi = 1$). The equivalent mass is equal to the constant average mass per unit of facade area (see section 7.4).

The logarithmic decrement of the aerodynamic damping is commonly not used: δ_a=0.

δ_d is the logarithmic decrement of the damping due to special devices (tune mass dampers, water tanks etc.). If special damping devices have been added to the structure, δ_d has to be calculated with suitable theoretical or experimental techniques.

$n_{1,x}$ is the natural frequency of the structure. The fundamental flexural frequency n_1 of multi-story buildings with a height larger than 50m can be estimated with:

$$n_1=46/h \text{ [Hz]}$$

Unfortunately, this provides very inaccurate results for the natural frequency and it's strongly advised to use annex A.4 of NEN 6702 to calculate the natural frequency.

S_L is the wind power spectra density function given by:

$$S_L(z,n) = \frac{6.8 * f_L(z,n)}{(1 + 10.2 * f_L(z,n))^{5/3}}$$

where $f_L(z,n)$ is a non-dimensional frequency determined by the natural frequency of the structure n_1, the mean velocity $v_m(z)$ and the turbulence length scale $L(z)$:

$$f_L(z,n) = \frac{n_1 * L(z)}{v_m(z)}$$

K_s is the size reduction function given by:

$$K_s(n) = \frac{1}{1 + \sqrt[2]{(G_y * \phi_y)^2 + (G_z * \phi_z)^2 + (\frac{2}{\pi} G_y * \phi_y * G_y * \phi_y)^2}}$$

with:

$$\phi_y = \frac{c_y bn}{v_m(z_s)} \quad \text{and} \quad \phi_z = \frac{c_z bn}{v_m(z_s)}$$

in which:

c_y and c_z are the decay constants (both equal to 11.5),

G_y and G_z are constants depending on the mode shape variation along horizontal y-axis and vertical z-axis respectively, for buildings an uniform horizontal mode shape variation and a parabolic vertical mode shape variation are assumed. NEN EN 1991-1-4 annex C Table C.1 gives G_y=1/2 and G_z=5/18.

7.3.3.5 Calculation of wind load F_w

The calculation of the wind load is quite complex for a 200m building. The structural factor c_sc_d is mainly responsible for this complexity. To prevent any recalculations every time something changes, a parametric wind calculation in Maple is made. To calculate c_sc_d, detailed information is required: the first natural frequency and the constant average mass per unit of height. This information is not available in this phase and therefore information of the current design by Zonneveld ingenieurs is used (see chapter 2). It is expected that the first natural frequency calculated at this design is more accurate than the estimation:

$$n_1=46/h=46/200=0.23 \text{ [Hz]}$$

At the current design (with a height of 190m), Zonneveld ingenieurs calculated the first natural frequency:

$$n_{1,cur,design}=0.193 \text{ [Hz]}$$

Because the new design is 10m higher, the frequency will be lower than the current design, resulting in a slightly higher acceleration. In this estimation this will not be taken into account.
Zonneveld also made a load calculation. At the ground level there is a total dead load of 804805kN (909kN/m^2). With this calculation it's very easy to take the 10m height difference into account. This results in an estimated mass per unit of length of:

$$m_{e1}=\frac{200}{190}*\frac{804805*10^3}{9.81*200}=4.3179*10^5 \text{ [kg/m]}$$

These values are entered in a Maple sheet that can be found in Appendix A.

In Figure 34 the peak velocity pressures in kN/m^2 as a function of the height is calculated with Maple. The values are equal to the values from the Dutch National annex (see Table 10, area II unbuild (onbebouwd in Dutch) and the calculation may be considered as valid.

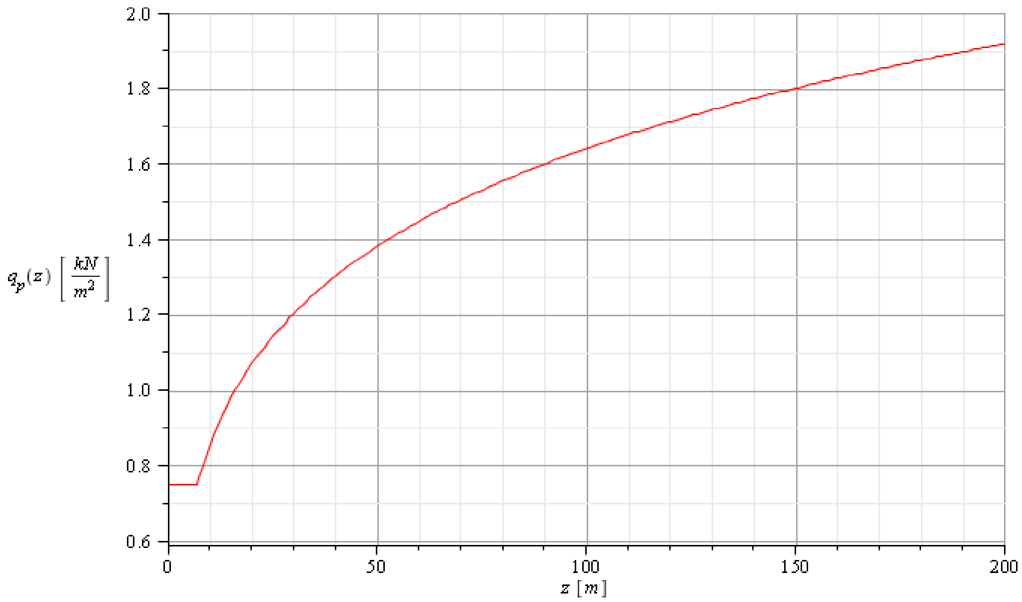


Figure 34 Peak velocity pressures as a function of the height[5]

[5] Unfortunately Matlab automatically plots the variable (z) on the horizontal axis. There are solutions to convert the axis, but the applied formula prohibits this.

Figure 34 was made to validate the Maple model. With the validated peak velocity pressures, the structural factor c_sc_d and the force coefficient c_f the wind force on the structure can be calculated.

The Maple calculation resulted in the following structural factor: c_sc_d=1.07. After the creation of the Maple sheet, a TNO Excel calculation was obtained. With this Excel sheet, the Maple sheet was validated and the value was identical: $c_sc_{d,TNO}$=1.07. This value is larger than 1.0, resulting in a magnification factor.

The force coefficient can be determined with Table 5: h/d=200/30=6.667. This results in c_f=(0.8-(-0.7))*0.85=1.3.

The calculation of the second order effect can be found in section 10.4.6. A SLS magnification factor of 1.08 is calculated. Due to many uncertainties, this value will be set at 1.1.

Now the wind pressure on the Zalmhaven tower can be calculated by Maple. Figure 35 shows the result. The reference height (see Figure 31) is included and the boxed sections are replaced by a line. The vertical line section at 170m is there because at the top box (the building width is equal to the height of the top box: 30m) the maximum pressure at 200m is used.

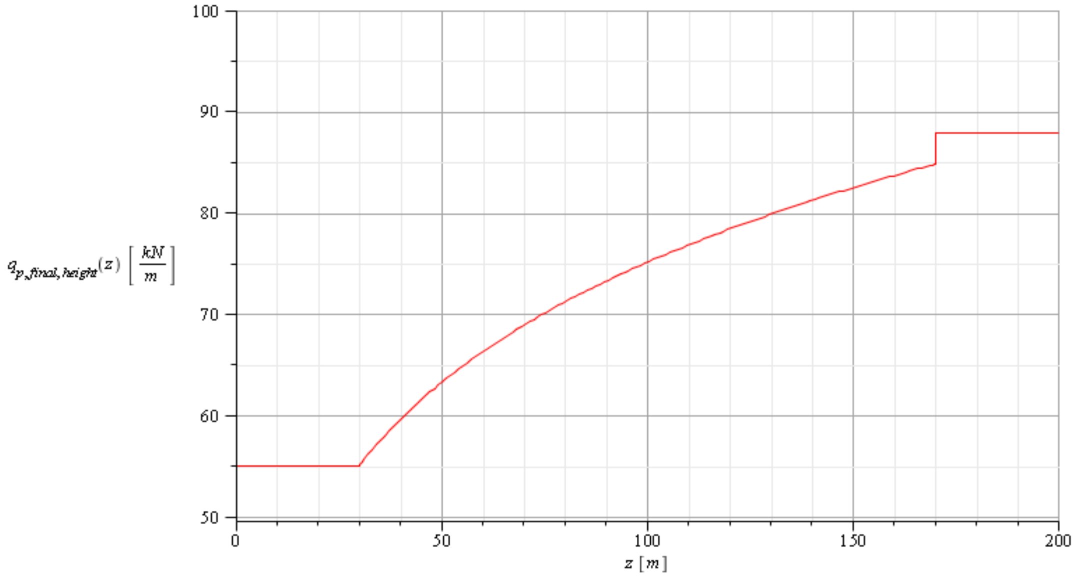


Figure 35 Wind pressure as a function of the height

At the section 0-30.50m a wind pressure of 55.07kN/m is obtained. At 51.85m this value increases to 64.55kN/m. At 100.65 and 149.45m a value of respectively 75.14 and 82.06kN/m is obtained. At the section 170.80-200 the maximum wind load of 88.10kN/m is encountered. When these values are compared with the results of the calculation made by Zonneveld ingenieurs (see report 2 of the literature study), it can be concluded that the Eurocode values are higher at the bottom (approximately 10%), see Figure 36.

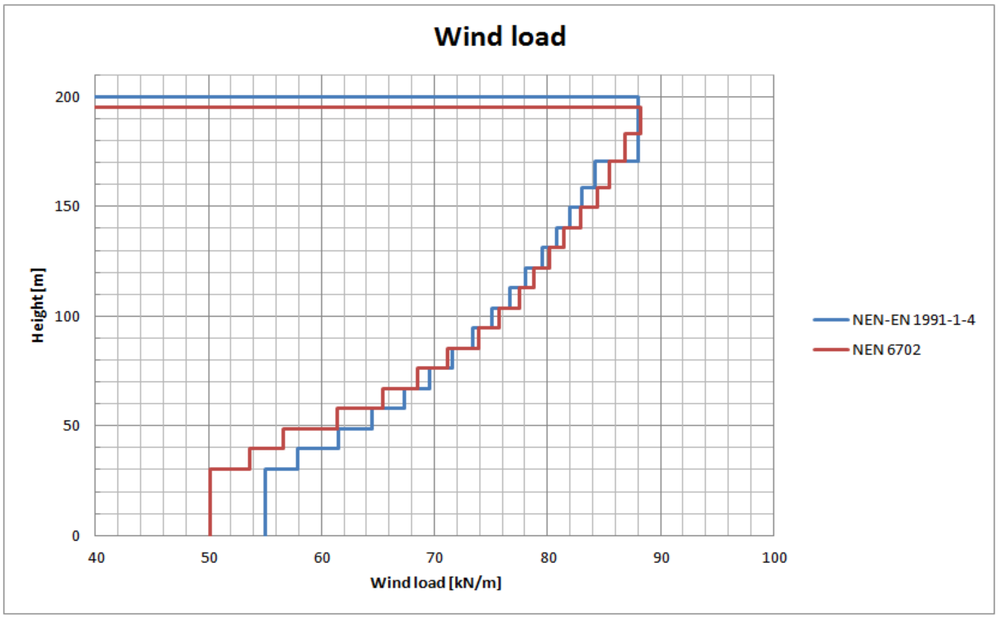


Figure 36 Difference in wind load

This is unexpected because the wind load values of Table 10 (NEN-EN 1991-1-4) are lower than Table 11 (NEN 6702) for area II. The fact the Eurocode load is higher can be explained by the following aspects:

- Zonneveld ingenieurs used the wind load for a build-up area (bebouwd in Dutch), but for a high rise tower the unbuild area load is required,
- in the Eurocode the force coefficient has increased from 1.2 to 1.3,
- Zonneveld ingenieurs uses a second order effect of 1.073. In this calculation the value is assumed at 1.1.

The structural factor slightly decreases this difference ($c_s c_d$=1.067 versus $C_{dim} \cdot \varphi_1$=1.088). When Zonneveld ingenieurs would apply the unbuild wind load and a height of 200m, Figure 37 is obtained.

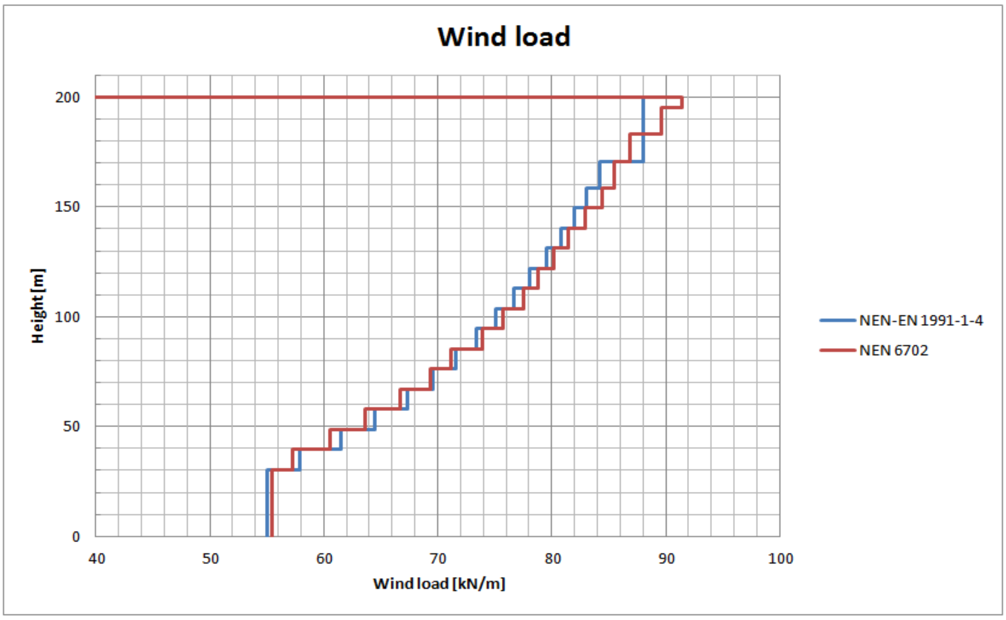


Figure 37 Adapted difference in wind load

At the bottom, the Eurocode values are slightly larger but from 85m and higher, the NEN 6702 values become larger. Between Figure 36 and Figure 37 the largest differences occur at the bottom, because the build and unbuild values of NEN 6702 are equal from 85m and higher (see Table 11).

It might be concluded that despite the higher force coefficient and the larger second order effect, the NEN-EN 1991-1-4 provides lower values than the NEN 6702 at higher altitudes. Below 85m, the NEN-EN 1991-1-4 provides slightly higher values because the difference between p_w and q_p is smaller (this are the peak velocity pressure values, see Table 10 and Table 11).

7.4 Acceleration due to cyclic wind loading

This section will be devoted on vibrations and accelerations due to cyclic wind loading. The building's natural eigenfrequency is an important measurement to ensure whether the building's comfort is still within an acceptable range.

Eurocode NEN-EN 1991-1-4 and the "NTA Hoogbouw (03-A Wind)[6]" report give guidelines to calculate the frequency and acceleration. Most FEM programs are currently able to calculate the natural eigenfrequency of a structure. These values are more accurate than the design formulas when the program is used correctly. But in order to obtain accurate values with a FEM program, a large amount of information has to be entered in the model. In Figure 38 the Dutch comfort criteria is depicted. It can be noted that office buildings have less strict requirements.

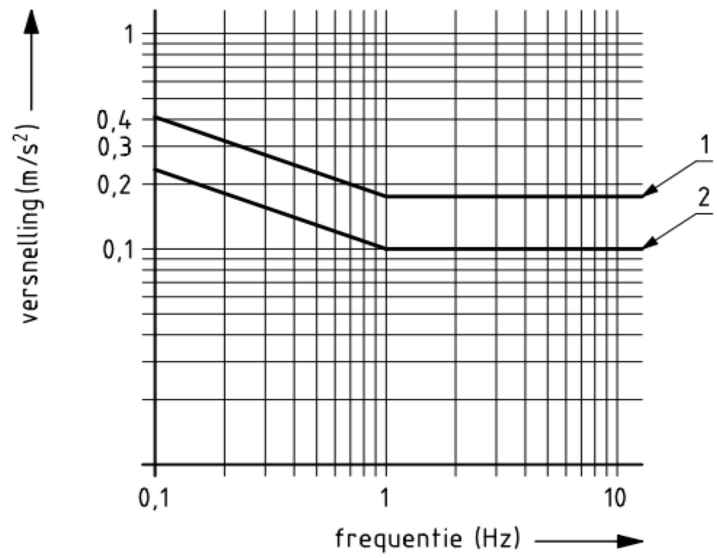


Figure 38 Comfort criteria for Dutch office buildings (1) and residential buildings (2) [NEN 6702]

In the Eurocode, it's recommended to use annex B to calculate the accelerations. Annex C gives an alternative calculation and the difference between the two calculations is less than 5%. Because annex B gives systematically lower values than annex C (small "mistakes" are made in annex B), it's advised to use annex C in the Netherlands. The following formulas can be found in the Eurocode NEN-EN 1991-1-4 annex C.4.

The characteristic acceleration of a building is given by (annex C.4 (3)):

$$\hat{a} = k_p * \sigma_a$$

[6] The National Technical Arrangements supplement the existing code. The NTA is a recommendation and it's not obligatory (similar to the Code of Good practice).

in which:

k_p is the peak factor, see section 7.3.3.2,

σ_a is the standard deviation of the acceleration.

This formula has the form of x=μ+k*σ. The expected value μ for the acceleration is equal to zero, because the positive acceleration is equal to the negative acceleration (not taking damping into account). The acceleration behaves as a sinus function around the x-axis.

The standard deviation of the acceleration can be calculated with (annex C.4):

$$\sigma_{a,x}(y,z) = c_f * \rho * l_v(z_s) * v_m^2(z_s) * R * \frac{K_y * K_z * \Phi(y,z)}{\mu_{ref} * \Phi_{max}}$$

in which:

c_f is the force coefficient for the structure or a structural element (see Table 6),

ρ is the air density, ρ=1.25kg/m^3,

$l_v(z_s)$ is the wind turbulence intensity at a height z_s above the terrain, see section 7.3.1.5,

$v_m(z_s)$ is the mean wind velocity at a height z_s above the terrain, see 0,

R is the square root of the resonance response factor, see 7.3.3.4,

K_y =1 (constant uniform value, given in table C.1 in NEN-EN 1991-1-4),

K_z =5/3 (constant parabolic value, given in table C.1 in NEN-EN 1991-1-4),

μ_{ref} is the reference mass per unit area on which the load acts,

$\Phi(y,z)$ is the vibration form (mode shape),

Φ_{max} is the vibration form (mode shape) value at the point with the maximum amplitude.

Since the acceleration is calculated at the highest occupied floor with the largest amplitude, the mode shape equals the maximum mode shape:

$$\frac{\Phi(y,z)}{\Phi_{max}} = 1$$

μ_{ref} is determined with annex F.5 (3) of NEN-EN 1991-1-4. According to F.5 (3) a good approximation of μ_{ref} is the mass per unit area of the structure at the point with the largest amplitude of the mode shape. If the weight of the structure is equally distributed over the height, the mass per unit area is obtained as following: divide the total dead load of the structure [kg] by the structure height [kg/m] and then divide it by the structure width [kg/m^2]. The strip that is created on the facade is the unit area on which the wind force acts at that point.
At the NEN 6702 section 10.5.3 it's stated that in order to calculate the accelerations of the building, it's allowed to use the dead load in combination with the instantaneous (momentane in Dutch) live load. If only the dead load is taken into account, no occupants will be present and therefore no one will experience any nuisance. The Eurocode deviates from this expression and states (see annex F.5 (3)) that only the load of the structure may be taken into account. This assumption increases the acceleration of the building.

The formula for the standard deviation of the acceleration can be rewritten as:

$$\sigma_{a,x}(y,z) = 1{,}3 * 1{,}25 * l_v(z_s) * v_m^2(z_s) * R * \frac{1{,}0 * 1{,}67}{\mu_{ref}} * 1 = 2.708 * l_v(z_s) * v_m^2(z_s) * R * \frac{1{,}0}{\mu_{ref}}$$

According to the NTA Hoogbouw (03-A Wind) report, $v_m(z_s)$ is determined with a lower basic wind velocity: 19.4m/s instead of 27m/s in area II. This is because the accelerations are calculated in the serviceability limit state and it has a return period of 1 year (the original value of 27m/s has a return period of 50 years). Because of this assumption, the acceleration is reduced with 61%.

When all the values are entered in the formula, a bending acceleration of $\hat{a}_{bending}$=0.050m/s^2 is obtained (this calculation can be found in Appendix A). Since the introduction of the Eurocode, it's also required to take the torsion acceleration into account. The torsion acceleration is calculated with the same formulas, but at the resonance response factor and the standard deviation the values for G_y, G_z, K_y and K_z have to be replaced with: G_y=3/8, G_z=3/8, K_y=3/2 and K_z=3/2. This results in a torsion acceleration of $\hat{a}_{torsion}$=0.062m/s^2. To calculate the total acceleration, the following formula has to be used (see NTA Hoogbouw (03-A Wind) report):

$$\hat{a}_{tot} = \sqrt{\hat{a}^2_{bending} + \hat{a}^2_{torsion}}$$

This results in a total acceleration of 0.080m/s^2. The allowable acceleration can be calculated with the first natural frequency (0.193Hz) and Figure 38: an acceleration of 0.17m/s^2 is allowed. This is larger than the actual acceleration and the building meets the requirements for the comfort. In practise accelerations above 0.15m/s^2 can be felt and the occupants will complain during a heavy storm.

Zonneveld also calculated the acceleration of the tower. This was done with NEN 6702 and the following formulas were used (section 10.5.3 and annex A.5):

$$a = \frac{1.6 * \varphi_2 * p_{w,1} * C_t * b_m}{\rho_1}$$

in which:
a is the acceleration,
φ_2 is a factor dependant on the eigenfrequency and damping of the building:

$$\varphi_2 = \sqrt{\frac{0.0344 * f_e^{-2/3}}{D * (1 + 0.12 * f_e * h) * (1 + 0.2 * f_e * b_m)}}$$

in which:
f_e is the eigenfrequency of the building:

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

in which:
a is the numerical value of the oscillation acceleration, depending on the static system and distribution of the mass: a=0.384m/s^2,
δ is the numerical value of the largest deformation of the structure as a result of the instantaneous load combination.

According to annex A.5 of NEN 6702, f_e may be multiplied by a factor of (1+20/h), creating a larger eigenfrequency and reducing the acceleration.

D is the adapted damping factor=0.01 for concrete buildings (if f_e<1Hz),
h is the height of the building,
b_m is the average width of the building.

$p_{w,1}$ is the variation in thrust on the building:

$$p_{w,1} = 100 * \mathrm{Ln}\left(\frac{h}{0.2}\right)$$

in which:
h is the building height,
0.2 is the roughness factor (area II, unbuilt).

C_t is the summation of the wind factors for thrust and suction=0.8-(-0.4)=1.2,
b_m is the average width of the building,
ρ_m is the mass of the building per metre building height.

When all the values are entered in the formulas, the following acceleration is obtained for a height of 200m:

$$a=0.081 m/s^2$$

Unlike the calculation of the Eurocode, the NEN 6702 only includes bending. The torsion acceleration is not considered (there is no method to determine the torsion frequency). The entire calculation can be found in appendix A.

The results of the NEN 6702 and NEN-EN 1991-1-4 are nearly identical, but in reality there are large differences between the calculations. The bending acceleration of the NEN-EN 1991-1-4 is 38% smaller than the bending acceleration of NEN 6702 (0.050 versus $0.081 m/s^2$).

This lower acceleration is created by several differences between the two calculation methods. For example, the NEN 6702 uses a damping value of D=0.01 (in SLS and for reinforced concrete buildings) and this is the relative damping in relation to the critical damping. This damping can be rewritten to the logarithmic decrement used by the Eurocode: d=2*π*D=2*π*0.01=0.063. But the Eurocode uses a value of d=0.1 (see section 7.3.3.4) for reinforced concrete buildings: the damping in the Eurocode has increased with almost 60% compared to the NEN 6702. The damping values should be considered as informative (other values may be used when they are properly substantiated), but during a preliminary design these values will likely be used and often the exact damping value can only be determined when the building is constructed.

The large difference between the codes is slightly diminished by the force coefficient and the weight of the structure. The Eurocode uses a lower mass (no instantaneous live load) and a higher force coefficient is applied (1.3 compared to the 1.2 of the NEN 6702), increasing the accelerations. Despite the differences, the end result of both calculations is nearly identical.

As shown before, the NEN 6702 only provides methods to calculate the bending eigenfrequency. Zonneveld ingenieurs used a FEM program(EsaPrimaWin) to determine the bending and torsion eigenfrequency. Now that the eigenfrequency of the torsion is known, the torsion acceleration can be calculated (same method as the bending acceleration). This resulted in the following values:

- $f_{e,ESA,bending}$=0.209Hz and this results in $a_{bending}=0.068 m/s^2$,
- $f_{e,ESA,torsion}$=0.286Hz and this results in $a_{torsion}=0.048 m/s^2$,
- $a_{tot}=\sqrt{(0.068^2+0.048^2)}=0.083 m/s^2$.

This FEM acceleration is comparable with the accelerations calculated with the NEN 6702 ($0.081 m/s^2$) and the Eurocode ($0.080 m/s^2$). Based on the Zalmhaven tower it may be concluded that the new and extensive formulas of the NEN-EN 1991-1-4 result in nearly the same values as the NEN 6702. If this conclusion still holds when the parameters change (different building) is unknown.

7.5 Wind interference

Dense city centres with tall buildings will locally influence the wind climate. In complex situations it's advised to use a wind tunnel study. The CUR recommendation 103 gives guidelines for this research. Also several studies by students have been done on the effects of wind around high rise buildings. Yoshihito Taniike researched interference mechanisms for enhanced wind forces on neighbouring tall buildings and Navin Narain researched the determination of wind loads and the effects of wind load on two towers.

7.5.1 Interference research of Yoshihito Taniike

Yoshihito Taniike [Taniike 1992] defines wind interference as following:

$$IF = \frac{Force\ on\ the\ building\ with\ the\ other\ building}{Force\ on\ the\ building\ without\ the\ other\ building}$$

Taniike researched the wind interference by placing a medium tower at a ground plate with x=0 and y=0 in a wind tunnel. Then he placed one of three towers (small, medium and large) at multiple locations. With this alignment, it was possible to visualise interference patterns. Out of the research project several conclusions could be made:

Fluctuating forces:

- A large building (same height but wider) upstream of the building results in larger fluctuating forces parallel (drag) to the wind direction. With increasing width, these forces will also increase. This is because of vortex shedding. Only when the building upstream is placed at a distance of y=7B and more, the interference will be equal to 1 (see Figure 39).
- On the other hand, a large building upstream will result in smaller fluctuating forces perpendicular (lift) to the wind direction.

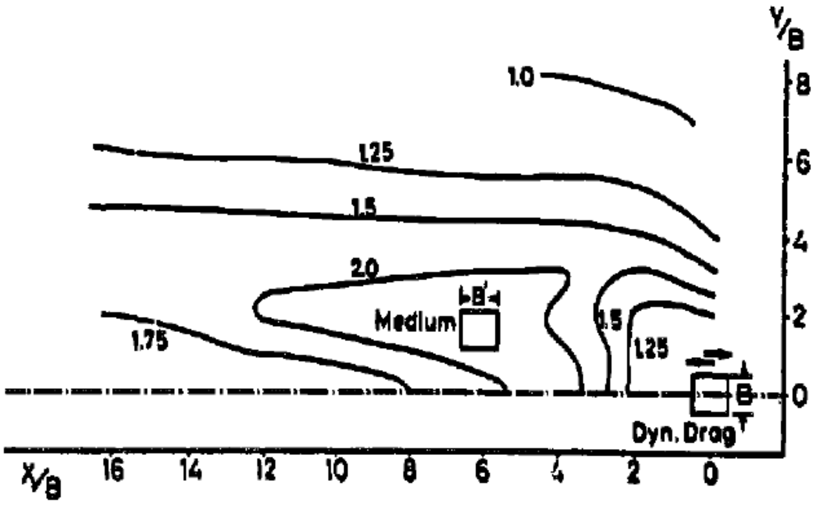


Figure 39 Interference factors for parallel fluctuating forces by a medium building [Taniike 1992]

Time average forces:

- When the time average forces of the three upstream buildings are calculated on the medium reference building, it can be concluded that the parallel (drag) forces are reduced. This is because of the reduced wind speeds. When the upstream building increases in seize, the reduction of the average forces also increases. Even negative factors are shown in Figure 40. Negative values mean that the forces work in the opposite direction of the wind.
- The reduction can also be seen by the time average forces perpendicular (lift) to the wind direction. Because of the coordinate definition, all the values are negative.

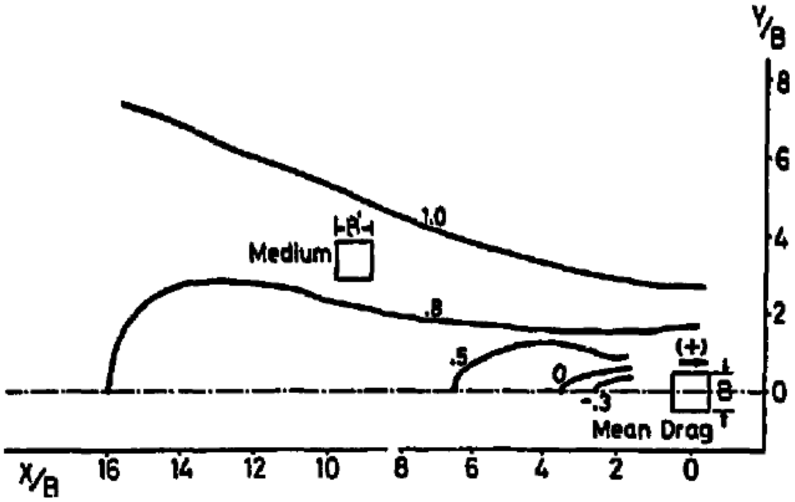


Figure 40 Interference factors for parallel time average forces by a medium building [Taniike 1992]

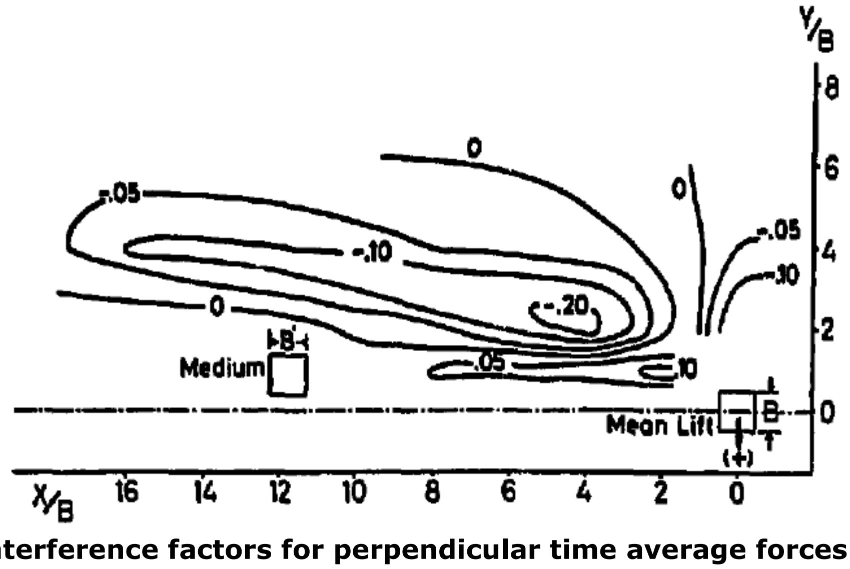


Figure 41 Interference factors for perpendicular time average forces by a medium building [Taniike 1992]

When the two buildings are placed next to each other (in the y direction) with a distance of 2B – 4B, a special situation occurs. The vortices of both buildings detach at the same time and as a results high wind speeds and a narrow flow section are created. Because of this an under pressure is formed and the two buildings are pulled to each other. This is visualised by Navin Narain in Figure 42. This figure also shows the coordinate system used by Taniike.

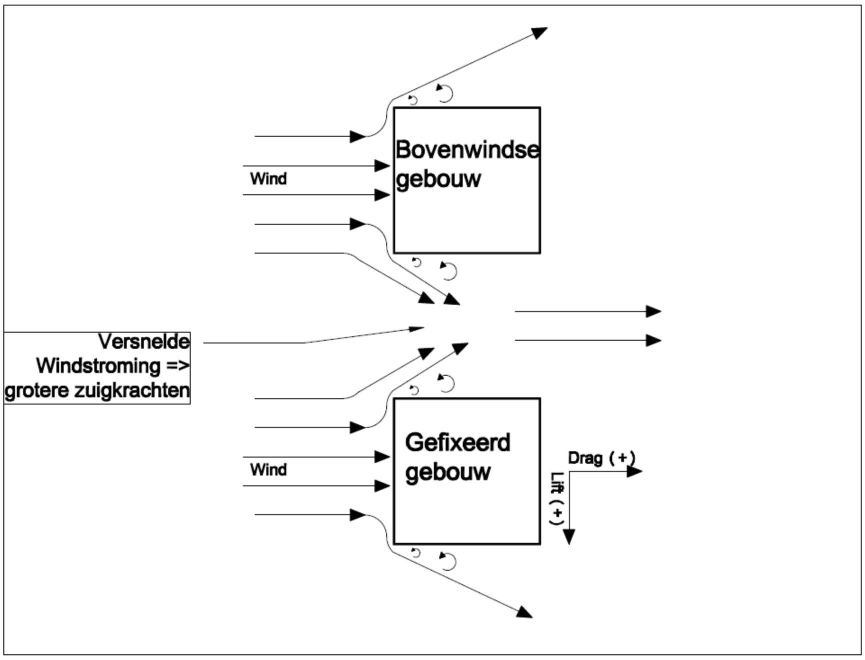


Figure 42 Under pressure because of the building configuration [Narain 2011]

7.5.2 Wind research of Navin Narain

Navin Narain [Narain 2011] also did a wind tunnel research. He used the ERASMUSPOORT project, consisting out of a 250m and a 300m tower. These two tower are constructed very close to each other in a special configuration (see Figure 43). Several conclusions can be made based on this research:

- Low and mid rise buildings have a positive influence on the high rise building. These buildings "protect" the high rise building and because of their presence, the wind speeds are reduced.
- The high rise building disturbs the upper wind layers and more wind will be directed towards the lower buildings. In most cases, the new high rise building has a negative influence on the mid and low rise buildings.
- Placing two tower very close to each other has mainly positive effects. When one of the buildings is placed behind the other in the main wind direction, the most benefits are obtained. At different configurations, problems may occur. This was the case at the ERASMUSPOORT project: at a wind angle of 285°, the torsion moment of the South tower increased with 11% because of the North tower (see Figure 44).

Narain concludes that it's very difficult to get a complete insight on all the parameters that have an effect on the wind interference: "it is hard to draw specific conclusions out of all the researches, because at every project the size, form, orientation and slenderness varies." Therefore Narain was not able to construct specific guidelines for wind interference.

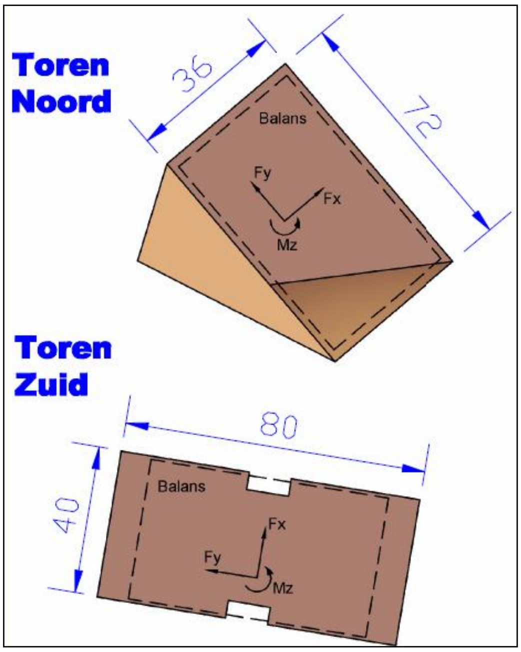


Figure 43 Dimensions of the ERASMUSPOORT towers [Narain 2011]

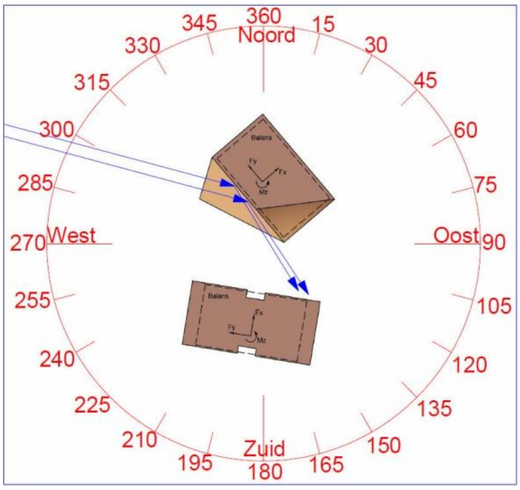


Figure 44 Effects of the North tower on the South tower [Narain 2011]

7.5.3 Recommendations for the Zalmhaven tower

Out of these two researches it can be concluded that wind interference is an important aspect. The low rise buildings could have a positive influence, but there is a small change that specific forces might increase. Furthermore, wind nuisance at ground level will probably occur because the 200m high tower will direct wind from the upper layers to the ground. Therefore, wind loads on surrounding buildings may increase. There are also three large towers near the Zalmhaven tower: Hoge Heren (103m high and 140m north of the tower) and Hoge Erasmus (93m high and 80m south-west of the tower) (see Figure 45). To calculate the actual forces a wind tunnel research is necessary.

Figure 45 High buildings near the Zalmhaven tower [Bing Maps 2011]

7.6 Vortex shedding

Besides wind interference, vortex shedding is also an important aspect in a high rise design. When wind passes a bluff object (a non streamlined body), alternating low pressure vortices are created downstream (see Figure 46). The object tends to move towards the low pressure zone, and because the vortices are alternating, the object starts to oscillate perpendicular to the wind direction. This vortex excitation is one of the aspects that distinguishes high rise from mid and low rise buildings.

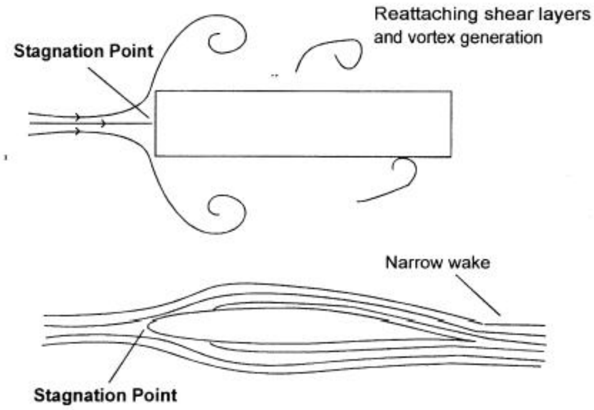


Figure 46 Vortex shedding at a bluff and streamlined object [Winter 2011]

The frequency at which vortices are shed from the building (the release frequency of the vortices) can be calculated with the following formula:

$$N = S * \frac{v}{b} \quad [Hz]$$

in which:

S is the Strouhal number,
v is the wind speed,
b is the building width.

The Strouhal number depends on the cross section shape and Reynolds number and ranges from 0.1 to 0.3. For a square cross section the Strouhal number is around 0.14 [Winter 2011].

When the frequency of the vortices matches the natural frequency of the building, resonance will occur. By rewriting the formula, the wind speed at which this resonance will take place can be calculated:

$$v = \frac{b * n_{natural}}{S} = \frac{30 * 0.1932}{0.14} = 41.4 \quad [m/s]$$

With the current design, resonance will occur at wind speeds of 41.4m/s=148.9km/h. This phenomena is depicted in Figure 47.

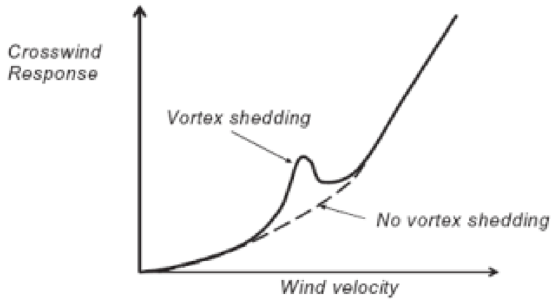


Figure 47 Perpendicular wind response [Winter 2011]

To prevent resonance from happening, the natural frequency can be enlarged or the cross section can be made more aerodynamic (see Figure 46). Increasing the stiffness can be done by enlarging the cross section of the stability structure, but this is rather expensive in high rise towers. Making small and clever aerodynamic adjustments has more influence and might even eliminate the resonance, i.e. buildings will always have vortex shedding, but resonance should be prevented.

Vortex shedding is a complex aspect and there are no simple design rules to calculate the amount of resonance. To really understand this phenomena, a wind tunnel research or/and computational fluid dynamics (CFD) analysis is recommended.

7.7 Snow load calculation

With the introduction of the Eurocode NEN-EN 1991, the calculation of snow load has also changed. In this chapter, the snow load for the Zalmhaven tower will be calculated. Because the tower has a tapered roof and a panorama view deck is located at the top with a load of 5kN/m^2, it is unlikely that the snow load will be governing.

According to the Eurocode NEN-EN 1991-1-3 [NEN-EN 2005], snow load on roofs shall be determined as following for permanent/temporary structures:

$$S=\mu_i*C_e*C_t*s_k$$

in which:

μ_i is the snow load shape coefficient. For a “lessenaarsdak” with an angle between 0° and 30°: μ_i=0.8. When a parapet (borstwering in Dutch) is placed around the roof, μ_i is given by the lowest value of:

$$\mu_i=2*h_1/s_k \text{ or } \mu_i=5$$

C_e is the exposure coefficient. For normal situations C_e=1.0 is recommended. For windy situations (very likely at 200m), the exposure coefficient may be reduced: C_e=0.8. Because it's unknown if a parapet is used, C_e=1.0 will be used.

C_t is the thermal coefficient. For non-glass roofs: C_t=1.0.

s_k is the characteristic value of snow load on the ground and can be calculated with the following equation (the Netherlands is located in Central West):

$$s_k=0.164*Z-0.082+A/336$$

in which:

Z is the zone number. For Rotterdam this is: Z=3

A is the height of the location above sea level: A=0.

This results in:

$$s_k=0.164*3-0.082=0.41kN/m^2$$

Unfortunately, the national annex specifies a different characteristic snow load (section 4.1, NEN-EN 1991-1-3):

$$s_k=0.7kN/m^2$$

The snow load on the roof is:

$$S=\mu_i*C_e*C_t*s_k=0.8*1.0*1.0*0.7=0.56kN/m^2$$

The value for the snow load is equal to the snow load from the Dutch code NEN 6702:

$$P_{rep}=C_i*p_{sn;rep}=0.8*0.7=0.56kN/m^2.$$

7.8 Conclusion

When NEN-EN 1991-1-4 is compared with NEN 6702, it can be concluded that generally the Eurocode gives lower values for the peak velocity pressure (see Table 10 and Table 11, the lower values can be found in area 2 and 3). This is remarkable, because the Eurocode gives higher values for the basic wind velocity. Also the reference height and the wind friction have changed, resulting in higher loads for high rise structures. The reason for the reduction is because the Eurocode has more possibilities to accurately calculate the peak velocity pressure. By applying a higher force coefficient (1.3 instead of 1.2), the difference is reduced between the codes. As a result of these values, the transition from NEN 6702 to NEN-EN 1991-1-4 for the Zalmhaven tower will not result in an increased structural area.

When the height of a building increases, the comfort of the occupants becomes decisive. The damping of concrete buildings is larger than that of steel variants and most concrete buildings fulfill the requirements. Because a 200m tower has never been constructed in the Netherlands before, the comfort levels of the building become a point of attention. With the Eurocode a new calculation becomes available that has more possibilities to accurately calculate the accelerations of the building. With the abundance of options (that make the calculation rather complex and difficult to understand) it's possible to achieve results that are comparable to the results obtained by a FEM analysis (ESA). If the results remain comparable when the project parameters change is unknown.

Wind interference and vortex shedding are two important aspects of high rise buildings. Unfortunately there are no design rules for these aspects because they depend on too many different variables. To get a clear insight, a wind tunnel research is recommended.

8 Foundation

A good geotechnical design is an important aspect of the overall building design. Large projects seldom win awards for the appearance, innovation or quality of their foundations. The foundation has to be build cheap, quick and it must work.
In section 8.1 the current design is elaborated. In section 8.2 and 8.3 the preliminary and alternative design are discussed. Section 8.4 ends with a conclusion.

8.1 Current design

Diaphragm walls are used for the foundation of the Zalmhaven tower. They are approximately 60m long and protrude trough the layer of Kedichem (a thick layer of clay). The first sand layer can't be used for the foundation because the bearing capacity is too low and it would result in large settlements for the surrounding buildings. The location of these diaphragm walls can be seen in Figure 48 (the tick dark lines in the lower section of the figure).

The stiffness of these walls was already calculated by MOS Grondmechanica in Rhoon and the values can be found in Table 12 (the 1.5x3.3 60m-NAP are applied). The load bearing capacity is shown in Table 13. More information about the foundation can be found in section 2.3.

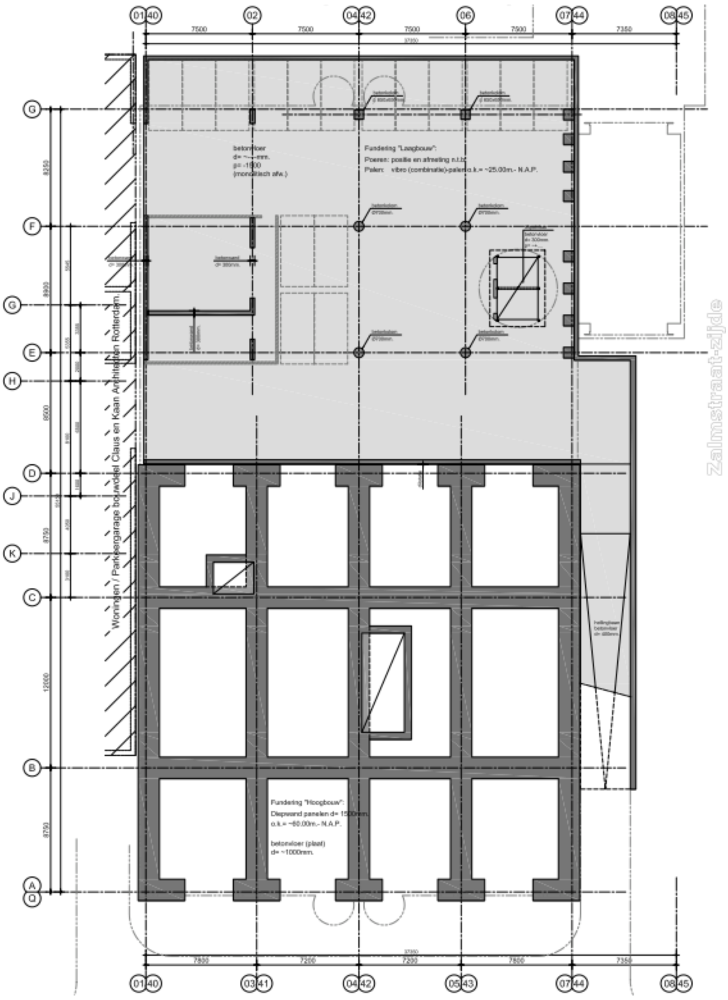


Figure 48 Location of the diaphragm walls

Table 12 Results of the settlements and stiffness

Panel level	Panel size	Point settlement	Elastic shortening	Top settlement	Stiffness
	[m^2]	[mm]	[mm]	[mm]	[kN/m]
50m-NAP	0.8x3.3	12	13	25	525650
	1.0x3.3	13	11	24	581165
	1.2x3.3	14	10	24	633555
	1.5x3.3	15	9	24	711230
60m-NAP	0.8x3.3	13	20	34	517355
	1.0x3.3	15	17	32	575555
	1.2x3.3	16	16	32	627225
	1.5x3.3	18	14	31	697190

Table 13 Results load bearing capacity

	Panel level = 50m-NAP		Panel level = 60m-NAP	
Panel size	$F_{r,max,Shaft,rep}$	$F_{r,net,d}$	$F_{r,max,Shaft,rep}$	$F_{r,net,d}$
[m^2]	[kN/m]	[kN]	[kN/m]	[kN]
0.8x3.3	4403	17080	6096	22635
1.0x3.3	4403	18445	6096	24270
1.2x3.3	4403	19810	6096	25905
1.5x3.3	4403	21860	6096	28360

The thinnest and shortest diaphragm wall has a shaft friction of 4403kN per meter wall circumference. This means F_{rep}=4403*(2*0.8+2*3.3)=36105kN of shaft friction per element. The total load bearing capacity of this wall is: $F_{tot,rep}$=42700kN. When this is divided by a reduction factor of 2.5 (see section 2.3), the total design load bearing capacity becomes: $F_{tot,d}$=17080kN. The thickest and longest diaphragm wall has a total design load bearing capacity of $F_{tot,d}$=21860kN.

The resistance is mainly based on friction. Therefore increasing the wall thickness has nearly no effect. For example 2 diaphragm elements with a dimension of 1.5x3.3 have a total load bearing capacity of 43720kN while 3 elements of 1.0x3.3 (same amount of concrete) have a resistance of 55335kN. Thinner elements will result in a reduction of concrete and reinforcement, but more elements have to be created. In total, it's estimated that using 1m thick diaphragm walls instead of 1.5m thick walls will result in a cost reduction of 25%[7].

8.2 Preliminary design

During preliminary design, it is often assumed that one-third of the total deflections is caused by the foundation. A simple way to incorporate the foundation into the design without any knowledge of the foundation is to limit the maximum deflections from w_{max}=h/500 to w_{max}=h/750. In a later stage, when a preliminary design is made for the foundation, the deflections because of the rotation can be calculated with the following formula (see also Figure 49) [Romeyn 2006]:

$$w_{foundation} = \frac{M * h}{r} = \frac{M * h}{\Sigma k * a_i^2}$$

in which:
M is the bending moment at the foundation [kNm],
h is the height of the structure [m],
k is the spring stiffness of the foundation piles and the ground [kN/m],

[7] This value is obtained in a consult with Robert Schippers from MOS Grondmechanica in Rhoon.

a_i is the lever arm [m].

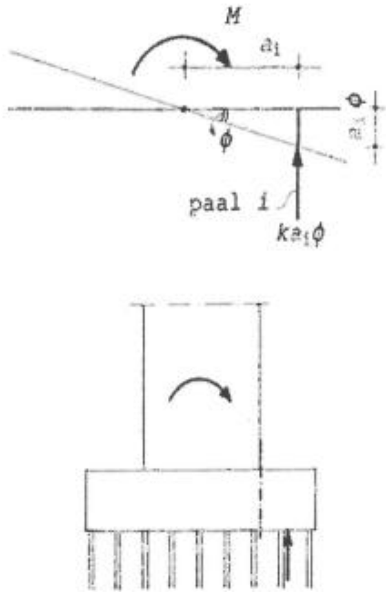


Figure 49 Deformation due to the foundation [Romeyn 2006]

The spring stiffness can be determined with:

$$\frac{1}{k} = \frac{1}{k_p} + \frac{1}{k_g} \rightarrow k = \frac{1}{\frac{1}{k_p} + \frac{1}{k_g}}$$

in which:
k_p is the spring stiffness of the pile [kN/m]:

$$k_p = \frac{EA}{l}$$

in which:
E is the Young's Modulus of the concrete,
A is the area of the pile,
L is the length of the pile.

k_g is the spring stiffness of the soil underneath the pile head [kN/m]

$$k_g = 90 * b * q_p$$

in which:
b is the width of the pile,
q_p is the load bearing capacity at the pile point.

When round foundation piles are used, the formula for the spring stiffness of the pile becomes:

$$k_g = 80 * D * q_p$$

in which:
D is the diameter of the pile,
q_p is the load bearing capacity at the pile point.

If k_g (load bearing capacity at the pile point) is unknown, $1/k_p + 1/k_g$ may be replaced by $1/(0.5*k_p)$ to give a first approximation.

The formulas described above are based on a pile and raft foundation. Figure 49 assumes that there is no deformation in the raft and therefore $r=\Sigma k*a_i^2$. In reality this is not the case. By applying a thick raft (the Zalmhaven tower uses a massive slab of 1m thick), the deformations are reduced. This thick slab also assures that the outer piles, which are the most effective, bear the largest load.

A pile and raft foundation has already been used in a number of high rise projects in the Netherlands. For example the Erasmus MC tower, were they used a 2m thick raft with 333 prefab concrete piles [Henkens 2010]. Abroad this foundation method is also popular for high rise structures, because eccentric loads are spread over the piles and different settlements are taken up by the raft and not by the superstructure. At the JuBi towers in The Hague they used a different system: a beam grid with prefab concrete piles. This system reduces the amount of required concrete, but the stiffness is also reduced. Another disadvantage is the execution of the formwork. Constructing a beam grid is far more complicated than constructing a solid foundation slab.

NEN-EN 1997-1 and NEN-EN 1997-2 give guidelines for the geotechnical design. At the moment this Eurocode section is not yet obligatory and the NEN 6740 may be used.

8.3 Alternative design

Are the diaphragm walls used in the current design the best solution for the foundation of the Zalmhaven tower? Soil removing foundations are time consuming to construct and soil relaxation takes place. A foundation with prefabricated concrete piles would be much faster, cheaper and because of the large amount of piles, the subsoil will be compacted. Unfortunately the weight of the tower is too high and the piles need to reach to the second Pleistocene sand layer at 55m-NAP. Prefabricated concrete piles are made in lengths up to 40m and the only possibility to apply these piles is to create a basement until 15m-NAP. The current design has no basement and therefore an additional 15m has to be excavated in order to apply prefab concrete piles. Besides the additional excavation, the tower is located in an urban and densely populated area, were vibrations, noise nuisance and settlements are unwanted.

A second alternative is to use bored piles. A diameter up to 2.5m is possible and the execution is free of vibrations. When one takes a closer look at the construction method, there are only small differences between bored piles and diaphragm walls. In order to prevent a very thick foundation slab, it is advised to place the bored piles beneath the core walls. To increase the stiffness, bored piles can also be placed around the perimeter of the building. With a diameter of 1.5m the same result as the diaphragm walls is obtained. Because large section can be excavated at once, the diaphragm method is preferred.

A third alternative is to use Tubex piles. A tubex pile consists out of a steel pile with a sharp steel point. The pile is screwed into the ground until the final depth is reached. Then the reinforcement is placed in the tube and the pile is finished by pouring concrete in it. With this method no soil is removed and no vibrations are produced. Technically, the length of the pile is unlimited, because steel sections can be welded on the pile. Normally Tubex piles with a length of around 30m are used, but length up to 60m have already been used in the Netherlands. The load bearing capacity of a Tubex pile goes up to 5000kN [Maes 2001] and this is considerably less than the diaphragm walls used in the current design (17080kN). Approximately 3.4 Tubex piles are needed for the thinnest diaphragm wall. Compared to the thickest and longest diaphragm wall, 5.7 Tubex piles are required.
The loss of load bearing capacity is compensated by the relative low price of a Tubex pile: €15000 for a 50m pile. For a diaphragm wall, a unit price of €400/m^2 can be used. This results in a price of 400*((2*0.8+2*3.3)*50)=€164000 per wall.

It can be concluded that the thinnest and shortest diaphragm wall has 3.4 times more load capacity, but is 11 times more expensive. Out of this conclusion it might seem that Tubex piles are the best solution, but a Tubex pile has several other disadvantages besides the lower load bearing capacity:

- It takes approximately one day to place a Tubex pile and this is equal to the construction time of a diaphragm wall. Because there are at least 3.4 Tubex piles per diaphragm wall required, the construction time of the foundation will increase when Tubex piles are used.
- To reach a load capacity of 5000kN, a Tubex pile with a diameter of 762mm will be used. When a minimal distance of 2.5D is applied between the piles, the centre to centre distance becomes 1.9m. Per diaphragm wall of 3.3m 1.5 Tubex piles can be placed. This means that there is not enough space to place all the Tubex piles underneath the walls of the structure. Piles have to be placed between the walls and the thickness of foundation raft has to increase.
- With the current design, diaphragm walls are placed around the circumference of the structure. With Tubex piles less piles can be placed around the circumference and this reduces the stiffness of the foundation. Zonneveld already made a calculation for a 220m high Zalmhaven tower and it was calculated that a foundation with Tubex piles was not stiff enough.

8.4 Conclusion

The current design with diaphragm walls was chosen because of the high stiffness, the depth of the load bearing sand layer and because of the preference of the contractor. In the previous section the question was raised if this is the best solution? Prefab piles are not a better solution because of the maximum length and vibrations during placement. Bored piles are almost equal to the diaphragm walls and there is no preference. Tubex piles are a good and cheap third solution, but the foundation stiffness will be reduced and the construction time increased. A stiffness calculation of the foundation has to show if it’s possible to use Tubex piles instead of diaphragm walls.

9 Stability systems in general

The demand for bigger and taller buildings has resulted in different structural systems. Engineers try to design the optimum system, where all aspects work together as an integrated whole. The design process is not limited or steered by rules and every building is an unique project.
When high rise buildings are compared to regular buildings, the effect of lateral loads stands out. Because of their height, the loads have to be transferred over a longer distance. The building behaves like a horizontal clamped beam and the stability and stiffness becomes dominant. If the stability and stiffness is not satisfied at a regular building, the fastest solution is to enlarge the cross-section of the structural elements. For high rise buildings this is possible, but a more economical solution is to change the structural system. This chapter will elaborate on what the term high rise means (section 9.1) and on the possibilities of different structural systems (section 9.2).

9.1 Definition of a high rise buildings

What is a high rise building? Literature can't give a deceive answer, because high rise is a subjective aspect. In [CTBUH 1995] the concept of tall buildings (similar to high rise buildings) is defined as:

"*A tall building is not defined by its height or number of stories. The important criterion is whether or not the design is influenced by some aspects of "tallness". It is a building in which tallness strongly influences planning, design, construction and use. It is a building whose height crates conditions different from those that exist in "common" buildings of a certain region and period."*

A more structural definition can be found in [Stafford Smith 1991]:

"*From the structural engineer's point of view, however, a tall building may be defined as one that, because of its height, is affected by lateral force due to wind or earthquake actions to an extent that they play an important role in the structural design. The influence of these actions must therefore be considered from the very beginning of the design process."*

According to both definitions, the Zalmhaven tower can be considered as a tall building.

9.2 Stability systems

If horizontal loads didn't exist, a building with double the height would have approximately the same material demand as two normal buildings. Because of the wind force this is not applicable and the material demand increases exponentially with the height.

An example is made for an concrete building, where the height is doubled [Hoenderkamp 2007]:

Vertical loads:

- Floors
  The dimensions of the floor are not a function of the building height. The dead load increases linear with the height and the influence factor is $\mathbf{2^0=1}$.
- Columns
  The dimensions of a column are a function of the height. Doubling the number of floors, will result in double the amount of vertical load: the influence factor is $\mathbf{2^1=2}$.

Horizontal loads:

- Shear
  The shear load increases linear with the height of the building. The influence factor for the shear load is $\mathbf{2^1=2}$.
- Bending moment
  The bending moment increases with the building height to the power of two. This results in an influence factor of $\mathbf{2^2=4}$.
- Sway index
  The sway index of a building is the maximum horizontal displacement at the top divided by the building height. The sway index increases with the building height to the power of three. The influence factor for the sway index is $\mathbf{2^3=8}$.
- Dynamic behaviour
  The dynamic actions increase rapidly with the building height. The influence factor for the dynamic behaviour is $\mathbf{2^4=16}$.

To cope with the increasing actions of lateral load, an optimal stability structure should be used in the design. There are several possibilities to choose from for concrete structures: rigid frames, shear walls, cores, tube structures, tube-in-tube structures, bundled tubes and combinations of these systems. When designing a steel structure, the engineer could also use outriggers or a mega-braced structure. Figure 50 and Figure 51 give an indication for the maximum number of stories that could be achieved with the stability system.

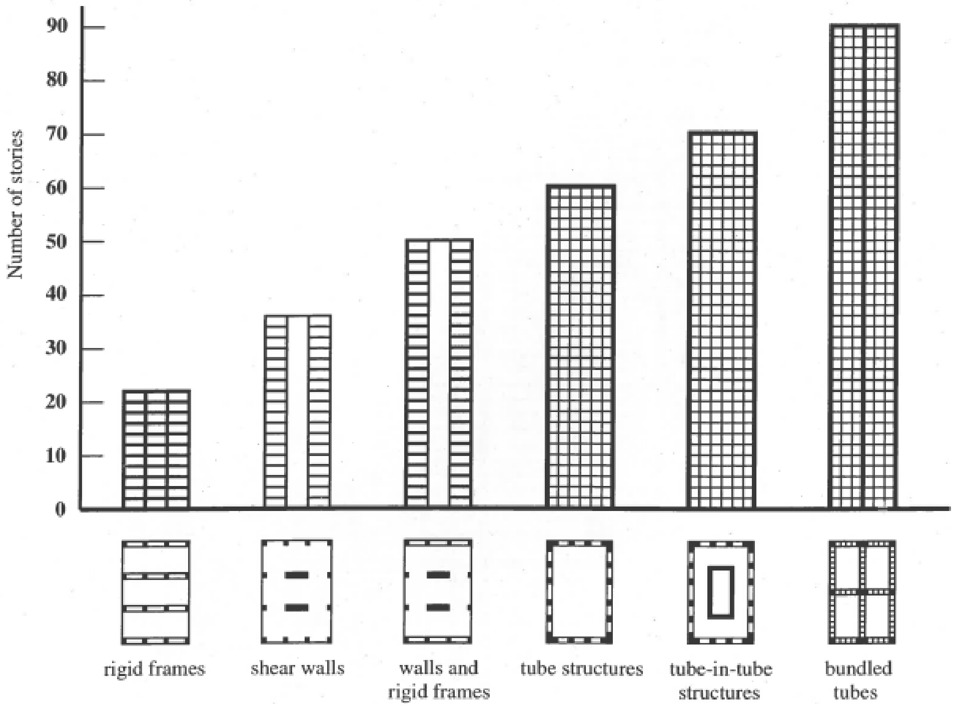


Figure 50 Stability systems in concrete [Hoenderkamp 2007]

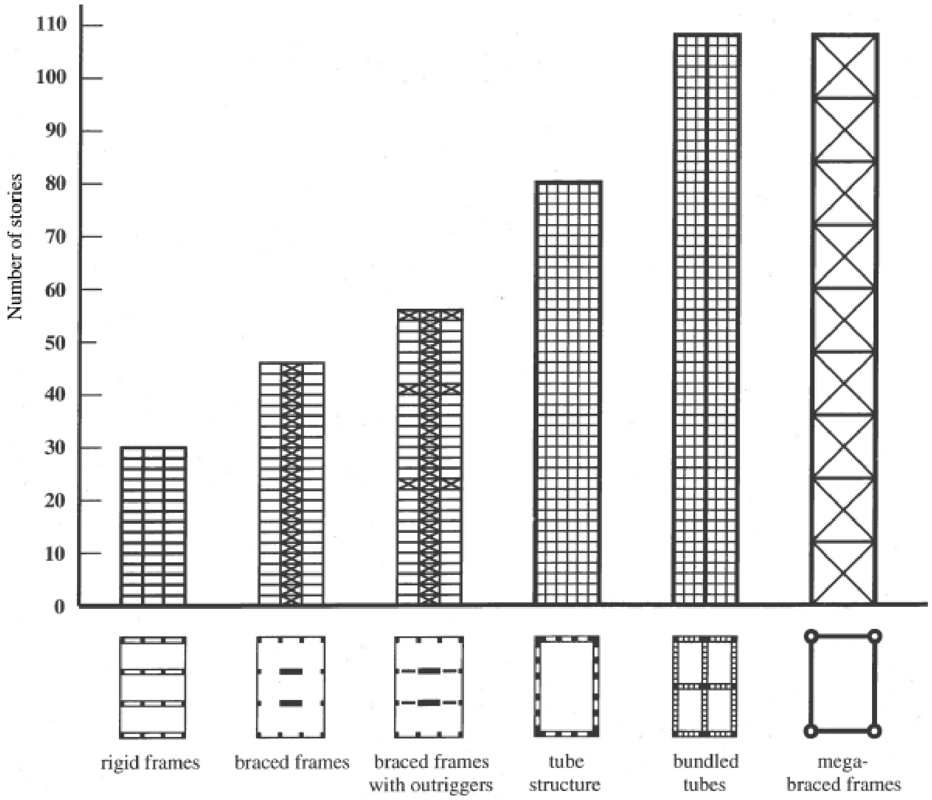


Figure 51 Stability systems in steel [Hoenderkamp 2007]

10 Stability systems for prefab structures

Simple and fast connections are essential for a prefabricated construction method. Hinges are normally applied and rigid frames are rarely used. Because of these connections the amount of options for the stability system are reduced to shear walls, cores, tube structures, tube-in-tube structures, bundled tubes and combinations of these systems. The engineer is left with plenty of options and designing a high rise prefabricated building should be achievable. This chapter will focus on the elements of the structure in section 10.1, the connections in 10.2 and the structural behaviour in section 10.3. Section 10.4 will continue with the response to lateral load and section 10.5 describes the element configuration. This chapter ends with a conclusion in section 10.6

10.1 Prefabricated elements

Prefabricated structures can be constructed out of walls, floors and columns. Walls and floors are more interesting than columns, because they also contribute to the stability system. In this section the elements will be shortly discussed.

10.1.1 Walls

Walls are essential elements in a structure. They are used for vertical load bearing, lateral stability, dividing functions and areas, fire protection and sound isolation. Prefabricated walls differ from cast in situ walls because of their connections. Because of these connections the structure is no longer monolithic and the wall response on loads changes. Without a proper connection, the sections will behave like separated walls and the stiffness will reduce with a factor of 4 ($2^3/2=8/2=4$). This is illustrated in Figure 52

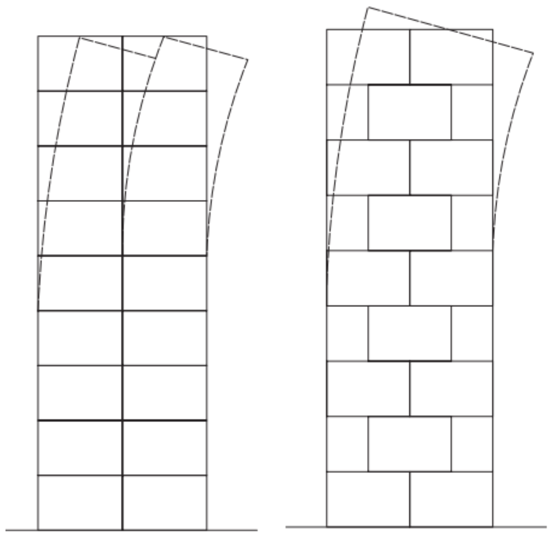


Figure 52 Connection action

To approach a monolithic behaviour, the connections should be able to transfer shear and normal forces. The connection type is responsible for the behaviour of the structure. Figure 53 shows the in-plane response of a prefabricated wall. In a) the shear forces due to wind load are shown, and b) shows the tension and compression forces due to wind load.

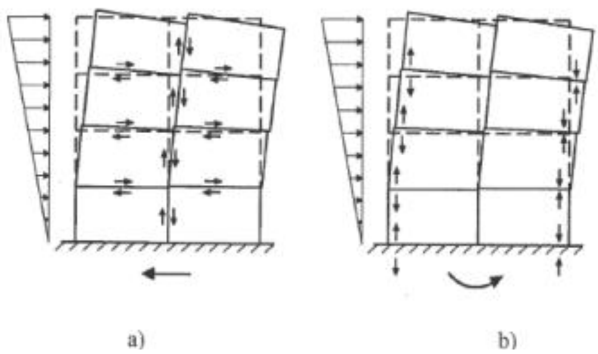


Figure 53 In-plane response of a prefabricated wall [Tolsma 2010]

For internal and core walls commonly plain concrete elements are used. If a facade tube is constructed it's also possible to use plain elements, but there is another possibility: sandwich elements. By using sandwich elements, different phases of the construction process are combined and this could lead to a building time reduction.

10.1.2 Floors

The floors transfer the vertical load to the load bearing structure by bending. They also transfer the wind load from the facade to the stability system by diaphragm action. To achieve diaphragm action, the floor must work as a stiff whole. This is no problem for cast in situ floors, but when prefabricated elements are used, connections become necessary. There are several possibilities to connect the elements:

Connection 1: Concrete filled joints
This is the most simple and fasted solution to transfer shear forces between the floor elements. The relatively small connection area results in a small shear force that can be transferred. Therefore, these connections are only made in low rise buildings or in combination with other connection methods. It is also possible to apply reinforcement in the joint, which increases the capacity. This connection is depicted in Figure 54.

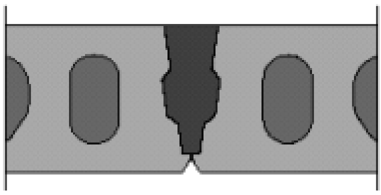


Figure 54 Joints filled with concrete [Betonson 2012]

Connection 2: Reinforced pressure layer
A reinforced pressure layer on top of a prefabricated floor can be seen as a thin cast in situ floor with lost formwork. This monolithic floor behaves as a stiff diaphragm and large shear forces can be transferred. When a point load is near the joint between two elements, the pressure layer also transfers a part of the load to the next element. The joints between the elements described in the previous connection are also filled.

Connection 3: Tension ties
When it's undesirable to apply a pressure layer, tension ties can be applied. Tension ties are normally placed around the floor (between the floor elements and the facade), but in several cases problems might arise with the available area. In Figure 55 the traditional location of a tension tie is shown. Figure 56 shows an internal tension tie, because there was no room for the tension ties at the edge.

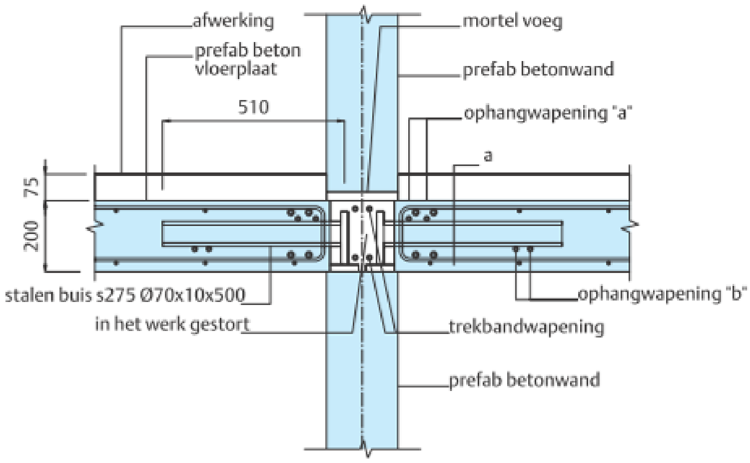


Figure 55 Floor support Waterstadtoren [Alphen 2005]

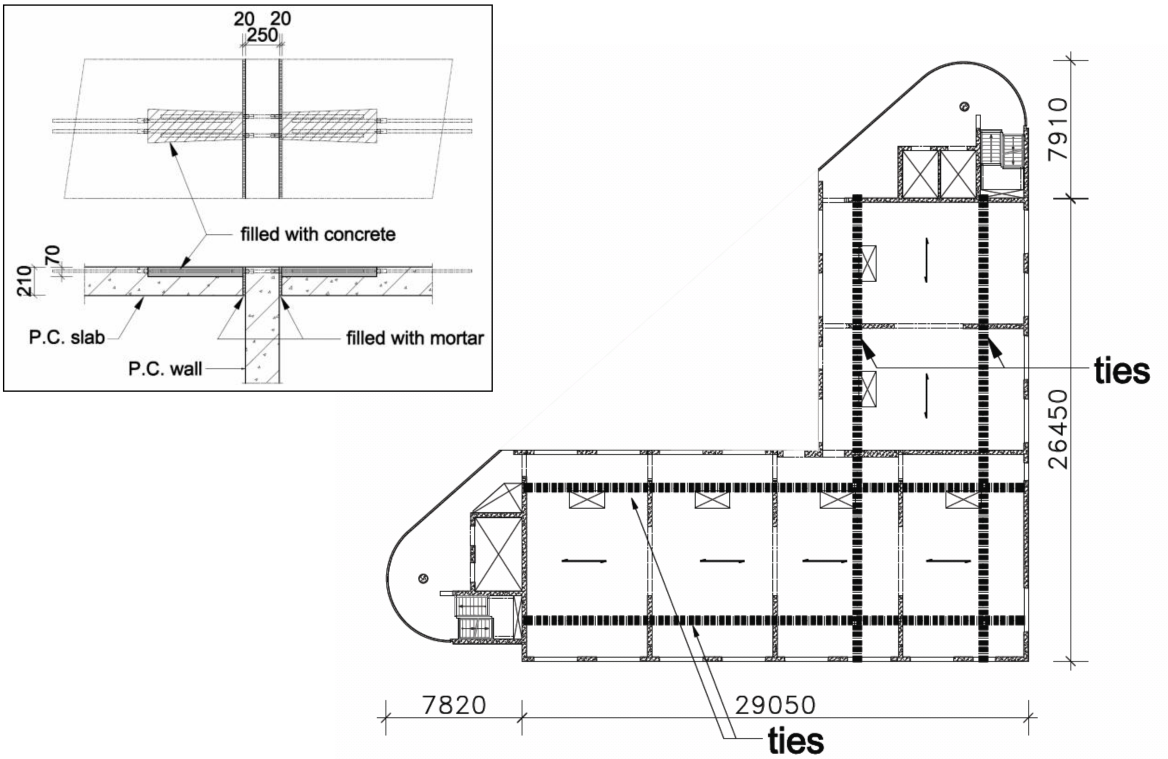


Figure 56 Tension ties layout [Corsmit PowerPoint]

Connection 4: Welded plates
The last solution is to use casted in steel plates. By welding two plates together, the separate floor elements are connected. This solution is used very seldom because the stiffness of this connection is relatively low and it's expensive.

For the floor, several different floor types can be used. Two groups can be distinguished: fully prefabricated floors and partially prefabricated floors. Cast in situ floors are not taken into account in this thesis, because the benefits of prefab are lost when cast in situ floors are used.

Hollow core slabs are an example of the fully prefabricated floors. These slabs are frequently used at office buildings because of their low dead load. For residential towers this system is less beneficial since the low dead load results in a low noise reduction. By using massive floor slabs of approximately 320mm thick this problem is overcome. By pre-installing all the ducts inside the massive floor slabs, the level of prefabrication is increased. Despite the fact that all the ducts are pre-installed, it's still possible to achieve a high level of flexibility. Special ring systems provide connections throughout the entire floor area and the toilet or kitchen can be placed in any corner of the room. It's even possible to place the mechanical ventilation in the floors.

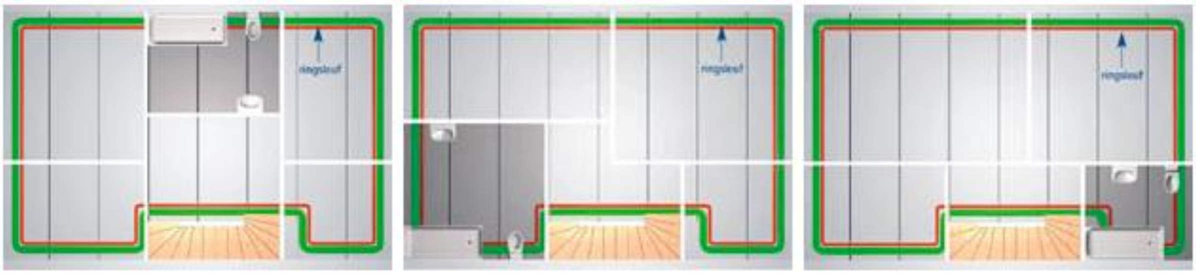


Figure 57 Duct floor from VBI [VBI 2012]

Composite plank and bubble deck floors belong to the partially prefabricated floors. The prefabricated part of the floor behaves as a lost formwork and the amount of weight that has to be transported by the crane is reduced. The required concrete can be poured with a concrete pump. Before the concrete is poured all the ducts and reinforcement have to be placed. Compared to fully prefabricated floor, time is lost during the execution.

10.1.3 Columns

In most structures prefab columns are used as vertical bearing elements. In some low rise projects the columns also provide the stability. This is done by making a moment resisting connection at the foundation and the connections with the floor beams are hinged. Moment resisting connections in high rise buildings are prevented as much as possible because they are expensive and labour intensive.

10.2 Possible connections

Already stated in the previous section, the vertical connections between the elements are responsible for the behaviour of the structure. With increasing height, the normal and shear forces will become larger (linear relation). Despite the increasing forces, the connections should still be very easy and fast to make on site. Beside this vertical connection between two elements, there are more locations were connections have to be made.

Four different locations can be highlighted with different types of connections:

- horizontal connections between two parallel wall elements,
- vertical connections between two parallel wall elements,
- vertical connections between two perpendicular wall elements (corner connections),
- connections between horizontal en vertical elements (floor connection).

For all the four locations several possible connections can be used. Figure 58 explains the division of these connections [CT4281 2005].

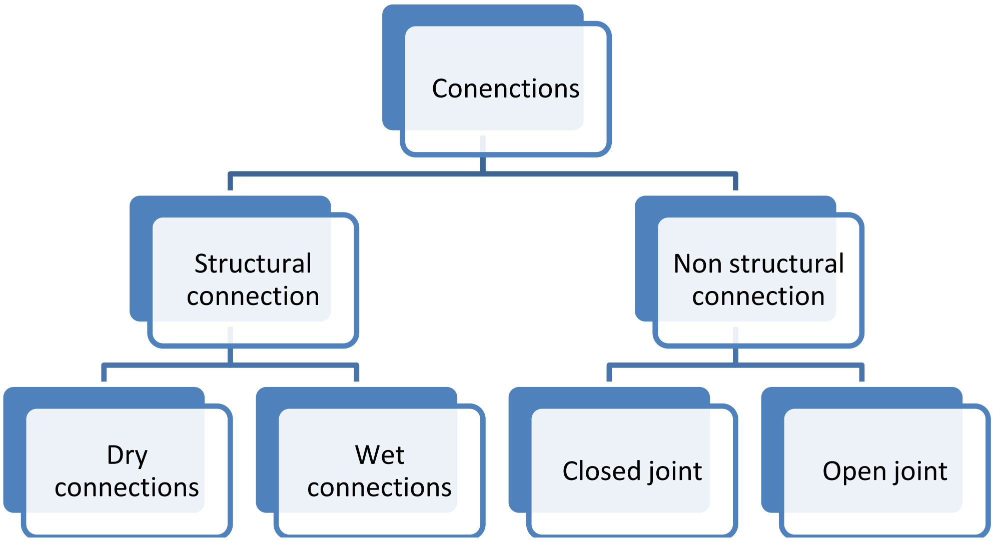


Figure 58 Division of the connections

A dry structural connection is a connection were no mortar is used. As a result, the connection doesn't have to dry before it can be loaded. Examples of dry connections are free supported connections (floor supports), welded connections, cold connections[8] and glued connections.
A wet structural connection is made with fluid mortar. When the mortar has dried, a relative stiff connection is obtained.

A non structural connection does not transfer any forces and can be left open (open joint). Because of requirements (water and air tightness, sound isolation, appearance and fire resistance) the joints are often closed. This can be done with rubber strips, elastic profiles or a joint kit (voegenkit in Dutch).

Besides the structural or non structural properties, joints are also important for the tolerances and movement of the elements. Prefabricated structures are composed out of prefabricated elements. When this element doesn't fit at the building site, delays are bound to happen. Elements can be made with a high dimensional tolerance, but it is easier (and cheaper) to incorporate tolerances in the elements and joints. Aside from tolerances, the elements will also move relative to each other under the influence of external loading and temperature differences. The joint or connection should be designed for this movement in order to prevent damage.

In this section several solutions will be reviewed for the four different connection locations. The two words "joint" and "connection" are a bit overlapping and therefore a word definition is used: a connection is the total physical link including the adjoining parts of the precast concrete elements and a joint is the space (area) between the two elements were they meet each other.

10.2.1 Horizontal connections between two wall elements

To connect two horizontal elements, commonly a structural wet connection is used. It's also possible to use a dry connection (for example a tooth connection), but several problems arise (reduced stiffness, difficult to execute, possible element splitting and a high level of accuracy is required). A well proven connection is the grouted starter bar. With this connection, reinforcement bars are protruding out of the bottom element and fit

[8] Two concrete elements are placed on top of each other without any intermediate material. Quay walls in hydraulic structures are often constructed this way.

in the corrugated sleeves of the top element. When the top element is placed and levelled, the joint and the sleeves are filled with mortar.
The placing of an element on starter bars is shown in Figure 59. The amount of starter bars in this figure is enormous and generally much less bars are used.

Figure 59 Placing of an element on starter bars [Bennenk]

There are several possibilities to fill the joint. For example:

- placing in a half plastic mortar bed (Figure 60 A),
- dry packing (Figure 60 B),
- overflow pouring with fluid mortar (Figure 60 C),
- pressure grouting (Figure 60 D).

With the first two options, the sleeves of the starter bars (gains) have to be filled afterwards with fluid mortar.

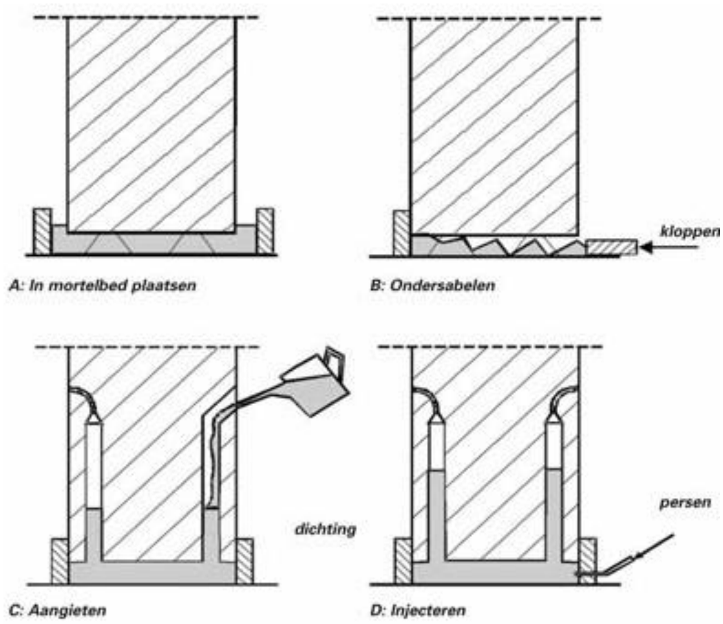


Figure 60 Execution method for joints [Bennenk]

A fast and widely applied fifth option is pump grouting (onderpompen in Dutch). Because of new innovations like a higher strength mixtures, faster strength development and better thixotropic properties, pump grouting has gained a higher quality level. Thixotropic mixtures become very fluid when energy is added to the mixture (for example during pouring of the mixture). When the energy is removed, the viscosity increases rapidly and the mixture becomes thicker. Another advantage is that the mortar can be placed very carefully in the joint and the mortar is thick enough to stay in the joint.
To apply this technique at an intermediate wall, one side of the wall is closed with a wooden strip and air can escape at this side. On the other side, the mortar is placed by pump grouting. It's possible to inspect the degree of filling from this side. At the end walls, the wooden strip is replaced by an elastic, relatively air permeable strip. This strip has the same size as the joint and it's placed between the elements (see Figure 61). This technique was applied at Het Strijkijzer and they achieved a minimal degree of filling of 98% [Huijben 2006]. It is quite difficult to achieve this level of filling with other techniques because of enclosed air bubbles.

Because of the thickness of the thixotropic mixture, the gains have to be filled with fluid mortar afterwards.

Figure 61 Elastic strip at a joint

At the Stadskantoor in The Hague, they applied a wet tooth connection with a masonry element configuration (see Figure 62). Because of the alternating windows (see Figure 63) there was no direct load path from top to bottom. To redirect the forces, a wet tooth connection was used. Because of this connection, the dimensional deviations had to be very low.

Figure 62 Wet horizontal tooth connection [Hurks Delphi Engineering 2012]

Figure 63 Alternating windows at the Stadskantoor in The Hague [Hurks Delphi Engineering 2012]

A third possibility to connect the elements is to use a welded connection. This dry connection uses steel plates that are casted into the concrete. Because this connection is located in a difficult place (near or behind the floor) and the stiffness is rather low, it's used very seldom in practice. The high price makes it even more undesirable.

10.2.2 Vertical connections between two wall elements

The vertical connections differ from the horizontal connections due to accessibility and the location. As a result, vertical connections have less to no normal forces and large shear forces occur. There are several possibilities to connect the elements:

- wet connections,
- dry connections,
- no connection.

Wet connections
Wet connections are widely used and they can transfer large shear forces when the mortar or concrete has hardened. During the construction and hardening time, these connections can't fulfil their structural role. Several different variants are used:

- Unreinforced smooth connection
  The joint between the two elements is filled with mortar or concrete. Because the interface is smooth and there is no reinforcement, this connection can only transfer compression.
- Reinforced smooth connection (see Figure 64)
  The reinforced smooth connection is comparable to the horizontal connection and compression and shear forces can be transferred. In some cases the connection can also be loaded with tension. It should be noted that the shear force depends on the normal stress. A different mechanism with friction and dowel action takes over.
- Reinforced tooth connection (see Figure 65)
  The reinforced tooth connection is similar to the reinforced smooth connection, but with some extra's. Because of the tooth's, the mortar is confined and pressure diagonals arise. These pressure diagonals in combination with a tension force in the reinforcement result in an increased shear resistance.
- Loop connection (see Figure 66)
  The loop connection is a variant of the reinforced smooth connection. Because of the loop recess (lussparing in Dutch), the production and execution is less difficult. Pressure diagonals are present between the recesses and the stiffness is higher than the reinforced smooth connection.

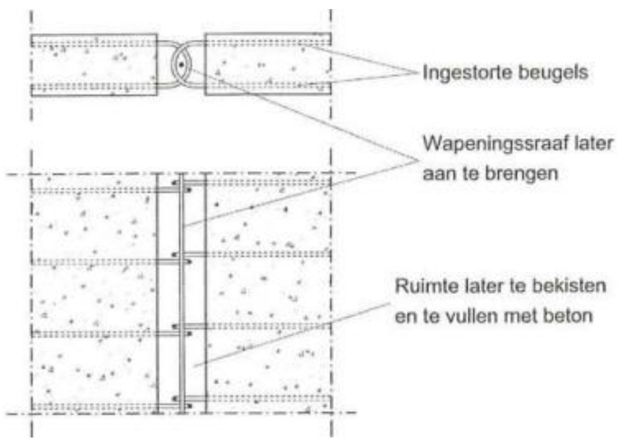


Figure 64 Reinforced smooth connection [Falger 2004]

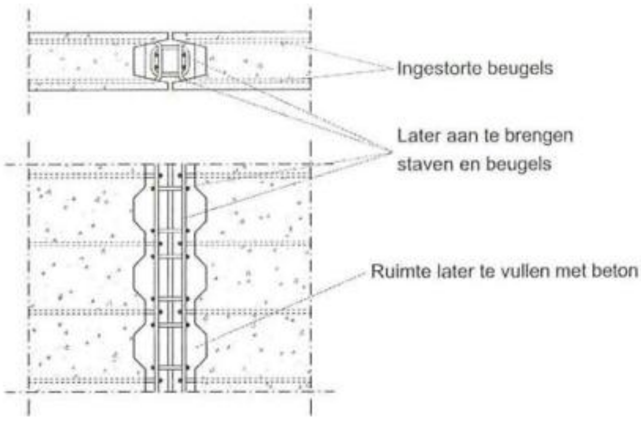


Figure 65 Reinforced tooth connection [Falger 2004]

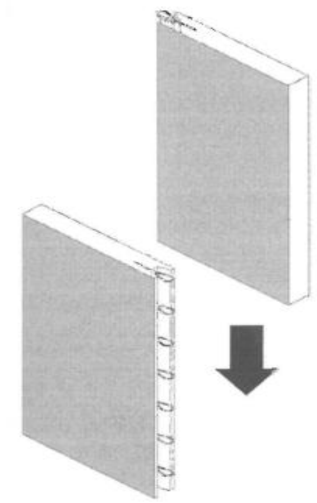


Figure 66 Loop connection [CT4281 2005]

Dry connections
A disadvantage of the wet connections is that the connection has to dry before it can transfer forces. Dry connections do not use mortar or concrete and they can immediately transfer forces. A disadvantage of dry connections is that the shear capacity is often lower than the capacity of wet connections. The two most applied dry connections are welded connections:

- Welded steel plates that are casted in the concrete
  In every element two or more plates are casted in the concrete. The plate is anchored in the concrete by reinforcement bars. After the elements are levelled, the plates are welded together on one side. Because the forces are concentrated around the steel plates, large deformations occur [Falger 2004].
- Welded steel profiles that are casted in the concrete
  With the casted in profiles, the elements have to be welded on both sides. In general these connections are stiffer than the casted plates [Falger 2004].

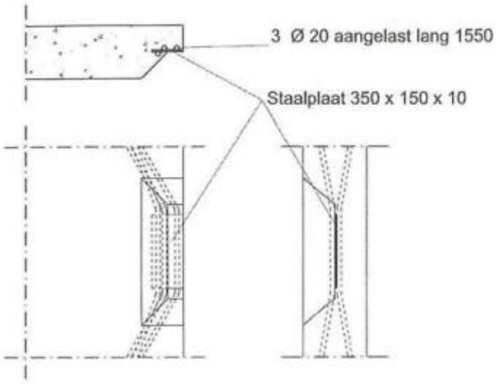


Figure 67 Welded steel plates that are casted in the concrete [Falger 2004]

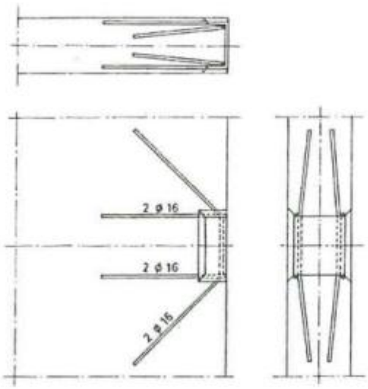


Figure 68 Welded steel profiles that are casted in the concrete [Falger 2004]

Just as the horizontal connection, the vertical welded connections are also rarely applied. The high price and the low stiffness are the main factors for the reduced interest.

There are other dry connect possible, for example a dry tooth connection (see Figure 69). This connection is also rarely used because of the small tolerances that are required. A variant of this connection is used as corner connection in section 10.2.3.

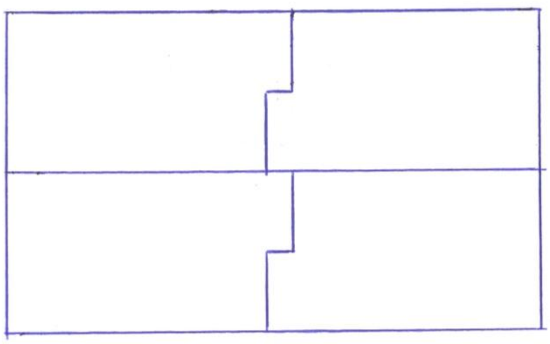


Figure 69 Dry tooth connection

No connection
When there is no structural connection between the elements, no force can be transmitted. Often a joint of 20 to 30mm is applied between the elements for dimensional tolerances. In buildings this joint often is closed with a non structural material because of building physics and fire safety requirements. Since the vertical joints don't fulfil a structural role, a different mechanism has to be used. Placing the elements in a masonry configuration is a technique first used in the Prinsenhof office building in The Hague. With

this configuration, there is a whole element above and beneath the joint. Shear forces that should be transferred by the connection are now taken up by the whole elements (dowel elements). This configuration is shown in Figure 70.

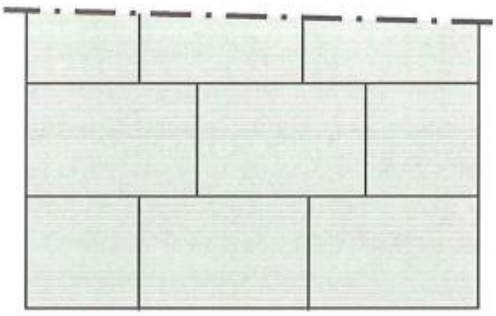


Figure 70 Masonry configuration [Falger 2004]

Depending on the configuration of the wall, up to 40% of the joints don't have to be connected (a 40% reduction is obtained when the width/height ratio of the elements is equal to 1). This results in an enormous reduction of time and costs. Combine this with the fact that this layout has approximately the same stiffness as the stiffest wet connection (the tooth connection) and the masonry configuration becomes even more attractive.

10.2.3 Vertical connections between two perpendicular wall elements

Corner connections are important to make sure that the stability system in the x and y direction works together. With this connection the flanges are activated and the stiffness increases. The variants for vertical wet and dry connections between two wall elements also apply for the corner connection. Beside these variants, there are three more possibilities:

- Interlocking Halfway Connection (IHC),
- Interlocking above Ceiling Connection (IACC),
- Staggered Connection (SC).

In Figure 71 the three corner connections are visualised.

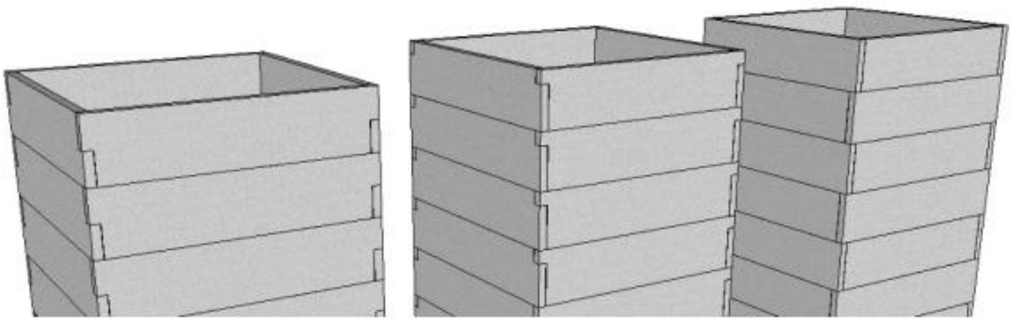


Figure 71 Dry corner connections: IHC, IACC and SC [Tolsma 2010]

These corner connections can also be transformed to intermediate connections, shown in Figure 72 (the IACC is not shown, but it's comparable to the IHC).

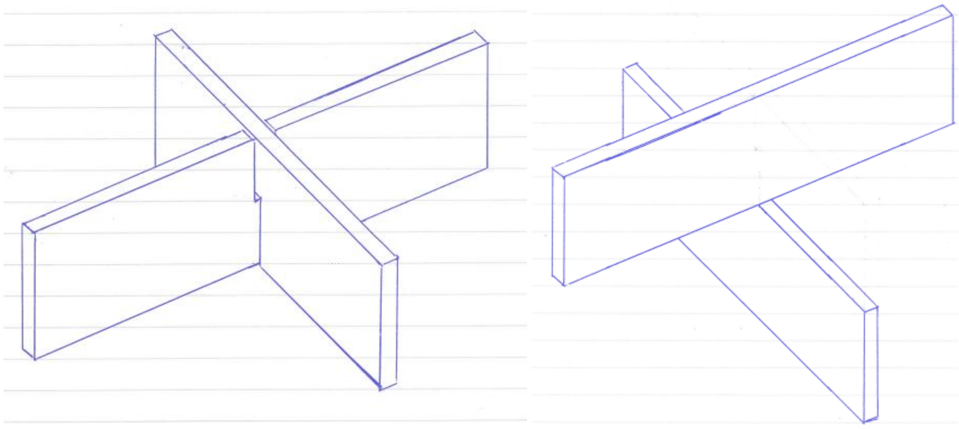


Figure 72 IHC (left) and SC (right)

The Interlocking Halfway Connection is visible when it is used for an internal core. Therefore the Interlocking Above Ceiling Connection was designed. The Staggered Connection is also aesthetically more pleasant. It is difficult to hide the joints of the IHC because the elements will move relative to each other. Due to this movement, the finishing might crack.

10.2.4 Connections between horizontal en vertical elements (floor connection)

The last connection to be discussed is the floor-wall connection. There are several possibilities:

- Corbel connection (Figure 73 A),
- Wall-floor-wall connection (Figure 73 B),
- Steel tube connection (Figure 73 C),
- Steel strip connection (Figure 73 D).

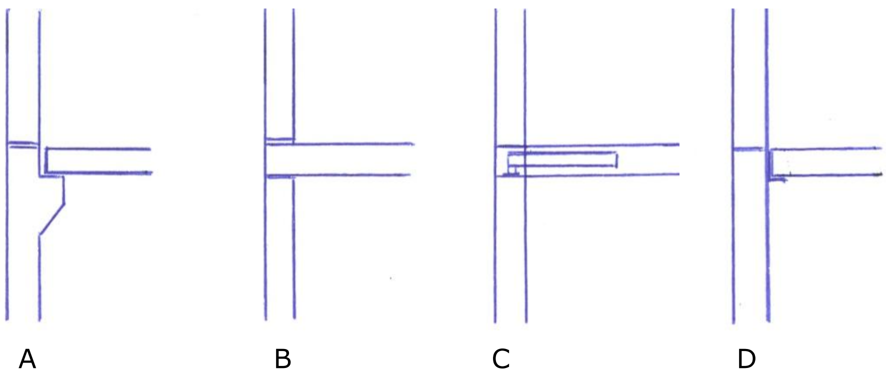


Figure 73 Connections between horizontal en vertical elements

Corbel connection
The corbel connection is used very often in office buildings because of its simplicity and the floor doesn’t interfere with the joint. For residential buildings, the corbel is less regular because of the aesthetics. A disadvantageous of this connection is the eccentricity of the floor support. As a result, a bending moment is created in the wall.

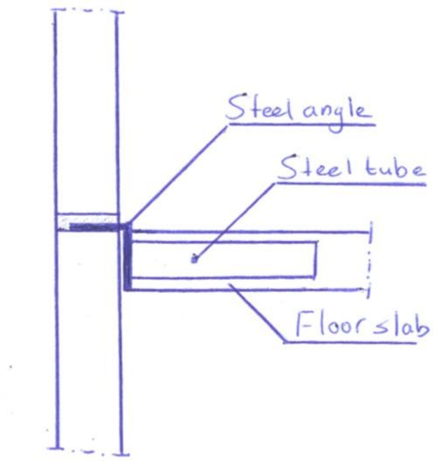
Wall-floor-wall connection
In low rise buildings this connection is applied very often. This is because the load bearing capacity of the floor determines the load bearing capacity of the wall and instead of one now two joints are necessary. Since there is no visual hindrance, it’s possible to apply this connection in residential buildings.

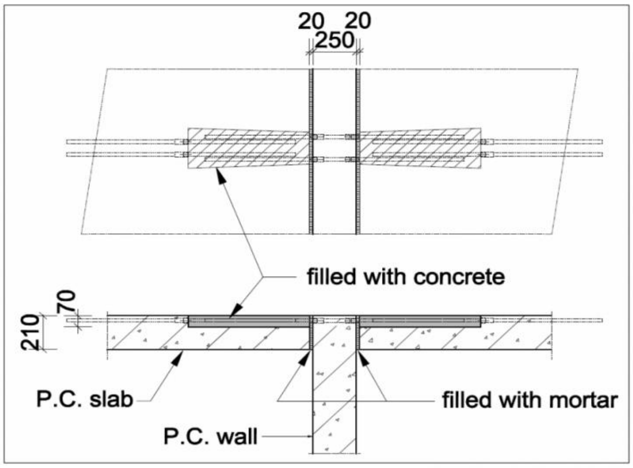
Steel tube connection
This connection maintains the aesthetic benefits of the wall-floor-wall connection, but the load capacity and double joint problem are prevented. Because of the thick joint, formwork is necessary. This connection was applied at the Waterstadtoren (see Figure 14).

Steel strip connection
The steel strip connection reduces the thickness of the joint and the aesthetic hindrance is limited. When the steel strip is kept clear, problems might arise with fire protection. At the Erasmus MC tower they applied both the corbel and steel strip connection (see Figure 74 and Figure 16). Due to problems, it was not possible to place a corbel on the load bearing facade elements and they were forced to use a steel strip. The steel strip is protected by a lowered ceiling. Just as the corbel connection also a small eccentricity is created by the steel strip connection.

Figure 74 Steel strip connection at the Erasmus MC tower

At Het Strijkijzer they reengineered the steel tube connection, to increase the building speed. The steel tubes in the floor slabs were maintained, two on both sides. At the end of every tube, a steel bearing angle was welded (hoeklijn in Dutch). This angle was placed on adjustment plates on top of the wall elements (see A of Figure 75).
Two internal tension ties were used in both directions for the structural integrity and to increase the cooperation between different floor fields (see B of Figure 75 and Figure 77). Four wet joints were used for every floor element to transfer the shear forces to the stability structure. The Porthos tower in Eindhoven used a similar system and in Figure 76 the wet joints and steel angles are visible.





A B

Figure 75 Steel angle-tube connection from Het Strijkijzer

Figure 76 Floors from the Porthos project [Architectenweb 2012]

In Figure 77 the floor layout of Het Strijkijzer is shown. Separate floor elements are denoted with VAXXXX and the red lines represent the structural tension ties. Per floor plate the four wet connections can be distinguished. Several floor plates contain six wet connections, because they are intersected by the tension tie (for example floor plate VA0301 in the left bottom corner).

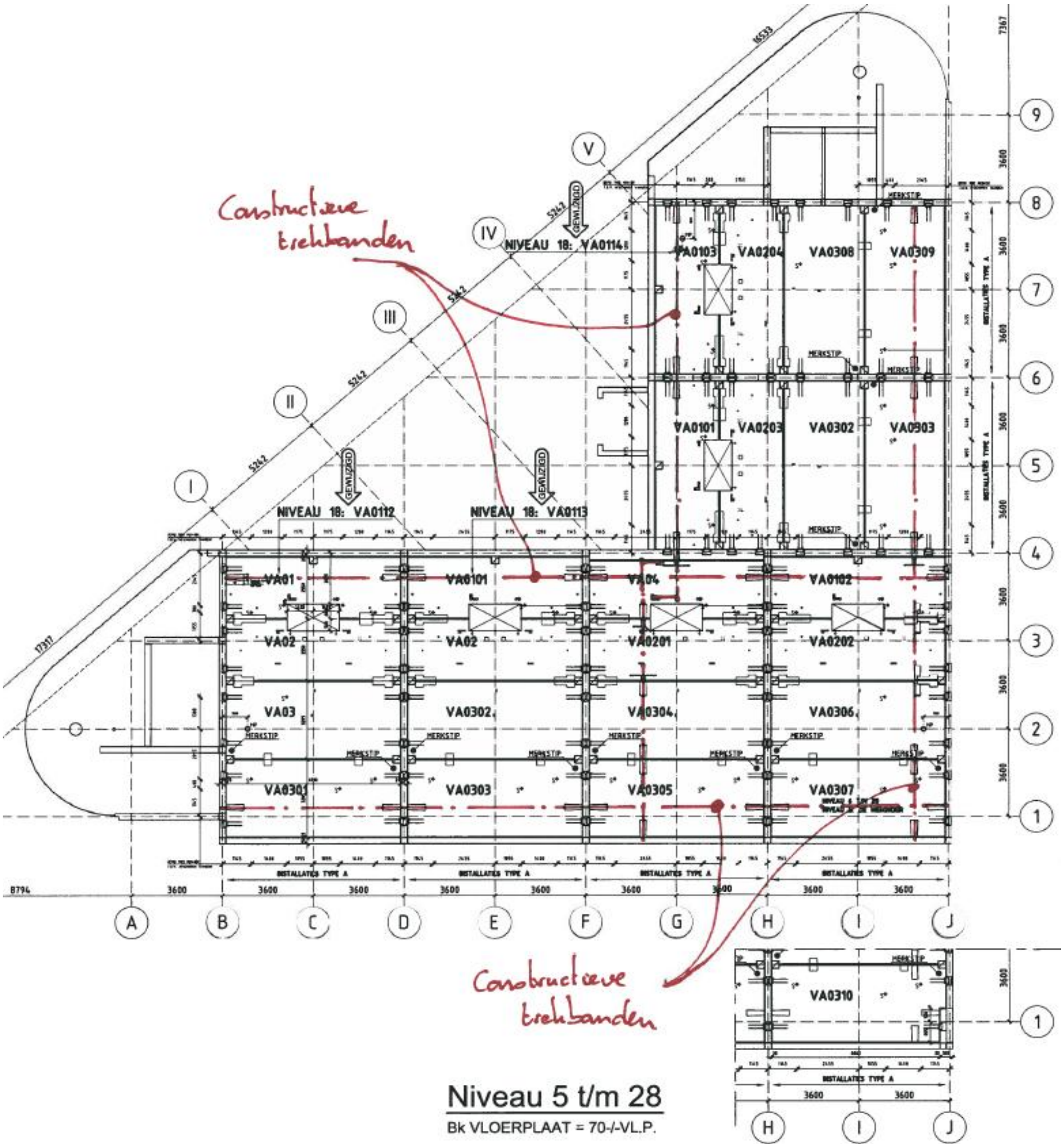


Figure 77 Floor layout from Het Strijkijzer [Hurks Beton]

When the building is loaded with wind in the y-direction (vertical direction in Figure 77), the wind is transferred from the facade to the floor fields B/D, D/F, F/H and H/J. Every floor field contains four floor elements and these elements transfer the wind force trough the wet connections to the stability structure. Because there is no structural topping, every floor element will work as a single diaphragm. To increase to cooperation of all the floor fields, four tension ties are applied. Compared with a system with a structural topping, this tension tie system will behave less stiff. The reason why this system is applied is because a considerable time reduction is obtained (only a thin screed layer is required and no reinforcement or ducts have to be placed).

When the building is loaded with wind in the x-direction (horizontal direction in Figure 77), the wind is transferred from the facade to the floor fields 1/4, 4/6 and 6/8. But in floor field 1/4 there are no wet connections between the floors and the internal walls (B, D, F, H and J) have to provide the diaphragm action. Since the walls are loaded in the weak direction, the vertical tension ties are extended to provide more stiffness.

By applying this system no structural topping was required and this resulted in a time reduction during the construction phase (ruwbouwfase in Dutch). Because of tolerances, a non structural screed layer was applied during the finishing phase (afbouwfase in Dutch). Generally, the finishing phase extends far beyond the construction phase and pouring concrete should be prevented as much as possible in the finishing phase.

10.3 Structural behaviour of the connections

In the previous section several possible connection were discussed for the four different locations. In this section, the structural behaviour of these connections will be examined.

10.3.1 Horizontal connection between two wall elements

For the horizontal connection there are three possibilities. The wet connection with starter bars is most common and has the most advantages. Therefore, only this connection will be discussed.

Normal stress
Horizontal connections endure a very large compression stress. The entire weight of the structure has to be transferred via these joints to the foundation. Several factors determine the strength of the connection [Falger 2004]:

- Strength of the mortar
  The mortar that is placed between the elements plays a role in the overall strength of the wall. A low strength mortar will result in a local strength reduction and this affects the entire structure.

- Degree of filling
  In section 10.2.1 several techniques to fill a horizontal joint were explained. Depending on the technique, air bubbles can be enclosed in the joint. Large amount of air bubbles have a negative influence on the strength and they should be avoided as much as possible. Pump grouting (onderpompen in Dutch) and overflow pouring with fluid mortar have the highest degree of filling: respectively 97 and 95%.

- Thickness of the joint
  Thick joints are more likely to fail than thin joints, especially when the joint is unreinforced. This phenomenon is caused by the friction (aanhechting in Dutch) between the mortar and prefab element. When the connection is loaded with compression, the mortar wants to expand perpendicular to the element. This is known as the Poisson effect. The friction prohibits this expansion and this has a positive effect on the strength of the connection (the concrete is confined). In the centre of the connection there is less friction and the concrete expends more. As a result tension forces occur in the mortar and the connection fails in an hourglass pattern.

- Reinforcement
  Reinforcement is mainly used to increase the tension and shear capacity, but it also increases the strength of the connection. By adding reinforcement, the tension forces due to the Poisson effect can be taken up and the thickness effect is reduced.

- Strength of the surrounding concrete
  The concrete elements above and beneath the joint might differ in strength and stiffness. As a result of different prevented deformations between the contact areas, higher tension stresses occur.

- The location of the joint
  The location of the joint is of large importance, because a confined joint could have a stiffness up to two times larger than a non confined joint.

In a parametric study from BFBN [Bennenk, chapter 8] it's concluded that the joint compression capacity mainly depends on the degree of filling and the strength class of the surrounding concrete. The strength class of the mortar is less important. The strength of a mortar joint can be calculated with VBC 1995, 9.17.3 and CUR-Aanbeveling 97. If the previous aspects are taken into account during the calculation and execution of the connection, a compression stiffness comparable to the surrounding elements may be used.

Tensile forces are a different aspect. Since the elements are relative smooth, no tensile force can be transferred trough the mortar joint (the adhesion is too low). The protruding starter bars can, but the steal area is relative small compared to the internal reinforcement area. Tensile stresses could lead in this case to high stresses in the reinforcement, which results in large deformations. Placing an enormous amount of starter bars in the connection may increase the tension capacity, but this is not a regular solution (see Figure 59). A better option is to prevent the tension in the first place.

The mechanism of transferring tension between two elements is based on ribbed (geribbeld in Dutch) gains and starter bars. Gains are thin metal tubes that are casted into the concrete element. The smaller starter bars from the bottom element protrude into the gains and the free area is filled with fluid mortar. When a tension force occurs, compression diagonals are created. This mechanism is shown in Figure 78.

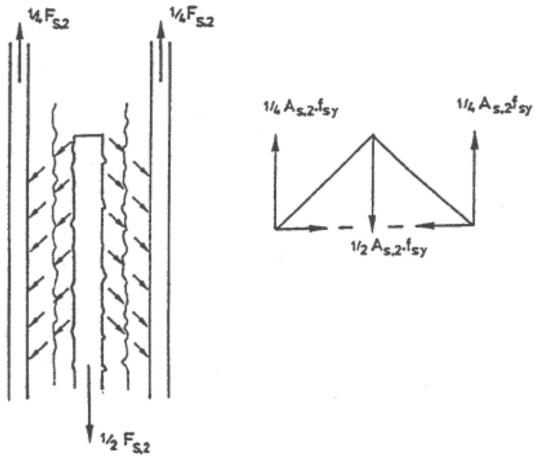


Figure 78 Force distribution between a gain and a starter bar [CT4281 2005]

A safe assumption is to take the normal stiffness equal to zero when the connection is loaded with tension. It is possible to calculate the reduced stiffness, but research done by Falger [Falger 2004, p. 73] has shown that the small difference can be neglected.

Shear stresses
Besides the normal stress, the connection is also loaded by shear force. This shear force is taken up the mortar and the starter bars. Several mechanisms play an important role:

1. adhesion between the mortar and prefab element,
2. friction between the mortar and prefab element
3. dowel action of the starter bars,
4. pull out of the starter bars,
5. normal stress.

In contrast to the compression stiffness that is equal to the surrounding concrete, the shear stiffness will be lower than the surrounding concrete. The smooth surface of the concrete is the cause of this reduction: there is little or no adhesion and mechanism 1 is removed. Furthermore, the shear force is more difficult to determine as it's dependent on the normal stress in the joint. When the connection is loaded with tension, also mechanism 2 and 5 will be equal to zero. To determine the shear resistance, Eurocode NEN-EN 1992-1-1, section 6.2.5 can be used:

$$v_{Rdi} = cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin(\alpha) + \cos(\alpha)) \leq 0{,}5\ v\ f_{cd}$$

in which:

v_{Rdi}	is the design shear resistance at the interface,
c and μ	are factors depending on the interface (for very smooth: c=0,25 and μ=0,5),
f_{ctd}	is the design tensile strength of the concrete,
σ_n	is the minimal normal stress in the joint that can coincide with shear force, positive for pressure, whereby $\sigma_n<0{,}6f_{cd}$ and negative for tension. It's advised to use $cf_{ctd}=0$ when σ_n is in tension,
ρ	is the area of the protruding bars divided by the connection area: A_s/A_i,
α	is the angle of the reinforcement (see Figure 79),
v	is the stiffness reduction factor:

$$v = 0{,}6[1 - \frac{f_{ck}}{250}]$$

In which

f_{ck} is the characteristic cylindrical concrete compression strength.

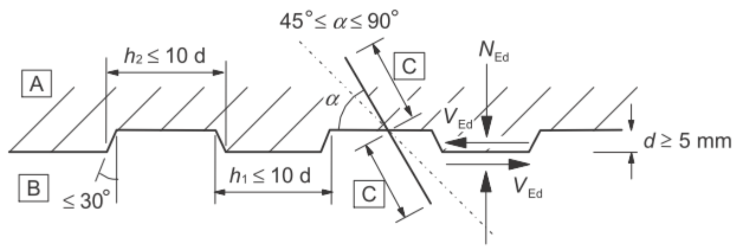


Figure 79 Explanation of factors in shear force calculation [NEN-EN 2005]

The first part of the shear resistance formula takes the adhesion into account. Because the concrete elements have a smooth surface, $c*f_{ctd}$ becomes equal to zero. The second part is responsible for the normal stress. In case the connection is loaded in tension, this value is also equal to zero. The third part is the contribution of the reinforcement. When a connection is loaded in compression and contains starter bars at an angle of 90°, the formula can be rewritten as:

$$v_{Rdi} = \mu\sigma_n + \mu\rho f_{yd} \leq 0{,}5\ v\ f_{cd}$$

Because the shear resistance depends on the normal stress, the value for v_{Rdi} will change over the height and the width of the structure. Several methods can be applied to prevent a repetitive calculation and a long construction time of the FEM model. For example, the shear resistance can be calculated per prefab element and the elements can be placed into classes: class 1 (0-5N/mm^2 compression), class 2 (5-10N/mm^2 compression) and so on.

Now the shear resistance is known, it's possible to calculate the shear stiffness. Research of Straman [Straman 1988] has shown that there is a linear relation between the shear force and the deformation. Although this research was done with vertical connections, the mechanism remains equal. Tests done in this research show that the connection will fail at a deformation of approximately 1mm. Other research was done on this subject, but the results from Straman are still used. In general, the shear stiffness will be calculated with the following formula:

$$K_u = \frac{v_{R,di}}{\delta_u} \; [N/mm^3]$$

in which:
v_{Rdi} is the shear resistance,
δ is the deformation at failure: δ=1mm.

10.3.2 Vertical connections between two wall elements

After the construction of the Prinsenhof project, Falger examined the element configuration of this building for his master graduation thesis. The main question was: what is the influence of the masonry element configuration with open vertical joints on the structural behaviour of a concrete stability structures. He used one of the walls of the Prinsenhof project as reference for his thesis. Falger only researched this wall in 2D and the connection with the two perpendicular facades was not included. Because the openings have a large influence on the behaviour, four different layouts were examined (see Figure 80).

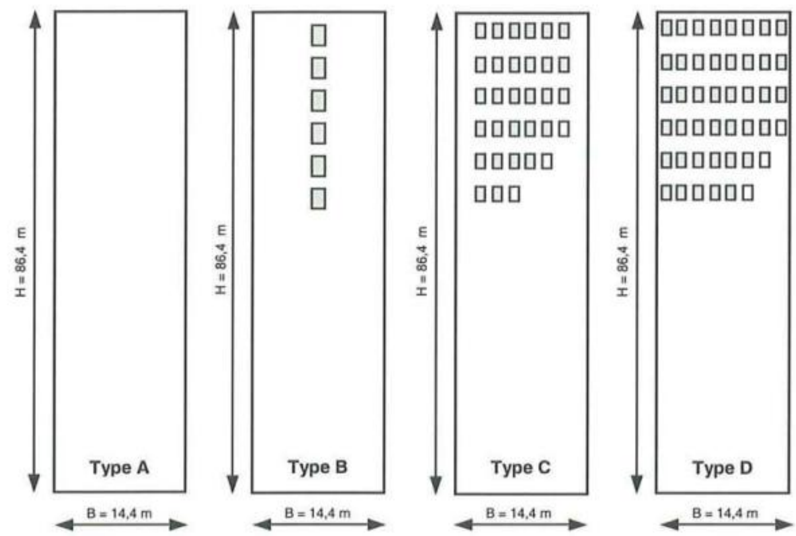


Figure 80 Four different structural layouts researched by Falger [Falger 2004]

Every layout consists out of 24 floors with a height of 3,6m (total 86,4m), a thickness of 0,3m and a width of 14,4m. In order to analyse the influence of an open vertical joint, six different vertical connections were considered:

1. monolithic,
2. reinforced smooth connection,
3. reinforced tooth connection,
4. welded steel plates,
5. welded steel profile,
6. non-structural open vertical joint.

For every layout type (A, B, C and D), these six connections were modelled. For connection 2, 3, 4 and 5 a traditional (non masonry) element layout was used. The location of the six connections in layout type B are depicted in Figure 81.

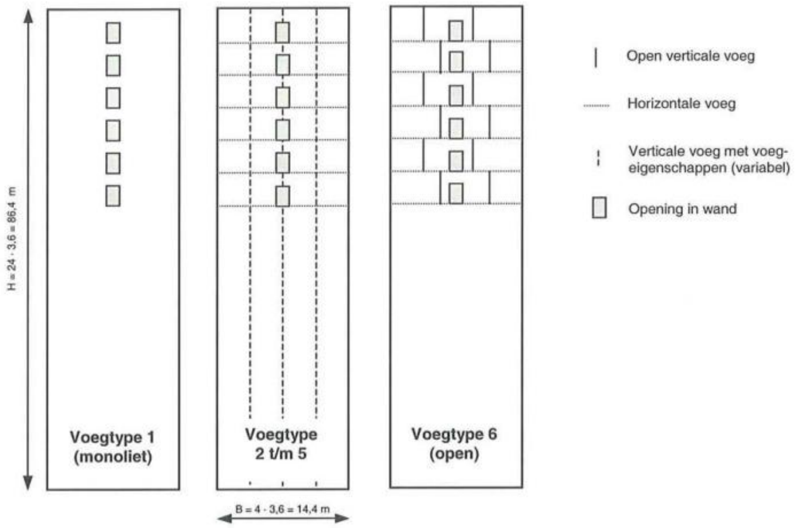


Figure 81 Six different connections for layout tybe B [Falger 2004]

The openings in Figure 82 are hxw=2.8x1.8m^2 and for connection 6 (open vertical joint), the minimal overlap of the elements is 0.25 times the element width (this minimal overlap is necessary for a proper interaction). To achieve this, two elements of 5.4m and one element of 3.6m are used (overlap of 1.8m).

In conclusion, there are four structural layouts and every layout has six different connections. Falger analysed all twenty-four variants with a Finite Element Method analysis (FEM) and in Table 14 until Table 17 the deformation and stiffness of the different vertical connections are shown.

Table 14 Deformation and stiffness vertical joint of structural layout A [Falger 2004]

	Type 1 (mono)	Type 2 (glad)	Type 3 (tand)	Type 4 (plaat)	Type 5 (UNP)	Type 6 (open)	Hand (monoliet)
$U_{x;top}$ [mm]	51,5	57,5	55,1	62,7	59,3	54,2	47,6
$U_{x;top}$ [%]	100,0	111,8	107,0	**121,8**	115,3	105,3	92,4
$K_{tt;voeg}$ [N/mm^3]	40*	1,31	3,60	0,56	0,90	-	40*
$K_{tt;voeg}$ [%]	100,0	3,3	9,0	1,4	2,3	-	-

Table 15 Deformation and stiffness vertical joint of structural layout B [Falger 2004]

	Type 1 (mono)	Type 2 (glad)	Type 3 (tand)	Type 4 (plaat)	Type 5 (UNP)	Type 6 (open)
$U_{x;top}$ [mm]	63,4	76,7	70,0	76,0	73,4	68,5
$U_{x;top}$ [%]	100,0	**121,0**	110,3	119,9	115,7	108,0
$K_{tt;voeg}$ [N/mm^3]	40*	1,31	3,60	0,56	0,90	-
$K_{tt;voeg}$ [%]	100,0	3,3	9,0	1,4	2,3	-

Table 16 Deformation and stiffness vertical joint of structural layout C [Falger 2004]

	Type 1 (mono)	Type 2 (glad)	Type 3 (tand)	Type 4 (plaat)	Type 5 (UNP)	Type 6 (open)
$U_{x;top}$ [mm]	63,7	70,5	68,1	75,5	72,3	67,6
$U_{x;top}$ [%]	100,0	110,7	106,8	**118,5**	113,5	106,1
$K_{tt;voeg}$ [N/mm^3]	40*	1,31	3,60	0,56	0,90	-
$K_{tt;voeg}$ [%]	100,0	3,3	9,0	1,4	2,3	-

Table 17 Deformation and stiffness vertical joint of structural layout D [Falger 2004]

	Type 1 (mono)	Type 2 (glad)	Type 3 (tand)	Type 4 (plaat)	Type 5 (UNP)	Type 6 (open)	Hand (monoliet)
$U_{x;top}$ [mm]	84,4	92,0	89,2	97,4	93,9	89,5	82,1
$U_{x;top}$ [%]	100,0	109,0	105,7	**115,5**	111,3	106,0	97,3
$K_{tt;voeg}$ [N/mm^3]	40*	1,31	3,60	0,56	0,90	-	40*
$K_{tt;voeg}$ [%]	100,0	3,3	9,0	1,4	2,3	-	-

From these four tables, it can be concluded that an open vertical joint in combination with the masonry configuration is the stiffest connection of them all. The deformations are smaller than those of traditional joints. When these connections are applied at a closed wall (Table 14), the deformations increase with merely 5.3% compared to a monolithic structure. The largest deformations occur at structural layout B: an increase of only 8.0%.

There are currently no design rules to calculate these connections and the only accurate method is to use a FEM analysis. It is possible to use hand calculations in order to prevent an extensive FEM analysis, but the results lack precision. With the increased possibilities and user friendliness of the modern software programs, a FEM analysis is preferred.

An important side note should be made with regard to the results. Falger used the full Young's modulus during all the calculations. As a result of creep and shrinkage, the compressed Young's may be reduced up to 2/3rd of the original value. As a result, the stiffness of the concrete is too high. Because this value was applied in all the calculations, the difference between the calculations will not change, but the absolute deflections may not be accurate.

10.3.3 Vertical connections between two perpendicular wall elements

The corner connections are important to activate the flanges. The stiffness of connection 1 to 5 obtained in the previous section could also be applied to the corner connections. The only difference between these two variants is that one of the elements of the corner connection is rotated 90 degrees. The stiffness of the 6th connection (masonry configurations) is not applicable because the overlap of the elements is less than 0.25 times the length.

Tolsma researched the application of 3 dry corner connections for high rise stability cores: the Interlocking Halfway Connection, the Interlocking Above Ceiling Connection and the Staggered Connection (see Figure 71). The staggered connection can be compared with connection 6 from Falger's research: the masonry configuration. Several interesting conclusions could be made:

- The smeared stiffness of the IHC and IACC is twice as large compared to the SC (see Table 18).
- The IHC is able to transfer considerable larger shear stresses than the SC.
- The IHC has the highest strength capacity. In Table 18 a factor 0.6 is applied because of the cyclic loading.
- When the stiffness of a precast corner connection is compared to a monolithic corner connection, the IHC shows an increase of lateral deflection of only 3.3%. The SC results is an increase of 5.9%. The IACC was not calculated because it failed under the load of the reference project. The displacements are depicted in Figure 82.

Table 18 Smeared connection stiffness and connection strength [Tolsma 2010]

	$K_{smeared} = \dfrac{K_{discrete}}{h \cdot d} [MN/m^3]$	$f_v = 0.6 \cdot \dfrac{F_r}{h \cdot d} [N/mm^2]$
IHC	1694	0.60
IACC	1582	0.21
SC	837	0.39

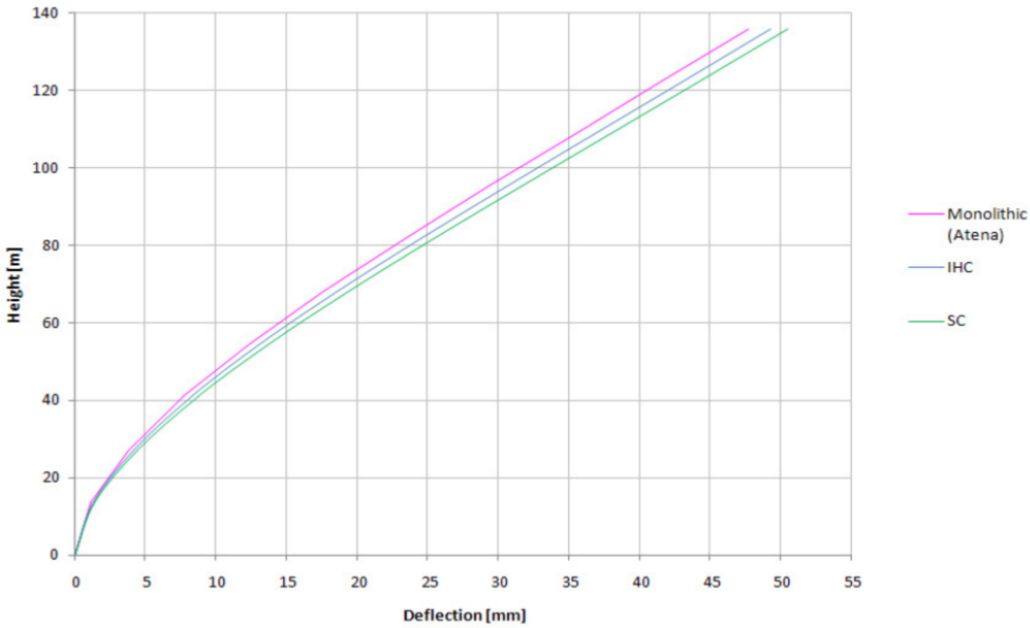


Figure 82 Influence of corner connections on lateral deflection [Tolsma 2010]

It can be concluded that the IHC is the best prefabricated corner connection compared to the two other connections, based on strength, shear stiffness en global deformations. Tolsma did not compare the three dry corner connections with other dry and wet corner connections. However it is possible to make an reasonable comparison based on the research from Tolsma and Falger. The IHC and SC deform 3.3 and 5.9% more compared to a monolithic closed structure, because less shear force is transferred to the flanges. The five connections researched by Falger deform 7 (tooth connection) to 21.8% (welded steel plate) more than a monolithic closed structure. Of course these two studies have been done with different parameters, and with new calculations the difference might be smaller between the tooth connection and the IHC. But it is unlikely that the two times higher stiffness of the IHC will decrease till a level where the tooth connection becomes stiffer. This assumption is made more plausible, because it was impossible for Tolsma to model a forty story 3D core with precast elements and a fine mesh. With the current state of computational capacity he decided to use two models. With the first complex 2D model he calculated the discrete stiffness of the connection. This stiffness was than imported to the second global 3D model as a smeared stiffness between the perpendicular core walls. In other words, Tolsma and Falger both calculated the stiffness of the connection in a 2D model, making it slightly easier to compare the results.

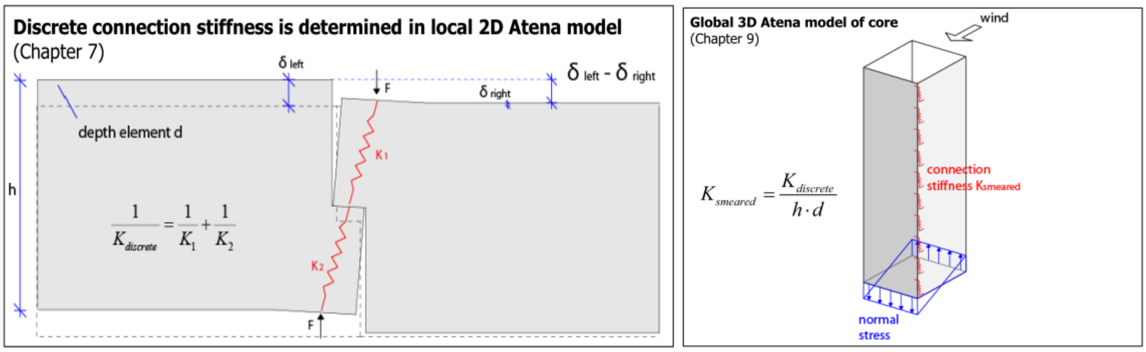


Figure 83 Calculation method Tolsma [Tolsma 2010]

Besides these corner connections there are also intermediate connections between two perpendicular wall elements (see Figure 72). Because the mechanism don't change, the stiffness remains equal. The overall deformations will decrease compared to the corner connection, because the flanges play a smaller role in the entire stiffness.

10.3.4 Connections between horizontal en vertical elements (floor connection)

These connections play an important role in the diaphragm action of the floor and they transfer the wind load from the floor to stability system. Commonly, tension ties are placed around the prefab floor elements and this area is filled up with fluid mortar. As a result, an edge beam is created that connects the single elements. Via this edge beam, shear forces can be transferred between the elements. The size of the edge beam determines the amount of the dowel action (see Figure 84).

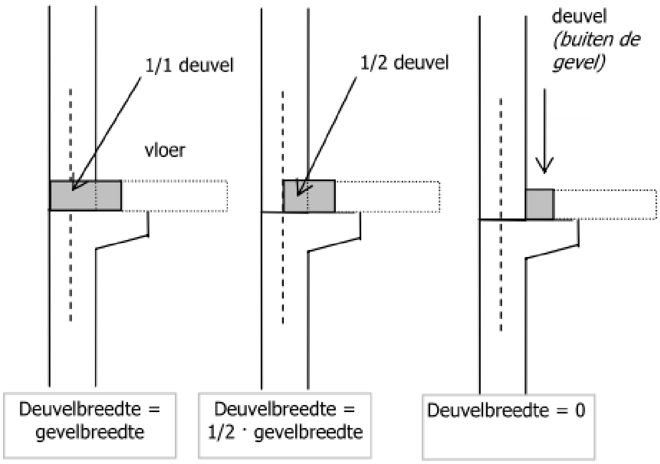


Figure 84 Dowel action of the edge beam [Pieterse 2007]

Besides the tension tie function and providing dowel action in the façade, the edge beam also increases the resistance against progressive collapse. Little was known on the exact behaviour of the edge beam and Pieterse, in combination with Stufib-studiecel 10, examined this topic for his master thesis. For this research Pieterse used a façade with a high of 36m (10 stories), a width of 10.8m and an element thickness of 0.3m. One of the research questions was: is it possible to connect the elements with an edge beam, using a traditional element configuration? A second question was: how large is the stiffness of an edge beam compared to other connections? To answer this last question, he used the research done by Falger and created a second façade with a height of 86,4m, a width

14,4m and a thickness of 0.3m. An overview of the element configuration can be found in Figure 85.

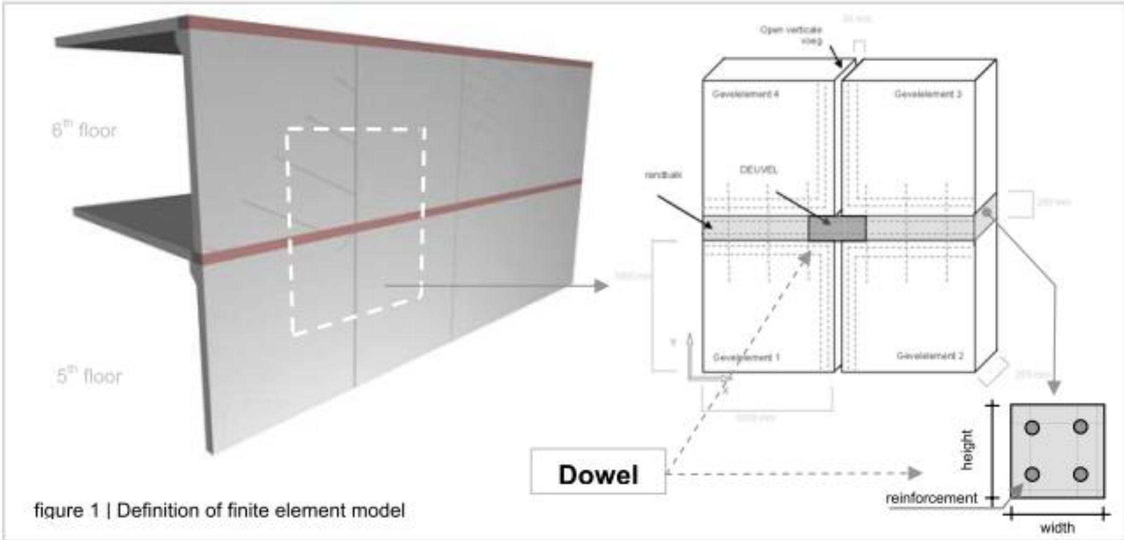


Figure 85 Overview of the element configuration [Pieterse 2007]
Several interesting conclusion were made by Pieterse:

- the façade with an height of 36m (h/w=3.3/1) satisfies the deformation requirements from the NEN 6702 when only dowels are used (B35; hxw=250x300mm^2),
- the deformation of a façade (h=36m) with dowels is approximately 50% larger than that of a façade with welded steel plate connections,
- the deformation of a façade (h=36m) with dowels is approximately 140% larger than the deformation of a monolithic façade,
- a façade with a height of 86,4m (h/w=6/1) does not meet the deformations requirements from the NEN 6702 when only dowels are used (B35; hxw=250x300mm^2).

The deformation of several connections is shown in Figure 86.

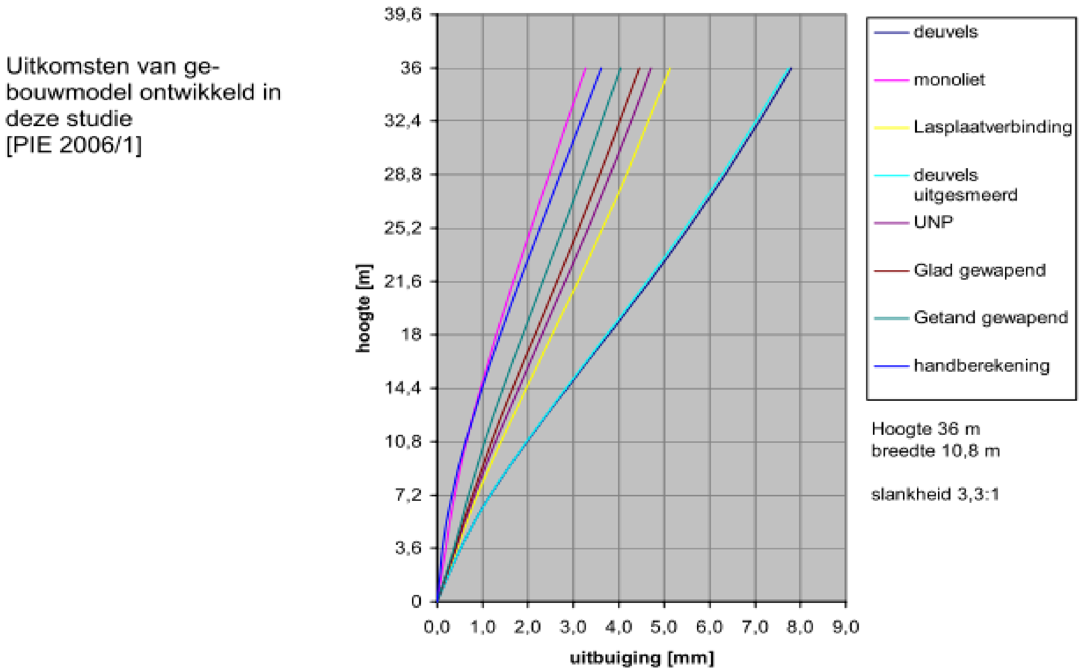


Figure 86 Deformation of different connections [Pieterse 2007]

In conclusion: dowels are a possible connection for low rise buildings. Because of the tension ties and floor connection it is rather simple to increase the joint area and construct a continuous dowel. The low stiffness compared to the other connection makes that this connection alone is not suited for high rise buildings.

The reinforcement bars needed in the structural concrete topping to transfer the shear force from the floor to the stability system are shown in Figure 87. In this figure also an increase of reinforcement around the edges can be seen. The amount and diameter of these coupling bars are based on the required shear force capacity. If a structural topping is unwanted, the tension ties can also be placed internally (see section 3.1 for an example).

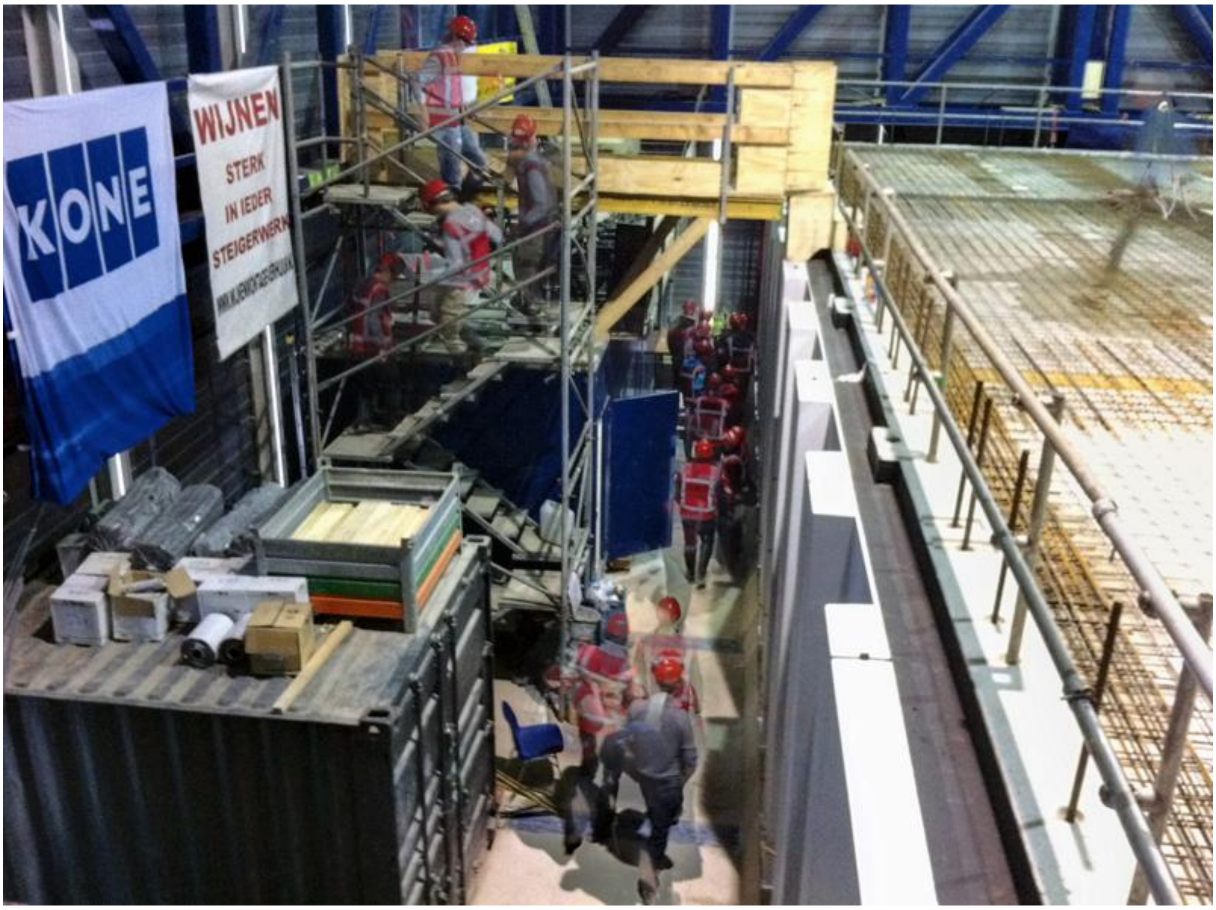


Figure 87 Structural topping reinforcement

10.4 Response stability system to lateral load

In chapter 9.2 it was determined that when the building height doubles, the shear force increases with a factor of 2, the bending moments with a factor of 4, the sway index with a factor of 8 and the dynamic behaviour with a factor of 16. This factor is based on a cantilevered beam fixed into the ground with a constant load (see Figure 88).

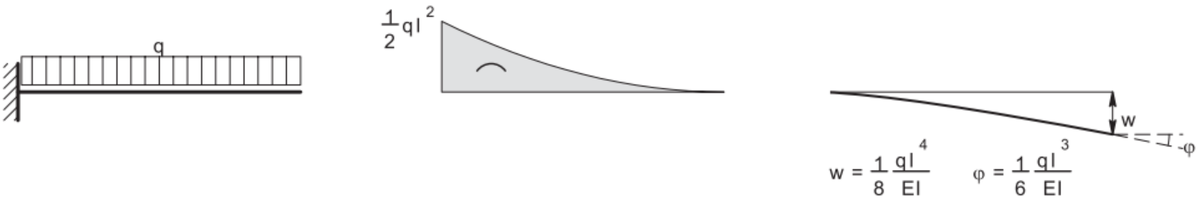


Figure 88 Fixed cantilever [Welleman]

In reality, the lateral load is not constant and increases over the height. This will result in a bending moment that increases more rapidly to the base. The amount of shear load remains the same, but the shape changes. As a result of the non linear lateral load the deformations also increase. When this is combined with functional requirements, challenges are created. Many high rise buildings have a public function in the lower section and large open areas are preferred. Combining this with a stiff stability system requires innovative solutions that change the response to lateral load.

Regular openings (shaft outlets, elevator and stair doorways) in stability structures also have a large influence on the response to lateral load. Because of these intersections, coupled systems are created, which form a highly complicated structural system.

To get a full insight in the response of a stability structure, the following actions are examined:

- deflection because of bending,
- deflection because of shear,
- deflection because of bending and shear,
- deflection and stresses because of a coupled structure,
- flange activation and shear lag,
- vibrations and acceleration due to cyclic wind loading.

10.4.1 Deflection because of bending

The derivation is based on the reader "An introduction to the Analysis of Slender Structures" from the course CIE 4190 [Simone 2010]. Consider Figure 89 (a). From simple geometric considerations follows:

$$\mathrm{ds} = \rho \mathrm{d}\theta \ \text{ and } \ \kappa = \frac{1}{\rho} = \frac{\mathrm{d}\theta}{\mathrm{ds}} \tag{10.1}$$

From Figure 89 (b) it follows that:

$$\frac{\mathrm{dv}}{\mathrm{dx}} = \tan\theta \tag{10.2}$$

Under the assumption of very small rotations we may set $\sin\theta \approx \tan\theta \approx \theta$ and $\cos\theta \approx 1$. Hence $ds=dx$, $\kappa=1/\rho=d\theta/dx$ and $dv/dx=\theta$. Taking the first derivative of θ and using the expression for the curvature we obtain:

$$\frac{\mathrm{d}^2\mathrm{v}}{\mathrm{dx}^2} = \kappa = \frac{1}{\rho} \tag{10.3}$$

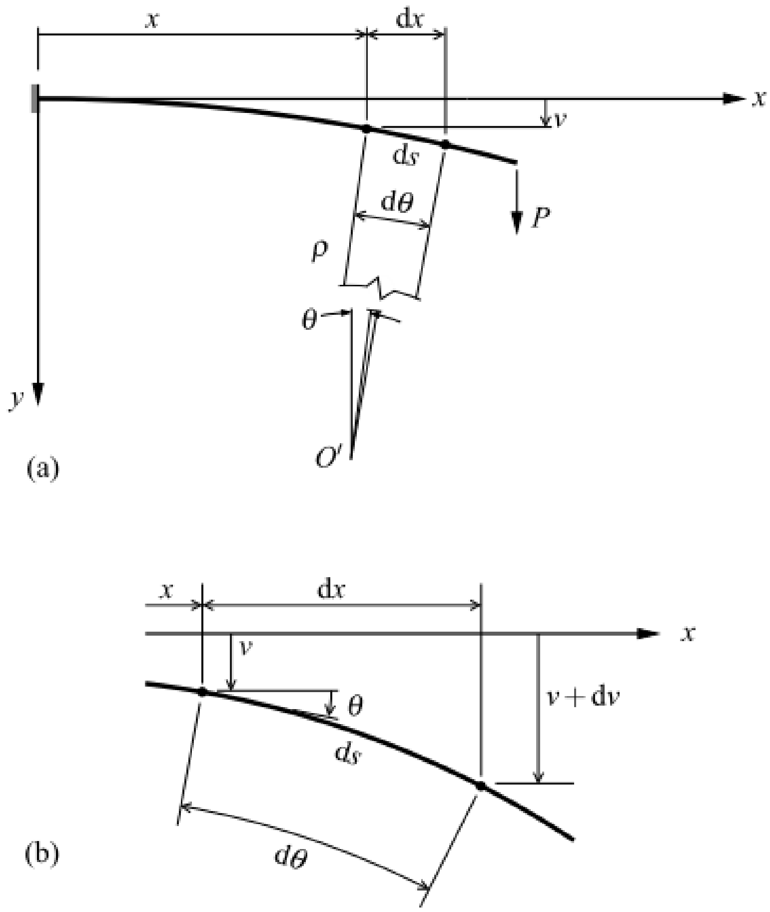


Figure 89 Kinematic assumptions [Simone 2010]

The relationship between load, shear force and bending moment is derived by expressing the equilibrium of an infinite small element dx of a beam in bending loaded with a distributed load of intensity q as shown in Figure 90. The relation between shearing forces and distributed load is obtained from equilibrium of forces in the vertical direction and reads as:

$$\Sigma F_y=0:\ V+dV-V+qdx=0$$
$$\frac{dV}{dx}=-q \tag{10.4}$$

The equilibrium equation obtained by summing moments about an axis through the left hand face of the element and orthogonal to the plane of the figure yields:

$$\Sigma M=0:\ M+dM-M-Vdx=0$$
$$\frac{dM}{dx}=V \tag{10.5}$$

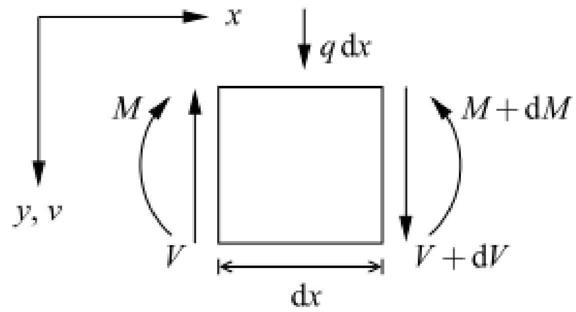


Figure 90 Element dx of a beam [Simone 2010]

The longitudinal strain is related to the stress by means of Hooke's law through the Young's modulus so that $\sigma_x = E\varepsilon_x = -E\kappa y$.

Consider Figure 91 representing a portion of a beam in bending where the internal moment at the right-hand cross section is replaced with the corresponding stress distribution. For equilibrium, the internal couple resulting from the sum of $\sigma_x dAy$ over the whole section must equal the internal moment M. Hence, its moment about the z axis is $dM = -\sigma_x dAy$. The equilibrium equation obtained by summing moments about the z axis yields:

$$M = \int dM = \int \sigma_x y dA = -\int E\kappa y^2 dA = -EI\kappa \tag{10.6}$$

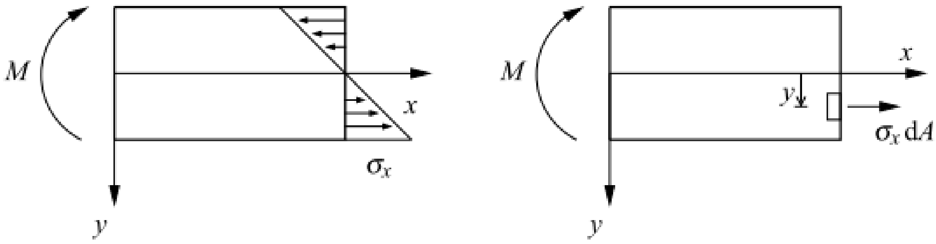


Figure 91 Internal bending [Simone 2010]

The differential equation of the deflection of a beam is obtained by eliminating the curvature κ from (10.3) and (10.6) to obtain:

$$M = -EI\frac{d^2v}{dx^2} \tag{10.7}$$

By making use of the relation (10.4) between shearing force and distributed load and (10.5) between shearing force and bending moment, (10.7) can be expressed as:

$$EI\frac{d^4v}{dx^4} = q$$

After integrating the formula four times, the following expression is obtained:

$$EIv = \frac{1}{24}qx^4 + \frac{1}{6}C_1x^3 + \frac{1}{2}C_2x^2 + C_3x + C_4$$

The four integration values can be solved by using four boundary conditions for the clamped beam:

x=0: w=0, x=0: φ=0, x=l: M=0 and x=l: V=0

This results in:

$$C_1=-ql,\ C_2=0{,}5ql,\ C_3=0 \text{ and } C_4=0$$

The equation becomes:

$$w = \frac{q}{24EI}(x^4 - 4x^3l + 6x^2l^2)$$

When x=l, the formula becomes:

$$w = \frac{1}{8} * \frac{ql^4}{EI}$$

10.4.2 Deflection because of shear

According to the Euler-Bernoulli beam theory, cross sections carry a resultant shearing force V (see Figure 90). But it's remarkable that the deformation associated to the corresponding shear stress is not taken into account. This problem has been solved by Timoshenko by approximating the effect of shear as an average over the cross section.

Shear deformation is described by the shear distortion γ, which is a result of the shear force shown in Figure 92. When the rotations are small, the following relation holds:

$$\gamma \approx \frac{dv}{dx} \tag{10.8}$$

When the material is assumed to be linear elastic, Hooke's law can be written as:

$$\tau = G\gamma \tag{10.9}$$

Considering an average expression of the shear force τ acting on a section, where A_s is the effective shear area, the following formula can be expressed:

$$\tau = \frac{V}{A_s} \tag{10.10}$$

When (10.8), (10.9) and (10.10) are combined, the following formula is derived:

$$\gamma = \frac{dv}{dx} = \frac{\tau}{G} = \frac{V}{GA_s} \tag{10.11}$$

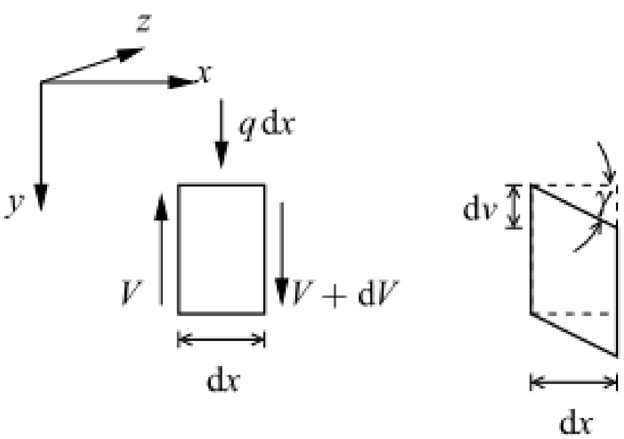


Figure 92 shear deformation by Timoshenko [Simone 2010]

The relation between shearing forces and distributed load is obtained from equilibrium of forces in the vertical direction and reads as:

$$q = -\frac{dV}{dx} \tag{10.12}$$

When (10.11) is combined with (10.12), it results in a second order differential equation where the quantity GA_s is known as the shear rigidity of the beam:

$$-GA_s\frac{d^2v}{dx^2} = q$$

After integrating the formula twice, the following expression is obtained:

$$GA_sv = \frac{-1}{2}qx^2 + C_1x + C_2$$

The two integration values can be solved by using two boundary conditions for the clamped beam:

x=0: w=0 and x=l: φ =0

This results in:

C_1=ql and C_2=0

The equation becomes:

$$w = \frac{q}{2GA_s}(2lx - x^2)$$

When x=l, the formula can be written as:

$$w = \frac{ql^2}{2GA_s}$$

10.4.3 Deflection because of bending and shear

In the two preceding sections, the deflections of a bending and shear beam where determined separately. With the Timoshenko beam theory, an extension of the Euler-Bernoulli beam theory, it’s possible to combine these two deflections:

$$w = \frac{q}{24EI}(x^4 - 4x^3l + 6x^2l^2) + \frac{q}{2GA_s}(2lx - x^2)$$

For the current design a calculation was made to determine the influence of shear deflection. The results are shown in Figure 93.

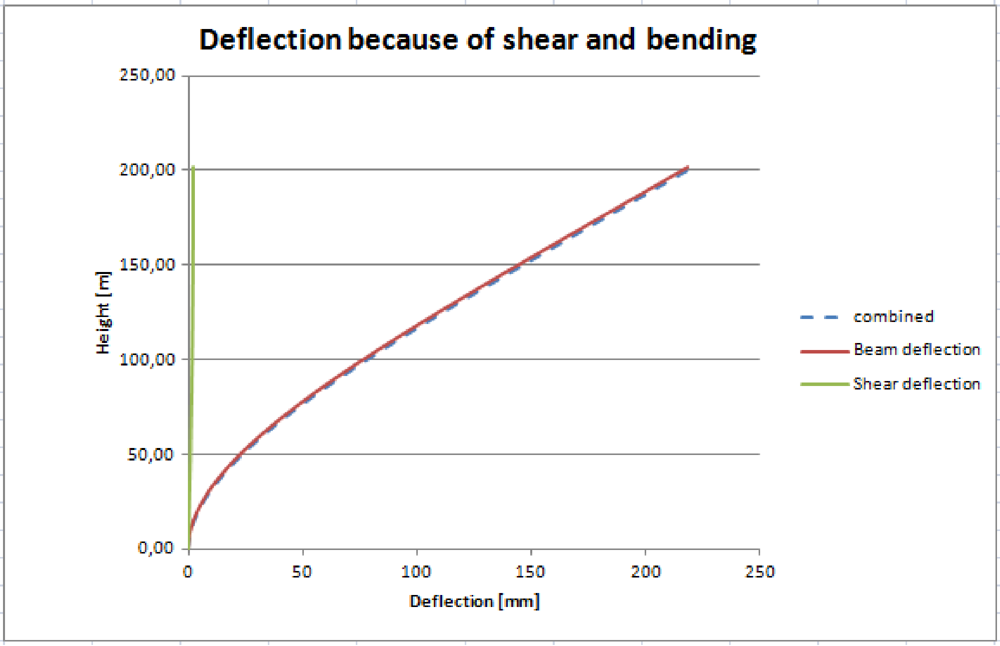


Figure 93 Bending and shear deflection

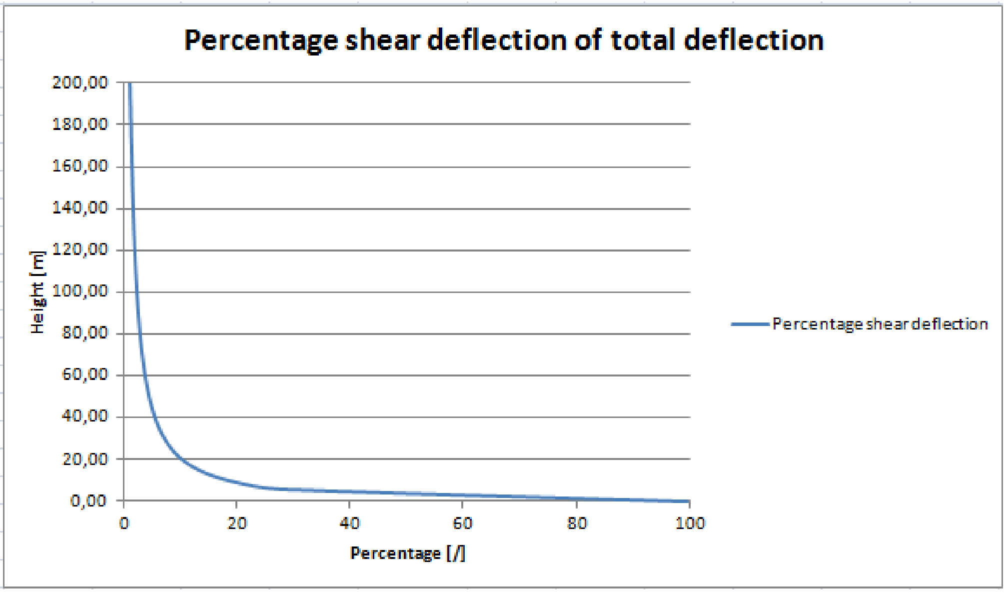


Figure 94 Percentage shear deflection of the total deflection

Figure 93 and Figure 94 show that the shear deformation diminishes with increasing height. At a height of 0m, the shear deflection is 100% of the total deflection. At 20m, the shear deflection is only 10% and at 200m, the shear deflection has been reduced to 1%. The entire calculation can be found in report 2 of the literature study.

10.4.4 Deflection and stresses because of a coupled structure

Openings in the wall are necessary because of functional requirements. The connection between the two sections is provided by thin concrete element, often denoted as beams or lintels (latei in Dutch). The interaction between the two separate walls is of large importance, since the moment of inertia increase with the height to the power of three. For example, if there would be no opening the moment of inertia is $1/12bh^3$. If no connection is created, or a very weak lintel is constructed, the moment of inertia will become $2x1/12b(0,5h)^3=1/48bh^3$. This is $2x0,5^3=1/4$ (75% reduction) of the stiffness of the original wall.

In other words: the interaction between the two walls is very important and depends on the shear stiffness of the connection. To calculate the shear stiffness, the ultimate shear stiffness capacity v_{Rdi} is divided by the deformation at failure δ_u:

$$K_u = \frac{v_{Rdi}}{\delta_u}\ [N/mm^3]$$

The shear stiffness of the connections can be found in section 10.3.2. The development of shear stress v_{Rdi} over the height of a structure is shown in Figure 95.

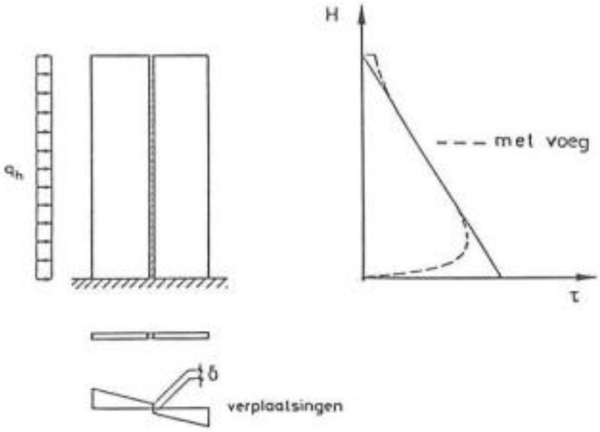


Figure 95 Shear stress τ over the height due to wind load [Stupré 1993]

In Figure 95 a displacement δ is visible between the two sections. This displacement is caused by the shear stiffness and the initial slip.

There are two extremes for the shear stiffness of the lintel:

- K -> ∞ (monolithic wall),
- K=0 (two separate walls).

The stiffness of the lintel will be in between these two extremes: 0<K<∞. This is displayed in Figure 96 and the non linear relation between H, δ and K is visible.

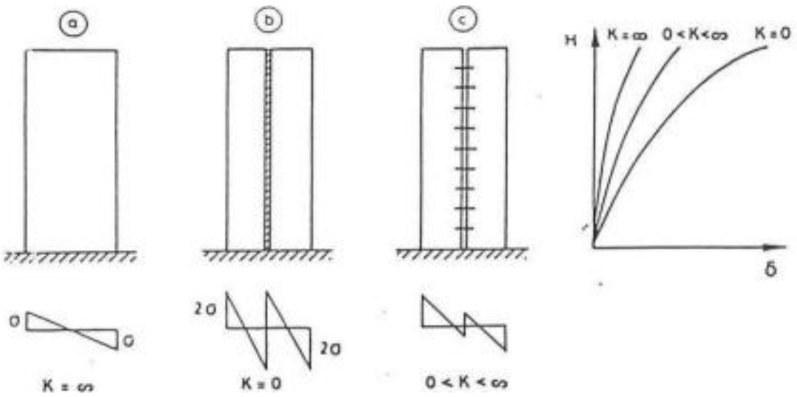


Figure 96 Influence of the K value on deflection and stress distribution [Stupré 1993]

Because this lintel is so important, large stresses occur due to moments. Combine this with the small area and cracks are formed: the stiffness is reduced (of the lintel and therefore also the entire wall). Because the deflection and the moment of inertia are inversely proportional, a 75% reduction of the moment of inertia will lead to a 75% increase of deflections.

10.4.5 Flange activation and shear lag

The corner connections are just as important as the lintel beams for the stiffness of the entire structure. Take for example an I-profile beam. Shear forces are transferred between the web and flanges and the two flanges contribute to the moment of inertia.

To illustrate the effect of flange activation, the moment of inertia of an I-profile beam with and without flanges is compared. For this example a HE500A is used:

No flanges: $I_{zz}=(1/12)*b*h^3=(1/12)*12*490^3=1{,}1765*10^8mm^4$
With flanges: $I_{zz}=(1/12)*b*h^3+2*A*a^2=(1/12)*12*444^3+2*(300*23)*233{,}5^2$
$=8{,}3994*10^8mm^4$

The flanges increase the moment of inertia with a factor of seven. When a small boxed girder is examined, the contribution factor reduces to a factor of four (two flanges instead of one).

The previous examples were relative small and with large box girders, the term effective width becomes important. The flanges and connection are not stiff enough and the flange area becomes too large to contribute entirely to the moment of inertia. Only the effective width can be taken into account and this results in large losses. This phenomenon is depicted in Figure 97.

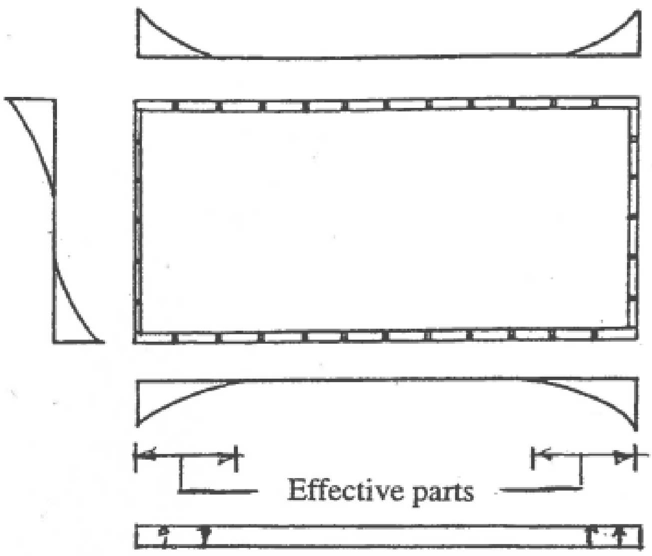


Figure 97 Shear lag and effective width [CT4281 2005]

The Bernoulli-Euler assumption that plane sections remain plane is often used for beam analyses [Kwan 1996]. According to this assumption, stresses because of bending in the flanges and webs should be linearly distributed (see Figure 98 (a)). However, this assumption only holds when there is no shear force in the structure or when it has an infinite shear stiffness. In reality, a shear force is often present in combination with a finite shear stiffness. This results in a shear flow between the web and flange panels and eventually to shear deformation of the elements. Because of this shear deformation, the longitudinal deformation in the centre of the flange and web would lag behind those at the web-flange junction. As a result, large stress concentration areas can be found near the corners and the stresses will be distributed as is shown in Figure 97 and Figure 98 (b).

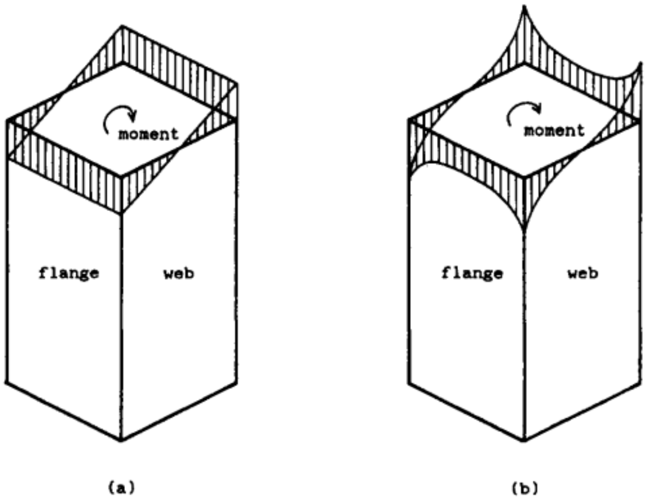


Figure 98 Axial stress distribution in a beam structure: (a) no shear lag: (b) shear lag [Kwan 1996]

To reduce shear lag, the stiffness of the wall has to be increased. This can be done by applying smaller openings or a stronger concrete mixture. Heykal Balbaid [Balbaid 2011] wrote a master thesis about the application of high strength concrete in tubular structures. He also examined the effect of three different concrete mixtures on shear lag effect. Compared to ordinary concrete (C35/45), a reduction of maximal 7% was possible with ultra high strength concrete (C180/200). In Figure 99 the normal forces are displayed for the three different concrete types.

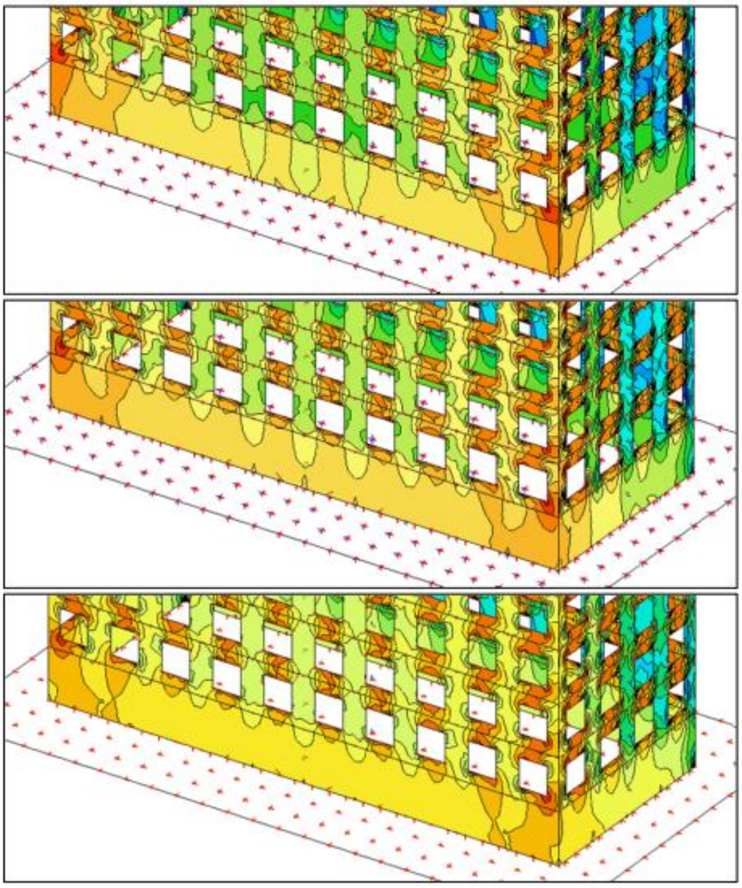


Figure 99 Shear lag: normal force in the OC model (top), HSC model (middle) and UHSC model (bottom) [Balbaid 2011]

In Figure 100 the normal force is compared to the average normal force. With the ordinary concrete (C35/45) the normal force can increase up to 50% in the corners. This is enormous and it should be taken into account during the design. Also the 7% reduction in section 1 and 5 is visible. It is questionable if the more expensive mixture is economically attractive when there is only a 7% reduction of shear lag.

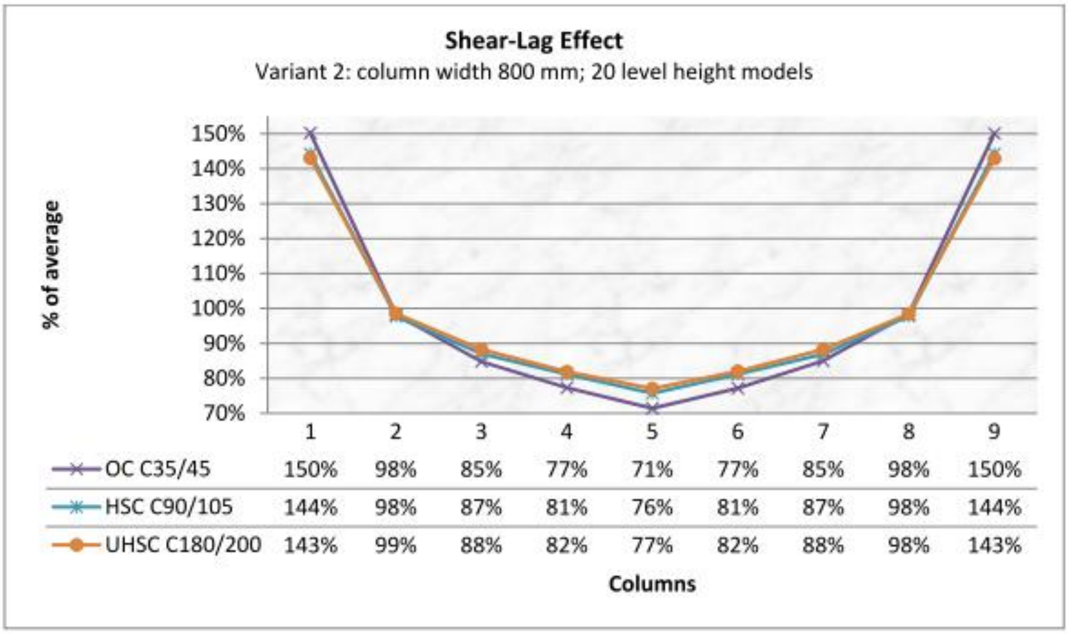


Figure 100 Shear lag in variant 2 [Balbaid 2011]

Several studies have been done to determine the shear lag and effective width. Prof.ir. A.K.H. Kwan is one of the researchers who wrote a paper about shear lag in shear cores and walls. He used a parametric finite element analysis to study the shear lag phenomenon. Unlike previous studies that neglected shear lag in the webs, many layers of elements are used for both the webs and flanges so that shear lag in the webs can also be taken into account. Based on research observations, the following empirical formulas for estimating the shear lag effects were developed for practical applications.

Table 19 Empirical formulas for shear lag coefficients at fixed end [Kwan 1996]

Load case	Shear lag coefficient α	Shear lag coefficient β
Point load at top	$\alpha = \dfrac{1{,}50}{1{,}00 + 0{,}76 * (H/a)}$	$\beta = \dfrac{1{,}25}{1{,}00 + 0{,}37 * (H/b)}$
Uniformly distributed load	$\alpha = \dfrac{1{,}59}{1{,}00 + 0{,}54 * (H/a)}$	$\beta = \dfrac{1{,}31}{1{,}00 + 0{,}24 * (H/b)}$
Triangularly distributed load (wind load)	$\alpha = \dfrac{1{,}56}{1{,}00 + 0{,}62 * (H/a)}$	$\beta = \dfrac{1{,}29}{1{,}00 + 0{,}28 * (H/b)}$

The maximum bending stress can be obtained with:

$$\sigma_m = \frac{M * a}{I_w(1 - 0{,}57 * \alpha) + I_f(1 - 0{,}80 * \beta)}$$

Where I_w and I_f are given by:

$$I_w = (4/3)*t_w*a^3$$

$$I_f = 4*t_f*a^2*b$$

The definition of a and b is given in Figure 101.

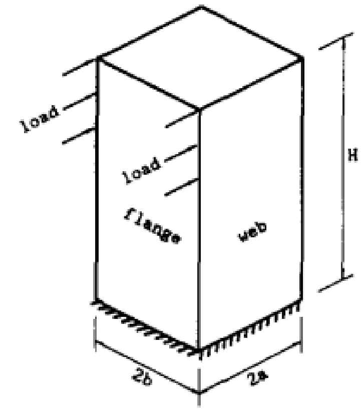


Figure 101 Core wall model [Kwan 1996]

After this research project, Kwan could make the following conclusions:

- The numerical results showed that the degree of shear lag in a cantilevered wall structure varies along the height and is generally greatest at the fixed end.
- The importance of shear lag increases in the order of point load case, triangularly distributed load case and uniformly distributed load case.
- The effects of shear lag in the web panels can be quite significant when the web panels are relatively short and wide, and hence, it should be prudent to also take into account the effects of any shear lag in the webs.
- Comparison with the finite-element results confirmed that the proposed formulas are sufficiently accurate for practical applications.

With the empirical formulas given by Kwan, it's possible to determine the maximum bending stress because of shear lag with a hand calculation. Modern Finite Element Method analysis software, such as Scia Engineer and AxisVM, are also capable of determining stress distributions because of shear lag.

10.4.6 Second order effects

In addition to the deflections caused by horizontal loads, the influence of vertical loads on the deformation should also be taken into account. Because of the first order wind deflection, the vertical weight will create an extra second order deflection and bending moment. This additional second order deflection creates a new eccentricity and a new second order deflection and bending moment is obtained. This continues until infinity (see Figure 102). Because the deformations rapidly decrease, only a first and second order calculation is made in practise.

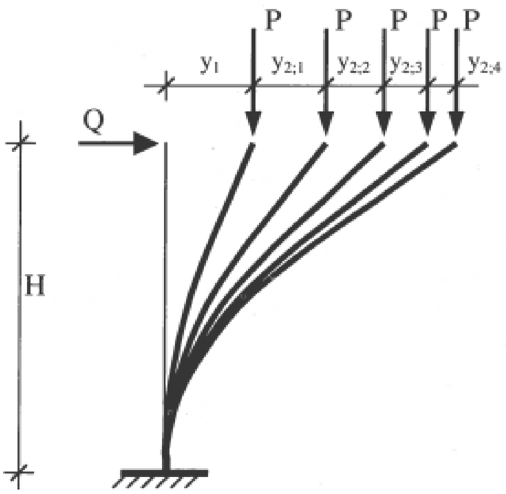


Figure 102 First order deflection and four steps of the second order [Hoenderkamp 2007]

Usually the second order effects are small (around 10%), but in extreme cases this may lead to structural failure. The effects can be calculated with a second order analysis and it's important to know in an early stage weather the structure is sensitive or not to these effects. A high rise tower can be schematized as seen in Figure 103.

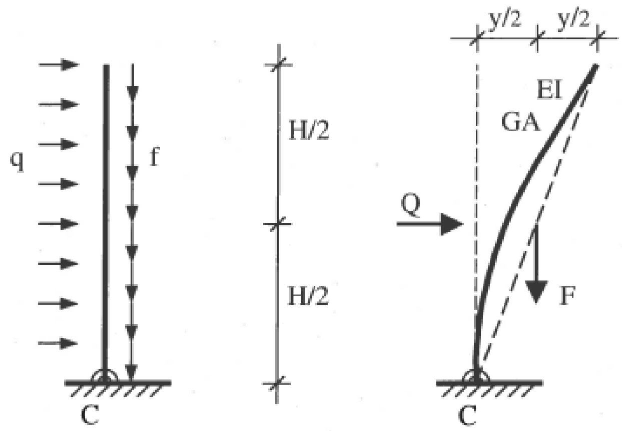


Figure 103 Schematization with uniformly distributed loads [Hoenderkamp 2007]

The total vertical load F acts at mid height (a linear distribution of the load is assumed). This schematization slightly underestimates the real behaviour, because the distributed load at the top creates a larger moment that the load at the base. The maximum expected error is about 3%. The total horizontal load also acts at mid height. An instable situation will occur when:

$$Q * \frac{H}{2} = F_{crit} * \frac{y_1}{2}$$

The critical load is then:

$$\frac{1}{F_{crit}} = \frac{y_f + y_b + y_a}{Q * H}$$

in which:

$$y_f = \frac{QH^2}{2C}$$

$$y_b = \frac{QH^3}{8EI}$$

$$y_a = \frac{QH}{2GA}$$

The three displacements are shown in Figure 104.

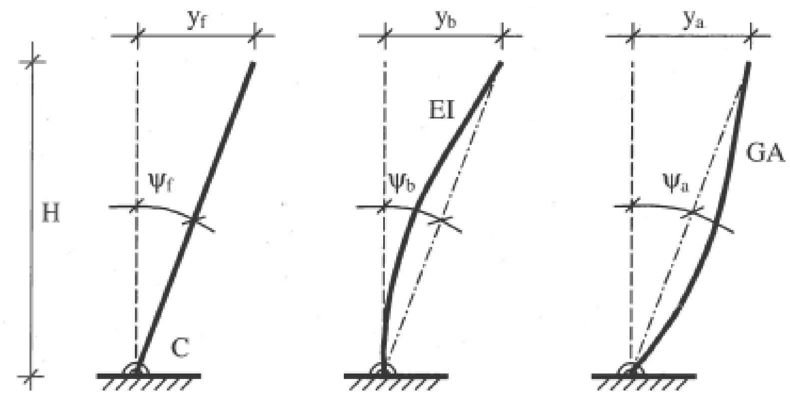


Figure 104 Deformations due to rotation, bending and racking shear [Hoenderkamp 2007]

After the deformations are substituted in the critical load, the following formula is obtained:

$$\frac{1}{F_{crit}} = \frac{H}{2C} + \frac{H^2}{8EI} + \frac{1}{2GA}$$

in which:
F_{crit} is the critical load which is used to calculate the second order effects,
H is the building height,
C is the rotation stiffness of the foundation,
EI is the bending stiffness of the building,
GA is the shear stiffness of the building.

A first estimation of the second order effects can be made by using the values calculated by Zonneveld ingenieurs:

I_{zz}=3.05*10^3m^4, E_d=2.17*10^7kN/m^2, C=8.52*10^9kNm/rad, $Q_{g,\varphi,k}$=804805.16kN and $Q_{q,\varphi,k}$=43204.72kN. The racking shear deflection is not calculated by Zonneveld, and this can be calculated with the following formula:

$$G = \frac{E}{2*(1+\upsilon)} = \frac{2.17*10^7}{2*(1+0.2)} = 9.04*10^6\ [\mathrm{N/mm^2}]$$

A=64,08m^2 (openings not included)

$$\frac{1}{F_{crit}} = \frac{200}{2*8.52*10^9} + \frac{200^2}{8*2.17*10^7*3.05*10^3} + \frac{1}{2*9{,}04*10^6*6{,}41*10^1} = 8.8*10^{-8}\ [\mathrm{kN^{-1}}]$$

This results in F_{crit}=11337417.4kN. Now the factor n can be calculated:

$$n_k = \frac{F_{crit}}{F_{total,k}} = \frac{11337417.4}{804805.16 + 43204.72} = 13.37$$

The magnification factor for Serviceability Limit State becomes n/(n-1)=13.37/12.37=1.08. The factor n for Ultimate Limit State can be calculated with:

$$n_d = \frac{F_{crit}}{F_{total,d}} = \frac{11337417.4}{1.3 * 804805.16 + 1.65 * 43204.72} = 10.1$$

The magnification factor for Ultimate Limit State becomes n/(n-1)=10.1/9.1=1.11.

The magnification factor for SLS is not larger than 1.1, but in ULS, the magnification factor is slightly larger. Since this exceedence is only marginal, it may be concluded that the structure is not sensitive for second order effects. The values in the calculation were used from the 190m design. The final magnification factors will therefore be smaller, because the stiffness will increase in a 200m design. Zonneveld ingenieurs also calculated the magnification factors for the monolithic tower and this resulted in n_k=1.07 and n_d=1.09 (see report 2 of the literature study). They did not include racking shear deformation and this results in a 1% smaller magnification factor. Furthermore, the magnification factors for the ULS were less severe (1.2 and 1.5 instead of 1.3 and 1.65).

10.5 Element configuration

The elements of a prefabricated structure can be placed in several configurations. The traditional and masonry (also known as staggered) configuration have already been mentioned several times. But there are more possibilities. For example the 2D masonry configuration or the rotated masonry configuration. In the next sections, these variants will be elaborated. In section 0 Several structural properties are discussed, based on the research of van Keulen [Keulen 2012].

10.5.1 Traditional configuration

With the traditional configuration, the vertical joints are continuous over the entire height of the structure. This method is applied very often at low and mid rise structures since it requires less different elements. This configuration isn't applied very often at high rise structures because it requires a labour intensive vertical connection to reach a high stiffness. The traditional configuration is shown in Figure 105.

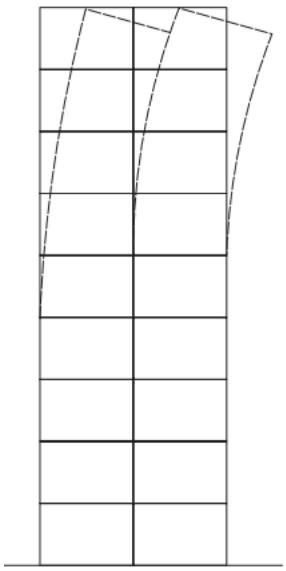


Figure 105 Traditional configuration [Falger 2004]

10.5.2 Masonry configuration

Since the completion of the Prinsenhof in 2005, the masonry configuration has been used several times (for example in the Eramus MC tower in Rotterdam and at the Stadskantoor in The Hague). By applying the masonry configuration, the vertical joint can be left open and this reduces time and costs. The stiffness of a masonry configuration is comparable to a traditional configuration with the stiffest wet connection (tooth connection). The masonry configuration is shown in Figure 106. Falger [Falger 2004] examined this configuration and concluded that it is most effective when the width of the element is two times larger than the height and when a minimal overlap of 0.25 times the length of the element is applied.

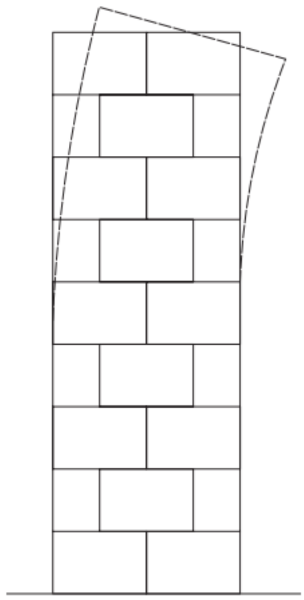


Figure 106 Masonry configuration [Falger 2004]

The Prinsenhof was not the first prefabricated structure to use the masonry configuration. The Egyptians constructed their pyramids 4500 years ago with prefabricated blocks weighing up to 2.5 ton [Wikipedia 2012]. By using this configuration they were able to construct the pyramid of Cheops with a height of 147m. This pyramid was for almost 4000 years the tallest structure in the world and it was beaten by the cathedral of Lincoln in the year 1311. The Egyptians have shown that using prefab can lead to extraordinary structures.

10.5.3 2D masonry configuration

By using a masonry configuration, the vertical joint is interrupted. The horizontal joint still continues from left to right. By using a 2D masonry configuration, the horizontal joint is also interrupted. An example is shown in Figure 107. When this configuration is examined in more detail, it can be concluded that the amount of continuous vertical joints increases compared to the normal masonry configuration. This will have a negative influence on the stiffness of the structure. Because long elements are now placed horizontally and vertically, the amount of different elements will increase. Combining this with the fact that only a quarter of the horizontal joint is interrupted explains why this technique has never been applied before.

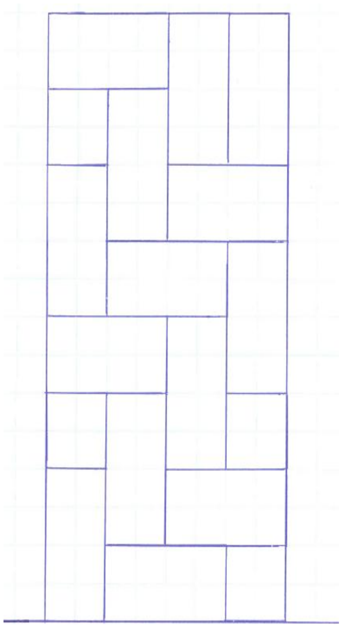


Figure 107 2D masonry configuration

10.5.4 Rotated masonry configuration

The rotated masonry configuration has no continuous horizontal joint and provides a larger horizontal shear capacity than the normal masonry configuration. As a result of the rotation, the vertical joints are now continuous. To maintain a high stiffness, a labour intensive vertical connection has to be applied. In most high rise structures the bending stiffness is more important than the horizontal shear capacity and the standard masonry configuration is preferred. Furthermore, the elements continue over two floors and the floor-wall connection becomes more complicated.

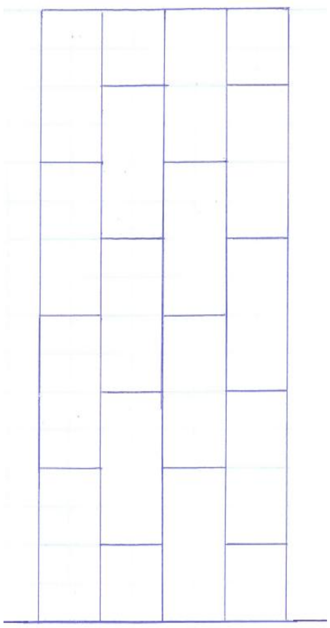


Figure 108 Rotated masonry configuration

10.5.5 Structural properties of the element configurations

In the previous sections several element configurations have been discussed. Van Keulen examined several layouts for his promotion research on the design, behaviour and construction of tall precast concrete structures. In Figure 109 the considered layouts are shown.

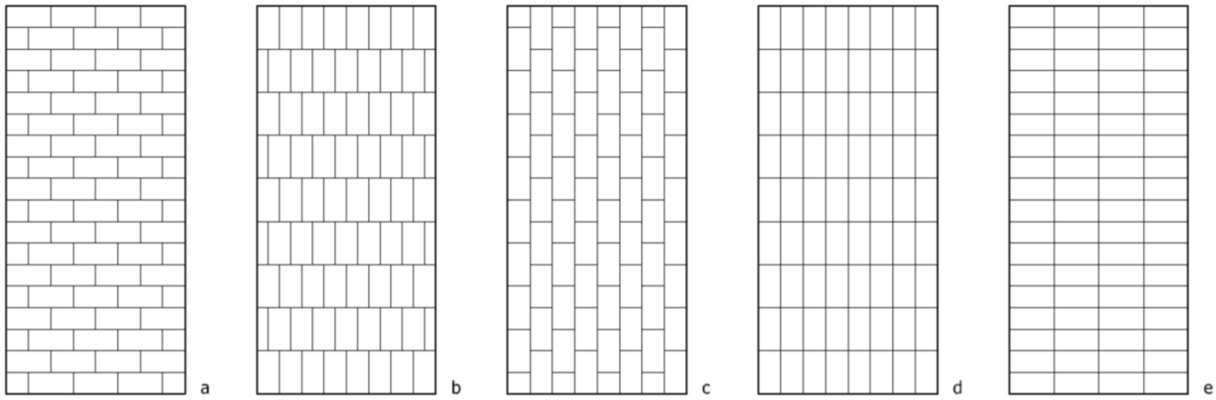


Figure 109 Different element configurations [Keulen 2012]

Figure 109a contains the masonry configuration with an open vertical joint. Figure 109b utilises a vertical masonry configuration, also with open vertical joint. Figure 109c until Figure 109e require a structural vertical connection. When the different models were analysed with a Finite Element Method (FEM) program, Figure 110 is obtained. In Figure 110 the deformations of the five models are depicted relative to a monolithic model. As a result of the lower stiffness of the precast models, an increase of deformation is plotted against the slenderness of the structure.

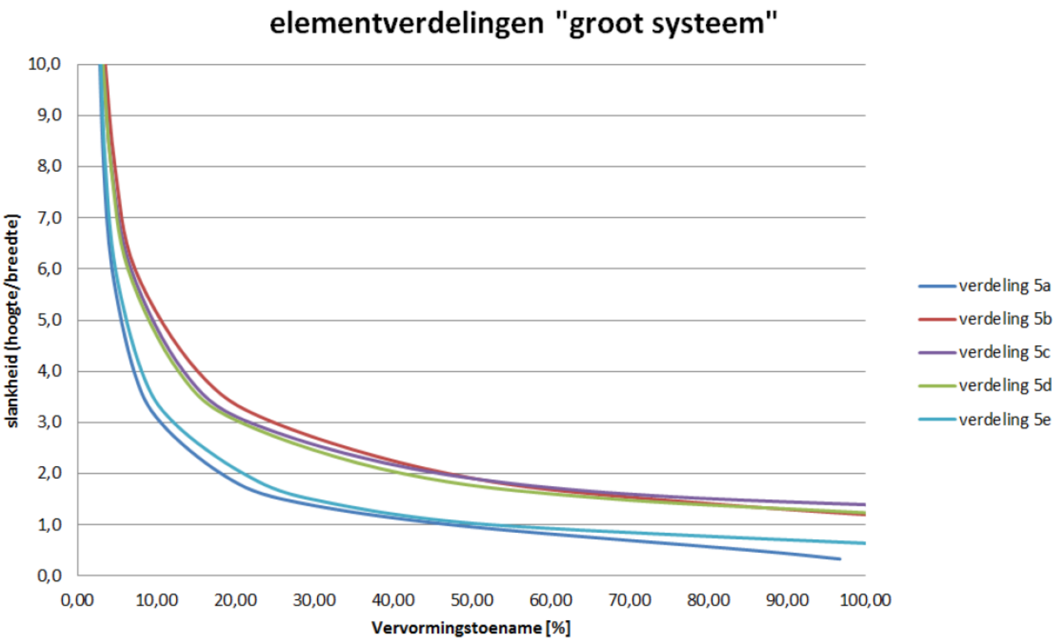


Figure 110 Results of different element configurations [Keulen 2012]

From Figure 110 it can be observed that Figure 109a provides better results than Figure 109b until Figure 109d. Therefore it can be concluded that a horizontal configuration is preferred. The relative good results of Figure 109e are remarkable. Apparently the low shear stiffness applied at the vertical connection provides enough resistance. When the size of the elements is varied, as shown in Figure 111, Figure 112 is obtained.

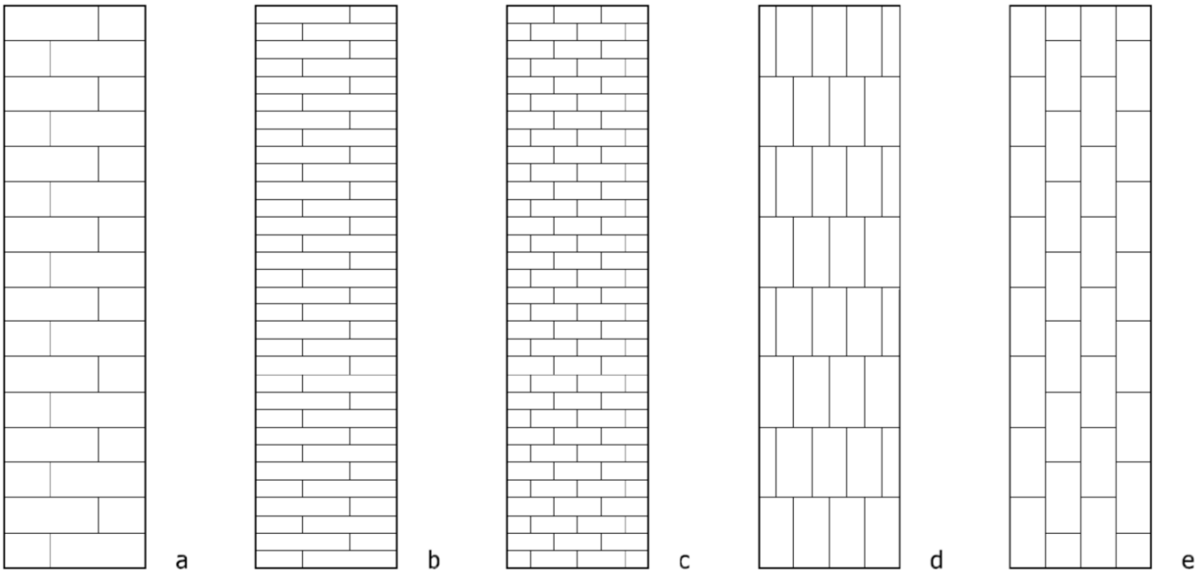


Figure 111 Different size configurations [Keulen 2012]

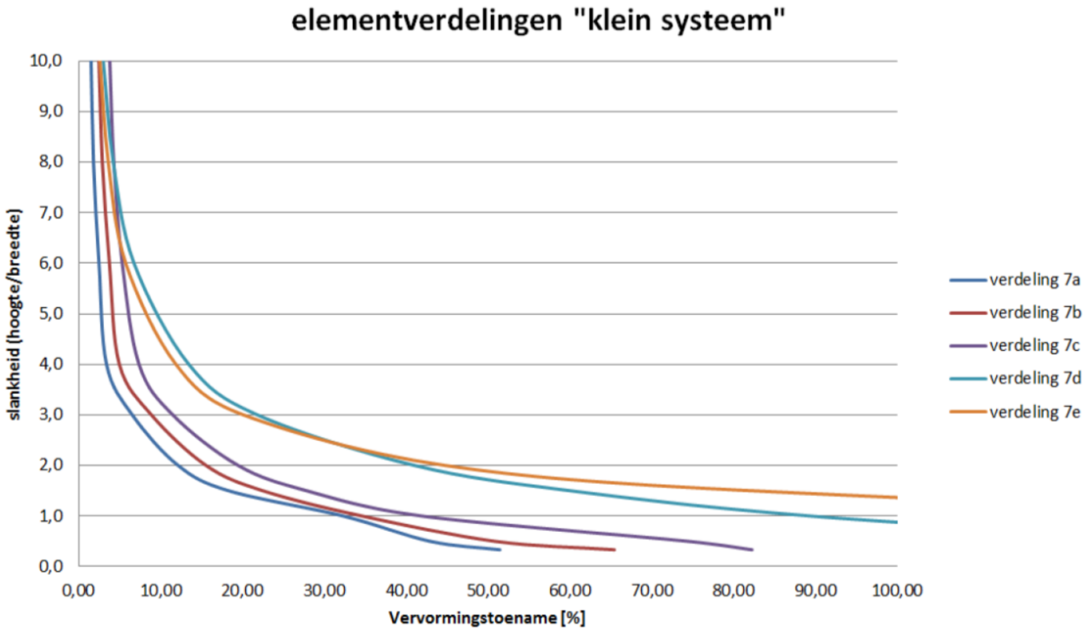


Figure 112 Results of different element sizes [Keulen 2012]

From Figure 112 it can be concluded that a few large elements provide better structural properties than many small elements. Nevertheless, the differences between Figure 111a and Figure 111c remains small. When the slenderness of the structure is known, Figure 109 until Figure 112 can be used to quickly determine the amount of additional deformation compared to a monolithic structure.

10.6 Conclusion

In chapter 10 many important aspects have been discussed for a prefabricated structure. In section 10.1 the elements that are required to construct a prefabricated structure are examined: walls, floors and columns. Section 10.2 describes the connections between these elements. When section 10.3 is also taken into account (structural behaviour), it can be concluded that the grouted starter bar connection and the masonry configuration are the best solutions for the horizontal and vertical connections. The Interlocking Halfway Connection is the stiffest vertical connection between perpendicular walls and for the floor the re-engineered steel tube connection contains the most benefits. Next, the response to lateral load is investigated (section 10.4). Shear force deformation is governing in the lower levels and shear lag will lead to locally increased forces. The lintel above the door opening is decisive for the stiffness of the structure and due to large forces it will probably be cracked. In section 10.5 several element configurations were studied. According to the research of van Keulen [Keulen 2012], a horizontal element configuration is preferred over a vertical configuration. The size of the elements also influence the structural properties: the wall elements should preferably be as large as possible.

11 Material properties

The development of High Strength Concrete (HSC) has evolved rapidly in the last few years. Due to many new researches, more knowledge is acquired on the properties and performance of this mixture. This also results in an improved performance of Ordinary Concrete (OC) because they have many factors in common. Today, even mixtures with a compression strength beyond 200N/mm^2 are developed.

Concrete mixtures can be divided into five categories [Walraven 2006]:

- Ordinary Concrete: up to C53/65,
- High Strength Concrete: C53/65 to C90/105,
- Very High Strength Concrete: C90/105 to C150/170,
- Ultra High Strength Concrete: C150/170 to C200/230,
- Super High Strength Concrete: from C200/230.

To create high strength concrete, the mixture of OC has to be changed:

- reduce the water cement ratio,
- increase the packing density of the powder content by adding puzzolanes and silica fume,
- improve the homogeneity by applying smaller aggregates,
- increase ductility by adding steel fibres.

In Table 20 various concrete mixtures are summarized. A large increase of Young's Modulus and compression strength is visible.

Table 20 Concrete mixtures with their properties [Balbaid 2011]

Mixture	C35/45	C70/85	C100/115	C200	BSI	Secutec S9	Ductal
Cement (kg/m^3)	360	475		1075	1100	-	710
Binder (kg/m^3)					-	1100	-
Silica fume (kg/m^3)	-	25		165	165	-	230
Quartz powder (kg/m^3)					-	-	210
Sand (kg/m^3)	790	785		1030	1050	-	1020
Bauxite 0-1mm (kg/m^3)					-	685	-
Bauxite 5-8mm (kg/m^3)					-	625	-
Gravel (kg/m^3)	1110	960		-	-	-	-
Steel fibers (kg/m^3)				235	235	200	40-160
Plasticizer (kg/m^3)	0.5	4.6		40	40	-	13
Water (kg/m^3)	145	150		200	200	200	140
Mass density (kg/m^3)	2405	2400		2810	2800	2850	2500
Water-cement factor (-)	0.4	0.3		0.16	0.15	0.18	0.15
Compression strength f_{ck} (N/mm^2)	35	70	100	200	180	183	200
Mean tensile strength f_{ctm} (N/mm^2)	3.23	4.6	5.2	7.8	16.8	9.5	21.7
Young's Modulus E_c (N/mm^2)	33500	39300	48600	55000	65000	64000	50000

Applying a higher strength concrete mixture requires more care and attention. During the production the mixture has to be carefully monitored, because mistakes in this process have a tremendous effect on the final performance of the concrete. Adding smaller aggregates, fillers and steel fibres in combination with a different mixture method also result in a lower production capacity of the concrete factory. Combine this with different pouring requirements and it's clear why high strength concrete needs more care and attention.

Balbaid researched the application of higher strength concrete in tubular structures for his master thesis. During his literature study he investigated the advantages and disadvantages of HSC and UHSC:

Advantages:

- Improved material strength and properties.
- When applying UHSC with fibres no additional steel reinforcement is required (in certain situations). When needed, pre-stressing still needs to be applied. The lack of additional reinforcement reduces production costs.
- A high density comes with a high durability. Consequently, concrete covering can be reduced or even neglected if no reinforcement bars are applied, reducing overall thickness.
- A high strength is achieved very fast after pouring, creating a higher build speed.
- Higher pre stressing can be applied.

Disadvantages:

- Higher strength concrete has a larger magnitude of autogenous shrinkage compared to OC. Most of the shrinkage occurs in the first few days after pouring.
- The hydration process in high strength concrete is very fast, resulting in a higher temperature production. This can result in cracks in the concrete.
- UHSC without fibres acts very brittle. Adding the fibres solves this problem.
- Production capacity at the concrete factory is reduced.
- Higher strength concrete is more expensive than OC.
- Until now there is no standardization when it comes to the strength classes of UHSC.

After the literature study, he designed and calculated two variants of a tubular office building. The window size was the only difference between the two variants. In variant 1 (see Figure 113), the windows are 2x2m^2 and in variant 2, they are 2.8x2m^2 (wxh). Both variants were then calculated with OC, HSC and UHSC. One of his research goals was to find out how many extra floors could be added to the building if the quality of the concrete mixture was improved and the amount of concrete per floor remained the same.

Figure 113 Rendered model of the building's structure (variant 1) [Balbaid 2011]

Al six models complied with the Eurocodes, but in the HSC and UHSC models there was room for optimization. To reduce the unused material he proposed three solutions:

- Reduce the beam height in the upper part of the structure.
- Replace the corner columns by steel columns.
- Apply a hybrid structure (lower section in HSC and the upper section in OC).

The next step was to compare the six models based on their price. Unfortunately, the optimizations mentioned above were not taken into account and the original six models were compared. To make a price comparison possible, first the price of each mixture had to be determined. This was done by consulting Dr.ir. Grünewald (TU Delft and Hurks Beton).

Ordinary Concrete (C35/45) is one of the most applied mixtures. The strength is sufficient for most regular applications and it contains large aggregates in combination with a low amount of cement. The price of this mixture is approximately €100/m^3.

High Strength Concrete (C90/105) uses finer aggregates and more cement. Synthetic fibres are added to improve the fire resistance of the concrete. Because of these factors, the price increases compared to OC. On average, the price is around €200-250/m^3. When steel fibres are added, the price increases to €400/m^3.

Ultra High Strength Concrete (C180/200) uses even more fine aggregates and cement paste than HSC. Also silica fume and bauxite are added to increase the packing density. The large amount of steel fibres increases the price even more. This results in a mixture price of around €1200/m^3.

The difference in strength, Young's Modulus and price is shown in Figure 114.

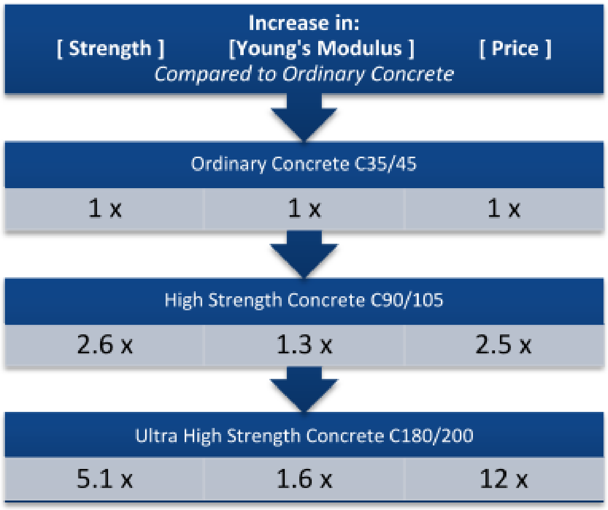


Figure 114 The difference in strength, Young's Modulus and price [Balbaid 2011]

With these unit prices it was possible to calculate to total price of each model. Prof.ir. van der Horst (TU Delft and Koninklijke BAM Groep nv) was consulted to obtain the distribution of the total building cost as seen in Figure 115.

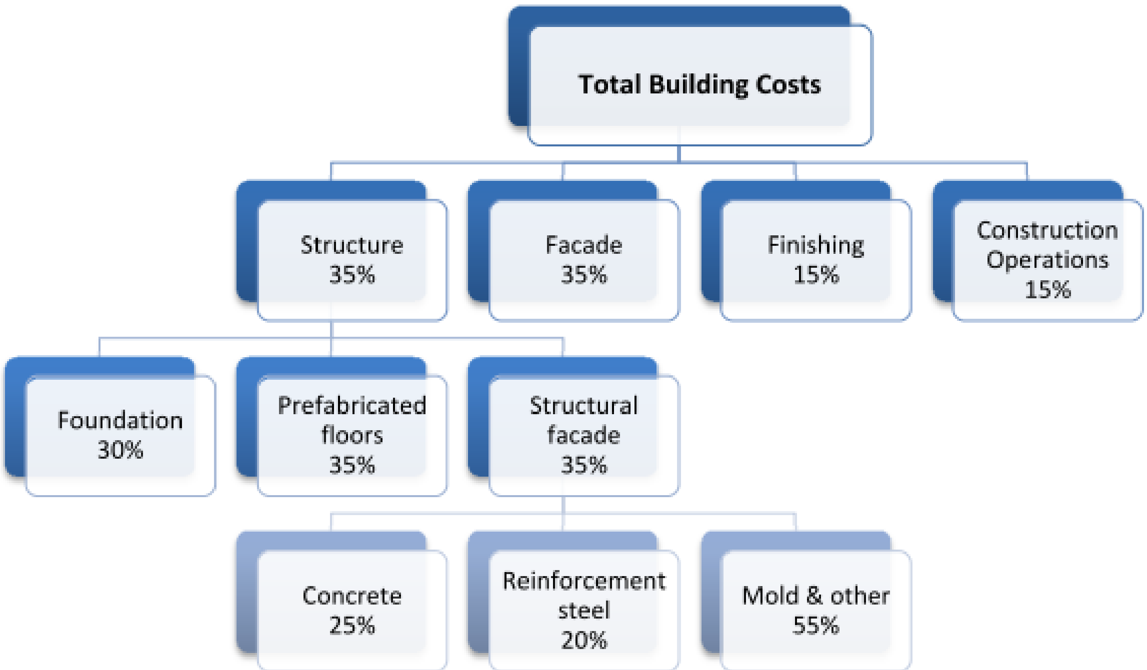


Figure 115 Total building costs [Balbaid 2011]

The costs were calculated per floor and only the structural costs that differ per model were taken into account. The result of this analysis is depicted in Figure 117.

Figure 116 Maximum building heights with three different concrete qualities [Balbaid 2011]

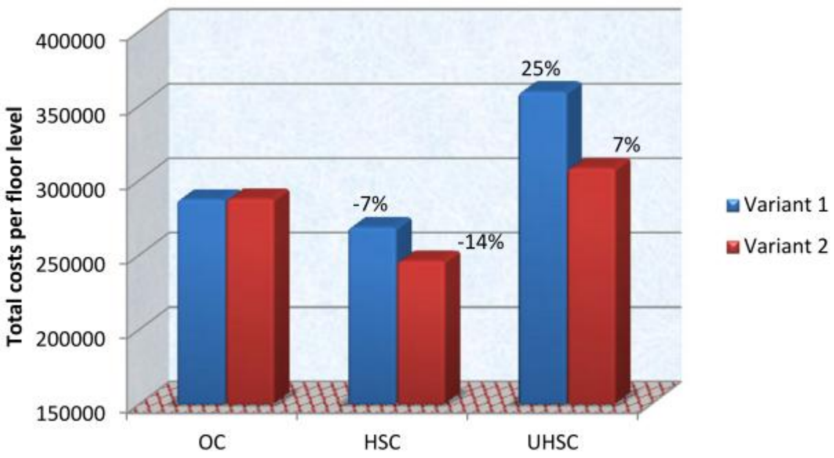


Figure 117 Total costs per floor level comparison [Balbaid 2011]

When OC is applied, it's possible to construct an office building with 30 floors for variant 1 and 20 floors for variant 2 (because the window openings are large, the maximum building height decreases). By applying HSC instead of OC, it's possible to construct 35 floors for variant 1 and 25 for variant 2. In other words: by applying HSC, the maximum amount of floors increases with 5 while the amount of concrete per floor remains equal. Although the HSC mixture is more expensive, the 5 extra floors result in a lower price per floor: 7% reduction for variant 1 and 14% for variant 2. The difference of 7% between these two HSC variants is because variant 1 has still some capacity left and variant two is near its maximum.
The structural concrete façade only takes 12% of the total building costs and the price of the mixture plays a small role. Nevertheless, the UHSC mixture is up to 5 times more expensive than OC and the additional 10 floors for variant 1 and 2 do not compensate the increased costs.

The application of higher strength concrete becomes more and more custom. The use of Ultra High Strength Concrete is at the moment reserved for prestige projects. A price reduction is necessary before it's widely introduced in the building industry. Applying High Strength concrete in building structures is more common and Bablaid's research shows that it provides a better performing structure and in some cases, a reduction in costs. The increased Young's Modulus is quite interesting for prefabricated structures because the stiffness is reduced by the joints. These two aspects may counteract each other without a large increase of costs.

12 Progressive collapse

Progressive collapse of a structure starts when a part of the structure isn't able to bear the load and fails. This increases the load on the surrounding structure and when this load cannot be endured, the surrounding structure fails: progressive collapse is now a fact.

The first known structure that collapsed because of progressive collapse is the St. Marks's Campanile in Italy [Wikipedia 2012]. On July 14, 1902 the 98,6m high tower collapsed because one of the load bearing walls failed. The progressive collapse of multiple floors of the Ronan Point building in London was a turning point for the engineering practice. A gas explosion in one compartment raised the floor of the apartment above and pushed the facade and load bearing walls outwards. The elements were easily blown away because the connections relied only on gravity for the integrity. The upper floor was not designed to bear the load without the supporting walls and collapsed. This mechanism and the momentum of the collapsed material (dynamic impact) resulted in a partly progressive collapse of the building (see Figure 118).

Figure 118 Progressive collapse at the Ronan Point tower [Wikipedia 2012]

Two examples of progressive collapse in the Netherlands are the balconies at the Patio Sevilla in Maastricht and the steel roof of the FC Twente football stadium in Enschede.

In Section 13.1, progressive collapse will be explained in more detail and section 13.2 will continue with possible solutions to prevent progressive collapse. Section 13.3 elaborates on the execution of a risk analysis for CC3 structures. This chapter will end with a risk analysis for the Zalmhaven tower (section 13.4) and a conclusion (section 13.5).

To construct this chapter, several articles have been used:

- NEN-EN 1991-1-7,
- NEN 6702,
- Stufib repport 8: Constructieve samenhang van bouwconstructies,
- NTA Higrise – part 3: Structural safety,
- IABSE report: Additional requirements for High Rise buildings in The Netherlands,
- graduation report of R.C. Siersma: Progressive Collapse of Building Structures.

12.1 The causes of progressive collapse

Progressive collapse is caused by extraordinary loads due to (un)foreseen situations, such as explosions, fires, collisions, earthquakes, hurricanes, design mistakes or malicious actions (for example: vandalism or a terrorist attack). It's not economically attractive to design all the elements in such a way that they will withstand these extraordinary forces and it's like that some element will fail. A solution to prevent progressive collapse is to apply a second load bearing system. In the event of an extraordinary situation, this system will take over and transport the loads via a different path to the foundation. Figure 119 shows two examples of progressive collapse.

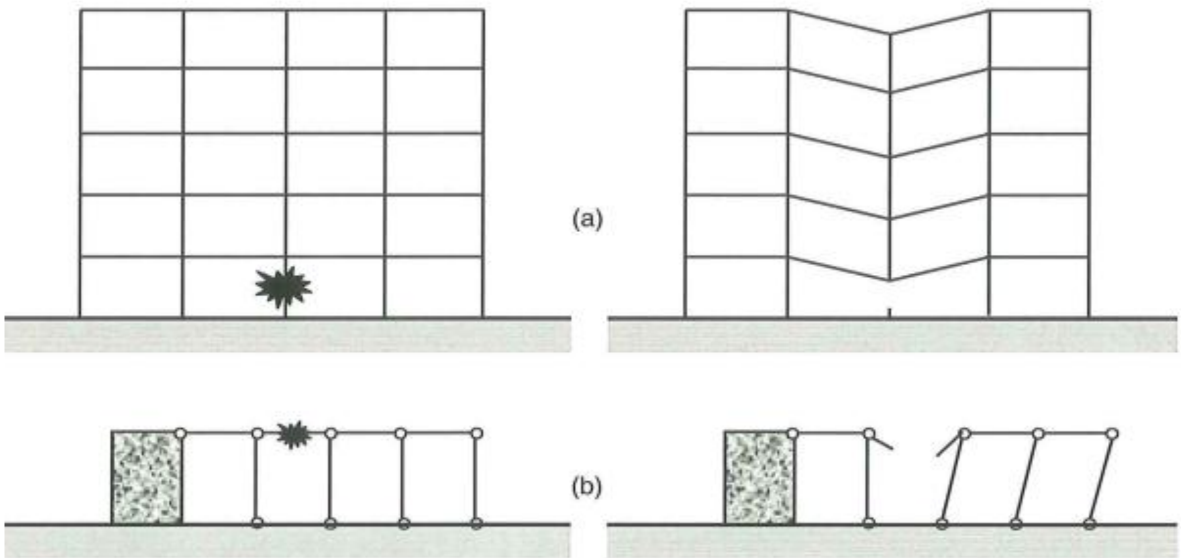


Figure 119 Progressive collapse [Hobbelman 1986]

In structure (a) one of the centre columns collapses and the span of the beams increases with a factor of two. The beams of the first floor are not designed to carry the floor load without an intermediate column and they start to deflect. Due to this deflection, a large positive bending moment will occur at the supports of all the beams in this bay and a tension force will be generated in the centre of the extended beam (cable action). As a result, this bay will probably collapse and it's likely that adjacent bays will be damaged or even might collapse as well.

In structure (b) a braced frame is shown. When a beam fails, the right part of the structure becomes unbraced. The connections are not designed to be moment resistant and this part of the structure will collapse.

As stated before it's unwanted or impossible to prevent the collapse of all the elements. The focus is more on progressive collapse: giving the occupants enough time to escape the building. This is similar to fire safety design, were the occupants have 30 to 120 minutes to escape the building during a fire. Clients (or the insurance company) often provide higher specifications than the codes because the codes do not take the economic considerations of a project into account. In fire safety design it's very common to apply sprinklers, manual fire extinguishers and less combustible material. For the same reasons a client may specify a higher safety level before an element fails or that actions are undertaken to prevent unforeseen situations (collision protection or an electric furnace instead of a gas furnace for example).

For the previous two structures, several actions can be taken to prevent progressive collapse. In structure (a) more reinforcement can be applied at the connections. When the column collapses, the beams will behave like cables and prevent a progressive collapse. It should be noted that large deformations will occur due to the cable action. Structure (b) should get second bracing structure to prevent progressive collapse.

12.2 Preventing progressive collapse

Preventing progressive collapse is in itself a risk analysis and the risk is composed out of a probability and a consequence. One could try to prevent the risk or the consequences can be limited. Impact protection for columns or non combustible materials are examples that (try to) prevent the risk. Applying extra strength or a second load bearing system will limit the consequence of the risk. According to NEN-EN 1991-1-7 and NTA Hoogbouw (03-B) special design situations as a consequence of the collapse of a structural part by an undefined cause has to be taken into account. In the Eurocode NEN-EN 1991-1-7 two different strategies are used for known and unknown extraordinary loads. These strategies are clarified with Figure 120.

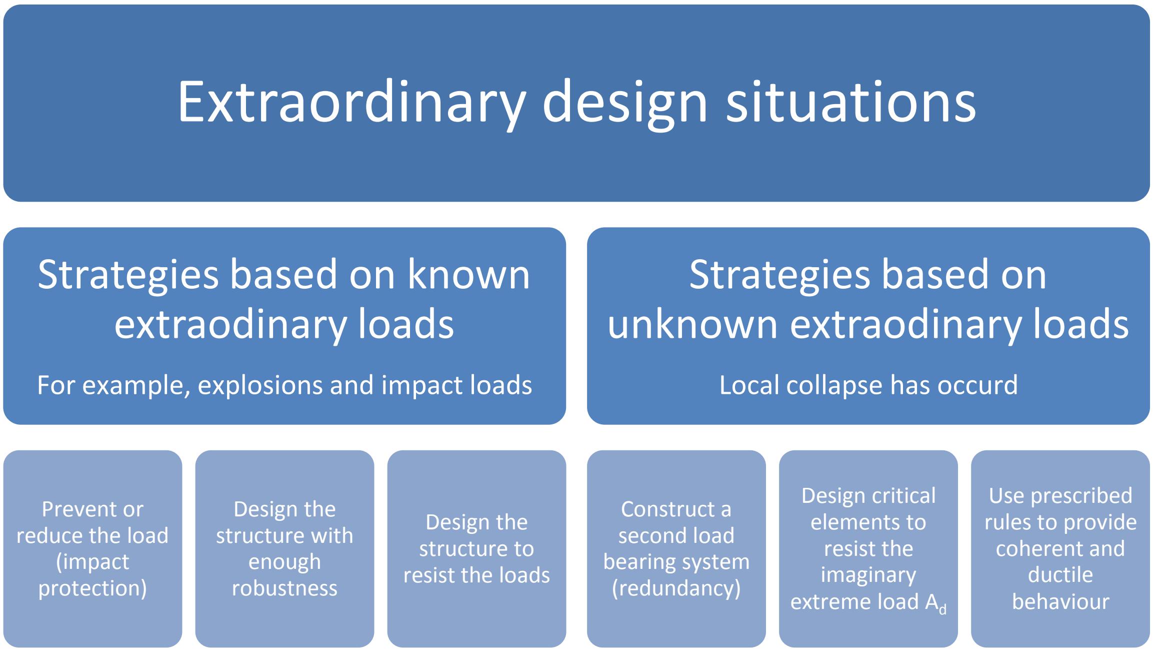


Figure 120 Strategies for extraordinary design situations

Depending on the consequence class, extraordinary design situations may be considered as following:

- CC1: a specific consideration for extraordinary loads is not necessary, except to ensure compliance with the applicable rules for robustness and stability as specified in NEN-EN 1990 until NEN-EN 1999,
- CC2a: in addition to the recommended strategies for CC1, horizontal tension ties or effective anchorage of raised floors connected to the walls have to be applied as defined in annex A.5.1 and annex A.5.2 of NEN-EN 1991-1-7 for structures with columns and load bearing walls,
- CC2b: in addition to the recommended strategies for CC1:

- o horizontal tension ties as defined in annex A.5.1 and annex A.5.2 of NEN-EN 1991-1-7 in combination with vertical tension ties as defined in annex A.6 of NEN-EN 1991-1-7 have to be applied in all the load bearing columns and walls, or as alternative,
  - o the building should be verified that when an imaginary removal of every load bearing column and every beam that supports a column, or any part of a load bearing wall as defined in annex A.7 of NEN-EN 1991-1-7 (in each case one element at a time), the stability of the building is maintained and or local damages do not exceed a certain limit.

  Where the notional removal of columns and sections of walls would result in an extent of damage in excess of the agreed limit, or another specified limit, then elements or wall section should be designed as a "key element" (see annex A.8 of NEN-EN 1991-1-7),
- CC3: a systematic risk analysis of the building has to be executed with both expected and unexpected threats (see Annex B of NEN-EN 1991-1-7).

One would expect CC3 structures to satisfy CC2b requirements (just like CC2a and CC2b have to satisfy the requirements of CC1), regardless of supplementary measures that might arise from the risk analysis. But the Risk analysis may indicate that other provisions, as described in the codes, are adequate without the CC2b requirements.

To which consequence class the structure belongs depends on its function and size. The height is often leading for the size and the different consequence classes are described in Figure 121.

Consequence class	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of $1^1/_2$ times the building height.
2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1 000 m^2 floor area in each storey. Single storey educational buildings All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey.
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators Buildings containing hazardous substances and /or processes

Figure 121 Classification of the consequence class [NEN-EN 1991-1-7]

In this section seven methods to prevent progressive collapse will be discussed. To provide a clear oversight, the same division for known and unknown extraordinary loads will be used as in Figure 120 (a fourth method is added to the strategy for unknown extraordinary loads).

12.2.1 Known extraordinary loads

Known extraordinary loads are explosions, collisions, fires, extreme weather and earthquakes. There are two possible solutions for known extraordinary loads. The first solution is to reduce or prevent the effect of the extraordinary load on the main load bearing structure (see section 13.2.1.1). The second solution is to make the structure resist the effect of the extraordinary load (see section 13.2.1.2 and 13.2.1.3). A problem with both techniques is that the design team should recognize all the extraordinary loads in an early project stage. This is quite difficult because the amount of information is relatively low in this phase. The problem also originates in the Eurocode because it doesn't specify all the known extraordinary loads and for which buildings they apply. It only provides a (short) list of which aspects should be taken into consideration when forming the list with known extraordinary loads (see NEN-EN 1991-1-7, 3.2 (1)):

- the taken measures to prevent an extraordinary load or the measures that are used to reduce the severity,
- the probability that a known extraordinary load occurs,
- the consequence of the collapse due to the known extraordinary load,
- the perception of the public,
- the acceptable risk level.

The following known extraordinary loads are specified in the Eurocode: fires, collisions and explosions. The NEN 6702 covers earthquakes (very shortly, see Figure 14 of chapter 9 of NEN 6702), fires, explosions, collisions and water pressure. In the following three sections, solutions will be discussed to reduce/prevent or resist known extraordinary loads.

12.2.1.1 Prevent or reduce known extraordinary loads

The simplest solution is to prevent the extraordinary load. Applying impact protection or non combustible materials are a good example. To apply impact protection the force should first be calculated. The Eurocode provides calculation methods and this will be explained in more detail in section 12.2.1.3.

Unfortunately, the Eurocode doesn't specify which actions could or should be taken to prevent or limit the extraordinary load if the force is known. It's up to the engineer to design the impact protection system or the amount of non combustible materials. In the case of an explosion design, assistance by a specialist is recommended by the Eurocode (NEN-EN 1991-1-7 5.3 (8)). Because the lack of guidance in the Eurocode to prevent or limit extraordinary loads, this should also be recommended for the other known extraordinary loads.

Internal explosions are a small exception on the lack of guidance. Four solutions are provided that might limit the consequence of an explosion [NEN-EN 1991-1-7]:

- "use venting panels with defined venting pressures,
- separate adjacent sections of the structure that contain explosive materials,
- limit the area of structures that are exposed to explosion risks,
- provide specific protective measures between adjacent structures exposed to explosions to avoid propagation of pressures."

12.2.1.2 Design the structure with enough robustness

Robustness is described as following: the ability of a structure to withstand events like fire, explosions, collisions or effects of human errors without being damaged to an extend not proportional to the original cause. One or multiple of the following three methods can be used to provide the structure with enough robustness (see section 3.2 (3) C from NEN-EN 1991-1-7):

1. Design certain parts of the structure, of which the stability is dependant, as key elements in order to increase the likelihood that the structure is preserved after an extraordinary event.
2. Design structural elements and use materials with sufficient ductility to absorb significant deformation energy without fracturing.
3. Incorporate sufficient redundancy in the structure in order to transfer the loads via an alternative load bearing system in case of an extraordinary event.

Method 1 and 3 are already part of the unknown extraordinary load strategy and they are explained in section 12.2.2.1 and 12.2.2.2. Method 2 contains a new aspect: ductility. Ductility, also known as formability, is the extent to which a material permits plastic deformation under a tensile stress. When a compressive stress is applied, it's called malleability. By applying a certain amount of reinforcement in and between concrete elements, the structure will behave ductile under extraordinary loads. This reinforcement ratio should not be too small (failure moment will approximate the cracking moment) or too large (the concrete fails before the steel has yielded) because this will lead to brittle failure. The Eurocode provides values for the minimum and maximum reinforcement area in the tension zone (see NEN-EN 1992-1-1/NB 9.2.1.1):

The value for $A_{s,min}$ should be equal to the smallest value of:

$$A_{s,min1} = 0.26 * \frac{f_{ctm}}{f_{y,k}} * b_t * d \geq 0.0013 * b_t * d$$

in which:
f_{ctm} is the average value for the axial tension strength of concrete,
$f_{y,k}$ is the characteristic yield level of reinforcement steel,
b_t is the average width in the tension zone,
d is the effective height of the cross section.

or

$$A_{s,min2} = 1.25 * A_{s,ULS}$$

in which:
$A_{s,ULS}$ is the required reinforcement area in the Ultimate Limit State.

The value for $A_{s,max}$ should not be larger than $0.04A_c$, where A_c is the concrete area. It should be noted that these values are applicable for concrete beams under bending. The minimum and maximum values for walls are not given. A wall can be seen as a vertical floor, but values for floors are also not specified. A third option is to divide the wall into small beam sections. Since a wall is normally loaded with compression and only by a small out of plane bending moment, it's likely that minimal reinforcement ratios are applied.

Besides the reinforcement ratio, the reinforcement itself should also meet several requirements:

$$k = \left(\frac{f_y}{f_y}\right)_k$$

and

$$\varepsilon_{uk} = j$$

Values for k and j are given in annex C of NEN-EN 1992-1-1. For example, bars and oriented wires of class A: k≥1.05 and ε_{uk}≥2.5%. These values are also shown in Figure 122.

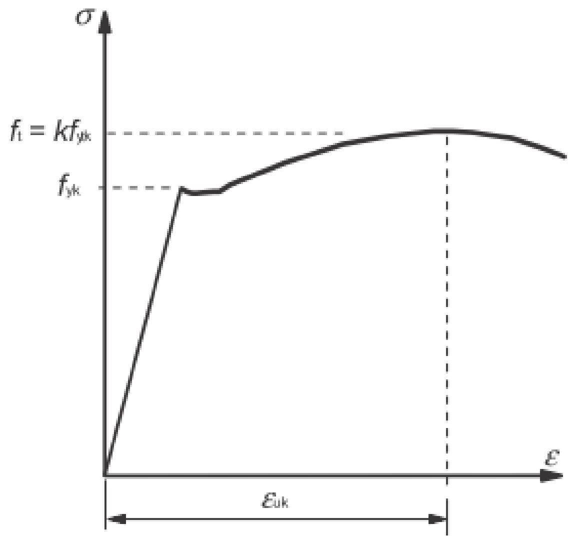


Figure 122 Ductility properties of hot rolled steel [NEN-EN 1992-1-1]

Aside from the reinforcement ratio and the steel requirements, there are also requirements for the concrete. Already mentioned before: the concrete will fail first if the reinforcement ratio is too high. This failure occurs when the strain limit (stuik in Dutch) is exceeded: ε_{cu3}. For concrete classes C12/15 until C50/60, the concrete shatters at 3.5% (see Figure 123). Higher strength classes are more brittle and failure occurs at a lower level. For C90/105 this is: ε_{cu3}=2.6%.

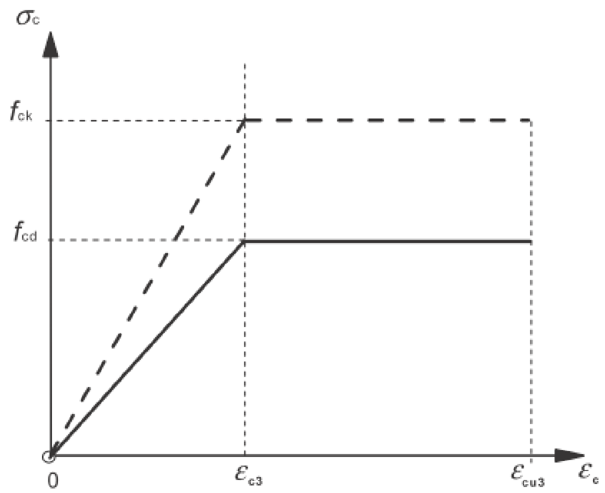


Figure 123 Bi-linear stress-strain relationship of concrete [NEN-EN 1992-1-1]

12.2.1.3 Design the structure to resist the load

In contrast to section 12.2.1.1 (prevent or reduce the load), the Eurocode does provide rules to design the structure for a certain extraordinary load. In NEN-EN 1991-1-7 methods can be found for collisions and explosions. Fire design methods can be found in NEN-EN 1991-2 and NEN-EN 1992-1-2. The reliability of the elements is guaranteed by applying safety factors.

Collisions
Impact loads due to the following objects are defined in NEN-EN 1991-1-7 (4.1 (1)):

- road vehicles,
- forklifts,
- trains,
- ships,
- the rough landing of a helicopter.

Buildings should take impact loads into consideration if:

- the building is used as a parking garage,
- forklifts are used inside,
- the building is situated near a road or railway,
- a helipad is present on the roof of the building.

The impact loads have to be determined with a dynamic calculation or it should be represented by an equivalent static force. This simplified static force model may be used to check the static equilibrium, strength and deformation of the structure. How this calculation should be executed is explained in annex C of NEN-EN 1991-1-7.

The calculation of dynamic loads goes beyond the scope of this thesis and is not explained in more detail. Because parking garages are often found in the basement of a high rise building, the indicative equivalent static loads are shown in Figure 124.

Category of traffic	Force F_{dx} [a] [kN]	Force F_{dy} [a] [kN]
Motorways and country national and main roads	1000	500
Country roads in rural area	750	375
Roads in urban area	500	250
Courtyards and parking garages with access to:		
- Cars	50	25
- Lorries [b]	150	75

[a] x = direction of normal travel, y = perpendicular to the direction of normal travel.

[b] The term "lorry" refers to vehicles with maximum gross weight greater than 3,5 tonnes.

Figure 124 Indicative calculation values for equivalent statically forces due to a collision [NEN-EN 1991-1-7]

More information about the calculation of forklift, ship, train and helicopter impacts can be found in chapter 4 of NEN-EN 1991-1-7.

Explosions

Section 5.1 (1)P of NEN-EN 1991-1-7 states: "Explosions shall be taken into account in the design of all parts of the building and other civil engineering works where gas is burned or regulated, or where explosive material such as explosive gases, or liquids forming explosive vapour or gas is stored or transported (e.g. chemical facilities, vessels, bunkers, sewage constructions, dwellings with gas installations, energy ducts, road and rail tunnels)". A simple solution to prevent the extraordinary load of an explosion is to remove all explosive materials. In practice this becomes quite difficult, since most buildings are heated with natural gas. When centralised heating is used and the gas stoves are replaced by electric stoves in every apartment, the extraordinary load can be limited to the basement. Now the load is confined to one location, it becomes easier to apply preventive measure as explained in section 12.2.1.1. Before these measures can be applied, the peak pressure an minimal required venting area has to be calculated. Annex D of NEN-EN 1991-1-7 gives (informative) formulas to calculate these values.

Fire

Fire is one of the best known extraordinary load for a structure. As a result, Fire Safety Engineering (FSE) has become an important aspect of the structural design. With FSE, risks are identified and preventing, controlling and mitigating the effects of a fire are a key aspects. A specific property of fire is that it may initiate other accidental loads (such as explosions) and besides the possibility of progressive collapse it also threatens the safety of the occupants trough smoke suffocation and blocked escape routes. To create a fire safe design, the engineer can consult the Eurocode and the Bouwbesluit 2012. In section 2.2.1 of the Bouwbelsuit 2012 a control table is given (see Figure 125) with twelve different user functions. Per function one is referred to a different section.

gebruiksfunctie	leden van toepassing											
	tijdsduur bezwijken									bepalingsmethode		verbouw
artikel	2.10									2.11		2.12
lid	1	2	3	4	5	6	7	8	9	1	2	*
1 Woonfunctie	1	2	3	–	–	–	–	–	–	1	2	*
2 Bijeenkomstfunctie												
a voor kinderopvang met bedgebied	1	–	–	–	5	6	–	–	–	1	2	*
b andere bijeenkomstfunctie	1	–	–	4	–	6	–	–	–	1	2	*
3 Celfunctie	1	–	–	–	5	6	–	–	–	1	2	*
4 Gezondheidszorgfunctie												
a met bedgebied	1	–	–	–	5	6	–	–	–	1	2	*
b andere gezondheidszorgfunctie	1	–	–	4	–	6	–	–	–	1	2	*
5 Industriefunctie	1	–	–	4	–	6	–	–	–	1	2	*
6 Kantoorfunctie	1	–	–	4	–	6	–	–	–	1	2	*
7 Logiesfunctie	1	–	–	–	5	6	7	–	–	1	2	*
8 Onderwijsfunctie	1	–	–	4	–	6	–	–	–	1	2	*
9 Sportfunctie	1	–	–	4	–	6	–	–	–	1	2	*
10 Winkelfunctie	1	–	–	4	–	6	–	–	–	1	2	*
11 Overige gebruiksfunctie												
a voor het personenvervoer	1	–	–	4	–	6	–	–	–	1	2	*
b voor het stallen van motorvoertuigen	1	–	–	4	–	6	–	–	–	1	2	*
c andere overige gebruiksfunctie	–	–	–	–	–	–	–	–	–	–	–	–
12 Bouwwerk geen gebouw zijnde												
a wegtunnel met een tunnellengte van meer dan 250 m	1	–	–	–	–	–	–	8	–	1	2	*
b ander bouwwerk geen gebouw zijnde	–	–	–	–	–	–	–	–	9	1	2	*

Figure 125 Control table of the Bouwbesluit 2012 for fire safety [Bouwbesluit 2012]

Most residential structures fall within section two and Figure 126 provides the minimal duration of fire resistance. Within this time, the structure may not collapse, in order to provide the occupant enough time to escape the building.

woonfunctie	tijdsduur van de brandwerendheid met betrekking tot bezwijken in minuten
Indien geen vloer van een verblijfsgebied hoger ligt dan 7 m boven het meetniveau	60
Indien een vloer van een verblijfsgebied hoger ligt dan 7 m en geen vloer van een verblijfsgebied hoger ligt dan 13 m boven het meetniveau	90
Indien een vloer van een verblijfsgebied hoger ligt dan 13 m boven het meetniveau	120

Figure 126 Minimal required fire resistance of a residential function [Bouwbesluit 2012]

Now the minimal duration is known, Eurocode NEN-EN 1991-1-2 and NEN-EN 1992-1-2 can be used to design the elements to meet the requirements. When prefab elements are used, the fire resistance is often calculated by the supplier (the design diagrams of hollow core slabs are a good example). Nevertheless, the overall fire resistance should still be calculated (single elements behave differently when combined in a structure). Since FSE has become a field of expertise within building engineering (concomitant with a high level of complexity), this will not be elaborated much further in this thesis.

Partial safety factors [CUR 2006]
Safety factors play an important role for the resistance of a structure or element. What is the reliability of an element and what is the probability that the element will function as it should? To overcome this uncertainty, safety factors are applied. With these factors, also two limit states are described: Serviceability Limit State (SLS) and Ultimate Limit State (ULS). The two limit states are described as following [CUR 2006]: "in the SLS, the functions of the system can barely be fulfilled, within the so-called usability boundaries. An example is the non-workability of a harbour, because the waves are temporarily too high. In the ULS the system permanently ceases to function because the failure and collapse of an object or objects. This, for instance, occurs if the breakwaters of a harbour entrance are washed away due to extreme conditions. As a result, under normal conditions, the wave heights in the harbour are too high for the workability."

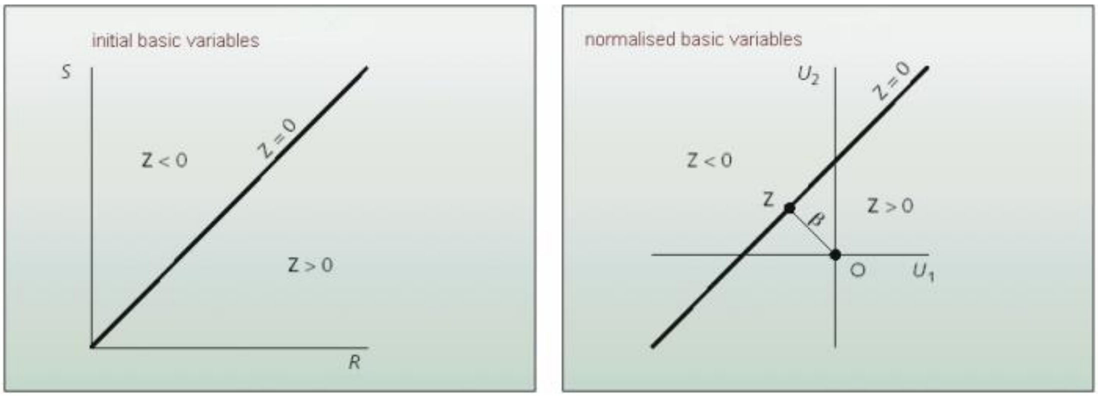
The state just before failure is the limit state. The reliability of the system is the probability that this limit state is not exceeded. When using the limit states, it's often possible to define a so-called reliability function:

$$Z=R-S$$

in which:
Z is the reliability function,
R is the strength or resistance to failure,
S is the load which leads to failure (solicitation).

When $Z=R-S=0$, the limit state is derived. The failure space is the area where $Z=R-S<0$. The element functions as it should when $Z=R-S>0$. This is clarified in Figure 127 A. The probability of failure is:

$$P_f=P(Z\leq 0)=P(S\geq R)$$



A B

Figure 127 Reliability function in the RS-plane [CUR 2006]

The distance between the origin and the line Z=0 is called the reliability index (see Figure 127 B):

$$\beta = \frac{\mu_R - \mu_s}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \frac{\mu_Z}{\sigma_Z}$$

in which:
β is the reliability index,
μ_z is the mean deviation of the reliability function,
σ_z is the standard deviation of the reliability function.

The previous text is only a slight introduction into probability calculations. For more background information the reader of the course CT4130 Probability in Civil Engineering is advised.

To clarify the relation between the reliability index and the partial safety factors, an example will be given for the partial wind load safety factor in the NEN 6702 and the NEN-EN 1990. Both calculations are based on the article Curus Windbelasting 2008 [Wit 2008].

<u>NEN 6702</u>
The peak velocity pressure is a function of the wind speed:

$$q=0.5*\rho*V^2$$

In this formula, ρ is the mass density of the air and V is the hour average of the wind speed at a height of 10m above the ground. Statistical data from the KNMI shows that in the western part of the Netherlands, the Hour average wind speed V is Weibull distributed with a mean deviation of 5.2m/s and a standard deviation of 2.6m/s. For structural purposes, the annual extreme values are more interesting. These values are characterized by a Gumbel distribution:

$$F_V(\xi) = e^{-e^{-\alpha(\xi-u)}}$$

The mean deviation and the standard deviation are given by (see also Annex C of NEN-EN 1990):

$$\mu = u + \frac{0.577}{\alpha} \text{ and } \sigma = \frac{\pi}{a\sqrt{6}}$$

For the Netherlands in area II, the following values are applicable:

$$u_1=20\text{m/s},\ \alpha=0.53\text{m/s},\ \mu_1=21\text{m/s and } \sigma=2.4\text{m/s}$$

In the NEN 6702, the representative value for the wind load is characterized by a hour average value with a repetition period of once every 12.5 years. On average, this means that once every 12.5 years this value is exceeded. The chance that the value will be exceeded in a random year is 1/12.5=0.08. This results in the following representative wind speed:

$$\mathrm{F_V}\left(\mathrm{V_{rep}}\right) = \mathrm{e}^{-\mathrm{e}^{-0.53(\mathrm{V_{rep}}-20)}} = 1 - 0.08 = 0.92$$

$$V_{rep}=25\text{m/s}.$$

Before the partial factor can be determined, the design values have to be defined: safety class 3 (this is approximately equal to Consequence Class 2 of the Eurocode) and the wind load is leading. This results in the following probability that the wind speed will be higher than the design wind speed (failure):

$$P_f(V>V_d)=\Phi(-\alpha*\beta_i)$$

in which:

α is the parabolic influence coefficient: $\alpha=-0.7$ for the leading load,

β is the reliability index: $\beta=2.6$ for leading wind load in safety class 3.

In combination with a standard normal distribution probability table of Figure 128 (take the first column and Z=1.8), the following probability is obtained:

$$P(V>V_d)=\Phi(-0.7*2.6)= \Phi(-1.8)=0.036$$

This probability holds for the entire life cycle of the structure (50 years) and this value should be divided by 50. Because only one in four wind directions is leading (see section 7.2), the value has to be divided by 12.5. This results in the following equation for the wind speed V_d:

$$F_V(V_d) = e^{-e^{-0.53(V_d-20)}} = 1 - \frac{0.036}{12.5} = 0.997$$

With this equation, V_d can be calculated: $V_d=31$m/s.

The partial safety factor can now be determined:

$$\gamma = \frac{0.5*\rho*V_d^2}{0.5*\rho*V_{rep}^2} = \frac{0.5*1.25*31^2}{0.5*1.25*25^2} = 1.54 \approx 1.5$$

	probabilities of non-exceedence for standard normal distribution									
U	**0.00**	**0.01**	**0.02**	**0.03**	**0.04**	**0.05**	**0.06**	**0.07**	**0.08**	**0.09**
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5754
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7258	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7518	0.7549
0.7	0.7580	0.7612	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7996	0.8023	0.8051	0.8079	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9430	0.9441
1.6	0.9452	0.9463	0.9474	0.9485	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9700	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9762	0.9767
2.0	0.9773	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9865	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9959	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.9978	0.9979	0.9980	0.9980	0.9981
2.9	0.9981	0.9982	0.9983	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9987	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998	0.9998
3.5	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998
3.6	0.9998	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999

Figure 128 Standard Normal Distribution probability table [CUR 2006]

<u>NEN-EN 1990</u>

The analysis for the Eurocode is more or less the same as for the NEN 6702. Only two principles have been changed: the repetition time for the representative wind load has been increased to once in every 50 years and the reliability index β has been increased to 3.3.

The reliability index is an interesting aspect. This index is coupled with the possibility of failure trough a normal distribution (see Figure 128 and Figure 129).

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,28	2,32	3,09	3,72	4,27	4,75	5,20

Figure 129 Relation between β and P_f [Wit 2008]

Because of economic reasons, the NEN 6702 makes a distinction between leading wind load and other leading loads, i.e. when the wind load is leading, lower reliability indices are allowed. As a result, the safety factor of 1.5 for wind load is maintained. This was already the case in previous standards, although this was never explicitly recognized. In the Eurocode this distinction was never made and this would result in an enormous increase of the safety factor for wind load in the Netherlands. To prevent this, the National Annex reintroduces the distinction, but only for CC2. The following statement is made in Annex C of the National Annex: "In the event that wind is the prevailing live load, the reliability index β in table C2 for the Ultimate Limit State and a reference period of 50 years may be reduced from 3.8 to 2.8." For CC1 and CC3 structures no reduction is provided, but it's assumed that the same reduction may be applied.

In practice, this uncertainty doesn't create large problems, because very few engineers will design a structure based on reliability indices and possibilities of failure. Furthermore, the requirements are very abstract and a large amount of background information is required before the calculation can be made.

In Figure 130 the recommended minimum values for the reliability index are shown according to the reference period. When wind load is leading (ULS and an reference period of 50 years), the following values may be assumed:

- RC3 (CC3): 3.3,
- RC2 (CC2): 2.8,
- RC1 (CC1): 2.3.

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

Figure 130 Recommended minimum values for the reliability index (ULS) [NEN-EN 1990]

Now the values for the reliability index are known, it possible to continue with the calculation of the partial safety factors.

The probability that a repetition time of once every 50 years is exceeded is 1/50=0.02. This leads to a representative wind speed of:

$$F_V(V_{rep}) = e^{-e^{-0.53(V_{rep}-20)}} = 1 - 0.02 = 0.98$$

$$V_{rep}=27.36\text{m/s}$$

V_{rep} is almost equal to the fundamental value for the basic wind speed used by the Eurocode: $v_{b,o}$=27m/s (see section 7.3.1.1, this value is probably rounded with no decimals).

To determine the calculation value of the wind speed, a reliability index of 3.3 is used in combination with a parabolic influence coefficient of -0.7. This results in the following probability that the wind speed will be higher than the design value (failure):

$$P_f(V>V_d)=\Phi(-0.7*3.3)=0.0107$$

This probability holds for the entire life cycle of the structure (50 years) and this value should be divided by 50:

$$F_V(V_d) = e^{-e^{-0.53(V_d-20)}} = 1 - \frac{0.0107}{50} = 0.999786$$

This results in: V_d=35.96m/s.

The partial safety factor can now be determined:

$$\gamma = \frac{0.5 * \rho * V_d^2}{0.5 * \rho * V_{rep}^2} = \frac{0.5 * 1.25 * 35.96^2}{0.5 * 1.25 * 27.36^2} = 1.73$$

This value is almost equal to safety factor for CC3: γ=1.1*1.5=1.65. The 1.1 factor in the partial safety factor was introduced in CC3 in order to meet the higher level of safety. This higher level of safety is mainly caused by the higher reliability index (3.3 instead of 2.6), but other factors in the wind model also contribute to a higher partial safety factor.

When the Eurocode partial safety factor would be calculated with a repetition period of once in every 12.5 years, a safety factor of 1.82 is obtained. This is unexpected, since lowering the repetition period would result in lower peak values. And in return, lower peak values should lead to a lower safety factor, but according to this calculation, this is not the case. To overcome this problem, the Eurocode specifies a higher reliability index when a shorter repetition period is used. This is shown in table B.2 from annex B3.2 from NEN-EN 1990 (see Figure 130).

If the National Annex didn't make a distinction between wind load and other loads (β=4.3), a safety factor of 2.1 should be applied instead of 1.65 for CC3 structures (an increase of 27%).

12.2.2 Unknown extraordinary loads

This section will explain four possible methods to prevent progressive collapse when the extraordinary load is unknown:

1. provide a second load bearing system,
2. design key elements,
3. use prescribed rules for a coherent structure,
4. apply non structural measures.

The Eurocode (see Figure 120) only specifies the first three (structural) methods. The fourth method is specified in the NTA HGBW part 3: Structural safety. Examples of unknown extraordinary loads are: design or execution errors in the main load bearing structure, material defects, terrorist attacks and abuse by the users.

12.2.2.1 Second load bearing system

Undefined accidental actions are unknown and it's likely that the element will fail. With horizontal tension ties, vertical tension ties or a combination it's possible to create a robust structure with a second load bearing path. If the structure does not remain coherent during the virtual removal of a single element, key elements have to be used (see section 12.2.2.2).

Horizontal tension ties for columns

Horizontal tension ties have to be applied along the perimeter of every floor and roof to connect the column and wall elements to the structure. Furthermore, internal tension ties have to be applied in two perpendicular directions at every column (see Figure 131). These tension ties have to be continuous.

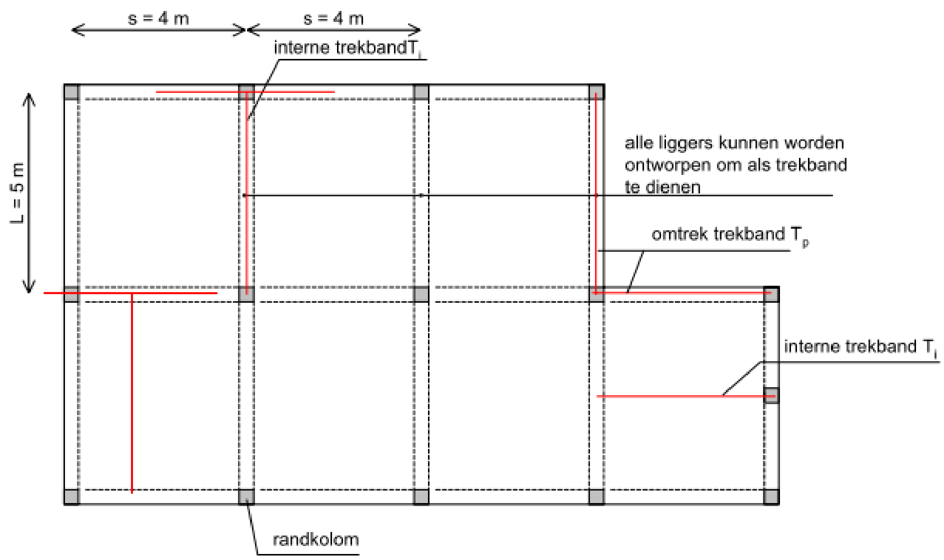


Figure 131 Example of an internal and external tension tie [Stufib 2006]

The horizontal tension ties may consist out of rolled steel sections, rebar or rebar meshes in concrete slabs. The tension ties may also consist out of a combination of the previous elements.

Every continuous tension tie, including its final anchorage, should be able to resist a tension force T_i (in case of an internal tension tie) or T_p (in case of a perimeter tension tie) in the extraordinary limit state:

$$T_i=0.8*(g_k+\Psi*q_k)*s*l\geq 75kN$$

$$T_p=0.4(g_k+\Psi*q_k)*s*l\geq 75kN$$

in which:

s is the distance between the tension ties [m],

L is the length of the tension ties [m],

Ψ is the load combination factor for extraordinary loads. Ψ_2 from formula 6.11b is used (table A1.3 from NEN-EN 1991-1-7), see Figure 132.

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions (**)	
	Unfavourable	Favourable		Main (if any)	Others
Accidental (*) (Eq. 6.11a/b)	$G_{kj,sup}$	$G_{kj,inf}$	A_d	ψ_{11} or $\psi_{21} Q_{k1}$	$\psi_{2,i}\ Q_{k,i}$
Seismic (Eq. 6.12a/b)	$G_{kj,sup}$	$G_{kj,inf}$	$\gamma_I A_{Ek}$ or A_{Ed}		$\psi_{2,i}\ Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National annex, depending on the accidental action under consideration. See also EN 1991-1-2.

(**) Variable actions are those considered in Table A1.1.

Figure 132 Calculation values for loads in extraordinary design and calculation situations [NEN-EN 1990]

The values for Ψ_2 can be found table A1.1 from NEN-EN 1990, see Figure 133.

Belasting	ψ_0	ψ_1	ψ_2
Voorgeschreven belastingen in gebouwen, categorie			
Categorie A: woon- en verblijfsruimtes	0,4	0,5	0,3
Categorie B: kantoorruimtes	0,5	0,5	0,3
Categorie C: bijeenkomstruimtes	0,25	0,7	0,6
Categorie D: winkelruimtes	0,4	0,7	0,6
Categorie E: opslagruimtes	1,0	0,9	0,8
Categorie F: verkeersruimte, voertuiggewicht ≤ 30 kN	0,7	0,7	0,6
Categorie G: verkeersruimte, 30 kN < voertuiggewicht ≤ 160 kN	0,7	0,5	0,3
Categorie H: daken	0	0	0
Sneeuwbelasting	0	0,2	0
Windbelasting	0	0,2	0
Temperatuur (geen brand)	0	0,5	0

Figure 133 Ψ values for structures [NEN-EN 1990]

The purpose of horizontal tension ties is clarified in Figure 134. When a column is removed, the membrane action from the horizontal tension ties takes over. The removal of a corner column (right hand side of Figure 134) result is a special case, since there is no equilibrium without a tension column (vertical tension tie).

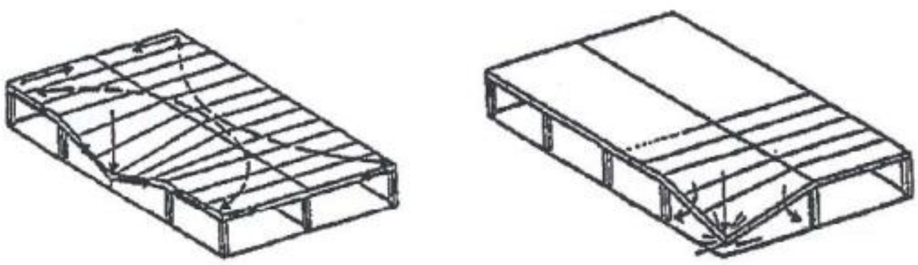


Figure 134 Membrane effect of horizontal tension ties [NEN-EN 1991-1-7]
Horizontal tension ties for load bearing walls

Horizontal tension ties for wall elements
Internal tension ties have to be distributed over the floors in both orthogonal directions (similar to the column tension ties) and perimeter tension ties have to be applied in the first 1.2m of the floor perimeter, as shown Figure 135.

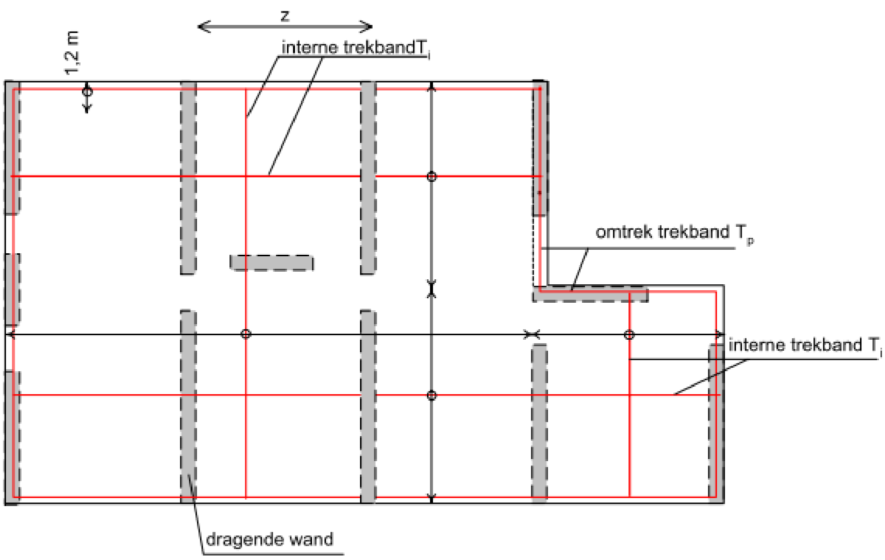


Figure 135 Example of horizontal tension ties for load bearing walls [Stufib 2006]

The calculation value for horizontal tension ties is determined as following:

for internal tension ties T_i is the largest value of:

$$F_t \ [kN/m]$$

or

$$\frac{F_t * (g_k + \Psi * q_k)}{7.5} * \frac{z}{5} \quad [kN/m]$$

For tension ties around the perimeter T_p=F_t:

in which:
F_t is the smallest value of 60kN/m or 20+4*n_s [kN/m],
n_s is the amount of floors,
z is the smallest value of:
- 5 times the height of the building, or
- the largest distance [m] in the direction of the tension tie, between the centre to centre line of the columns or other vertical load bearing elements, regardless of the distance that has to be spanned by:
  - only a floor plate, or
  - a combination of beams and floor plates.

According to A.7 from NEN-EN 1991-1-7, the nominal length of the wall that has to be imaginary removed is maximal 2.25*H, were H is the floor height.

Vertical tension ties
Every column and wall should be provided with a continuous tension tie from the foundation up to the rooftop. In case the building contains a frame structure (for example, a steel structure or reinforced concrete structure), the columns and walls that bear a vertical load should be able to resist an extraordinary tension load equal to the

maximum value of dead and live load working on that column on any given floor. This extraordinary load does not have to be taken into account simultaneously with the dead and live load (i.e. at every floor level, the column is loaded by a single floor. In the event a column at the level below fails, the floor load becomes a tension load. This maximum tension load has to be taken into account, even if there is no (dead or) life load).

The vertical tension tie may be taken into account if:

- the masonry walls have a minimal thickness of 150mm and a minimal compression strength of 5N/mm^2 conform NEN-EN 1996-1-1,
- the free height H [m], measured between the floor and the ceiling, is smaller than 20*t, were t is the wall thickness [m],
- they are designed to transfer a tension force T, with the largest value of:

$$T = \frac{34 * A}{8000} * \left(\frac{H}{t}\right)^2 \quad [N]$$

or

T=100kN per meter wall

  in which:
  A is the load bearing cross section of the wall from the top view [m^2],
  H is the free height from the floor till the ceiling [m],
  t is the wall thickness [m],
- the vertical tension ties have been placed at a maximum distance of 5m along the wall and if they are not located more than 2.5m from a unsupported end of the wall.

12.2.2.2 Key elements

Sometimes it's impossible to construct a second load bearing path: it might be physically impossible or economically unattractive. Therefore the Eurocode gives a second possibility: a "key element" may be used. Annex A.4, c) states: "Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, or other such limits specified, then such elements should be designed as a "key element"".
And annex A.8 continues: "a key element should be able to resist and extraordinary load A_d, applied horizontally and vertically (one at a time) on the element and all the connected components, taking into account the ultimate strength of the components and the connections. The application of this load is described in EN 1990 6.11b and it may be applied as a concentrated or uniform distributed load. The recommended value of A_d for a building structure is 34kN/m^2".

Besides the previous mentioned aspects, the NTA (National Technical Arrangements) also requires that a key element should have an increased capacity so it can take a load as calculated with the fundamental load combination, multiplied by an additional partial factor for the loading $\gamma_{f;as}$=1.2.

Key elements are frequently applied in structures, but these elements represent a weakness in the structural system. This is because the failure of a key element often result in the collapse of (a large part of) the structure. It is advised to prevent key elements as much as possible within the economical boundaries.

12.2.2.3 Use prescribed rules for a coherent structure

A coherent structure is obtained when all the single elements are tied together. The vertical and horizontal tension ties provide a coherent and ductile structure when they are applied properly. It should be noted that the NEN-EN 1991-1-7 doesn't specify how the tension ties should be incorporated in the reinforcement layout. Aside from the tension ties, the NEN-EN 1992-1-1 does provide detailing rules for torsional, shear and punching reinforcement. Chapter 9 also provides rules for the anchoring of an intermediate and end support. In Figure 136, an example is shown how to detail torsion reinforcement. More information can be found in chapter 9 of NEN-EN 1992-1-1.

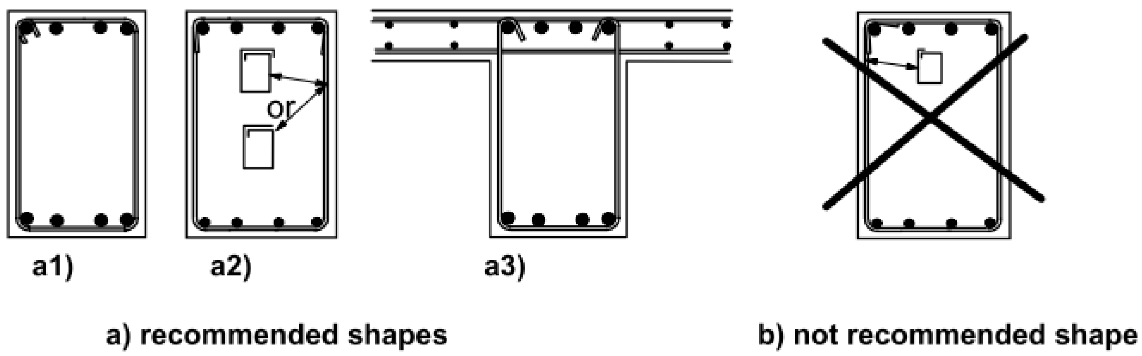


Figure 136 Detailing of torsion reinforcement [NEN-EN 1992-1-1]

When a coherent structure is created, the redundancy is increased. It should be noted that this might also have an negative effect: during an extraordinary failure, the collapse might not be limited to one bay. Due to the coherency, surrounding bays could collapse as well.

12.2.2.4 Non structural measures

Non structural measures are for example quality assurance, prevention of mistakes, elimination of already made mistakes and social control. Per unknown extraordinary load, these solutions will be explained. The following text is taken from [Terwel 2011]:

"Design errors

Mistakes are inevitable during the design phase. The first preventive measure is that a client should appoint a capable coordinating structural engineer, who will be responsible for the integral structure.

The main coordinating structural engineer can show his capability by answering the following questions ([4][9] B1.1.2):

- "Does the engineering company have any experience with similar projects (similar in size and complexity)?
- What is the vision of the engineering company on the project?
- Can the engineering company proof that its internal checking is sufficient?
- Does the engineering company have qualified personnel?
- Does the engineering company have the right facilities?
- Can the engineering company show positive references?
- Is the engineering company investing in knowledge development?
- Is the engineering company able to think out of the box?"

The coordinating structural engineer should execute the following tasks:

[9] SPEKKINK, D.E.A., Compendium aanpak constructieve veiligheid, Vrom Inspectie et al., 2008.

- Stating a program of requirements for the structure.
- Making the structural design and present it with drawings and calculations.
- Communicating with checking parties.
- Guarding the integration of different parts of the structure.
- Analysing and pointing out structural elements in design and/or execution which are vulnerable to failures (risk analysis).
- Cooperating with the third party which executes the second opinion.

The coordinating structural engineer should make an introduction on the statical calculations with the following subjects:

- Building codes which have to be applied.
- Description of the structure.
- Amount and position of loads.
- The load combinations.
- Description of the way the vertical loads will be transferred to the foundation.
- Description of the way the horizontal loads will be transferred to the foundation.
- Description of the situation in case of fire.
- Description of the way accidental loads are taken into account.
- Summation of the elements which are part of the main load bearing structure.

This document should be available for all structural engineers involved in the project.

The second preventive measure is that a client should make a demarcation of responsibilities between several structural engineering companies. For a high rise project usually several specialized structural engineers are appointed for different parts of the building. These structural engineers often have different principals. For instance the structural engineers which design the precast concrete elements often work in order of the contractor.

The third measure is that the structural design (together with detail engineering) of the main load bearing structure should be checked on content by an independent party.
The municipality will sometimes check the design too, but for quality control one cannot count on this checking, for checking by the municipality is optional not obligatory (see discussion [5][10]).

Execution errors
For the prevention of mistakes in the execution phase two measures are issued.
First, the contractor should make a quality assurance plan for the execution phase, hand it over to the client for permission and act in accordance with the plan. In this document the tasks, authorization and responsibilities of the involved execution parties are described, together with the (checking) procedures.
Second, the contractor should make a project plan detail engineering, in which the design tasks of the contractor, the planning and control of tasks are mentioned. This plan should be given to the main structural engineer for approval and all subcontractors should act in accordance with this project plan.

Additional control on the execution of the main load bearing structure is necessary by an independent party (not the contractor). The checker should make a description of the necessary checking, which should be approved by the coordinating structural engineer. Most important is the checking of the execution of elements that cannot be checked after completion (e.g. reinforcement). A clear administration of the checking should be available."

---

[10] MOESKER, H. C. W. M., Bouw- en woningtoezicht na Vie d'Or, *Tijdschrift voor bouwrecht,* 2007.

Material Defects
The prevention of material defects should be perused during the entire life cycle of the structure, from the factory until the demolition. A quality assurance plan is an effective measure and all the subcontractors should provide one. At every mile-stone, the materials and elements should be checked for defects and if they meet the requirements as stated.

Terrorist attack and abuse by users
Quality control does not provide a solution for malicious actions. Therefore social measures have to be taken. Security barriers can be placed, to prevent unauthorised persons to enter the building. By limiting the accessibility of key elements in the structure, abuse becomes more difficult.

"Other measures?
Besides the given measures some other measures were considered, but were not presented in the NTA.
Stating a minimum budget, for instance, seems to be useful, but price fixing within an industry is prohibited by European legislation. Clients awareness of not only tendering on lowest prize is increasing ([6][11]).
Stating a reasonable amount of time for design and execution might be useful, but will vary from project to project.
A users handbook (with loads etc) is not prescribed to avoid the problem of misuse, because it is assumed that professional clients for this size of buildings will ask for a handbook themselves, to use as a guideline for e.g. maintenance."

12.3 Risk analysis for CC3 structures

If a structure is categorised as a Consequence Class 3 structure, no predefined measures can be taken. Instead, a risk analysis has to be preformed, to gain a thorough understanding of potential hazards for the integrity of the structure and their consequences. The reason for this enhanced level of security is not that these structures are less predictable than CC1 and CC2 structures, but because the consequences of a collapse are far greater. Each building reacts different to accidental loads and it would go far beyond the intention of the regulations to provide measures capable of safeguarding all buildings against every imaginable accidental event.

Annex B of NEN-EN 1991-1-7 provides guidance for performing a qualitative or quantitative risk analysis. Figure 137 shows an overview of the proposed steps.

[11] HOLST, A., Gunnen op waarde maakt creatief, *Cobouw*, 06-01-2011, SDU, Den Haag, 2011.

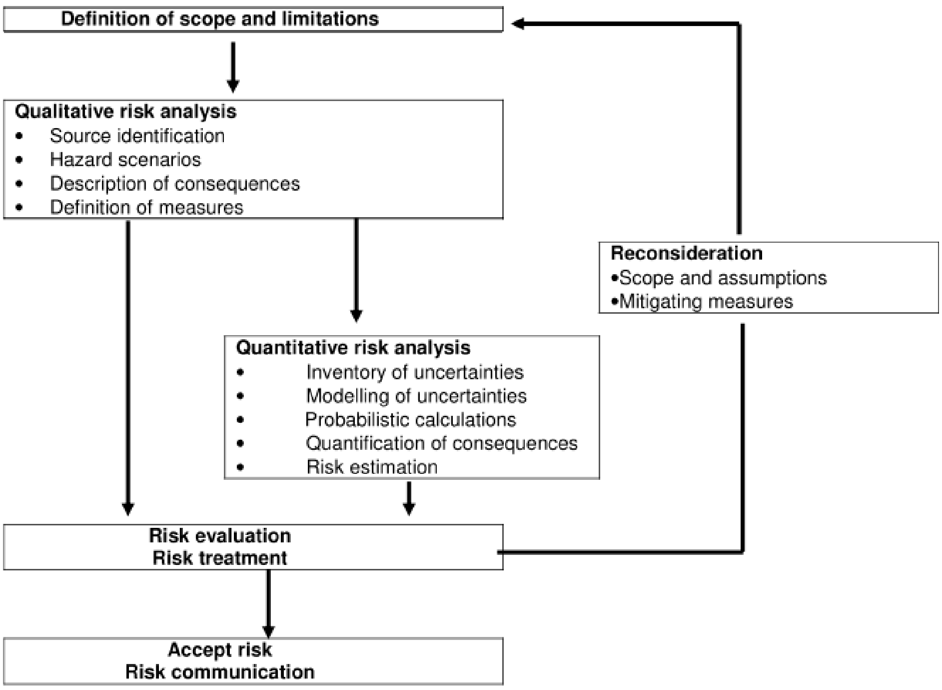


Figure 137 Overview of the risk analysis for CC3

The following text is taken from [Siersma 2005]:

"Step 1: Definition of scope and limitations
In this phase, the risk analysis describes what it sets out to do. The subject and background are mentioned and the goals of the analysis are stated. In this phase, the engineer must acquire, if not yet the case, an extensive and thorough understanding of the building's load bearing structure and structural properties. All other non-structural properties that may be relevant in relation to potential hazards, have to be described. This includes the buildings physical environment, its use/function and other human and organisational circumstances. All these must be stated sufficiently detailed. Finally, the assumptions and simplifications made have to be documented, since they are important in judging the outcome of the analysis.

Step 2: Qualitative risk analysis
The risk analysis should contain at least a qualitative part and, if possible and relevant, a quantitative part. The qualitative risk analysis should identify all hazards and corresponding scenarios. To be able to asses these possible scenarios, an understanding of the load bearing structure as a system is essential. For this reason, application of a (combination of) technique(s) from a wide variety is suggested. These may include, for instance, a Potential Hazard Analysis, fault tree and event tree, among others.
The outcome of this qualitative risk analysis may be presented as a matrix that, of course qualitatively, shows the risk level associated with certain hazards.

Step 3: Quantitative risk analysis
A quantitative risk analysis seeks to assign a quantitative label to the risks resulting from the hazard scenarios identified. To do this, the probability of hazards is estimated and the damage is expressed in numbers. The probability of occurrence of a certain hazard may be based on (expert) judgement and may for that reason differ quite substantially from actual frequencies.

Since this process is far from an exact science, uncertainties in data and models used and the assumptions upon which the analysis is based have to be considered carefully. This consideration may imply a sensitivity analysis.

A quantitative risk analysis may not always prove feasible or desirable. It is therefore not necessarily a part of the risk analysis as proposed by the Eurocode.

Step 4: Risk evaluation
Risk evaluation entails weighing the (qualitative or estimated quantitative) risk resulting from hazards against some level of acceptable risk. In this context, both the individual acceptable level of risk and social acceptable level of risk are important. The risk evaluation will establish whether or not certain risks can be accepted.

It is important to realise that the weighing of a risk against an acceptable risk is possible when both are expressed verbally, instead of numerically. This may be the case when the quantitative part of the risk analysis does not prove feasible or when uncertainties in its outcome would make any conclusion of little to no value.

Step 5: Risk treatment
If the risk associated with a certain hazard cannot be accepted, measures have to be taken to mitigate them, i.e. bringing the risk level of a certain hazard to a level that can be accepted, by either reducing the probability of its occurrence or the magnitude of its consequences.

The Eurocode brings forward a global set of structural and non-structural measures[12]:

- elimination or reduction of the hazard (e.g. modify design concept, combat hazards),
- by-passing of the hazard (e.g. protection of the structure by means of safety barriers or a sprinkler system),
- controlling the hazard (e.g. checks, warning systems, monitoring),
- overcoming the hazard (e.g. providing an alternate load path, providing sufficient reserves, increasing robustness).

Note that these measures may address both unspecified causes and identified actions. Only the last of these four categories of measures may contain truly structural measures. This exemplifies the statement that dealing with accidental loads is about more than just structural improvements in the building's design.

Step 6: Modification and revision
Modification may be necessary if, with the selected mitigation measures, the design of the structure cannot be accepted in relation to the hazards considered. This is clarified in Figure 137.
If it can, risk communication with the general public is essential. As mentioned in [14][13], the public's perception to collapse event carries in it a strong social and subjective element. Though the deliberation in the former parts of the risk analysis may appear to give an exact outcome, there is in fact no such thing: nor is the calculation of risk, or in the levels of acceptance. All of these have a basis of (a combination of) assumption, estimation and subjectivity.

Step 7: Presentation of the results
Results of the analysis are presented as a list of hazards with their accompanying consequences and probabilities. The degree of acceptance of each of these risks should be discussed. Also, the data source that were used and assumptions/simplifications that were made should be stated, along with a notion of the sensitivity of the outcome to variations in input. In this way, the validity and limitations of the analysis become clear."

---

[12] Siersma used the prEN 1991-1-7 for his report. In the current Eurocode, six instead of four measure are provided (see Figure 120).
[13] Koot, A.J.; Wiltjer, R.H., 2004. *Constructief ontwerpen met tweede draagweg.* BV Nieuws 3-2004, Betonvereniging.

Before the actual risk analysis for the Zalmhaven tower can be preformed, several aspect have to be considered. For example: what is disproportional collapse and in which phase should it be implemented?

Disproportional collapse
The failure of an element may result in progressive collapse and eventually in the collapse of the entire structure. But is the collapse of the entire structure due to the failure of an element disproportional? The answer to this question lies in the definition of the scale of the failing element, in relation to the scale of the entire structure.

The following definition from the Eurocode NEN-EN 1991-1-7 Annex A4 will elaborate on that:

"(1) Adoption of the following recommended strategies should provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse."

With this "disproportionate level of collapse" is meant that there is a disproportion between the level of initial collapse and the level of total collapse. For example: the structural collapse following a core collapse can hardly be considered as disproportional, since the core entails almost the entire load bearing structure. In contrast, the collapse of (a large part of) the structure due to the failure of only one column may very well be considered disproportional.

To get a better grasp of the scale of an element, two definitions are made (disregarding the exact definition of an "element") [Siersma 2005]:

"Physical scale:	*the size of an element or building component in relation to the entire structures size,*
Functional scale:	*a notion of the magnitude of the consequences of an element's or buildings component's failure; or: importance for the building structures stability or functioning.*

Key elements may have a large or small physical scale, but will always have a large functional scale; this why their identification is essential. Disproportionate collapse may be the case, if (upon failure of the element) the extent of collapse exceeds the functional boundaries of the element."

The core of the previous example can be categorised as a key element[14] with a large physical and functional scale and therefore the collapse cannot be considered as disproportionate.

It's easy to see why disproportional collapse should be prevented. The initial failure is limited to only one element or component and this has a corresponding risk and consequence. If this local failure results in a far greater overall failure, the associated risk and consequence is far greater as well. This implies that great caution is appropriate with respect to elements with a small physical size and a large functional scale (for example key elements).

[14] The Eurocode doesn't specify the physical scale boundaries of a key element and the interpretation of the engineer is leading. In practice, an entire core is never defined as a key element. It's more likely that a wall section of the core or column is entitled as key element.

The design process paradox
Annex B.4.1 of NEN-EN1991-1-7 states the following with respect to the qualitative risk analysis:

"The following should be taken into account in defining the hazard scenarios:

(...)

- the concept of the structure, its detailed design, materials of construction and possible points of vulnerability to damage or deterioration."

This brings an important aspect to light: the Eurocode statement mentioned above implies that the structures detailed design should be finished before the risk analysis can be applied. This is rather logical, since the hazards depend on the total design. However if a problem occurs it becomes difficult, costly and inefficient to apply measures that mitigate risks at the end of a design process. On the other hand, performing a risk analysis in an early design stage with little information might result in measures that, upon completion, might not be effective ore unnecessary. This paradox is shown in Figure 138.

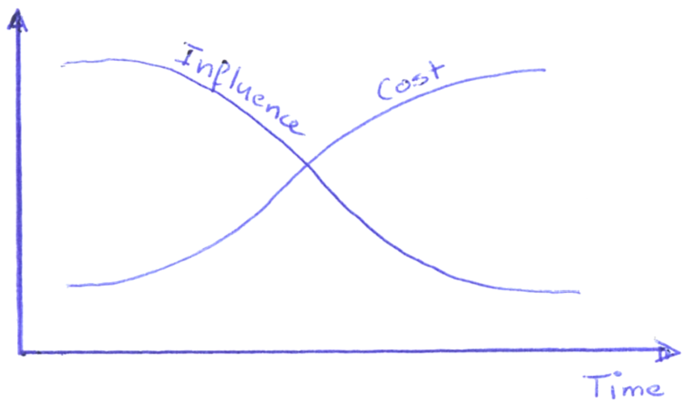


Figure 138 Distribution of cost and influence over time in the design process

In [Siersma 2005] the following is stated:

"The problem with any (structural) risk reducing measure is that the effect will only become clear after finishing the analysis, whether the measure may affect the fault tree on a high level, or on a lower level.
Risk reducing measures taken in an early phase of the design process (on basis of an apriori demand for an alternate load path) may be more effective, more efficient and more reliable, but the arguments on which their application is based, may be insufficiently strong in the light of a risk (scenario)-based approach[15]."

This paradox cannot be solved and [Siersma 2005] advises to use the risk analysis alongside the course of the design process: "It should be started at the beginning of the design process and constantly be updated when choices in the design process are being made. In this way, it is used as a tool that constantly reflects on the engineer's choices with regard to the structural design."

[15] A risk-based approach is a strategy to identify and mitigate risk.

12.4 Risk analysis for the Zalmhaven tower

With the previous section, it's now possible to perform a risk analysis for the Zalmhaven tower. This will be a preliminary risk analysis, because the design process has yet to begin.

12.4.1 Introduction

The Zalmhaven tower will be the highest prefabricated tower of the world. Located near the Maas river and the Erasmus bridge, the tower stands within the high rise centre of Rotterdam. Besides the high rise tower of 200m, several low rise projects will be constructed to create more liveliness. An overview of the project can be found in Figure 139. More information about the project can be found in chapter 2.

Figure 139 Artist impression from the Zalmhaven tower and low rise [Zalmhaven]

12.4.2 Step 1: Definition of scope and limitations

The goal of a risk analysis is to provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse. The focus lies on hazards/risks, accidental loads and event scenarios. The relations between these aspects are depicted in Figure 140.

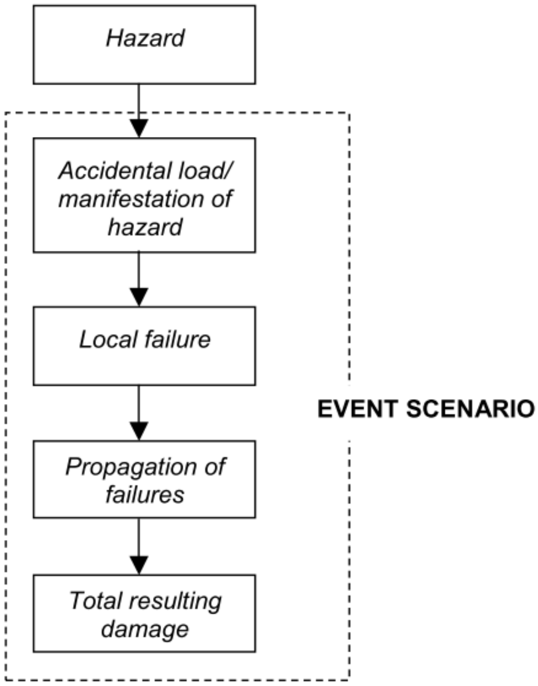


Figure 140 General sequence for progressive collapse [Siersma 2005]

Objectives
The goal of preventing progressive collapse is limiting the extent of failure, i.e. limiting the length of the event scenario. A hazard or risk may result in an accidental load that may result in local failure (this is acceptable). However, the structure should prevent the propagation from local failure to progressive failure.
The analysis tries to bring as many hazards from the category "unknown extraordinary loads" to the category "known extraordinary loads" (see Figure 120). It should create a thorough understanding of potential risks for the integrity of the structure, the event scenarios and their consequences. A fourth important aspect of the risk analysis is the statement whether the imposed risks can be accepted and, when this is not possible, how they should be mitigated.

Assumptions and simplifications
To create a risk analysis that can be preformed, several assumptions have to be made. For example:

- Risks for adjacent buildings are not taken into account, since they do not threaten the structural integrity of the Zalmhaven tower.
- Only casualties from structural failure are considered. Other risks that may threaten occupants, for example suffocation due to an excess of carbon dioxide, are discarded.
- Human casualties outside the building resulting from a structural collapse are included in the considerations. Other external human casualties are excluded.
- Economic damage is of minor importance and the inaccessibility of the structure or surrounding properties after a collapse is not taken into account.
- Only the main load bearing structure will be considered. The failure of a facade panel is outside the scope of this analysis.
- Risks and failures during the construction phase are discarded. This is a large simplification, since most risks and failures occur during this phase. Two important factors that contribute to a larger risk of failure are the incomplete structure and the large amount of construction workers that are present with construction equipment. This phase is discarded due to the limited time for this study.

- Since no prefab structure has been designed yet, the current cast in situ layout will be used to supplement the prefab design.

General features
The load bearing structure can be shortly described as following:

- the tower has a height 202.25m, 65 stories and no basement,
- the storey height in the lobby is 6.1m, the floors have a height of 3.05m and the 65th floor has a height of 4m,
- the floor plan of the building is 30m by 30m,
- the building is stabilised by two 500mm thick shear walls in the x-direction and three 400mm shear walls in the y-direction,
- the floors span from shear wall to shear wall (7.8m) and no internal columns are used. The east and west facade (left and right facade in Figure 141) contain vertical load bearing columns, in order to bear the weight of the floors,
- the foundation consist out of a 1m foundation slab in combination with 1.5m thick diaphragm walls with a length of 63.2m,
- the high rise tower is dilatated from the low rise buildings.

See Figure 141 and Figure 142 for an impression of the load bearing structure.

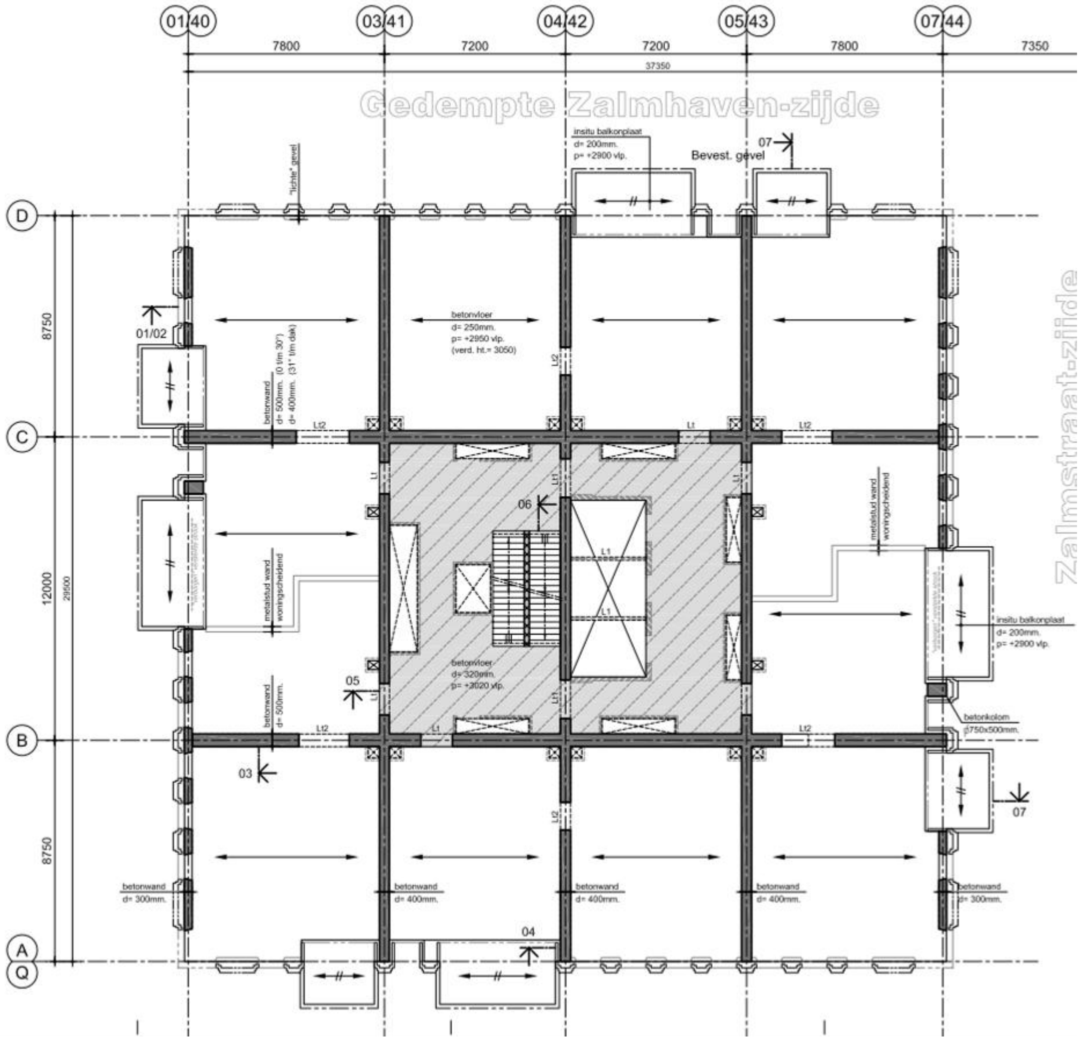


Figure 141 An average floor plan of the Zalmhaven tower

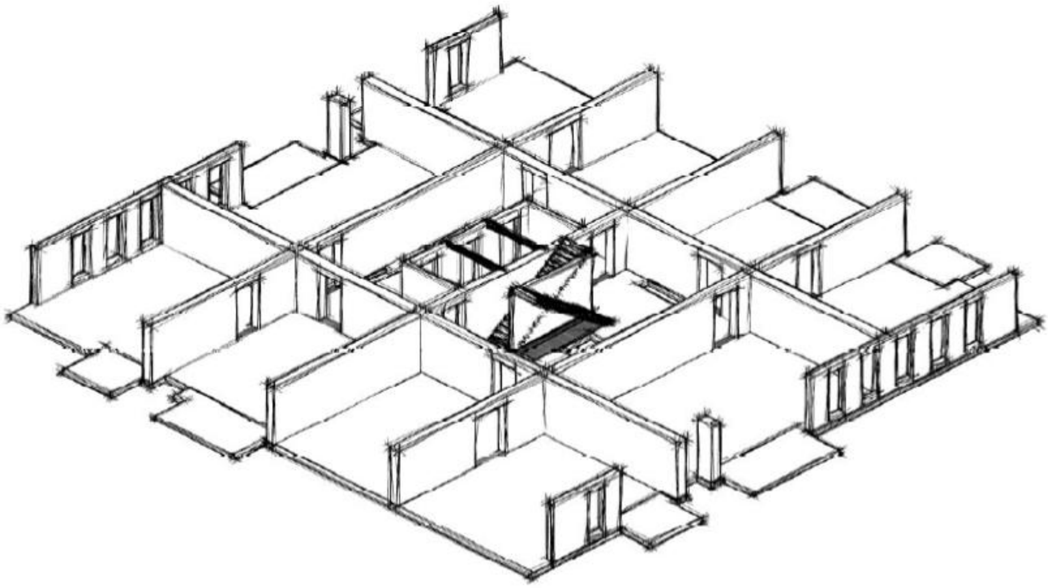


Figure 142 Building layout of the Zalmhaven tower

Floor system
For the floors, 300mm thick massive prefab slabs are used with integrated ducts. The floors are supported by the shear walls and the load bearing facade (see Figure 141). To provide diaphragm action, the separate floors are connected via tension ties. As a result, no structural topping is required. The span direction of the floors can be found in Figure 141.

Special features in the building's design
The Zalmhaven tower can be considered as an icon structure, since it will become the highest structure of the Netherlands and the highest prefabricated structure of the world. Aside from this, the structure does not contain any special features (for example cantilevering floors or a pre-stressed core).

Human and organisational circumstances
Human and organisational circumstances concerning the building can be described as following:

- The Zalmhaven tower has a residential function. A change of function cannot be discarded, but this is highly unlikely.
- The building is accessible at several floors via adjacent low rise buildings. The Zalmhaven tower doesn't provide any vertical transport for these buildings and it is only accessible for the occupants of the tower. This is achieved by using an electronic key.
- Since there is no basement, all the installations are placed on the first and second floor. The required area is relative small, because the building's temperature is controlled with district heating (the centre and the Kop van Zuid are connected to this system).
- Because of the district heating, the building is not equipped with a gas system. Furthermore, no hazardous materials are being stored or used in the building.

Environmental circumstances

<u>Surroundings</u>
An overview of the surroundings is given in Figure 143.

Figure 143 Arial view from the south

The surroundings include:

- the two towers, known as the "Hoge Heren", 140m to the north with a height of 103m,
- the "Hoge Erasmus" tower, 80m to the south-west with a height of 80m,
- several low rise residential building to the east of the three high rise towers,
- the plinth building, surrounding the high rise tower (the building left and right from the red box will be demolished).

Because of the dilatation between the high rise tower and the low rise plinth building, no structural connection is created. The tower is not dependant on other structures.

<u>Traffic</u>
The Zalmhaven tower is located near the Houtlaan. This is a small urban road with a speed limit of 50km/h. Only local people will use this road and Figure 144 gives an impression.

Figure 144 Impression of the Houtlaan [Google 2012]

Level 2 and 3 of the tower are partly used as parking level and therefore the northern structural walls become subjective to impact loads. This can be seen in the building sections in Report 2 of this study.

The third largest airport of the Netherlands, Rotterdam The Hague Airport, is located 6km to north of the Zalmhaven tower. The flight paths are orientated east-west and the Zalmhaven tower is not located within the flight path. The traffic consists mostly out of relative small aircrafts (Boeing 737-700 and Fokker 50).

12.4.3 Step 2: Qualitative risk analysis

To perform the qualitative risk analysis, the Eurocode states that several methods can be used, such as a Fault Tree Analysis (FTA), Event Tree Analysis (ETA), Potential Hazard Analysis (PHA) or a HAzard and OPerability studies (HAZOP). How these methods should be implemented is not discussed in the Eurocode.
For the risk analysis of the Zalmhaven tower a Fault Tree Analysis will be used, since this method is regularly applied in Civil Engineering.

The Fault Tree Analysis was developed in 1962 to evaluate the launch control system of American ballistic missiles. Since the introduction, the FTA has become a widely used method throughout many industries and professions (for example Safety and Reliability Engineering). A FTA is top-down analysis were deductive reasoning is applied to create a conclusion (top event) that necessarily follows from the stated premises (lower events). An example of a FTA is shown in Figure 145 and Figure 146 explains the symbols that are used in the FTA.

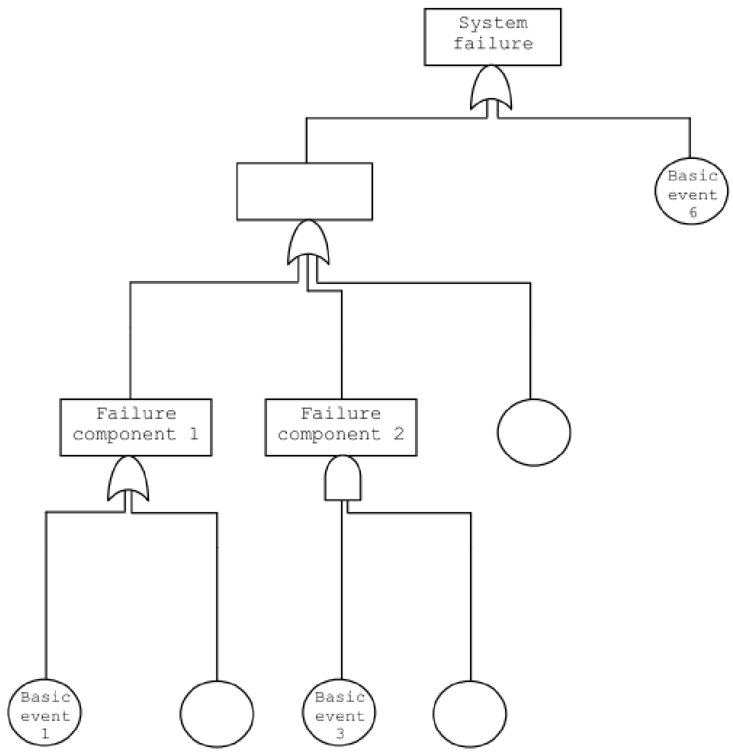


Figure 145 Example of a Fault Tree Analysis [Siersma]

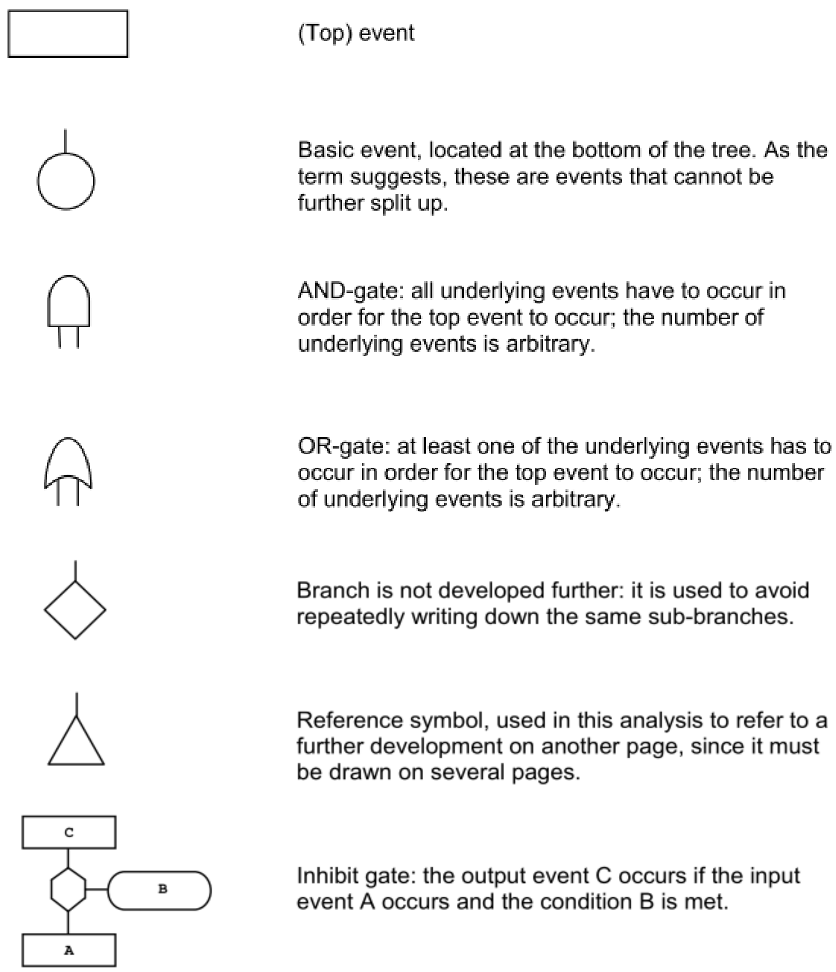


Figure 146 Symbols used in a Fault Tree Analysis [Siersma 2005]

An undesired event, for example the collapse of an entire structure, is placed in the top event box. Different component failures leading to the top event, for example the collapse of several columns, are represented by lower events in the tree. Every path or branch in the tree represents a different failure mode and the length of a path determines the amount of required progressive collapse before the top event is reached. The FTA is performed by stating a clear top event and answering the questions "what might cause this event?" and "how can this happen?".

A complex structure such as the Zalmhaven tower creates a large fault tree (see report 2 of the literature study). All the events are connected by an OR-gate and to increase the readability, these gates have been left out. The level of detail of the analysis is relative low, since the analysis is preformed before any prefab designs have been made. The FTA will be expanded when more information becomes available.

The FTA can be found in report 2 of the literature study. Several important events require a small elaboration:

A **Top event: "Collapse of the building structure over the full height"**
The top event contains the collapse of the building structure over the full height since the emphasis of the analysis lies on the complete event scenario (from accidental load to the total resulting damage, see Figure 140). In other words, the end-goal of this analysis is to create a structure with an acceptable level of robustness that can sustain localised failure without a disproportionate level of collapse.

B/C/D **Events: "Shear wall failure", "Foundation failure" and "Collapse of the floor system over the full height"**
Shear wall failure together with foundation failure and the collapse of the floor system over the full height are the three sub-top events. The failure of one of these systems will automatically result in the failure of the entire structure. Systems or elements with a short path may be considered as sub-top events.

E **Event: "Strength exceeded (S>R)"**
This is the most common event in the entire FTA. It can be concluded that the Solicitation (the imposed load) and the Resistance are the two most important aspects in the structural design.
To create a safe design, the resistance should be larger than the solicitation (R>S), but Figure 147 shows that this is physically impossible. The bell-shaped lines of the reliability density curves extend to infinity and a certain probability that S>R has to be accepted. This probability is represented by the hatched area of Figure 147 and should be as small as possible. It should be noted that in the FTA failure can only occur when the solicitation is higher than expected or when the resistance is lower than expected (or a combination of both), while it's also possible that failure occurs when both values are within the expected boundaries.

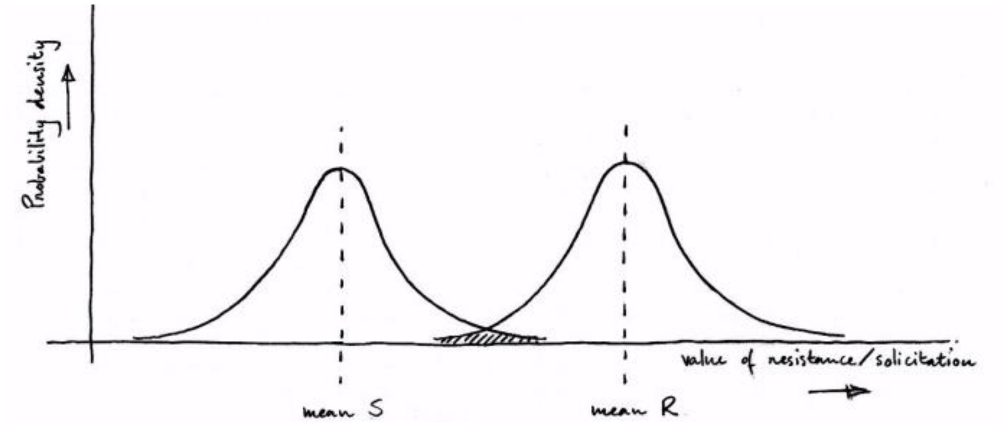


Figure 147 Resistance versus Solicitation (imposed load) [Siersma 2005]

12.4.4 Step 3: Quantitative risk analysis

If it's feasible and desirable, the next step will be performing a quantitative risk analysis. This analysis is used to assign quantitative levels (expressed in numbers) to the risk from the identified hazard/risk scenarios. To do this, detailed information is required about the probability of occurrence of the basic events and determining the damage that results from the considered basic event. There are several aspects that make this process not only difficult, but also labour intensive [Siersma 2005]:

- "There is little relevant data available on probabilities of occurrence of various unforeseen events. Compared to other branches of industry (airline industry, power industry) there has not yet been a systematic collection and preservation of data concerning actual collapse failure cases. The data that does exist may best be sought in the insurance industry, but the way that this industry documents failure cases leaves little basis for application in building engineering [13][16]. The most reliable source of information on the occurrence of accidental loads may therefore be expert judgment; this may be gathered by, for instance, Delphi (-like) techniques [13].
- Besides a quantitative description of the probabilities of occurrence of events, a quantitative description of the resulting extent of damage poses problems as well. As mentioned in [14][17], structure reaction to a local collapse is a complex dynamic process, that is, even with the best (computerized) analysis tools available, very hard to predict. Moreover, the correlation between the extent of structural damage and the amount of casualties would need further clarification.
- If the fault tree diagram were to be quantitatively expended, this would require determining a risk magnitude for each failure path in the tree. These are more than the amount of hazards, since an event scenario does not necessarily have to develop, until finally the top event is reached. This exemplifies the tremendous amount of work needed to quantify just this tree."

[16] Lemer, A.C., McDowell, B.D., (1991). *Uses of Risk Analysis To Achieve Balanced Safety In Building Design and Operations.* National Academy Press, Washington D.C., USA.
[17] Siersma, R.C., (2004). *Progressibe Collapse of Building Structures: Literature Study.* TU Delft, Delft, The Netherlands.

The NEN-EN 1991-1-7 states in Annex B4.2 (1) that:

"The risk analysis will be terminated at an appropriate level, taking into account for example:

- the objective of the risk analysis and the decisions to be made,
- the limitations made at an earlier stage in the analysis,
- the availability of relevant or accurate data,
- the consequences of the undesired events.

The limitations mentioned above in combination with the statement of the Eurocode imply that performing a quantitative risk analysis is not necessary for this study.

12.4.5 Step 4: Risk evaluation

During the evaluation, the risks from encountered events are compared with the level of acceptable risk. As a result of this evaluation, certain risk may or may not be accepted. The level of risk has to be expressed verbally, since no quantitative risk analysis has been performed. It's also possible to use a relative risk level to compare different risks.

The relative risk comparison is based on the following principle: the severity of initial failure and the severity of the resulting damage are classified with a scale of 1 to 5, were 1 refers to the smallest severity and 5 refers to the largest severity. For every event the severity of the initial failure and the severity of the resulting damage has to be determined. The next step is to combine these two values into a relative risk level. This is done by adding the two values and not by multiplying them. The reason for this method lies in the use of the severity class. The relation between the actual probability of initial failure (or the extent of the resulting failure) and the severity classification is logarithmic, see Figure 148.

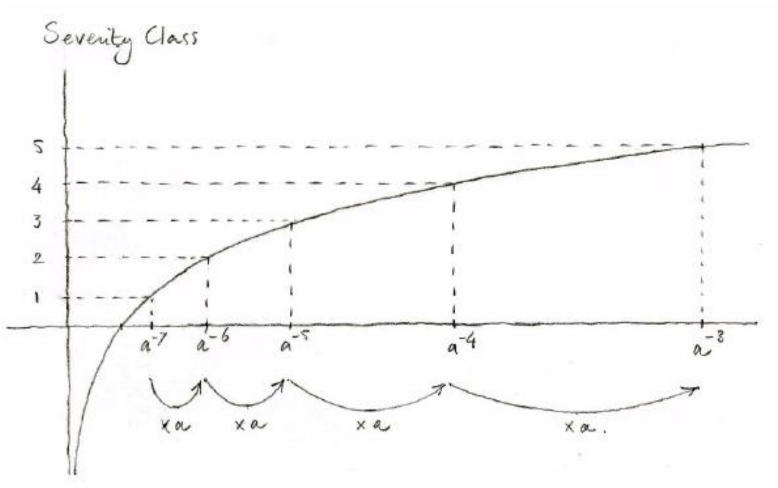


Figure 148 Relation between actual probability of initial failure or the extend of resulting failure and the severity class [Siersma 2005]

This means that:

$$\text{severity (if)} \approx {}^{a}\log(\text{probability of initial failure})$$

and

$$\text{severity (rd)} \approx {}^{a}\log(\text{extend of resulting failure})$$

where a is some unknown constant. The definition:

risk=probability (of initial failure)*damage (extend of resulting failure)

can be approximated by:

$$risk=a^{severity\ (if)}*a^{severity\ (rd)}=a^{severity\ (if)+severity\ (rd)}$$

Since “a” remains unknown, the comparison between different event scenarios (different tree branches) doesn’t become more accurate than for instance the verbal method mentioned before. On the other hand, relative risk levels for different event scenarios can be compared very easily with this method. This relative comparison should be treated with care, since the differences between the risks may be distorted. This distortion is inevitable, since this method is not exact (no quantitative values) and depends on the interpretation of the engineer.

The following scenarios and severities can be distinguished:

Scenario 1: Resistance lower than anticipated (shear wall failure)
This scenario can be caused by design errors, inadequate materials/prefab elements and fire load. The severity of initial failure is classified as level 2. The probability that crucial design errors are made and inadequate materials/prefab elements are used is relatively low. On the other hand, it’s likely that a fire may occur in the lifetime of the structure. The severity of damage is classified as level 5, because the lower resistance of a wall element or entire wall may lead to the collapse of the entire structure.

Scenario 2: Solicitation higher than anticipated (shear wall failure)
A higher solicitation could be the result of impacts, explosions, design errors, extreme wind conditions and skewness of the structure. This scenario has a large amount of possible causes that could result in a larger solicitation than anticipated and therefore this is classified as level 3. The possibility that this solicitation acts on a large shear wall section is present and therefore the severity of damage is classified as level 5.

Scenario 3: Resistance lower than anticipated (vertical foundation failure)
Vertical foundation failure from a lower resistance than anticipated can be caused by design errors (insufficient amount of diaphragm walls or wrong dimensions/depth), an improper concrete mixture or inadequate materials (faulty reinforcement). The probability that the entire foundation has a lower resistance than anticipated is relative low. Therefore, the severity of initial failure is classified as level 1. The monolithic design of a foundation has a positive influence on the integrity and the foundation will act as a coherent structure. As a result, the severity of resulting damage is classified as level 2. This value is relatively low, since an uniform vertical displacement has only small consequences.

Scenario 4: Solicitation higher than anticipated (vertical foundation failure)
This scenario can be caused by higher loads from the shear walls and facade columns (design error on the loads). Negative adhesion on the diaphragm walls is a third cause. The severity of initial failure is classified as level 1. The severity of damage is classified as level 2.

Scenario 5: Resistance lower than anticipated (horizontal foundation failure)
A low effective soil stress, a high effective water pressure and an insufficient horizontal diaphragm capacity (design error or inadequate materials) may result in a lower resistance than anticipated. Due to the

monolithic design, the severity of initial failure is classified as level 1. The severity of damage is classified as level 2.

Scenario 6: Solicitation higher than anticipated (horizontal foundation failure)
A higher horizontal solicitation can be the result of local excavations, extreme surface loads and horizontal overload via the shear walls (extreme wind loads). The severity of initial failure is classified as level 1. The severity of damage is classified as level 2.

Scenario 7: Resistance lower than anticipated (rotational foundation failure)
A lower resistance than anticipated can be caused by a local lower diaphragm resistance (different soil conditions), local inadequate materials and local liquefaction of the soil. It's expected that the probability of local failure is greater than global failure and the severity of initial failure is classified as level 2. If one of these risks (or multiple) does occur, a large amount of damage may be expected. Therefore, the severity of damage is classified as level 3.

Scenario 8: Solicitation higher than anticipated (rotational foundation failure)
Local higher loads from the shear wall (local overload) or larger bending moments (extreme wind load) may result in a higher solicitation than anticipated. The solicitation should be very large to create rotational failure and therefore severity of initial failure is classified as level 2. When the solicitation is high enough, a large amount of damage may be expected. Therefore, the severity of damage is classified as level 3.

Scenario 9: Resistance lower than anticipated (collapse of one story)
A lower resistance can be caused by design errors (wrong dimensions/assumptions), production errors, inadequate materials and due to physical conditions (fire load). The severity of initial failure is classified as level 2, since the probability is low that the floors do not meet the requirements. The severity of damage is classified as level 4, since the dynamic load of a collapsing floor may threaten all the underlying floors.

Scenario 10: Solicitation higher than anticipated (collapse of one story)
A higher solicitation than anticipated could originate from horizontal overload (extreme wind load and impacts/collisions) or vertical overload (too much vertical weight, explosions and impacts). The severity of initial failure is classified as level 3. This severity is larger than the severity of the resistance, since a higher solicitation is more likely. The severity of damage is classified as level 4, since the dynamic load of a collapsing floor may threaten all the underlying floors.

Scenario 11: Failure facade column (floor support failure)
In contrast to the other scenarios, this scenario is not yet divided into two groups (resistance lower than anticipated and solicitation higher than anticipated) at this level. This is because more events lie in between the low resistance (or high solicitation) and the failure of a facade column. For example column overload and failure of the horizontal or vertical support. An explosion, fire, collision, design errors and inadequate materials might cause the facade column to fail. The severity of initial failure is classified as level 3, since many aspects may cause a failure. The severity of damage is classified as level 4, since the collapse of a column might cause the floor to fail.

Scenario 12: Failure multiple facade columns (floor support failure)
The severity of initial failure of multiple columns is also classified as level 3, because an explosion or fire will likely take out multiple columns at the façade. The severity of damage is classified as level 5, since the collapse of multiple columns will cause the floor to fail.

The severity levels are summarized in Table 21.

Table 21 Relative risk level

Scenario	Severity (if)	Severity (rd)	Relative risk level
1	2	5	7
2	3	5	8
3	1	2	3
4	1	2	3
5	1	2	3
6	1	2	3
7	2	3	5
8	2	3	5
9	2	4	6
10	3	4	7
11	3	4	7
12	3	5	8

It can be concluded that scenario 1 & 2 (shear wall failure), 9 & 10 (collapse of one story) and 11 & 12 (floor support failure) contain the largest relative risk. Within scenario 1 & 2 and 9 & 10, the solicitation contains the highest relative risk level. This is because a higher solicitation than anticipated is more likely to occur than a lower resistance than anticipated.

The next step of the risk evaluation is to compare the relative risks to a level of acceptable relative risk for every scenario. No quantitative or relative acceptable risk level is available and therefore this is expressed verbally:

Scenario 1 & 2: These two event scenarios contain a relative risk level that is too high (7/10 and 8/10). Basic events such as design errors, collisions, explosions should be prevented and the resulting progressive collapse should be mitigated.

Scenario 3 to 6: These four event scenarios contain a low level of relative risk. The monolithic design of a foundation has a positive influence on the integrity and the foundation will act as a coherent structure. Nevertheless, this level can't be ignored because due to the location of the foundation it's difficult to verify the result afterwards (the entire building is constructed on top of it). Measures should be taken to reduce the relative risk level.

Scenario 7 & 8: The rotation of the foundation contains a higher relative risk level compared with the previous four scenarios, because it's more likely that local failure will occur and the resulting damage is more severe. The relative risk levels should be reduced.

Scenario 9 & 10: The relative risk level connected to the event scenario of a collapsing floor is too high to be accepted. Design errors and inadequate materials should be prevented and the solicitation of vertical overload, explosions and fires should be reduced.

Scenario 11 & 12: The failure of a single façade column (or multiple) results in an unacceptable high relative risk level. Actions have to be taken to reduce this level

It can be concluded that all the scenarios should be taken into account during the design and actions have to be taken to reduce the relative risk levels. The required actions to accomplish an acceptable relative risk level are discussed in the next section.

12.4.6 Step 5: Risk treatment

The risk associated with the twelve scenarios cannot be accepted and measures have to be taken to mitigate them. Section 12.2 provides many solutions and for every scenario several recommendations will be made. Which measure or combination of measures will be used for the final design cannot be decided yet since no design has been made.

Scenario 1: Resistance lower than anticipated (shear wall failure)
There are five solutions to reduce the relative risk level of scenario 1: prevent or reduce the load (fire load), design the structure with enough robustness (design errors and inadequate materials/prefab elements), construct a second load bearing system (reduce resulting damage), use prescribed rules for a coherent structure (reduce resulting damage) and apply non structural measures (design errors and inadequate materials/prefab elements).

Scenario 2: Solicitation higher than anticipated (shear wall failure)
For the second scenario there are six possibilities to mitigate the relative risk level: prevent or reduce the load (impacts and explosions), design the structure with enough robustness (design errors and skewness), design the structure to resist the load (impacts, explosions and extreme win loads), construct a second load bearing system (reduce resulting damage), use prescribed rules for a coherent structure (reduce resulting damage) and apply non structural measures (design errors).

Scenario 3: Resistance lower than anticipated (vertical foundation failure)
The monolithic foundation has a low relative risk level. Nevertheless, it’s still advised to design the foundation with enough robustness (design errors and inadequate materials/concrete mixture) and to apply non structural measures (design errors). The use of prescribed rules will create a more coherent and ductile structure, which will reduce the severity of the resulting damage.

Scenario 4: Solicitation higher than anticipated (vertical foundation failure)
Design errors may result in a higher load on the structure or more negative adhesion. Enough robustness and non structural measures in combination with prescribed rules will reduce the relative risk to an acceptable level.

Scenario 5: Resistance lower than anticipated (horizontal foundation failure)
A low effective soil stress, a high effective water pressure cannot be reduced or prevented and the structure should be designed to resist the load. Designing the structure with enough robustness and non structural measures will reduce relative risk of design errors. The use of prescribed rules will create a more coherent and ductile structure, which will reduce the resulting damage.

Scenario 6: Solicitation higher than anticipated (horizontal foundation failure)

Horizontal loads due to local excavations or local extreme surface loads can be prevented or reduced. The structure can also be designed to resist the load. The previous solution also applies for large horizontal loads from the core (extreme wind loads). Prescribed rules will create a more coherent and ductile structure.

Scenario 7: Resistance lower than anticipated (rotational foundation failure)
This event scenario is mainly based on local events. Using non structural measures will ensure a low probability of local failure (local lower diaphragm resistance or local inadequate materials). Using prescribed rules and designing the structure with enough robustness will ensure a low relative risk level for the resulting damage.

Scenario 8: Solicitation higher than anticipated (rotational foundation failure)
This event scenario is caused by high local vertical loads from the shear walls (local overload on the floor) or large bending moments from the shear walls (extreme wind conditions). Local overload can be prevented or reduced or the foundation can be designed for these large loads. Using prescribed rules and designing the structure with enough robustness will ensure a low relative risk level for the resulting damage.

Scenario 9: Resistance lower than anticipated (collapse of one story)
A lower resistance (collapse of one story) can be caused by design errors (wrong dimensions/assumptions), production errors, inadequate materials and an inadequate state of the floor (fire load). Non structural measures should be taken to prevent a lower resistance. Using prescribed rules, designing the structure with enough robustness and constructing a second load bearing system will ensure a low relative risk level for the resulting damage.

Scenario 10: Solicitation higher than anticipated (collapse of one story)
A higher solicitation than anticipated could originate from horizontal overload (extreme wind load and collisions) or vertical overload (too much vertical weight, explosions and collisions). The severity of collisions and explosions can be limited by prevention or reduction of the load. Using prescribed rules, designing the structure with enough robustness and constructing a second load bearing system will ensure a low relative risk level for the resulting damage.

Scenario 11: Failure facade column (floor support failure)
An explosion, fire, collision, design errors and inadequate materials might cause the facade column to fail. Prevention or reduction of the load, designing the structure to resist the load and non structural measures can be used to limit the severity of initial failure. Prescribed rules, designing the structure with enough robustness and constructing a second load bearing system will reduce the severity of the resulting damage.

Scenario 12: Failure multiple facade columns (floor support failure)
The measures for a single column also apply for multiple columns, but more attention should be applied to the severity of initial failure and the resulting damage. For example, the second load bearing system is in this case of more importance than with the failure of one column.

To reduce the relative level of risk, all the strategies from the Eurocode are used, except one: design a key element. A key element is applied when it's physically impossible or economically unattractive to construct a second load bearing path. This risk analysis is performed before the design is made and every solution can still be applied. When the risk analysis is executed in a later stage of the design process, key element may become necessary. Besides the six strategies of the Eurocode, a seventh strategy is applied: non structural measures. This strategy is mentioned by the NTA HGBW part 3: Structural safety.

12.4.7 Step 6: Modification and revision

Since no structural design has been made yet, no modifications or revision can be applied.

12.4.8 Step 7: Presentation of the results

The Zalmhaven tower is a CC3 structure and a risk analysis has to be performed. A Fault Tree Analysis is used for the risk analysis and the fault tree can be found in report 2 of the literature study. Executing a FTA is very laborious, but it provides the engineer with much needed information. It should be noted that the risk analysis is not able to identify all relevant hazard. It’s likely that events or relations between events and failures are overlooked and as a result (important) failure modes may be forgotten.

Besides the identification of hazards (at the bottom of the tree), the Fault Tree Analysis diagram also show (partial) failure paths. With increasing length, the structure becomes more robust and ductile. This is because the progression of an event scenario may be stopped at every intersection. The increasing length of the failure paths also results in more different relations between the events and it becomes more difficult and time consuming to determine the amount of resulting damage.

An important aspect of a Fault Tree Analysis is that is utilizes Boolean logic [Wikipedia 2012]. Boolean logic cannot clarify the degree of failure: an element fails or it doesn’t. A level in between, for example a large deformation of a floor just before failure, doesn’t exist and warning signs cannot be taken into account.

The simplifications and assumptions made for the risk analysis can be found in section 12.4.2 at Step 1. With these simplifications and assumptions the probability of initial failure and the resulting damage hav been quantified with a relative scale. The results can be found in Table 22.

Table 22 Relative risk level

Scenario	Severity (if)	Severity (rd)	Relative risk level
1	2	5	7
2	3	5	8
3	1	2	3
4	1	2	3
5	1	2	3
6	1	2	3
7	2	3	5
8	2	3	5
9	2	4	6
10	3	4	7
11	3	4	7
12	3	5	8

The relative risk levels of the twelve scenarios are unacceptable high and measures have to be taken. Determining the degree of acceptance per scenario is quite difficult, since it’s not an exact science and the public’s perception also plays an important role (an airplane killing 100 people has more social impact than 100 car crashes each killing 1 person). Scenario 3 until scenario 6 have a relative risk level of only 3, but simple solutions during the design and construction (for example a good quality control system) might prevent expensive solutions after the completion. Scenario 7 and 8 also contain a low relative risk level, but more actions are required. Scenario 1, 2, 9, 10, 11 and 12 contain the highest relative risk level and one or multiple solutions may be applied to reduce the risk and occurring damage to an acceptable level.

The previous risk analysis was performed before any prefab structural design was made. Therefore the risk analysis only contains global risks (failure of the foundation or shear wall) and several global causes have been examined (the solicitation is too high or the resistance is too low). When the prefab structure is compared to a monolithic variant, the global failure modes remain identical. When the comparison is executed on a local scale, an important difference can be distinguished: the connections. Prefab distinguishes itself from cast in situ by the connections, resulting in a les coherent structure. Although the starter bars connect the different prefab elements, the amount of reinforcement is commonly lower than within the element and the smooth surface of the elements prohibits a proper bond between the two elements. Therefore the structural properties of the connection will always be lower than the surrounding concrete (this mainly refers to the tensile properties). It may be concluded that the connections are the strength and weakness of prefab concrete when compared to cast in situ concrete. Therefore the division and properties of the connections should be an important aspect of the structural design of a prefab building.

The sensitivity of the outcome to variations in the input is still very large for the current analysis. This is because the analysis is performed before the actual structural design is made. When this analysis is performed in a later or final stage, the outcome will become less sensitive to the input. One might wonder if it’s recommended to perform a risk analysis before any structural design is made. The answer lies in the amount of experience of the engineer. An expert who has performed many risk analyses might already know which risks should be mitigated and what kind of unknown extraordinary load may occur. A junior engineer lacks experience and a preliminary risk analysis may provide important insights and knowledge.

12.5 Conclusion

Progressive collapse is an important aspect of the structural design. Several structures have collapsed due to (un)foreseen situations and the resulting extraordinary loads. Depending on the consequence class, different measures have to be taken. For CC1 structure no extraordinary loads have to be considered and for CC3 structures a systematic risk analysis is obligatory. The reason for this enhanced level of security is not that these structures are less predictable than CC1 and CC2 structures, but because the consequences of a collapse are far greater. To prevent casualties in the future, the Eurocode distinguishes two different strategies for extraordinary design situations at CC3 structures. The first strategy provides three solutions for known extraordinary loads (for example explosions, collision, fires, extreme weather and earthquakes):

1. prevent or reduce the load,
2. design the structure with enough robustness,
3. design the structure to resist the load.

The second strategy provides three solutions for unknown extraordinary loads (design and execution errors, material defects, terrorist attacks and abuse by users):

1. construct a second load bearing system,
2. apply key elements,
3. use prescribed rules to provide a coherent and ductile structure.

A fourth solution for unknown extraordinary loads is specified in the “NTA HGBW part 3: Structural safety”: apply non structural measures (for example: quality assurance, prevention of mistakes, elimination of already made mistakes and social control).

To perform the previous mentioned risk analysis, the following seven steps have to be executed:

- step 1: definition of scope and limitations,
- step 2: qualitative risk analysis,
- step 3: quantitative risk analysis,
- step 4: risk evaluation,
- step 5: risk treatment,
- step 6: modification and revision,
- step 7: presentation of the results.

With these steps, a risk analysis is performed for the Zalmhaven tower. Although the prefab structure has yet to be designed, several interesting conclusions could be made. The shear walls in combination with the facade columns and the floors contain the highest relative risk. To prevent disproportional collapse, measures should be taken to mitigate the relative risk level. A second load bearing system is a good example that will reduce the severity of the resulting damage. To reduce the severity of initial failure, the extraordinary loads have to be prevented or reduced. The foundation contains the lowest relative risk level and non structural measures in combination with a high robustness will reduce this relative risk to an acceptable level.

The previous solutions are based on a global scale for a precast structure. When the structure is compared with a cast in situ variant, the global risks remain comparable. On a local scale an important distinction can be made: the connections. The connections are the strength and weakness of prefab concrete and the division and properties should be an important aspect during the upcoming structural design.

Performing a risk analysis provides insight to how the structure performs and reacts. Failure paths are discovered and progressive collapse is prevented. On the other hand, it should be noted that the risk analysis is not able to identify all relevant risks. It’s likely that events or relations between events and failures are overlooked and as a result (important) failure modes may be forgotten. A second important aspect of a risk analysis performed in an early stage is the sensitivity of the outcome to variations in the input. Because only very little is known and everything could still change, the sensitivity to variations is still very high. This has a positive effect: simple changes can be applied to prevent expensive solutions after the construction, but the high sensitivity has also a negative influence: nothing is certain and the applied measures may not be required when the structure is finished. This is a common problem from the design paradox and the risk analysis should constantly be updated during the design and construction process.

Part 2: Construction methodology

In part 2 of the literature study, the construction methodology will be elaborated. With the construction methodology the following is meant: the entire construction process, including the building method, element configuration, transport from the factory to the final location, tolerances and the planning. Part 2 will start in chapter 14 construction methodology criteria. By composing a list with criteria, different concepts can be evaluated. This list also provides insight in the aspects which are crucial for an optimal construction methodology. In chapter 14 the building method will be reconsidered with the previous mentioned criteria. In chapter 15 the transport system is examined and the influence factors in relation to the transport length are determined. Consequently several transport methods are considered (preliminary design) in chapter 16 and with the criteria of chapter 13 a (temporary) decision is made. In chapter 17 the tolerances are examined. Chapter 18 concludes part 2 with the cycle times.

13 Construction methodology criteria

A large part of the project cost are generated during the construction process of a building. These costs are composed out of personnel, equipment and materials. The design has a large influence on the cost and a small change can make a large difference. Therefore one of the primary structural design criteria is cost. During the structural design of the project, it's important to understand what the consequences are of different design alternatives and how they influence the construction of the project. Therefore it's impossible to create an optimal structural design without taking the construction methodology into consideration. In chapter 6 criteria can be found for an optimal structural design. This chapter will discuss criteria for the construction methodology. By defining the criteria first, different construction methodologies can be optimized during the design. This will result in better solutions and the criteria may also provide guidance for decisions between different concepts.
The reader CIE4170 Construction Technology of Civil Engineering Projects [Horst 2011] is used to create this chapter.

13.1 Primary criteria

The criteria are divided into two groups: primary and secondary criteria. The following primary criteria are essential to achieve an optimal and integrated construction methodology.

<u>Cost</u>
As stated before, the construction process has quite an impact on the overall project cost. Therefore the construction should be simple, fast and efficient. The costs for material can be divided into three main cost components:

- Formwork and falsework: the high price for formwork and falsework is mainly based on the skilled labour that is necessary to place and modify it. In Western Europe, the price for skilled labour is increasing faster than the price for materials (there's a factor 10 between the prices). As a result, optimizing formwork and falsework is more important than ever. This is illustrated in Figure 149. Two examples of this optimization are sliding formwork and prefab concrete (the formwork and falsework is placed in a relative small controlled area). The amount of required personnel is reduced, as well as the construction time. A third optimization is the standardization of formwork, which results in a reduced price per square meter.
- Concrete: the price for concrete is relatively low compared to reinforcement, formwork and falsework. Although the price for material has increased, it did not grow as fast as the price for skilled labour. To optimize the cost, waste products can be added to the concrete (for example blast furnace slag). By using new casting and compaction techniques (for example self compacting concrete) or a sophisticated management for the concrete hardening process, the costs are reduced even more.
- Reinforcement steel: the reinforcement is responsible for one third of the total price. This is relatively high if one takes in consideration that only 0.3 to 0.4% of the total weight of a wall is steel. The amount of required steel could be reduced by adding synthetic fibres that take over the role of reinforcement. To reduce the price for reinforcement, mats could be prefabricated, the design could be computer aided and the diameter and shape should be normalized.

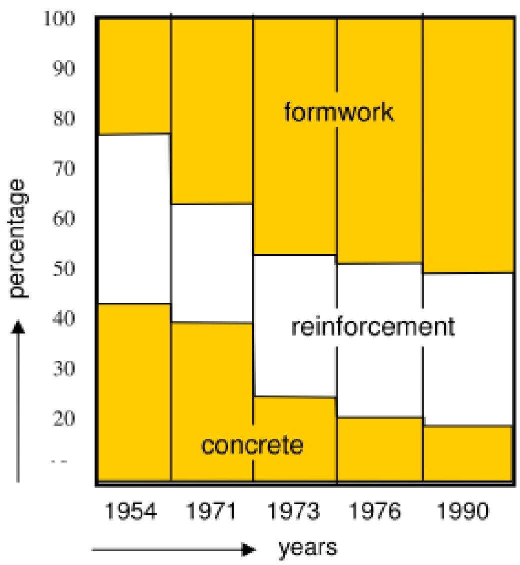


Figure 149 Contribution of cost components for concrete works in the building industry [Horst 2011]

In addition of the material costs, equipment also contributes to the total cost. For example: purchase costs (or rental costs), running costs, assembly and disassembly costs, interest costs and residual value (in case the equipment is purchased). Labour is the third cost factor that has the largest influence.

Quality
The demand for quality is increasing compared to the past and the construction methodology has a large influence on the final quality of the project. Despite of a high quality design, the end result may not be satisfying if the construction method, equipment and temporary structures do not meet the requirements. The attention to detail and quality assurance of construction workers also contributes to the total quality.

Risk
During the construction of the project, most delays are encountered. Large financial expenses in combination with low amount of influence and short time schedules make this a phase with high risks. Construction site management in combination with an advance planning should minimize these risks. Risks can be expressed in money and this makes comparing risks more viable. For example, applying a hoisting shed reduces the risk of delays (and as a result the construction time is reduced), but generally increases the initial investment. By expressing the risk in money, a hoisting shed can be compared with traditional cranes.

Time
Time is one of the most important factors of the construction methodology. If the deadline is not met, the contractor will receive large fines and the client is unable to exploit the project. The time criterion is interwoven with the three other primary criteria and it can be expressed in money, quality and risk. For example: when the construction time is reduced by 50%, the project interest is reduced and the client will receive revenues in an earlier stage. As a result of this time reduction, the time schedule will become more compact and delays will result in more risks. Therefore the construction methodology should be optimal for all three primary criteria.

The sensitivity and accuracy of the three primary criteria should be examined because they are based on estimations. During the first phases of the design, the sensitivity for errors may be very high and the accuracy very low. Decisions based on inaccurate criteria may result in incorrect solutions.

13.2 Secondary criteria

Redundancy and flexibility

Redundancy is the ability to absorb the unforeseen without disproportional damage, or the ability to cope with changing requirements without large adjustments. To prevent large adjustments, flexible systems should be applied. Gains in prefab elements with a diameter larger than the protruding reinforcement are a good example. This flexibility is essential for preventing delays and unnecessary costs.

Dependency

The weather is an important aspect for high rise buildings and a weather independent system is desirable. Dependency on other aspects, for example facade elements that have to be placed by tower cranes, should also be reduced as much as possible. This is because dependant elements or systems form a weak link in the construction methodology (they rely on other systems or elements for their success). The term sensitivity can also be used and a weather independent system will have a low weather sensitivity.

Repetition

Repetition is a key aspect for all prefab and cast in situ structures. Repetition reduces the construction time (learning effect), the costs and the risks (the elements become interchangeable). In other words, repetition has a positive influence on the primary criteria. Besides applying repetition at elements, it could also be useful for systems or methods. For example: constructing every floor in the same order.

Construction area

Besides time and money, the construction process also requires a certain minimal building area. Traditional systems with storage need more space than industrial variants with Just in Time delivery. According to the requirements, an optimal choice should be made for every location. Because many high rise projects are constructed in densely populated areas, systems with a small footprint are preferred. At the JuBi project in The Hague, they used a logistical manager, a building site ticket and JiT because the free space was extremely small: there was only a strip of 10m between the facade and the street [Herwijnen 2011]. To make traffic possible in this 10m, the tower cranes were placed on a portal structure (see Figure 150). The access, storage and manoeuvrability at the site depend on the construction methodology.

Environmental impact

The construction of a high rise building results in nuisance for the surroundings. Large quantities of waste material have to be removed, traffic problems occur and the construction creates high noise levels. Demands that limit the negative effects for the surroundings are increasing and nuisance should be taken into consideration during the construction. For example, entirely prefabricated structures reduce the amount of intermediate products and as a result the amount of waste material is reduced (packaging of separate products).

Figure 150 Portal structure for the tower cranes [Herwijnen 2011]

The primary and secondary criteria mentioned before can be considered as general criteria, applicable for the entire construction methodology. For the transport system at the building site the primary and secondary criteria are too general and it becomes difficult to judge different concepts. Therefore these criteria will be examined in more detail by determining the transport characteristics and the boundary conditions.

With the four primary criteria, one question remains: which criterion is leading? The answer to this question isn't straightforward because the primary criteria are interwoven. Therefore the four primary criteria should be considered as a whole.

14 Building method

In chapter 5 a vision was given for the construction methodology. This vision contained two aspects: the building method and the transport system. For the building method prefab elements with a masonry configuration were considered to be the best solution. The first part of the literature study didn't provide any reasons to change the building method. Because of the new acquired construction methodology criteria this decision will be shortly re-examined.

The first primary criteria is costs. On average, a prefabricated element is more expensive than an identical element cast on site. As a result, prefabricated projects are often more expensive than cast in situ projects. Because a higher quality (controlled environment), lower risk (outsourcing of work) and a shorter construction time (assembling instead of constructing) can be achieved, the higher costs may be counteracted. Because the division of these aspects is highly project specific, it may not always be the case but often the lower risk and shorter construction time result in an economically more attractive project.

From the secondary criteria, a cast in situ structure scores better on redundancy or flexibility. This is because many design aspects can still be changed until the concrete is poured and errors or deviations are more easily adsorbed. With prefab concrete, more information is required at an earlier stage and everything has to fit the first time, every time. Furthermore, cast is situ structure are less crane dependant. When the weather dependency is considered, prefab with the small amount of wet connection has an advantageous over cast in situ concrete. Both methods contain a high amount of repetition and can be considered equal. When the construction area is examined, prefab concrete contains a slight advantageous over cast in situ structures because the elements only have to be assembled (requiring less equipment and personnel). When the environmental impact is analysed, the assembling process of prefab concrete also provides more benefits, resulting in less waste and noise nuisance for the surroundings. Because the dimensions of prefab elements are larger than that of liquid concrete, the city centre of Rotterdam may encounter more nuisance from the higher amount of transport.

When all the aspects are considered, it may be concluded that prefab concrete provides many benefits, but there are also disadvantageous. Per project it should be considered if the benefits (mainly quality, risk and construction time) outweigh the disadvantageous. Because the cast in situ design is already made by Zonneveld ingenieurs, it becomes very interesting to examine if prefab concrete is able to provide the considered benefits. Therefore the prefab building method will be applied in this thesis.

15 Transport systems

The transport system has a considerable influence on the building process. In this chapter insight is provided in the relations between these two aspects, the corresponding influence factors and the properties. The building height in relation to the transport system and building process is a key aspect in this chapter.
A transport system includes all the means necessary to transport a product from the factory to its final position. This physical flow of products can be divided into three separated phases:

- phase 1: from the factory to the building site,
- phase 2: vertical transport from the building site to the construction floor,
- phase 3: horizontal transport on the construction floor to the final position.

The influence factors of phase 2 and 3 will be discussed first in section 15.1 because these aspects have already been examined by [Meij 2012]. Based on this information, phase 1 will be analysed in section 15.2. This chapter will end with a conclusion in section 15.3.

15.1 Influence factors for phase 2 and 3

Phase 2 and 3 start when the materials arrive at the building site. To compare the possible solutions on a qualitative level, an understanding of the influence factors is required. This is done by determining the transport system properties, the building process properties, the internal relations and the boundary conditions. These four aspects are explained in more detail below:

- **Transport system properties:** the characteristic properties of the transport system, for example the load capacity or transport speed.
- **Building process properties:** the characteristic properties of the building process which depend on the transport system, for example the transport time or the amount of loads that have to be transported (elements, material and equipment). The building process properties determine the construction time.
- **Internal relations:** the relations between the design and transport system properties (the wind sensitivity increases with the building height) and the relations between the design and the building process properties (a large building height result in a longer transport time). There is also an internal relation between other production factors (the construction team) and the building process properties: dethatching of the elements (afpikken in Dutch).
- **Boundary conditions:** the influence of the surroundings and the regulations on the transport system and building process properties.

The four aspects are also clarified in a relation diagram in Figure 151.

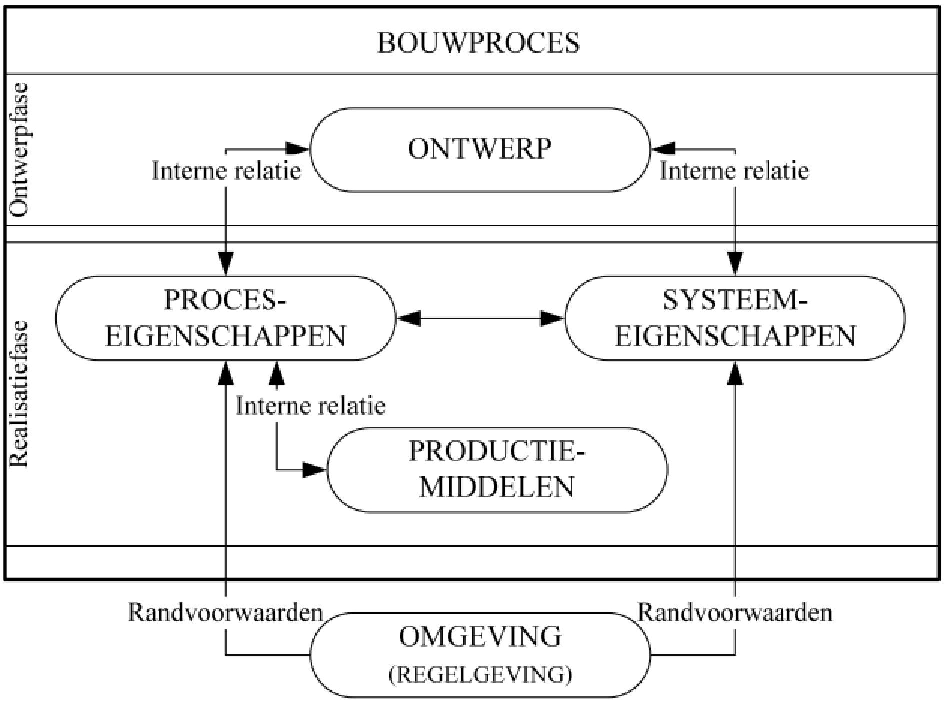


Figure 151 Relation diagram for phase 2 and 3 [Meij 2012]

The transport system properties and building process properties will be discussed in section 15.1.1 and 15.1.2. The internal relations and boundary conditions are determined for both properties and are included in the corresponding section. In section 15.1.3, the transport system properties are linked to the building process properties. This section will end with the building process properties for the entire height in section 15.1.4 and a conclusion in section 15.1.5.

15.1.1 The transport system properties

The transport system properties are the characteristic properties of the transport system. Five different properties can be distinguished:

1. Load capacity: the maximum weight that can be transported per cycle.
2. Load size: the maximum size that can be transported (width, height and length).
3. Transport speed: the speed of the loaded, unloaded, horizontal and vertical transport system.
4. Visibility of the transport: the amount of direct visual view of the operator on the load and the transport route.
5. Wind sensitivity: the response of the load (during transport) and the transport system to the wind. This is an important property that distinguishes high rise from low and mid rise buildings.

The transport system properties are limited by the boundary conditions. Three groups with boundary conditions can be distinguished: surroundings, design and legislation. These boundary conditions will be discussed per group.

Boundary conditions from the surroundings
One transport system property is limited by the surroundings:

- Factory capacity: the prefab concrete factory might limit the weight or dimensions of the element. In general, the transport capacity will be leading because most factories have a large capacity (Hurks Beton can produce elements up to 80 ton).

Boundary conditions from the design
The transport system properties are also affected by the design. Because of this influence, the transport system should be considered during the structural design. The following design aspects will influence the transport system:

- Structure: the element layout of the structure determines the size and mass of the elements. A large element size and mass is beneficial for the stability of the structure, but may conflict with the load and size capacity of the transport system.
- Degree of prefabrication: a high degree of prefabrication could lead to an increased amount of material that has to be transported (prefab floors instead of cast in situ floors). It's also possible that weight of the elements increases (sandwich elements instead of concrete load bearing inner leafs).
- Building layout and repetition: the amount of unique elements is determined by the layout and the repetition factor.

Boundary conditions from legislation
The national legislation is the only direct boundary condition that influences the transport system properties. The following limitations are encountered [Wikipedia 2012]:

- the maximum height is 4m,
- the maximum width is 2.55m,
- the maximum length is 16.5m (truck with trailer),
- the maximum Gross Vehicle Weight (GVW, this is the weight of the vehicle, driver,
  passenger(s) and cargo) is limited to 50 ton.

These conditions apply per truck. If the transport system is designed for a higher capacity, two elements should be delivered and vertically transported at once. When transport by water is applied, no limitations are encountered.

There are also two aspects that have an indirect influence: the Eurocode and the requirements for the submission of a building permit.

The Eurocode provides boundary conditions for the wind sensitivity via the code NEN-EN 1991-1-4. Trough a complex calculation, wind speeds are converted to loads which affect the wind sensitivity. The Eurocode also provides boundary conditions for the safety (material factors and limit states).

In 2003 VROM (Ministry of Housing, Spatial Planning and the Environment) introduced requirements for the submission of a building permit (besluit indieningvereisten aanvraag bouwvergunning in Dutch). These requirements are part of the construction safety plan and should prevent safety hazards outside of the building area. In the construction safety plan a distinction is made between the construction site, construction safety/danger zone and the transport zone (hijszone in Dutch). For vertical transport of prefab elements, the following rules apply:

- The transport zone has an area that is at least equal to the object that is transported and this is supplemented with the transport height related construction safety zone. In The Hague the construction safety zone is defined as 1/10 of transport height plus 2m. The maximum transport height is limited to 200m.
- If the object is able to rotate, the transport zone will be equal to the largest dimension of the object in both directions.
- The object may not be rotated or transported above a public street or used structures.

The legislation applied in The Hague (1/10 of transport height plus 2m) is determined by the maximum horizontal deflection due to the wind. When measures are taken to reduce this deflection (for example a guided transport), the construction safety zone may be reduced.

Internal relations in the transport system
[Meij 2012] created a relation diagram for the internal relations, which is depicted in Figure 152.

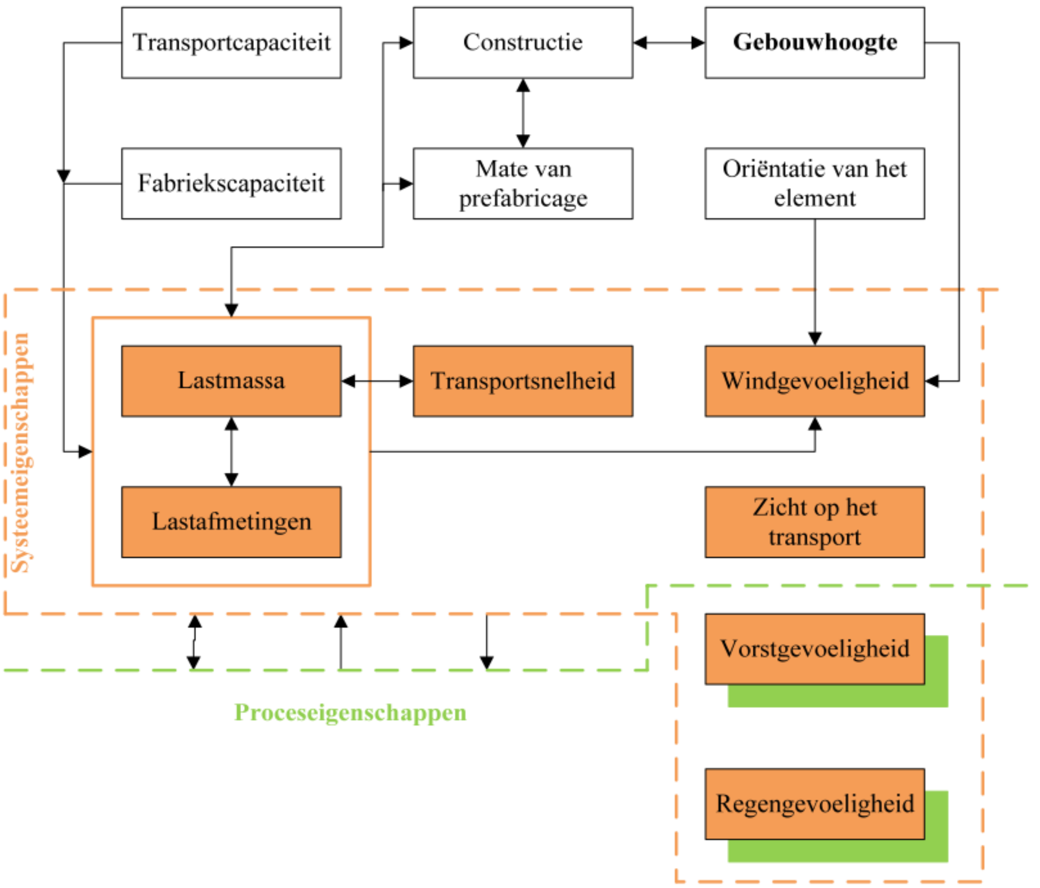


Figure 152 Internal relation diagram for the transport system properties [Meij 2012]

The orange blocks in Figure 152 represent the transport system properties, the green blocks represent the transport process properties and the boundary conditions are represented by the white blocks. The relations between these three aspects are indicated by the arrows. Four important relations can be identified in this relation diagram:

1. The building height and the orientation influence the wind sensitivity of the transport system. With an increasing building height, the hoisting cable will also increase. Accordingly, the wind sensitivity of the transport system will increase while the weather conditions remain equal[18]. Depending on the orientation of the element, the load area may change.
2. With increasing heights, the wind speed will become larger. To prevent more delays because of the wind (windverlet in Dutch), the wind sensitivity has to be reduced.
3. The building height affects the structure. As the building height increases, the element size and mass should also increase in order to provide enough strength,

[18] Provided that the loads are transported outside of the building perimeter and no additional measures are taken (for example a guided transport system).

stiffness and stability. Subsequently, this has an effect on the capacity of the transport system.
4. The transport speed is influenced by the mass that has to be transported. An increased mass will result in a lower transport speed and vice versa.

By reducing the wall thickness over the height of the structure, it's possible to increase the size of the elements. Consequently, the amount of transport movements can be reduced. This has a positive effect on point 3 of the previous enumeration. On the down side, the repetition factor will be reduced. In Figure 153 the relation between the mass and transport speed is shown.

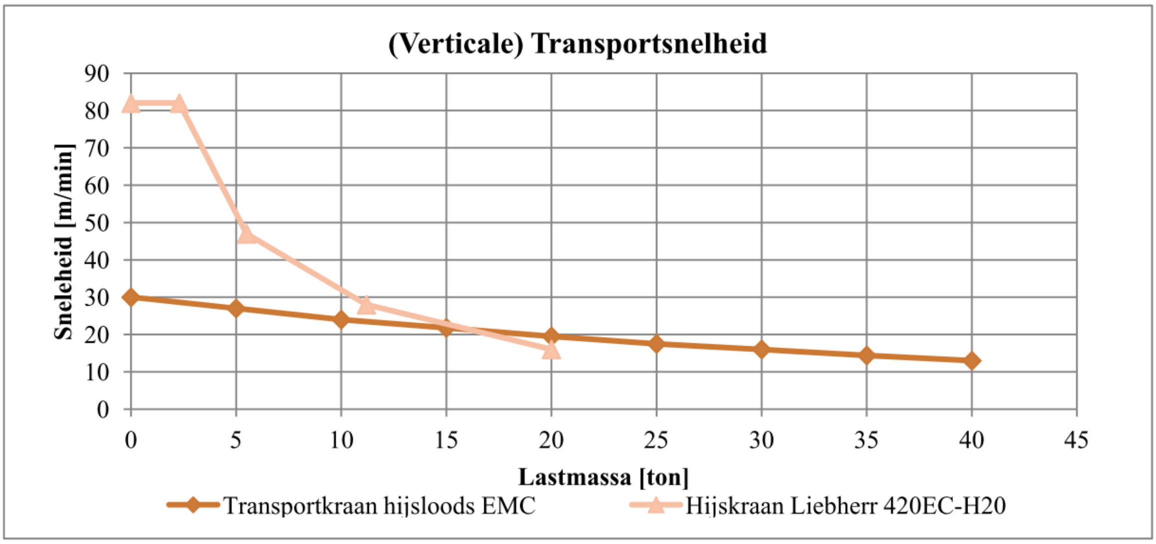


Figure 153 (Vertical) Transport speed [Meij 2012]

The Erasmus MC crane has a relative low transport speed because an overhead gantry crane is used. With this gantry crane it was impossible to use multiple gears. The speed curve of the Liebherr crane is characteristic for normal cranes[19].

To reduce the construction time as much as possible, the number of loads within a certain time period should be as large as possible, i.e. the efficiency of the transport system must be optimal. To calculate the efficiency, the transport speed is multiplied by the load that is transported. Figure 154 shows that the transport system has the highest efficiency when the transported load is maximal.

[19] The Liebherr 420EC-H 20 has a maximum load capacity of 20 ton. For a correct comparison a Liebherr 630EC-H 40 or any other tower crane with a capacity of 40 ton should be used.

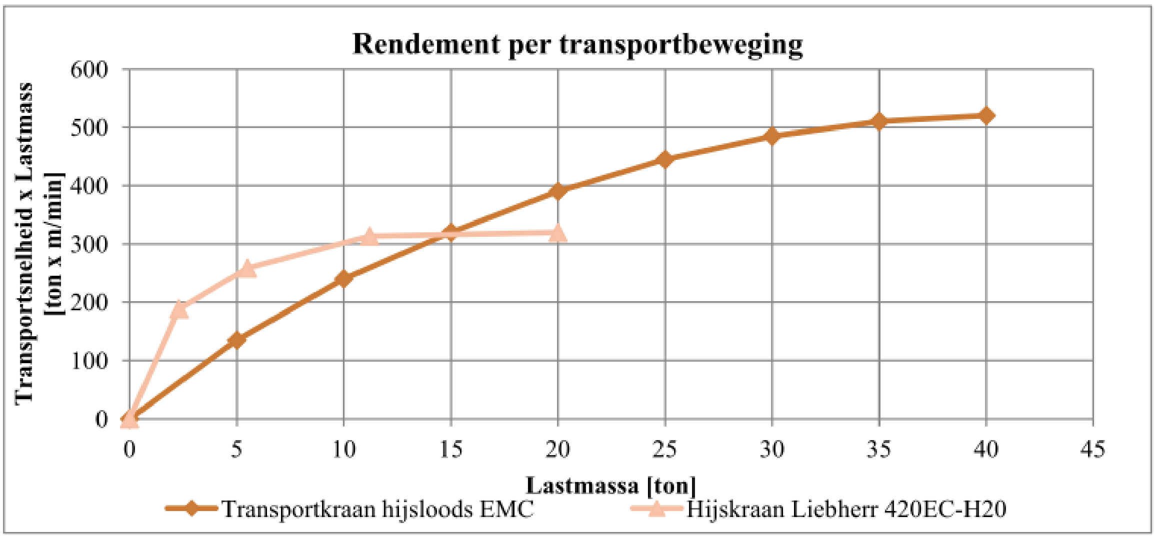


Figure 154 Efficiency per transport movement [Meij 2012]

Furthermore, Figure 154 also illustrates that the relative slow overhead gantry crane of the Erasmus MC has a higher efficiency at large loads (>15 ton).

15.1.2 Building process properties

The building process properties, which depend on the transport system, determine the utilization ratio (bezettingsgraad in Dutch) and the construction time. The four properties are defined per storey:

1. <u>Transport time:</u> the time that is required to transport an element horizontally or vertically, including the return time. The lead time of the actions are excluded. The vertical transport time is an important property which distinguishes high rise from low and mid rise buildings.
2. <u>Lead time of the actions:</u> the time that is required before the following action can take place. This time is composed out of several intervals: attaching and detaching, positioning, adjusting and stabilising of the element. This aspect is also known as the required time for the crane related actions.
3. <u>Amount of elements that have to be transported:</u> the amount of prefab elements that are required per floor level.
4. <u>Amount of other materials that have to be transported:</u> the amount of other materials that are required per floor level. For example scaffolding, mortar, equipment, rebar and struts.

There are two properties left that may be considered as a building process and transport system property:

1. Sensitivity for frost: the extent to which low temperatures affect the realisation. Adjustments in the transport system properties may protect sensitive actions and as a result the construction time could be reduced (transport system). Vice versa, reducing the amount of sensitive actions (building process) will affect the transport system.
2. Sensitivity for precipitation: the extent to which the precipitation affects the realisation. In consensus with the sensitivity for frost, this property is also affected by the building process and transport system properties.

Boundary conditions from the surroundings
One building process property is limited because of the surroundings:

- Delivery of prefab elements: the transport method determines how the elements are attached. The orientation of the element during transport (by water or via the road) relative to the orientation during the vertical transport is an important aspect. The delivery principle (JiT or storage on site) also determines the possibility that an element is not available.

Boundary conditions from the production factors and the design
The building process properties are also affected by the design. Because of this influence, the building process should be considered during the structural design. Besides the design, the production factors also influence the building process. The following boundary conditions can be distinguished:

- Production rate of the construction team: the production rate determines the required time before a certain action is completed. A high and efficient production rate is beneficial for the construction time.
- Production of the prefab elements**:** the capacity, storage and available time determines if the concrete factory can produce enough elements. When the required production rate is not achieved, multiple factories can be used. In this thesis it's expected that this boundary condition will not result in any problems.
- Layout of the building: the amount of elements are defined by the dimensions of the building. With increasing dimensions, the horizontal transport time will become larger.
- Structure: the main load bearing system in combination with the element configuration, element properties and connections define the amount of (different) elements and other required materials that have to be transported.
- Degree of prefabrication: a relative high degree of prefabrication will reduce the required labour at the construction site. As a result, the sensitivity to precipitation and frost will be reduced. Furthermore, the degree of prefabrication is related to the amount of materials that have to be transported.
- Connections: the connection between the elements determines the lead time of several actions. This connection also influences also the sensitivity for wind and frost.

Boundary conditions from legislation
No direct legislation is available that will influence the construction process properties. There are two aspects that have an indirect influence: (structural) safety and noise nuisance.

The (structural) safety should be guaranteed at all times. Fall restraints, falling objects and safe scaffolds are a few examples that have to be considered. The construction site may be closed when the requirements aren't met.

The regulations relating to noise nuisance differ per municipality. Generally, the following rules apply:

- Construction activities are authorized between 7:00 and 19:00.
- During the evening and the night, no demolition activities (or other activities with a high noise level) are allowed within the vicinity of dwellings.
- Construction activities are not allowed on Sunday.
- An exemption of these rules is possible, when special circumstances apply.

Internal relations in the building process
Figure 155 shows the internal relation diagram of the building process properties. The orange blocks in Figure 155 represent the transport system properties, the green blocks represent the transport process properties and the boundary conditions are represented by the white blocks. The relations between these three aspects are indicated by the arrows.

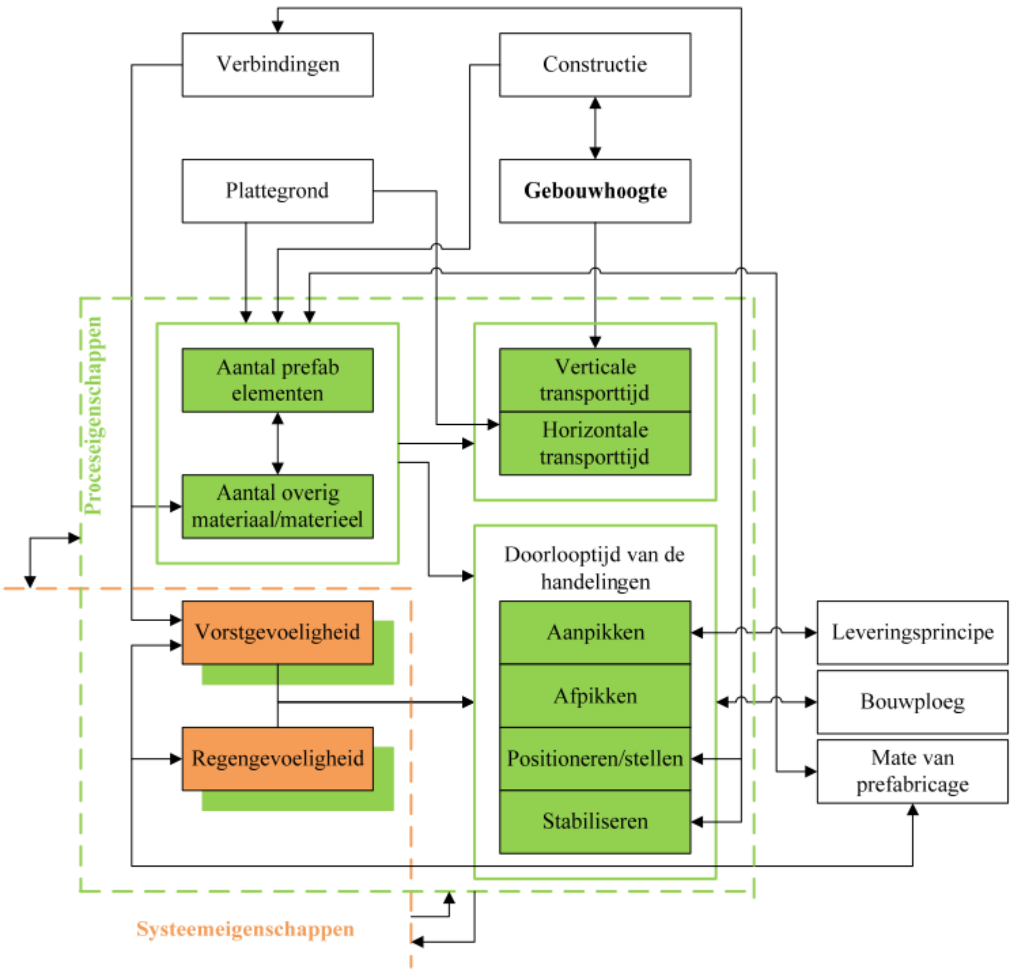


Figure 155 Internal relation diagram for the building process properties [Meij 2012]

Figure 155 shows that only one building process property depends on the building height: the vertical transport time. When the building height increases, the elements have to be transported over a longer distance, resulting in a longer vertical transportation time.

Besides this relation, there are also two relations between the building process properties:

- The amount of prefab and other materials has a positive relation with the transport time: a reduced amount of materials (larger loads) will result in a reduced transport time.
- The amount of prefab and other materials has also a positive relation with the lead times. When the amount of materials is reduced (larger loads), the lead time will be reduced as well.

It can be concluded that the optimization of the amount of materials has a positive influence on the other building process properties.

15.1.3 Interaction between transport system and building process properties

In the previous two sections, the internal relations between the boundary conditions and specific properties (transport system and building process) have been determined. The transport system and building process were separated and the relations between these two properties will be determined in this section.

The relation diagram is depicted in Figure 156. The orange blocks represent the transport system properties and the green blocks represent the transport process properties. The bold properties have a direct relationship with the building height.

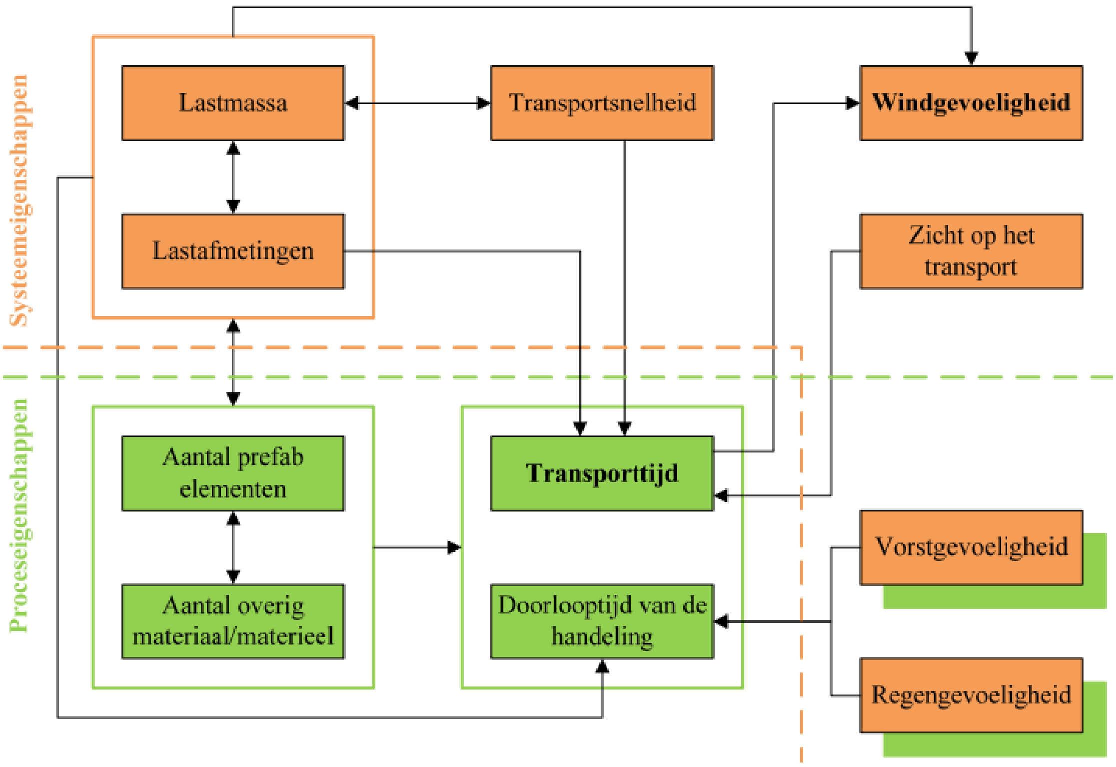


Figure 156 Relation diagram between transport system and building process [Meij 2012]

Three important relations that stand out with increasing building height:

1. Transport time: the transport time is influenced the size of the load (large loads are difficult to control), the transport speed, the visibility on the transport and the building height.
2. Wind sensitivity: the wind sensitivity depends on the mass and size of the load, the transport time and the building height. When the building height increases, the crane cable will become longer, the wind speed will increase and the transport time will become larger[20]. As a result of these three aspects, the wind sensitivity will increase. To limit the amount of delays, the wind sensitivity has to be reduced. Small and heavy loads or a guided transport system are two examples that reduce the wind sensitivity.
3. The amount of loads that have to be transported: the amount of loads (elements, material and equipment) are determined by the maximum mass per transport movement and the size of the load.

[20] Wind is a time dependant phenomena. A longer transport time will increase the change that higher wind speeds occur.

15.1.4 Building process properties for the entire height

Until now, the building process properties have been reviewed per floor level. For the realisation of the project it's relevant how the properties relate to the entire construction height and construction time. For this purpose, the temperature, precipitation and wind speed have to be included.

The results of the analysis performed by [Meij 2012] can be found in Figure 157.

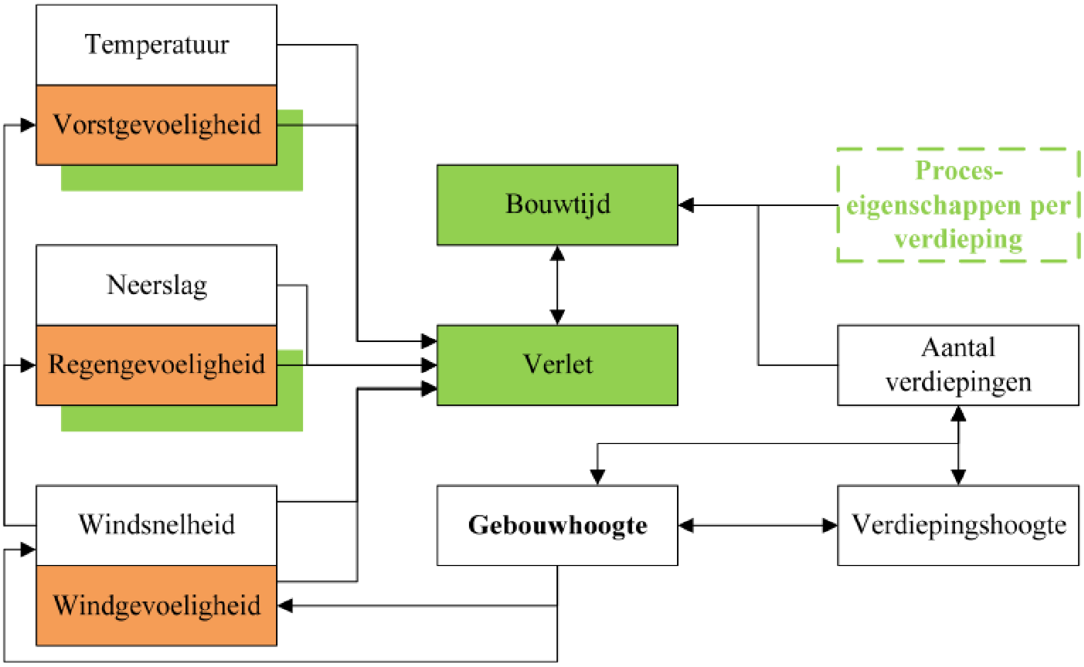


Figure 157 Relation diagram for the building process properties over the entire height [Meij 2012]

Three relations can be distinguished in the relation diagram which influence the building process properties over the entire height:

1. With an increasing building height, the wind speeds will become larger. In return, larger wind speeds will result in more wind delays (windverlet in Dutch)
2. Labour is a weather critical factor and the risk of frost and precipitation delays will increase with larger building heights (the delays are wind dependant).
3. The total construction time depends on the amount of floor levels. With a longer construction time, the risk of weather delays will increase (the delays are time dependant).

15.1.5 Conclusion

The transport system is a key component of the construction phase. Properties of this system will influence all the other phases and vice versa. It's important to have a thorough understanding of the (im)possibilities of a transport system in an early stage. When multiple transport systems are examined, the following aspects should be considered:

- the economical and technical feasibility of the transport system,
- the element size, mass and configuration.

When the construction process of a 200m prefabricated tower is compared with the construction process of low and mid rise prefabricated buildings, a relevant transport system property and a relevant building process property can be distinguished:

- the sensitivity for the wind,
- the vertical transport time.

Without a reduced sensitivity for the wind, the amount of delays will increase. This will threaten the construction time and the economical feasibility of the project. When a traditional vertical transport system is applied in a high rise project, problems might occur with the vertical transport time. The efficiency per transport movement has to be optimal to reduce the vertical transport time at a high rise project. A larger transport capacity doesn't necessarily mean that the transport time can be reduced. The best solution to achieve the highest efficiency is to transport the maximum load at every vertical movement.

In order to transport the maximum load every time, the capacity of the transport system should be adjusted to the mass and size of the elements. This adjustment is slightly complicated because the mass and size of the elements depends on several boundary conditions, whereby transport via the road results in the most limitations.

The mass and size of the elements are also indirectly influenced by the building height. An increasing building height requires more strength, stiffness and stability. To reduce the vertical transport time, the element thickness can be limited at higher levels. As a result, the mass can be reduced (multiple elements in one movement) or the element size can be increased (less movements).

Finally, the total construction time will also be influenced by the building height. Apart from the increasing number of floor levels, the amount of delays will also increase because:

- the probability of a delay is time dependant,
- the wind speeds increase with larger building heights,
- frost and precipitation delays are influenced by the wind speed (when the labour is weather dependant).

15.2 Influence factors for phase 1

Now the influence factors for phase 2 and 3 have been determined, it's possible to redo the analysis for phase 1. Phase 1 will start at the prefab factory and ends at the building site. There are two known solutions for this phase: transport by water or via the road. To compare both solutions on a qualitative level, the same four aspects are used:

- transport system properties,
- transport process properties (the term building is replaced by transport),
- internal relations,
- boundary conditions.

The transport system properties and transport process properties will be discussed in section 15.2.1 and 15.2.2. The boundary conditions and relations are included in the corresponding chapter. In contrary to phase 2 and 3, phase 1 is not determined per floor level, but over the entire transport length. Furthermore, there is no relation with the building height. In section 15.2.3, the transport system properties are linked to the building process properties. This section will end with a conclusion in section 15.2.4.

15.2.1 The transport system properties

The following three different transport system properties can be distinguished:

1. Load capacity: the maximum weight that can be transported per cycle.
2. Load size: the maximum size that can be transported (width, height and length).
3. Transport speed: the speed of the transport system when it's loaded.

The transport system properties are limited by the boundary conditions. Three groups with boundary conditions can be distinguished: surroundings, design and legislation. These boundary conditions will be discussed per group.

Boundary conditions from the surroundings

The following properties of the transport system are limited because of the surroundings:

- Factory capacity: the prefab concrete factory might limit the weight or the dimensions of the element. In general, the transport capacity via the road will be leading because most factories have a large capacity (Hurks Beton can produce elements up to 80 ton).
- Vertical transport capacity: the mass and size of the elements can be limited by the vertical transport system (phase 2 and 3).

Boundary conditions from the design

Three design aspects will influence the transport system:

- Structure: the element layout of the structure determines the size and mass of the elements. A large element size and mass is beneficial for the stability of the structure, but may conflict with the load capacity and size of the transport system.
- Degree of prefabrication: a high degree of prefabrication could lead to an increased amount of material that has to be transported (prefab floors instead of cast in situ floors). It's also possible that weight of the elements increases (sandwich elements instead of concrete load bearing inner leafs).
- Building layout and repetition: the amount of unique elements is determined by the layout and the repetition factor.

Boundary conditions from legislation

The national legislation is the only direct boundary condition that influences the transport system properties. The following limitations are encountered [Wikipedia 2012]:

- the maximum height is 4m,
- the maximum width is 2.55m,
- the maximum length is 16.5m (truck with trailer),
- the maximum Gross Vehicle Weight (GVW, this is the weight of the vehicle, driver,
  passenger(s) and cargo) is limited to 50 ton.

These conditions apply per truck. If the transport system is designed for a higher capacity, two elements should be delivered and vertically transported at once. When transport by water is applied, no limitations are encountered. For the transport speed the same legislation applies as for all other vehicles and there are no special requirements.

Internal relations in the transport system

When the internal relations are studied, it can be concluded that there are no relations between the three transport system properties and the transport distance. A longer distance doesn't influence the load capacity or transport speed. There are two relations between the three transport system properties: the transport speed is influenced by the mass and size that has to be transported (an increased mass or size will result in a lower transport speed and vice versa).

15.2.2 Transport process properties

The transport process properties, which depend on the transport system, determine the utilization ratio (bezettingsgraad in Dutch) and the transport time. These properties are determined over the entire transport length and five properties have been defined:

1. Transport time: the time that is required to transport an element from the factory (or a JiT depot) to the building site, excluding the return time. The return times are excluded since multiple transport systems will be used. The lead time of the actions are also excluded.
2. Lead time of the actions: the time that is required before the following action can take place. This time is composed out of two intervals: attaching (or detaching) and making room for the next transport system (for example a truck).
3. Amount of elements that have to be transported: the amount of prefab elements that are required for one floor level.
4. Amount of other materials that have to be transported: the amount of other materials that are required per floor level. For example mortar and rebar.
5. Sensitivity for traffic jams: the extent to which traffic jams affect the transport process. Adjustments, for example JiT or storage at site, may reduce the sensitivity and resulting delays.

Boundary conditions from the surroundings

The following properties of the transport process are limited because of the surroundings:

- Orientation of the elements: the orientation of the element during transport (by water or via the road) relative to the orientation during the vertical transport is an important aspect. Elements may be transported vertical, horizontal or under an angle. The vertical transport system at the factory and the building site play an important role in the orientation.
- Traffic: the amount of traffic determines the transport time. Solutions might be necessary to reduce the transport time (local storage or JiT delivery from a nearby depot).
- Availability of the vertical transport systems: the time that the transport system has to wait, before elements or materials can be (un)loaded.

Boundary conditions from the production factors and the design

The transport process properties are also affected by the design and the production factors. The following boundary conditions can be extinguished:

- Operation speed of the ground personnel (attaching and detaching): a high operation speed results in a short lead time.
- Production of the prefab elements: the capacity, storage and available time determines if the concrete factory can produce enough elements. When the required production rate is not achieved, multiple factories can be used. In this thesis it’s expected that this boundary condition will not result in any problems.
- Structure: the main load bearing system in combination with the element configuration, element properties and connections define the amount of (different) elements and other required materials that have to be transported.
- Degree of prefabrication: the degree of prefabrication is related to the amount of materials that have to be transported.

Boundary conditions from legislation

No direct legislation is available that will influence the construction process properties, i.e. there are no rules that limit the transport time or the amount of materials that have to be transported.

In the case of exceptional transport (when the legislation boundary conditions of the transport system are exceeded), it’s preferred to transport the load during the night. This will reduce the impact on the traffic and there are indirect boundary conditions for the noise levels.

Internal relations in the transport process
There are two transport system properties that have a direct relation with the transport length: a longer transport distance will result in a longer transport time and an increased sensitivity for traffic jams. The other properties (lead time and amount of elements/materials) have no relation with the transport length. Between the properties, there is one relation: the transport time has a positive relation with the traffic jam sensitivity (when the transport time increases, the possibility of traffic jams increases as well).

15.2.3 Interaction between transport system and building process properties

When the relations between the transport system and building process properties are studied, two important relations can be distinguished:

1. Transport time: the transport time is influenced the size and the mass of the load (large loads are difficult to transport) and the transport speed. Extraordinary loads may have to be transport during the night.
2. The amount of loads that have to be transported: the amount of loads (elements, material and equipment) are determined by the maximum mass per transport movement and the size of the load.

15.2.4 Conclusion

A transport system for phase 1 is comparable with a transport system of a low or mid rise building. The only differences are the increased mass, size and amount of the elements.

During the structural design, boundary conditions for the mass and size of the elements should be considered. The National legislation limits the dimensions and the mass of the transport system properties and this is directly related to the structural properties.

The sensitivity of the horizontal transport system to traffic increases when more loads are transported over a longer distance. To prevent delays, the sensitivity should be reduced. JiT and storage on the building site are two possible solutions.

Finally, the total construction time of the structure might be influenced by the transport time per load and the amount of loads that have to be transported by the horizontal transport system. Multiple transport flows can be applied to ensure this method will not influence or determine the total construction time.

15.3 Conclusion

The transport system is a key component of the construction phase. Properties of this system will influence all the other phases and vice versa. The physical flow of products can be divided in three phases:

- phase 1: from the factory to the building site,
- phase 2: vertical transport from the building site to the construction floor,
- phase 3: horizontal transport on the construction floor to the final position.

For phase 1, 2 and 3 influences factors have been determined. The transport system of phase 1 is comparable with a transport system for a low or mid rise building. The only differences are the increased mass, size and amount of the elements.
During the structural design, boundary conditions for the mass and size of the elements should be considered. The National legislation limits the dimensions and the mass of the transport system properties and this is directly related to the structural properties.

When phase 2 and 3 of a 200m prefabricated tower are compared with the construction process of low and mid rise prefabricated buildings, a relevant transport system property and a relevant building process property can be distinguished:

- the sensitivity for the wind,
- the vertical transport time.

Without a reduced sensitivity for the wind, the probability of delays will increase. This will threaten the construction time and the economical feasibility of the project.

The efficiency per transport movement has to be optimal to reduce the vertical transport time. A larger transport capacity doesn't necessarily mean that the transport time can be reduced. The best solution to achieve the highest efficiency is to transport the maximum load at every vertical movement. In order to transport the maximum load every time, the capacity of the transport system should be adjusted to the mass and size of the elements. This adjustment is slightly complicated because the mass and size of the elements depends on several boundary conditions, whereby transport via the road results in the most limitations.

The mass and size of the elements are also indirectly influenced by the building height. An increasing building height requires more strength, stiffness and stability. To reduce the vertical transport time, the element thickness can be limited at higher levels. As a result, the mass can be reduced (multiple elements in one movement) or the element size can be increased (less movements).

16 Preliminary design of the construct methodology

In the previous chapter detailed criteria was provided for the transport system and building process. Based on the results, several solutions will be examined for the three phases.

Section 16.1 will start with phase 1. Phase 2 and 3 are enclosed in section 16.2. This chapter ends with a conclusion in section 16.4.

16.1 Phase 1: transport from the factory to the building site

During phase 1 the materials will be transported from the factory to the building site. This can be done by water or via the road. Transport by water has several benefits compared to transport via the road:

- The ship has a very large capacity with almost no restrictions for the prefab elements. Without the transport restrictions, the elements will be limited by the factory and the building site.
- By using water instead of the road, the busy centre of Rotterdam is relieved of extra transport.

The Rotterdam at the Kop van Zuid is one of the projects where they applied transport by water. Figure 158 shows the building site and a barge with materials. The cranes are able to transport these materials from the barge to the final location.

Figure 158 Transport by water at The Rotterdam [De FotoVlieger 2011]

Unfortunately, water transport at the Zalmhaven tower also comes with several disadvantages:

- not every prefab factory is located near the water,
- the Zalmhaven tower isn't located next to the water and the elements have be transported over land (200m). This transhipment of elements requires an additiona crane, special transport vehicles and possibly an exemption for the maximum weight.

Between the ship dock and the building site there are no obstacles. In order to create this ship dock, the waterbus has to be temporarily diverted to a nearby dock. This is depicted in Figure 25.

Figure 159 Building location and loading dock [Google 2012]

Transport via the road is the most common applied method in the Netherlands. Nevertheless, this method is subjected to several limitations [Wikipedia 2012]:

- the maximum height is 4m,
- the maximum width is 2.55m,
- the maximum length is 16.5m (truck with trailer),
- the maximum Gross Vehicle Weight (GVW, this is the weight of the vehicle, driver,
  passenger(s) and cargo) is limited to 50 ton.

These values are applicable for the entire transport combination. A truck with trailer weights approximately 14 ton (the truck weights 6 ton and the heavy load trailer 8 ton) and therefore the cargo is limited to 36 ton. If one of these limits is exceeded, a permit has to be acquired for exceptional transport. Placing the element at an angle in a flatbed trailer may prevent the need for a permit (see Figure 160).

Figure 160 Flatbed trailer with a sluice door at an angle [Pultrum 2012]

When transport via the road is preferred, another aspect has to be taken into account: traffic. Delayed trucks may threaten the schedule and this risk should be prevented. Just in Time (JiT) delivery is a solution applied at many construction projects. At the JuBi project in The Hague this principle was extended with a construction site ticket (bouwplaatsticket in Dutch) and a logistic operator [Herwijnen 2011]. The logistic operator supervised the crane schedule and a construction site ticket was used to organise the transport and storage of all the materials. To ensure that all the materials would be unloaded at the designated time, every delivery had to be registered in advance at the logistical operator. This application was done via the building site ticket and the supplier had to indicate when and where it would be delivered (basement or fifth floor), how long it would take and which resources were acquired (forklift, tower crane or nothing at all). The logistic operator would then assess if it's possible to unload the delivery. When the application is granted, the supplier will receive an approved building site ticket. On the corresponding day, the truck has to report at the storage depot of JuBi at the Binckhorstlaan in The Hague (10 minutes away from the building site). When the transport was on time with the correct cargo, the foreman at the depot would contact the logistic foreman at the building site. He determined whether the transport could drive to the building site or if it had to wait (for example when there is a delay). When the transport was approved, the truck could drive to the building site and deliver its cargo. Besides the JiT transport from the factory to the building site via the depot at the Binkhorstlaan, this depot was also used for temporary storage (there was no storage at the building site). The prefab concrete inner leafs were one of the materials that had to be stored. When the elements were required, they would be placed on a truck and transported to the building site. By applying this system, all the materials arrived at the desired time and this resulted in a very efficient and organised process [Herwijnen 2011].

This principle can also be applied to the Zalmhaven tower. A vacant area at the Maashaven Noordzijde (marked with an A in Figure 161) or the Rijnhaven Zuidzijde could be used as storage depot. Both plots contain approximately 10 000m^2 of free space and they are 2.5km or 10 minutes away from the building site.

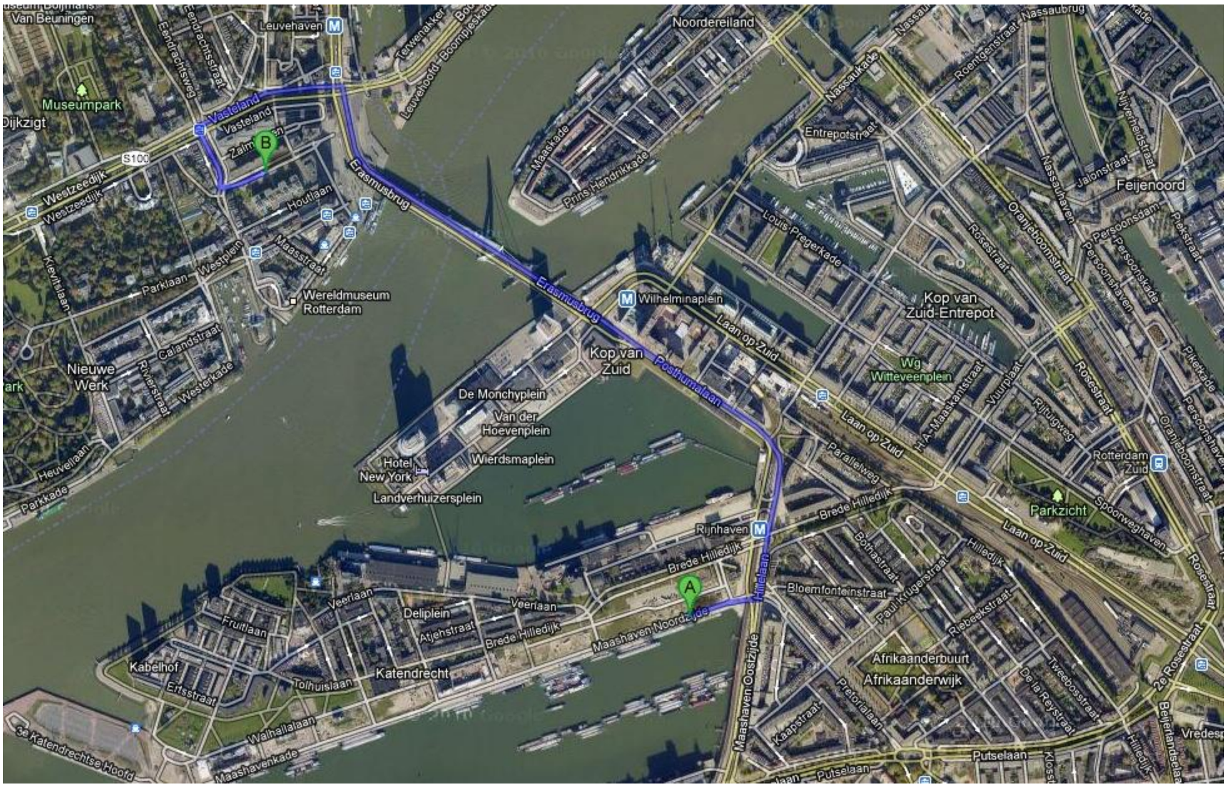


Figure 161 JiT truck waiting area [Google 2012]

By using this technique applied at the JuBi tower in The Hague, traffic delays can be reduced to a minimal and the correct elements are delivered at the desired time.

16.2 Phase 2 and 3: Transport on the building site

During phase 2 and 3, the elements will be transported from the ground level to their final position. There are two methods available for phase 2 and 3: tower cranes or a hoisting shed.

16.2.1 Tower cranes

Tower cranes are a well known transport system for cast in situ and prefab buildings. Two prefab buildings that are constructed with tower cranes are Het Strijkijzer (single crane) and the Prinsenhof (two cranes) in The Hague. To transport the elements, the following crane related actions are required:

1. attaching of the element,
2. element orientation for transport,
3. vertical transport,
4. horizontal transport,
5. placing and adjusting,
6. stabilising,
7. detaching,
8. returning horizontal (unloaded),
9. returning vertical (unloaded).

When tower cranes are applied, the transport flows are normally non-separated, i.e. the elements are transported from the ground level to their final location without transhipment. Eliminating the transhipment reduces the total amount of crane related actions, but increases the amount of actions per crane. Furthermore, the cycle times[21] will depend on the building height and this is adverse for high rise buildings.

To construct a prefabricated building, large and heavy elements are preferred. Tower cranes with a load capacity of 40 ton and more are uncommon in the Netherlands, but they are available. To construct the E-ON power plant at the Maashaven, a Liebherr 630 EC-H from KraanTechniek Nederland was used. This crane has a load capacity of 40 ton at a range of 18m. The maximum range is 80m and this reduces the capacity to 8.5 ton. The Liebherr 1250 HC 40 and the Potain MD 1100 250 LCC 100[22] are two other tower cranes that are able to transport 40 ton.

Figure 162 shows the load capacity at a certain radius of a Liebherr 1250 HC 40 tower crane. This crane has a maximum radius of 79.6m and at this distance a load of 11 ton can be transported. Between 0 and 30m, the tower crane is able to transport the maximum load of 40 ton. The maximum load is set at 40 ton because higher loads will damage the driving unit, trolley (loopkat in Dutch) and the cables.

m r	m/kg	30,0	35,0	40,0	45,0	50,0	55,0	60,0	65,0	70,0	75,0	79,6
79,6 (r = 83,7)	5,2 – 27,0 40000	35410	29630	25520	22350	19780	17660	15880	14370	13060	11920	11000
73,8 (r = 77,9)	5,2 – 28,3 40000	37360	31290	26980	23660	20970	18750	16880	15290	13920	73,8 m 13000	
62,1 (r = 66,2)	5,2 – 30,0 40000	40000	33570	28990	25450	22590	20230	62,1 m 17500				
50,5 (r = 54,6)	5,2 – 31,0 40000	40000	35360	30810	27200	50,5 m 24000						
38,8 (r = 42,9)	5,2 – 31,0 40000	40000	35490	38,8 m 32000								

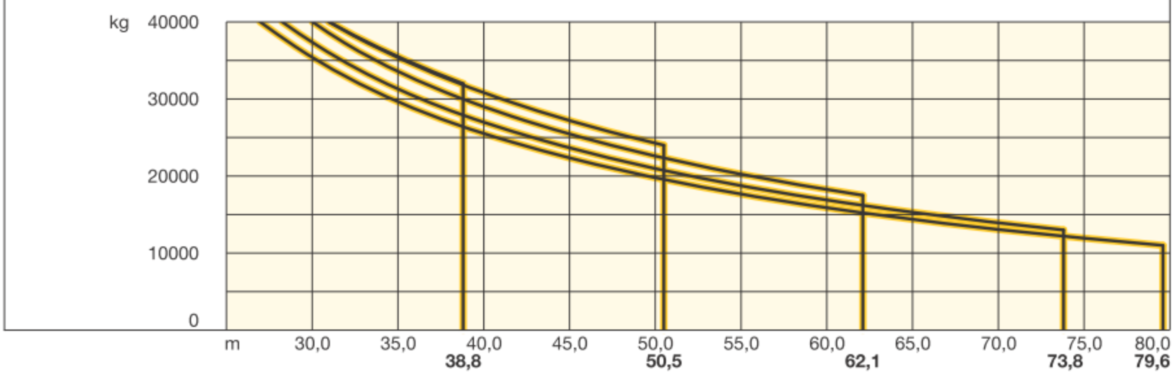


Figure 162 Load capacity of a Liebherr 1250 HC 40 tower crane [Liebherr 2009]

With a range of 30m (or 18m from the Liebherr 630 EC-H), two tower cranes are able to reach the entire floor plan of the Zalmhaven tower. A positive aspect of the tower cranes is the ability to attach the elements at multiple locations at the ground level. This makes storage on site possible without the need to transport the stored elements or materials. On the down side, transporting elements and materials from multiple locations may lead to a reduced visibility on the transport. The jibs should be placed at different heights to prevent possible collision. Optimizing the turning circles could prevent delays.

---

[21] A cycle time is the total time to complete a building layer. This aspect will be discussed in more detail in chapter 18.1.
[22] Two Potain MD 1100 tower cranes would be used to construct the Erasmus MC tower if the hoisting shed wasn't used.

The design of a crane makes it susceptible to high wind loads. On average, tower cranes are operational up to wind speeds of 27m/s (72km/h). Depending on the model, these wind speeds may change. If a tower crane is used during higher wind speeds, it will start to sway. This has no structural consequences (to a certain level), but this swaying results in unmanageable loads during transport. Beside the crane, the load is also affected by the wind speed. According to Abomafoon 3.07, the load will become uncontrollable at wind speeds higher than 15m/s (54km/h). If the construction workers are unable to control the element, the construction process will be delayed. The unguided transport of an element by a tower crane has also a negative influence on the drop safety zone (a swaying element requires a larger area). This may not be available at a small building site.

A tower crane has a maximum free standing height due to the wind load (the maximum height of a Liebherr 1250 HC 40 crane is 79m). To increase the transport height of the crane, it should be stabilised by the building. Figure 163 shows several cranes at the JuBi project in The Hague that are connected to the core by enormous steel triangles. At a later stage, the cranes were connected with a smaller steel structure to the facade (see Figure 164). During the structural calculations, these additional horizontal forces should be taken into account.

Figure 163 Horizontal connection between a tower crane and the building [Zonneveld 2011]

Figure 164 Horizontal connection between a tower crane and the building [Cement 2011]

The availability of a tower crane also plays an important aspect. Cranes with a capacity up to 20 ton are widely available in the Netherlands, but cranes with a capacity up to 40 ton are less abundant. When very powerful cranes with a high load capacity are considered (for example a Potain MD 1100), it’s very likely that the crane has to be imported from abroad. Applying cranes with a very low availability may impose a problem: as a result of the low availability, the crane may be rented out to another project shortly after the completion date. When the project encounters large delays, the crane may have to be dismantled before the project is finished or considerably large fines have to be payed. During the design of the construction method this should be taken into account.

16.2.2 Hoisting shed

By applying a hoisting shed, an indoor construction area is created. This method has several benefits compared to a traditional system with tower cranes:

- The weather dependency is reduced enormously, since all the construction activities take place inside the hoisting shed. To obtain a fully weather independent system, the vertical transport should be guided.
- A low weather dependency will increase the quality of the construction process. The mortar between the elements is not affected by low temperatures or by an excess of rain water. Also the quality of the working conditions increases, providing a better working environment for the construction workers. Internal lights and sound insulating walls make it possible to work during the evening and early morning.

- By applying a guided vertical transport system, the drop safety zone becomes equal to the largest element that is transported. This is especially beneficial for project with a small construction area. Aside from the reduced drop safety zone, the hoisting shed is placed on top of the building: reducing the required construction area at ground level even more (at the JuBi project they applied tower cranes on a portal structure since there was no free space).
- A hoisting shed also provides a working platform around the building and tasks can easily be performed at the outside of the facade. One of the important tasks is sealing the joints with EPDM (a synthetic rubber), to create a water tight structure. Many high rise buildings struggle with the water tightness at higher levels and the platform provides an optimal working area. Placing lightning conductors and cleaning the windows are several other tasks that can easily be executed. This platform around the building also increases the safety of the construction workers (fall protection for example).
- The hoisting shed also provides enough room for a lunch shack (schaftkeet in Dutch), toilet and equipment container. Consequently, the construction workers don't have to travel during their break and they can stay within the hoisting shed during the entire shift. This is an important consideration since the transport of personnel becomes leading in high rise structures.
- By applying two gantry cranes, the hoisting shed is able to separate the transport flows. This results in cycle times that are independent of the height and the horizontal distance influences the efficiency (see section 18.1).

Applying a hoisting shed also induces several problems:

- A hoisting shed contains relative high investment costs. The two hoisting sheds already applied in the Netherlands were project specific. There is no standard system available and the Zalmhaven tower project is unable to completely reuse the hoisting shed of the Erasmus MC tower.
- It's likely that the hoisting shed has to be dismantled at the top of the building. This requires a crane with a very large load capacity and height.
- The two hoisting sheds already applied in the Netherlands used the facade for support. Consequently, the facade has to stay open for a certain time period (at the Delftse Poort the facade was placed in a different phase. At the Erasmus MC the four windows could be placed after the truss was removed).

Because of these disadvantageous, several projects from abroad, which utilised a hoisting shed, are examined. Between 1991 and 1998, the Japanese construction industry constructed several projects contained a hoisting shed. Table 23 shows a short summary. Based on these projects, it's analysed if the previous three problems may be solved.

Table 23 Buildings in Japan with a hoisting shed [Cousineau 1998]

System	Company	Year	Type	Structure	Levels
SMART System	Shimizu	1991-93	Office	Steel	20
ABCSystem	Obayashi	1991-94	Residential	Steel	10
T-Up	Taisei	1992-94	Office	Steel	34
MCCS	Maeda	1992-94	Office	Steel	10
SMART System	Shimizu	1994-97	Office	Steel	30
Big Canopy	Obayashi	1995-97	Residential	Prefab concrete	26
MCCS	Maeda	1995-98	Office	Steel	8

The Big Canopy from Obayashi is the only system applied at a prefabricated concrete building. In Figure 165 Big Canopy is depicted. This system depends on four large towers to support the roof and the overhead gantry cranes. The Big Canopy was not equipped with walls and the weather sensitivity is slightly better than a traditional system (there is no large jib that is affected by wind and the roof partly shelters the construction area from the rain).

Figure 165 Big Canopy from Obayashi [Obayashi 2012]

The SMART System from Shimizu has more similarities with the hoisting shed of the Erasmus MC tower. SMART stands for Shimizu Manufacturing System by Advanced Robotics Technology and it was used for the Juroku Bank, Nagoya in 1991 (Figure 166 A) and the Nisseki building, Yokohama in 1994 (Figure 166 B). Figure 167 shows the two buildings during construction.

A B

Figure 166 Buildings constructed with the SMART System [Wikipedia 2011]

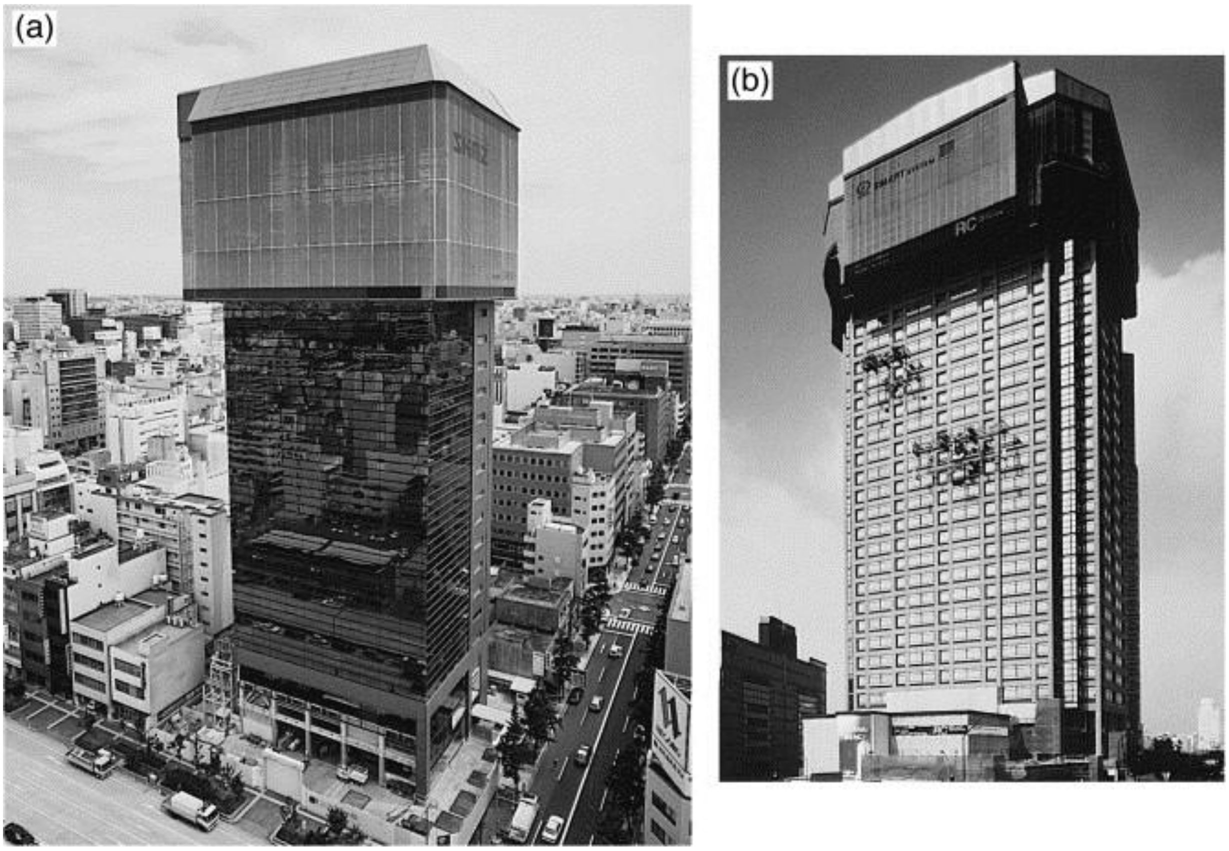


Figure 167 SMART System during Construction [Maeda 1998]

The SMART System is an integrated automatic hoisting shed for buildings with a steel load bearing structure. The process starts when a steel column is attached to an automated and self-moving trolley. Then a barcode is scanned to calculate the fasted route to the final location of the column. After the vertical transport, the trolley uses several of the 24 available gantry cranes to transport the element horizontally. When the column arrives at its final location, the trolley automatically places the element with the help of several lasers. When the element is levelled with the help of a construction worker, an automated welding robot is placed around the column. After the column is welded to the previous column, the welding robot is detached and the trolley automatically returns to the loading platform. These steps are also executed for the beams and prefabricated floors. This process is depicted in Figure 168.

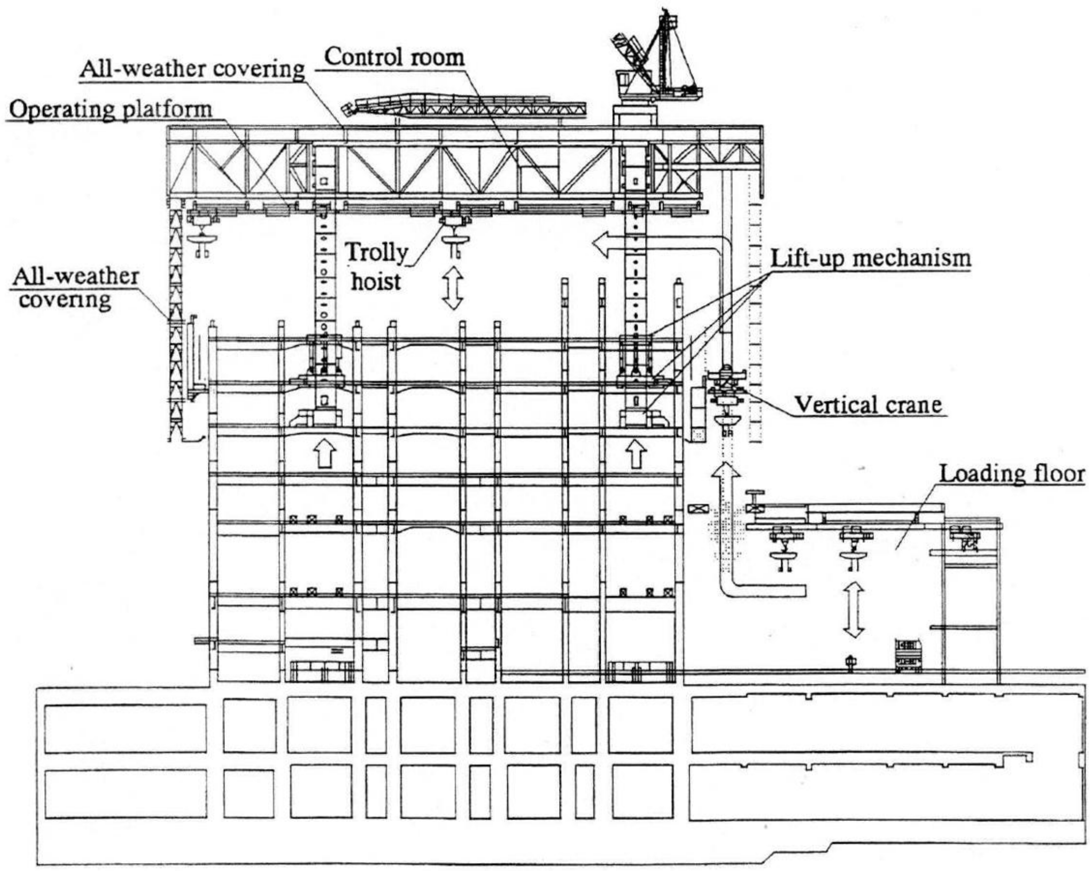


Figure 168 Operation of the SMART System [Maeda 1997]

By applying robots, the amount of construction workers is reduced enormously. The tasks of the few remaining construction workers was to level the columns, attach and detach the welding robot, maintenance of the robots and the overall supervision. The total man-hours on site were reduced with 50%.

To operate this system, 24 gantry cranes and 10 trolleys are required. This high amount of gantry cranes is necessary because the hoisting shed is supported by four internal columns and due to the dimensions of the building (see Figure 168 and Figure 169). The four supporting columns and the layout of the facade limit the freedom of the gantry cranes. With a square layout and external supports (this was the case with the Erasmus MC tower hoisting shed) less gantry cranes would be required.

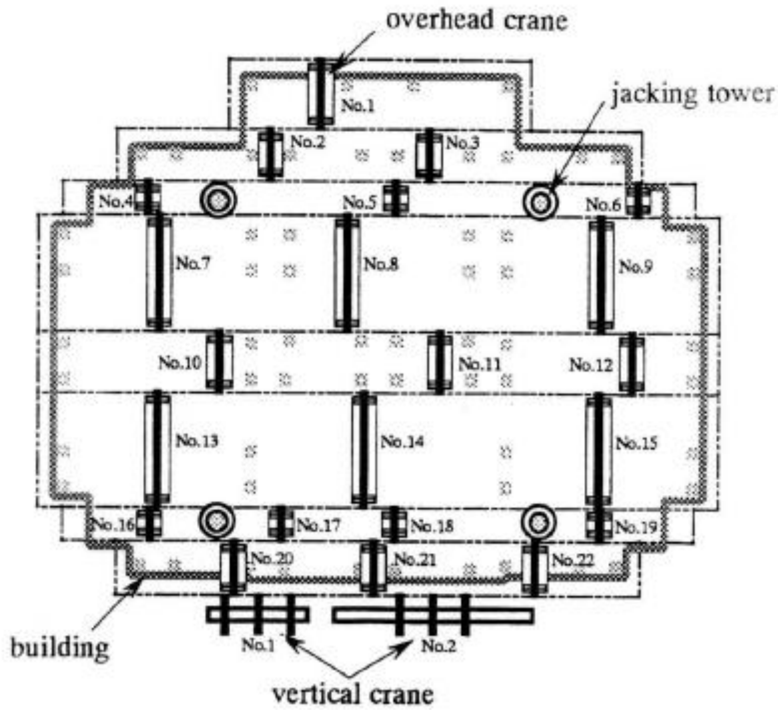


Figure 169 Layout of the SMART System at the Yokohama building [Maeda 1997]

When a column, beam or floor elements has to be placed at the left upper corner of Figure 169, gantry crane 20, 17 and 13 would line up. The trolley could than travel from gantry crane 20 to number 13. Gantry crane 13 would than travel horizontally and line up with gantry crane 10, 7 and 4, to place the element in the designated corner. This entire process is controlled by computers and no physical labour is required. Several operators monitor this process to prevent any problems.

The SMART System has several benefits compared with the hoisting shed of the Erasmus MC tower:

- Building speed: the Nisseki building in Yokohama has a floor area of 2100m^2 and a construction cycle of one floor every 5.5 days was achieved. When this is compared with the Erasmus MC tower (830m^2 could be finished every 4 days) it can be concluded that they were able to construct almost 180m^2 floor per day more between 1994 and 1997. Unfortunately, it’s unknown how many shifts they used at the Nisseki building and how many construction workers per shift.
- Jack capacity: the SMART System at the Nisseki building had a dead load of 1650 ton and within 2 hours the next floor was reached. The Erasmus MC hoisting shed had a dead load of 450 ton and required 5 hours to reach the next floor.
- Weather dependency: since there are no external supports, the facade is immediately closed after construction. This is beneficial for the finishing phase (afbouwfase in Dutch). Furthermore, no trusses have to be transported to support the hoisting shed.

This astonishing achievement, based on computers and automated systems, was already applied in 1991 (the same year the World Wide Web was introduced to the public) and can be considered as a technological innovation. The SMART System was only applied twice in Japan, because the construction firm Shimizu struggled to make a profit on the project. The traditional construction projects also had financial difficulties and the economic crisis of 1997 made an end to these extraordinary systems. The financial problems were mainly created by the enormous amount of innovation and the reusability of the systems: the hat truss of the SMART System and T-Up hoisting shed were used as load bearing structure for the top floor. This resulted in less transport at the end of the project, but very little of the hoisting shed could be reused at the next project.

With the increasing requirements for working conditions, new technology and the increasing price for labour, it’s just a matter of time before these systems will be reapplied in Japan. If the Netherlands is ready for these automated systems is a whole different question, but the technique of internal columns is interesting.
With the projects from abroad and the two hoisting sheds used in the Netherlands it may be possible to design a system based on one of these of these projects. As a result, the amount of innovation will be considerably lower, reducing the costs. The projects from Japan have also shown that it’s not required to dismantle the hoisting shed: the structure may be used as hat truss. Since this will result in the loss of the hoisting shed, it may not be the most economical solution. By using internal columns of the SMART system, the facade isn’t interrupted, increasing the weather independency of the system.

During the enumeration of the benefits of the hoisting shed relative to tower cranes, also the separated transport flows were mentioned. In Table 24 the critical actions required during the transport are depicted.

Table 24 Critical path activities of a separated and non-separated transport system

Separated transport		Non-separated transport	
Vertical transport	Horizontal transport	Vertical transport	Horizontal transport
1. Attaching of the element		1. Attaching of the element	
2. Element orientation for transport		2. Element orientation for transport	
3. Vertical transport of the element		3. Vertical transport of the element	
4. Element orientation for storage			4. Horizontal transport of the element
5. Detaching of the element			5. Element orientation and adjustment
6. Returning vertical (unloaded)	7. Attaching of the element		6. Detaching of the element
	8. Element orientation for transport		7. Returning horizontal (unloaded)
	9. Horizontal transport of the element	8. Returning vertical (unloaded)	
	10. Element orientation and adjustment		
	11. Detaching of the element		
	12. Returning horizontal (unloaded)		

The amount of actions, 12 for a separated system and 8 for a non-separated system determine the cycle time per floor to a significant extend. It should be noted that while the separated system contains more actions, the actions per system are less (6 versus 8).

At the separated system the vertical gantry crane will return for a new element (step 6) after the element is detached. When the horizontal gantry crane returns from its cycle, a new element should be ready for pick up, i.e. step 1 to 6 should require less time than step 7 to 12. If this requirement isn’t met, the construction workers has to wait on the vertical transport, which is disadvantageous for the cycle time. At the non-separated system all the actions are performed by one transport system. To prevent the construction workers from waiting, at least two tower cranes are required.

When the load capacity and transport speed of a hoisting shed is compared with four different tower cranes, several important conclusions can be made (see Figure 170 and Figure 171):

- At a low mass (between 0 and 15 ton), the tower cranes are able to transport the load much faster than the gantry crane of the Erasmus MC tower. This is because the gantry crane of is not equipped with gears.
- At a high mass (between 15 and 40 ton), the gantry crane is slightly faster than the Liebherr 1250 HC 40 (65kW) and Liebherr 640 EC-H 40 (65kW). These two heavy load cranes are more common in the Netherlands than the Potain MD 1100 (180kW). The Liebherr 1250 HC 40 is normally equipped with a 65kW drive unit, but this can be replaced by a 110kW unit. With this more powerful drive unit, the overall transport speed of the Liebherr tower cranes becomes faster than that of the gantry crane.
- The efficiency will also increase when it's possible to transport heavy elements faster. Compared with the other systems, the Potain MD 1100 has an enormous efficiency, but this is mainly caused by the very powerful drive unit (180kW).

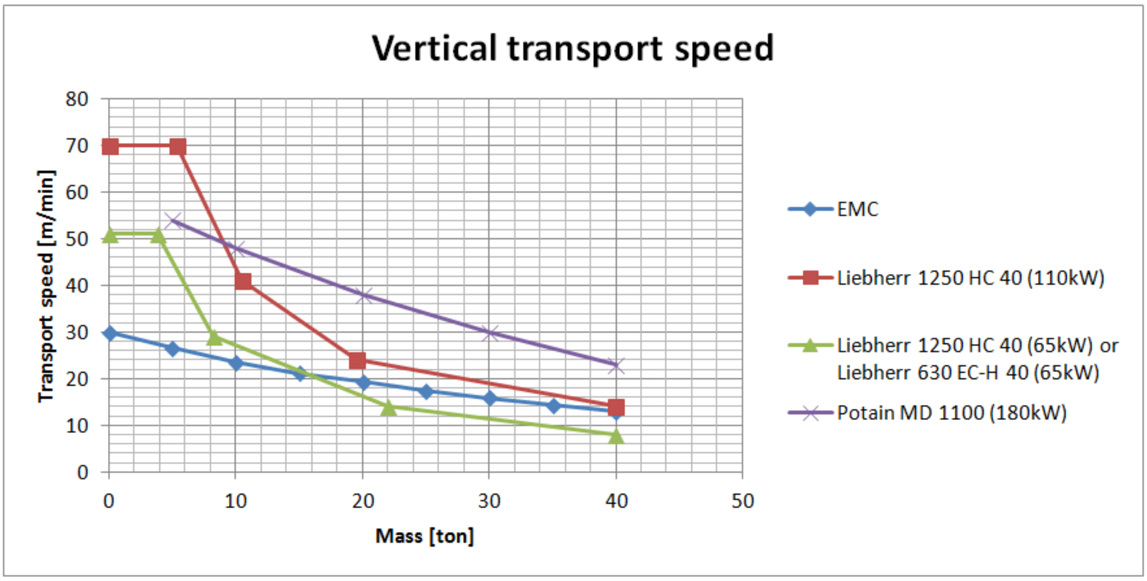


Figure 170 Vertical transport speed

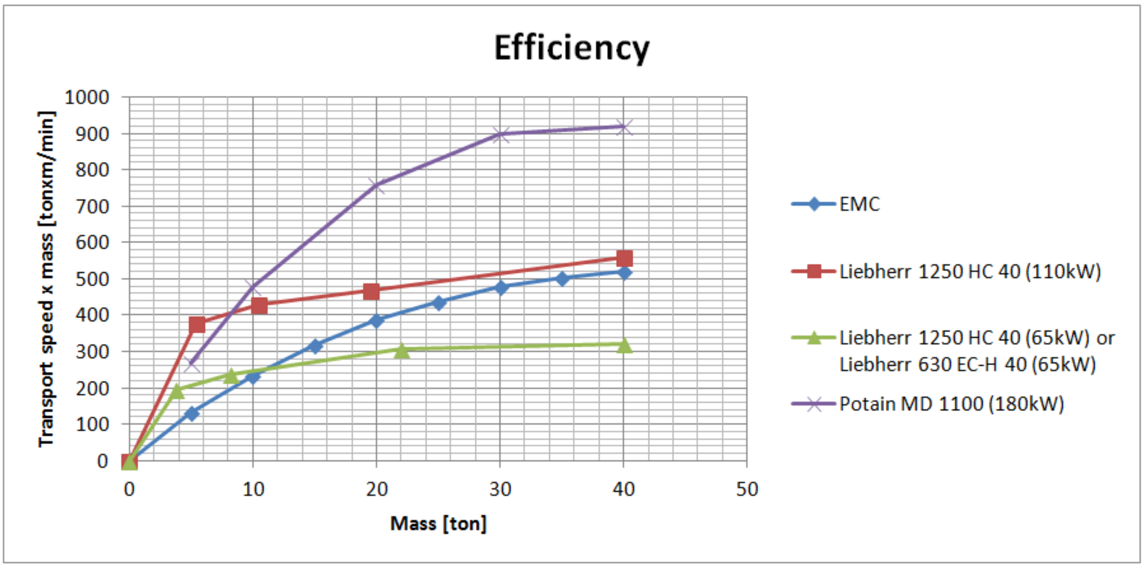


Figure 171 Efficiency per movement

The performance of the gantry crane of the Erasmus MC tower would be approximately equal to the Liebherr 1250 HC 40 (110kW) if it was also equiped with gears. Figure 153 and Figure 154 in section 15.1.1 might have indicate that the gantry crane had a higher transport speed and efficiancy, but the Liebherr 420EC-H 20 used in this example is unable to transport 40 ton.

16.3 Decision final transport systems

In chapter 13 multiple aspects have been considered which influence the construction methodology. Based on the criteria, a decision will be made between the different possible solutions of the transport system in phase 1, 2 and 3.

Phase 1

For phase 1 there are two solutions: transport by water or via the road. When the costs are considered, it may be concluded that transport by water is more expensive. This is because the elements have to be lifted out of the ship and onto a special transport vehicle. This transhipment may also endanger the quality of the elements because an additional step is required. The quality of the entire process may be considered equal. When JiT is applied at the road transport, the risk of delays of both methods is considerably low. When the transport time is considered, it's clear that transport via the road is much quicker. On the primary aspects it may be concluded that transport via the road contains the most benefits. From the secondary criteria, the redundancy or flexibility of trucks is considerably larger because only one (or a few, depending on the size) element is transported per truck. If the truck has to wait for the element or the truck breaks down, this won't result in any large problems. To increase the redundancy or flexibility of a ship, trucks may be used as back-up. This also holds for the dependency criteria. Repetition is not a critical criteria and both processes contain a high amount of repetition. When the construction area is considered, both methods provide storage outside the construction area. The environmental impact of the transport method is considerably lower for transport by water. This is because less trucks have to drive through the busy centre of Rotterdam. All in all, it may be concluded that the traditional method of trucks via the road provides the most benefits. Therefore this method will be applied during the design.

Phase 2 and 3

For phase 2 and 3 there are also two solutions: tower cranes or a hoisting shed. When the costs are considered, it may be clear that the hoisting shed will be more expensive. This is mostly due to the project specific design of the hoisting shed. When a modular system is applied, which allows for reuse, the cost may be reduced considerable, but it will always be higher than one or two tower cranes. But this disadvantage is accompanied by a higher quality, shorter construction time and lower risk (the system is weather independent). The redundancy or flexibility of the tower cranes are slightly better because if one crane ceases to operate, the other crane isn't affected. This also holds for the dependency: the horizontal gantry crane of the hoisting shed depends on the vertical gantry crane. When the weather dependency is considered, the hoisting shed with a guided vertical system is able to provide a nearly weather independent system, while this is nearly impossible with tower cranes. The repetition is not a critical criteria and both processes contain a high amount of repetition. On the construction area and environmental impact the hoisting shed provides better results than the tower cranes. When all the criteria are considered, it may be concluded that the hoisting shed provides the best results. In order to apply this special system, higher costs are required.

16.4 Conclusion

In this chapter several solutions for the transport systems were considered. Two possible solutions for phase 1 are: transport by water or via the road. Transport via the road results in mass and dimensional limitations, but in order to apply transport by water, the elements and materials have to be transhipped. For phase 2 and 3, tower cranes or a hoisting shed can be applied. Since hoisting sheds are often project specific, several projects from abroad have been analysed. The SMART system of Shimizu, which was applied twice in Japan, provides many new possibilities. For example the high production speed, short jacking time of the entire hoisting shed and the possibility to use the hoisting shed as hat truss.

Based on the criteria of chapter 13, the methods of phase 1 and phase 2 and 3 were compared. Due to the lower costs, flexibility and dependency, transport via the road is recommended for phase 1. The better quality, lower risk and reduced time outweigh the higher costs of the hoisting shed and the original recommendation of the vision for the construction methodology (see chapter 5) is maintained.

During the structural design, the limitations of transport via the road and the building shed should be considered in order to create an optimal structural design. During the structural design also the construction methodology will be finalised (especially the design of the hoisting shed), providing an optimal coherency between the two aspect.

17 Tolerances

Tolerances are essential during the manufacturing of the elements and during the assembly at the construction site. The allocation of the measuring points at the construction site is a third aspect which, in combination with the other two aspects, determines the quality, assembly speed and additional costs. Clients, contractors, architects and engineers must consent on universal dimensions and tolerances for the intermediate and finished end product (the building). Besides theses mutual agreements, the national legislation also specifies tolerances. For example, the NEN 2886[23] formulates maximum tolerances for buildings and the NEN 3682 contains general rules and guidance for dimensional control in the building field. NEN 2887 until NEN 2889 are developed for the allocation, assembly and manufacturing tolerances. These standards provide maximum allowable dimensional tolerances that can be achieved by "proper workmanship". There may be situations in which a higher degree of tolerances is required or necessary, but this higher degree can only be achieved if all three disciplines (allocation, assembly and manufacturing) reduce the dimensional tolerances to a minimum. Large dimensional tolerances of one discipline will neutralise the high amount of accuracy of the other two disciplines. A disadvantage of reducing tolerances is that it usually involves a significant cost increase.

The structural design is the first phase where tolerances are taken into account. Due to inaccuracies of the concrete template, shrinkage of the concrete, damage during transport and mistakes/inaccuracies at the construction site, it may be possible that the element won't fit. To prevent unnecessary repair costs, the joints and gains are slightly enlarged: the elements are smaller than they should be and the larger joints are filled with mortar.

When the factory starts to produce the elements, there will always be a small deviation from the dimensions provided by the engineer. The allowed tolerances are specified in NEN 2889 and Figure 172 provides an overview. Manufacturers may individually or collectively specify smaller tolerances than the previous mentioned standard. An example of the latter can be found in the BELTON publication "Connections in precast" (see Figure 173) and "Facades in precast". During the production of the elements, it's also possible to integrate measuring points for the assembling phase.

Hurks Beton, a large manufacturer of prefab elements, uses the tolerances of the NEN 2889 for all their elements. For every project it's examined if smaller tolerances are required and possible.

[23] These standards are not yet replaced by the Eurocode and the NEN is still applicable.

produkt	grootte [1]					vorm [1]					voorzieningen [2]	
	lengte [3]	breedte	dikte	hoogte	diagonaal [4]	kromte	buiging [5]	scheluwte	haaksheid			
									kop-eind	opleg-vlak	eenling	groep
	mm	mm	mm	mm	mm	mm/m	mm/m	mm	mm	mm	mm	mm
kolommen	–	7	7	11	–	1,4	–	5	10	6	11	5
balken:												
≤ 10 m NVS [6]	11	–	7	11	–	1,4	1,4	8	10	6	11	5
≤ 10 m VS [7]	17	–	7	11	–	2,0	2,8	10	14	6	14	5
> 10 m VS [7]	21	–	8	11	–	2,0	2,0	14	16	8	14	5
spantvormige elementen	11	7	7	11	–	1,4	2,0	10	10	6	11	5
vloerplaten NVS [6]	28	12	12	–	28	2,0	1,6	8	20	–	50	–
vloerplaten VS [7]	28	12	12	–	28	1,0	2,0	8	20	–	50	–
vloerplaten TT [8]	21	7	7	7	21	2,0	2,8	10	20	6	28	5
wanden	11	–	7	8	11	1,4	–	8	10	–	11	5
gevelelementen – binnenspouwbladen	7	–	5	7	9	2,0	–	8	10	–	11	5
trapelementen	14	11	11	–	–	2,0	–	8	10	–	11	5
balkonelementen	7	7	5	–	9	1,4	2,0	8	10	–	11	5

1) Zie illustraties in de bijlage C bij NEN 3682.
2) Bijvoorbeeld een opening c.q. sparing is eenling.
Maatafwijkingen bij een voorzieningengroep zijn maatafwijkingen in de onderlinge posities van eenlingen binnen een groep.
3) Voor de lengte van balken als onderdelen van systeemvloeren gelden dezelfde maximaal toelaatbare maatafwijkingen als bij vloerplaten zijn vermeld.
4) Bedoeld worden alle diagonalen.
5) Buiging ten opzichte van berekende doorbuiging of opbuiging.
6) NVS: niet-voorgespannen.
7) VS: voorgespannen.
8) Vloerplaten TT: dubbel T vloerplaten.

Figure 172 Maximum allowable dimensional deviations for concrete elements (A_i) [NEN]

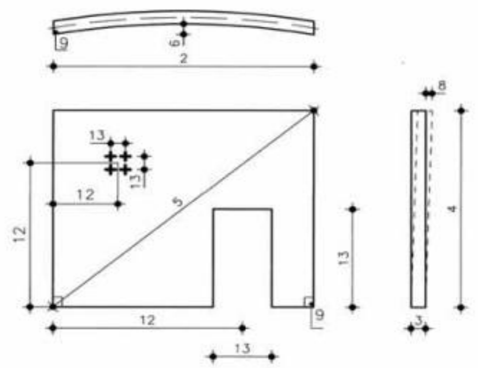
MAXIMAAL TOELAATBARE MAATAFWIJKIGEN VAN WANDEN, SPOUWBLADEN EN TOPPEN

Maximaal toelaatbare maatafwijkingen

Onderdeel		pos.nr.	volgens Belton
Grootte	lengte	2	9 mm
	dikte	3	7 mm
	hoogte	4	7 mm
	diagonaal	5	11 mm
Vorm	kromte	6	1,4 mm/m
	scheluwte	8	8 mm
	* haaksheid E	9	8 mm/m

Voorzieningen	pos.nr.	malzijde	stortzijde
plaats/eenling	12	9 mm	11 mm
afmeting/groep	13	5 mm	8 mm

** In twee richtingen*



Afwerking (NEN 6722)
indien zichtzijde geldt:
oppervlakte beoordelingsklasse IC

Figure 173 Tolerances specified by BELTON [Bennenk]

When the elements are placed at their final location, a level (waterpas in Dutch) can be used to adjust the angle of the elements. A total station is a more advanced tool and the angles in all three directions can be determined. With the integrated measurement points it’s possible to accurately determine the dimensional deviations and the element can be placed on the desired location. These points can also be used the measure settlements and deformations during the construction.

A prefabricated high rise building contains a large number of stacked concrete elements. The load bearing walls can be compared with a standard masonry wall: the prefab stones are laid on a mortar bed and the vertical joints are filled with mortar. Due to the thick joints there are no dimensional problems, but these masonry walls are often not perfectly straight and contain many inaccuracies. This could also occur at the load bearing walls of the Zalmhaven tower if the execution process isn’t performed properly. During the design it will be examined what the influence is of stacking a high amount of elements on top of each other.

18 Cycle time

The cycle time includes the sum of activities that are required to complete a building layer. By calculating the required time per floor, the total construction time can be obtained. In this chapter, the cycle time will be considered in relation to the vertical transport time. The influence of the transport system is taken into account in this consideration.

To create this chapter, the master thesis report of van der Meij [Meij 2012] is used. This chapter will start with

18.1 Influence factors

As mentioned in the introduction, the cycle time includes the sum of activities that are required to complete a building layer. To determine the cycle time of a building layer, the following three questions have to be answered:

1. Which activities have to be executed to complete the layer?
2. How much time does this activity requires?
3. Which activities can be executed parallel?

When the cycle time of a floor is determined, it can be optimized. The following four aspects can be considered during the optimization:

1. reduce the required time per activity,
2. reduce the amount of activities in the critical path,
3. reduce the transport time,
4. reduce the amount of elements that have to be transported.

Reducing the required time per activity (for example placing a floor element in 20 minutes instead of 30 minutes) has only a small relation with the transport system (the (gantry) crane is able to return quicker to the starting point for the next element) and is therefore not discussed in more detail. The activity vertical transport is affected by the building height, but this will be examined in step 3.

The amount of activities has a large influence on the cycle time. These activities can be separated into two categories: crane related and crane unrelated activities (kraangebonden and kraanongebonden handelingen in Dutch). The critical path crane related actions of a separated (hoisting shed) and non-separated (tower crane) are shown in Table 24.

Table 25 Critical path activities of a separated and non-separated transport system

Separated transport		Non-separated transport	
Vertical transport	Horizontal transport	Vertical transport	Horizontal transport
1. Attaching of the element		1. Attaching of the element	
2. Element orientation for transport		2. Element orientation for transport	
3. Vertical transport of the element		3. Vertical transport of the element	
4. Element orientation for storage			4. Horizontal transport of the element
5. Detaching of the element			5. Element orientation and adjustment
6. Returning vertical (unloaded)	7. Attaching of the element		6. Detaching of the element
	8. Element orientation for transport		7. Returning horizontal (unloaded)
	9. Horizontal transport of the element	8. Returning vertical (unloaded)	
	10. Element orientation and adjustment		
	11. Detaching of the element		
	12. Returning horizontal (unloaded)		

The elements which are discussed in Table 24 are for example:

- structural wall elements,
- facade elements,
- floor elements,
- scissor stairs,
- concrete aerated blocks for several internal walls,
- bathroom units.

The amount of actions, 12 for a separated system and 8 for a non-separated system determine the cycle time to a significant extend. It should be noted that while the separated system contains more actions, the actions per system are less (6 versus 8).

The crane unrelated actions do not require the presence of the crane and can therefore be executed parallel with the crane related actions. For a precast element several actions can be enumerated:

- determining the correct location of the element (in x, y and z relative to a reference point),
- cleaning the connection,
- creating the connection (with or without formwork),
- removing the braces (schoren in Dutch).

In order to execute these actions simultaneously with the crane related actions, enough personnel should be available on the construction floor. If this requirement is satisfied, the crane unrelated actions have no influence on the cycle time.

The transport time and the amount of elements that have to be transported are the two last aspects which influence the cycle time. Just as the amount of actions, the amount of elements also significantly influences the cycle time: less elements results in less transport time, less actions and consequently the cycle time is reduced. Before the transport time can be examined, the following aspects have to be discussed: the utilization ratio of the (gantry) crane (bezettingsgraad in Dutch) and the norm time (normtijd in Dutch).

The utilization ratio is the percentage of time which the transport system is utilised. For example, the maximum utilisation ratio of a tower crane is 80%. This implies that minimal 20% of the cycle time the tower crane is inactive, waiting to transport a new element. This boundary is set at 80% to increase the robustness of the system and schedule. As a result, breaks (not included in the schedule), short delays and human errors won't affect the schedule. At a hoisting shed, the maximum utilisation ratio is set at 90% since a hoisting shed has a higher robustness (less susceptible to weather delays). If the utilisation ratio is too high, multiple transport systems can be applied, the amount elements that have to be transported can be reduced or the transport speed can be increased.

The scheduled time which one element utilises the transport system is called the norm time. This norm time includes the transport time and the required time for the crane related actions (for example attaching, adjusting and detaching of the element). In other words, the norm time is the total time between moment of attaching the element to the crane and the moment the crane returns for the second element (step 1 to 6, 7 to 12 or 1 to 8 in Table 24). In practise, often a constant value is used for the crane related actions, based on the size accuracy, accessibility and visibility of the actions.

The last aspect which remains is the transport time. Unlike the crane related actions, the transport time increases over the building height. When a hoisting shed is considered with a separated transport system, Figure 174 is obtained.

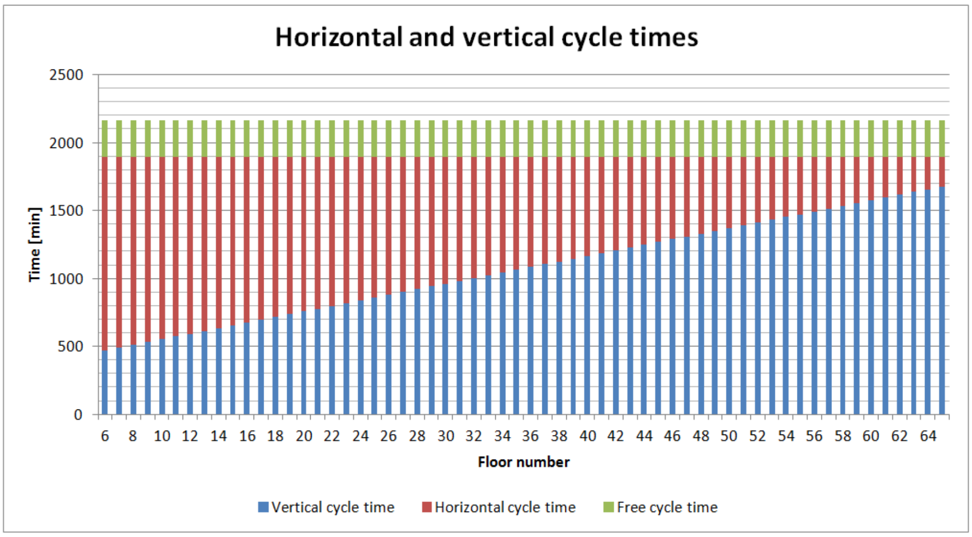


Figure 174 Horizontal and vertical cycle times of a separated system

The blue lines represent the vertical cycle times. Due to the increasing height, the cycle times increase. The red lines represent the horizontal cycle times. Since the horizontal transport isn't affected by the height[24], the cycle times remain constant. The green lines are the remaining unused cycle time (at least 10% at a hoisting shed and 20% at a tower crane). At the fifth floor the total height is only 15.25m, resulting in a very short vertical transport time. Since the horizontal time remains constant, the vertical transport system is only utilised very little (less than 25% of the total available cycle time). Since the efficiency is very low at the start of a project (the construction workers aren't familiar yet with the process), the vertical gantry crane may be utilised as second horizontal

[24] The horizontal distance and the amount of crane related actions remains identical at all the floors.

transport system. This additional second horizontal system should be planned very carefully since the two gantry cranes may obstruct each other.

At the 65th floor, the vertical transport time has increased considerably and the horizontal and vertical cycle time are nearly identical. It should be prevented that the vertical transport cycle becomes larger than the horizontal cycle since this ensures a stagnation of the work on the construction floor (the construction workers have to wait for the new elements). When this requirement is fulfilled, a separated transport system isn't affected by the building height and constant cycle times can be maintained. At a non-separated system, the height affects the cycle time (see Table 24) and it's nearly impossible to acquire a constant cycle time. It's possible, but then very large buffers have to be included in the process, making the cycle time very inefficient at the bottom of the tower. Furthermore, when they are near the top floor, the construction workers have to wait for the elements, making also the top of the construction inefficient.

18.2 Conclusion

In conclusion, the cycle time includes the sum of activities that are required to complete a building layer. Per activity, there is a certain norm time, which contains the crane related actions and transport time. When a separated system is applied, the horizontal cycle time should be leading, which isn't affected by the height. As a result, constant cycle times are obtained over the height, ensuring an optimal and efficient construction process.

19 Abstract of the literature study

During the literature many aspects have be considered and examined. The subject of these aspects is based on the following research question:

What do I have to know in order to design the structure and construction methodology of a 200 meter tower in Rotterdam?

To answer this question, first the current design was analysed in combination with several reference projects. To acquire more knowledge on these projects, several experts were interviewed. In combination with the building vision, created during the orientation phase, a basis was created for the literature study.

The next step was to examine the structural aspects required for the design, combined in section 1: Structural Design. This section starts with the criteria for a structural design in chapter 6 and they are divided into two groups (primary and secondary criteria). These criteria are essential to create an optimal and integrated design and also provide guidance for decisions between different concepts.

Chapter 7 continues with the load on the structure. Wind load is governing in a high rise design and there are several differences between the NEN-EN 1991-1-4 and the NEN 6702. The values for the basic wind velocity have been increased in the Eurocode and the reference height in combination with the wind friction have changed. This should result in higher loads for taller buildings, but when the wind load for the Zalmhaven tower is calculated, it can be concluded that the Eurocode provides slightly lower values at higher altitudes. The reason for this difference can be traced back to the higher amount of possibilities to accurately calculate the wind load in the Eurocode.
A 200m tower has never been constructed in the Netherlands before and the comfort levels become a point of attention at these heights. With the Eurocode a new calculation becomes available that has more possibilities to accurately calculate the accelerations of the building. With the abundance of options (that make the calculation rather complex and difficult to understand) it's possible to achieve results that are comparable to the results obtained by a FEM analysis and the NEN 6702. If this conclusion remains valid when a different project is considered or when the project parameters change considerably is unknown.
Wind interference and vortex shedding are two important aspects of high rise buildings. Unfortunately there are no design rules for these aspects because they depend on too many project specific variables. To get a clear insight, a wind tunnel research is recommended.

In chapter 8 several alternatives for the foundation were reviewed. Based on the analysis, a primary conclusion was made: diaphragm walls are the best solution because of the very large stiffness, unlimited length and the low amount of vibrations. The high price per diaphragm walls is a considerable disadvantage. A stiffness calculation of the foundation has to show if it's possible to use Tubex piles instead of diaphragm walls.

General structural layouts for the stability structure are discussed in chapter 9. Chapter 10 continues with the stability systems for prefab structures. First the elements that are required to construct a prefabricated structure are investigated: walls, floors and columns. When the structural behaviour of the connections between these elements is examined, it can be concluded that the grouted starter bar connection and the masonry configuration are the best solutions for the horizontal and vertical connections. The Interlocking Halfway Connection is the stiffest vertical connection between perpendicular walls and for the floor the re-engineered steel tube connection contains the most benefits. Next, the response to lateral load is examined. The shear force deformation is governing in the lower levels and shear lag will lead to locally increased forces. The lintel above the door opening is decisive for the stiffness of the structure and due to large forces it will probably be cracked. When several element configurations are studied, it can be concluded that the standard masonry configuration provides the stiffest structure with the least amount of required labour. Furthermore, to obtain the best structural properties, the elements should be one floor level high and as large as possible.

The application of higher strength concrete is discussed in chapter 11. The use of Ultra High Strength Concrete is at the moment reserved for prestige projects. A price reduction is necessary before it's widely introduced in the building industry. Applying High Strength concrete in building structures is more common and Bablaid's research shows that it provides a better performing structure and in some cases, a reduction of costs. The increased Young's Modulus is quite interesting for prefabricated structures because the stiffness is reduced by the connections.

Chapter 12 contains a significant aspect of a high rise design: progressive collapse. Several structures have collapsed due to (un)foreseen situations and the resulting extraordinary loads. Depending on the consequence class, different measures have to be taken. For CC1 structure no extraordinary loads have to be considered and for CC3 structures a systematic risk analysis is obligatory. The reason for this enhanced level of security is not that these structures are less predictable than CC1 and CC2 structures, but the consequences of a collapse are greater. To prevent casualties in the future, the Eurocode distinguishes two different strategies for extraordinary design situations at CC3 structures. The first strategy provides three solutions for known extraordinary loads and the second strategy provides three solutions for unknown extraordinary loads. A fourth solution for unknown extraordinary loads is specified in the "NTA HGBW part 3: Structural safety".
Although the prefab structure for the Zalmhaven tower has yet to be designed, several interesting conclusions could be made from the performed risk analysis. The shear walls in combination with the facade columns and the floors contain the highest relative risk. To prevent disproportional collapse, measures should be taken to mitigate the relative risk level. A second load bearing system is a good example that will reduce the severity of the resulting damage. To reduce the severity of initial failure, the extraordinary loads have to be prevented or reduced. The foundation contains the lowest relative risk level and non structural measures in combination with a high robustness will reduce this relative risk to an acceptable level.
Performing a risk analysis provides insight to how the structure performs and reacts. Failure paths are discovered and the propagation of collapse should be prevented. On the other hand, the risk analysis is not able to identify all relevant risks. It's likely that events or relations between events and failures are overlooked and as a result (important) failure modes may be forgotten.
When a prefab structure is compared to a cast in situ variant, it can be concluded that on a global scale the risks remain comparable. On a local scale an important distinction can be made: the connections. The connections between elements are subject to more errors and commonly have a lower reinforcement ratio than the surrounding elements. Due to this lower reinforcement ratio and other connection properties (smooth concrete surface), the resistance will be lower than the surrounding concrete. Therefore the connections should be a point of interest during the structural design.

With chapter 12 the structural part of the literature study comes to an end. Many aspects that are required for a structural design have been examined and it can be concluded that there are multiple solutions for specific prefab problems (reduced stiffness and special connections): a 200m prefabricated tower seems to be structurally feasible.

Part 2 contains the construction methodology and starts with criteria in chapter 13. Based upon these criteria, the decision for prefabricated concrete is reconsidered in chapter 14. Despite the higher initial costs, it's most likely that the shorter construction time in combination with the higher quality and lower risks will create a positive end result.

Chapter 15 continues with the transport system, which has a considerable influence on the building process. When the transport system and process properties of the horizontal transport (from the factory to the building) are considered in relation to the building height, it may be concluded that the transport system of a high rise building is comparable to that of a low or mid rise building. The only differences are the increased mass, size and amount of the elements. It should be noted that the National legislation limits the dimensions and the mass of the transport system properties, directly influencing the structural properties.
When the transport system and building process properties of the vertical and horizontal transport system on site are examined in relation to the height, it can be concluded that the sensitivity for weather delays and the transport time increase at taller buildings. To obtain an economical viable project, the wind sensitivity should be as low as possible and the efficiency per transport movement as high as possible.

With these influence factors a preliminary design was drafted for the transport systems in chapter 16. Transport by water or via the road was considered in combination with tower cranes or a hoisting shed. With the construction methodology criteria, transport via the road in combination with JiT and a hoisting shed was considered to be the best solution.

The two final chapters contain the tolerances and cycle time. Tolerances are an interesting aspect at prefab concrete because the structure is composed out of a large amount of stacked elements. This composition might require a different technique to absorb deviations. To calculate the construction time, the cycle time per floor is of considerable interest. When a separated system is utilised (for example a hoisting shed), the cycle time will become independent from the building height (as long as the horizontal cycle is leading). This is advantageous for high rise buildings because the vertical transport time will increase significantly at larger heights.

Corresponding to part 1, no limitations have been encountered in part 2 preventing a 200m building from being constructed.

20 Design recommendations

In the previous chapter a short abstract was provided of the entire literature study. In this chapter several important recommendations are provided which should be taken into account during the design of the structure and the construction methodology:

- The determination of the wind load and building acceleration are based on the Eurocode and cast in situ design. With the prefab properties it has to be examined if the values remain in the same order of magnitude or if there are considerable differences.
- From the structural properties and the risk analyses it may be concluded that the connections are the strength and weakness of prefab concrete. During the structural and construction methodology design the connections should be one of the most important aspects. In this literature several formulas have been provided to calculate the connection properties, but what is the influence of different stiffness values on the behaviour of the structure?
- A prefabricated building of 65 levels contains a large amount of stacked elements. How does this influence the division of tolerances of a structure? Are traditional methods applied at low and mid rise buildings sufficient or is a new procedure required?
- With a high amount of floors, the cycle time becomes an important aspect of the total building time. The relation between the cycle time and height have been examined, but what is the relation between the cycle time and the mass of the elements (large elements have a positive effect on the structural properties)?

These four points are interesting subjects which haven’t been examined yet. During the research phase of this graduation thesis, an answer will be formulated on for these subjects.

Bibliography

Books

- Bennenk, H.W. *Handboek Prefb-Beton.* bfbn.
- Cousineau, L. & Miura, N. (1998). *Construction Robots.* Amerika, ASCE.

Readers

- CUR (2006). *CT 4130: Probability in Civil Engineering.* Delft: TU Delft.
- Hoenderkamp, J.C.D. (2007). *High-rise structures, preliminary design for lateral load.* Eindhoven: TU Eindhoven.
- Horst, A.Q.C. van der (2011). *Lecture notes CT4170 Construction Technology of Civil Engineering Projects.* Delft: TU Delft.
- CT4281 (2005). *Designing and Understanding Precast Concrete Structures in Buildings.* Delft: TU Delft.
- Ridder, H.A.J. de and Soons, F.A.M. (2006). *Integraal Ontwerpen in de Civiele Techniek – Ontwerpproject 2.* Delft: TU Delft.
- Romeyn, A. (2006). *Constructieleer 3A Dictaat deel I, CT3051A.* Delft: TU Delft.
- Simone, A. (2010). *An Introduction to the Analysis of Slender Structures.* Delft: TU Delft.

Journals/Articles

- Cement online
    - Font Freide, J.J.M., Prumpeler, M.W.H.J. & Woudenberg, I.A.R. (2006). *Het Strijkijzer; Landmark voor Den Haag*. Cement, vol. 2006-1, p. 37-41.
    - Alphen, R.E. van & Vambersky, J.N.J.A. (2005). *Woontoren volledig in prefab uitgevoerd.* Cement, vol. 2005-4, p. 53-57.
    - Henkens, G. & Splinter, B. (2010). *Erasmus MC in aanbouw.* Cement, vol. 2010-3, p. 22-28.
    - Herwijnen, R. Van & Reuvers, M. (2011). *Organisatie van de uitvoering.* Cement, vol. 2011-7, p. 20-26.
    - Huijben, R.N. (2006). *Onderpompen van prefab-betonwanden.* Cement, vol. 2006-8, p. 46-49.
    - Kock, J.W.G.J. de (2006). *Woontorens met rugzak.* Cement, vol. 2006-4, p. 29
    - Keulen, D, van (2012). *Vervormingen prefab wandconstructies*. Cement, vol. 2012-6 p. 80-84.
    - Köhne, J.H. (1991). *Hoogbouw dwingt tot nieuwe uitvoeringstechniek.* Cement, vol. 1991-4, p. 12-19.
    - Lindhoud, A.C. (2003). *Een verzameling dozen*. Cement, vol 2003-7, p. 53-58
    - Robbemont, A. (2011). *Buis in buis verzorgt stabiliteit.* Cement, vol. 2011-7, p. 14-19.
    - Walraven, J.C. (2006). *Ultra-hogesterktebeton: een material in ontwikkeling.* Cement, vol. 2006-5, p. 57-61.

- Others
    - Convenant Hoogbouw (2009). *NTA Hoogbouw (03-A)nl.* Nederlandse Normalisatie-instituut: Delft.
    - Council on tall buildings and urban habitat (1995). *Structural systems for tall buildings.* United States of America, McGraw-Hill inc..
    - ING (2010). *Samen duurzaam bouwen aan innovatie.* Presented at Bouwend Nederland at 2 October 2010.
    - Geurts, dr.ir. C.P.W., Bentum, ir. C.A. & Steenbergen, dr.ir. R.D.J.M. (2011). *Euocode EN 1991: Windbelasting 1, 2 and 3.* Bouwen met Staal.
    - Hayden, T. (2009). *Crowding Our Planet.* National Geographic, p. 10-29.

- Kwan, A.K.H. (1996). *Shear Lag in Shear/Core Walls.* Journal of structural engineering (www.ascelibrary.org), vol. September 1996.
- Maeda, J. & Miyatake, Y. (1997). *Improvement of a Computer Integrated and Automated Construction System for High-rise Building and its Application for RC (Rail City) Yokohama Building.* IAAC.org
- Nederlandse Normalisatie-instituut (2002). *NEN-EN 1990.* Nederlandse Normalisatie-instituut: Delft.
- Nederlandse Normalisatie-instituut (2005). *NEN-EN 1991-1-4.* Nederlandse Normalisatie-instituut: Delft.
- Nederlandse Normalisatie-instituut (2005). *NEN-EN 1991-1-3.* Nederlandse Normalisatie-instituut: Delft.
- Nederlandse Normalisatie-instituut (2005). *NEN-EN 1992-1-1.* Nederlandse Normalisatie-instituut: Delft.
- Nederlandse Normalisatie-instituut (2007). *NEN 6702.* Nederlandse Normalisatie-instituut: Delft.
- Stafford Smith, B, Coull, A (1991). *Tall Building Structures: Analysis and Design.* John Wiley & Sons in..
- Straman, J.P. (1988). *Geprefabriceerde stabiliteitsconstructies, de invloed van de verticale voegen.* TU Delft, Delft.
- Stufib (2006). *Constructieve samenhang van bouwconstructies.* Stufib, Bundschoten.
- Stupré commissie 53 (1993). *Verticale voorspanning van ruimtelijke kernelementen, Stupré rapport 24.* Nieuwegein.
- Terwel, K., Wijte, S., Windt, J. van der (2011). *Additional requirements for High Rise buildings in The Netherlands.*
- Vambersky, J.N.J.A. *Superhoog in Prefab*. Beton in Beeld 006-2007.
- Wit, S. de, Vrouwenvelder, T. (2008). *Cursus Windbelasting 2008.*

Master thesis
- TU Delft
  - Balbaid, H. (2011). *Application of Higher Strength Concrete in Tubular Structures.* Delft: TU Delft.
  - Falger, M.M.J. (2004). *Prefab stabiliteitsconstructies met open verticale voegen, Onderzoeksrapport.* Delft: TU Delft.
  - Meij, M. van der (2012). *Een bouwmethode van geprefabriceerde betonnen hoogbouw (>200m)*. Delft: TU Delft.
  - Narain, N (2011). *Winbelasting en Hoogbouw.* Delft: TU Delft.
  - Siersma, R (2005). *Progressive collapse of building structures.* Delft: TU Delft.
  - Tolsma, K.V. (2010). *Precast concrete cores in high-rise buildings.* Delft: TU Delft.
  - Winter, U.M. (2011). *Super high-rise in Rotterdam*. Delft: TU Delft.
- Others
  - Taniike, Y (1992). *Interference mechanism for enhanced wind forces on neighboring tall buildings*. Japan: Kyoto University,.

Internet:
- Architectenweb. http://www.architectenweb.nl/aweb/producten/product_detail.asp?productID=7187. Used on 15-04-2012.
- Betonson. http://www.betonson.com/producttypen/geisoleerde-kanaalplaatvloer/verwerking/voegen-4/default.aspx. Used on 28-02-2012.
- Bing maps. http://www.bing.com/maps/. Used on 17-11-2011.
- Bouwbesluit 2012. http://www.rijksoverheid.nl/onderwerpen/bouwregelgeving/documenten-en-publicaties/besluiten/2011/08/29/bouwbesluit-2012-staatsbladversie.html. Used on 30-03-2012.

- Google maps. http://maps.google.nl/. Used on 15-11-2011.
- Het strijkijzer. http://www.hetstrijkijzer.nl/strijkijzer?waxtrapp=nzfmbEsHunObnOhIYW. Used on 02-12-2011.
- Hurks Delphi Engineering. http://www.hurksdelphi-engineering.nl/show/nl/projecten/item/43,Stadskantoor_aan_de_Leyweg.html. Used on 01-03-2012.
- Nieuwbouw Erasmus MC. http://www.erasmusmc.nl/nieuwbouw/bouw/planningophoofdlijnen/. Used on 01-12-2011.
- Obayashi. http://www.thaiobayashi.co.th/html/html/oba_corp.php?page=technology. Used on 25-04-2012.
- Svensk Betong. http://www.svenskbetong.se/component/content/article/348.html. Used on 10-12-2011.
- Top100.nl. http://www.top010.nl/html/woontoren_zalmhaven.htm. Used on 16-11-2011.
- Top100.nl. http://www.top010.nl/html/de_hoge_heren.htm. Used on 16-11-2011.
- Top100.nl. http://www.top010.nl/html/hoge_erasmus_rotterdam.htm. Used on 16-11-2011.
- VBI. http://www.vbi.nl/?pageID=75&Title=VBI_leidingvloer. Used on 09-03-2012.
- Wikipedia. http://en.wikipedia.org/wiki/Ronan_Point. Used on 12-03-2012.
- Wikipedia. http://nl.wikipedia.org/wiki/Piramide_van_Cheopshttp://egypte-info-site.nl/piramiden-verhaal.html. Used on 08-03-2012.
- Wikipedia. http://en.wikipedia.org/wiki/Progressive_collapse. Used on 06-04-2012.
- Wikipedia. http://en.wikipedia.org/wiki/Boolean_algebra. Used on 06-05-2012.
- YPDGL. http://precastdesign.com/projects/high-rise/dalian_xiwang_gallery.php#13_dalian_xiwang/Xiwang_01.jpg. Used on 10-12-2011.
- Zalmhaven. http://www.zalmhaven.com/?item=downloads&count=true&selected=4. Used on 16-11-2011.

Others

- Corsmit PowerPoint presentation. Received from Jan Font Freide personally.
- Hurks Beton. Personal e-mails with Ron Vonken.

Appendices

Appendix A: Maple calculations

Appendix A contains a Maple sheet that is used to calculate the wind forces on the building. This sheet also contains the calculation for the acceleration of the building by the Eurocode and NEN 6702.

According to the NTA Hoogbouw (03-A Wind) report, $v_m(z_s)$ is determined with a lower basic wind velocity: 19.4m/s instead of 27m/s.

The basis of this Maple sheet is written by C. van der Ploeg and the sheet has been adapted for this thesis.

Calculation of windforce

Extreme wind pressure at location

General parameters

$\rho := 1.25 :$

$c_{dir} := 1.0 :$

$c_{season} := 1.0 :$

$zmax := 200 :$

$z0II := 0.05 :$

$k_l := 1.0 :$

$c_0 := z \rightarrow 1.0 :$

Basic windspeed [m/s]

$v_{b0} := 27 :$

Terrain categori III

$z0 := 0.2 :$

$zmin := 7 :$

$q_p := z \rightarrow \frac{1}{1000}\left(1 + 7 \cdot I_v(z)\right) \cdot 0.5 \cdot \rho \cdot v_m(z)^2 :$

$c_e := z \rightarrow \frac{q_p(z)}{q_b} :$

$q_b := 0.5 \cdot \rho \cdot v_b^2 :$

$v_b := c_{dir} \cdot c_{season} \cdot v_{b0} :$

$v_m := z \rightarrow c_r(z) \cdot c_0(z) \cdot v_b :$

$v_m(z) :$

$F_i := i \rightarrow piecewise\left(x = 0, 0, x \leq \Delta_{i,1}, k_{i,1} \cdot x, \Delta_{i,1} < x \leq \Delta_{i,2}, k_{i,2} \cdot x + \left(k_{i,1} - k_{i,2}\right) \cdot \Delta_{i,1}, \Delta_{i,2} < x, 0\right) :$

$c_r := z \rightarrow piecewise\left(z < zmin, k_r \cdot \ln\left(\frac{zmin}{z0}\right), zmin \leq z \leq zmax, k_r \cdot \ln\left(\frac{z}{z0}\right)\right) :$

$c_r(z);$

$$\begin{cases} 3.555348061\, k_r & z < 7 \\ k_r \ln(5.000000000\, z) & 7 \leq z \text{ and } z \leq 200 \end{cases} \quad \textbf{(1.1)}$$

$\xrightarrow{\text{simplify symbolic}}$

$$\begin{cases} 3.555348061\, k_r & z < 7. \\ 8.000000000\ 10^{-9}\, k_r \left(2.01179739\ 10^{8} + 1.25000000\ 10^{8} \ln(z)\right) & z \leq 200. \\ 0. & 200. < z \end{cases} \quad \textbf{(1.2)}$$

$$k_r := 0.19 \left(\frac{z0}{z0II}\right)^{0.07}:$$

$$l_v := z \rightarrow piecewise\left(z < zmin, \frac{\sigma_v}{v_m(zmin)}, zmin \leq z \leq zmax, \frac{\sigma_v}{v_m(z)}\right):$$

$$\sigma_v := k_r \cdot v_b \cdot k_l:$$

$$q_p(z);$$

$$455.6250000 \left(\frac{1}{1000} + \frac{7}{1000} \right. \quad \textbf{(1.3)}$$

$$\left. \begin{cases} \dfrac{\begin{matrix} 0.2812664141 \\ 0.2093619720 \end{matrix}}{\begin{cases} 0.7443546812 & z < 7 \\ 0.2093619720 \ln(5.000000000\, z) & 7 \leq z \textbf{ and } z \leq 200 \end{cases}} & \begin{matrix} z < 7 \\ 7 \leq z \textbf{ and } z \leq 200 \end{matrix} \end{cases} \middle/ \left(\begin{cases} 0.7443546812 & z < 7 \\ 0.2093619720 \ln(5.000000000\, z) & 7 \leq z \textbf{ and } z \leq 200 \end{cases}\right)^2 \right)$$

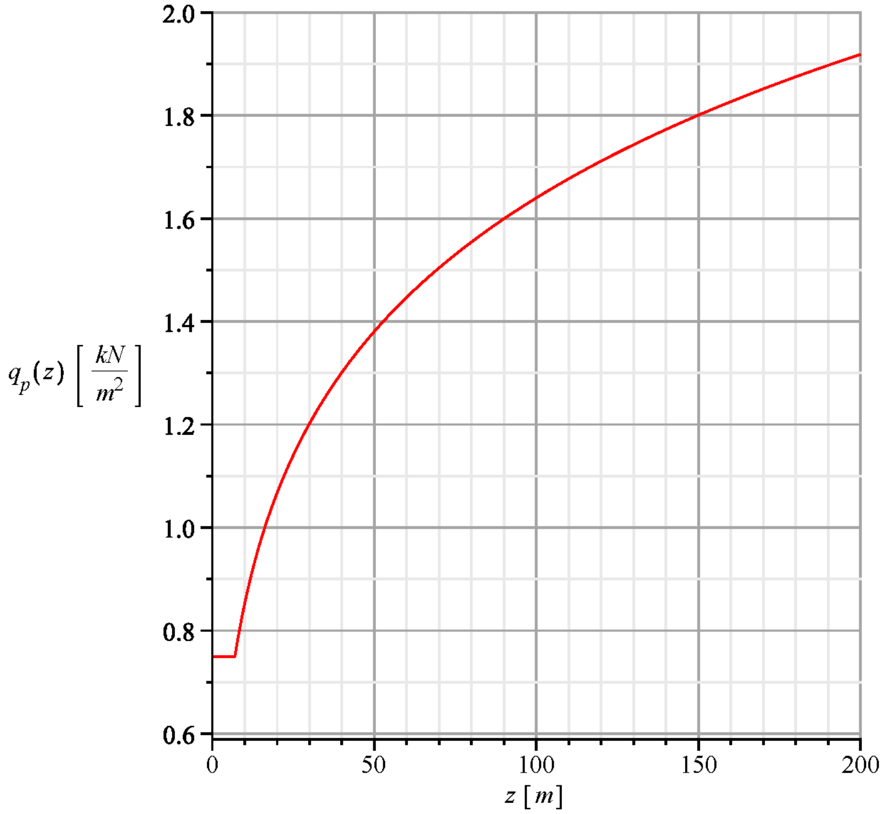
$\xrightarrow{\text{simplify symbolic}}$

$$\begin{cases} 0.7494761698 \\ 1.366875000\ 10^{-19} \left(3.369550951\ 10^{9} + 2.093619720\ 10^{9} \ln(z)\right) \left(6.00829633\ 10^{8} + 6.9787324\ 10^{7} \right. \\ 0. \end{cases}$$

$\overset{\text{simplify}}{=}$

$$\begin{cases} 0.7494761698 & z < 7. \\ 0.01997115334 \left(\ln(z) + 8.609437915\right) \left(1.609437912 + \ln(z)\right) & z \leq 200. \\ 0. & 200. < z \end{cases} \quad \textbf{(1.5)}$$

$$plot\left(q_p(z), z=0..200, q_p=0.6..2.0,\ labels=\left[z\,[m], 'q_p(z)\left[\frac{kN}{m^2}\right]'\right], gridlines=true\right)$$



Building parameters

Building width [m].

$bb := 30:$

Building height [m].

$h := 200:$

Force coefficient for building shape [/].

$c_f := 1.3:$

Mass per unit of height [kg/m].

$m_e := 431785.5035:$

First natural frequency [Hz].

$n1 := 0.193:$

Second order effect [/].

$n := 1.1:$

Structural factor cscd

Reference height [m].

$zs := \max(0.6 \cdot h, zmin)$:

Background factor B

Back ground factor allowing for the lack of full correlation of the pressure on the structure surface,

$B :='B'$:

Turbulent length scale ar reference heigt:

$\text{alpha} := 0.67 + 0.05 \cdot \ln(z0)$:

$zt := 200 : Lt := 300$:

$$L := z \to piecewise\left(z < zmin,\ \text{Lt} \cdot \left(\frac{\text{zmin}}{zt}\right)^{\text{alpha}}, zmin \leq z, \text{Lt} \cdot \left(\frac{\text{z}}{zt}\right)^{\text{alpha}}\right):$$

$$B := \text{sqrt}\left(\frac{1}{1 + \frac{3}{2} \cdot \text{sqrt}\left(\left(\frac{bb}{L(zs)}\right)^2 + \left(\frac{h}{L(zs)}\right)^2 + \left(\frac{bb}{L(zs)} \cdot \frac{h}{L(zs)}\right)^2\right)}\right)$$

0.6483841927 **(3.1)**

Resonance responce factor R

The resonance response factor allows for turbulence in resonance with the considered vibration mode of the structure

Dimensilose freqency: structure / wind:

$$f_L := (z, n) \to \frac{n \cdot L(z)}{v_m(z)} :$$

Wind power spectra density function:

$$S_L := (z, n) \to \frac{6.8 \cdot f_L(z, n)}{\left(1 + 10.2 \cdot f_L(z, n)\right)^{\frac{5}{3}}} :$$

$G_y := \frac{1}{2} : G_z := \frac{5}{18} : \phi_y := \frac{c_y \cdot bb \cdot n1}{v_m(zs)} : \phi_z := \frac{c_z \cdot h \cdot n1}{v_m(zs)} : c_y := 11.5 : c_z := 11.5 :$

Size reduction function:

$$K_s := n \rightarrow \frac{1}{1 + \mathrm{sqrt}\left(\left(G_y \cdot \phi_y\right)^2 + \left(G_z \cdot \phi_z\right)^2 + \left(\frac{2}{\pi} \cdot G_y \cdot \phi_y \cdot G_z \cdot \phi_z\right)^2 \right)} :$$

Logarithmic decrement of damping:

$deltas := 0.1 :$

$deltaa := \frac{c_f \cdot \mathrm{rho} \cdot bb \cdot v_m(zs)}{2 \cdot n1 \cdot m_e};$

$$0.01057675385 \qquad (3.2)$$

$deltad := 0 :$

$\mathrm{delta} := deltas + deltad;$

$$0.1 \qquad (3.3)$$

Fundamental vibration

$\Phi_1 := z \rightarrow \left(\frac{z}{h}\right)^{\xi} :$

$\xi := 1.0 :$

Resonance responce factor

$R := evalf\left(\mathrm{sqrt}\left(\frac{\pi^2}{2 \cdot \mathrm{delta}} \cdot S_L(zs, n1) \cdot K_s(n1)\right)\right)$

$$1.040063352 \qquad (3.4)$$

Peak factor:

$T := 600 :$

$\nu := evalf\left(\max\left(n1 \cdot \mathrm{sqrt}\left(\frac{R^2}{B^2 + R^2}\right), 0.08\right)\right) :$

$k_p := \max\left(\mathrm{sqrt}\left(2 \cdot \ln(\nu \cdot T)\right) + \frac{0.6}{\mathrm{sqrt}\left(2 \cdot \ln(\nu \cdot T)\right)}, 3\right) :$

$$(3.5)$$

Structural factor:

$cscd := evalf\left(\frac{1 + 2 \cdot k_p \cdot I_v(zs) \cdot \mathrm{sqrt}\left(B^2 + R^2\right)}{1 + 7 \cdot I_v(zs)}\right);$

$$1.067966771 \qquad (3.6)$$

Extreme pressure over height

if $h \leq bb$ **then** $q_{p,final} := z \rightarrow piecewise(z > 0, q_p(h))$ **end if**
if $bb < h \leq 2\,bb$ **then** $q_{p,final} := z \rightarrow piecewise(z \leq bb, q_p(bb), z > bb, q_p(h))$ **end if**
if $h > 2\,bb$ **then** $q_{p,final} := z \rightarrow piecewise(z \leq bb, q_p(bb), bb < z < h - bb, q_p(z), z \geq h - bb, q_p(h))$ **end if**

$$z \rightarrow piecewise(z \leq bb, q_p(bb), bb < z \text{ and } z < h - bb, q_p(z), h - bb \leq z, q_p(h)) \quad \textbf{(4.1)}$$

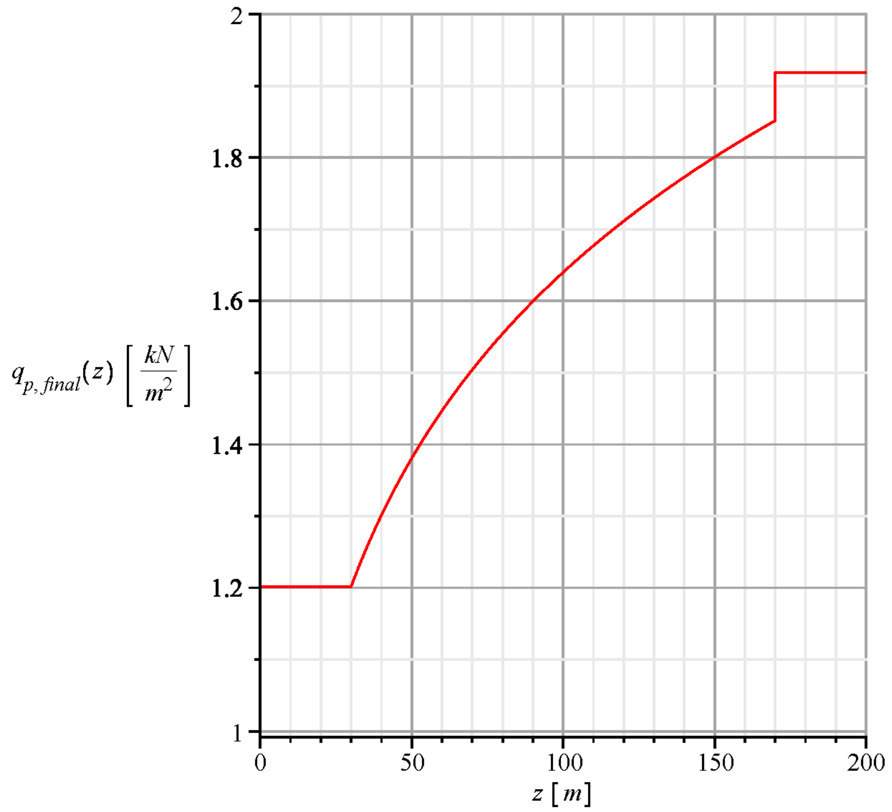
$q_{p,final}(z)$

$$\begin{cases} 1.201882244 & \\ 455.6250000 \left(\frac{1}{1000} + \frac{7}{1000} \begin{cases} \dfrac{0.2812664141}{0.2093619720} & \\ \hline \begin{cases} 0.7443546812 & z < 7 \\ 0.2093619720 \ln(5.000000000\, z) & 7 \leq z \text{ and } z \leq 200 \end{cases} \end{cases} \right. & z \quad 7 \leq z \text{ a} \\ 1.918656061 & \end{cases}$$

$\xrightarrow{\text{simplify symbolic}}$

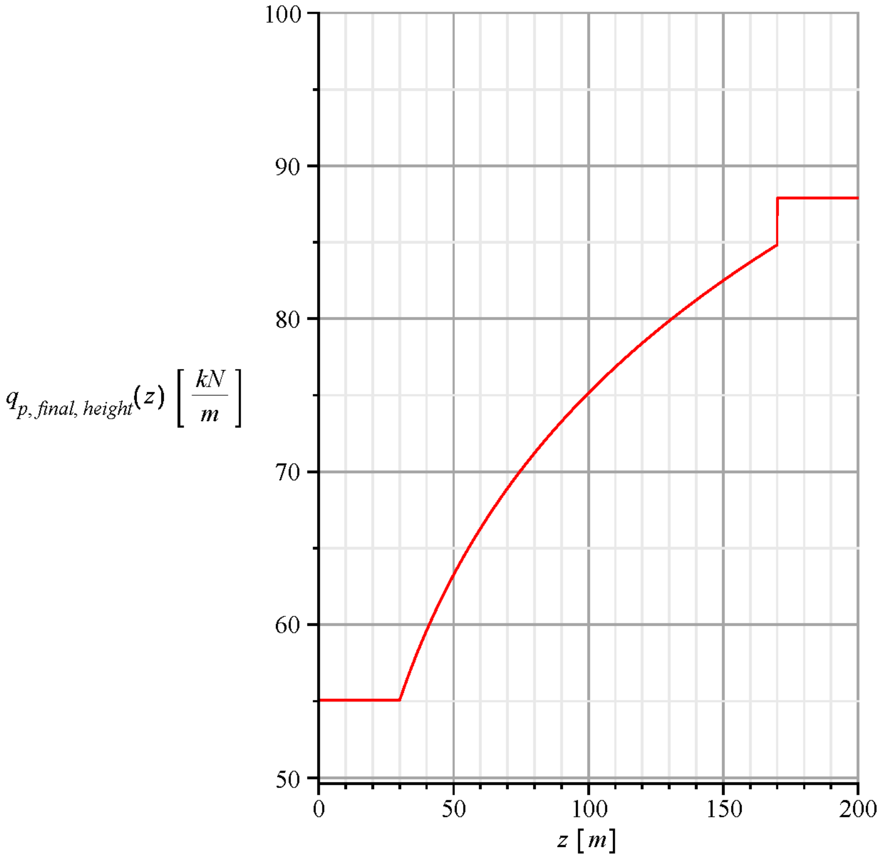
$$\begin{cases} 1.201882244 \\ 1.366875000\ 10^{-19} \left(3.369550951\ 10^{9} + 2.093619720\ 10^{9} \ln(z)\right) \left(6.00829633\ 10^{8} + 6.9787324\ 10^{7} \ln(z)\right) \\ 1.918656061 \end{cases}$$

$plot\left(q_{p,final}(z), z = 0..h, q_{p,final} = 1.0..2.0, labels = \left[z\,[m], 'q_{p,final}(z) \left[\frac{kN}{m^2}\right]'\right], gridlines = true\right)$



Total horizontal load over the height (kN/m)

$$plot\left(cscd \cdot bb \cdot n \cdot c_f \cdot q_{p,\,final}(z),\ z = 0\,..h,\ q_{p,\,final} = 50\,..100,\ labels = \left[z\ [m], 'q_{p,\,final,\ height}(z)\ \left[\frac{kN}{m^1}\right]'\right],\ gridlines = true\right)$$



To calculate the acceleration, $v_m(z_s)$ is changed from 27m/s to 19.4m/s.

Building acceleration NEN-EN 1991-1-4

$$z := 200 : c_f := 1.3 : br := 30 : \rho := 1.25 : K_y := 1.0 : K_z := \frac{5}{3} : \mu_{ref} := \frac{847163.1579 \cdot \left(\frac{1000}{9.81}\right)}{z \cdot br};$$

$$14392.85012 \quad (6.1)$$

$$l_{v1} := \frac{\sigma_v}{v_m(zs)}$$

$$0.1563249956 \quad (6.2)$$

$$v_m(zs)$$

$$25.98191186 \quad (6.3)$$

$$\sigma := \frac{c_f \cdot \rho \cdot K_y \cdot K_z \cdot l_{v1} \cdot v_m(zs)^2 \cdot R}{\mu_{ref}};$$

$$0.01572614079 \quad (6.4)$$

$$a := k_p \cdot \sigma;$$

$$0.05030095112 \quad (6.5)$$

Building acceleration NEN 6702

$$br := 30 : C_t := 1.2 : H := 200 : De := 0.01 : f_e := 0.193 : \rho_1 := \frac{892642.1053 \cdot \left(\frac{1000}{9.81}\right)}{H};$$

$$4.549653952\ 10^5 \quad (7.1)$$

$$\phi_2 := \mathrm{sqrt}\left(\frac{\left(0.0344 \cdot f_e^{\left(-\frac{2}{3}\right)}\right)}{De \cdot (1 + 0.12 \cdot f_e \cdot H) \cdot (1 + 0.2 \cdot f_e \cdot br)}\right)$$

$$0.9205973951 \quad (7.2)$$

$$p_{w,1} := 100 \cdot \ln\left(\frac{H}{0.2}\right)$$

$$690.7755279 \quad (7.3)$$

$$a_{\max} := \frac{1.6 \cdot C_t \cdot br \cdot \phi_2 \cdot p_{w,1}}{\rho_1}$$

$$0.08051018101 \quad (7.4)$$