# High-Rise

## Exploring the Ultimate Limits


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*Picture on cover: Tower of Babel by M.C. Escher (Woodcut 62.1 x 38.6 cm, 1928)*

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# Abstract

It has always been human's desire to build ever taller structures. Look at ancient structures such as the pyramids of Egypt, the Mayan temples in Central America or the Gothic cathedrals in France. The mid 1800's can be marked as the beginning of the era of the modern skyscraper and the city of Chicago as its birthplace. Due to a lack of available land for new buildings, the inhabitants of Chicago were forced to build skywards. Later on New York followed Chicago in constructing ever taller skyscrapers. Furthermore, the evolution of the steel structure, the introduction of the electric motor and the development of Otis' safe lift system cleared the way for the race for the skies to commence. Until recently, the United States of America were the most important contestants in this race. However, since the turn of the 1990s Asian countries started to get involved and have become the leading contestants of today. Driven by their fast economic growth, these new contestants want to promote their country and its cities by constructing ever taller skyscrapers. The question which comes into mind is: For how long can this race for the skies continue? Will it be possible to build ever higher, or will we eventually encounter insuperable difficulties which bring the race for the skies to a close? Encouraged by this question the goal of this master's thesis is:
*"To gain a good insight into the challenges which will be encountered when designing and constructing an ultra-tall skyscraper, with the aim to find the ultimate limit to the height of the skyscraper."*
This goal consists of two parts. In the *first* part of the thesis we will gain a good insight into the subject. This means that each of the encountered challenges will be carefully examined. In the *second* part of the thesis this knowledge is used to explore the limits to high-rise.

When designing and constructing a skyscraper, a wide range of challenges will occur. These challenges vary from structural and serviceability challenges to safety and economical challenges. In this thesis, 14 different challenges are examined:

**Foundation:** The load-bearing capacity of the building's foundation can seriously limit the building's height. The load-bearing capacity of the foundation is determined by the properties of the subsoil and the chosen foundation type.

**Load-bearing Structure:** It goes without saying that the load-bearing structure of a skyscraper can seriously limit its height. There is a wide variety of building structures which can be used: frame structures, shear wall structures, core structures, tube structures, outrigger-braced structures and finally special high-rise structures like spatial frames and suspended structures.

**Slenderness:** The depth of skyscrapers is limited by the desire to have sufficient daylight into the

building. These limitations to the depth of the building mean that together with the height of the building, the slenderness of the skyscraper increases. A more slender building is more susceptible to vibrations and it is harder to safeguard the stability of the structure.

**Comfort:** Due to their high slenderness, skyscrapers are more susceptible to oscillations. Oscillations can occur when the building resonates under the action of external loads, e.g. seismic ground accelerations, wind action and man-induced excitations. Constant vibrations and movements of the building have a negative effect on the comfort felt by the building's occupants.

**Earthquakes:** In certain areas of the world earthquakes occur. The lateral movements caused by these earthquakes can seriously damage a structure or can even cause it to collapse. Apart from this, an earthquake is often accompanied by other hazards which can easily destroy a structure.

**Influence on the Surroundings:** A skyscraper influences its surroundings and vice versa. Anticipated unfavourable effects caused by a new skyscraper, can impose a limit to its height.

**Organising the Building Site:** Because most high-rise projects are realised in densely populated city centres, organising the building site becomes a top-priority. Furthermore, high-rise projects are complex and require large quantities of building materials.

**Vertical Transportation:** In both the realisation and operating phase of the skyscraper the focus lies most in vertical transportation. Due to the limited amount of space, it is very likely that bottlenecks will occur. This can seriously limit the height of the structure.

**Fire Safety:** The most terrifying event that can happen in a building is the outbreak of a fire. This concerns in particular high-rise buildings since there is no possibility of escaping through the windows of the building. Additionally all firefighting and rescue operations have to be carried out inside the building, resulting in strict requirements for the structural safety and stability of the building.

**Terrorist Attacks:** Since the terrorist attacks on the World Trade Centre in New York City, there is a general concern about the safety of high-rise buildings with respect to terrorist attacks. This anxiety could end the desire to build ultra-tall skyscrapers.

**Evacuating the Building:** Despite the great height of the building and its increasing population, it must still be possible to safely evacuate all of the building's occupants in time. This puts pressure on the skyscraper's escape routes. The taller the building, the larger these problems will be.

**Economical Feasibility:** Often skyscrapers are constructed in order to promote a company or investor. However, they still have to be seen as real estate ventures which means that the buildings have to be economically feasible. Some concerns which can negatively influence the economical feasibility of a high-rise building are: its high construction and maintenance costs and the building's gross-nett floor ratio.

**Sufficient Economical Support:** The location of an ultra tall-skyscraper has be to capable to economically support such an enormous structure. In other words, the economical environment has to be sound.

**Market Instability:** Like the economy, the real estate market is subjected to cycles. However, the cycles of the real estate market are much more extreme. When skyscrapers grow ever taller, the time span between initiation and completion will increase as well, resulting in higher risks for the investors.

With the careful examination of the challenges, the first goal of this thesis is achieved: obtaining a good understanding into the thesis' subject. Now the limits to high-rise buildings can be explored. In order to do this, the thesis' subject is reduced to a more manageable size in three consecutive steps. *First*, the list of 14 challenges is reduced to a list of six. *Second*, a location for the building is chosen: The Netherlands. And *third*, a benchmark skyscraper is described in which the geometry of the building is established. For a skyscraper it is most efficient to have its load-bearing structure on the perimeter of the building, since this creates the largest possible internal lever arm. Therefore, the benchmark skyscraper is given a rectangular "Tube Structure" with a centre-to-centre distance of 2.5 metres between the columns. Two types of columns are examined; concrete columns and steel box columns.

Now, the limits to the benchmark skyscraper can be established for the six remaining challenges. These limits are established by using the regulations and building codes which are valid in The Netherlands. Only when these codes fail to give an explanation, building codes from other nations will be used. The computed limits are given in the table 1. It can be concluded that the found limits are lower than what has already been achieved in high-rise construction worldwide. This means that the assumed geometry of the benchmark skyscraper has proven to be too much confining.

| Challenge | Concrete | Steel |
|---|---|---|
| Load-bearing Structure: | *344* | *700* |
| Foundation: | *344* | *344* |
| Economical Feasibility: | *475* | *475* |
| Comfort: | *380* | *520* |
| Vertical Transportation: | *720* | *720* |
| Evacuating the Building: | *154* | *154* |

**Table 1:** *Overview of the limits imposed on the benchmark skyscraper by each challenge (in metres)*

Due to the somewhat unsatisfactory results, a closer look has to be taken into each of the six challenges, to consider how the earlier found limits can be pushed skywards. A series of possible solutions are assessed. It turns out that some of these solutions are shared by multiple challenges. These correlations can be best visualised by making a web diagram which contains all the possible solutions. From this diagram follows that altering the skyscraper's form seems to be the most promising solution.

Changing the shape of the building can stretch the skyscraper's limits with regard to its load-bearing structure, comfort criterion and economical feasibility. Changing the building's form can be achieved

in three ways: By changing the footprint of the building, by giving the building a tapering shape or by creating a compound structure, i.e. interconnecting multiple slender towers to form one structural entity.
Keeping the gross floor area of the building constant, seven alternative building footprints are examined: a U-shape, a I-shape, a cross-shape, a T-shape, a Y-shape, a shifted rectangular shape and a circular shape. Changing only the footprint of the building proves to have no structural advantage. This is caused by the fact that the dimensions of the footprints are predetermined by the requirement that the skyscraper should show a similar behaviour for wind coming from all directions. This means that the possible increase of the cross-section's moment of inertia is limited. Together with an increase of the wind load due to a larger width of the building, no structural gain can be achieved. Only the circular shape shows a favourable behaviour, because the wind can flow easier around the building. A Y-shaped footprint proves to behave most stiff, resulting in smaller accelerations of the skyscraper and better comfort. Changing the building's footprint has no significant effect on the gross-nett floor ratio of the building.
Giving the building a tapering shape results in a significant gain with regard to the limit of the skyscraper's load-bearing structure. This is caused by the decreasing lateral and vertical loads acting on the structure. However, due to the large height of the structure, tensile forces are introduced into the columns of the building. This is highly undesirable. The limits with regard to the comfort criterion drop, due to the building's increased eigenfrequency and decreased mass. Also, no improvement can be obtained with regard to the gross-nett floor ratio of the skyscraper. The gross-nett floor ratio of the building can even deteriorate when there is no tapering building core applied.
Creating a compound structure proves to be a good solution: showing good results and higher limits all along the line. A major advantage is that no tensile forces have to be accommodated any longer by the windward columns of the structure.

When the other three remaining challenges are considered, it can be concluded that the limits to these challenges can be stretched to match those found for the load-bearing structure, comfort and economical feasibility of the building. The previous considered solutions for stretching the limits to these challenges, can be considered to be adequate.

It can be concluded that it is not possible to find an absolute limit to high-rise. This because the limits which are found, are largely dependent on the made assumptions and boundary conditions which are determined in the beginning. However, this report gives a well-founded reasoning that compound structures are the most promising solution when we want to build ultra-tall skyscrapers in the near future. So, in order to push the limits to high-rise, we have to turn to compound structures.
For pushing the limits to high-rise, a different approach than the one used in this thesis should be followed. In this report, the following approach was chosen: At first, all of the building's boundary conditions were determined. Based on this, the limits to the skyscraper were established. Instead, it would be better to predetermine the height of the skyscraper. Based on this target one can deduce which structure is needed to achieve this height and which concessions have to be made with regard to the building's serviceability and safety. By following this alternative approach, the thesis' subject is much more confined and easier to manage. This will also push the researcher to be creative and to think beyond conventional solutions.

*"It must be tall, every inch of it tall. The force and power of altitude must be in it, the glory and pride of exaltation must be in it. It must be every inch a proud and soaring thing, rising in sheer exultation that from bottom to top it is a unit without a single dissenting line."*

**Louis H. Sullivan (1856 – 1924)**

# Preface

This Master's thesis is the final piece of my study at the faculty of Civil Engineering and Geosciences at Delft University of Technology. I can look back on six great years in which I have had the opportunity to experience life as a student in Delft.

The first three years of my studies I experienced the diversity of the field of Civil Engineering. Later on I decided to specialise myself in the field of Building Engineering to learn more about the commercial and industrial building industry. During this period I have been fortunate to study for half a year in Gothenburg, Sweden at the Chalmers University of Technology as a part of the Erasmus Exchange Program. This has been a fantastic experience which I can recommend to every student.

The last few months of my studies I worked on this final master's thesis. In relation to this I would like to thank my graduation committee, Prof. dipl. ing. J.N.J.A. Vambersky, Ir. K.C. Terwel, Prof. ir. P.G. Luscuere, Prof. ir. C.H.C.F. Kaan and Ir. F.H. Middelkoop, for guiding me during this period despite their busy agendas. Their valuable input during the several meetings encouraged me and gave me the opportunity to improve my work.
I also express my gratitude to the following people for the discussions I had with them about my thesis' subject: David Scott, chairman of the Council on Tall Buildings and Urban Habitat (CTBUH); Antony Wood, executive director of the CTBUH; Johannes de Jong, member of the steering committee of the CTBUH; Henk-jan Schuurman, senior advisor at Oranjewoud and finally Bart Leclercq, associate director WSP Middle East.
I also want to thank my fellow graduates of room 0.72. Who made sure there was always a nice atmosphere to work in and with whom I had nice discussions about my thesis' subject ... or other very interesting things.
I also would like to thank my friends for helping me to take my mind off the thesis, when this was needed.
Finally, I want to thank my parents, my sister and Sylvie. Their mental (and, in case of my parents, also financial) support helped me to persevere, also when, sometimes things did not go as planned. Thanks a lot!

Arne Dijkstra
Delft, September 2008

*"The only way to discover the limits of the possible is to go beyond them into the impossible."*

**Arthur C. Clarke (1917 – 2008)**

# Contents

# Introduction

On the 27th of March 2008 the Burj Dubai in Dubai, United Arab Emirates, broke the record of being world's tallest structure on land. It overtook the 628.8 metres tall KVLY television mast in Blanchard, North Dakota, United States by 1.7 metres. Still under construction, the Burj Dubai already claimed the title of being world's tallest skyscraper, surpassing the 508 metres tall Taipei 101 in Taipei, Taiwan. Although its exact height is still top-secret, it is said that the Burj Dubai will eventually reach to a height well over 800 metres. This means that the Burj Dubai will undoubtedly hold the title of being "world's tallest skyscraper".
Nevertheless, new competitors have already emerged on the scene. Plans for other super-tall skyscrapers in Kuwait, Dubai and Saudi Arabia have already leaked out. It is said that the new skyscraper in Dubai, the Al Burj, will be a staggering 1050 metres tall. One thing is certain: The Burj Dubai will not be world's tallest for all that long.

It has always been human's desire to build ever taller and taller structures. Look at ancient structures such as the pyramids of Egypt, the Mayan temples in central America or the Gothic cathedrals in France. The mid 1800's can be marked as the beginning of the era of the modern skyscraper, and the city of Chicago as its birthplace. Chicago had become the focal point of commerce in the United States. Due to a large fire which broke out in 1871, almost the entire city was destroyed. However, the city was quickly rebuilt. Due to lack of available land for new buildings, the inhabitants of Chicago were forced to build skywards.
At the beginning, most of these skyscrapers were masonry wall bearing structures with thick walls. This limited the height of the buildings. But, the evolution of the steel structure and the invention of the mechanical lift and ventilation, cleared the way for the race of building world's tallest to commence.

Until recently the United States were the most important contestant in this race for the skies. In 1996, eight out of the ten world's tallest buildings were located in the US. Nowadays, Asian countries are the new contestants, counting eight out of the ten world's tallest skyscrapers on the Asian continent. Asia's involvement started at the turn of the 1990s decade. Driven by their fast economic growth, countries like Japan, Taiwan, Malaysia and China constructed super-tall skyscrapers in order to promote their country and its cities. Asian countries like the UAE, South Korea and Saudi Arabia have recently joined the race by starting to build their own super-tall skyscrapers. By building the Russia Tower in Moscow, Russia is also trying to get involved as, for now, the only true contestant on the European continent.

All the countries mentioned above, are competing to have world's tallest building within their country's borders. As soon as plans for a new record skyscraper are revealed, new design teams are immediately put together to make sure this building will not hold this title for long.

The question which comes into mind is: For how long can this race for the skies continue? Will it be possible to build ever higher, or will we eventually encounter insuperable difficulties which will bring the race for the skies to a close?

In this Master's thesis, the ultimate limits to a skyscraper will be explored. We will examine which challenges will be encountered when designing and building an ultra-tall building. Furthermore, will be determined which solutions are most promising when building an ultra-tall skyscraper.
This report is a valuable reference work for he or she who is interested in the skyscraper.

# Chapter 1

# Problem Description

## 1.1 Problem

When designing and constructing a skyscraper, a broad range of challenges will emerge. All of these challenges have to be tackled in order to complete the skyscraper. Most of these encountered challenges are related to the building's height. This often means that when the skyscraper gets taller, the encountered challenges will be larger. Some of the challenges may even relate exponentially to the height of the building, resulting in real brainteasers.

The challenges which will be encountered during the development of a skyscraper are listed below:

- The foundation of the building.
- The load-bearing structure of the building.
- The slenderness of the building.
- The comfort requirements of the building's occupants.
- Earthquake hazards.
- The skyscraper's influence on its surroundings.
- Organising the building site.
- The vertical transportation of people, goods and building services.
- The fire safety inside the building.
- The threat of terrorist attacks on the building.
- The evacuation of the building in case of an emergency.
- The economical feasibility of the building.

- The presence of sufficient economical support for the building.
- The instability of the real-estate market.

In the end, one of the encountered challenges becomes so complex that it becomes impossible or too costly to solve. This challenge has become the limiting factor in the building's design and will set a limit to the height of the skyscraper.
To the question: "How tall can we build?" Most engineers will answer: "The sky is the limit". But....how high is the sky?

## 1.2 Goal

The goal of this Master's thesis consists out of two parts. In the first part of the thesis we will gain a good insight into the thesis' subject. This means that each of the challenges will be carefully examined. In the second part of the thesis this knowledge is used to explore the limits to high-rise.

The goal of this thesis can be summarised in one single sentence:
*"To gain a good insight into the challenges which will be encountered when designing and constructing an ultra-tall skyscraper, with the aim to find the ultimate limit to the height of the skyscraper."*

## 1.3 Approach

To gain a proper insight into the thesis' subject, an extensive literature review has to be conducted. A good understanding has to be gained in each of the challenges. Additionally, knowledge about how to tackle these challenges has to be acquired.
After this the thesis' subject is reduced to a more manageable size by ruling out some of the examined challenges. Thereafter, the height limit for each of the remaining challenges will be computed.
Based on these calculated limits it can be determined whether it is possible to focus in on one challenge in particular, or whether it is preferable to continue to examine all the remaining challenges.

## 1.4 Outline of the Report

The *second* chapter in this report gives a short exposition of the history of the modern skyscraper. Technologies applied in high-rise construction today, can be comprehended better when the historical background is known. This chapter describes the birth of the modern skyscraper and describes its development throughout the decades that followed.
In the *third* chapter of the report, the challenges which are encountered when developing a skyscraper will be described. The chapter will provide understanding in each of the challenges and examines how each of the challenges can be tackled. This chapter gives a broad review on the existing knowledge and technologies which are used in high-rise construction.

In chapter *four*, four reference projects will be discussed. This chapter will discuss the tallest completed skyscraper of today and three new record skyscrapers which are still under construction. For each of the buildings the main characteristics will be described. Additionally, the building's structural system and wind engineering will be discussed.
In chapter *five*, the thesis' subject is reduced to a more manageable size. This is done in three consecutive steps.
In chapter *six*, computations are made in order to determine the limits to each of the remaining challenges. At the same time this chapter gives a valuable overview of the calculation methods and building codes which have to be used. The chapter is concluded with an overview of the challenges and their corresponding limits.
In chapter *seven* is considered how the limits found in the previous chapter can be pushed skywards. For each of the challenges a tree diagram is put up to depict the possible measures which can lead to higher skyscrapers. In the end all these separate tree diagrams are combined into one large web diagram from which can be determined which of the measures are most promising.
In chapter *eight* a study is made to determine to which extend the shape of the building influences its height. In this chapter recommendations are made to indicate which building form is best when one wants to build an ultra-tall skyscraper.
In chapter *nine*, feedback is given on how the changed form of the skyscraper influences the other challenges.
In chapter *ten*, the thesis' report is concluded by listing the conclusions which can be drawn from this thesis and by considering the recommendations.

# Chapter 2

# Historical Background

This master's thesis is an exploratory study to the ultimate limits to high-rise. To be able to determine these extremes, we have to consider the current knowledge in the field of building technology. This chapter will describe the evolution of the modern skyscraper and examines the developments in the building technology which made this evolution possible.

## 2.1 History

It has always been human's desire to build tall buildings. In the beginning tall buildings were primarily constructed motivated by political or religious reasons, e.g. the pyramids in Egypt, the Mayan temples in Central America, the Gothic cathedrals of France, etc. (figure 2.1).

But, even in ancient times multistory buildings were used to house people. It is claimed that the Romans built already ten-storey tenement buildings. These buildings were made out of wood. It is said that Emperor Augustus later limited the height of these buildings in order to reduce the risk of fire. In addition to this, the Romans started to use incombustible construction materials like brick and even concrete. In ancient urban centres like Babylon, Athens and Byzantium four-storey apartment buildings constructed from mud-brick with timber floors were quite common (Schueller [41]).
In the following centuries the common building materials were stone, brick and timber. It was not until the turn of the nineteenth century that the traditional way of constructing buildings changed dramatically.

Chicago can be considered as the birthplace of the modern skyscraper. In 1871, a large fire destroyed a major part of the city, reducing almost 17.5 thousand buildings to ashes. In the years that followed, Chicago grew explosively and it quickly began to strain against its natural boundaries. On the north and west side, the city was closed in by the Chicago River, on the east and south side the city was enclosed by Lake Michigan and the railways. The citizens were forced to build skywards in order to solve this problem. Later on New York followed Chicago in constructing ever taller buildings.
Another major development that further paved the way for the skyscraper, was the introduction of the safe lift system in 1853. Although lifts were already in use for over a decade, Elisha Graves Otis

(a) The Great Pyramids of Giza, Egypt

(b) Mayan temple at Tikal, Guatemala

(c) Gothic Cathedral in Chartres, France

**Figure 2.1:** *Ancient structures*

invented a system that prevented a lift car from falling down if the hoisting cable should break. Later the steam-operated lifts were replaced by lifts working on an electric motor. This made the lift a practical solution for the vertical transportation in tall buildings. This allowed the building's owner to ask higher rents from tenants on the higher floors, making the building more economical.
A race for the skies was initiated, a contest which still rages on today.

When we look to the evolution of the skyscraper in a structural perspective, we can divide the development of the skyscraper into three different periods. In each of these periods another building material dominated high-rise construction.
The *first period* has already been briefly discussed in the preceding text of this chapter. During this period masonry load-bearing walls were used to accommodate the gravitational and lateral forces acting on the building. Later on in this period the masonry interior walls were replaced by a system of iron beams and columns, creating a so-called "cage structure". First cast-iron was used for both the beams and columns. Later on the cast-iron beams were replaced by wrought-iron beams which had a much better tensile capacity. Steel was also making its appearance in the high-rise construction. However, the outside walls of the skyscrapers were still constructed of masonry. Because of the high dead load of a masonry wall, the thickness of the walls had to increase together with the height of the building in

order to accommodate its own weight at the base. As a consequence of this, the percentage of floor area occupied by the vertical structural elements (walls and columns) was large compared to the total gross floor area of the building, making it highly inefficient. The height of high-rise was limited because the resisting surface area at the base could not become reasonably larger (Schueller [41]). Masonry construction literally reached its peak in 1891 with the construction of the Monadnock Building in Chicago (figure 2.2). This 17-storey building reached a height of 60 metres. The masonry walls are 2.13 metres thick at the base and they occupy 15% of the gross floor area at ground level.

**Figure 2.2:** *The Monadnock Building in Chicago, US*

In the *second period* the cage construction was further developed into a frame structure. The exterior load-bearing walls were replaced by a frame of iron columns and beams, making the structure of the building a true skeleton. Now that the load-bearing masonry walls had been replaced, the lateral forces acting on the structure had to be resisted by the frame. The limits to the height of a skyscraper imposed by the masonry walls, were removed, making the construction of taller skyscrapers possible. The second period meant a boom in high-rise construction. Ever taller skyscrapers were built at an incredible pace. The first iron frame structure was already built in 1797, for the construction of The Flaxmill in Shrewsbury, England. This is the oldest iron framed building in the world and is seen as the "grandfather of the skyscrapers". However, it was not until the year 1885 that the first skyscraper with an iron frame structure was constructed in Chicago. With a height of 55 metres (10 floors), the Home Insurance Building (figure 2.3) is generally considered as world's first true skyscraper. An other breakthrough was the completion of the Masonic Temple in 1892 (figure 2.4). The 92 metres tall building was the tallest of its time. The facade frames were laterally braced with diagonals forming a vertical truss, the forerunner of modern shear wall and braced frame structures (Smith and Coull [45]).

Improved design methods and construction techniques allowed the height of steel frame structures to increase steadily. In 1913 the Woolworth Building in New York was completed (figure 2.5). Reaching a

**Figure 2.3:** *Home Insurance Building in Chicago, US*

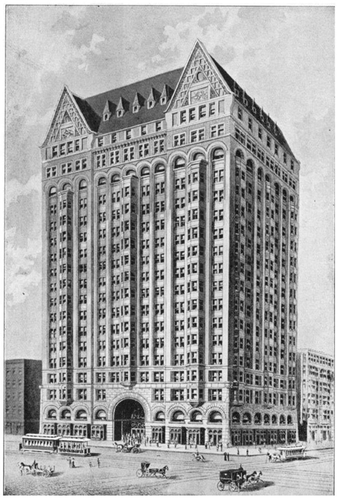


**Figure 2.4:** *Masonic Temple in Chicago, US*

height of 241 metres, for 17 years it held the record of being world's tallest before it was overtaken by the Trump Building (283 metres) and the Chrysler Building (319 metres)(figure 2.6), both completed in 1930. Already a year later, in 1931, the Empire State Building took the title (figure 2.7). With its 102 Floors and 381 metres height, it was the first skyscraper which was taller than the Eiffel Tower in Paris, France.

**Figure 2.5:** *Woolworth Building in New York, US*

**Figure 2.6:** *Chrysler Building in New York, US*

**Figure 2.7:** *Empire State Building in New York, US*

The economic depression of the 1930s and the following Second World War put a temporary ending to race for the skies. Only in 1973 the World Trade Centre's Twin Towers in New York (figure 2.8) (415 and 417 metres) overtook the Empire State Building. In 1974 the 442 metres high Sears Tower in Chicago (figure 2.9) snatched the title of being world's tallest. The most recent example of a skyscraper with a steel structure is the Taipei 101 in Taipei, Taiwan (figure 2.10). At the time of writing this building is the tallest, completed skyscraper in the world, counting 101 floors and reaching a height of 509 metres.

**Figure 2.8:** *World Trade Centre in New York, US*

**Figure 2.9:** *Sears Tower in Chicago, US*

**Figure 2.10:** *Taipei 101 in Taipei, Taiwan*

In the *third period*, the use of concrete is introduced in high-rise construction. In comparison to the rapid introduction of steel into high-rise construction, the developments in reinforced concrete were slow. Prior to the First World War reinforced concrete was primarily used for the foundations of buildings and occasionally for the construction of floor slabs. Nevertheless, already in 1903 the world's first reinforced concrete skyscraper was completed in Cincinnati in the United States. The Ingalls Building (figure 2.11) counts 16 floors and reaches a height of 65 metres. The tallest steel structure high-rise building at that time was the Park Row Building in New York City. With its 119 metres this building had almost twice the height of the Ingalls Building, illustrating the lead of steel in high-rise construction. It was only after the Second World War that reinforced concrete became more popular. Before, concrete structures in high-rise buildings had been treated merely as a substitute for steel, by exactly imitating steel frames. This practice resulted in heavy members and especially large column sizes for the lower floors of tall buildings (Schueller [41]). It were pioneers like Joseph DiStasio and Frank Lloyd Wright who were the first to apply reinforced concrete in its full potential. These developments led in 1964 to the completion of the world's first true skyscrapers in reinforced concrete: the 60-stories (179 metres) tall Marina Towers in Chicago (figure 2.12). A race was initiated, resulting in the completion of the 197 metres tall Lake Point Tower in 1968 in Chicago (figure 2.13). Only three years later, another record was set by the 218 metres tall One Shell Plaza in Houston.

In the United States skyscrapers have mainly been constructed using steel. Now, with other parts of the world joining in the race for the skies, reinforced concrete is being used in skyscrapers which are

**Figure 2.11:** *Ingalls Building in Cincinnati, US*

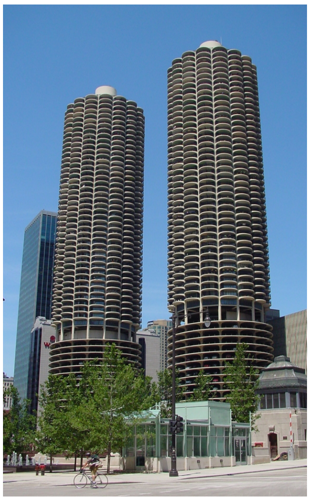


**Figure 2.12:** *Marina Towers in Chicago, US*

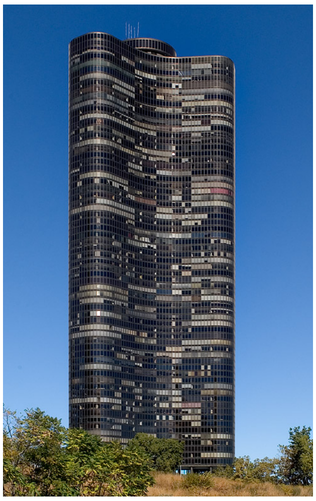


**Figure 2.13:** *Lake Point Tower in Chicago, US*

comparable to the tall steel skyscrapers. In 1997 the Petronas Towers (452 metres) in Kuala Lumpur in Malaysia (figure 2.14) were the first skyscrapers, mainly constructed out of concrete, to claim the title of being world's tallest. The building has a concrete core and concrete mega-columns in the facade of the building. Interaction between the core and the columns is made possible by steel outriggers. Although, the Petronas Towers have been overtaken by the steel structure of the Taipei 101, the combination of steel and reinforced concrete in high-rise construction seems to be the best solution for the next generation of skyscrapers. In Dubai, in the United Arab Emirates, a new record-holder is under construction. The Burj Dubai (figure 2.15) is expected to be completed in the year 2009 and is estimated to reach a height of approximately 800 metres (the exact height of the tower is top-secret). The Burj Dubai is again largely constructed out of reinforced concrete. Only the tip of the tower is made of a steel frame structure.

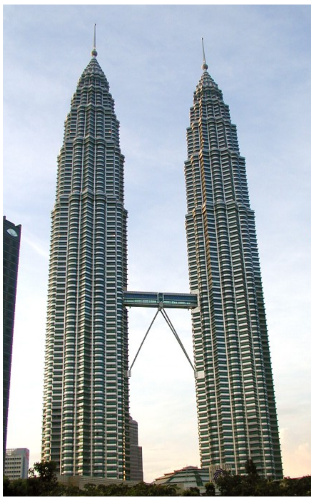


**Figure 2.14:** *Petronas Towers in Kuala Lumpur, Malaysia*

**Figure 2.15:** *Burj Dubai under construction in Dubai, UAE*

# Chapter 3

# Challenges in High-rise Construction

This chapter will give an overview of the challenges which are encountered when designing and constructing a skyscraper. Each section will deal with a challenge and will give a detailed explanation based on found literature.

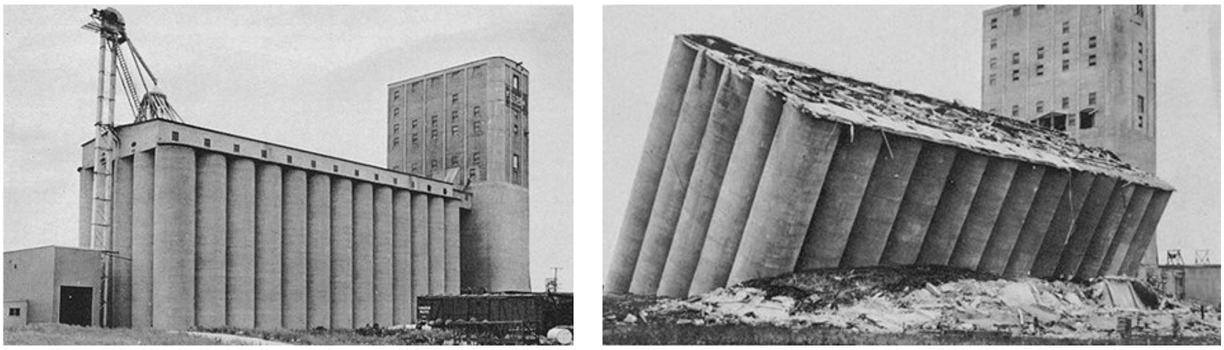
## 3.1 Foundation

The local ground conditions and the behaviour of the subsoil are major risk factors in the building industry, particularly in high-rise construction. Insufficient load-bearing capacity of the subsoil can result in serious settlements of the structure. These settlements can cause serious damage to both the structure itself, as to structures and infrastructure in the building's surroundings. A well-known example is the tilting of a grain silo in Winnipeg, Canada in 1913 (see figure 3.1). Geotechnical failure caused the structure to tilt. This was caused by an uneven loading on the foundation of the structure.
Excessive settlements are not always intolerable. The in the previous chapter discussed Monadnock Building (figure 2.2) has settled approximately 60 centimetres since its completion and virtually no damage has occurred.

### 3.1.1 Subsoil Types

We can distinguish four different kind of subsoil properties (Eisele and Kloft [11]):

- Subsoil with young sediments, e.g. peat, silt or clay. This soil has a low load-bearing capacity.
- Subsoil with geologically prestressed sediments and loosely packed sand or gravel. This soil has a limited load-bearing capacity.
- Subsoil with densely packed sand or gravel. This soil has a good load-bearing capacity.
- Subsoil consisting of stones and rock. The load-bearing capacity of this type of soil is uncertain. The capacity to bear loads is largely dependent on the properties of the rock. Soft rock types



(a) Grain silo before geotechnical failure of the soil (b) Grain silo after geotechnical failure of the soil

**Figure 3.1:** *Grain silo in Winnipeg, Canada*

like marl, sandstone and limestone have a poor load-bearing capacity. On the contrary, hard rock types like basalt and quartzite have a high load-bearing capacity.

The rate at which settlements occur, depends on the permeability of the soil types. Peat, silt and clay are so-called cohesive soils. This means that these soils have a very low permeability and water is squeezed out slowly when a load is applied on these soil layers. As a result of this the layers are compressed slowly and a substantial part of the final settlement of the building is due to long-term consolidation movements (Schueller [41]). On the contrary, the settlements of noncohesive soils, like sand or gravel, are primarily of an immediate nature. Water is squeezed out quickly through the large pores in between the soil's grains.

### 3.1.2 Foundation Types

The ground conditions determine the type of foundation which has to be applied to transfer the heavy loads from the skyscraper onto the subsoil underneath the building. Three different kind of foundation types can be used in high-rise construction.

**Shallow foundations:** Shallow foundations can only be used when the subsoil has a good load-bearing capacity. This foundation type is usually by far the most economic option. The downside of choosing this type of foundation is that significant settlements occur, even if the soil conditions are satisfactory. We can further divide this group into three different types of shallow foundations:

*First*, the spread footings. This is nothing else than a widening at the base of the load-bearing member. The members transfer their forces directly onto the subsoil underneath the building. Due to the widening, the vertical forces are spread over a larger area. A single column can be supported by a separate "isolated footing" or a group of columns can be supported by a single "combined footing". Load-bearing walls or rows of columns are generally supported by a so-called "strip footing".

*Second*, the raft foundation (figure 3.2). A raft foundation is basically one large continuous footing upon which the entire building rests. This type of shallow foundation is used for soils with a lower load-bearing capacity. The raft distributes the vertical forces from the load-bearing elements over the entire footprint of the building. A raft foundation is often applied when small soft soil areas have to be bridged, or to prevent unequal settlements of the different load-bearing elements. A disadvantage of the raft foundation is that natural eccentricities in the subsoil can cause the structure to tilt. In European high-rise design, the maximum allowable tilt is determined at 1:800.

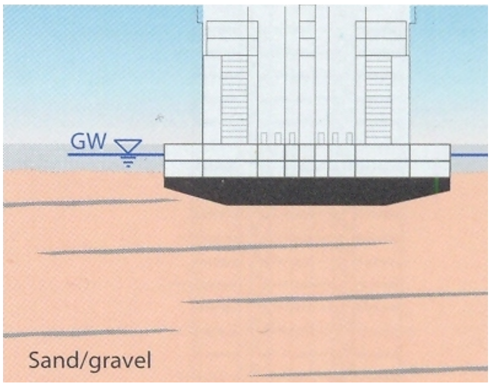



**Figure 3.2:** *A raft foundation*

*Third*, a floating or compensated raft foundation. In this case, so much soil is excavated that the weight of the soil removed plus any uplift from water pressure is replaced by the combined gross loading of the building (Schueller [41]). By doing this, the pressure in the subsoil remains the same, thus theoretically resulting in no settlements. However, in practise limited settlements do occur. This type of foundation has been applied in some high-rise buildings in Houston, US. Note that the uplift water pressure can fluctuate throughout a year due to changes in the groundwater level.

**Deep foundation:** Deep foundations are required when building on soils with a low load-bearing capacity. With this type of foundation the total load of the high-rise building is transferred to the load-bearing layer via the use of piles or caissons. The thin foundation slab underneath the building is not capable of carrying any loads. This task is solely fulfilled by the piles and caissons.

A wide variety of piles can be applied. The piles can be categorised into two groups, i.e.: large displacement piles and small displacement piles (Chew Yit Lin [9]). These categories express to which extend the soil is "pushed away" when the pile is driven into the ground. Due to this displacement of the soil, the piles exert pressure on the subsoil, resulting in higher friction stresses between the pile and the soil.
Caissons are basically nothing else then a small displacement pile with a large diameter. A caisson is a shell, box or casing constructed in the ground. Afterwards, the void is filled with concrete.
Although a deep foundation is costly, the settlements which occur are minimal. Most high-rise projects in Europe, Asia and in the United States are constructed with a deep foundation.

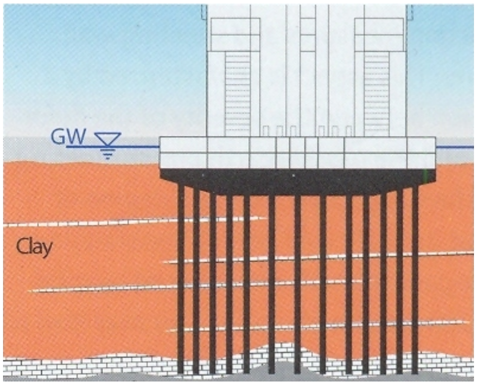



**Figure 3.3:** *A pile foundation*

**Pile and raft foundation:** Pile and raft foundations can be used when the subsoil has a limited load-bearing capacity. A Pile and Raft foundation consists of a shallow load-bearing raft, which is supported by short piles. The raft and the piles, both take part in accommodating the imposed loads. A Pile and Raft Foundation can be seen as a combination of a Shallow Foundation with a Deep Foundation.
The total settlements of the Pile and Raft Foundation can be controlled by changing the ratio of load-bearing capacity of the piles to the total load (the so-called pile-raft factor), letting the piles carry a larger part of the total load. In this way the settlements and tilt of a structure can be decreased. However, the costs of the foundation will increase.

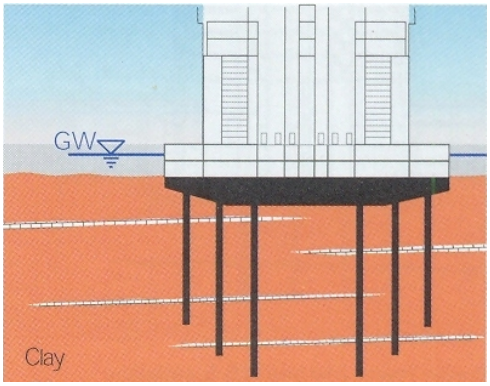



**Figure 3.4:** *A pile and raft foundation*

## 3.2 Load-bearing Structure

It goes without saying that the magnitude of the vertical forces is far greater in a high-rise building than in a low-rise building. The deadweight of the structure and the life loads are increasing approximately linear to the height of the structure (figure 3.5).

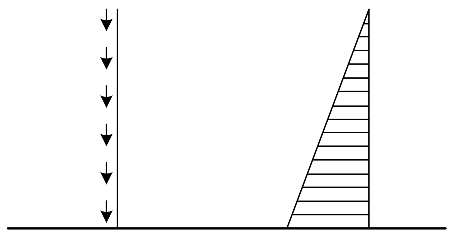



**Figure 3.5:** *Vertical loads acting on a skyscraper*

Due to the increasing height of the building, the horizontal forces acting on the building will increase as well. This is caused by the increased wind load acting on the skyscraper. The lateral load increases in a nonlinear fashion. We can schematise a high-rise building as a cantilever beam, fixed on one side into the ground (figure 3.6). A distributed load (the wind) acting on this beam will result in moments and shear forces at the base of the beam/building.

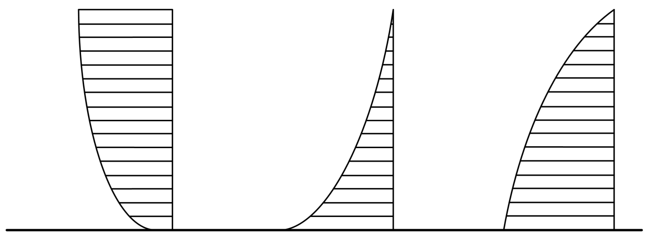



**Figure 3.6:** *Horizontal loads acting on a skyscraper*

Over the past years a lot a different structures have been applied in high-rise construction. This section will give an overview of the different applied structures and will categorise them into several groups.

### 3.2.1 Frame Structures

The first true skyscrapers in the world were build by using a frame structure. We can distinguish three different kind of frame structures: Rigid frame structures, Braced frame structures and Semirigid frame structures.

**Rigid frame structures:** A rigid frame structure is built-up out of portal frames. The horizontal loads cause large moments in the connections between the columns and the beams of the frame. In order to withstand these moments, the connections are moment-resisting and have sufficient stiffness to hold the angles between the members virtually unchanged under loading. Instead, the rigid frame structure responds to the lateral loading primarily through flexure of its beams and columns.

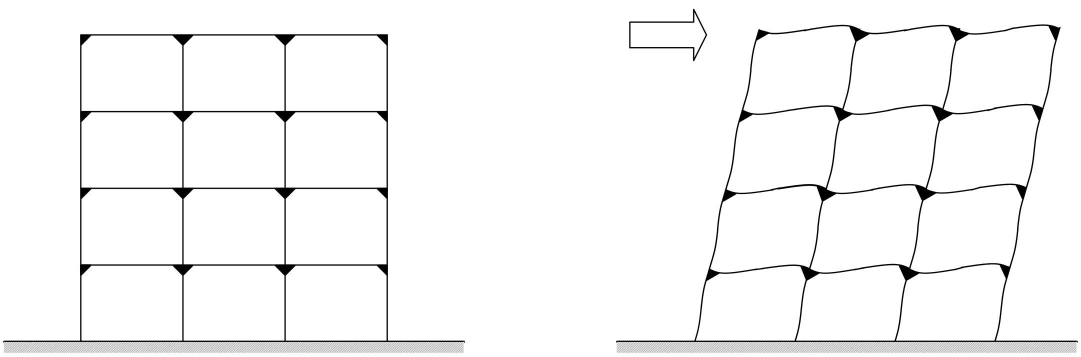


**Figure 3.7:** *Rigid frame structure*

The total lateral deflection of a rigid frame is caused by two components: the "Overturning moment" component and the "Shear racking" component.
*The overturning moment* in a building causes compression in the leeward columns of the frame and tension in the windward columns of the structure (see figure 3.9). As a result of these forces, the columns shorten on the leeward face and lengthen on the windward face, causing the building to rotate. This rotation causes the lateral deflection of the structure.
*The shear racking component* is the flexure which occurs in the beams and columns of the portal frames (see figure 3.10). The bending of the individual members results in a distortion of the entire frame, causing a lateral deflection of the structure.
Pure rigid frame systems are not efficient for ultra-tall buildings. As the height of the building increases, the moments in the members of the portal frames become substantial, causing them to flexure excessively. To solve this problem large columns and beams have to be applied, making this type of structure heavy and expensive to build.

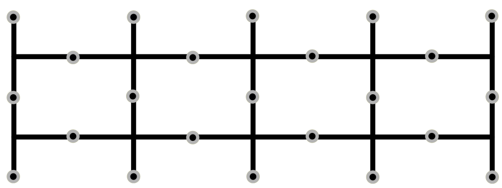


**Figure 3.8:** *An unloaded rigid frame structure*

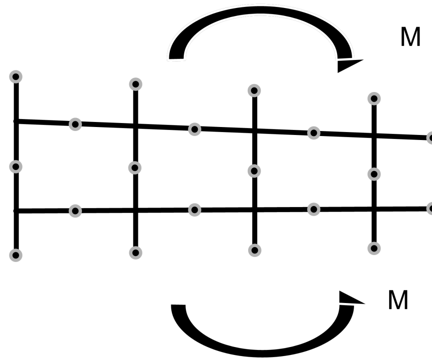



**Figure 3.9:** *The overturning moment component*

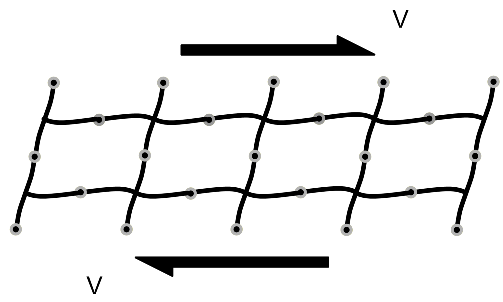



**Figure 3.10:** *The shear racking component*

**Braced frame structures:** To improve the behaviour of a frame structure, stiffening braces can be added, creating a braced frame structure. In this system the members of the portals are not any longer subjected to bending because the lateral forces are absorbed by the braces in the frame. Due to this, all members in a braced frame structure are only subjected to axial loads, creating an efficient structural system (Taranath [47]). It means that the system behaves much stiffer, resulting in less lateral deflection.

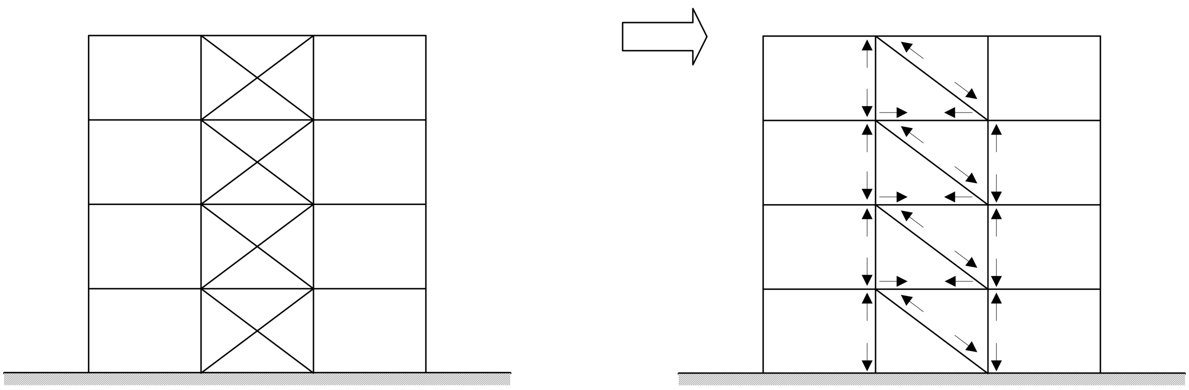


**Figure 3.11:** *Braced frame structure*

Like a rigid frame structure, a braced frame structure has to cope with moments and shear forces. The moments are taken by the columns of the structure, while the shear forces are taken by the structure's bracing.

The bracing can have different configurations. The most effective ones are the simple diagonal bracing, a double diagonal bracing (X-bracing) and a so-called K-bracing. However, the need for window and door openings sometimes requires a bracing of a different configuration. Examples of these are given in figure 3.12. However, these arrangements are generally less efficient, resulting in bending of the structural members.

When applying a braced frame structure, there is a risk that because of the elastic shortening of the columns (due to the high vertical loads), the tension diagonals will be slackened. In this way they can only take their assumed tensile force when the entire structure is highly deformed. To solve this problem, the tension diagonals inside a braced frame structure have to be prestressed. Due to this prestressing the diagonals will not slacken when the structure of the building shortens. This means that they can immediately accommodate a tensile force.

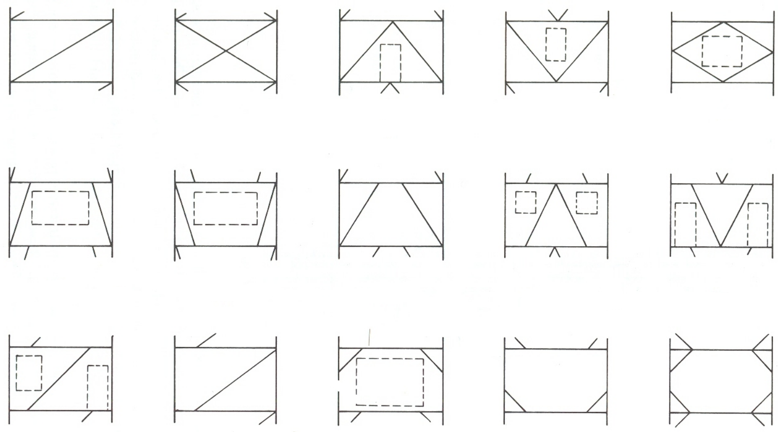


**Figure 3.12:** *Examples of bracing configurations*

**Semirigid frame structures:** This structure can be seen as an intermediate form of the Rigid frame structure and the Braced frame structure. Both the portal frames and the bracing in the structure absorb the lateral loads. This means that flexure of beams and columns will occur, however, this is substantially limited by the structure's bracing.
It has to be said that most braced frame structures can, to a certain degree, be regarded as a semirigid structure. It is not possible to construct a 100 percent braced frame. The connections between the columns and the beams of the frame will always posses a certain stiffness. Due to this stiffness the frame will partly behave as a portal frame, which means that some flexure in the structure's beams and columns will occur.

### 3.2.2 Shear Wall Structures

A Shear wall structure works in a similar way as a Braced frame structure. However, instead of a braced portal, a shear wall absorbs the lateral forces. Reinforced concrete walls are highly suitable for this. These shear walls are often combined with the service core inside the building, or they serve as a partition wall. The shear walls can be planar, but are often L-, T-, I- or U-shaped in order to increase their flexural stiffness.

When the shear walls are constructed out of concrete, a point of interest is that no, or only limited tensile forces may occur in the walls. This because the tensile strength of concrete is low and excessive reinforcement in the shear wall is undesirable. In order to solve this problem, it is usual to locate the shear walls in such a way that they attract a sufficient amount of gravity dead loading to counteract the tensile stresses in the wall caused by the lateral loads.

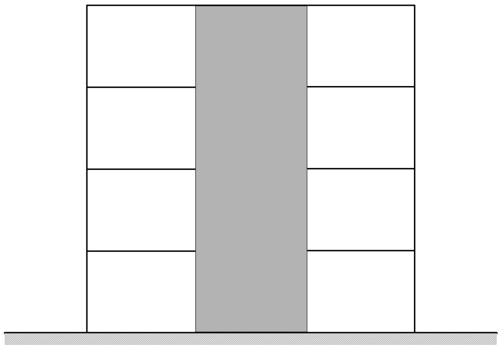


**Figure 3.13:** *Shear wall structure*

We can divide shear wall structures into two groups: Non-twisting structures and Twisting structures.

**Non-twisting structures:** If the resultant lateral force has no eccentricity, the building will not twist. In other words, if the axis of the resultant lateral force intersects the torsion centre of the structure, no twisting will occur. This is for example the case when a building is symmetric about the axis of the resultant force. To safeguard the buildings stability, only shear walls parallel to the direction of the lateral loads are needed.

**Twisting structures:** If the torsion centre of the building does not coincide with the axis of the resultant lateral force, the building will twist. This happens if the plan of the building is asymmetric. In addition to the parallel walls, shear walls perpendicular to the direction of the lateral loads are needed in order to safeguard the stability of the building. For twisting structures, a minimum of three shear walls is needed. It has to be noted that the axis of these walls may not intersect in one point.

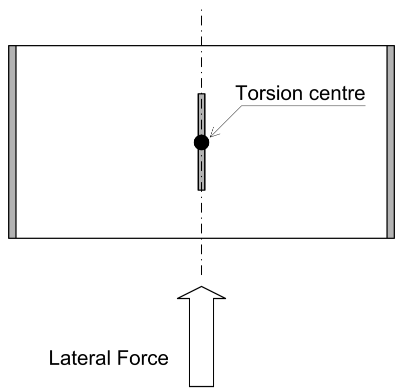



**Figure 3.14:** *Non-twisting structure*

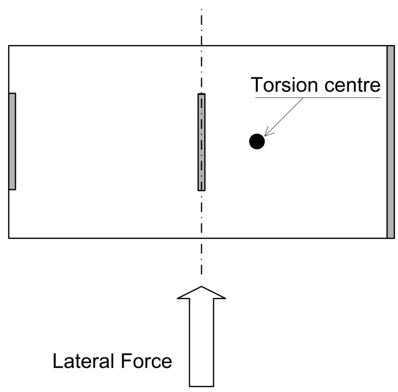



**Figure 3.15:** *Twisting structure*

When multiple shear walls are applied in the same plane, the walls can be coupled, creating a so-called "Coupled Shear wall structure". In a Coupled shear wall structure the walls are rigidly connected to each other, allowing them to interact. This interaction between the walls results in a much stiffer system than the enumerated stiffness of each separate wall. The connecting elements are subjected to both horizontal and vertical interactive forces. The coupling of the shear walls can be realised by means of the floor slabs or by means of special connecting beams.

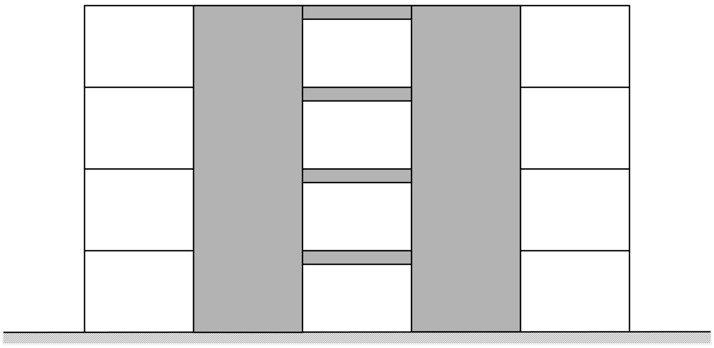


**Figure 3.16:** *Coupled shear wall structure*

### 3.2.3 Core Structures

Inside a skyscraper the lift shafts and stairwells are generally located in the centre of the building. To ensure that the lift shafts and stairwells remain operational during an emergency, they have to be surrounded by fireproof walls. Reinforced concrete walls are well suitable for this protection. Besides this protective function, these walls can also play an important role in the building's stability. The four walls surrounding the lift shafts and staircases form a box section, a so-called building core.
The moments of inertia of such a reinforced core are large, making it suitable to accommodate substantial lateral loads. The core's moments of inertia are reduced due to the presence of openings in the core. Openings are necessary for doorways (figure 3.17). This negative effect can be counteracted by partially closing the core again by means of connecting beams or by using the floor slabs of the building (figures 3.18 & 3.19).

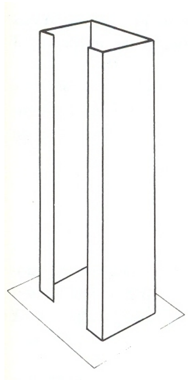


**Figure 3.17:** *Open core section*

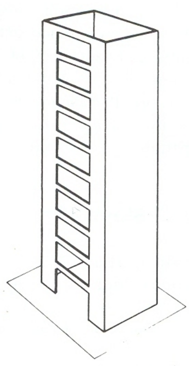


**Figure 3.18:** *Core partially closed by beams*

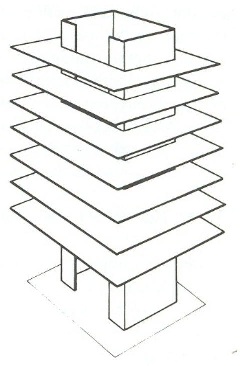


**Figure 3.19:** *Core partially closed by the floor slabs*

The behaviour of a core under loading is similar to that of a thin-walled beam. Besides bending, the twisting action of a core structure should be analysed (figure 3.20). This analysis should include both the warping and shear torsion in the core. Both are caused because the base section of the core is prevented from rotating by the building's foundation.

**Warping:** Warping means that an initially plane cross-section will not remain plane under loading. The effects due to warping include vertical stresses in the walls of the core and shear and bending in beams or slabs that connect across openings in the core section (Smith and Coull [45]).

**Shear torsion:** The twisting movement of the core will result in high shear forces in the bottom of the core.

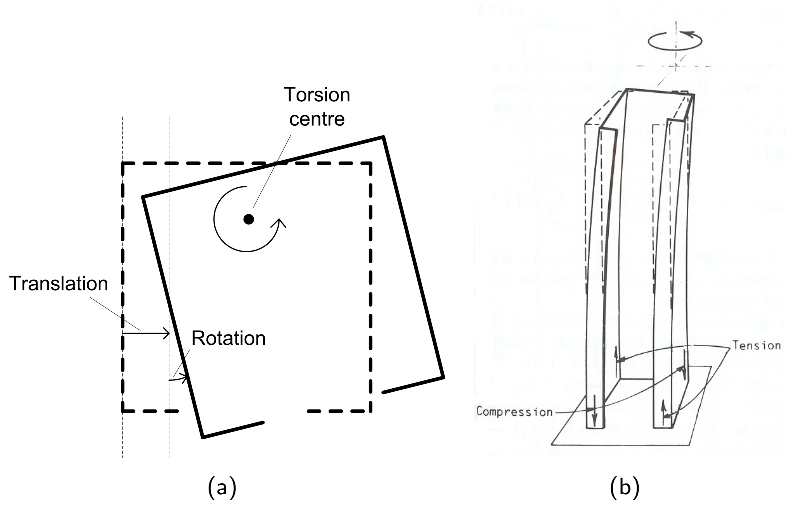



**Figure 3.20:** *The twisting action of the core*

### 3.2.4 Tube Structures

In the mid-1950s, engineers realised that the conventional way of building skyscrapers by using a frame structure had reached its peak with the construction of the Empire State Building in New York in 1931 (figure 2.7). A solution had to be found in order to further increase the height of a skyscraper. The solution they came up with, was the Tube structure. In the three decades that followed, this became the most applied structure.
In a Tube structure, the facade of the building is formed into a rigid body. This can be done by applying rigidly connected beams and closely-spaced columns around the perimeter of the building. This is called a "Framed-Tube Structure". The basic design philosophy behind this type of structure is to place as much of the load-bearing structure as possible on the perimeter of the building. This maximises the flexural rigidity of the cross-section. This can be explained by analogy with an ordinary thin-walled beam. The facades parallel to the wind act as the webs of the tube, while the facades perpendicular

to the wind direction will act as the flanges. Because the flanges are located as far from each other as possible, the large internal lever arm creates a very rigid structure when it is subjected to bending.

Although a Framed-Tube structure has a tube-like form, its behaviour is much more complex. In an unperforated tube, the overturning moment will result in a constant stress in the tube's flanges; a compressive stress on the leeward side and a tensile stress on the windward side. The webs of the tube are exposed to a linear stress pattern (see figure 3.21). Due to the presence of openings in the framed-tube, it becomes less rigid. This causes the columns in the middle areas of both the flange and the web planes to "evade" the stresses, which results to higher stresses in the corner columns of the building. This phenomenon is known as the "Shear-lag Effect" (figure 3.21).

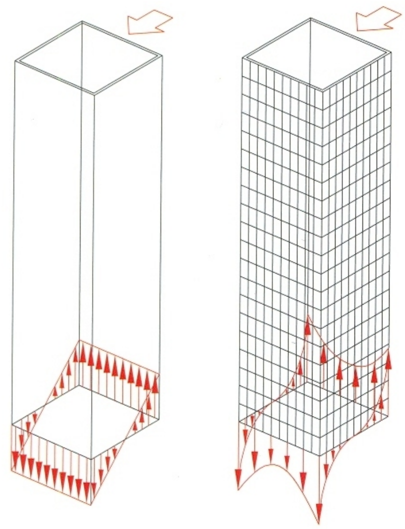


**Figure 3.21:** *The shear-lag effect in a Tube structure*

To counteract the Shear-lag effect, three alternative tube structures were developed:

**Braced-Tube Structures:** In these structures the frame structure in the facade of the building is stiffened by a bracing system (figure 3.22). Due to the bracing, the behaviour of both the flange planes and the web planes is much closer to that of unperforated plane. As a consequence of this, the Shear-lag effect is dramatically reduced.

**Bundled-Tube Structures:** A second solution is the implementation of internal web frames across the entire width of the building (figure 3.23). Due to the internal web planes, the stresses in the planes of the tube are more equally distributed which brings the behaviour of the Bundled-Tube Structure much closer to that of an unperforated tube. The most striking example of a Bundled-Tube structure is the Sears Tower in Chicago, United States (figure 2.9).

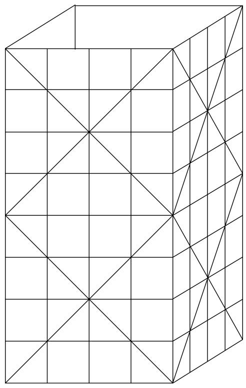


**Figure 3.22:** *Braced-tube structure*

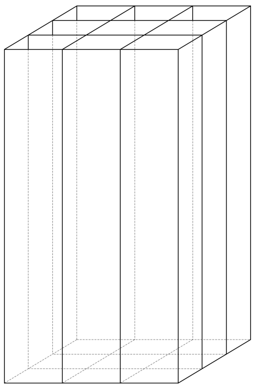


**Figure 3.23:** *Bundled tube structure*

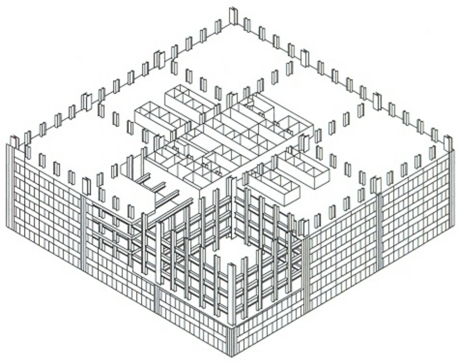


**Figure 3.24:** *Isometric view on the bundled tube structure inside the Sears Tower*

**Tube-in-Tube Structures:** The third option to reduce the Shear-lag effect, is to connect the outer tube with an additional tube inside the building. This system is known as a Tube-in-Tube system. The presence of the inner tube will reduce the vertical stresses in the outer tube, reducing the magnitude of the Shear-lag effect and its consequences. Coupling the outer tube to the inner tube can be realised by means of the floor slabs.

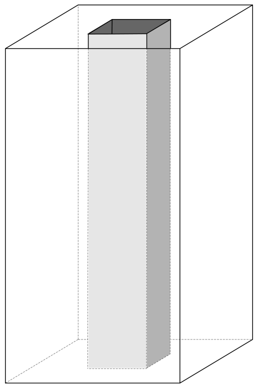


**Figure 3.25:** *Tube-in-tube structure*

### 3.2.5 Outrigger-Braced Structures

To limit the lateral deflection due to horizontal loading, a new type of structure is used in most modern skyscrapers: the so-called Outrigger-Braced structure. An outrigger-braced structure consists of a main core which is connected to super columns in the facade of the building by means of large horizontal cantilevers (Smith and Coull [45]) (figure 3.26). By applying outriggers, the internal lever arm of the structure is increased, which results in a much stiffer behaviour of the building when it is subjected to bending. The outriggers will impose a compression force on the leeward columns and a tensile force on the windward columns of the structure (see figure 3.27).

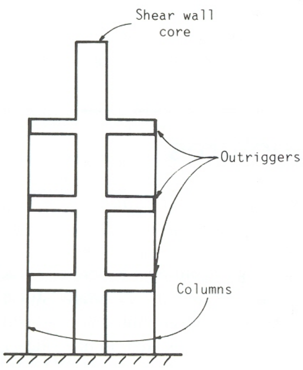



**Figure 3.26:** *Outrigger-braced structure*

The outriggers are generally constructed out of steel. The height of the outriggers can take up as much as two stories. Depending on the total height of the structure, multiple outriggers can be constructed. If one outrigger is applied, the optimum performance is achieved with the outrigger placed at two-third of the building's height. Smith and Coull [45] give a method to determine the optimum outrigger locations when multiple outriggers are applied. In order to obtain an optimum performance, the outriggers have to be placed at:

$$1/(n+1), 2/(n+1), 3/(n+1).....n/(n+1) \tag{3.1}$$

While the outrigger system is very effective in increasing the structure's flexural stiffness, it has to be noted that it does not increase the building's resistance to shear forces. These are still mainly accommodated by the core of the skyscraper.

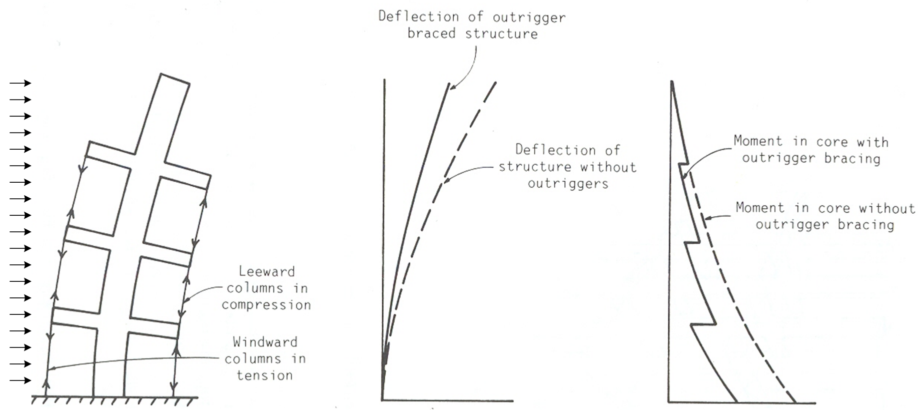



**Figure 3.27:** *Behaviour of an outrigger-braced structure under lateral loading*

### 3.2.6 Special High-Rise Structures

A special breed of high-rise structures, are the Special High-Rise structures. This term can be seen as a collective expression for all the building structures which can not be assigned to the structure types described above. Two types of special high-rise structures will be briefly discussed: Spatial Frame structures and Suspended structures.

**Spatial Frame Structures:** Spatial frame structures are using super-sized spatial frames as their load-bearing structure. A very good example of this, is the Bank of China in Hong Kong (figure 3.28). The giant spatial frame is able to cope with both the gravitational and the lateral forces acting on the building. A spacial frame structure is stable by itself and, in theory, does not need an inner core or shear walls to safeguard its stability. The floors of the building are directly connected to the giant frame structure.

**Suspended structures:** Yet an other example of a special high-rise structure can be found in Hong Kong: the 179 metres tall Hong Kong & Shanghai Bank. This building has a so-called suspended structure.
A suspended structure transfers the vertical loads upwards via tension members into horizontal trusses. These trusses transfer the load to one or more cores. The cores eventually transfer the loads onto the foundation of the building. Because the floors are suspended from a structure above, the vertical load-bearing members are only loaded with a tensile force which prevents them

from buckling. This allows vertical load-bearing members with a small cross-section. The shear forces due to lateral loading can only be resisted by the cores of the building.
An advantage of a suspended structure is that the conventional construction process can be reversed. In other words the building can be constructed from top to bottom instead of starting at the base of the building to build skywards. After the core(s) of the building have been completed, each floor can be completed on the ground before it is hoisted up. The benefits of this are that the floor construction can take place on the ground and that workers can start working on the facade of the building as soon as the floors are hoisted into place. No dangerous building activities are going on above their heads.

**Figure 3.28:** *The Bank of China in Hong Kong*

**Figure 3.29:** *The Hong Kong & Shanghai Bank in Hong Kong*

### 3.2.7 Load-bearing Floor Systems

The load-bearing floor systems which are applied in high-rise construction differ not much from systems used in the conventional building industry. In each project an evaluation has to made to determine which floor system is best suitable. Common used floor systems are:

**Flat-slab in reinforced concrete:** An advantage of this system is that it has a flat underside. Because of this, there are no barriers for pipes and cables suspended under the floor slabs. An additional suspended ceiling to mask the floor beams is not required.

**Floors with reinforced concrete beams:** This dramatically reduces the thickness of the floor slabs. However, the advantages of a flat-slab floor system do not apply any longer.

**Floors with reinforced concrete suspender beams:** In this system virtual beams are created in the floor slab by applying additional reinforcement. By doing this, shallower floor beams can be realised.

**Composite floor:** This floor system combines steel with cast-in-situ concrete. The concrete is applied in the compressive layer of the floor and the steel is used to accommodate the tensile forces. Metal studs secure the interaction between the two layers.

**Prestressed concrete slabs:** The slenderness and span of an ordinary flat-slab reinforced concrete floor can be increased by using prestressed concrete instead. The downside of applying this floor system is the more complex construction method.

**Flat-slab floors with displacement bodies:** This system is in principle the same as an ordinary flat-slab reinforced concrete floor. However, the dead-weight of the floor is reduced by applying displacement bodies in the concrete. An example of this floor system is BubbleDeck, a system which is currently emergent in Europe.

**Precast concrete elements:** These elements are prefabricated in a factory and already contain the necessary reinforcement. A structural concrete topping assures that the separate elements act together to accommodate the loads.

## 3.3 Slenderness

The European regulations with respect to the entry of daylight into a building are more strict than regulations applied in the United States and Asia. The required entry of daylight limits the depth of the floors of the building. European skyscrapers are generally 30–40 metres wide at their base while the base of skyscrapers in the US and Asia is around 50–60 metres wide. The limitations to the depth of the building mean that together with the height of the skyscraper also the slenderness of the building is increasing. Most of the currently built skyscrapers have a slenderness around 1:8/1:9.
A more slender building is more susceptible to vibrations and it is harder to safeguard the stability of the building. Therefore the slenderness of the building could seriously limit the building's height.

## 3.4 Comfort

As mentioned in the previous section, a skyscraper becomes more susceptible to oscillation when the slenderness of the building increases. Oscillations can occur when the building resonates under the action of external loads. External loads which can cause the resonance of a building are seismic ground accelerations, wind action and various man-induced excitations. The latter includes blasts, vehicular, rail and pedestrian traffic, machinery vibrations, etc. (Isyumov and Tschanz [19]). Apart from the seismic ground accelerations, displacements caused by these external loads are usually not large enough to inflict damage to the structure of the building. However, constant vibrations and movements of the building have a negative effect on the comfort felt by the building's occupants. Generally, the public does not expect buildings to vibrate or move. When it does occur, it is judged to be an indication of inferior quality of the structure, making the occupants to feel insecure. Like the rolling movements of a ship on open sea, the oscillations of the building can cause the occupants to develop dizziness or nausea. This is known as "building sickness".

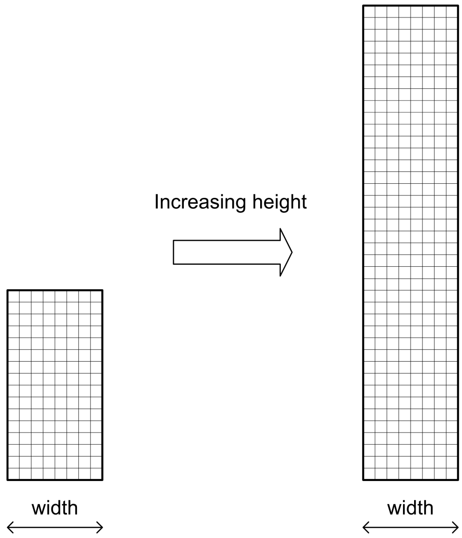



**Figure 3.30:** *The slenderness of the building is increasing with the height*

Wind-induced motions of tall buildings can persist for hours in a row and is therefore the major factor which endangers the comfort for the building's users. Therefore wind-induced oscillations will be considered in the remaining part of this section.

### 3.4.1 Wind Induced Vibrations

The magnitude of the wind-induced vibrations is dependent on several factors (Melbourne and Cheung [29]), i.e. the wind speed, the damping of the building, the building massiveness and the cross-wind force spectrum.
Generally the acceleration of the building is accepted as the criterion to evaluate the building's comfort. In his book Oosterhout [35] states that the perception of motion as a serviceability limit state is becoming increasingly important due to the following developments in the building industry:

- Skyscrapers tend to become more and more slender. This because the height of the structures increases, but the width of the base of the buildings remains practically unchanged.
- In the building industry more and more high strength materials are applied. As a consequence, structures can be lighter and more flexible, and still meet the strength requirements.
- Less damping is present due to the use of high strength materials.
- Fewer non-structural components which provide inherent stiffness and damping. Moreover these elements are often assembled in such a way that they are free to move.
- Non-structural materials are lighter and less rigid.
- Modern Building Codes have relatively low safety factors, allowing for lighter structures.

### 3.4.2 Human's Perception and Acceptance

Human's perception and acceptance of tall building oscillation depends on both psychological and physiological considerations. Both are highly subjective and therefore it is difficult to quantify them. In his book Oosterhout [35] quantifies several psychological and physiological factors which influence human's perception of motion in a building. As a result of extensive "moving room" experiments in the past, the physiological effect are quite clearly understood. However, the psychological effects are still not entirely clear and require further studies.

**Frequency or period of the building:** The perception thresholds tend to increase as the frequency decreases for low frequency harmonic solutions.

**Age:** People of a younger age are more sensitive to motion than older aged people.

**Body posture:** The sensitivity to motion is linked to the distance of a persons head to the floor. The larger this distance the more sensitive the body is to motion.

**Expectancy to motion:** Perception thresholds decrease if a person has prior knowledge that motion occurs.

**Body movement:** A moving person will start to perceive motions at a higher acceleration level than a standing person.

**Visual cues:** Visual signs, e.g. movements of hanging lights, can greatly influence a persons perception to motion. Another example is the apparent lateral swinging of the horizon near the corners of the building, which appears due to torsional motions.

**Acoustic cues:** Sounds caused by rubbing partition walls or the whistling of the wind can focus a persons attention on the lateral movement of the building.

**Type of motion:** Persons are more sensitive to bi-axial movements than to uni-axial movement.

The acceleration restrictions for the design of residential high-rise buildings are more severe than those for skyscrapers with a commercial function. This is because residential buildings are occupied for more hours and are therefore more likely to experience the design storm event. Additionally, studies demonstrate that people are less sensitive to motion when they are at work than when they are at home. Moreover, people are more tolerant to their work environment than to their home environment (Eisele and Kloft [11]).

### 3.4.3 Suppressing wind-induced vibrations

There are several ways in which the wind-induced oscillations of a skyscraper can be suppressed:

**Adding more structure:** A straightforward solution is to add more structure to the building. This to reduce its displacement or to increase the mass of the building, yielding a longer sway period for the same building stiffness (Tamboli [46]). Applying this solution can be very costly to achieve significant improvements in occupant comfort. Moreover, adding additional structure can cause a redistribution of the loads within the load-bearing system of the building. This is not necessarily desirable with regard to the capacity for which the load-bearing structure was designed (Eisele and Kloft [11]).

**Damping the structure:** A better approach is to increase the damping of the structure. This can be done by adding many sources of damping throughout the building or by concentrating the damping mechanism at one or two locations.
The use of many energy dissipating devices throughout a building to reduce the building's response to dynamic inputs, has become an accepted approach for high-rise buildings (McNamara [28]). Viscous dampers have proven to be the most effective. These dampers are applied in the structural frame of the building. To improve the effectiveness of the common "Diagonal viscous damper" a so-called "Toggle Brace Damper" system (TBD) can be applied. How this system functions is explained in the paper of McNamara [28]: "The main intent of the damper installation is to reduce accelerations. In order to do this the viscous dampers need to provide a large force output at very low displacement levels. In order to insure reliability at this small movement and to keep the number and cost of the dampers to a minimum, a motion amplification device is needed. A TBD system is a very effective mechanism to amplify inter-story motion."
In order to be effective the viscous dampers have to be applied to a large number of joints. This can make this solution very costly.

**Figure 3.31:** *A Diagonal Viscous Damper installed in a building*

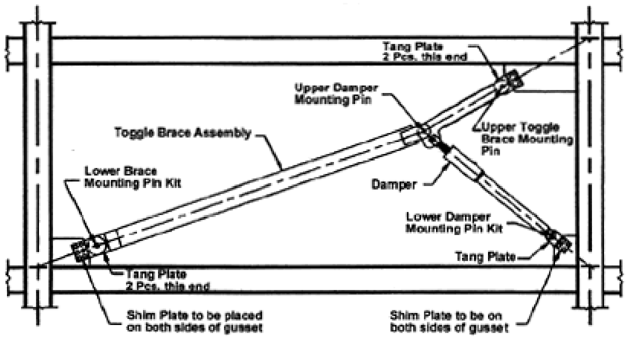



**Figure 3.32:** *A Toggle Brace Damper*

Concentrated damper systems are usually located near the top of the building. There are two suitable systems for concentrated damping: a "Tuned Mass Damper" (TMD) system or a "Tuned Liquid Column Damper" (TLCD) system.

A *TMD* is a mass block which is connected to the building by a suspension system, which allows the block to move "freely". Additionally the mass block is connected to the building by a set of dashpots or in other words, large "shock absorbers". In case of wind-induced oscillations, the mass block at the top of the building moves out of phase with the building's swaying. This will drive the dashpots, which will convert a part of the kinetic energy of the buildings motion into heat, reducing the oscillations. In order to be effective, the eigenfrequency of the damper should be close to the eigenfrequency of the building so both will start to oscillate simultaneously. The principle of a TMD is illustrated in figure 3.33. The main mass, M, represents the mass of the building. The spring, K, connecting the main mass to the ground represents the stiffness of the building. The damping of the building is represented by the viscous damper C. The secondary mass, m, is the mass of the TMD. The spring and damper which connect this mass to the main mass, represent the stiffness and damping of the suspension system by which the TMD is connected to the building.

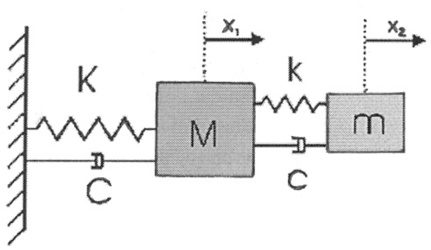



**Figure 3.33:** *Principle of a TMD*

When applying a TMD system, it is necessary to limit the motions of the TMD mass under very high wind loading to prevent the mass from damaging the structure of the building. For this reason nonlinear hydraulic dampers and hydraulic buffers are installed around the mass block (Irwin and Breukelman [18]). There are several types of TMD's on the market: a Simple Pendulum Damper Application, a Nested Pendulum Damper Application or a Hybrid Mass Damper (HMD) system. The simple pendulum damper application (figure 3.34) and the nested pendulum damper application function both in a similar way. The only difference between the two is the configuration of the suspension system. The height of a nested damper is much lower, making it possible to apply this TMD in buildings which have insufficient height available to fit a simple pendulum TMD. The downside of this is that the design of a nested TMD is much more complex. A HMD system consists of a Active Mass Damper (AMD) installed on a TMD (see figure 3.35). The AMD is a block mass which is guided along a ball screw shaft and propelled by a AC servomotor. The AMD makes it possible to control the vibration of the TMD, improving its effectiveness under strong winds or small-to-medium earthquakes (Okuda et al. [33]).
The disadvantages of using a TMD are that it requires a lot of space for mounting and operating it and that additional structure for supporting the extra weight is needed. Commonly a TMD weighs about 1% of the total building weight (Irwin and Breukelman [18]).

*Tuned Liquid Column Dampers* are similar to TMD's. The difference is that the mass is now water or some other kind of liquid. The system consists of a U-shaped tank, i.e two vertical columns and a horizontal passage connecting them (Irwin and Breukelman [18]). The horizontal passage is fitted with screens and sluice gates. The columns of water have their own natural period of oscillation which depends on the geometry of the tank. Again the eigenfrequency of the system has to be close to the eigenfrequency of the building, making it possible that the building's kinetic energy is transferred to the water in the tank. When the water passes the screens and sluices in the horizontal passage, the kinetic energy is dissipated, reducing the lateral movements of the building.

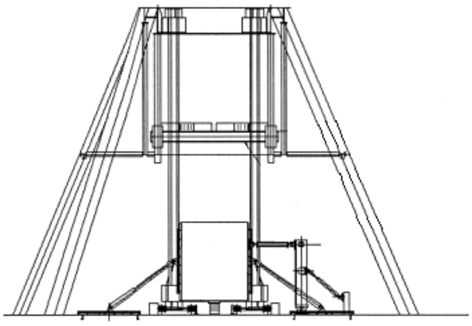


**Figure 3.34:** *Simple pendulum damper*

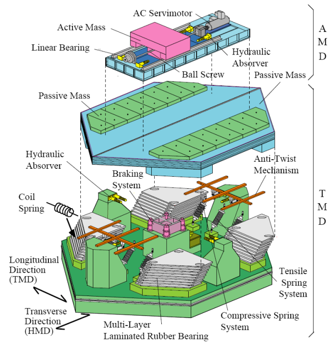



**Figure 3.35:** *Hybrid mass damper (HMD)*

A big advantage of a TLCD is that it can be combined with a water supply used for fire suppression. Often, water tanks for fire extinguishing water are already incorporated in the design of the building and the building structure is already capable to accommodate these high loads.

**Changing the shape of the building:** The final solution to reduce the building's wind-induced oscillations is by giving the building a more aerodynamic shape. This is a promising solution, but currently there is no extensive knowledge in this field. Although, it is known that cut or round corners will reduce the wind forces on the structure (Melbourne and Cheung [29]).

## 3.5 Earthquakes

The outermost layer of the earth consists of a 150 kilometres thick crust, the lithosphere. This is divided in seven large and a few smaller tectonic plates (figure 3.36). Because these plates are constantly moving in respect to each other, high tensions are built up between the plates. If the stress capacity is surpassed locally, a sudden movement occurs. This is experienced by humans as an earthquake. The lateral movements caused by the earthquake can seriously damage structures and can, in some cases, even cause buildings to collapse. Large cities like Tokyo, Auckland, Mexico City, Manila, Santiago, Los Angeles, San Francisco, Athens, Istanbul, Tehran and Vancouver are all located near the borders of the tectonic plates. Large earthquakes have already occurred in these cities or are expected to do so in the near future. This highlights the fact that the threat of earthquakes to high-rise buildings is relevant.

### 3.5.1 Earthquake waves

An Earthquake emits several types of waves. These waves can be divided into *Space Waves* and *Surface Waves* (Eisele and Kloft [11]). Space Waves travel faster than Surface Waves and will therefore be experienced earlier.

**Space waves:** There are two types of space waves, P-waves and S-waves:

*P-waves* (figure 3.37a) are the fastest travelling space waves. They can propagate in solid rock, but in liquid materials as well. The motion of these waves is comparable with that of a sound wave.

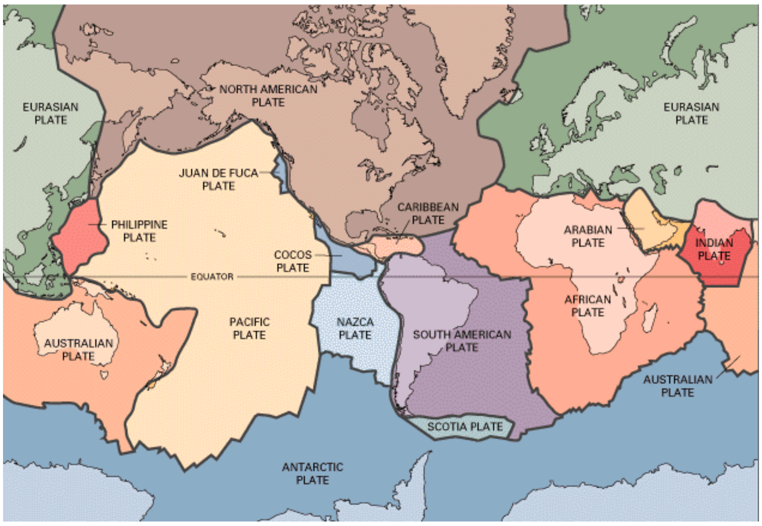



**Figure 3.36:** *Tectonic plates in the lithosphere*

As it spreads, it alternately compresses and dilates the subsoil. The effect of P-waves is similar to that of a loud sonic boom, making the windows of the building to tremble. The structure of the building generally experiences little damage as a consequence of these waves. This is because the waves only cause tremors in the vertical direction of the building. In this direction the structure is designed to resist high loads. The second type of wave to reach the earth's surface are the *S-waves* (figure 3.37b). These waves can only propagate in solid rock. The motion of these waves is a shear movement perpendicular to the direction of travel. The waves introduce horizontal tremors into a structure, generally causing severe damage to the building because the horizontal forces exceed the design loads.

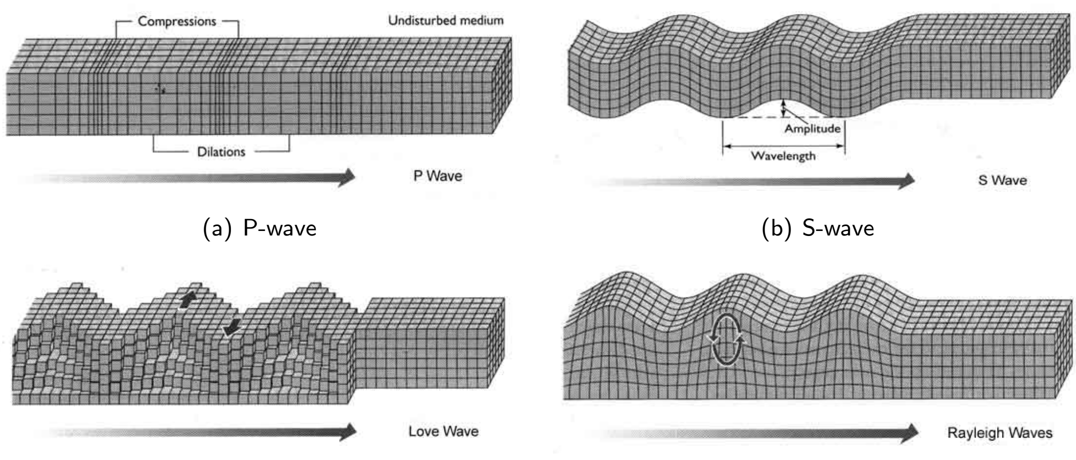
**Surface waves:** Surface waves can be divided into Love waves and Rayleigh waves. These waves are called surface waves because the motion of the waves is restricted close to the ground surface. As mentioned before, surface waves travel at a lower speed than the space waves.

The first surface waves that will be experienced are the *Love waves*. The motion of Love waves (figure 3.37c) is comparable with that of S-waves, moving the ground from side to side in the horizontal plane. This movement is particulary damaging to the foundation of structures.

The Rayleigh waves (figure 3.37d) are experienced last and can be compared to the movements of a rolling ship, causing both vertical and horizontal movements. Because structures are already severely damaged by the previous waves, these waves often cause the collapse of the building.

### 3.5.2 Building's Performance During an Earthquake

Although earthquakes may not even occur during the lifetime of a structure, structures should be designed to withstand them in areas where earthquakes are likely to occur. There are four major factors which can influence the performance of a structure during an earthquake (Hutchinson et al. [16]):




(a) P-wave (b) S-wave

(c) Love wave (d) Rayleigh wave

**Figure 3.37:** *Earthquake waves*

**Structure configuration:** Earlier experiences have shown that high-rise buildings with an irregular configuration in the floor plan or in elevation, suffer greater damage than buildings with a regular configuration. Regularity in stiffness, strength and mass is desirable.

**Material characteristics:** For a good earthquake resistance materials with the following material characteristics are favourable: high ductility, high strength-to-weight ratio, homogeneity and ease in making full strength connections. We can conclude that steel and reinforced concrete possess these characteristics, provided that the structure is well detailed. Prestressed and precast concrete have a limited performance due to the limited strength of the connections.

**Structural framing system:** e.g.: during an Earthquake a "Building Frame System" performs better than a "Bearing Wall System" because the frame systems are considered to have a greater energy absorbing capacity. With "Moment Resistant Frame Systems" the capability to absorb energy is highly dependent on the detailing of the members and connections.

**Non-structural components:** Non-structural components, e.g. partition walls and facade walls, can damp the swaying movements of a building induced by an earthquake.

### 3.5.3 Additional Hazards

Earlier earthquakes proved that not only the seismic shaking of the ground caused problems. Also other hazards, associated with earthquakes, can cause the collapse of structures (Lew and Naeim [25]).

**Surface fault rupture:** When an earthquake fault ruptures, it can sometimes extend all the way up to the ground surface. This results in large movements of the ground and permanent deformation across the fault plane. These large displacements can not be accommodated by most structures.

**Liquefaction:** Liquefaction can occur in loosely packed, saturated soils. Due to the shaking, the soil is condensed resulting in smaller pores in the soil (figure 3.38). The groundwater in these pores becomes over-pressurised, resulting in the liquefaction of the soil (Verruijt [50]). The load-bearing capacity of the soil is lost, causing the collapse of the structure. When the structures are supported by a deep foundation which extends to deeper non-liquefiable ground layers, the effects will be minimal or non-existent.

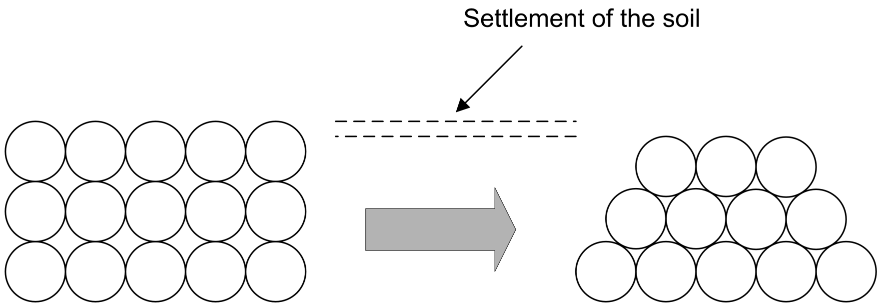



**Figure 3.38:** *Liquefaction*

**Landsliding:** Landslides can occur during an earthquake because the heavy ground shaking can cause instability in slopes that are only marginally stable under stable conditions. The sliding soil can destroy entire structures.

**Tsunami:** When the epicentre of an earthquake lies on the bottom of an ocean or sea, the kinetic energy of the earthquake is transferred to the water column above the epicentre, resulting in an upward motion of the water column. This is visible as a small ripple at the water surface. Due to the gravitational forces this upward motion is converted to a horizontal motion of the water. Together with new vibrations in the seabed, this results in multiple small waves which travel away from the epicentre at a high velocity. These waves have the same circular form and behaviour as the waves which occur when a rock is thrown into a pond. A stronger earthquake transfers more kinetic energy to the water, which results in waves travelling at higher speeds.
When the tsunami approaches the coast, the so-called "ground-effect" occurs: the front of the wave is slowed down by the upwards sloping seabed. The back of the wave still travels at maximum speed, resulting in compression of the wave. The water in this compressed wave can only travel upwards, creating a higher wave. The more the tsunami approaches the shore, the higher the resistance at the front of the wave, the more the height of the wave increases. Water in between the tsunami and the shore is sucked into the approaching wave, increasing the height of the wave even further.
When the wave finally approaches the coast, a wall of water flushes over the land. Enormous forces are unleashed upon structures in the coastal area. Even if a structure survives this first impact, it has to withstand the suction of the withdrawing water which flows back into the ocean after the wave has struck the shore. Very often these first waves are followed by new waves.
With major cities lying along the ocean's shores, tsunamis are a threat to reckon with.

**Fire:** The structure of a building may be designed to withstand an earthquake, the question is if the electrical equipment and the electrical cabling inside the building is also capable to withstand vigourous shaking. An earthquake could cause a short circuit in these electrical systems, which could lead to the outbreak of a fire. A fire can still cause the collapse of the building.

## 3.6 Influence on the Surroundings

When building a skyscraper, the building has an influence on its surroundings. These influences can be problematic and have to be identified in the early stage of the design process. As at this stage most of these problems, if not all, can be prevented by adapting the design of the building. In his master's thesis, Leeuwen [24] examines the effects of a high-rise building on the surroundings.
Leeuwen [24] divides the effects into four categories: i.e. harmonisation with the surroundings, building physics, accessibility and safety.

### 3.6.1 Harmonisation with the Surroundings

The newly constructed skyscraper should be in harmony with its surroundings and the public should not regard the building as a nuisance. In case of a tall skyscraper, the area in which the people's perception has to be taken into account, increases. This simply because the building is visible from greater distances. Considering this category, Leeuwen [24] identifies a number of aspects:

**Perception at street level:** People should consider their stay in the streets surrounding the building as pleasing. With regard to this aspect, the configuration of the building, the design of the public areas around the building, the architecture of the building and the perception of safety in the surroundings of the building are important features. An uncomfortable perception of the neighbourhood can cause vacancies in the surrounding office and residential buildings.

**View:** A high-rise building can seriously influence the view from neighbouring properties. The view is especially important for residential buildings, as it is part of a comfortable living. Moreover it influences the monetary value of the properties.

**Privacy:** The occupants in adjacent buildings will feel uncomfortable if the occupants in the newly constructed skyscraper have a view inside their building. To prevent this, the distance in between the buildings should be sufficient.

**Skyline:** A tall skyscraper can dominate the skyline of a city. It should be decided whether this is desirable or not. An example of this is the project "Overhoeks" in Amsterdam in The Netherlands. Five high-rise buildings are planned for construction. There are several action committees which protest against the plans. They consider it as unacceptable that the modern buildings will be visible from the old, monumental city centre.

**Town planning:** The building has to blend in with the surrounding buildings.

**Flight paths:** When there is an airport in close proximity of the skyscraper. The building can interfere with the flight paths of the airplanes landing and taking off from the airport. This often leads to height restrictions for buildings near airports.

**Telecommunication:** Skyscrapers can cause interference in telecommunication signals.

### 3.6.2 Building Physics

Building physics are not only important inside the building. The skyscraper also influences the building physics in its surroundings. Leeuwen [24] discusses six different aspects:

**Sunlight/shade:** A tall skyscraper can cast a shade on adjacent properties, influencing the entry of daylight into these buildings and thus influencing the comfortable living.

**Reflections:** The facade of the building can cause inconvenient and in some cases even dangerous reflections of sunlight.

**Noise:** During the construction and the utilisation phase, a large building like a skyscraper will attract a lot of activity into the area. The increased activity in an area can be a nuisance because it is often accompanied with an increased noise production. The sound insulation of buildings in the area is insufficient against these increased noise levels. In addition to this, a new building can reflect noises from the street, resulting in higher noise levels for people in neighbouring buildings.

**Air quality:** Due to the increased activity in the area, an increased flow of traffic is to be expected. As a consequence of this, more exhaust gasses are emitted in the area, lowering the air quality. Additionally, the building may influence the air flow in the area. In areas where the wind velocity is low, the polluted air can build up to unacceptable levels.

**Ventilation:** As said above, the new building can change the air flows in the area. This can seriously jeopardise the functioning of the natural ventilation systems in the neighbouring buildings.

**Wind:** Due to the changed air flows, wind speeds can locally reach high levels. High wind velocities on street level are unpleasant for pedestrians. Wind speeds at ground level should be minimised. However this clashes with earlier discussed aspects as the air quality and the ventilation. Also the occurrence of wind gusts has to be prevented to assure the pedestrian's comfort.

### 3.6.3 Accessibility

The construction of a skyscraper will attract a lot of new activity into an area. During both the construction and the utilisation phase of the building, a lot of traffic and people will travel towards and from the building. These increased traffic flows reduce the accessibility of the area. Leeuwen [24] expounds this category into the following aspects:

**Accessibility by car:** The area in which the new building is located has to be accessible by car. When a city district is not easy accessible as a consequence of the construction of a new building, firms which are housed in this area can suffer substantial economic losses. For residents in the district, poor accessibility can be a major nuisance.

**Accessibility by public transport:** The area in which the new building is located has to be accessible by public transport. This for the very same reasons as mentioned above for the accessibility by car. If necessary, the existing public transport has to be upgraded to match the increased demand.

**Parking facilities:** The demand for parking facilities in the area will increase because of the increased traffic flow into the city district.

**Accessibility emergency services:** The completion of the building should not jeopardise the area's accessibility for emergency services.

**Capacity utilities:** The existing utility networks, i.e. gas, water, electricity and sewage systems, have to have a sufficient overcapacity to supply the additional demand created by the new building. If necessary, the existing capacity has to be upgraded.

**Capacity municipal services:** The municipal services should have sufficient overcapacity so they are still able to service the city district adequately.

### 3.6.4 Safety

The last category examined by Leeuwen [24], is the safety around the newly constructed building. The new skyscraper can jeopardise the safety of the people in its surroundings in several ways:

**Increased wind loads:** As mentioned earlier, a new building can change the air flow in the area. If these changes mean an increase in the wind velocity, the wind loads on the surrounding structures will increase. These structures are not designed for these increased loads.

**Settlement of the subsoil:** Because a skyscraper is transferring high forces to the subsoil underneath the building, settlements may occur. Even if the building is constructed on a deep foundation, settlements can occur. These settlements can cause damage to the neighbouring infrastructure and buildings.

**Risks in the realisation phase:** Building activities in a densely populated area can cause substantial safety risks to those in close proximity of the building site, e.g. falling building materials.

**Risks in the operation phase:** It should be considered whether the skyscraper imposes safety risks upon its surroundings during the operating phase. An example of this is the tumbling-down of facade elements. The design of a high-rise structure is based on the "global structural load" which is generally given by the building codes. This load introduces high forces into the structure of the building and causes the structure to deform. The dimensions of the structural elements of the building are deduced from these loads and deformations. Depending on the shape of the building, local wind turbulence may occur. This turbulence can locally cause loads which exceed the global structural load. This local turbulence load can reach up to 5 $kN/m^2$. If the fastenings of facade elements are not able to withstand these increased forces, the elements will come loose. Moreover, the turbulence causes pressure fluctuations on the facade elements. The design load for the fastening systems must take these fluctuations into account to prevent damage due to fatigue. These kind of turbulences especially occur at the corners of the building and at the edges of its roof.

**Risks in the demolition phase:** Is has to be taken into account that the service life of a building is not infinite. Therefore the demolition phase should already be considered in the design phase of the building. Demolishing a building in a densely populated area may involve safety risks for the surroundings, e.g. falling debris.

## 3.7 Organising the Building Site

Because most of the high-rise projects are realised in already developed, densely populated city centres, there is not much space to set up a building site (figure 3.39). However, site offices, facilities, storage depots, workshops, etc. all have to be accommodated on site. Therefore organising the building site becomes a top-priority.

In order to prevent future problems a good planning has to be set up right from the start of the project. This plan records where the different functions have to be allocated in each phase of the building process. The plan should also schedule the on-site deliveries of building materials in order to prevent large stocks of materials on site which consume a lot of space. To solve this specific problem an additional storage can be set up just outside the city, where the limited amount of space is less of an issue. The building materials are only transported to the building site when directly needed, so-called just-in-time delivery. This solution was for example used when erecting the Empire State Building in New York City.

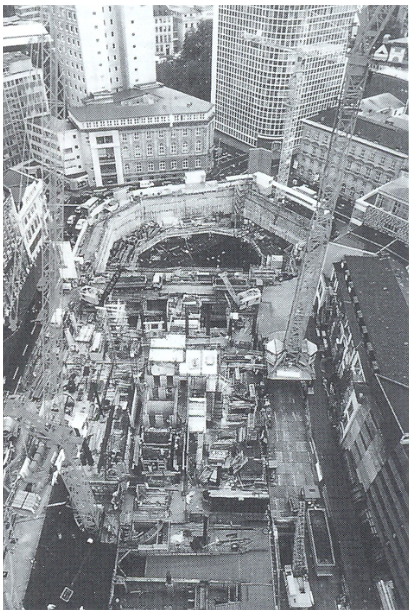


**Figure 3.39:** *Site organisation for the Main Tower in Frankfurt, Germany*

## 3.8 Vertical Transportation

### 3.8.1 Vertical Transportation in the Realisation Phase

When constructing a skyscraper the focus lies most in vertical processes. Due to the limited amount of space on the building site, it is very likely that the bottlenecks will occur in the vertical transportation during construction (Eisele and Kloft [11]). The vertical transportation consists of the transportation of both materials and workers. The machinery commonly used for vertical transportation are: cranes, site lifts and concrete pumps.

**Cranes:** In general, self-erecting cranes are used for the vertical transportation of building materials on site. The use of cranes on the building site introduces a range of logistical problems. One of these problems is neighbour-related concern. This concern involves two different cases. In the first case problems occur because of the simultaneous use of multiple cranes. While operating, the cranes may get in each others way, which will limit their effectiveness. The same problem can occur with neighbouring properties. The owners of these properties have to consent to the sweeping movements of the cranes above their property. In a built environment the presence of other high-rise can also seriously hinder crane operations on the building site.
To improve the effectiveness of cranes, an intensive planning has to be made in the preparation phase of the building project. In this planning important parameters like the reach of the cranes, maximum dimensions and weight of the transported goods and operating speed of the cranes have to be taken into account. This should result in a low degree of downtime of the cranes.

**Site lifts:** Site lifts are used for both transporting loads and persons. The vertical reach of the lifts is expended as the progress of the building structure, resulting in a longer travelling time for its users.
Relevant parameters are the load-bearing capacity of the lifts and their hoisting speed. This speed can reach up to 1.5 m/s.

**Concrete pumps:** Concrete pumps are deployed for transporting a continuous flow of concrete. Transportation in both the horizontal and vertical direction is possible. The pumps are capable of transporting concrete to heights of approximately 300 metres. When it is required that the concrete is transported over greater distances, intermediate pumping stations are required. No thorough research has yet been conducted to verify whether the pumping of concrete under high pressure has an influence on the quality and strength of the concrete. This problem is addressed in the paper of Seo et al. [42].

### 3.8.2 Vertical Transportation in the Operating Phase

In the operating phase, like in the realisation phase, most of the transportation in high-rise buildings is concentrated vertically. Not only people and goods have to be transported, but also the various building services require vertical transportation.

The invention of the lift and in particular Otis' lift safety device introduced a boom in the high-rise construction. Today the use of lifts for vertically transporting people and goods is indispensable. Three

different kind of lifts can be distinguished: passenger lifts, freight lifts and firefighter lifts. Passenger lifts and freight lifts are used for the transportation of respectively people or freight. The primary objective of the firefighter lifts is to transport firemen in case of an emergency. In Europe a clear distinction is made between ordinary lifts and firefighter lifts. In the US, all lifts are used as firefighter lifts. Firefighter lifts have to have a separate electricity supply and should be made entirely out of non-combustible materials. Additionally the lifts have to have separate lift shaft to prevent the spread of fire.
Lifts are usually incorporated in the very same (concrete) core which safeguards the overall stability of the building and accommodates the shear forces acting on the skyscraper. When a building gets taller the lifts form the bottleneck in the vertical transport. Solutions have to be found in order to keep the time, lost by travelling, acceptable.

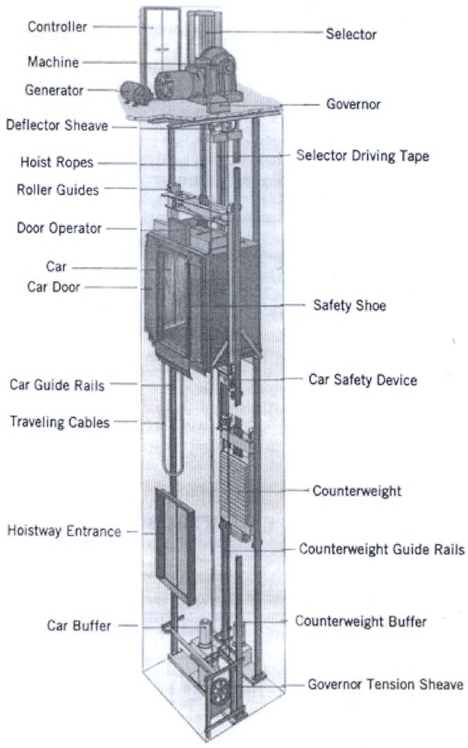



**Figure 3.40:** *The lay out of a passenger lift*

When planning and designing a lift system for a building, there are several parameters which are important for the performance of the system (Eisele and Kloft [11] and Tervonen et al. [48]):

**Number of floors:** The number of floors in the building.

**Usable Floor Space:** The usable floor space is an important parameter to determine the total lift capacity which is required. For office buildings this parameter is expressed in the amount of nett $m^2$ of floor surface used for office space. In hotels and residential buildings this is expressed in respectively the number of rooms and the number of apartments in the building.

**Number of Passengers:** The number of passengers is based on the population of each floor. This is strongly related to the usable floor space on each of the floors.

**Required Handling Capacity:** The required capacity of the lift system is expressed in the up-peak handling capacity. This figure is the percentage of the population per 5 minutes that can be transported from the lobby to the upper floors. The usual recommendation states that the up-peak handling capacity for a building should be 15%.

**Average Waiting Time or Average Interval Time:** Time elapsed before another lift arrives.

**Cab Capacity:** This is the number of passengers which can be carried by one lift cab. It is assumed that lifts are filled to 80% of the rated load. Although it is possible to fill the lift up to the rated load, passengers are often not willing to squeeze in an already crowded lift cab.
In case of a freight lift the maximum load which can be carried by the cab is relevant.

**Time Lost at Stop:** This is the time difference between a trip from A to B without intermediate stops to a trip from A to B with one intermediate stop. This time is influenced by the acceleration of the lift cab and the speed in which the doors of the lift open and close.

**Speed:** The speed in which a lift travels.

When all these parameters have been optimised, the floor area which is occupied by the lift system can be determined. The total floor area needed for a lift system consists of the shaft space plus the waiting area for passengers.

Lifts travelling at high speeds are only effective if they travel over longer distances so the maximum speed can indeed be reached. However it is not the maximum speed itself that forms the limiting factor in lift-use, these limits are set by the accelerations and decelerations of the lift cab. Accelerations and decelerations are experienced as uncomfortable by the passengers inside the lift car.
The performance of lift installations has received increased attention throughout the last decades, and passengers expect a better lift service and ride comfort (Li et al. [26]). The experienced ride comfort depends on three factors: the experienced physical movements, jerk and noise.

*Physical movements* are experienced as lateral and vertical movements. Lateral movements are the horizontal accelerations and vibrations of the lift cab. These are caused by the following factors (Li et al. [26]):

**Guide Rail Alignment:** A proper alignment of the guide rail will reduce lateral vibrations of the lift cab dramatically.

**Guide Roller Configuration:** For better comfort, rather guide rollers than guide shoes should be used, considering that these rollers have a perfectly round shape.

**Static Balance of Car Frame:** When the compensating ropes or travelling cables are mounted off-centre, the unbalance will create lateral vibrations of the lift car during operation.

**Lift Well Configuration and Air Displacements:** When a lift moves in the lift shaft, it will create a wind velocity which exerts a pressure on the sides of the car. When the car passes for example a floor, these pressures are changed abruptly, causing vibrations in the lift cab.

**Car Speed effects** The previous discussed factors are amplified when the car speed increases.

The second comfort criterium is the experienced *jerk*. Jerk is the rate of change of the vertical acceleration and is the motion felt by passengers.

The final comfort criterium is the *noise* which is produced by a lift. Noises produced by a lift system can influence the passenger's confidence in the safety of the system. An example of this is the earlier discussed air displacement in the lift well during lift operation. The abrupt pressure changes can cause the doors of both the lift cab and the landing doors on the floors to vibrate. This produces an uncomfortable noise.
Although lift comfort is a very important aspect, at the moment of writing, there are no international standards established to clearly specify the acceptance standard applicable to lift ride quality.

Besides choosing the right type of lift for the job, also the way in which the lifts are deployed can influence their effectiveness to a great extend.
Lifts can be operated in two different ways. In the first system, the conventional controls, a passenger chooses in which direction he wants to travel and takes the next lift travelling in this direction. The passenger only indicates inside the lift at which floor he needs to stop (Eisele and Kloft [11]). The second type of lift control is the "Group Dispatching system". In this system the passenger already selects the floor while outside the lift. Next, a computer system determines which lift cab the passenger has to take. In this way people with the same destination can be grouped together in one lift car. Because of this, the number of intermediate stops can be minimised together with the travelling time. The downside of this system is that the waiting times can be substantial because the passengers can not simply take the next lift in the desired direction. Group Dispatching systems are relatively new on the market, and are not yet fully perfected. Improvements can be expected in the near future.

Another way of influencing the effectiveness of lifts, is to change the way in which the lifts are configured. There are three different lift configurations possible (see figure 3.41):

**One group for all floors:** This is the conventional way of configuring lifts and is still applied in lower high-rise buildings (up to approximately 20 floors). Each lift is serving every floor.

**Lift groups from main lobby:** If a building gets taller it pays off to divide the building in groups. Each group is serviced by its own lifts which are all leaving from the main lobby of the building. This division reduces the amount of intermediate stops, reducing the travelling time and waiting time of the passengers. One drawback of this system is that together with the height of the building the required space for the lift shafts is increasing excessively.

**Stacked lift groups and skylobbies:** This configuration is mainly applied in modern high-rise projects above 45 floors. Skylobby's are directly connected to the main lobby on ground level by means of several express lifts. From each of the skylobbies, lift groups service the floors. This configuration saves space because the different lift groups can be stacked on top of each other. A drawback is that passengers have to transfer at the skylobbies in order reach their destination.

A double-deck lift is sometimes wrongly suggested as a lift configuration which improves the effectiveness of a lift system. The double deck only means that more passengers can be transported in one lift car.

The handling between floors is not improved. A stop may be required for the passengers on one deck, while it is not required for the passengers on the other lift deck. The large capacity of the double-deck lifts are only advantageous when all the passengers have the same destination. Therefore double-deck lifts are very suitable for connecting a skylobby to the main lobby.

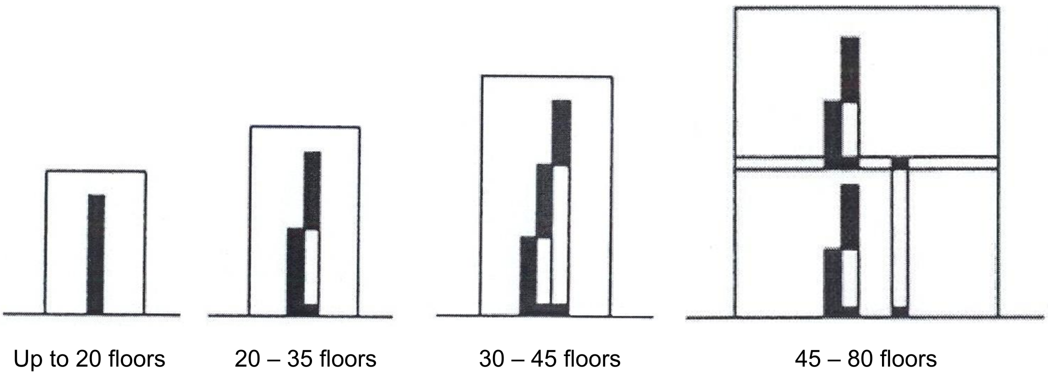
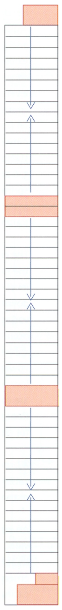



**Figure 3.41:** *Different lift configurations, influenced by the height of the building*

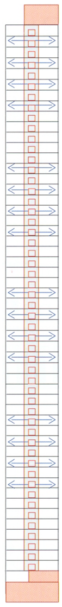
As mentioned in the introduction of this subsection, a range of building services also require vertical transportation. These services are listed below:

- Heating
- Cooling
- Water supply
- Sewage systems
- Fire-extinguishing systems
- Power supply

When a building gets taller it becomes necessary to decentralise the equipment areas for the building services. A centralised equipment area at the base of the building is only practical for buildings up to heights of approximately 25 floors. When taller, problems with for example the water supply will occur. Getting water to heights above 25 floors requires the use of heavy pumping stations.
The decentralisation of the building services can be done by designing several service floors from which the adjacent floors are serviced (figure 3.42a). In order to be efficient, each service floor should not serve more than 20 floors. The drawback is that large vertical shafts are needed for the cables and pipes which connect the floors to the service floor. This shaft means a "loss" of valuable letting space. To prevent large vertical shafts, building services can also be deployed at each individual floor (figure 3.42b). However, this way of arranging building services is less efficient than the use of service floors.



(a) Decentralisation of the building services by service floors



(b) Decentralisation of the building services on every floor

**Figure 3.42:** *Decentralisation of building services*

## 3.9 Fire Safety

The most terrifying event that can happen in a building is the outbreak of a fire. This concerns in particular high-rise buildings since there is no possibility of escaping through the windows of the building. Additionally all firefighting and rescue operations have to be carried out inside the building, resulting in strict requirements for the structural safety and stability. Generally the smoke production proves to be much more lethal than the fire itself, claiming victims through suffocation.

To prevent fire and smoke from spreading a couple measures have to be taken:

**Compartments:** Compartments limit the spread of a fire and smoke. These compartments are separated by walls with a high fire resistance.

**Material use:** The use of incombustible building materials will limit the spread of a fire. When combustible materials or materials not resistant to fire are used, they have to be protected to shield them from the fire, e.g. insulation materials and steel.

**Fire Detection:** An early detection of a fire is a very important measure to protect the occupants of a building against fire. In the beginning fires are still relatively easy to extinguish. Modern detection systems can detect smoke, heat, flames and even gas in an early stage and subsequently trigger the alarm.

**Sprinkler systems:** A sprinkler system is a very effective tool in firefighting.

**Firefighting installations:** It goes without saying that proper firefighting installations are necessary and essential to control fires. Necessary installations are: firefighter lifts, fire-extinguishing systems and installations for radio communication between the firemen in the building and the command post outside the building.

A very important phenomenon which occurs during fires in high-rise buildings is the so-called stack effect. During a fire the building will act as a giant chimney. All the smoke produced by the fire will travel all the way up to the top of the building. This means that a fire at the base of the building causes smoke to spread to all the floors above. To prevent this, compartments should also be implemented in the vertical shafts of the building.

## 3.10 Terrorist Attacks

Since the terrorist attacks on the World Trade Centre in New York City on the 11th of September 2001 there is a general concern about the safety of high-rise buildings with respect to terrorist attacks. This anxiety could end the desire to build ultra-tall skyscrapers. We should ask ourselves the question whether it is possible to design skyscrapers in such a way that they are capable to resist a terrorist attack.

To be able to answer this question, the structure of the two towers of the World Trade Centre is taken as the starting point. The structure of these towers was a so-called framed-tube system. The facade of the building was made of a closely spaced steel grid which formed the framed tube (figure 3.43). The central core of the building was constructed out of steel columns. This core was solely capable to accommodate the vertical, gravitational loads. The lateral loads acting on both towers were accommodated by the framed tube in the facade of the building. In the top of the towers, large hat trusses were constructed (figure 3.44). These trusses performed as outrigger structures, ensuring interaction between the outer framed-tube and the building's core. The floor structure of the towers was made out of slender steel trusses supporting a concrete floor slab.

The structure of the WTC towers proved to be robust, despite the huge forces unleased upon the structure by the impact of the airplanes, it did not collapse. The strong exterior frame of the building proved to be effective against the impact of the airplanes. Because of the high strength of the facade, it absorbed a major part of the impact's energy, decreasing the damage inflicted to the rest of the structure. An additional advantage of the facade was that the rigid steel grid was able to redistribute forces around the damaged areas, preventing the facade structure to collapse. Also the hat trusses played an important role in this redistribution of the forces, transferring forces from the facades to the cores of the towers (Ali [2]). An earlier, less known terrorist attack on the towers in February 1993 already proved that the structure was able to withstand a bombing at its base. In the end we can conclude that the structure of the WTC proved to be effective against an exterior attack.
Eventually, the fragility to fire caused the structure to collapse. The steel structure in the building was provided with a spray-on fire coating. Using this fireproofing is cost effective and is generally good for normal fires, however it is questionable for high impact loads and fierce fires (Ali [2]). Another major factor was the malfunctioning of the sprinkler system inside the building.

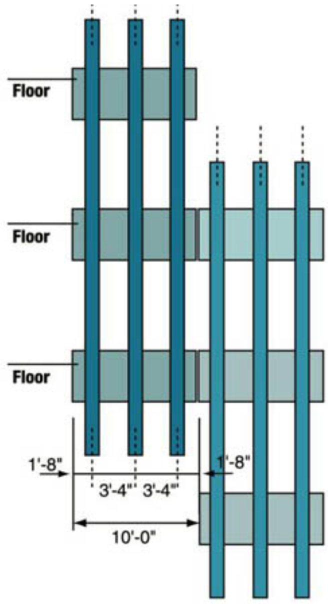



**Figure 3.43:** *The facade structure of the World Trade Centre*

To protect a building better against fire, concrete instead of steel, could be applied in the structure. Concrete is a non-combustible material, it has a high thermal inertia and it tends to remain in place during a fire. Another advantage of concrete structures are their high stiffness and their robustness. A drawback of new types of high-strength concrete is the spalling of the concrete when it is exposed to extreme temperatures.

Most of the occupants of the WTC which died in the attack, were the ones which were trapped above the floors of the impact zone. The egress routes in the core of the towers were blocked, leaving them no option for escape. The core of a building should be strengthened to withstand high impact loads or explosions, leaving it intact for evacuating people. This can be done by strengthening the core with steel plates (Ali [2]) or by applying a double concrete wall around the core (Baily [4]). Another solution can be to have multiple cores distributed over the building. These cores should be strategically located to minimise the risk of all being simultaneously disabled during an attack.

There are two different theories about what triggered the collapse of the WTC towers. One theory considers the collapse of the inner cores to be responsible for bringing down the towers. Another theory assumes that the collapse of the damaged floors initiated a progressive collapse of the building. In his paper Reitzel [40] explains an interesting measure to prevent progressive collapse of a building. His idea is to apply shock absorbers in a building. In case of a collapse the shock absorbers will absorb the kinetic energy of the collapsing stories, protecting the underlying structure. The shock absorbers are designed as double walls filled with a fluid that can be squeezed out through valves in the walls, absorbing the dynamic forces (Reitzel [40]). It is possible to combine these shock absorbing floors with floors needed for the installations of the building services. One might say that the shock absorbing floors act as a kind of airbag for the building.

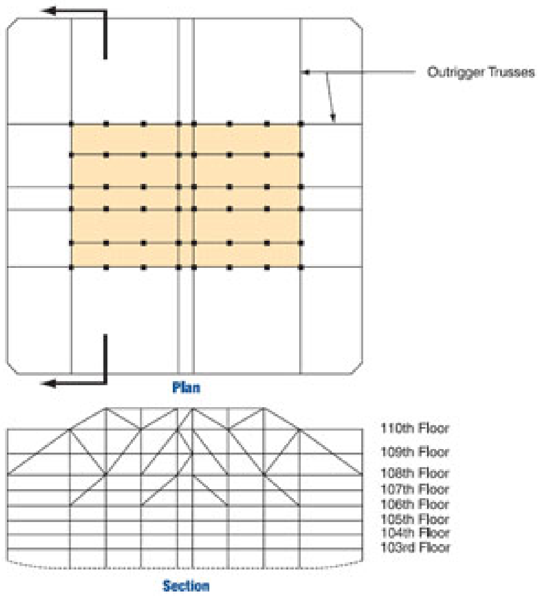



**Figure 3.44:** *The configuration of the hat trusses of the World Trade Centre*

## 3.11 Evacuating the Building

In case of an emergency it is important to be able to evacuate a building quickly and safely. The evacuation situation can be divided into three phases (Hakonen et al. [12]):

1. Recognition time
2. Reaction time
3. Egress time

The recognition time is the time occupants take to become aware of the emergency. The reaction time is the time from awareness to the time persons start to move out from the building. The egress time is the period that elapses after the reaction time, till all the occupants of the building are evacuated. The total elapsed time from the beginning of the emergency until the evacuation of all the occupants is called the evacuation time (figure 3.45).

The escape routes in high-rise buildings consist of the corridors, lobbies and staircases in the building. Besides serving as an escape route, these routes are also used by the rescue personnel to enter the building during an emergency. This two-way traffic in the staircases leads to substantial longer evacuation times due to congestion. Moreover, in planning stairs, only the floor population and the walking distance to the stairs or exit routes are considered. According to Hakonen et al. [12], no attention is paid to the total population in the building, or how fast people exit the building. Needless to say, these factors become more critical when the number of floors in the building increases. When building an ultra-tall skyscraper, despite the great height of the building and the large number of occupants, it must still

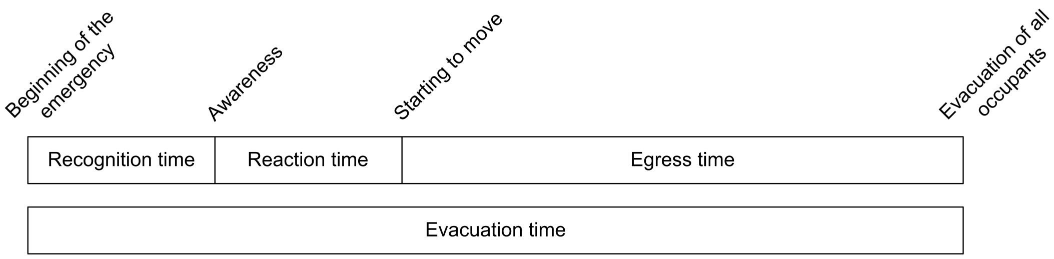
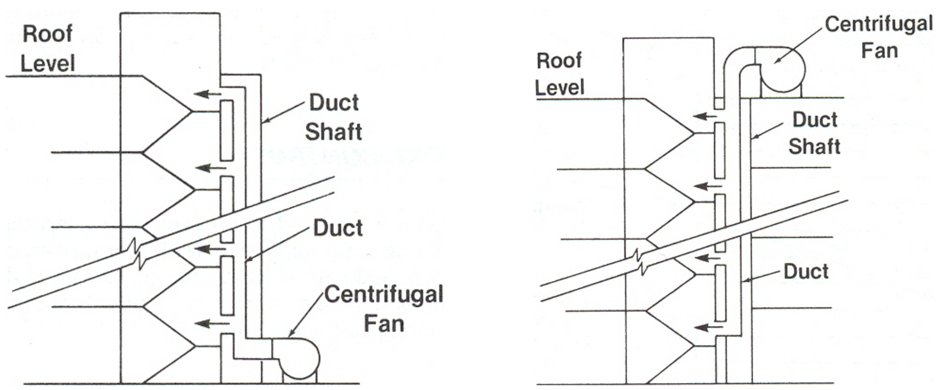



**Figure 3.45:** *The evacuation time of a building*

be possible to safely evacuate all the users in time. To make this possible existing evacuation methods should be improved or new ones have to be implemented.

The most important thing is that escape routes are kept free from fire and smoke. Therefore the walls must be resistant to fire and incombustible materials should be used. Electrical and data communication cables must never be laid along essential escape routes because these cables are fire hazards by themselves (Eisele and Kloft [11]). Stairwells should be "flushed out" with air to prevent the build up of smoke in the staircases. This can be done by automatically opening windows in the staircases, or in case of an internal staircase ventilation with pressurised air (figure 3.46). This air will simply "push out" the smoke when this finds its way into the stairwell. This very same system is being used to keep the firefighter lift shafts and its connecting lobbies free from smoke. Air is blown into the base of the shaft and flows into the lobbies through openings, pushing the smoke away from the lobbies.




(a) Stairwell pressurisation with fan located at ground level

(b) Stairwell pressurisation with roof-mounted fan

**Figure 3.46:** *Stairwell pressurisation*

Currently, lifts are not used in the evacuation of buildings. Only the firefighter lifts are used to evacuate disabled or injured people, but these lifts can only be deployed by the emergency personnel. The ordinary freight and passenger lifts can not be used in case of an emergency. They automatically return to the evacuation floor, parked with open doors and switched off. The reasons to put the lifts out of action during an emergency are mentioned by Baily [4]:

- Due to unpredictable nature of fire propagation, lift reliability can not be assured.
- Smoke propagation and accumulation is unpredictable, thus the safety of passengers can not be assured.
- The presence of water from sprinkler systems or fire hoses could disable lifts and entrap passengers.
- Human behaviour during the emergency may be very different under various circumstances and is thus unpredictable.
- Power availability during a fire may not be relied upon due to possible damage to power feeders and associated equipment.

Firefighter lifts, although available, are in reality not often used by the emergency personnel. This is because the personnel has insufficient knowledge about the condition of the lift and about the reliability of the lifts supporting systems. In this case, the Firemen use rather the stairwells for transporting equipment, rescuing people and fighting fires.

To improve the speed and effectiveness of the evacuation of a high-rise building, the possibility of using the lifts in the evacuation plan can be considered. In a high-rise building, lifts are by far the fastest way out. Using lifts for evacuation purposes is especially useful in the beginning of the emergency, even before emergency personnel have arrived. As time passes, the more likely a fire will reach the lift shafts or machine rooms, putting the lifts out of order. The direct use of a lift system for evacuation requires a system which can operate in an emergency situation without a rescue worker operating it. Important for such a system is a supporting monitoring system which constantly determines whether or not it is safe to use the lift. Smoke, gas and temperature sensors are placed in the lifts hoistway, machine room and evacuating floors to ensure the safety. As soon as these sensors measure a value which indicates that using the lift is not safe, the lift will be ordered to travel to the nearest safe landing where it will be switched off. To make lifts suitable for evacuation purposes, the lifts should be switched automatically or manually in an emergency drive mode. This emergency drive mode overrides the standard drive mode (Brlund et al. [8]). In their article Brlund et al. [8] explain how a lift in the evacuation mode should function in order to optimise the lifts handling capacity so that passenger transportation towards exit floors consume as little time as possible. First of all the dispatching of the lifts will be done automatically. Manual dispatching, as in the currently used firemen's mode, can never compete with the efficiency of automatic dispatching. Second, the lifts will only stop at emergency evacuation floors and calls on intermediate floors are disregarded. The passengers can only exit the lift at emergency egress floors. Third, the lift cars are filled to maximum capacity at each stop to further increase traffic handling capacity. Finally, the normal comfort standards should be neglected once the lifts are in evacuation mode to further improve the efficiency. Acceleration and deceleration shall be carried out at maximum power, without compromising the machinery (Brlund et al. [8]). To make lifts suitable for evacuating people, hardened lift cabs have to be applied, e.g. no smoke may enter into the lift cars, the electronics should be waterproof, and a back-up power supply should be available (Baily [4]).
It should be noted that when using lifts for the evacuation, stairwells do not become redundant. Hakonen et al. [12] state that the fastest way out of a building is to use both stairs as lifts.

To improve the evacuation of high-rise buildings it may be necessary to think beyond existing ways of evacuating occupants from a buildings. Problems occurring at conventional means of egress are for example (Shimshoni [43]):

- The limited capacity, the physical difficulty and the slow rate of evacuation of occupants through stairwells.
- The limited capacity, the physical difficulty and the slow rate at which the emergency personnel can access the building through the stairwells.
- The limitations of evacuating disabled or injured people.
- The lack of alternative evacuation routes when one route is blocked.

In his paper Shimshoni [43] enumerates a couple of alternative evacuation methods which are already available on the market.
A *"Controlled descent device"* lowers a person at a controlled rate of descent on the outside of a building (figure 3.47). The person is connected via an individual harness to a rescue line or descent rail on the exterior of the building.
Another method is the use of *"escape chutes"* (figure 3.49). These systems are similar to the inflatable escape-chute system which is used for evacuating passengers out of large airplanes. The escape chutes can be made of fire-resistant fabric or netting.
The last method mentioned in the article is the *"Platform rescue system"*. A platform rescue system is defined as an enclosed platform or cabin which moves along the exterior of the building (figure 3.48). This system is suitable for both evacuating the occupants from a building as for transporting rescue personnel up the building. The system is installed on the roof of the building and can, in case of an emergency be controlled by the rescue personnel on the ground. In contrary to the other two systems, this system is capable of evacuating a large flow of evacuees.


**Figure 3.47:** *A controlled descent device*

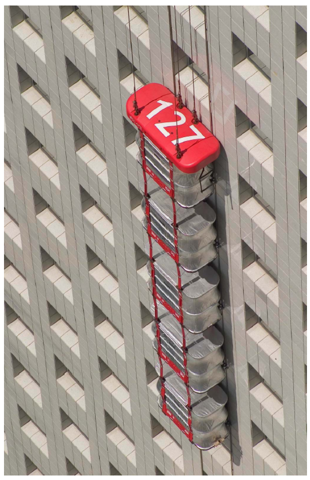



**Figure 3.48:** *A platform rescue system*

(a) The Baker life chute

(b) Advanced modular evacuation system

**Figure 3.49:** *Escape chutes*

Another promising solution for the evacuation problem is the use of skybridge linkages in between skyscrapers. These linkages make horizontal evacuation possible at high altitudes. When an emergency occurs in one building the skybridges make it possible to flee to the other building and safely evacuate. When multiple high-rise buildings are connected to each other with multiple skybridges, a lot of alternative evacuation routes are created. That this is a very real solution for improving the evacuation efficiency is shown by the fact that five out of the seven official proposals for the World Trade Centre re-design competition include linkages between towers at higher altitudes. In his paper Oldfield [34] analyses the skybridge connections in each of the proposals. He concludes that skybridges are no longer a fantasy, but a real proposal for increased evacuation efficiency.

## 3.12 Economical Feasibility

In her book Willis [53] explains that buildings, and especially high-rise buildings, can be seen as real estate ventures. The structures are erected by investors in order to generate income out of them. This is achieved by letting out floor space in the building, or as a long-term investment in high-value urban land. Because of this, the economical feasibility of the building is an important factor. The generated income should at least cover the expenses of the building. This means that an economical building height will be the height at which the marginal cost of creating another floor is equal to the sales price realised for that floor (Watts et al. [52]).

Besides serving as an investment, it is often said that high-rise buildings are not about economics alone. When building a skyscraper there is often a lot of prestige involved in the project. This is described by Helsley and Strange [14]. "The premium may be a matter of taste alone. An oversized building is a good match for the oversized ego of the investor."
Additionally, a skyscraper build by a company can serve as an advertisement or may signal the fitness of

the company to the outside world. Examples of this can be found in the past. When Frank W. Woolworth constructed the the Woolworth Building in New York, he encouraged his architect Louis Horowitz to build the in that time tallest skyscraper in the world. Woolworth realised that, after completion, the building would be mentioned in every newspaper and would be pointed out to every tourist (Helsley and Strange [14]). An other example is the race between two large American automobile companies in the 1920's: General Motors and Chrysler. Both were constructing their headquarters: the Manhattan Company Building and the Chrysler Building. The race was on to construct the world's tallest building in order to signal the fitness of the companies.
Similar, skyscrapers can also be built in order to promote a city instead of a company. The construction of a new skyscraper can cause a boost in the regeneration of a city or region (Watts et al. [52] ). Recent examples of this are London's Docklands, Shanghai's Lujiazui Trade and Enterprise Zone and Dubai.

(Willis [53]) states in her book that, even if a building is built to promote a company or to boost an investor's personal ego, they still have to be seen as real estate ventures. To support this opinion, she gives the following arguments:
Very often, in buildings which are owned by a company, a considerable amount of the floor space is let out to other tenants in order to cover the regular expenses of the building. This makes these buildings real estate ventures as well.
Additionally, investors are more willing to invest in distinctive skyscrapers because the returns on their investments will be larger. Prime locations, rich materials and above all, the height of the skyscraper give the building publicity which will benefit the income which is generated from it (Willis [53]). This means that it does not solely serve as an ego boost.
She concludes that every skyscraper, owned by a company or investor, can be seen as a money-generating property. The buildings are part of a marketplace where space is for sale, and location and image have value (Willis [53]). Therefore the economical feasibility of the building plays an important role. A skyscraper will never be built with the sole purpose of prestige. When the building is not economically feasible, it is almost certain it will not be built at all.

Below are listed some concerns which can negatively influence the economical feasibility of a building.

**High construction costs:** Constructing a skyscraper will result in relatively higher construction costs than the construction of an ordinary low-rise building. We can mention two main reasons for this. The first is that relatively more material has to be used as the height of the building increases. This can be ascribed to the exponential increase in the horizontal loads caused by the wind forces. Higher loads mean that a larger amount of building material is needed to accommodate these forces. A second reason is the complexity of building a skyscraper.
When the terrain on which the skyscraper is built, was first in other urban use, additional costs have to be taken into consideration. The revenues generated from the building should cover both the out-of-pocket costs of construction and the opportunity cost of lost rents from the previous use (Helsley and Strange [14]).

**High maintenance costs:** The maintenance costs for a high-rise building can mount up to considerable amounts.

**Ratio G.F.A. to L.F.A.:** The ratio "Gross Floor Area" to "Lettable Floor Area" is one of the most important factors. This ratio expresses how much floor surface is "lost" to other functions from

which no revenues can be generated. In skyscrapers this is the amount of space which is taken by stairwells, lift shafts, building services and sanitary fittings.
A solution to this problem is to locate these "space losses" in areas which are not suitable for letting out, e.g. in the centre core of the building due to the lack of entry of daylight.

In the past, most high-rise buildings only had one function, e.g. commercial, office, residential or hotel. However multi-use tall buildings have much more potential than single-use buildings (Kim and Elnimeiri [23]). One of the reasons that multi-use buildings have not been constructed in large numbers, is that they are difficult to design efficiently.
When a building accommodates multiple functions each of these functions needs its own entrance and separate lift system. In order to reach the higher functions, one needs to travel through the other functional areas of the building. These lifts take up valuable space.
It is difficult to obtain an optimum design for a multi-use building. Preferably, from a tenant's perspective, the lowest floors of the building should be used for parking followed by the floors used for commercial purposes, the next floors should be used for office space, the next for hotel function and finally for residential purposes. However, from a structural point of view, this is unfavourable. At the bottom of the building the centre-to-centre distances between the columns will be the smallest. This makes the lower floors of the building more suitable for hotel or residential functions. Offices need longer floor spans in order to make them more flexible in their lay-out. The challenge is to balance these issues (Kim and Elnimeiri [23]).
Another problem is that every function in the building requires another storey height. These irregularities could cause problems in the design and construction of the building. This could also mean that the building is more vulnerable to earthquakes (see relevant section).
Very often, special functions are accommodated at the top floors of the skyscraper, e.g. an observatory or a restaurant. These functions need their own lifts. These lifts have to run through the entire building. Kim and Elnimeiri [23] calculated that in most cases, these lifts take up almost two percent of the gross floor area.

## 3.13 Sufficient Economical Support

The location of an ultra tall-skyscraper has be to capable to economically support such an enormous structure. In other words, the economical environment has to be sound. Watts et al. [52] mention a couple of attractors:

**Human Capital:** the availability and concentration of "human capital" relates to the available amenities for companies and investors.

**The Business environment:** economic regulation and taxation, proximity to financial markets and ease of doing business within a city, all contribute to attracting international business to locate in a city.

**Market Access:** The availability of capital and the volume of trade, combined with the provision of legal and accounting services to the financial sector make cities more attractive to tenants in the commercial sectors.

**Accommodation availability:** The quality, availability and occupancy cost of office space are influencing the attractiveness of a city to firms.

**Transport infrastructure:** The provision of infrastructure that is of sufficient quality, safety and capacity and which provides access to global transport hubs, is essential.

## 3.14 Market Instability

The economy in countries is generally subjected to cycles. Years of prosperity are followed by years of depression. A similar, more extreme pattern can be seen in the real estate development. Low interest rates and a high money supply is a driver for investments in capital-intensive projects such as land and large buildings. "The lowered opportunity cost of investment when debt-servicing costs are reduced, leads to larger investments hence taller buildings" (Watts et al. [52]). Large profits from buildings completed early in the cycle, attract other developers and investors into the market. The consequence of this is that a lot of projects are initiated at the same time resulting in an oversupply.
This oversupply will reinforce "black leverage". Black leverage means that when a skyscraper becomes available to let in an economic downturn, it will suffer from under-occupancy and rising interest rates on the large debts (Watts et al. [52]). This same happened to the Empire State Building in New York, which soon became known as the "Empty State Building", with more than forty vacant floors. The overbuilding puts pressure on the real estate market for a number of years, resulting in hardly any activity or even a complete shutdown in construction.

Besides the more extreme cycles, the real estate cycle also has a time lag of about 1–3 years compared to the economic cycle. This phenomenon is caused by the large time span between the initiation and the completion of building projects. This lag disguises the magnitude of all development until well after the supply of new space has exceeded demand (Willis [53]). The demand has decreased due to the worsened economy. Additionally, it is not possible to simply stop the construction midway to respond to the declining economy.

When skyscrapers grow ever taller, the time span between initiation and completion will increase as well, resulting in higher risks for the investors. These risks will discourage investors to invest in a super-tall skyscraper.

# Chapter 4

# Reference Projects

## 4.1 Taipei 101

| | |
|---|---|
| **Location:** | Taipei, Taiwan |
| **Completion date:** | 2004 |
| **Client:** | Taipei Financial Centre Corp. |
| **Architect:** | C.Y. Lee & Partners Architects/Planners |
| **Structural engineer:** | Thornton-Tomasetti Engineers, Evergreen Consulting Engineering, Inc. |
| **Vertical transportation consultant:** | Lerch, Bates & Associates, Inc |
| **Contractor:** | Kumagai Gumi, Taiwan Kumugai, RSEA Engineering, Ta-You-Wei Construction |
| **Height:** | 508 metres |
| **Stories:** | 101 |
| **Use:** | Mixed: commercial, office |
| **Principal structural materials:** | Steel |

The Taipei 101 is currently the tallest completed skyscraper in the world. Initially the owner of the building, Taipei Financial Centre Corporation, planned several towers of more modest height. When it appeared that all the investor-occupants wanted their office space in the tallest one, it was decided to build one single tall tower, despite the higher costs.

### 4.1.1 The Building

The unusual design of the tower can be traced back to the regularly spaced joints of a bamboo stalk. Furthermore the design is influenced by the tiers of the Chinese pagodas and the popularity of "lucky number 8" in China. Latter is expressed in the eight equally shaped modules of the tower, each with eight floors. Above these modules the tower is completed by a spire, rising from the 91st floor and containing 10 additional floors. The last 60 metres of the spire are not fit for habitation (figure 4.1).

The skyscraper has a large shopping mall at its base. The total amount of floor surface is 412 500 m$^2$. Underneath the building lies a five stories deep basement.

**Figure 4.1:** *Taipei 101 in Taipei, Taiwan*

### 4.1.2 The Structural System

The foundation of the building includes drilled concrete piers. Two different kind of piers were used: Supporting the tower are 380 piers with a diameter of 1.5 metres and with lengths ranging from 40 to 60 metres. Supporting the surrounding shopping mall are piers with a 2.0 metres diameter. Their length ranges from 5 to 28 metres.
The main structure of the building consists of 24 steel box columns of which 16 columns form the core of the building in a 4 x 4 configuration. This core houses the buildings 48 lifts. The core columns are rigidly linked to each other by steel diagonal and chevron bracing. The remaining eight columns are located on the perimeter of the building and are so-called "super-columns" with dimensions of 3 $\times$ 2.4 metres. These super-columns are connected to the building's core by steel outrigger trusses (figure 4.2). In this way the super-columns are engaged in accommodating the lateral forces acting on the tower. The mechanical floors on each of the eight modules provided ideal opportunities for accommodating these outriggers (Joseph et al. [21]). All the steel box columns are assembled from 8 cm thick steel plates.

### 4.1.3 Wind Engineering

Wind tunnel tests revealed that the vortex generation at the building's corners, caused large crosswind oscillations at certain wind speeds. Therefore, alternative corner shapes were examined in the wind

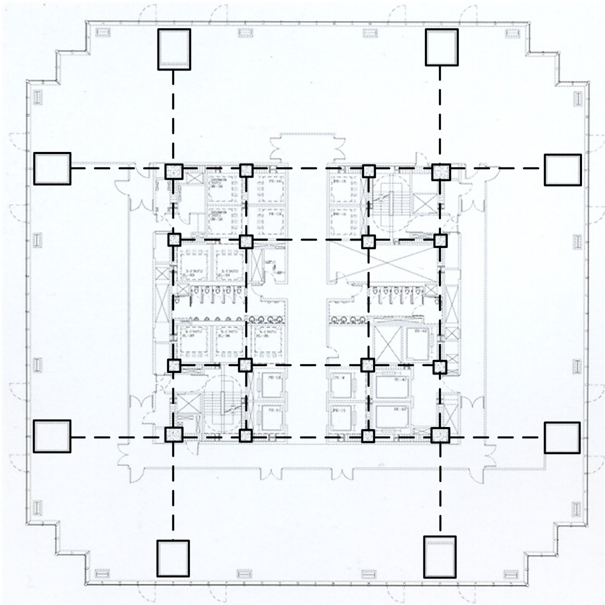


**Figure 4.2:** *The typical floor plan of the Taipei 101, with 16 columns in the core of the building and eight super-columns at the perimeter of the building*

tunnel. It was determined that a "double stair-step" notched corner gave the best results, so this corner type was implemented in the design of the Taipei 101.
Even after this reduction, the wind forces still caused a large deflection of the structure. This was mainly due to the fact that the structure of the Taipei 101 is constructed out of steel, a lightweight material which is vulnerable to lateral swaying when imposed to lateral loads. Although its seemingly unsuitable properties for utilising in high-rise construction, steel was preferred above concrete for three reasons:

1. Minimising the cost of the towers foundation by keeping the building weight low.
2. Minimising the seismic forces by keeping the mass of the building low.
3. To benefit from a skilled local steel construction industry.

Additional solutions had to be found to improve the building's dynamic behaviour under lateral loading. The option of adding more stiffness to the building by adding more steel was thought to be too expensive. Instead, it was decided to fill the steel box columns of the building up to the 62nd floor with concrete, creating a much stiffer structure.
Still, the structure did not meet the comfort criterion. Therefore three TMDs were installed in the tower. The main TMD occupies levels 87 till 91 as a centrepiece of a public lounge (figure 4.3). The TMD is a Simple Pendulum Damper, a 726 ton sphere of stacked steel plates, suspended from four pairs of steel cables. The lengths of the cables are adjustable to tune the TMD to match with the behaviour of the tower. The other two, much smaller, TMD's are applied in the spire of the building. Each of them is "only" weighing 5 tons.

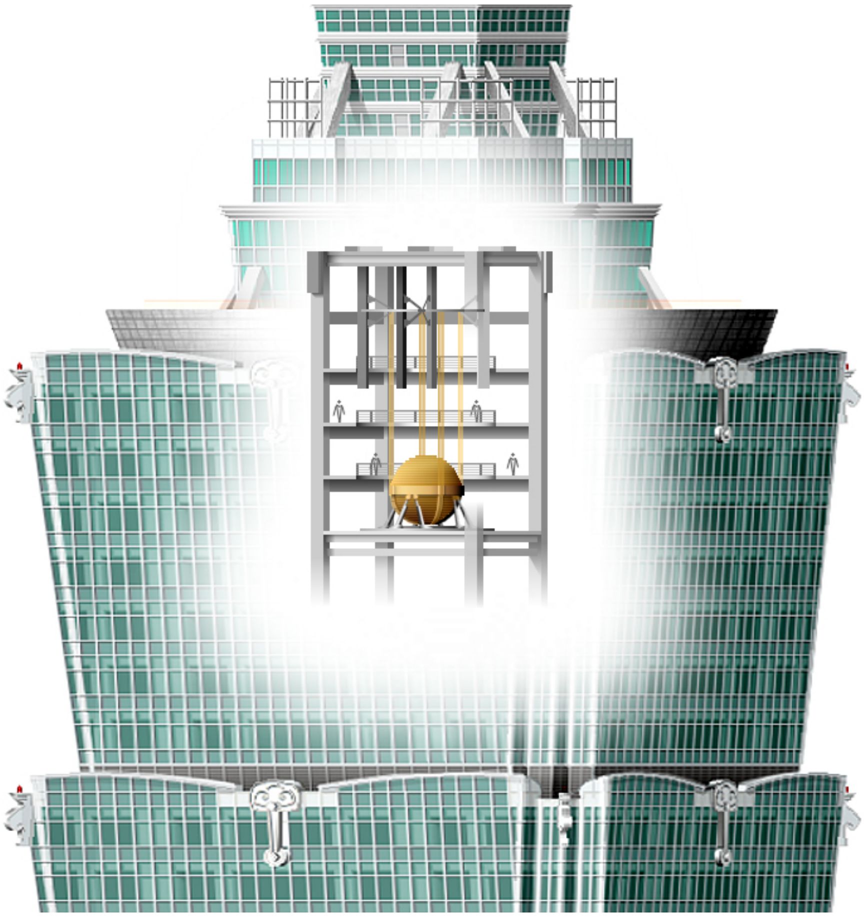


**Figure 4.3:** *The main TMD inside the Taipei 101*

## 4.2 Burj Dubai

| | |
|---|---|
| **Location:** | Dubai, United Arab Emirates |
| **Expected completion date:** | 2009 |
| **Client:** | EMAAR Properties |
| **Architect:** | Skidmore, Ownings & Merrill LLP (SOM), Adrian D Smith, FAIA, Consulting Design Partner |
| **Structural engineer:** | Skidmore, Ownings & Merrill LLP (SOM) |
| **Mechanical engineer:** | Skidmore, Ownings & Merrill LLP (SOM) |
| **Vertical transportation consultant:** | Lerch, Bates & Associates, Inc. |
| **Contractor:** | Samsung-BESIX-Arabtec |
| **Height:** | More than 800 metres |
| **Stories:** | 160 |
| **Use:** | Mixed: hotel, residential, commercial, office |
| **Principal structural materials:** | Concrete |

The Burj Dubai, currently under construction in Dubai, will be the tallest structure in the world when it is completed in the year 2009. The exact height of the building is top-secret, but it is expected that the building will rise to a height well above 800 metres. The importance of the being world's tallest building and remaining this for a considerable period of time is illustrated even better by the fact that the height of the tower was changed when construction had already started. This due to rumours telling that designs for even a higher skyscraper were made elsewhere. The consequence was that the structure of the building had to be reinforced with an additional 12,000 tons steel.

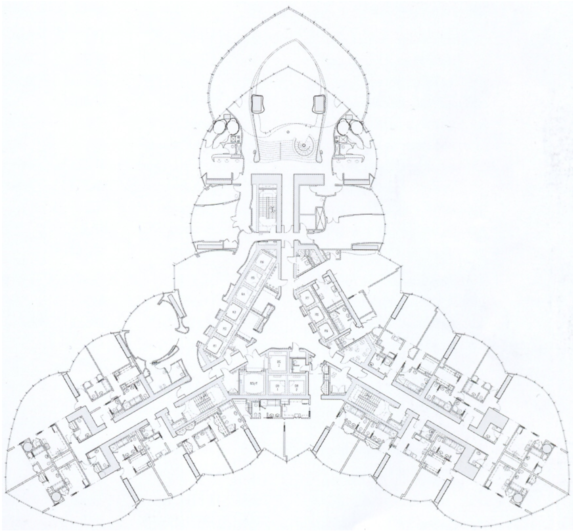
### 4.2.1 The Building

The Burj Dubai is part of a larger town planning developed by EMAAR properties. The ground plan of the building has a Y-shape and is designed by analogy with the hymenocallis, a desert flower which can be found in the desert surrounding Dubai.
The building can be divided in two parts: the building's plinth of 279,000 m$^2$ and the tower itself which contains 186,000 m$^2$ of floor surface. Underneath the tower lies a five stories deep basement.
The tower can be divided into 5 different zones, each with a different function. In the *first* zone of the building (until floor 39) a hotel is planned. The *second* zone (until floor 108) is reserved for residential use. In the *third* zone (until floor 153), offices and stores will be realised. The *fourth* zone (until floor 160) of the building is used for communication equipment. The top of this zone will be at a height of approximately 605 metres. The *final* zone, the next 200 metres of the building, consists of a steel structure which forms the spire of the building. This part of the structure is not fit for habitation and has merely an aesthetical function.
The Burj Dubai has a tapered shape. Because of this the floor surface decreases over the building's height. At ground level the floor surface measures 3,500 m$^2$ while the upper floor measures only 650 m$^2$ (figure 4.4).

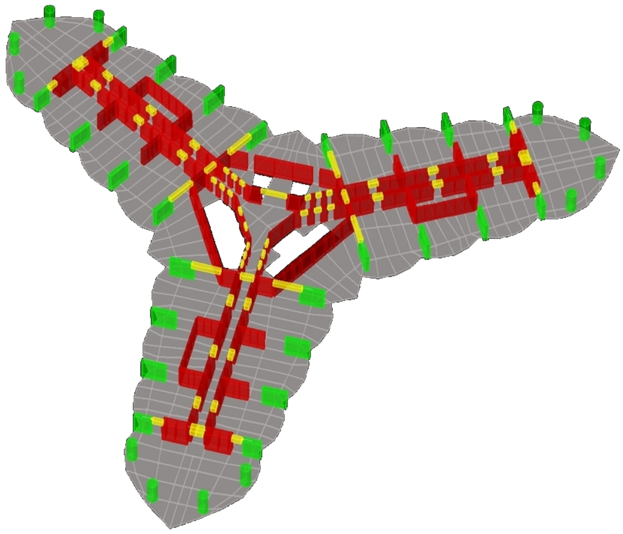
**Figure 4.4:** *The Burj Dubai in Dubai, UAE*

### 4.2.2 The Structural System

The foundation of the Burj consists of 200 drilled concrete piles. Each of them is 50 metres long and has a diameter of 1.50 metres. The load-bearing capacity of one single pile is equal to 3,000 tons. The 4,000 $m^2$ concrete base plate of the building is 3.70 metres thick.
The major structural element is the central concrete core of the building (figure 4.5). This core has a hexagonal shape and runs all the way up to a height of approximately 575 metres. After that, the size of the core is diminished to two-third and later to one-third of its original size. Eventually it reaches a height of 620 metres. The building's stairwells and 54 lifts are situated inside the core. Protruding from the core are the three wings of the building, which gives it its distinct Y-shape. In the centre of each wing a buttress, formed by concrete corridor walls, is supporting the core. At the end of these buttresses, concrete "hammerhead walls" are constructed. The facade of the building is supported by 60 cm thick slab-shaped concrete columns. The tower is serviced by five separate mechanical zones, located approximately 30 floors apart. At these floors, outrigger walls engage all the perimeter columns of the building. This allows these columns not only to participate in resisting gravitational loads, but also in accommodating the lateral loads acting on the building.
Because of the close spacing of the load-bearing walls in the building, the spans of the floor slabs are small. Therefore a flat-slab reinforced concrete floor of only 200 mm thick is sufficient.
In the entire structure 260,000 $m^3$ of reinforced concrete and 35,000 tons of steel (from which 31,000 tons reinforcing steel) are used.



(a) Floor plan of the Burj Dubai seen from above



(b) 3D model of the floor plan of the Burj Dubai

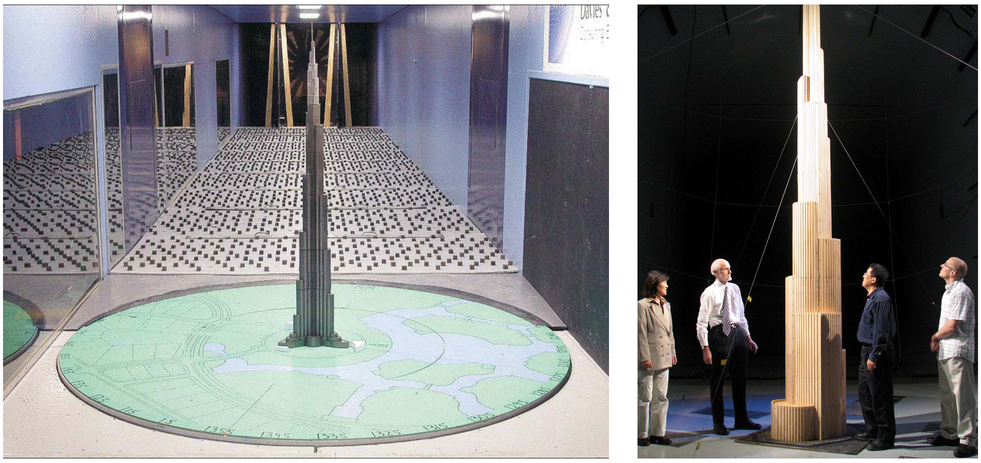
**Figure 4.5:** *Floor plan of the Burj Dubai*

### 4.2.3 Wind Engineering

Because the entire structure of the Burj Dubai is constructed out of concrete it behaves stiff under lateral loading. Extensive research to the behaviour of the tower under wind loading, has been done. For this research two scale models of the building have been constructed for wind tunnel testing (figure 4.6). With these tests not only the deformation and acceleration of the tower were tested, also the local wind pressure on the building's facades and the wind comfort for pedestrians around the building were examined.

Through these wind tunnel tests it was determined that the building will have a deflection of 1,450 mm at the top at wind speeds which have a probability of occurring once in fifty years. The corresponding acceleration of the building at these circumstances is 0.024 g. The deflection of the upper office floors will be 1,250 mm with an acceleration of 0.010 g. The upper residential floors will have a lateral movement of 540 mm with a maximum acceleration of 0.007 g. These results were satisfactory. However, it was discovered that measures had to be taken to improve the comfort for pedestrians on the terraces of the building.

The designing engineers have calculated that the tower does not need a TMD to dampen the building's lateral swaying. However, space is reserved in the building's structure to implement a TMD afterwards, if this proves to be necessary.



(a) A 1:500 scale model of the Burj Dubai

(b) A 1:50 scale model of the top of the Burj Dubai

**Figure 4.6:** *Scale models of the Burj Dubai*

## 4.3 Russia Tower

| | |
|---|---|
| **Location:** | Moscow, Russia |
| **Expected completion date:** | 2011 |
| **Client:** | STT Group |
| **Architect:** | Fosters and Partners |
| **Structural engineer:** | Halvorson and Partners structural engineers, Waterman Group |
| **Vertical transportation consultant:** | Lerch, Bates & Associates, Inc. |
| **Height:** | 600 metres |
| **Stories:** | 118 |
| **Use:** | Mixed: hotel, residential, commercial, office |
| **Principal structural materials:** | Concrete and Steel |

In September 2007, construction started on the Russia Tower in Moscow. When completed, the tower will not only be Europe's tallest skyscraper, it will also be among the tallest skyscrapers in the world. A remarkable achievement, since Europe has always played an insignificant role in building super-tall skyscrapers.

The client, STT Group, first approached Fosters and Partners with an idea of building three separate medium-rise buildings with heights ranging from 30 to 60 floors. Limited by the dimensions of the site, this resulted in three slender, inefficient towers. To improve the efficiency of both the structural system and the applied building services in the towers, the possibility of linking the three buildings was discussed. This eventually resulted in a plan for one, much taller, tower (figure 4.7).

### 4.3.1 The Building

The three separate towers evolved into three wings radiating from a central spine, giving the floor plan of the building a Y-shape (figure 4.8). Linked together, one wing stabilises the other two. Each of these wings is only 21 metres across, offering great potential for the entry of daylight into the building.
The building has a tapering shape, which means that the width of the building reduces when the height increases. At a height of approximately 500 metres a five-level observation deck will be realised. Above this observation deck, the wings of the building will rise another 100 metres towards the sky, to a height of 600 metres. This part of the building is not fit for habitation. It only provides space for communication masts. The total floor surface of the building will be 320,000 $m^2$. Underneath the building will be a six floors deep basement.

**Figure 4.7:** *The Russia Tower in Moscow, Russia*

### 4.3.2 The Structural System

The central concrete core in between the wings of the building is braced by a series of sloping concrete columns, the so-called "fan columns", which all come to one point at the base of each wing (figure 4.9). In this way, the main structure of the building shows a strong resemblance to that of a cable stayed mast. However, instead of tension cables, the tower has sloped columns in compression to prop the spine against wind loads and which are able to carry gravity loads (Warner and Halvorson [51]).

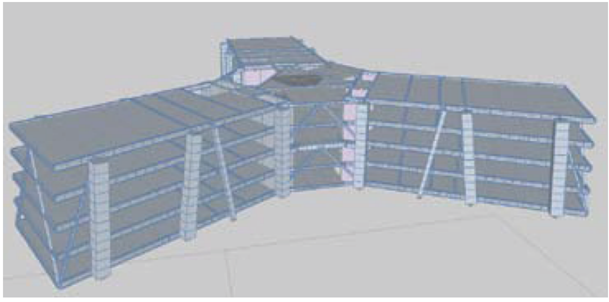


**Figure 4.8:** *Four storey portion of the tower*

Because the fan columns are capable of carrying both vertical and horizontal loads, the moments and shear forces in the core of the building are reduced considerably. The horizontal force components in the sloping columns are balanced between the three wings, introducing a tension force in the base plate of the building.
Still, there was one challenge remaining: torsion. The core of the building alone was not stiff enough to withstand the torsion. Therefore the wings of the building should perform as closed sections to participate in resisting the turning motion of the building. This stiffness was achieved by adding "reverse fan columns" to the structure of the building (figure 4.9). Where the fan columns hit the central core of the building, reverse columns deflect at the same relative angle to the core and extend upwards (Warner and Halvorson [51]).
In addition to the concrete columns and core of the building, steel is used for perimeter girders in the towers wings. The perimeter spans vary due to the fan layout of the columns, reaching a maximum span of 18 metres at certain places. This span is too large for ordinary concrete beams.
At the office levels, composite steel trusses span the 21 metres wide wings of the building. Because a shallower floor framing was desired at the hotel and residential floors, interior steel columns were introduced to reduce the floor spans. The loads from these columns are transferred to the perimeter fan columns via storey-high trusses at the mechanical levels of the building (Warner and Halvorson [51]).

### 4.3.3 Wind Engineering

A three-dimensional computer model of the building was made to determine whether the strength of the building's facade was sufficient to resist the wind loads.
The lateral accelerations of the tower were within the safety and comfort limits, so the structure did not require any additional damping. This is mainly due to the stiff lateral behaviour of the braced core.

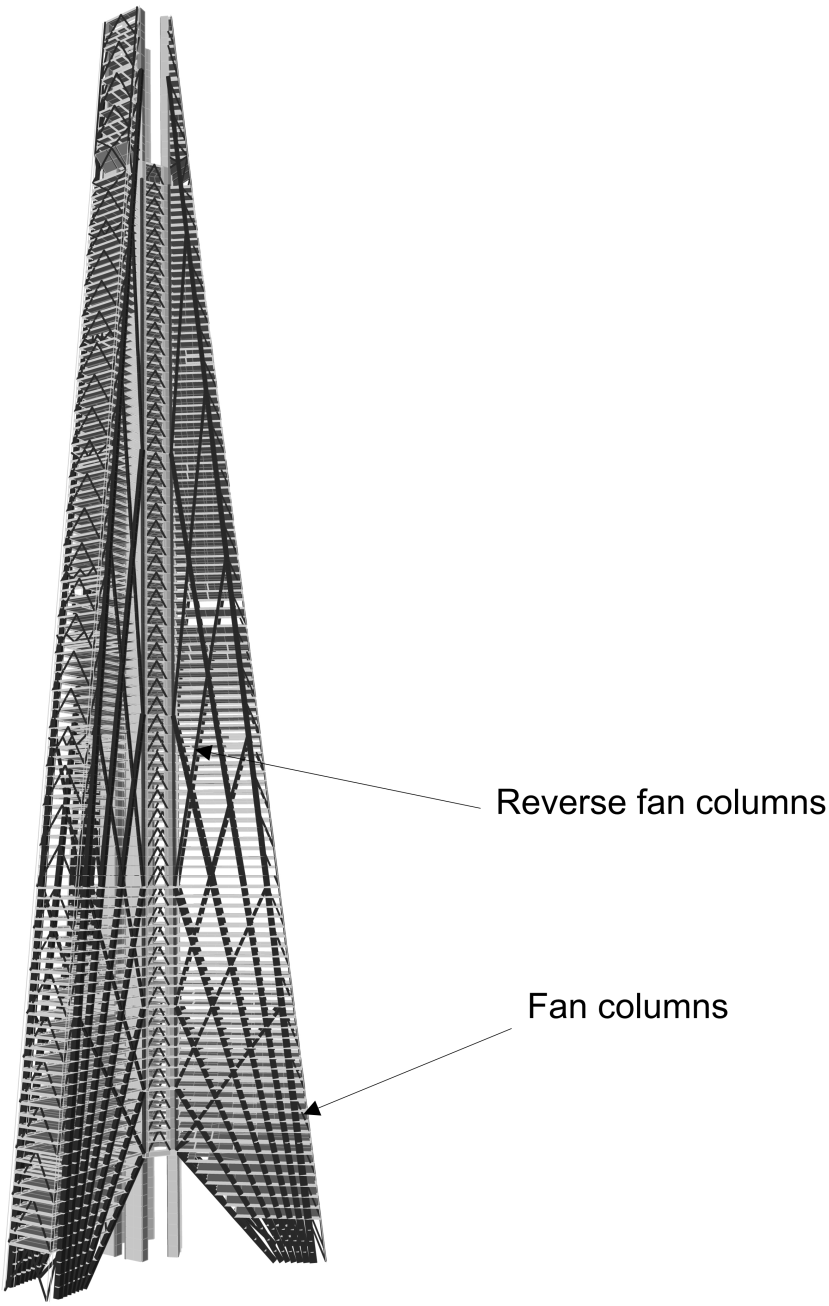



**Figure 4.9:** *Overview of the structure of the Russia Tower*

## 4.4 International Commerce Centre

| | |
|---|---|
| **Location:** | Hong Kong, China |
| **Expected completion date:** | 2010 |
| **Client:** | Harbour Vantage Management Limited |
| **Architect:** | Kohn Pedersen Fox Associates PC, Wong & Ouyang (HK) Ltd. |
| **Structural engineer:** | ARUP (HK) |
| **Mechanical engineer:** | JRPL |
| **Vertical transportation consultant:** | Lerch, Bates & Associates, Inc. |
| **Height:** | 484 metres |
| **Stories:** | 118 |
| **Use:** | Mixed: hotel, commercial, office, subway station |
| **Principal structural materials:** | Concrete |

When completed, the International Commerce Centre will be Hong Kong's tallest skyscraper. Together with the currently tallest building in Hong Kong, the Two International Finance Centre (415 m), it will create a virtual gateway to Hong Kong's Victoria Harbour (figure 4.10).

**Figure 4.10:** *The International Commerce Centre, with on the background the Two International Finance Centre*

### 4.4.1 The Building

The International Commerce Centre (the ICC) is the centrepiece of a large office, retail and residential complex, currently under development. The complex is build above a major stop on the city's subway line. The ICC will contain an entrance to this important subway station.
When completed, the building will have 4 basement levels. Atop of these levels sits a four-storey podium for retail and parking services.
At the bottom of the tower the facade flares into four canopies of different lengths. These canopies

protect the entrances of the building against the monsoon rains and winds, as well as against the downdraught wind from the tower itself.
The top floor of the building ends approximately 12 metres below the actual top of the building. These last 12 metres consist of cantilevered facade panels which are supported by a steel frame on the building's rooftop. This steel frame will also support the building's maintenance equipment.
The total floor surface in the building will be 232,250 $m^2$.

### 4.4.2 The Structural System

In the previous discussed projects, large-diameter bored concrete piles were used for the foundations. However, the use of this type of pile was not possible for the foundation of the ICC. The deep layer of solid rock and the presence of a fault zone directly underneath the building made this option unworkable (Reid [38]). Therefore a friction pile foundation consisting of 242 barrettes was used. To avoid that damage is inflicted to the surrounding structures, shaft-grouted barrettes were applied instead of driven barrettes. Each barrette pile can support a gravity load of approximately 2,600 tons.
Like in the previous discussed projects, the main structural element in the building is a central core. The core will be constructed out of reinforced concrete and will contain the building's stairwells and lift shafts. The internal dimensions of the concrete core are 30.5 by 32.5 metres. The typical floor plan surrounding the core, measures 60.5 by 60.5 metres. Along the perimeter of the building eight super-columns will be constructed (figure 4.11). To engage these in accommodating the lateral loads, the super-columns are connected to the building's core by outriggers. These outriggers are located at at four different levels throughout the total height of the building. The super-columns were originally designed as composite columns with embedded steel sections to minimise the column area. However, the design was changed to reinforced concrete because of a dramatic rise in steel prices during the design phase (Reid [38]). For the very same reason, only three of the four outriggers are constructed out of steel. The fourth will be made of prestressed concrete. The downside is that the structural depth of this concrete outrigger will be 21.5 metres against 14.1 metres of its steel counterparts.

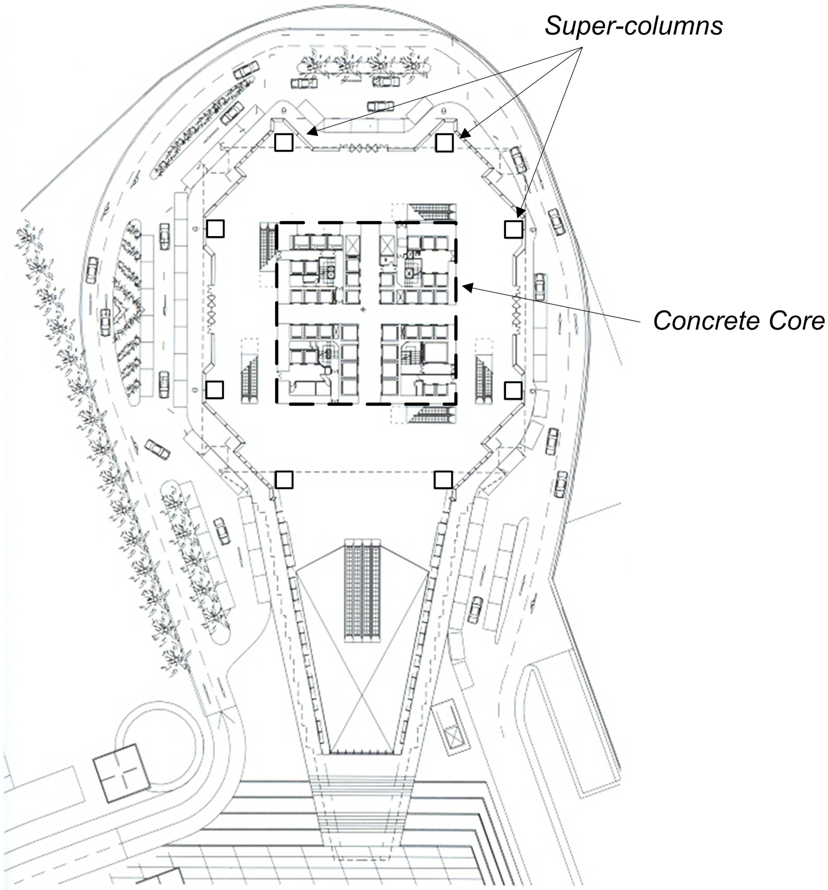



**Figure 4.11:** *The floor plan of the ICC at ground level, with eight super-columns at the perimeter of the building*

# Chapter 5

# Converging the Thesis' Problem

In the first chapter of this report, the goal of this thesis has been described: "To gain a good insight into the challenges which will be encountered when designing and constructing an ultra-tall skyscraper, with the aim to find the ultimate limit to the height of the skyscraper."
Through chapters 2, 3 and 4, extensive knowledge and understanding has been gained about the skyscraper. This means that we can focus now on the second part of the thesis' goal, finding the limits to high-rise. To be able to do this, the thesis' problem has to be further confined first. This chapter will deal with this convergence of the thesis' problem.

Confining the thesis' problem is done in three steps:
*First*, a selection is made from the 14 challenges described in chapter three of this report. Some of the challenges will be disregarded and some will be taken into the next phase of the thesis.
*Second*, it is determined what the location of the building will be. In this way wind and soil conditions are fixed, which further converges the thesis' problem.
*Third*, the form of the building and the geometry and dimensions of the structure will be determined. This is achieved by describing a benchmark skyscraper. Defining a benchmark skyscraper makes it possible to compare the challenges' limits to each other.

## 5.1 Step 1 – Selecting the Challenges

### 5.1.1 System Boundaries

In chapter three of this report 14 challenges which influence the height of a building were listed. In order to be able to make a first selection from this list, we will first define the *system boundaries* of the thesis' problem.
What can be learned from chapter three is that the surroundings of a building can seriously influence its height. Without taking the surroundings of a building into consideration, greater heights can be achieved. Since the goal of this thesis is to determine the ultimate limits to the height of a skyscraper, only those challenges which are imposed by the building itself will be taken into consideration. The

challenges imposed on the building by its surroundings, are disregarded. This means that the system boundaries of the thesis problem lie just outside the facade of the building, just above its roof and just underneath its foundation. This is illustrated in figure 5.1. The upper picture shows a skyscraper in the middle of a city. The building and its surroundings are constantly interacting with each other. In the bottom picture, system boundaries are applied. In this case the surroundings of the building have no longer influence on the skyscraper, and visa versa.

Interesting to see is that this approach is comparable with what is happening in Dubai at this very moment. The Burj Dubai is the centrepiece of a larger town planning. The buildings, roads, parks, etc. surrounding the tower will be constructed after the Burj is completed. This means that they will not interfere with the construction activities. Furthermore, these surroundings are designed in such a way that they do not impose a limit to the height of the Burj Dubai. The surroundings are somewhat inferior to the building. Obviously, this "ideal situation" is not present at most building sites.

Based on the chosen system boundaries, some of the challenges can be disregarded. These challenges are external influences and therefore, lie outside the system boundaries of the thesis' problem:

- Influence on the surroundings
- Organising the building site
- Sufficient economical support
- Stability of the real-estate market

(a) A skyscraper is affected by its surroundings


(b) When system boundaries are applied, the skyscraper is not any longer affected by its surroundings

**Figure 5.1:** *System boundaries*

### 5.1.2 Further Disregarded Challenges

Besides the challenges mentioned above, three others will be disregarded as well. These are: earthquake hazards, terrorist attacks and the fire safety of the building.

**Earthquakes**
Earthquakes only occur in a small area of the globe, only at places located near the borders of the tectonic plates (described in chapter 3). At most locations in the world, no or only moderate earthquake loads have to be taken into account. At these locations the forces induced by the wind are often larger than the earthquake loads which have to be taken into account. This means that the wind loads are governing. Therefore, earthquake hazards are not considered in the remaining part of this thesis.

**Terrorist attacks**
The terrorist attacks on the World Trade Centre on the 11th of September 2001, caused a general concern about the safety of high-rise buildings. However, the construction of high-rise buildings has not only continued ever since, but has even experienced a booming development. This can perhaps be best explained when we compare the use of skyscrapers with the use of airplanes. Despite numerous attacks and hijacks in the past and the assumption that airplanes are still a potential target for terrorists, we are still willing to use airplanes for going on holiday or going on a business trip. We simply accept there is a risk involved as soon as we are getting on an airplane. This same reasoning might be valid for the use of high-rise buildings. Obviously, like on airports, precautionary measures have to be taken in order to minimise the risk of an attack. Terrorist attacks will not be considered in the remaining part of this thesis.

**Fire safety**
Fire safety has been discussed in section 3.10. In this section, five measures were given to improve the fire safety of a building:

1. The use of fire compartments. Both horizontally as vertically.
2. The use of incombustible materials for shielding of combustible materials.
3. The application of a fire detection system inside the building.
4. The installation of a sprinkler system inside the building.
5. The installation of firefighting installations and equipment.

These measures are independent on the height of the building. Therefore, this challenge does not impose a limit to the height of a building and can be disregarded in the rest of the thesis. An aspect related to fire safety and which is dependent on the building's height, is the time it takes to entirely evacuate the building in case of a fire. A taller building will take more time to be fully evacuated. This important aspect of fire safety is captured by the challenge "Evacuating the Building".

### 5.1.3 Remaining Challenges

In the previous two sections has been decided to disregard seven challenges out of the original list of 14 challenges. This means that seven challenges are still remaining.

Because the challenge *"Slenderness"* is so closely related to the challenges "Load-bearing structure", "Comfort" and "Economical Feasibility", it is chosen to categorise the slenderness of the building under these challenges. Therefore, "Slenderness" will not be considered any longer as a separate challenge.

So finally, six challenges will be taken into the next phase of the thesis. These remaining challenges are listed below.

- Foundation
- Load-bearing structure
- Comfort
- Vertical transportation
- Evacuating the building
- Economical feasibility

## 5.2 Step 2 – Locating the Building

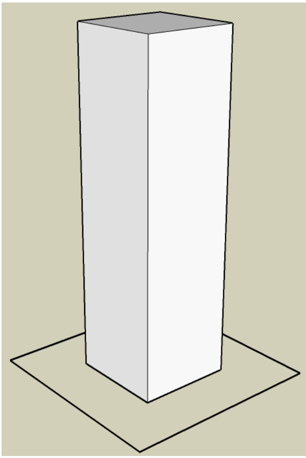
The design of a building is highly influenced by its location on the globe. Wind and soil conditions vary from place to place. This means that at some locations the conditions are more ideal for building an ultra-tall building than at others. We will hold on to one location in order to make a fair comparison between the different challenges.

For now, we will only consider the situation in The Netherlands. This means that the Dutch wind and soil conditions are considered.
Additionally, only the building codes which are valid in The Netherlands (National codes or European codes) will be taken into account. If, on certain aspects, these codes fail to give an explanation, building codes from other nations will be used.

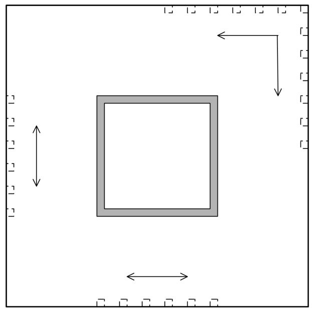
## 5.3 Step 3 – Describing a Benchmark Skyscraper

To further confine the thesis' problem, a benchmark skyscraper will be described. The benchmark skyscraper has a rectangular shaped footprint, which is continuous over the full length of the building. The structure will not have a spire on top and there are no basement levels constructed underneath the building (figure 5.2a).
When we look to the super-tall skyscrapers which have already been constructed, their slenderness ratios lie around 1:8. At the time of writing, the Petronas Towers in Kuala Lumpur are the most slender super-tall skyscrapers. The slenderness ratio of these towers is 1:8.6 . A similar slenderness ratio is given to the benchmark skyscraper.
The average storey height inside the building will be 3.5 metres.
For a skyscraper it is most efficient to have its load-bearing structure on the perimeter of the building.

Due to the large internal lever arm, the structure is capable to accommodate a large overturning moment at its base. This efficient way of arranging the load-bearing structure is also applied to the benchmark skyscraper. This means that the load-bearing columns are placed in the facade of the building, creating a so-called "Tube Structure" (figure 5.2b).
The shear forces are accommodated by a concrete core in the centre of the building.



(a) Form of the bench-mark skyscraper



(b) The floor plan of the bench-mark skyscraper

**Figure 5.2:** *The bench-mark skyscraper*

For the facade of the benchmark skyscraper, a wall-to-window ratio of 38% is assumed. This ratio is based on the article written by Vartiainen [49].
The most important aspect when choosing a wall-to-window ratio is the yearly average daylight availability (DA) in percentage of the lightning requirement during office hours. In his article Vartiainen [49] states that the optimum can be found in a wall-to-window ratio of 38%, with a DA of 88%. We consider a facade section of 10 metres long and one storey high (3.5 metres). The facade's parapet will be 1.40 metres wide and the centre-to-centre distance of the columns is 2.5 metres (figure 5.3). A simple calculation tells us that the width of the columns in the facades of the building is restricted to 920 mm. This will be rounded down to columns of 900 $\times$ 900 mm.

Now that the thesis' problem is confined, we can make a first computation to determine the limits to the benchmark skyscraper for each of the remaining challenges. This will be described in the following chapter.

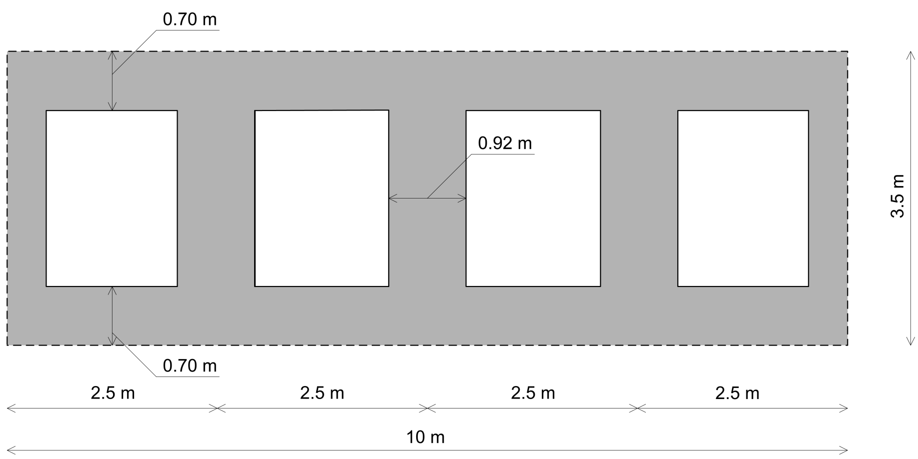



**Figure 5.3:** *Facade section of the bench-mark skyscraper*

# Chapter 6

# Limitations to the Benchmark Skyscraper

In the previous chapter the thesis' problem has been converged into a more manageable problem by ruling out eight challenges out of the original list of 14 challenges. Furthermore, a benchmark skyscraper was described which fixed the building's location, geometry and dimensions.
In order to find out which of the remaining six challenges is the most limiting to the height of a skyscraper, the ultimate limit to each of the remaining challenges is determined. This will result in a list of six limits. Based on this list we can conclude which of these challenges is the most restricting to the skyscraper's height.

Before we can start calculating the limits to each of the challenges, the loads which are acting on the benchmark skyscraper have to be determined. This will be presented in the first section. Subsequently, each of the challenges will be considered in a separate section.

## 6.1 Loads Acting on the Structure

The benchmark skyscraper will be exposed to both vertical and horizontal loads. To determine these loads, the Dutch building code NEN 6702 and the European building code NEN-EN 1991-1-4 are used.

### 6.1.1 Vertical loads

When a skyscraper gets taller, it is not surprising that the vertical forces acting on the structure increase as well. This subsection lists the vertical loads which are acting on the benchmark structure.

**Floor structure**
We assume that the floor structure of the building is a 280 mm thick concrete slab. On top of this slab will be a finishing layer which has a thickness of 50 mm. This brings the total floor thickness at 330 mm.
The concrete density is assumed to be 24 kN/m$^3$. This means that the representative value of the

deadweight of the floor is:

$$q_{g;rep} = 0.33 \times 24 \approx 8.0\mathrm{kN/m^2} \tag{6.1}$$

For partition walls, ceilings, pipes, etc. an additional 1.5 kN/$m^2$ is added to the deadweight. This gives a total deadweight of:

$$q_{g;rep;tot} = 8.0 + 1.5 = 9.5\mathrm{kN/m^2} \tag{6.2}$$

A skyscraper can have various user functions, e.g. residential, hotel, office or commercial function. The Dutch building code NEN 6702 gives different values for the variable load for each of these functions. The representative value for a residential and hotel function is 1.75 kN/$m^2$. The representative values for office and commercial functions are respectively 2.5 kN/$m^2$ and 4.0 kN/$m^2$. This means that the most unfavourable situation would be when the floors of the building fulfill a commercial function. The Dutch building code states that in high-rise buildings, only one floor should be assumed to be fully loaded. The load on the other floors may be multiplied with a factor $\psi$. For buildings with a commercial function, the factor $\psi$ is 0.5 . The representative value for the variable floor load is:

$$q_{p;rep} = 0.5 \times 4.0 = 2.0\mathrm{kN/m^2} \tag{6.3}$$

This gives us a total representative floor load of:

$$q_{rep;floor} = 9.5 + 2.0 = 11.5\mathrm{kN/m^2} \tag{6.4}$$

According to NEN 6702, the following safety factors have to be applied: $\gamma_g = 1.2$ for dead loads and $\gamma_p = 1.5$ for variable loads. This means that the total design floor load will be[1]:

$$q_{d;floor} = 1.2 \times 9.5 + 1.5 \times 2.0 = 14.3\mathrm{kN/m^2} \tag{6.5}$$

**Columns**

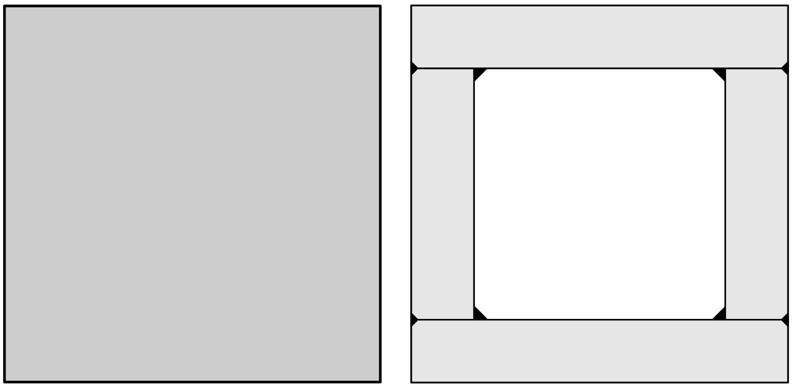
The columns in the facade of the building have outer dimensions of 900 $\times$ 900 mm. The columns can be constructed out of concrete or steel box sections. Figure 6.1 depicts the configuration of both columns.

The density of the concrete is again assumed to be 24 kN/$m^3$. In case the columns are constructed out of concrete, this means that the deadweight of the columns (design value) is:

$$q_{d;column} = 1.2 \times (0.9 \times 0.9 \times 24) = 23.4\mathrm{kN/m} \tag{6.6}$$

We assume that the steel columns are constructed out of 150 mm thick steel plates. The outer dimensions of the columns are 900 $\times$ 900 mm, which brings the surface of the column's cross-section on

[1] Lateron, it turned out that this floor load is slightly overestimated. Assuming a smaller floor load will result in smaller vertical loads and consequently in higher limits.



(a) Concrete column (b) Steel column

**Figure 6.1:** *Column configurations*

0.45 $\mathrm{m}^2$. The density of steel is 78.5 kN/m$^3$. In case the columns are constructed out of steel, the deadweight of the columns (design value) is:

$$q_{d;column} = 1.2 \times (0.45 \times 78.5) = 42.4\mathrm{kN/m} \tag{6.7}$$

**Facade**
The facade of the building has an estimated deadweight of 1.0 kN/m$^2$. This gives a design load of:

$$q_{d;facade} = 1.2 \times 1.0 = 1.2\mathrm{kN/m}^2 \tag{6.8}$$

Each facade column supports a 2.5 metres wide strip of facade. The facade load per column per metre building is therefore equal to:

$$q_{d;facade} = 1.2 \times 2.5 = 3.0\mathrm{kN/m} \tag{6.9}$$

### 6.1.2 Lateral loads

The lateral loads on a skyscraper are primarily caused by the wind and will vary along with the velocity of the wind, i.e. a higher wind velocity results in larger lateral loads.
The Dutch building code NEN 6702 is only usable to determine the lateral loads on buildings with a height below 150 metres. Since it is likely that our benchmark skyscraper will exceed this height, the NEN 6702 can not be used. Therefore the European building code EN 1991-1-4 is used. This code describes the lateral loads for buildings up to heights of 200 metres, but can also be used for estimating the wind loads on buildings exceeding 200 metres.
In combination with EN 1991-1-4, the national annex for the Netherlands is used.

The procedure to determine the wind forces acting on a structure, is extensive. To keep this chapter readable, the entire procedure is precisely described in appendix B. Now, only the outlines of this calculation procedure will be discussed.

The wind force which is acting on a structure is determined by the following equation:

$$q_w = c_s c_d \times c_f \times q_p(z_e) \times b \tag{6.10}$$

With:
$c_s c_d$ = structural factor, which has a minimum value of 0.85
$c_f$ = force coefficient
$q_p(z_e)$ = peak velocity pressure at reference height $\mathrm{z_e}$
$b$ = width of the building

For the determination of the structural factor and the force coefficient is referred to appendix B.
The wind force is primarily determined by the peak velocity pressure, $q_p(z_e)$. This pressure is changing over the height of the building. In the Eurocode a schematisation for this wind profile is given. This profile is given in figure 6.2. In order to keep the calculations for the benchmark skyscraper simple, the wind profile is assumed to be constant over the entire height of the building. In other words, we assume that $q_p(z_e) = q_p(h)$.

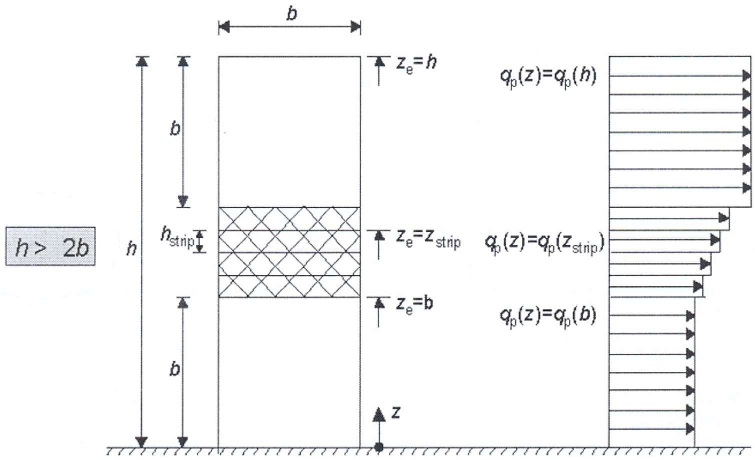



**Figure 6.2:** *Wind profile along the height of the building according to the EN 1991-1-4*

The peak velocity pressure is calculated using equation 6.11.

$$q_p(z) = (1 + 7l_v(z)) \times \frac{1}{2} \times \rho \times v_m^2(z) \tag{6.11}$$

With:
$l_v(z)$ = turbulence intensity
$\rho$ = air density, for which 1.25 kg/m$^3$ is given in the national annex
$v_m^2(z)$ = mean wind velocity

For the determination of the turbulence intensity is referred to appendix B.
The mean wind velocity, $v_m^2(z)$, is calculated by the following formula:

$$v_m(z) = c_r(z) \times c_0(z) \times v_b \tag{6.12}$$

With:
$c_r(z)$ = roughness factor
$c_0(z)$ = orography factor, taken as 1.0 unless otherwise specified
$v_b$ = basic wind velocity

For the determination of the roughness and orography factor is referred to appendix B.
The basic wind velocity is given by equation 6.13.

$$v_b = c_{dir} \times c_{season} \times v_{b,0} \tag{6.13}$$

With:
$c_{dir}$ = directional factor, 1.0 recommended
$c_{season}$ = season factor, for the Netherlands equal to 1.0
$v_{b,0}$ = fundamental value of basic wind velocity

In the national annex of the Netherlands is determined that the fundamental value of basic wind velocity is equal to 27 m/s.

Note that the by equation 6.10 determined wind forces, are representative forces. The results have to be multiplied with the corresponding safety factor to obtain the design value of the wind force, $q_{w;d}$. The safety factor which has to be applied is 1.5 .

Now, this calculation procedure is applied to the benchmark skyscraper. Because the magnitude of the lateral wind forces acting on the structure are dependent on the height of the building, no fixed value can be given for the benchmark skyscraper, since its height is variable. By implementing the calculation procedure in spreadsheet software, the relation between the height of the building and the lateral wind forces acting on the building, can be depicted in a graph (figure 6.3).
Figures 6.4 and 6.5 give the relation between the height above ground level and respectively the mean wind velocity, $v_m^2(z)$, and the peak velocity pressure, $q_p(z_e)$).

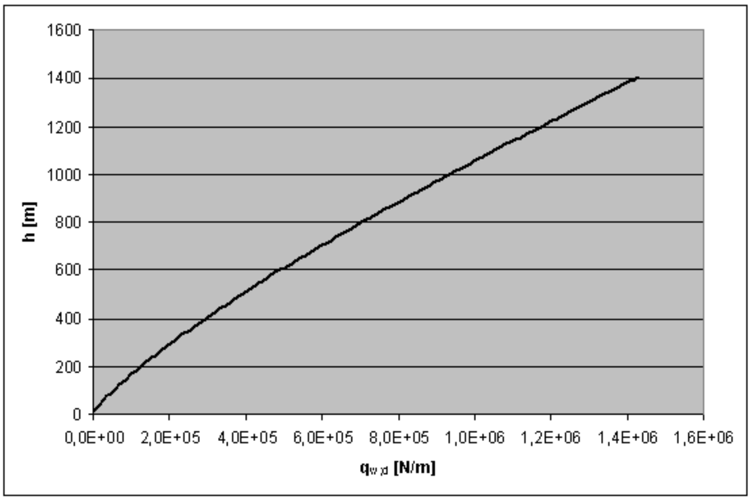



**Figure 6.3:** *Design value of lateral force due to wind*

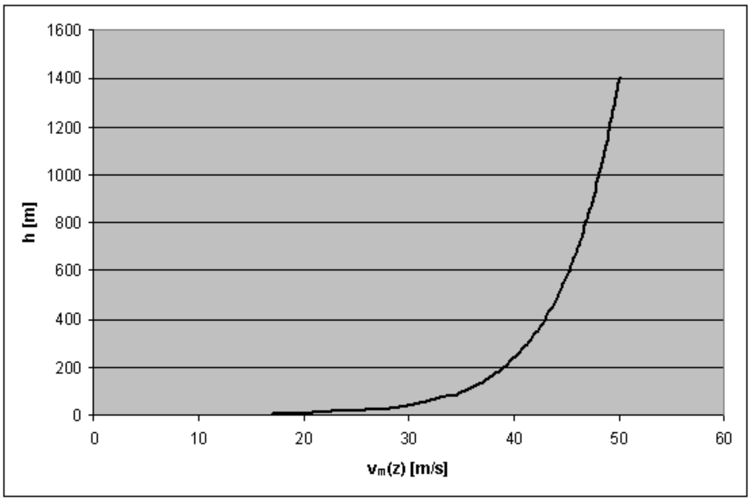



**Figure 6.4:** *The mean wind velocity along the height of a building*

## 6.2 Load-bearing structure

According to the description given in chapter 5, the load-bearing structure of the benchmark skyscraper is a tube structure with a concrete core in the centre of the building.
Important is that in these first calculations we assume that the core of the building solely accommodates shear forces and vertical loads. The core will not contribute in accommodating the overturning moment at the base of the structure. This means that the overturning moment is only resisted by the tube structure in the facade of the building.

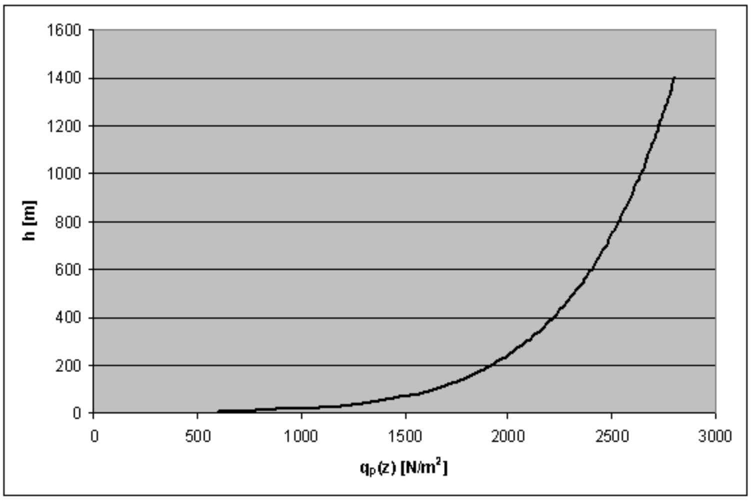



**Figure 6.5:** *The peak velocity pressure along the height of a building*

### 6.2.1 Limit

The floor loads of the benchmark skyscraper, are carried by both the columns in the facade of the building and the concrete core in the centre of the building. In this case we assume that the width of the core, is half the width of the skyscraper. This means that the storey depth *d* on either side of the core, is a quarter of the total width *b* of the skyscraper ($d = \frac{1}{4}b$).
Figure 6.6 displays how the floor loads are divided over the structural elements. Surfaces 1, 2, 3 and 4 are carried by the columns in the facade of the building. Surface 5 is carried by the core of the building.

In chapter 5 was decided that the centre-to-centre distance in between the facade columns will be 2.5 metres. Given this, the floor load which is introduced in each column at each floor level can be determined by using equation 6.14.

$$F_{N;d;floor} = 2.5 \times \frac{1}{2}d \times q_{d;floor} = 2.5 \times \frac{1}{8}b \times q_{d;floor} \tag{6.14}$$

With:
$d$ = storey depth
$b$ = width of the building
$q_{d;floor}$ = design floor load, which was calculated at 14.3 $\mathrm{kN/m^2}$

$\mathrm{F_{N;d;floor}}$ multiplied by the number of floors in the building gives the total vertical force in the columns due to the floor loads.
The weight of the building's facade and the dead weight of the structure's columns should be added to the load of the floors to calculate the total force in each column due to vertical loads:

$$F_{N;d} = F_{N;d;floor} \times m + q_{d;column} \times h + q_{d;facade} \times h \tag{6.15}$$

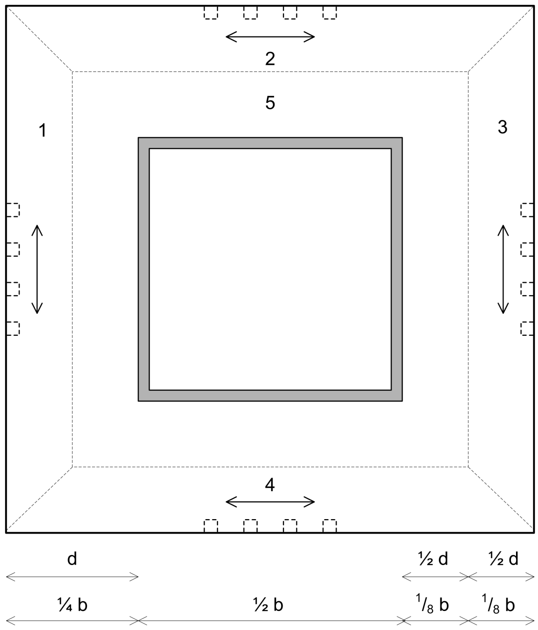



**Figure 6.6:** *Floor loads divided over the structural elements*

With:
$m$ = number of floors
$F_{N;d;floor}$ = design value floor load, see equation 6.14
$q_{d;column}$ = design value dead weight column
$q_{d;facade}$ = design value dead weight facade

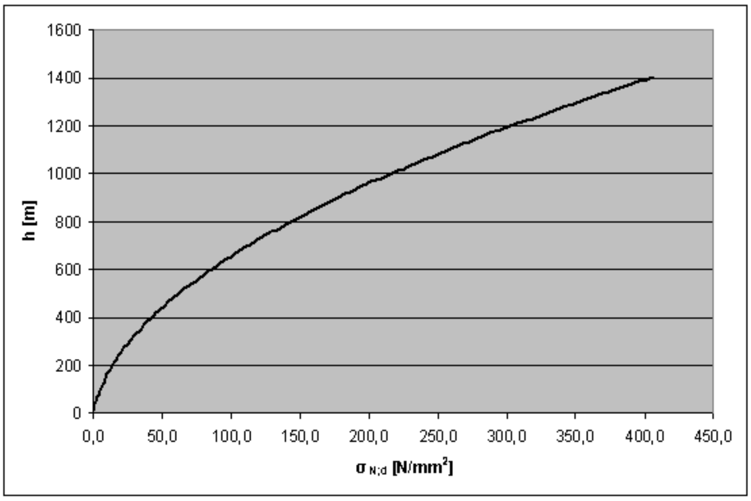
The vertical stresses in the column due to the vertical loads, can be calculated by dividing the total force $\mathrm{F_{N;d}}$ by the surface of the column's cross-section.

$$\sigma_{N;d} = \frac{F_{N;d}}{A_{column}} \tag{6.16}$$

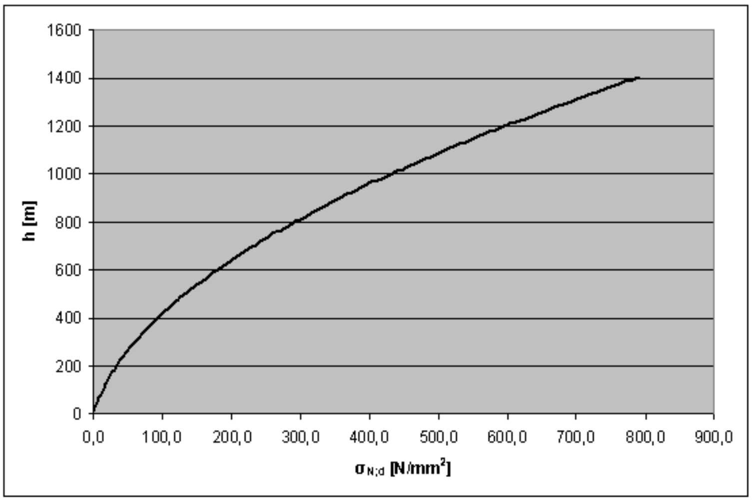
In figure 6.7 the stresses in the columns due to the vertical loads, are plotted against the building's height. This is done for both the materials concrete and steel. Note that the graphs do not show a linear curve. This can be explained by the fact that together with the building's height, the width of the building increases. A wider building means an increase of floor loads.

The lateral loads acting on the structure, will introduce vertical forces into the columns of the skyscraper as well. In order to determine these forces, first the moment at the base of the skyscraper has to be determined. This moment is known as the overturning moment. The overturning moment due to the wind forces acting on the structure can be calculated using the following formula:

$$M_d = \frac{1}{2} \times q_{w;d} \times h^2 \tag{6.17}$$




(a) Concrete column




(b) Steel column

**Figure 6.7:** *Stresses in the columns due to vertical loads*

With:
$q_{w;d}$ = design value of the wind force
$h$ = height of the building

Next, the stresses in the columns can be calculated by using equation 6.18:

$$\sigma_{M;d} = \frac{M_d \times z_i}{I} \tag{6.18}$$

With:
$M_d$ = Overturning moment at the base of the building (equation 6.17)
$z_i$ = Distance between the median of the cross-section and the outermost fibre
$I$ = Moment of inertia of the building's cross-section

The overturning moment is accommodated by the building's tube structure. Because this tube is a closely spaced frame structure, it will behave less rigid than a closed tube. In the third chapter of this report this phenomenon has been described as the "shear-lag effect". This means that, due to the lack of stiffness, the forces are not equally distributed over the columns in the flanges and the webs of the tube. Because the corners of the tube structure behave much stiffer, they will attract more forces. The shear-lag effect is taken into account by multiplying $\sigma_{M;d}$ with a factor $\gamma$. To determine the size of this factor, we will take a closer look into how the stresses are divided over the flanges of the tube structure. The stress distribution in the flanges follows a curved line (see figure 6.8). The assumption is made that the middle columns in the flanges are only loaded up to 50% of their maximum load-bearing capacity[2]. The curved stress distribution in the flanges can be approximated by a linear curve declining from 100% on the edges of the flange to 50% in the middle of the flange (see figure 6.8). From the approximated linear curve follows that the average overall strength of the flanges can be approximated at 75% of the flanges' maximum load-bearing capacity (see figure 6.9). Therefore, the multiplication factor $\gamma$ will be:

$$\gamma = \frac{1}{0.75} = 1.33 \tag{6.19}$$

In graph 6.10 the stresses in the columns due to the lateral loads are plotted against the building's height. In these results the shear-lag effect is taken into account, i.e. the values of $\sigma_{M;d}$ are multiplied by the multiplication factor $\gamma$.

Finally, the total compressive stress in the columns is obtained by adding up $\sigma_{N;d}$ and $\sigma_{M;d}$. In figure 6.11 the total compressive stress in the facade columns is plotted against the height of the building.

$$\sigma_d = \sigma_{N;d} + \sigma_{M;d} \tag{6.20}$$

When the facade columns are constructed out of concrete, we assume that high-strength concrete will be used. In the Eurocode EN 1992-1-1 the concrete with the highest strength mentioned, is C90/105. The

[2]This is purely an assumption. This percentage depends on the ratio between the dimensions of the columns and beams in the tube structure. An exact estimation can be made using computer modelling software.

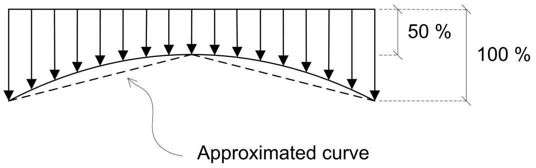



**Figure 6.8:** *Stress curve in the flanges of the tube structure*

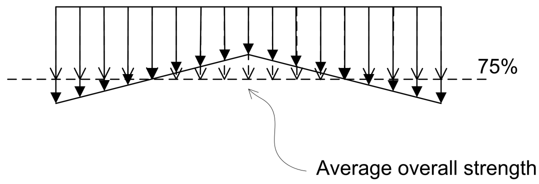
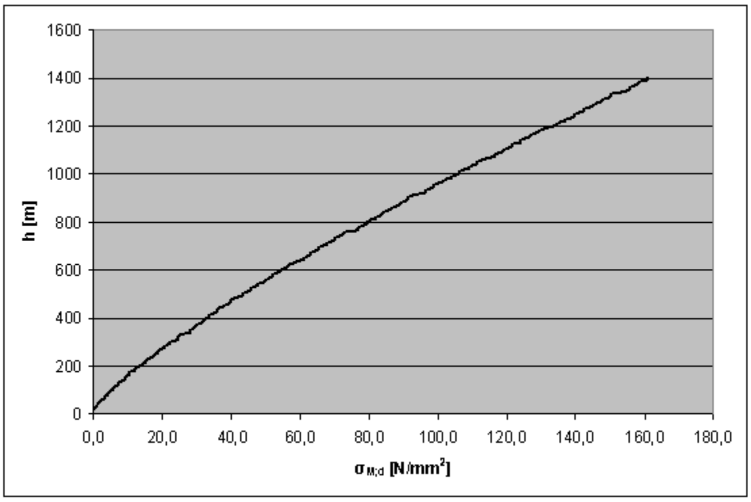
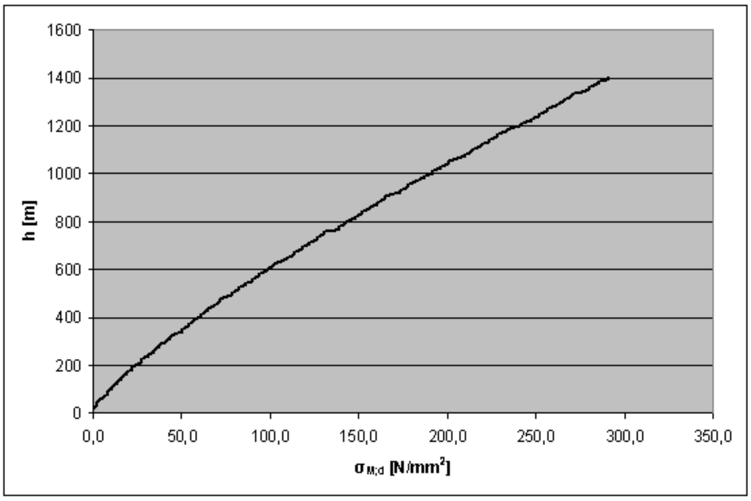



**Figure 6.9:** *The average overall strength of the flange*

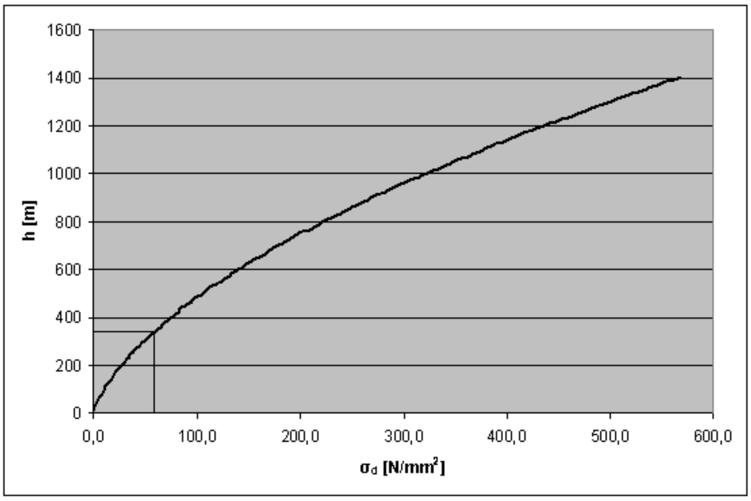



(a) Concrete column

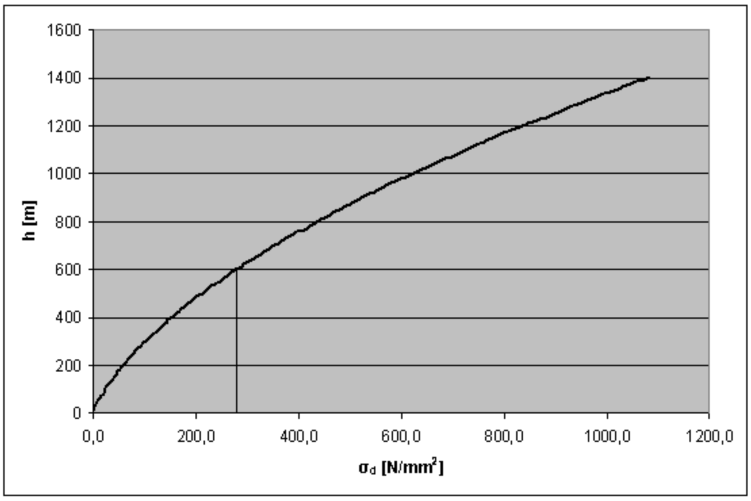



(b) Steel column

**Figure 6.10:** *Stresses in the columns due to horizontal loads*




(a) Concrete column




(b) Steel column

**Figure 6.11:** *Stresses in the columns due to both the vertical and horizontal loads*

characteristic compressive cylinder strength, $\mathrm{f_{ck}}$, of this concrete is 90 N/mm$^2$. The design compressive strength can be calculated using the following formula:

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} \tag{6.21}$$

With:
$\alpha_{cc}$ = coefficient taking account of long term effects and of unfavourable effects resulting from the way the load is applied. A value of 1.0 is recommended
$f_{ck}$ = characteristic compressive cylinder strength
$\gamma_c$ = partial safety factor for concrete

The partial safety factor $\mathrm{f_{ck}}$ is equal to 1.5. The design compressive strength of C90/105 concrete is:

$$f_{cd} = \frac{1.0 \times 90}{1.5} = 60\mathrm{N/mm^2} \tag{6.22}$$

From figure 6.11a can be read that the benchmark skyscraper with concrete facade columns, can have a maximum height of approximately 344 metres.

For the steel columns, steel grade S355 is examined. This steel has a yield stress of 355 N/mm$^2$. However, due to the thick steel plates which are used, a reduced yield stress of 275 N/mm$^2$ has to be taken into account. From figure 6.11b can be read that the benchmark skyscraper with steel facade columns, can have a maximum height of approximately 600 metres.

### 6.2.2 Conclusion

From this section can be concluded that the limit to the load-bearing structure of a skyscraper largely depends on the used building material. When steel is used for the load-bearing structure, the limit height is much higher then when concrete is used for the load-bearing structure of the skyscraper.
Considering the calculations in this section, we can conclude that the load-bearing structure of the benchmark skyscraper limits the height of a skyscraper to *344 metres* when constructed out of concrete and to *600 metres* when constructed out of steel.

## 6.3 Foundation

In the third chapter of this report a number of possible foundation types have been discussed. The possible foundation types are: shallow foundations, deep foundations and pile-raft foundations. To ensure a good load-bearing capacity of the building's foundation, pile foundations are generally used in high-rise construction. Therefore we will only consider a pile foundation to determine the height limit of the benchmark skyscraper with regard to its foundation. There are two different kinds of pile foundations: a driven pile foundation and a bored pile foundation.

*A driven pile foundation* consists of prefabricated piles which are driven into the ground. Both concrete and steel piles can be used. The largest prefabricated concrete piles on the market are rectangular piles of 500 × 500 mm. The maximum length of these piles is 35 metres. There is a wide variety of steel piles on the market. The pile which is most used, is the tube foundation pile. These steel tubes can have diameters up to 3000 mm, with a wall thickness of 25 mm. The length of the piles is often limited by the handling capacity and transportation. However, piles with lengths up to 65 metres can be produced. Driven piles are rarely used in the foundation of super-tall skyscrapers. The main reason for this is the so-called "group effect". This means that that when the first pile is driven into the ground, the soil compacts, making it harder to hammer the adjacent piles into the subsoil. As a consequence of this, the centre-to-centre distance in between the driven piles can not be too small. If for example a column needs to be supported by multiple piles, a thick concrete base plate is needed to spread the vertical column loads equally over the piles. A thick base plate adds extra weight to the skyscraper's foundation and is expensive to build.
*A bored pile foundation* is constructed by first excavating the soil and subsequently placing the piles. Because the piles do not need to be driven into the ground, the group effect is not an issue.
Often, bored piles are constructed out of concrete. The concrete can be simply poured into the hole which has been excavated. Concrete bored piles with a diameter of 2 metres are not an exception. The length of the piles can be as much as 100 metres. Because of these enormous dimensions, one single bored pile is often capable of accommodating the vertical loads from one column. In contrary to a driven pile foundation, a thick base plate is not needed any longer.

Because a bored pile foundation is most promising, we will only consider this foundation type in order to determine the benchmark skyscraper's limit with regard to its foundation.
The limits will be determined, using the Dutch national building code. The code which considers the load-bearing capacity of pile foundations is the *NEN 6743*.

### 6.3.1 Limit

The load-bearing capacity of the foundation is mainly determined by the local soil conditions. Even within a small country as the Netherlands, these local soil conditions can differ a lot. In order to be able to compute a limit to the building's foundation, we will first describe a representative subsoil for the Netherlands. The major economic centres of the Netherlands are located in the western part of the country. In this area, the first soil layer which is capable of supporting a structure, is a sand layer at approximately 20 metres below the ground's surface. Above this sand layer the soil consists of clay and peat. These are layers with a poor load-bearing capacity.

In NEN 6743, the maximum load-bearing capacity of a single foundation pile is calculated by the following formula:

$$F_{r;max;d} = \frac{\xi_{M;N} \times F_{r;max;i}}{\gamma_{m;b}} \tag{6.23}$$

With:
$\xi_{M;N}$ = factor determined in table 6.1
$F_{r;max;i}$ = the representative load-bearing capacity of one single pile, in kN

$\gamma_{m;b}$ = factor for the ground conditions. Established in table 3 of the Dutch building code NEN 6740 (figure 6.2).

| M | N | | | | | | |
|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 7 | ≥10 |
| 1 of 2 [a] | 0,72 | 0,76 | 0,77 | 0,78 | 0,78 | 0,79 | 0,80 |
| 3 ≤ M ≤ 6 | 0,76 | 0,80 | 0,82 | 0,83 | 0,84 | 0,84 | 0,85 |
| 7 ≤ M ≤ 9 | 0,78 | 0,84 | 0,86 | 0,87 | 0,88 | 0,89 | 0,90 |
| ≥ 10 | 0,79 | 0,85 | 0,87 | 0,88 | 0,89 | 0,89 | 0,91 |

**Table 6.1:** *Factor $\xi_{M;N}$, depending on the number of piles M, and on the number of drilling tests N*

The representative load-bearing capacity of the pile, $F_{r;max;i}$, is determined by the following formula:

$$F_{r;max;i} = F_{r;max;punt;i} + F_{r;max;schacht;i} \tag{6.24}$$

With:
$F_{r;max;punt;i}$ = representative load-bearing capacity of the pile head
$F_{r;max;schacht;i}$ = representative load-bearing capacity of the pile shaft

The load-bearing capacity of the pile head and the pile shaft are determined by equations 6.25 and 6.26.

$$F_{r;max;punt;i} = A_{punt} \times p_{r;max;punt;i} \tag{6.25}$$

$$F_{r;max;schacht;i} = O_{s;\Delta L;gem} \times p_{r;max;schacht;z;i} \tag{6.26}$$

With:
$A_{punt}$ = surface pile head, in m$^2$
$p_{r;max;punt;i}$ = maximum pile head resistance in kN/m$^2$
$O_{s;\Delta L;gem}$ = the average circumference of the pile shaft
$p_{r;max;schacht;z;i}$ = maximum pile shaft friction, in kN/m$^2$

$p_{r;max;schacht;z;i}$ is given by the following formula:

$$p_{r;max;schacht;z;i} = \alpha_s \times q_{c;z;a} \tag{6.27}$$

With:
$\alpha_s$ = factor depending on the soil type
$q_{c;z;a}$ = cone resistance, in MPa

| type geotechnische constructie | partiële materiaalfactor ($\gamma_m$) | | grenstoestanden: uiterste, 1A gunstig [1] | grenstoestanden: uiterste, 1B gunstig [1] | grenstoestanden: uiterste, 1A ongunstig [2] | grenstoestanden: uiterste, 1B ongunstig [2] | grenstoestanden: bruikbaarheid, 2 |
|---|---|---|---|---|---|---|---|
| alle geotechnische constructies | $\gamma_{m;g}$ | volumieke massa van grond | 1,1 | 1,1 | 1 | 1 | 1 |
| Funderingen van gebouwen: | | | | | | | |
| op palen (op druk belast) | $\gamma_{m;b1}$ | zonder onderzoek [3] | 1,4 | 1,4 | 1 | 1 | 1 |
| | $\gamma_{m;b2}$ | met proefbelasting | 1,25 | 1,25 | 1 | 1 | 1 |
| | $\gamma_{m;b3}$ | voor proefbelaste palen en ankers | 1,15 | 1,15 | 1 | 1 | 1 |
| | $\gamma_{m;b4}$ | uit sonderingen | 1,25 | 1,25 | 1 | 1 | 1 |
| op staal | $\gamma_{m;\phi'}$ | tangent van de hoek van inwendige wrijving | 1,15 | 1,15 | 1 | 1 | 1 |
| | $\gamma_{m;c1}$ | cohesie (draagvermogen van funderingen) | 1,6 | 1,6 | 1 | 1 | 1 |
| | $\gamma_{m;fundr}$ | ongedraineerde schuifsterkte | 1,35 | 1,35 | 1 | 1 | 1 |
| andere constructies: palen/anker (trek, horizontaal belast) | $\gamma_{m;b1}$ | zonder onderzoek | 1,4 | 1,4 | 1 | 1 | 1 |
| | $\gamma_{m;b2}$ | met proefbelasting | 1,4 | 1,1 | 1 | 1 | 1 |
| | $\gamma_{m;b3}$ | voor proefbelaste palen en ankers | 1,25 | 1,25 | 1 | 1 | 1 |
| | $\gamma_{m;b4}$ | uit sonderingen | 1,4 | 1,4 | 1 | 1 | 1 |
| | $\gamma_{m;\phi'}$ | tangent van de hoek van inwendige wrijving | 1,2 | 1,2 | 1 | 1 | 1 |
| | $\gamma_{m;c2}$ | cohesie (gronddrukken evenwicht taluds) | 1,5 | 1,5 | 1 | 1 | 1 |
| | $\gamma_{m;fundr}$ | ongedraineerde schuifsterkte | 1,5 | 1,5 | 1 | 1 | 1 |
| vervormingen: | $\gamma_{m;Cc}, \gamma_{m;C\alpha}, \gamma_{m;Csw}$ | | 1 | 1 | 0,8 | 0,8 | 1 |
| | $\gamma_{m;Cp}\ \gamma_{m;Cs}$ | | 1,3 | 1,3 | 1 | 1 | 1 |
| | $\gamma_{m;E}$ | | 1,3 | 1,3 | 1 | 1 | 1 |

1) "Gunstig": als een verhoging van de waarde van de betreffende parameter tot een gunstiger resultaat leidt.
2) "Ongunstig": als een verhoging van de waarde van de betreffende parameter tot een ongunstiger resultaat leidt.
3) Hierbij moet gebruik zijn gemaakt van slagdiagrammen die bij het heien zijn opgenomen, of anders geldt, voor toestanden 1A en 1B (gunstig): $\gamma_{m;b1} = 1,8$.

**Table 6.2:** *Table 3 from the Dutch building code NEN 6740*

It was assumed that the load-bearing layer lies at 20 metres below the ground's surface. The concrete bored pile will have a diameter of 2 metres and a length of a 100 metres. This means that the foundation pile protrudes 80 metres into load-bearing sand layer. At this depth, the pile head resistance can assumed to be fully developed. According to the NEN 6743 the maximum pile head resistance which may be taken into account for a sand layer, is 15 MPa. This is equal to 15000 kN/m$^2$.

Using equation 6.25, the maximum load-bearing capacity of the pile head can be determined:

$$F_{r;max;punt;i} = \frac{\pi}{4} \times 2^2 \times 15000 \approx 47000\text{kN} \tag{6.28}$$

However, the load-bearing capacity of the pile head of a bored pile is largely dependent from its settlements. In contrary to a driven pile, a bored pile is likely to show large settlements. During the construction phase, the initial stresses in the soil are removed due to the excavation of the soil. After construction, the bored pile will settle until these soil stresses are restored. Due to these settlements a reduced load-bearing capacity of the pile head has to be taken into account.
We assume that the allowable settlement of the pile head $w_{punt;i}$ is 100 mm. The diameter $D_{eq}$ of the pile is 2000 mm. From figure 6 in NEN 6743 (see figure 6.12) follows that a reduction factor of 0.66 should be applied on the load-bearing capacity of the pile head. So, the maximum load-bearing capacity of the pile head is approximately 31000 kN.

Also the pile's shaft friction will reduce due to the settlements. This reduction factor can be determined by using figure 7 from the NEN 6743 (see figure 6.13). However, from this figure can be learnt that with a settlement $w_{punt;i}$ of 100 mm, this reduction factor can be neglected.
The piles's shaft friction is calculated by using equations 6.26 and 6.27. The cone resistance $q_{c;z;a}$ fore moderate packed sand is 15 MPa (15000 kN/m$^2$). For bored concrete piles $\alpha_s$ has a value of 0.006. Equation 6.27 gives:

$$p_{r;max;schacht;z;i} = 0.006 \times 15000 = 90\text{kN/m}^2 \tag{6.29}$$

According to equation 6.26 the maximum load-bearing capacity of the pile shaft is:

$$F_{r;max;schacht;i} = \pi \times 2 \times 80 \times 90 \approx 45000\text{kN} \tag{6.30}$$

It should be noted that possible downdrag is not taken into consideration. This phenomenon occurs when the soil around the pile shows larger settlements than the pile itself. Due to the friction between the soil and the pile shaft, an additional load has to be accommodated by the piles.

Now both the representative load-bearing capacity of the pile head and the pile shaft are determined, the total representative load-bearing capacity can be determined by using equation 6.24:

$$F_{r;max;i} = 31000 + 45000 = 76000\text{kN} \tag{6.31}$$

The design value of the load-bearing capacity of the piles can now be determined using equation 6.23. For a large construction project $\xi_{M;N}$ can be taken 0.80 (table 6.1). From table 6.2 follows that for

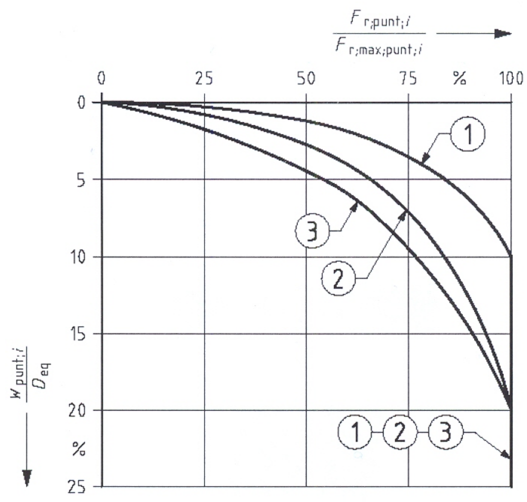



**Figure 6.12:** *Figure 6 from NEN 6743, graph 3 applies to bored piles*

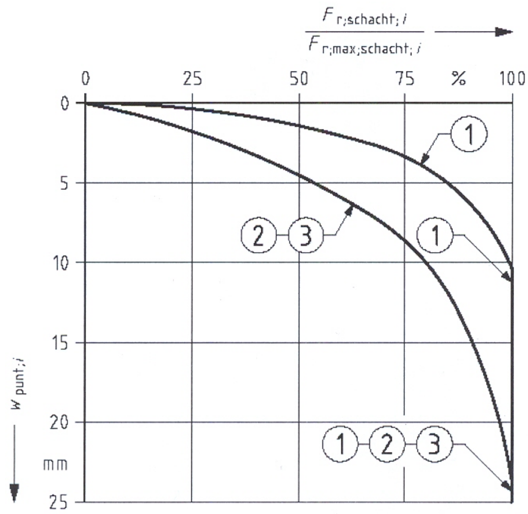



**Figure 6.13:** *Figure 7 from NEN 6743, graph 3 applies to bored piles*

$\gamma_{m;b}$ a value of 1.25 should be taken.

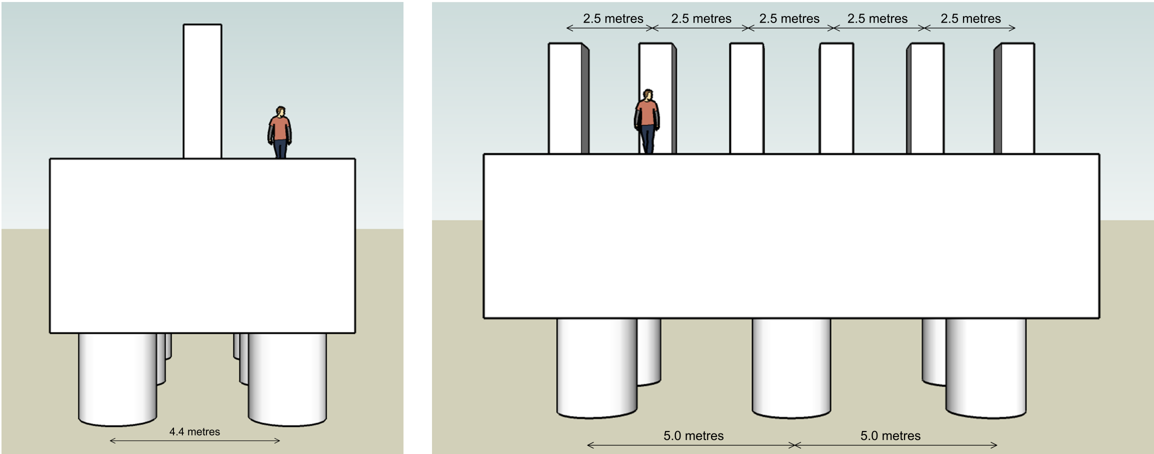
$$F_{r;max;d} = \frac{0.80 \times 76000}{1.25} \approx 48500\text{kN} \tag{6.32}$$

When we compare this with the Burj Dubai, which is currently under construction in the United Arab Emirates, we can conclude that this is a plausible figure. In chapter 4 of this report has already been mentioned that the load-bearing capacity of each pile in the foundation of the Burj equals to 3,000 tons. 3,000 Tons equals to 30,000 kN. Because the bored piles in the benchmark's foundation are much longer and wider, an additional 18500 kN can be accommodated.

According to the Dutch building code, the minimum centre-to-centre distance of bored piles is 2.2 times the diameter of the pile. This means that the centre-to-centre of the bored piles should be at least 4.4 metres. Since the columns in the benchmark skyscraper have a centre-to-centre distance of 2.5 metres, two columns are supported by one row of piles. One row of piles contains two piles. So, in the end this means that the vertical loads from one column are accommodated by one pile. This is illustrated in figure 6.14.
This means that the total compressive stress in the concrete columns can increase to a value of:

$$F_{r;max;d} = \frac{48500 \times 10^3}{900 \times 900} \approx 60\text{N/mm}^2 \tag{6.33}$$

This value is equal to the maximum compressive strength of the concrete. From figure 6.11 follows that this corresponds to a limit of 344 metres.




(a)

(b)

**Figure 6.14:** *Layout of a part of the benchmark's foundation*

### 6.3.2 Conclusion

Considering this section, we can conclude that a bored pile foundation is the best solution for the benchmark skyscraper's foundation. This foundation type limits the height of the benchmark skyscraper to approximately *344 metres*.

## 6.4 Economical Feasibility

In chapter 3 of this report has been mentioned that the ratio "Gross Floor Area" to "Lettable Floor Area" is an important indicator to asses the economical feasibility of a building. Therefore this ratio will be used to find the limit to our skyscraper with regard to its economical feasibility.

An aspect which determines whether or not floor area is suitable for letting out, is the entry of daylight into a building. A source of natural light is very much appreciated by the users of the building. This wish for sufficient daylight restricts the depth of the floors in the building and thereby influences its economical feasibility.

In the Netherlands, there are two different regulations with respect to the entry of daylight into buildings.

**ARBO-wet:** The first regulation which discusses this aspect, is the Dutch Health and Safety at Work Act, "ARBO-wet". This act regards a room as a working environment, when the people in the room are working there for more than two hours a day. In section 6.4 of the act is written that the window surface of a room should be at least $\frac{1}{20}$ of the total floor surface of the room.

**Bouwbesluit:** The second act which considers this subject, is the Dutch Building Regulation (Jong and Pothuis [20]), "Bouwbesluit". In this regulation a table is given (table 3.133) in which the minimum required window surface in a room is described for several functions of use. The requirement is both expressed in a minimum value and in a minimum % of the total floor surface of the room. An abstract of this table is given below (table 6.3).

| Function of use | | Limiting value [m$^2$] | [%] |
|---|---|---|---|
| 1 | Residential function | 0.5 | 10 |
| 2 | Gathering function | - | - |
| 3 | Health Care function | 0.5 | 5 |
| 4 | Industrial function | - | - |
| 5 | Office function | 0.5 | 2.5 |
| 6 | Hotel function | 0.35 | 7 |
| 7 | Educational function | 0.5 | 5 |
| 8 | Sport function | - | - |
| 9 | Commercial function | - | - |

**Table 6.3:** *Abstract of table 3.133 from the Dutch Building Regulations*

The minimum values given in table 6.3 are so-called equivalent values. The equivalent window surface can be calculated by using the following formula, which is given in the Dutch building code NEN 2057:

$$A_{e;i} = A_{d;i} \times C_{b;i} \times C_{u;i} \tag{6.34}$$

With:
$A_{e;i}$ = the equivalent window surface
$A_{d;i}$ = the actual window surface
$C_{b;i}$ = degree of obstruction
$C_{u;i}$ = exterior reduction of daylight entry

Both the requirements in the Dutch Health and Safety at Work Act as in the Dutch building regulations do not impose strict limits to the depth of a building. Therefore a limit will be set, based on the current European building practise. In Europe, offices are generally 7.2–9.0 metres deep.
To keep a high-rise project economically feasible, real estate agents aim for a nett floor surface in between 80 and 70%. This means that the stairwells, elevator shafts and the shafts for the vertical transportation of building services, may not claim more than 20–30% of the gross floor surface of the building. Since in this chapter the economical feasibility is assessed by regarding the entry of daylight into the building, it means that daylight should be able to penetrate into 80–70% of the total floor surface inside the building.

### 6.4.1 Limit

An office building does not only consists of office space, also other facilities are required, e.g. corridors, meeting rooms, toilets, kitchenette, archives, post room, photocopying room, etc.. All these facilities

do not necessarily have to have a direct source of daylight. Therefore, they can be positioned in the middle of the building. Estimated is that these facilities need an additional floor strip of 3.6 metres wide. Added up to the required depth for the offices, this means that the storey depth on each side of the core is 10.8–12.6 metres.
For the slenderness of the benchmark skyscraper a ratio of 8.6 was settled. This ratio is used to determine the limit to the height of the benchmark skyscraper with regard to the economical feasibility.

For the strict approach we have the following parameters:

- Storey depth: 10.8 metres
- Nett floor surface: 80%
- Slenderness ratio: 1:8.6

Figure 6.15 shows the geometry of the benchmark's cross-section. When variable *c* increases, the total width of the building *w* will increase as well since the storey depth *d* is fixed at 10.8 metres. However, when variable c is increased, the gross-nett floor ratio is deteriorating. This is illustrated by figure 6.16 in which the gross-nett floor ratio is plotted against variable c.

From figure 6.16 can be derived that in order to fulfill the demand of 80% nett floor surface, the core of the building can have the maximum dimensions of 17 $\times$ 17 metres. In this case, the total width of the building will be:

$$17 + (2 \times 10.8) = 38.6\text{m} \tag{6.35}$$

With the slenderness ratio taken into account, this means the maximum height of the building is:

$$38.6 \times 8.6 = 332\text{m} \tag{6.36}$$

For the favourable approach we have the following parameters:

- Storey depth: 12.6 metres
- Nett floor surface: 70%
- Slenderness ratio: 1:8.6

From figure 6.16 can be derived that the core of the building has the maximum dimensions of 30 $\times$ 30 metres. In this case, the total width of the building will be:

$$30 + (2 \times 12.6) = 55.2\text{m} \tag{6.37}$$

With the slenderness ratio taken into account, this means the maximum height of the building is:

$$55.2 \times 8.6 = 475\text{m} \tag{6.38}$$

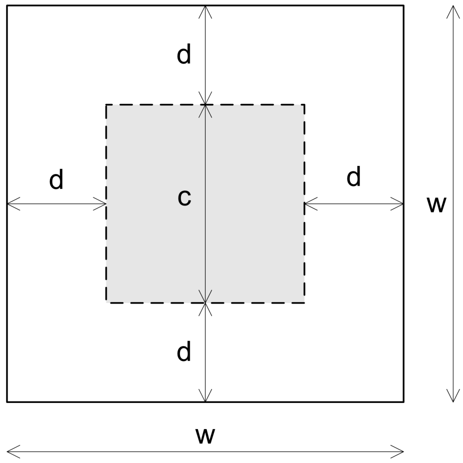



**Figure 6.15:** *Geometry cross-section benchmark skyscraper*

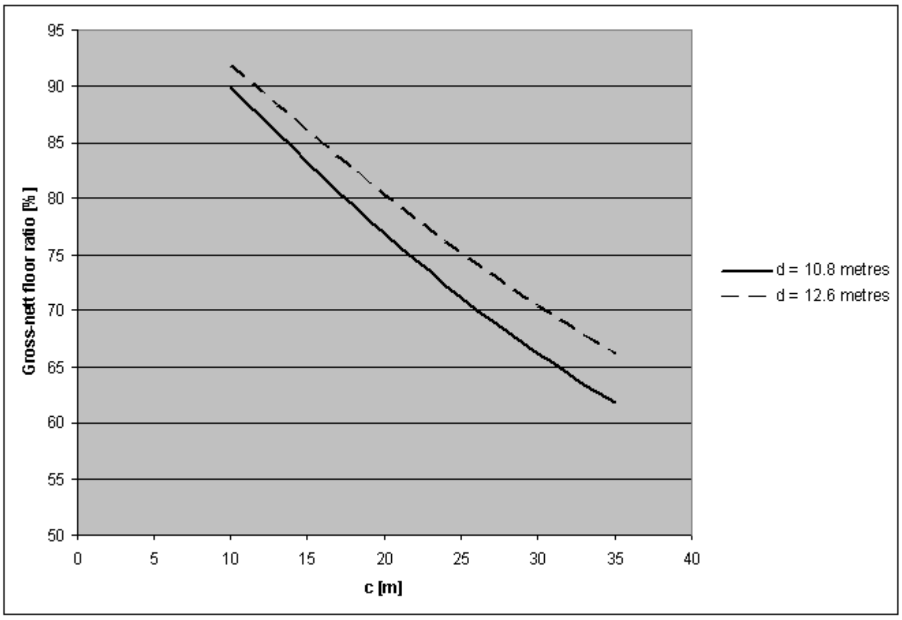



**Figure 6.16:** *Nett floor surface in relation to the width of the buildings core*

### 6.4.2 Conclusion

From these simple calculations follows that the limit, imposed by the economical feasibility of the building, lies in the range of *332 metres to 475 metres*.

## 6.5 Comfort

Generally, the accelerations experienced by the occupants of a building, are accepted as the criterion to evaluate the building's comfort. Therefore, the national building codes impose limits to the acceleration of structures. In the Netherlands, there are two operative regulations with regard to the comfort criterion.

**NEN 6702:** The first valid regulation is the Dutch Building Code NEN 6702. The limiting peak acceleration values which are applied by this regulation, are given in a graph (figure 6.17). The graph contains two curves. Curve 1 applies to floors with an industrial, office or educational function. Curve 2 applies to floors with a residential, gathering, health care, hotel, sport or commercial function.

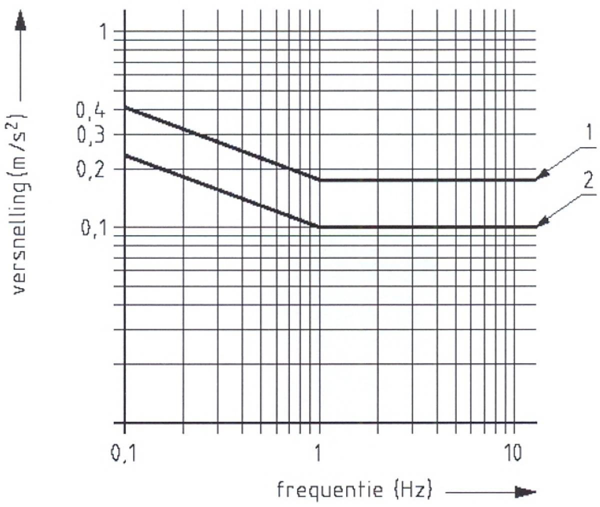



**Figure 6.17:** *Acceleration limits as described in NEN 6702*

**ISO 6897:** The second valid regulation is the International Standard ISO 6897. In this standard the limiting values are given for frequencies between 0,063 to 1 Hz. The values given in table 6.4, apply to buildings used for general purposes, during the worst 10 consecutive minutes of a wind storm with a return period of at least 5 years.
In the international standard, the acceleration limits are given in root-mean-square (rms). This notation is also used in the national building codes of the US and Canada. By converting the rms values in ISO 6897 into peak acceleration values, we can compare them to the values which are given by the Dutch building code.

A relation between rms acceleration and peak acceleration can be established by introducing a peak factor $g_p$ (Oosterhout [35]). This factor is described by the following formula:

$$g_p = \sqrt{2ln\left(f_e T\right)} + \frac{1}{sqrt3} \times \frac{1}{\sqrt{2ln\left(f_e T\right)}} \tag{6.39}$$

With:
$f_e$ = eigenfrequency of the structure in Hz
$T$ = time in s

By multiplying the rms acceleration values with this factor, peak acceleration values are obtained. Since ISO 6897 considers a storm of 10 consecutive minutes, T should be taken 600 s.
In the third column of table 6.4, the to peak acceleration converted values are given.

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

**Table 6.4:** *Acceleration limits as described in ISO 6897*

When we compare the acceleration limits of the NEN 6702 with those given in the ISO 6897, we can conclude that the targets described in the international standard are slightly more strict. Therefore these targets will be taken into account when determining the limit to the height of the benchmark skyscraper with respect to the occupant's comfort.

### 6.5.1 Limit

To exactly calculate the building's accelerations, already in the design phase of the project, requires a series of complex calculations. Therefore the designers of super-tall skyscrapers often test a scale-model of the building in a wind-tunnel instead. Trough wind-tunnel testing, reliable data can be obtained. A drawback of determining the accelerations of a building through wind-tunnel testing is that when the structure accelerates more than the targets given in the building codes, it may be difficult to adapt the design of the building since this is often already in far developed stadium.

Performing an elaborate calculation to determine the accelerations of the benchmark skyscraper, lies beyond the scope of this thesis. Therefore, only a simple calculation method will used to determine the limit to the benchmark skyscraper with regard to the comfort criterion.

The method which is used, is described in the Dutch building code NEN 6702.
In this code, the acceleration of the building is calculated by the following formula:

$$a = 1.6 \times \left( \frac{\rho_2 \times p_{w;1} \times C_t \times b_m}{\rho_l} \right) \tag{6.40}$$

With:
$\rho_2$ = factor dependent from the eigenfrequency and damping of the building
$p_{w;1}$ = variation in thrust on the building in N/m
$C_t$ = summation of the wind factors for thrust and suction
$b_m$ = the average width of the building in m
$\rho_l$ = the mass of the building per metre building height in kg/m

$p_{w;1}$ is given by equation 6.41.

$$p_{w;1} = 100 \times ln \left( \frac{h}{0.2} \right) \tag{6.41}$$

With:
$h$ = height of the building in m

$\rho_2$ is given by equation 6.42.

$$\rho_2 = \sqrt{\frac{0.0344 \times f_e^{-\frac{2}{3}}}{D \times (1 + 0.12 \times f_e \times h) \times (1 + 0.20 \times f_e \times b_m)}} \tag{6.42}$$

With:
$f_e$ = eigenfrequency of the building in Hz
$D$ = damping factor
$h$ = height of the building in m
$b_m$ = the average width of the building in m

To calculate the eigenfrequency $\mathrm{f_e}$ of the building, NEN 6702 gives the following suggestion:

$$f_e = \sqrt{\frac{a}{\delta}} \tag{6.43}$$

With:
$a$ = value dependent on the distribution of the mass of the building in m/s$^2$ (see figure 6.18)

$\delta$ = deflection at the top of the building in m, when one considers the building as a cantilevered beam loaded in the direction of the vibrations by a distributed load equal to the deadweight of the building (see figure 6.19)

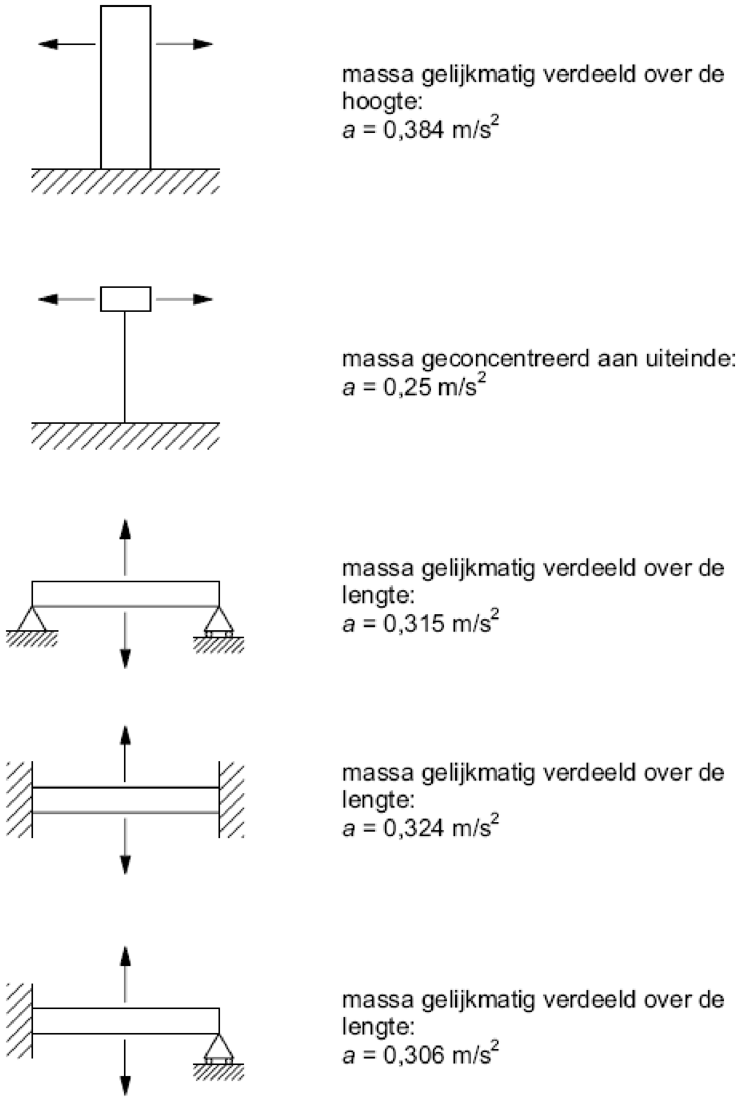



**Figure 6.18:** *Values a*

To further clarify the calculation procedure, we will briefly run through the equations and its variables: First, we will look at equation 6.43. From figure 6.18, which is given in NEN 6702, follows that the parameter *a* equals to 0.384 m/s$^2$ when considering a skyscraper.
The deflection $\delta$ is determined by using the equation which is given in figure 6.19. In this equation, variable q is the total deadweight of the structure. The total deadweight of the structure is the summation of the deadweight of the skyscraper's floors, load-bearing elements and facade:

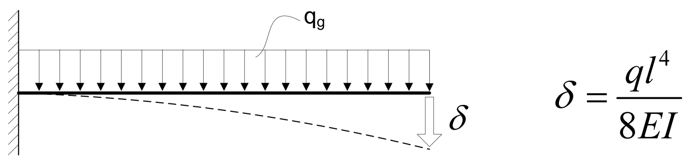



**Figure 6.19:** $\delta$

**Floors:**
The representative deadweight of the floor was 8.0 kN/m$^2$ (see beginning of this chapter). This gives a design value of $8.0 \times 1.2 = 9.6\mathrm{kN/m^2}$. The total floor surface of one storey is $55.2 \times 55.2 = 3047\mathrm{m^2}$. The total deadweight per storey is: $9.6 \times 3047 = 29252\mathrm{kN}$. Considering a storey height of 3.5 metres, the deadweight of the floors is $\frac{29252}{3.5} = 8358\mathrm{kN/m}$.

**Vertical load-bearing elements:**
The deadweight of a concrete column is 23.3 kN/m. On the perimeter of the 55.2 $\times$ 55.2 metre rectangular shaped cross-section are 88 columns. The total deadweight of the facade columns is therefore $88 \times 23.3 = 2050\mathrm{kN/m}$. It is assumed that the deadweight of the concrete core in the centre of the building is half of this, 1025 kN/m. This gives a total deadweight for the vertical load-bearing elements of: $2050 + 1025 = 3075\mathrm{kN/m}$.

**Facade:**
The deadweight of the facade elements is 1.2 kN/m$^2$. The perimeter of the building is $4 \times 55.2 = 220.8\mathrm{m}$. This means that the total deadweight of the facade elements is: $1.2 \times 220.8 = 265\mathrm{kN/m}$.

**Total:**
The total deadweight per metre building height is: $8358 + 3074 + 265 \approx 11.7 \times 10^3\mathrm{kN/m}$. For the sake of convenience, the same deadweight is assumed for the steel benchmark structure.

Variable I in the equation of figure 6.19 stands for the cross-section's moment of inertia. The benchmark's moments of inertia are: $3.8 \times 10^4\mathrm{m^4}$ when it is constructed out of concrete and $2.1 \times 10^4\mathrm{m^4}$ when it is constructed out of steel. However, again we have to take the shear-lag effect into account. The shear-lag effect will cause a reduction in the cross-section's moments of inertia. To take this effect into account, the moments of inertia are multiplied with a factor $\gamma$. Previously, this factor was estimated to have a value of 0.75. Therefore, the "new" values for the cross-section's moments of inertia become: $2.85 \times 10^4\mathrm{m^4}$ and $1.58 \times 10^4\mathrm{m^4}$.
The variable E in the equation of figure 6.19 is the Young's modulus of the used building material. The Young's modulus of uncracked C90/105 concrete is 44,000 $\mathrm{N/mm^2}$. When it is cracked the E-modulus decreases approximately with 70% to 13,200 $\mathrm{N/mm^2}$. The Young's modulus of steel is 210,000 $\mathrm{N/mm^2}$.
Variable l stands for the skyscraper's height. This parameter is variable.

Now, we will have a closer look to equation 6.42.
In the NEN6702 is written that, when the eigenfrequency of the structure is lower than 1.0 Hz, the damping factor D will have a value of 0.01. Tall skyscrapers usually have a eigenfrequency below the 1 Hz, so D will be set at 0.01.
In the previous section two limit values have been found for the width of the building. These limits were based on the required nett floor surface and the storey depth of the building. These values are 38.6

and 55.2 metres. Since we are striving for finding the ultimate limits, we will consider a benchmark skyscraper with a width of 55.2 metres. By doing this, we accept a gross-nett floor ratio of 70%. This means that the average building width $b_m$ will be 55.2 metres.
Again, the skyscraper's height *h* is the only parameter in the equation which is variable.

The actual along-wind acceleration is computed by using equation 6.40.
Variable $C_t$ is, according to NEN 6702, $0.8 + 0.4 = 1.2$.
Variable $p_{w;1}$ is computed in equation 6.41. Parameters $b_m$ and $\rho_l$ have already been discussed with respect to equations 6.42 and 6.43.

The only parameter which is left unknown, is the height of the building. By conducting an iterative calculation, the ultimate height to the building can be found. These iterations are made by using spreadsheet software Excel.
The result of this iteration is given in the table below (table 6.5).

| Material | Eigenfrequency | Building height |
|---|---|---|
| Concrete | 0,026 Hz | ± 620 m |
| Steel | 0,024 Hz | ± 1100 m |

**Table 6.5:** *Limits to the height according to NEN 6702*

The Dutch building code NEN 6702 is only considering the along-wind acceleration. However, due to the vortex shedding, a high-rise building is also showing cross-wind excitations. In case of a slender building, these across-wind accelerations generally exceed the along-wind accelerations. Predicting these across-wind accelerations has proven to be difficult. Consequently, the most accurate method to determine the across-wind motions is by wind tunnel testing.
Nevertheless, in the National Building Code of Canada (the NBCC) a tentative formula based on a wide range of turbulent boundary layer wind tunnel studies is given to determine the peak acceleration $a_w$ at the top of a skyscraper (Smith and Coull [45]):

$$a_w = f_e^2 \times g_p \left(b_m \times b_m\right)^{\frac{1}{2}} \left(\frac{a_r}{\rho g \sqrt{\beta}}\right) \tag{6.44}$$

With:
$f_e$ = eigenfrequency of the building in Hz
$g_p$ = peak factor, see equation 6.39
$b_m$ = the average width of the building in m
$\rho$ = average density of the building in kg/m$^3$
$g$ = acceleration due to gravity in m/s$^2$
$\beta$ = structural damping ratio for which the values are given in figure 6.20

$a_r$ is calculated by the following equation:

$$a_r = 0.0785 \times \left( \frac{v_h}{f_e \sqrt{b_m \times b_m}} \right)^{3.3} \tag{6.45}$$

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

A structural damping ratio of 0.040 is assumed for a concrete structure and a ratio of 0.025 is applied to a steel structure.
By conducting an iterative calculation, the limit to the height of the building can be found. The results are given in the table 6.6 below. From these results can indeed be concluded that the across-wind accelerations of a skyscraper are more problematic than the along-wind accelerations.

| Material | Eigenfrequency | Building height |
|---|---|---|
| Concrete | 0,069 Hz | ± 380 m |
| Steel | 0,109 Hz | ± 520 m |

**Table 6.6:** *Limits to the height according to the NBCC*

| Form of Construction | Damping Ratio $\beta$ Service | Ultimate |
|---|---|---|
| RC core, cantilever floor, light weight cladding | 0.016 | 0.022 |
| RC columns, slab floors, few internal walls | 0.030 | 0.070 |
| RC frame, few internal walls | 0.030 | 0.070 |
| RC frame, shear walls | 0.030 | 0.080 |
| RC shear core and columns, some internal walls | 0.040 | 0.120 |
| RC frame, some internal walls | 0.040 | 0.120 |
| RC all forms, many internal walls | 0.050 | 0.160 |
| Steel frame, no internal walls | 0.005 | 0.007 |
| Steel frame, few internal walls | 0.025 | 0.060 |
| Steel frame, many internal walls | 0.040 | 0.150 |

**Figure 6.20:** *Typical structural damping ratios*

## 6.5.2 Conclusion

Based on the calculations in this section we can conclude that the across-wind accelerations of a building are normative. The comfort demand limits the height of the benchmark skyscraper to *380 metres* when the load-bearing structure is constructed out of concrete and to *520 metres* when it is constructed out of steel.

## 6.6 Vertical Transportation

As mentioned in the third chapter of this report, people, goods and several building services all require vertical transportation in a skyscraper. The vertical transportation of people and goods is realised by using lift systems, while the transportation of building services requires wires, pipes and shafts.
The most efficient way of arranging the building services in a tall high-rise is by applying a service floor every 20 floors, from which the adjacent floors are serviced. When a skyscraper gets taller, more of these modules are added. Within each of these modules, the space required for the vertical shafts will not increase. However, the vertical shaft space required for the cables and pipes supplying each module, will. This space increases approximately linear to the height of the building. This is totally different when we look at the space which is required for lift systems. This space will increase somewhat exponentially compared to the building's height. The reason for this is that despite the increasing height of the building, the building's occupants demand that a travel to their destination floor will not take much longer than the travelling time in a medium-rise building. Since the acceleration of a lift cab is limited by the comfort requirements, more and more lifts are needed when the building gets taller.
An other difference between the transportation of people & goods on one hand and the transportation of building services on the other, are the allowable "intermediate stops" for the transported building services at each service floor. This means that the technology for vertically transporting building services 20 floors up, is not much different from the technology needed for transporting building services a 100, 150 or 200 floors up. However, the building's occupants will not agree on more than one intermediate stop, when travelling up the building. Therefore, lifts that can bridge larger distances are required when the height of a building increases. To make this possible, improved lift technologies are needed.
We can conclude that the lift systems inside a skyscraper are more restricting to the building's height than the vertical transportation of building services. Therefore, we will only consider the limits to the lift systems when searching for the ultimate limit to vertical transportation.

There are two possible ways to find the limit with regard to the vertical transportation of people and goods. The *first* is by looking what is technically possible, the *second* approach is by looking what is allowed by to the national building codes.

### 6.6.1 Technical Limit

There is a general consent that the technical limit to a conventional lift system lies around 700 to 750 metres. According to Johannes de Jong, member of the steering committee of the CTBUH and director in the KONE lift company, it should even be possible to build conventional lift systems up to travels of a 1000 metres. However, he also states: "A travel of 1000 metres requires about 60 tons of ropes to lift a 1600 kg pay load. This hardly makes sense anymore. Also the energy needed to move all this weight becomes very high." His personal opinion is that vertical travels exceeding 500 metres should be avoided, due to ecological reasons.
The technical limit of a conventional lift system is set at a 1000 metres.

### 6.6.2 Legal Limit

The national building codes which are applied in countries, do also impose a limit to the maximum possible vertical travel. In the Netherlands, the European building code *EN 81-72* is observed. In this code there are two regulations given which limit the length of a vertical travel:

**Article 5.2.2** A firefighter's lift shall serve every floor of the building.

**Article 5.2.4** The firefighter's lift shall reach the furthest floor from the fire service access level within 60 s, after the closing of the lift doors.

The result of *article 5.2.2* is that firemen have to be able to reach every floor in the building, using only one lift without any intermediate transfers. The height limit imposed by this article is therefore equal to the technical limit, a 1000 metres.

To examine the consequences of *article 5.2.4*, we consider the current fastest lift in the world. This lift is installed in the Taipei 101 in Taiwan and has a top speed of 60.6 km/h. In their article Li et al. [26] consider $1.0\ \mathrm{m/s^2}$ as the maximum allowable acceleration of a lift cab. A simple calculation can tell us which distance a lift with this top speed and acceleration can travel within 60 seconds:

$$60.6\mathrm{km/h} = 16.83\mathrm{m/s} \tag{6.46}$$

To accelerate from 0 to 16.83 m/s will take:

$$t = \frac{v}{a} = \frac{16.83}{1.0} = 16.83\mathrm{s} \tag{6.47}$$

It will take the same amount of time to decelerate the lift car to a halt. This means that $(16.83 \times 2 =)$ 33.66 seconds are "lost" by accelerating to top speed and decelerating from the top speed. 26.34 Seconds are remaining in which the lift cab can travel at top speed. Now the covered distance in 60 seconds can be determined.
Travelled distance during acceleration and deceleration of the lift cab:

$$s = 2 \times \left(0.5 \times a \times t^2\right) = 2 \times \left(0.5 \times 1.0 \times 16.83^2\right) = 283.25\mathrm{m} \tag{6.48}$$

Travelled distance during top speed of the lift cab:

$$s = v \times t = 16.83 \times 26.34 = 443.13\mathrm{m} \tag{6.49}$$

Total travelled distance:

$$s_{total} = 283.25 + 443.13 = 726.37\mathrm{m} \tag{6.50}$$

This means that the legal limit of a conventional lift system is set at a 726 metres.
To make longer vertical travels possible, lifts which can travel at higher speeds are necessary. However, there is always the problem that the acceleration and deceleration of the lift cab is limited by its occupants. This reduces the distance at which the elevator cab can travel at top speed.

### 6.6.3 Conclusion

With regard to the vertical transport, the limit imposed by the national building code is more restricting to the building's height than the limits imposed by the available lift technology; *720 metres*.

## 6.7 Evacuating the Building

To evacuate a skyscraper during an emergency, the buildings occupants rely primarily on the staircases inside the building. Only firefighter lifts can be used during an emergency and these lifts can only be used in an evacuation when they are operated by emergency personnel. Therefore, when determining the limit to the height of a skyscraper, this chapter will only focus on the use of staircases.

In their article Siikonen and Hakonen [44] state that in most situations, two staircases are a minimum requirement for high-rise buildings. The benchmark skyscraper will have three staircases inside its core (figure 6.21). In the article by Hakonen et al. [12] is stated that 1.2 metres wide stairs fulfil the requirements for most buildings.

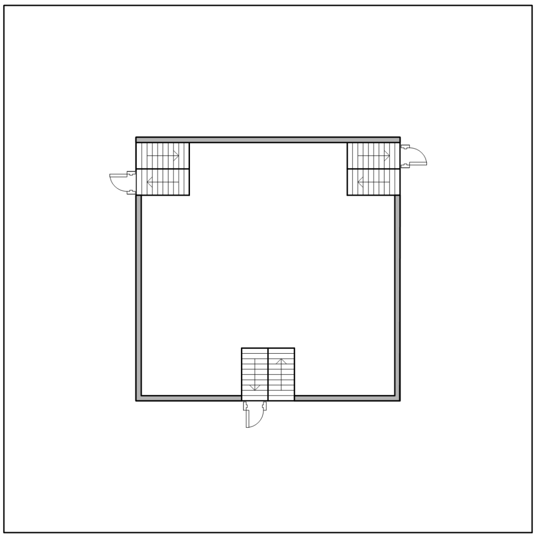


**Figure 6.21:** *Staircases inside the buildings core*

The population at each of the floors, is estimated at 120 persons. Since there are three staircases present in the building, this means that at each floor 40 persons will use one staircase.
The requirement for evacuating high-rise buildings, currently ranges from five to 30 minutes (Siikonen and Hakonen [44]). In order to find the ultimate limit, we assume a maximum evacuation time of 30 minutes, which is equal to 1800 seconds.

Nowadays, computer simulation software is used to model the evacuation of a building. Those models have proven to be a proper instrument in predicting the evacuation time for a building. However, in the past, numerous formulas have been developed to estimate the evacuation time of a building. Three of those formulas will be described in this section. Using these models, the maximum height of the building is determined.

**Muller (1966, 1968, 1969):** Muller gives the following formula:

$$t_{muller} = \frac{3h_G}{v} + \frac{P}{\left(\frac{b \times f_0}{0.6}\right)} \tag{6.51}$$

With:
$h_G$ = floor height in metres
$P$ = number of persons in building
$b$ = stair width in metres
$v$ = flow velocity down the stairs in m/s
$f_0$ = flow rate per unit stair width of 0.6 m

The variable P can be changed in n×p, where n is the number of floors in the building and p is the population at each floor which uses one staircase. Equation 6.51 can be rewritten as:

$$n \times p = \left(t - \frac{3h_G}{v}\right) \times \left(\frac{b \times f_0}{0.6}\right) \tag{6.52}$$

According to Muller the flow velocity is 0.3 m/s in a normal situation and 0.2 m/s in a congested situation. The flow rate is 0.5 persons per second per 0.6 m stair width. Put in equation 6.52 gives:

$$40n = \left(1800 - \frac{3 \times 3.50}{0.2}\right) \times \left(\frac{1.2 \times 0.5}{0.6}\right) \tag{6.53}$$

From this follows that n ≈ 44 floors. With a storey height of 3.5 metres, this means that the building will be: $3.5 \times 44 = 154$ metres.

**Pauls (1980, 1982, 1984):** Pauls gives the following formula:

$$t_{pauls} = 0.68 + 0.081 \times p^{0.73} \tag{6.54}$$

With:
$p$ = evacuation population per metre of effective stair width.

Equation 6.54 can be rewritten as:

$$p = \sqrt[0.73]{\frac{1}{0.081} \times (t - 0.68)} \tag{6.55}$$

Pauls considers the usable portion of a stair width to begin at a distance of 150 mm from a boundary wall on each side of the staircase. This means that the stairs in the benchmark skyscraper have an effective width of 0.9 metres.

The evacuation population $p$ is: $p = \frac{40n}{0.9} = 44\frac{4}{9}n$.
Put in 6.55 gives:

$$44\frac{4}{9}n = \sqrt[0.73]{\frac{1}{0.081} \times (30 - 0.68)} \tag{6.56}$$

From this follows that n ≈ 72 floors. With a storey height of 3.5 metres, this means that the building will be: $3.5 \times 72 = 252$ metres.

**Melink & Booth (1975):** Melink & Booth give two formulas. In one case, there is congestion on the stairs and the occupant flow is maximum all the time. In the other case, occupants can walk freely. The evacuation time is the maximum of these two (Siikonen and Hakonen [44]).

$$t_{1;melink\&booth} = \frac{nN}{F_s W} + t_s \tag{6.57}$$

$$t_{n;melink\&booth} = \frac{N}{F_s W} + nt_s \tag{6.58}$$

With:
$t_1$ = egress time (congestion)
$t_n$ = egress time (free walk)
$n$ = number of floors
$N$ = number per floor per floor per staircase
$F_s$ = nominal occupant flow on stairs (persons/m/s)
$W$ = width of the staircase
$t_s$ = walking time between adjacent floors (free walk)

Equations 6.57 and 6.58 can be rewritten as:

$$n = \frac{(t_1 - t_s) \times F_s W}{N} \tag{6.59}$$

$$n = \frac{t_n - \frac{N}{F_s W}}{t_s} \tag{6.60}$$

The effective width of the stairs is again 0.9 metres. A typical value for $t_s$ is 16 seconds and $F_s$ is 1.1 persons per metre per second. Put in equations 6.59 and 6.60 gives:

$$n = \frac{(1800 - 16) \times 1.1 \times 0.9}{40} = 44 \tag{6.61}$$

$$n = \frac{1800 - \frac{40}{1.1 \times 0.9}}{16} = 110 \tag{6.62}$$

From this follows that the egress time with congestion is normative. This equation gives the maximum height of the building with respect to evacuation. With a storey height of 3.5 metres, the building will be: $3.5 \times 44 = 154$ metres.

### 6.7.1 Conclusion

When we compare the outcomes of the three models, we see that both the models of Muller and Melink & Booth come up with the same limit of 154 metres. The model of Pauls gives a limit of 252 metres and is therefore not normative. The limit of the benchmark skyscraper with regard to the evacuation of its occupants, is *154 metres*.

## 6.8 Overall Conclusions

In table 6.7 below, an overview is given of the limits found in this chapter. A couple of conclusions can be drawn from this table:

- The found limits are lower than what has already been achieved in high-rise construction worldwide. They show discrepancy with reality. Especially the limits found for the evacuation of the benchmark building are eye-catching.
- There is not one challenge which clearly stands out from the rest, which means that it is not possible to point out one single challenge which limits the height of a skyscraper.

| Challenge | Concrete | Steel |
|---|---|---|
| Load-bearing Structure: | *344* | *700* |
| Foundation: | *344* | *344* |
| Economical Feasibility: | *475* | *475* |
| Comfort: | *380* | *520* |
| Vertical Transportation: | *720* | *720* |
| Evacuating the Building: | *154* | *154* |

**Table 6.7:** *Overview of the limits imposed on the benchmark skyscraper by each challenge (in metres)*

# Chapter 7

# Stretching the Limits

In the fifth chapter of this report, a list of six challenges was drawn up. Each of these challenges imposes a limit to the height of a skyscraper. In the previous chapter, a limit was established for each of these challenges by assuming a benchmark case.
However, the obtained results were unsatisfactory. This because the calculated limits, although in line with the height of buildings in the Netherlands, are much lower than what has already been achieved in high-rise construction worldwide. Instead of following the original idea of focussing on one of the challenges to determine the ultimate limit to high-rise, it would be more valuable to consider how the limits can be stretched for each of the challenges.

In this chapter we will take a close look into each of the six challenges and we will consider how each of the earlier found limits can be pushed skywards. Each section will start with a tree diagram which depicts possible measures to achieve this. In the text following the diagram, each of the measures will be further explained.
At first we will hold on to the fact that the skyscraper will be constructed in the Netherlands. In the last paragraph we will consider if the height of a skyscraper can be increased when it is located somewhere else on the globe.

## 7.1 Load-bearing Structure

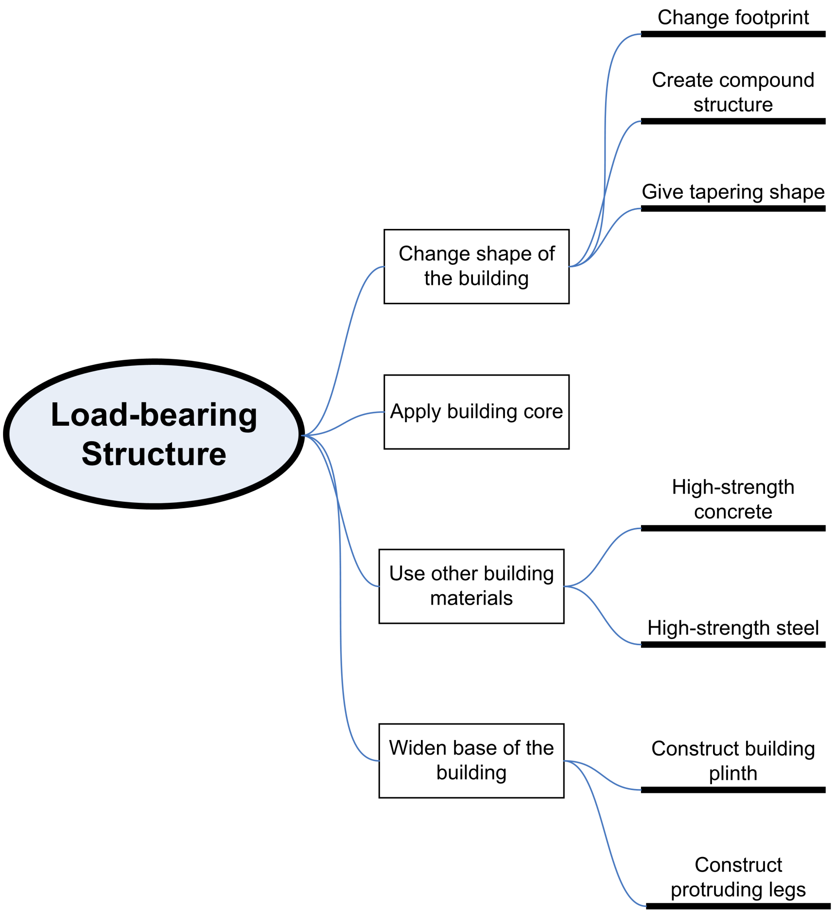



**Figure 7.1:** *Possibilities for stretching the limit of the load-bearing structure*

As depicted in the tree diagram above (figure 7.1), there are four possible approaches which can lead to a higher limit for the load-bearing structure of a skyscraper. These possible approaches are: change the shape of the building, widen the base of the skyscraper, apply a building core inside the building or use other building materials. Each of these approaches will be further described below.

### 7.1.1 Change the Shape of the Skyscraper

In chapter 5 of this report has already been claimed that it is most efficient to have the load-bearing structure in the facade of the building. Due to the large internal lever arm the building is capable of accommodating a larger overturning moment at its base. Bearing this in mind we can describe three different ways in which changing the shape of the building could help in stretching the limit:

**Change the footprint of the building:** When the building has a wider footprint, the internal lever arm will be larger which means a larger overturning moment can be resisted. A drawback of this measure is the decreasing nett-gross floor ratio (see also chapter 6). This could endanger the economical feasibility of the building.

**Create a compound structure:** When multiple slender towers are interconnected they form one structural entity. The internal lever arm of the combined towers is much larger than the the internal lever arm of the towers separate. This means that the compound structure is more efficient in accommodating the lateral forces.

**Give the building a tapering shape:** The previous two measures increased the internal lever arm of the structure, which makes it able to withstand larger lateral loads. Instead, this measure reduces the loads acting on the structure. Due to its tapering shape, the building's surface exposed to the wind is smaller at higher altitudes. This results in smaller lateral wind loads and a dramatic reduction in the overturning moment at the base of the structure.

### 7.1.2 Widen the Base on the Skyscraper

When the base of the building is widened, the larger internal lever arm enables the building to resist larger lateral forces. The difference with the measures described before is that the skyscraper is only widened at its base, where the overturning moment is the largest, and not over the entire length of the building. The new critical point will be at the transition zone between the widened base and the actual tower. Here, the load-bearing structure of the building should be able to resist the overturning moment. There are two ways in which the base of a skyscraper can be widened:

**Construct a plinth at the base of the tower:** A building plinth is a substructure of several stories high, which is much wider than the tower itself.

**Construct legs which protrude from the tower:** Protruding legs can increase the overturning moment which can be resisted by the skyscraper. These legs can be solely structural or can be a part of the building.

### 7.1.3 Apply a Building Core Inside the Building

It is assumed that the core inside the benchmark building is only capable of accommodating shear forces and vertical forces. The core does not participate in accommodating the overturning moment at the base of the structure. However, when stiff connections are made between the building's outer tube and its core, the core can participate in accommodating the overturning moment. This leads to reduced forces in the columns in the facade of the building.
The degree in which the building core is participating, is determined by the stiffness and rigidity of the core, i.e. the openness of the core.

### 7.1.4 Use Other Building Materials

The use of building materials with higher strengths can be an answer for stretching the limits of the load-bearing structure. For both concrete and steel, high-strength classes are available on the market. Due to the higher compressive strength of the material, columns with similar dimensions can accommodate larger compressive forces.

**High-strength concrete:** Concrete with strength class C90/105 was applied in the benchmark skyscraper. Nowadays, concrete mixtures with a cylindrical compressive strength of 125 MPa can be applied in the building industry.

**High-strength steel:** Steel with a yield stress of 650 N/mm$^2$ has already been applied in several civil engineering structures in Japan.

## 7.2 Foundation

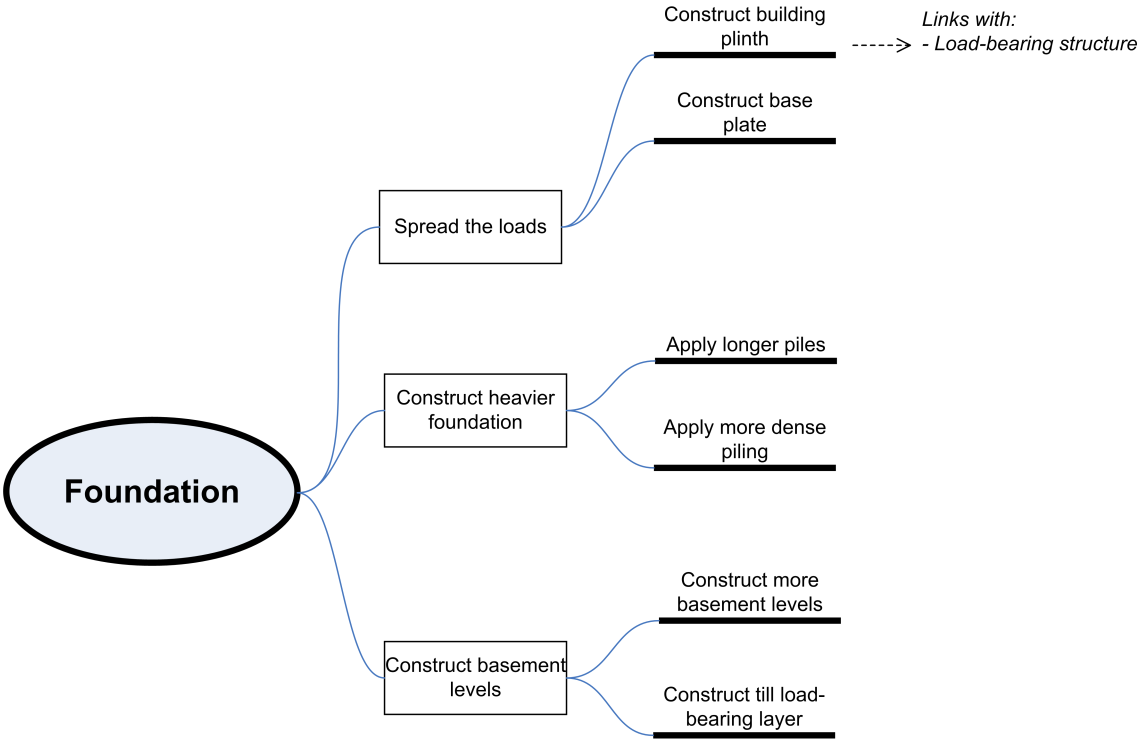



**Figure 7.2:** *Possibilities for stretching the limit of the foundation*

Figure 7.2 gives three possible approaches for stretching the limit for the foundation of a skyscraper: spread the loads, construct a heavier foundation, construct basement levels or build on stiffer soils. Each of these approaches will be further described below.

### 7.2.1 Spread the Loads

By spreading the loads from the building over a larger area, the pressure on the buildings foundation can be reduced. The spreading of the loads, can be established in two possible ways:

**Construct a plinth at the base of the tower:** This solution has already been suggested for stretching the limits of the load-bearing structure, by increasing the internal lever arm of the structure. However, a building plinth will also spread the forces from the tower over a larger area.

**Construct a base plate underneath the building:** A base plate is a thick concrete slab on top of the foundation's piles. A base plate distributes the forces more equally over the underlying piles. By applying a thick base plate, it is possible to load all the piles to their maximum load-bearing capacity. In this way a very efficient foundation can be created. A thicker base plate will result in a better distribution of the forces over the piles.

### 7.2.2 Construct a Heavier Foundation

A straightforward solution for stretching the limits of the building's foundation, is by constructing a heavier foundation. If a foundation is capable of resisting larger forces, a higher skyscraper can be constructed. A heavier foundation can be achieved in two ways:

**Apply longer piles:** Longer piles will generate more friction with the subsoil, which will increase their load-bearing capacity.

**Apply more dense piling:** When more piles are applied within the same area, the load-bearing capacity of the foundation will increase. Note that a very dense pile foundation is difficult to construct. Especially for a driven pile foundation. The driving of one pile will compact the soil around it, making it harder, if not impossible, to drive the neighbouring piles into the soil. If a dense pile foundation is required, a bored pile foundation will be a better option.

### 7.2.3 Construct Basement Levels

The construction of basement levels underneath the building, can make a taller building possible. To construct the basement levels, soil is excavated. This means that the weight of this soil is removed from the soil underneath it. In the initial situation this subsoil was capable of carrying the weight of the excavated soil. Since the total weight of the new basement levels (including life loads) is lower than the weight of the removed soil, the soil underneath the basement is capable of carrying some additional loads. This means that the weight of some of the floors above ground can be carried directly by the subsoil, without the use of any additional piles.
An other advantage of constructing basement levels, is the upward pressure of the groundwater. This upward pressure adds to the load-bearing capacity of the subsoil. However, it should be noted that this same upward groundwater pressure makes the construction of basement levels complex and costly. Additionally, fluctuation in the groundwater table causes a fluctuation in the upward groundwater pressure on the structure. This should be taken into account. Based on this, basement levels can lead to higher buildings in two ways:

**Construct more basement levels:** More basement levels, means a deeper building pit. More soil is excavated, so more weight is removed from the soil underneath it, which means that more floors above ground can be carried directly by the subsoil.
A deeper basement also means a larger upward groundwater pressure which adds to the load-bearing capacity of the soil.

**Construct basement levels till load-bearing layer:** In the benchmark case we assumed that the load-bearing layer in the soil was situated at 20 metres below ground level. If a the basement of the building is constructed to this depth, forces can be directly transferred to the load-bearing layer in the soil, without the need of piles. This can be compared with a pad foundation. Generally, pad foundations have a higher load-bearing capacity than pile foundations.

## 7.3 Comfort

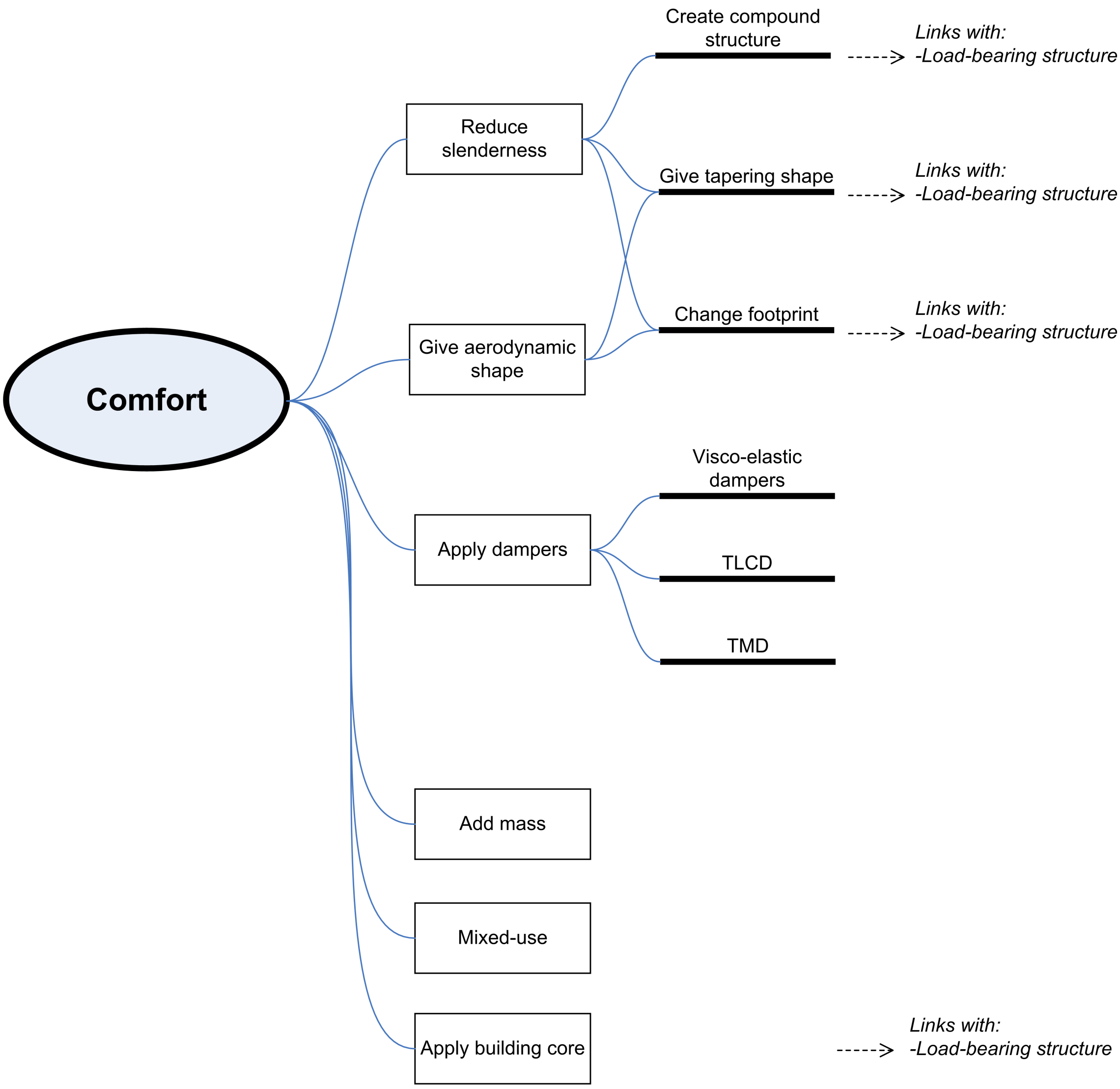



**Figure 7.3:** *Possibilities for stretching the limit of the comfort demands*

Figure 7.3 gives six possible approaches for stretching the limit with regard to the comfort demands in a skyscraper: reduce the slenderness of the building, give the building an aerodynamic shape, apply dampers, add mass, mix of functions in the building or apply a building core inside the building.
Each of these approaches will be further described below.

### 7.3.1 Reduce Slenderness of the Building

Buildings become less susceptible to oscillations when the slenderness of the building is reduced. The slenderness of a building can be reduced in three ways:

**Change the footprint of the building:** By making the footprint of a building wider, the slenderness of the building decreases. However, a wider building is unfavourable with respect to the entry of daylight into the building. A widening of the building's footprint has already been suggested in favour of stretching the limit of the load-bearing structure.

**Create a compound structure:** When multiple towers are rigidly interconnected, the overall slenderness can be much smaller than the individual slenderness of the towers. Note that it is impossible to create a fully rigid connection between two buildings. Therefore, the buildings will not fully act as a structural entity. Creating a compound structure, has already been suggested in favour of stretching the limit of the load-bearing structure.

**Give the building a tapering shape:** By giving the skyscraper a tapering shape, the slenderness of the building decreases. Like the two previous measures, giving the building a tapering shape has also been suggested for the load-bearing structure of the building.

### 7.3.2 Give the Building an Aerodynamic Shape

The vortex shedding due to wind on the skyscraper can cause the building to oscillate. The vortex shedding can be reduced when a more aerodynamic shape is given to the building. There are two ways in which this can be established:

**Change the footprint of the building:** Although there is currently no extensive knowledge in how a building's form influences the vortex shedding, it is known that buildings with round or cut corners show a better behaviour.

**Give the building a tapering shape:** By giving the building a tapering shape, the wind swirls upwards around the building. This improves the building's behaviour with respect to vortex shedding.

### 7.3.3 Apply Dampers

An other way to fulfil the comfort demands despite greater heights, is to apply dampers in the structure of the building. The dampers dampen the building's movements so it will meet the comfort demands. When dampers are applied where they are most needed, the height of a building could be, with respect to comfort demands, theoretically infinite.
In the third chapter of this report, three types of dampers have been discussed:

**Visco-elastic dampers:** These dampers are applied in the structural frame of the building.

**Tuned Mass Dampers (TMD):** These dampers include a mass block which is connected to the building by a series of dashpots.

**Tuned Liquid Column Dampers (TLCD):** A TLCD is similar to a TMD. The difference is that the mass is now water or some other kind of liquid.

### 7.3.4 Add Mass

When the mass of a skyscraper is increased it will show less swaying due to the wind acting on the structure. This explains why the steel Taipei 101 needs a TMD, while the much taller concrete Burj Dubai does not need any dampers at all. However, adding mass is contradictory with the goal of this thesis. If we want to explore the ultimate limits to high-rise, it is necessary to have structures which are as light as possible.

### 7.3.5 Mix of Functions in the Building

In the third chapter of the report has already been mentioned that the comfort demands for floors with a residential or hotel function are more strict than the demands for floors with an office or commercial function.
Due to this, a mixed-use building offers an opportunity for stretching the limits with regard to the comfort criterion. On the top floors of a skyscraper one will experience higher accelerations than on the lower floors. When the higher floors have a commercial function, the comfort demands can still be reached due to the less strict acceleration restrictions. The lower floors, where one will experience less accelerations, can be used for a residential purpose.

### 7.3.6 Apply a Building Core Inside the Building

For stretching the limits of the load-bearing structure, it has already been suggested to construct a building core inside the building which will accommodate a part of the overturning moment at the base of the skyscraper. An other advantage of applying a core in the building is that the building will behave more stiff. Due to this increased stiffness, the building will show less swaying. This means that less movement will be felt by the building's occupants.

## 7.4 Economical Feasibility

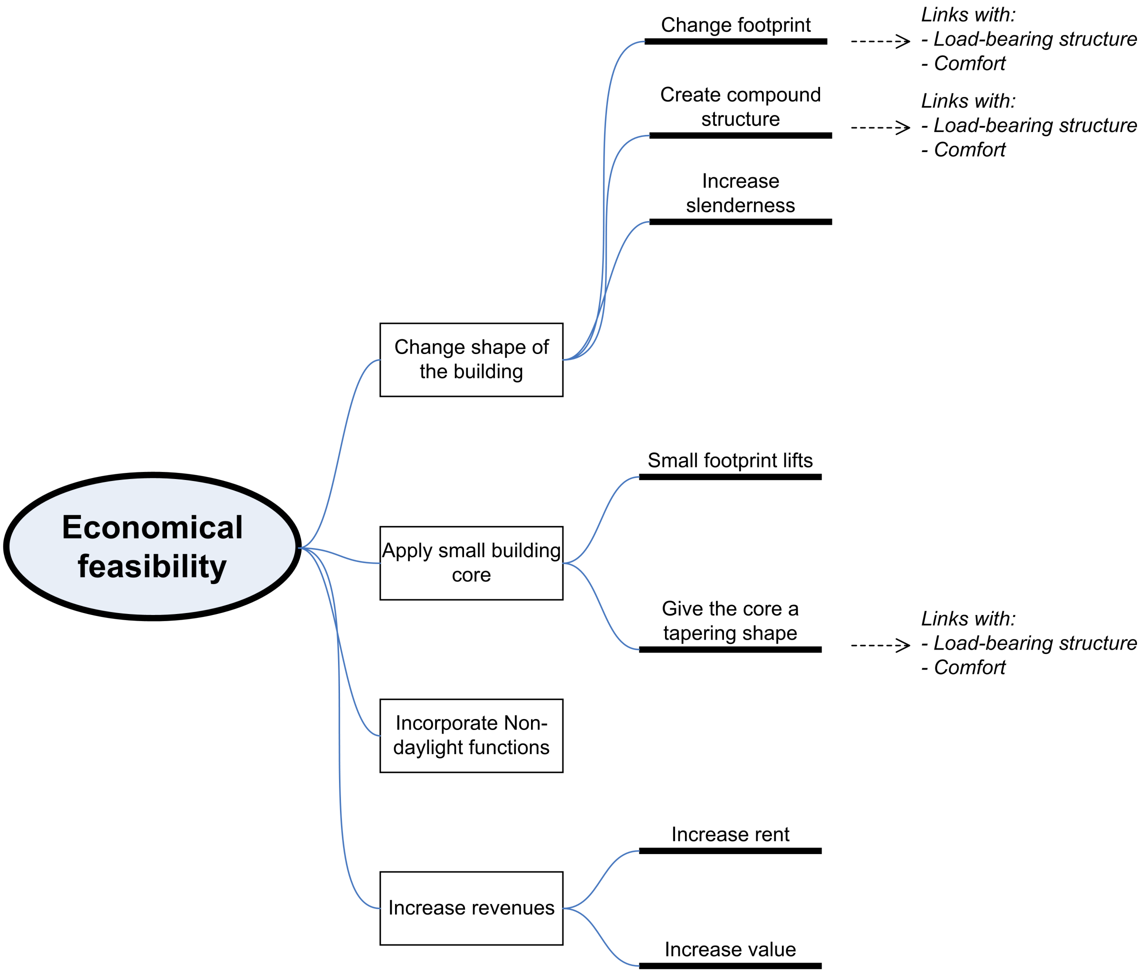



**Figure 7.4:** *Possibilities for stretching the limit of the economical feasibility*

Figure 7.4 gives four possible approaches for stretching the limit with regard to the economical feasibility of a skyscraper: change the shape of the skyscraper, apply a small building core, incorporate functions which do not require daylight or increase the building's revenues.
Each of these approaches will be further described below.

### 7.4.1 Increase the Building's Revenues

The most straightforward way to improve the economical feasibility of a building, is by increasing the revenues generated from it. This can be done in two ways:

**Increase the rent:** By increasing the rent, more money can be generated from the same building.

However, the building's tenants have to be willing to pay this extra rent.

**Increase the value of the building:** Revenues are not only expressed in terms of money, but can be expressed in terms of value as well. A landmark structure can have a poor gross-nett floor ratio, but because of its high value it can still be considered as a economic building.

### 7.4.2 Change the Shape of the Skyscraper

In this report floor surface is considered to be usable floor space when there is a sufficient entry of daylight. Therefore it is favourable to have most of the floor area near the facade of the building, so daylight can enter easily. A way to realise this, is by changing the shape of the building. This can be done in several ways:

**Change the footprint of the building:** In the benchmark case was assumed that the skyscraper has a rectangular footprint. Since daylight can only enter trough the windows in the facade of the building, there is a large area in the centre of the building's footprint which is unreachable for daylight. This area can be accounted for as unusable floor space. By changing the shape of the building's footprint, the surface area of this unusable floor space can be decreased. This will result in a more favourable gross-nett floor ratio.

**Increase the slenderness of the building:** By increasing the slenderness of a skyscraper, the building's width is decreased. This will result in a more favourable gross-nett floor ratio.
However, like mentioned in the previous section, a more slender building is more susceptible to oscillations which endangers the comfort of the building's occupants. Additionally, a slender tower is structurally unfavourable.

**Create a compound structure:** A better solution is to create a compound structure by interconnecting multiple towers. This means that the individual towers can be constructed more slender, while the overall slenderness of the system remains sufficient. Creating a compound structure is a solution which has already been suggested in relation with stretching the limits of the load-bearing structure and the comfort criterion.

### 7.4.3 Apply a Small Building Core

The building's core can be considered as unusable floor space. This because daylight can not enter inside the core. Therefore building cores are normally used to accommodate the stairwells and lift shafts of the building. A smaller building core means that a more favourable gross-nett floor ratio can be achieved. A small building core can be achieved in two ways:

**Smaller footprint lifts:** In some cases the footprint of the building's lifts determine the dimension of the building core. When the footprint of the lifts is reduced, it might be possible to reduce the size of the building core as well.

**Give the core a tapering shape:** The higher up the building, the fewer lifts are needed for the vertical transportation of people and goods. Therefore the size of the building core can be reduced. Also from a structural point of view, a smaller sized building core is permissable higher up a skyscraper. A tapering building core is even more desirable when the building itself has a tapering shape as well.

### 7.4.4 Incorporate Functions which do not Require Daylight

By incorporating functions into the building which do not require daylight, floor space which was at first considered to be unusable, can now be turned into usable floor space and more revenues can be generated from the building. Examples of functions which do not need the entry of daylight are: shopping centres, museums, sport facilities, cinemas, theatres, etc.. In many existing high-rise buildings these kind of functions can be found in the plinth underneath the tower.

## 7.5 Evacuating the Building

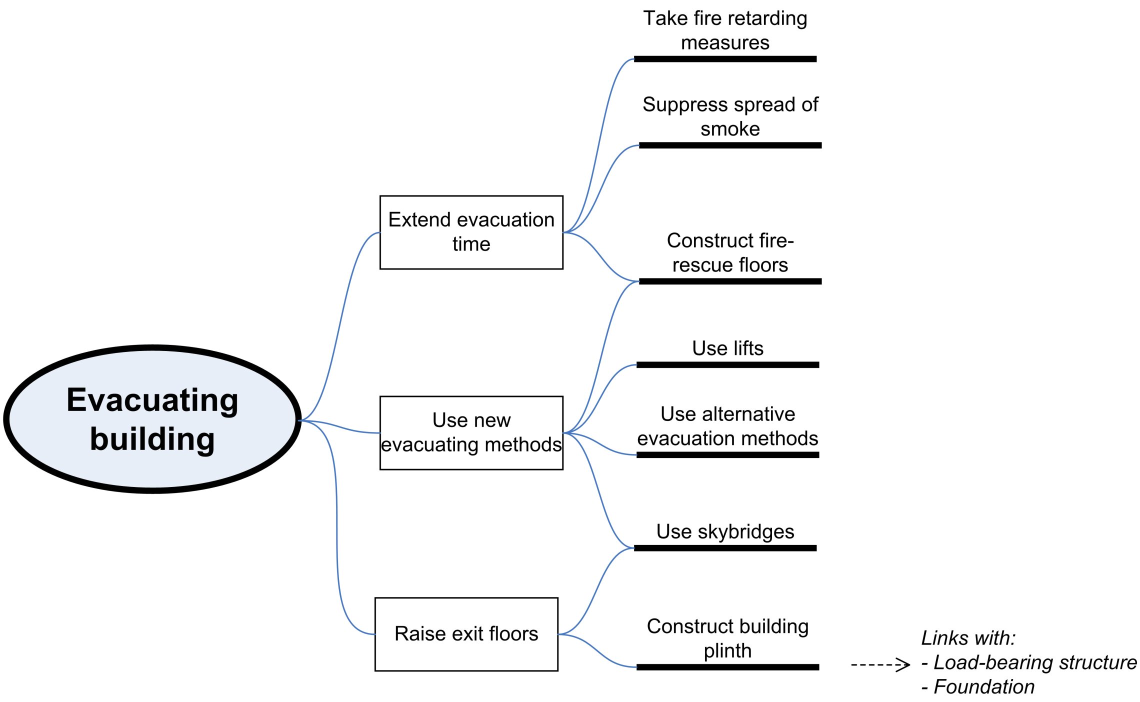



**Figure 7.5:** *Possibilities for stretching the limit for evacuating the building*

In figure 7.5 three approaches are given for stretching the limit for evacuating a skyscraper. These approaches are: extend the evacuation time, use new evacuation methods or raise emergency exit floors.

### 7.5.1 Extend the Evacuation Time

In the benchmark case a maximum evacuation time of 30 minutes has been assumed. This means that all the building's occupants have to leave the skyscraper within half an hour. This seriously limits the height of the building. By extending the allowable evacuation time, higher skyscrapers are achievable. There are three ways in which the evacuation time of a building can be extended:

**Take fire retarding measures:** When the spreading of fire is delayed, it is confined to a small area of the building. This means that the rest of the building stays intact and free from fire. This part of the building can still be used for evacuating people out of the structure. In modern skyscrapers fire retarding measures are already applied, e.g.: the use of fire compartments, the installation of a sprinkler system and coating combustible materials with fire-resistant materials.

**Suppress the spread of smoke:** In case of a fire, the smoke production is often much more lethal than the fire itself. Thick smoke often claims victims through suffocation long before the actual fire reaches these floors. The smoke disorientates people which are trying to find their way out of the building. By preventing smoke from spreading throughout the entire building, the building's occupants have more time to escape.

**Construct fire-rescue floors:** An other way to extend the evacuation time is by constructing fire-rescue floors inside the building. These floors function as a hide-out for the building's occupants during a fire. Here, people can await their rescue or wait till the fire is extinguished so they can escape safely from the building. No smoke may find its way into the fire-rescue floors and back-up power and water supplies should be present in these spaces.
A similar solution can be found in high-rise buildings in Hong Kong; the so-called "Fire-Refuge Floors". According to the Hong Kong Fire Safety Department, a refuge floor is a protected floor that serves as a refuge for the occupants of the building to temporarily assemble and rest in case of an fire. So, these floors have about the same function as the suggested fire-rescue floors. The only difference is that on a refuge floor, the building's occupants are expected to stay only for a short while and subsequently continue their escape. While on Fire-rescue floors people are able to lodge till after the emergency.
In 1996 requirements for refuge floors were published by the Hong Kong Buildings Department. The most important requirements are listed below:

- Refuge floors should be provided in residential buildings exceeding 40 storeys and in buildings with an other function exceeding 25 floors. A refuge floor should be at not more than 25 storeys from any other refuge floor. This can also be street level or the roof of the structure.
- There is no occupied accommodation or accessible mechanical plant room at the same level as the refuge floor.
- The nett area for refuge should be not less than 50% of the total gross floor area of the refuge floor and should have a clear height of at least 2300 mm.
- The area of refuge has to be fully separated from the remaining areas of the building and has to be protected by fire resistant walls and doors.
- At least two walls of the refuge floor must be open to permit sufficient natural cross ventilation through the refuge space.
- Any staircase passing through a refuge floor should interrupted. Instead, the exit route has to be diverted to pass over a part of the refuge area before it is continued to exit downwards.
- The floors have to be fitted with an emergency lighting system.
- A refuge floor should be provided with fire service installations.
- A refuge floor should be served by a fireman's lift. The lift doors should not open onto the refuge floor in normal operation.

The Burj Dubai will be world's first super-tall skyscraper which is equipped with fire-refuge floors.

### 7.5.2 Use New Evacuation Methods

Currently, the evacuation of a high-rise building is still relying on the use of staircases. However this makes the evacuation of disabled people difficult. Even a well able-bodied person will struggle to walk down a 800 metres tall building while being in a panic situation, experiencing high temperatures and breathing in smoke. Therefore, new methods for evacuating a building should be developed:

**Construct fire-rescue floors:** When multiple fire-rescue floors are applied in the building, the building's occupants do not have to go all the way to the bottom of the tower in order to reach safety.

**Use lifts:** In chapter 3, it has already been suggested that in order to improve the speed and effectiveness of the evacuation of a high-rise building, the possibility of using the building's lifts for evacuation should be considered. Using lifts for the evacuation of people instead of stairs, is by far a quicker. However, to be able to use elevators in the evacuation, all kinds of special measures have to be taken (see chapter 3).

**Use alternative evacuation methods:** To improve the evacuation of a high-rise building, it may be necessary to think beyond existing evacuation methods. In chapter 3, a couple of these methods have already been mentioned.

**Use skybridges:** When multiple high-rise buildings are connected to each other by skybridges, it allows people to evacuate via the other skyscrapers. Because there is no emergency situation in these buildings, the building's staircases and lifts are fully operational.

### 7.5.3 Raise Emergency Exit Floors

When the emergency exit floors are located higher up the building, the occupants have to descent less to be able to escape from the skyscraper. This will reduce the evacuation time of the building. Raising emergency exit floors can be done in two ways:

**Construct a plinth at the base of the tower:** Constructing a building plinth has already been suggested for stretching the limits of the load-bearing structure and the foundation of a skyscraper. A building plinth can also help in quickly evacuating a high-rise building. The building's occupants can flee away from the tower via the roof of the building plinth. In this way, apart from the ground floor, an additional emergency exit floor is created.

**Use skybridges:** When at different levels of the building skybridges are connecting to neighbouring buildings, there are multiple emergency exit floors at which people can escape from the building.

## 7.6 Vertical Transportation

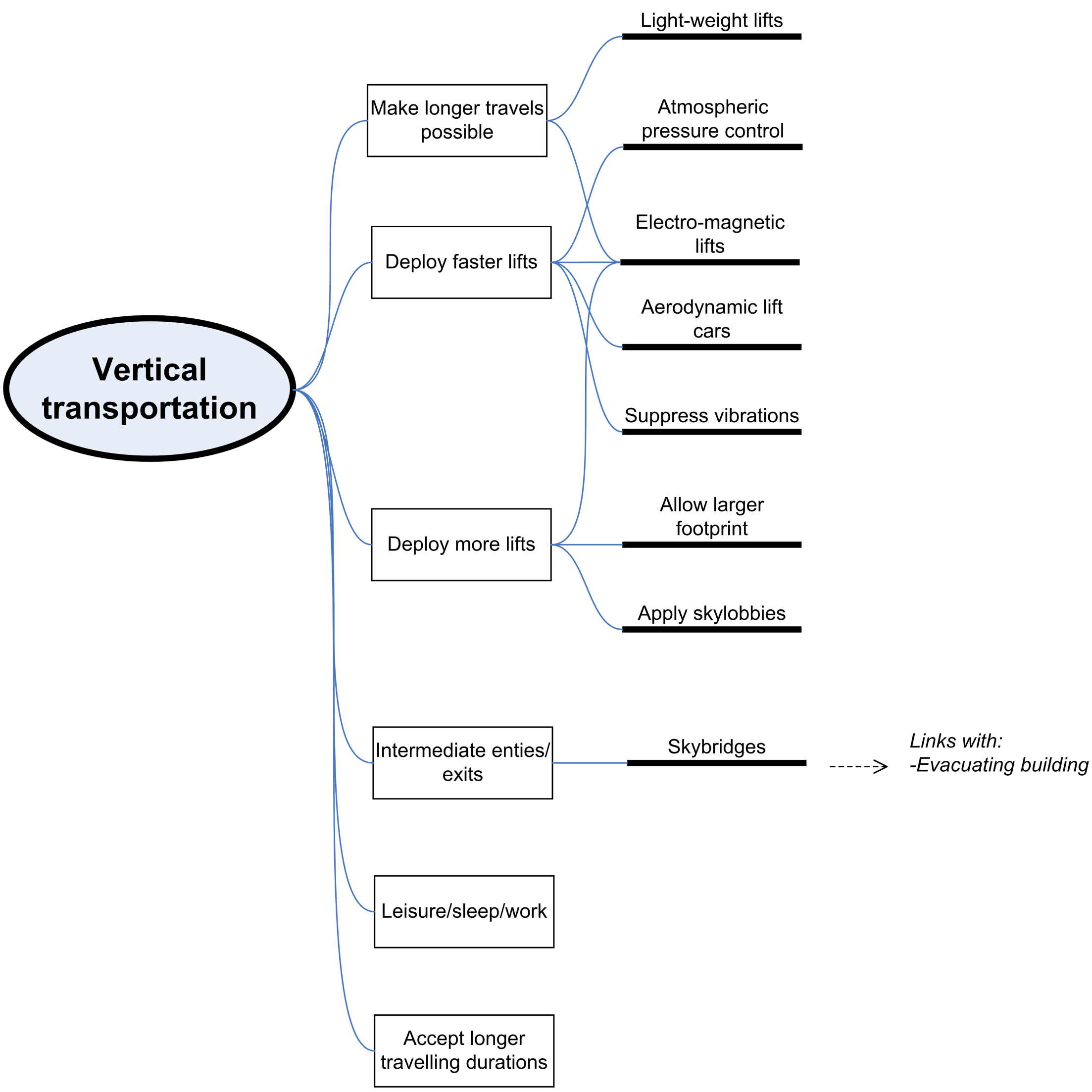



**Figure 7.6:** *Possibilities for stretching the limit of vertical transportation*

In the previous chapter two limits to the vertical transportation were found. The first limit is imposed by the available lift technology, the second limit is imposed by a nation's building regulations. In addition to this, the occupants of a skyscraper will not accept much longer lift travelling times.
For stretching these limits, figure 7.6 gives six possible approaches: make longer travels possible, deploy faster lifts, deploy more lifts, accept longer travelling durations, intermediate entries/exits or work, leisure and sleep in one building.

### 7.6.1 Make Longer Travels Possible

The technical limit of the present-day lift technology lies at approximately 1000 metres. If we want to build buildings which exceed this limit, new lift technologies should become available. Two of these technologies which are known to be under development are:

**Light-weight lift ropes:** A problem is that for travels of a 1000 metres, 60 tons of ropes are required to lift a 1600 kg pay load. This enormous weight makes longer travels impossible.
The ThyssenKrupp lift company is currently developing Kevlar ropes to replace the conventional steel cables. The Kevlar ropes weigh only 20 % of the weight of a steel cable. This means that the ratio, total weight of the rope to the lifted pay load is much more favourable. Less powerful electro motors can be applied for turning the lift sheaves. This will lead to substantial savings on energy consumption. The lift sheaves are the pulleys which hoist the lift car up and down its shaft.
Apart from this, there are two other major advantages of using Kevlar ropes. *First*, the modulus of elasticity of a Kevlar rope, is larger than the elasticity modulus of a steel cable. This leads to less stretch in the ropes. *Second*, smaller sheaves can be applied. For conventional steel ropes, the sheave to rope ratio is 40:1. With Kevlar ropes a ratio of 21:1 is possible.

**Electro-magnetic lifts:** The Japanese company Toshiba aims to be world's first lift company to commercialise this concept in the spring of 2008. The principle of this new lift system is based on the technology used in the Maglev trains. By using strong electromagnets these trains can move frictionless along a track by hovering above it. The same electromagnets make it possible to power the train forward: Electromagnets in the track behind the train repel the vehicle, while magnets in front of the train attract it. This results in a forward motion. The direction and strength of the magnetic field can easily be manipulated by respectively altering the direction of the electric current in the electromagnets or increasing the electric current. Worlds first commercial application of the Maglev train can be found in Shanghai in China. This train transports passengers from and to the airport of the city.
Freed from the use of hoist ropes, travels of more than a 1000 metres are made possible. Furthermore, this new lift system requires less energy to power the lift cabs and a much smoother and quieter ride can be provided. The possibilities of this new system seem endless: it may be possible to deploy multiple lift cars in one shaft, reducing the footprint of the lift shafts in the building. Lifts may even be able to travel horizontally, revolutionising the lift concept.

### 7.6.2 Deploy Faster Lifts

In order to meet the demands imposed by the building regulations, faster lifts are needed. When buildings grow taller, faster lifts are also necessary to keep the travelling time acceptable. There are several ways in which a faster lift can be realised without compromising the comfort demands:

**Atmospheric pressure control:** With very tall buildings, the difference in atmospheric pressure between the starting floor and the destination floor can cause problems (Manukata et al. [27]). Due to the fast lifts, a sudden change in atmospheric pressure is experienced by the lift's occupants

and this may cause discomfort due to "ear-popping". This problem can be solved by installing an "Atmospheric pressure control system" in the lift car. The working of this system is explained in the article of Manukata et al. [27]: "Normally, a pressure change begins slowly, speeds up and slows again. To enhance riding comfort without changing the hoistway length and travelling time, it is necessary to control the pressure change from the start of the journey until it ends." In this way a fixed change rate can be realised for the atmospheric pressure inside the lift car. This fixed rate lies below the maximum change rate which would have been experienced if the lift was not equipped with an atmospheric pressure control system. The first application of such a system has been installed in two lifts of the Taipei 101 in Taiwan.
When buildings grow taller, fitting an atmospheric pressure control system in the lift car alone, may not be enough because the realised fixed change rate lies above the comfort demand. In this case, the gradual change of the atmospheric pressure should already start in the lift lobbies, when people are waiting for their lift.

**Aerodynamic lift car:** By giving the lift cab an aerodynamic shape, the air resistance generated by the lift car, will decrease. This means that less wind noise will be experienced by the lift's occupants. Additionally, less wind resistance will also lead to savings on the energy consumption of the lift.

**Suppress vibrations:** When lifts are travelling at higher speeds, vibrations in the hoist ropes and of the lift car are likely to occur. These vibrations have to suppressed. In the lifts of the Taipei 101 this is achieved by installing suppressors against rope deflection and by installing an active control system on the lift cars. This system cancels the vibrations by moving a counter mass in the opposite direction.

**Electro-magnetic lifts:** Because electro-magnetic lifts can move frictionless along their guide rail, less or no vibrations will be experienced even when the lift car travels at higher speeds.

### 7.6.3 Deploy More Lifts

The most straightforward solution to prevent long travelling times, is by deploying more lifts in the skyscraper. Fitting more lifts in one building can be achieved in several ways:

**Allow larger lift footprint:** The easiest way to fit more elevators in a building is by increasing the footprint of the lifts. However, a larger lift footprint will decrease the gross-nett floor ratio of the building, which will endanger the economical feasibility of the building. Therefore, this is a disputable solution.

**Apply skylobbies:** A better way of fitting more elevators in a building, is by stacking the lifts on top of each other. In this way, the footprint of the lifts will not increase. Stacking lifts on top of each other, can be done by applying skylobbies. At these skylobbies, passengers have to transfer to other lifts (see chapter 3).

**Electro-magnetic lifts:** Like in the previous approaches, the introduction of the electro-magnetic lift system, could offer a solution. Because the lift cars are not connected to any hoist ropes, multiple cars can operate in one single shaft. A computer system makes sure that the dispatching of the

lift cars happens as efficient as possible.
Because the electro-magnetic lift cars are not operated by hoist ropes, it may be even possible to move the cars horizontally in the future. In this way a ring can be created inside the building, which will further increase the efficiency of the transportation system.

### 7.6.4 Accept Longer Travelling Durations

In the previous subsection, three ways were given in which the lift travelling times could be limited despite an increasing building height. However, we should ask ourselves if this is fair. Should we not accept longer travelling times when we want to have taller buildings? When taking the lift is seen as a part of our commute, it may become more acceptable. After all, a lot of commuters already accept that they have to wait for the bus, metro or train.

### 7.6.5 Intermediate Entries/Exits

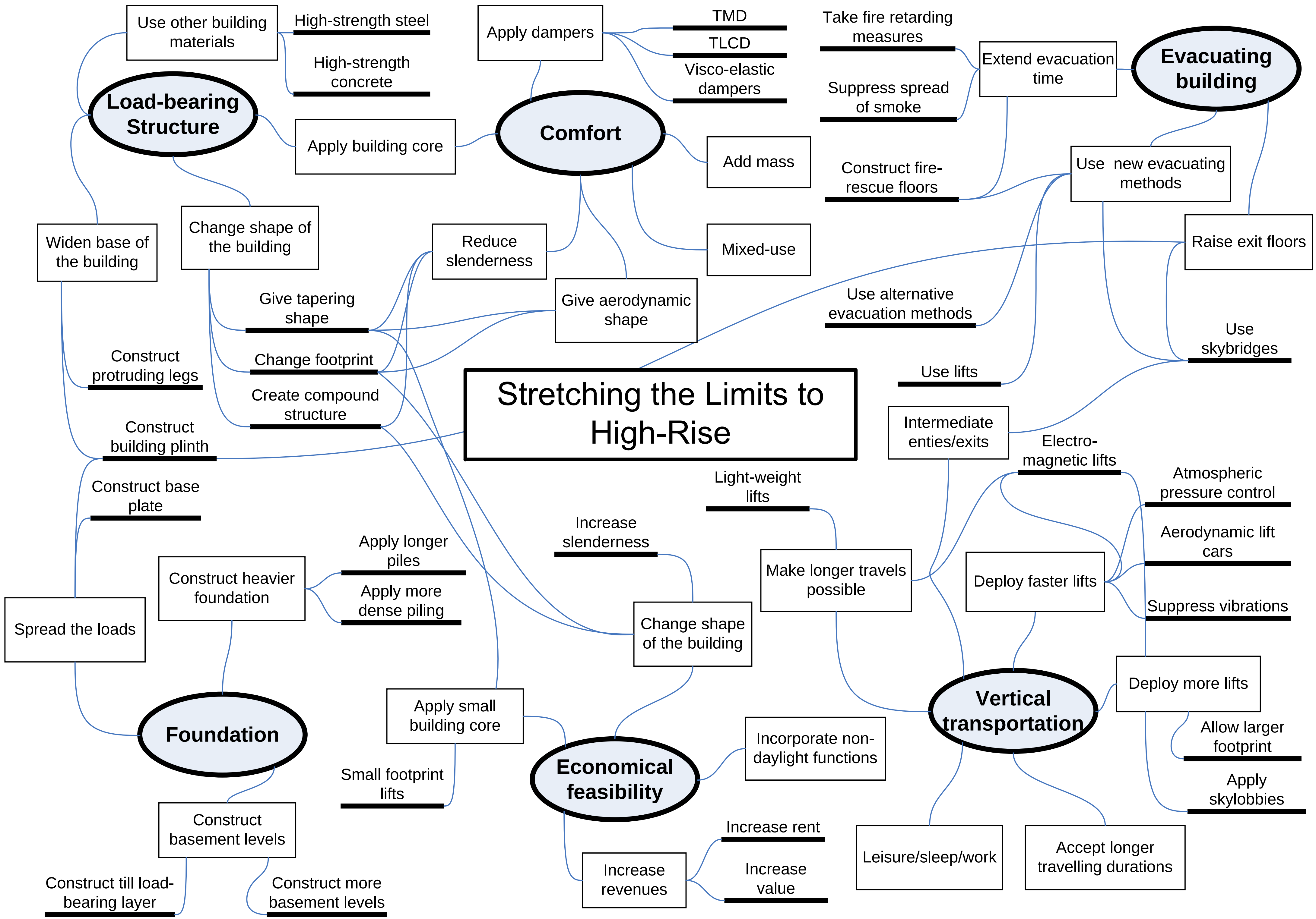
When people can enter and exit a building at multiple levels, lifts do not have to travel over the full length of the skyscraper. All of the previous mentioned problems can be solved: no travels longer than a 1000 metres are necessary, no high-speed lifts are needed and in case of an emergency, emergency personnel can enter the building at different levels. These intermediate entries and exits can be created by connecting the building with skybridges to neighbouring high-rise buildings. The use of skybridges has already been suggested for stretching the limit to the evacuation of a building.

### 7.6.6 Work, Leisure and Sleep in One Building

A last, a futuristic solution would be to have work, leisure and sleep combined in one building. This means that people do not even have to leave the building any longer. This will relieve the vertical transportation system of the building. Whether this is a desirable development, is questionable.

## 7.7 Combining the Challenges

From the tree diagrams given in the previous sections of this chapter, can be concluded that some of the solutions for stretching the limits of a skyscraper are shared by multiple challenges. These correlations can be best visualised by making a web diagram which combines all of the six tree diagrams. This web diagram can be found on the next page.
From the web diagram can be learned that changing the shape of the skyscraper is one of the most promising solutions for stretching the limits to high-rise. Altering the skyscraper's form stretches the limits with regard to the building's load-bearing structure, comfort demand and economical feasibility. All the other measures simultaneously push the limits of two challenges at the most. Therefore, the next chapter will further examine the effects of changing the shape of the building.


Stretching the Limits to High-Rise
Load-bearing Structure
Use other building materials
High-strength steel
High-strength concrete
Apply building core
Comfort
Apply dampers
TMD
TLCD
Visco-elastic dampers
Add mass
Mixed-use
Reduce slenderness
Give aerodynamic shape
Evacuating building
Extend evacuation time
Take fire retarding measures
Suppress spread of smoke
Construct fire-rescue floors
Use new evacuating methods
Raise exit floors
Use alternative evacuation methods
Use skybridges
Use lifts
Widen base of the building
Change shape of the building
Give tapering shape
Change footprint
Create compound structure
Construct protruding legs
Construct building plinth
Construct base plate
Spread the loads
Construct heavier foundation
Apply longer piles
Apply more dense piling
Foundation
Construct basement levels
Construct till load-bearing layer
Construct more basement levels
Apply small building core
Small footprint lifts
Increase slenderness
Change shape of the building
Economical feasibility
Incorporate non-daylight functions
Increase revenues
Increase rent
Increase value
Light-weight lifts
Make longer travels possible
Intermediate enties/exits
Electro-magnetic lifts
Atmospheric pressure control
Aerodynamic lift cars
Suppress vibrations
Deploy faster lifts
Vertical transportation
Deploy more lifts
Allow larger footprint
Apply skylobbies
Leisure/sleep/work
Accept longer travelling durations

## 7.8 Changing the skyscraper's location

In the previous sections was hold on to the fact that the skyscraper will be located in The Netherlands. However, when the building is erected somewhere else, the conditions may be more favourable, which results in higher limits. Below, some of these favourable conditions are described:

**Soils with a better load-bearing capacity:** In most parts of The Netherlands the subsoil has a poor load-bearing capacity. This limits the strength of the skyscraper's foundation. When it is decided to build the skyscraper in an area where the subsoil has a better load-bearing capacity (e.g. rocky soils), a higher building is possible.

**Lower wind loads:** When the building is built in an area where smaller wind loads can be taken into account, smaller lateral loads are acting on the structure.

**Dominant wind direction:** When the skyscraper is built on a location with one dominant wind direction, the building can be orientated in such a manner that the strongest direction of the structure is orientated in the direction of this normative wind load. This was also done in the design process of the Burj Dubai. Through wind tunnel tests was determined which orientation of the tower was best in order to minimise the building's wind-induced accelerations.

# Chapter 8

# Change the Shape of the Building

By changing the shape of the building, the limit to the load-bearing structure of the building can be stretched. Changing the shape of the building will also have a favourable effect on the limits with regard to the comfort criterion and the economical feasibility of the building.
In the previous chapter, three different options were given for changing the shape of a skyscraper:

1. Change the footprint of the building.
2. Give the building a tapering shape.
3. Create a compound structure.

The earlier considered benchmark skyscraper can be schematised as a cantilevered beam fixed on one side into the ground. This schematisation is depicted in figure 8.1.

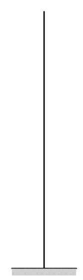


**Figure 8.1:** *Schematisation benchmark skyscraper*

When the footprint of the building is changed, the schematisation of the building remains the same. Also when the building is given a tapering shape, the schematisation remains unchanged. However, when a compound structure is created, the schematisation of the building changes because multiple "cantilevered beams" are now interconnected. Figure 8.2 depicts some examples of schematisations for compound structures.

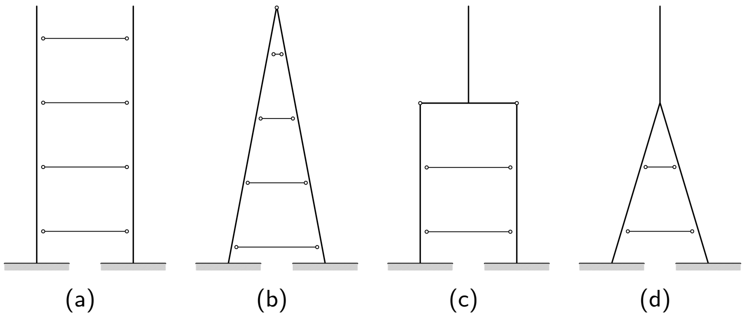


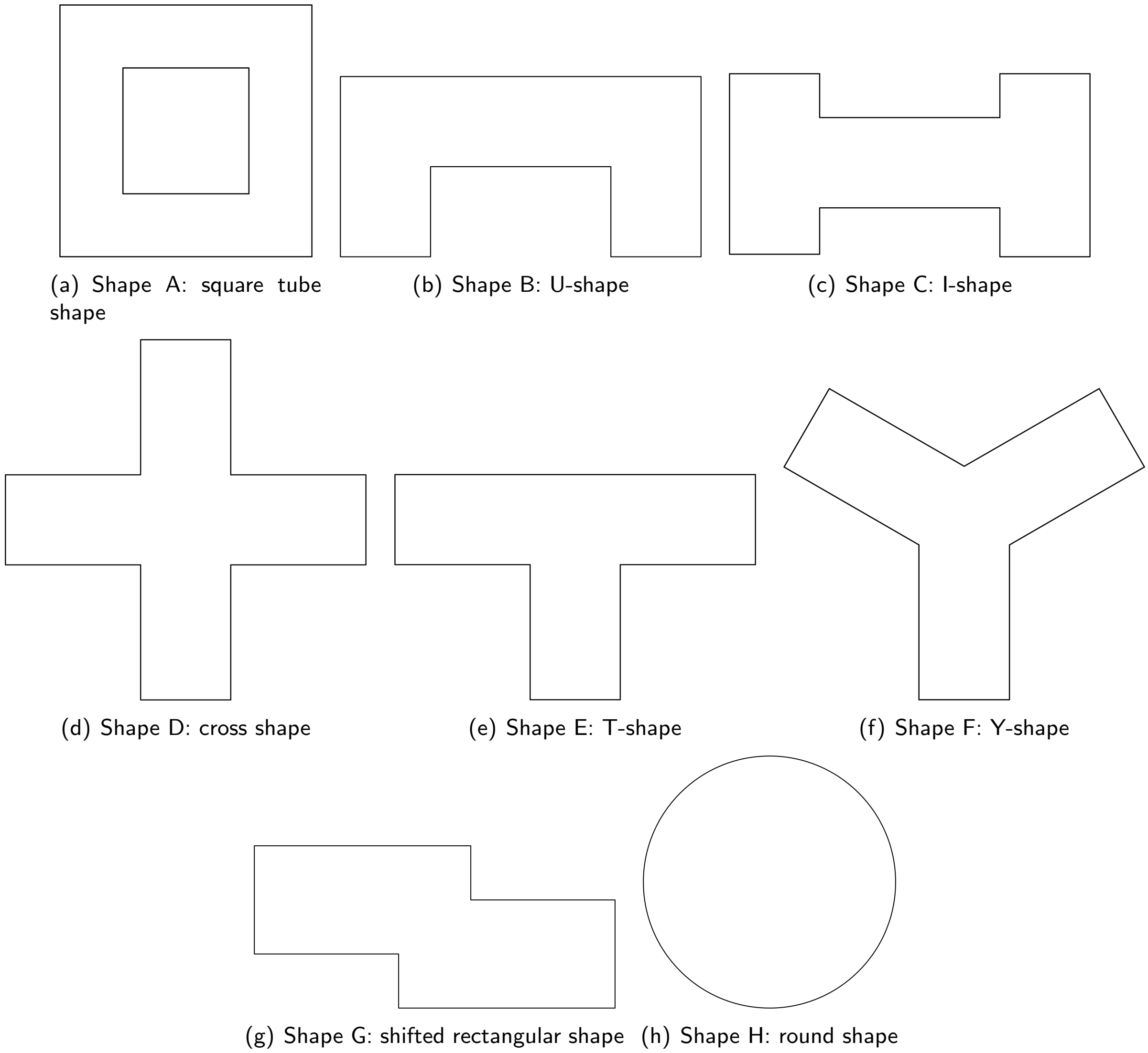

**Figure 8.2:** *Possible schematisations for a compound skyscraper*

The options for changing the shape of the building have already been qualitatively considered in the previous chapter. It was explained how each of the possibilities contributes to stretching the limits of the load-bearing structure, economical feasibility and comfort of the building. In this chapter we will take this a step further by considering the limits quantitatively. By doing so, for each of the three challenges an estimation can be made to which height each limit can be stretched.

## 8.1 Change the Footprint of the Building

In the previous chapter has been explained that by widening the footprint of the building, a larger overturning moment can be resisted by the building's structure and the skyscraper becomes less susceptible to oscillations. However, a wider cross-section endangers the entry of daylight into the building. Therefore, besides changing the dimensions of the footprint, also the shape of the building's cross-section has to be altered. Changing the form of the building's footprint can result in a more favourable gross-nett floor ratio. A number of possible building forms is given in figure 8.3.

Like with the benchmark skyscraper, it is assumed that the load-bearing structure of these forms is located in the facade of the building. Again, a wall-to-window ratio of 38% will be considered. This means that the facade columns will have a centre-to-centre distance of 2.5 metres.



(a) Shape A: square tube shape (b) Shape B: U-shape (c) Shape C: I-shape

(d) Shape D: cross shape (e) Shape E: T-shape (f) Shape F: Y-shape

(g) Shape G: shifted rectangular shape (h) Shape H: round shape

**Figure 8.3:** *Possible cross-sections for a skyscraper*

### 8.1.1 Load-bearing Structure

In order to judge whether changing the form of the building's cross-section improves the building's capacity to accommodate the overturning moment at the base of the structure, the moments of inertia for each shape are computed.
In chapter 6 was determined that with a storey depth $d$ of 12.6 metres and a minimum required gross-nett floor ratio of 70%, the footprint of the benchmark skyscraper has dimensions of $55.2 \times 55.2$ metres. This gives the building a gross floor area of 3047 $m^2$. To make a fair comparison between the benchmark case and the new building shapes, the gross floor area will be kept the same.

**Shape A**
Initially, this shape seems to be a promising solution. The footprint of the skyscraper can be wider because an atrium in the centre of the building assures that sufficient daylight can enter onto the floors. However, when the building gets taller, only a limited amount of daylight will find its way all the way to the bottom of the atrium. This means that the advantage of having an atrium is undone. A solution could be to leave large openings in the building, to ensure that daylight can reach the bottom of the atrium. However, by doing so, the schematisation of the entire building changes and will become more like a compound structure (see figure 8.4). Compound structures will be discussed hereafter and therefore shape A will not be further considered.

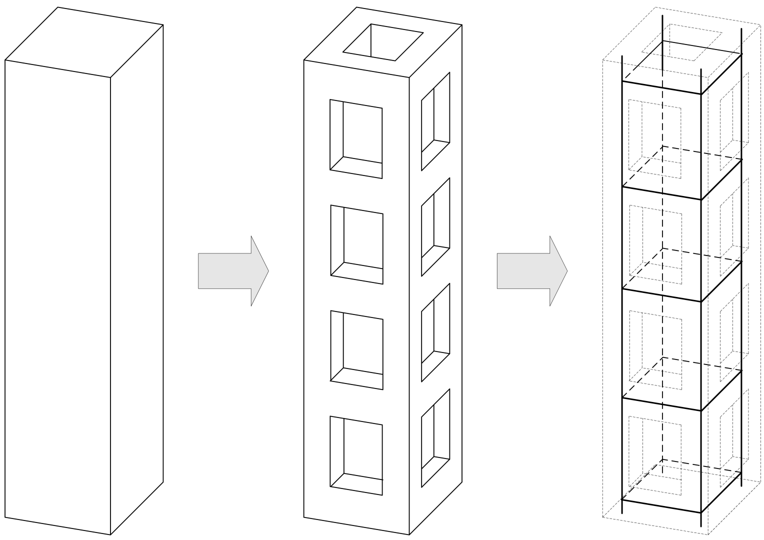


**Figure 8.4:** *A skyscraper with footprint shape A will become a compound structure*

**Shape B**
The second building form is a U-shape. The relation between the body and the legs of the U is described in figure 8.5a. Both the length of the body and the length of the legs of the U-shape are dependent on the variable $l$.
In figure 8.5b, $d$ represents the storey depth, which is, like in the benchmark case, equal to 12.6 metres. When the width $w$ of the building is larger than twice the storey depth d, there will be an area in the middle of the building where daylight can not enter. The width of this area is indicated by the variable

*c.*

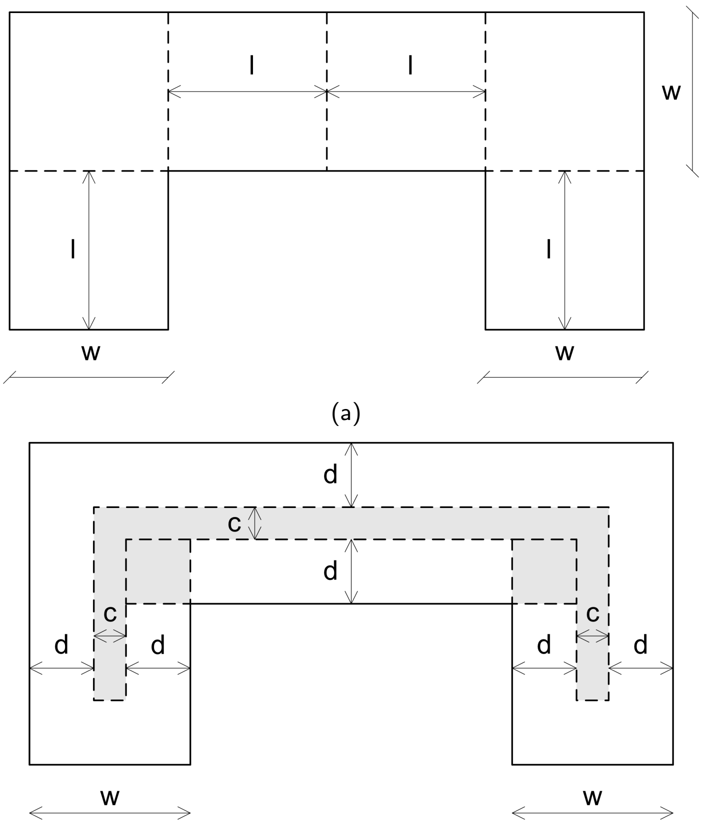



**Figure 8.5:** *Geometry of shape B*

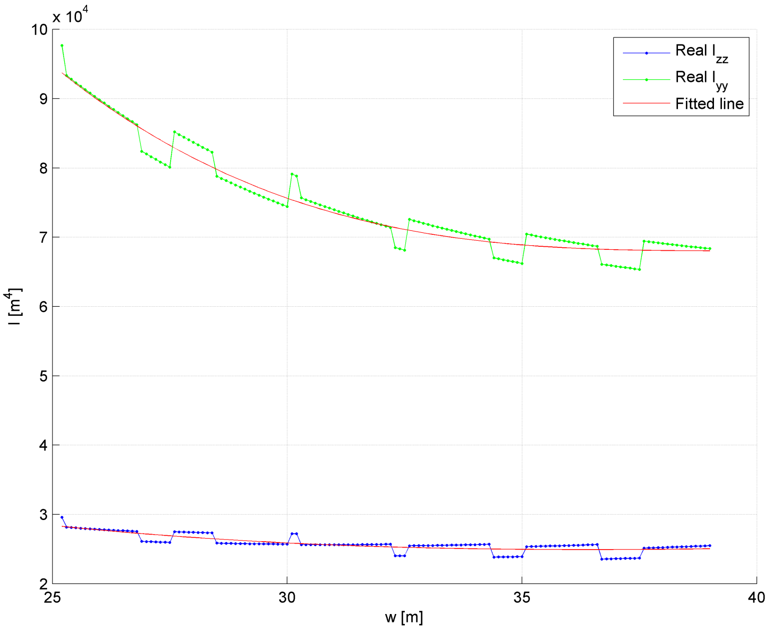
When the width of the building $w$ increases, $l$ will decrease since the gross floor area of the building is kept constant at 3047 m$^2$. A decrease of variable l means that the internal lever arm of the load-bearing structure decreases. This will cause a reduction in the cross-section's moment of inertia. However, the total width of the building decreases as well which means that a smaller facade surface is subjected to wind forces. This means that, despite a smaller moment of inertia, the vertical stresses in the columns might decrease. In order to determine whether this is the case, the minimum width of variable $w$ (which is equal to twice the building depth $d$) is slowly increased with steps of 0.1 metre. For each step, the vertical stresses in the columns of the load-bearing structure are calculated. In this way the optimal dimensions for the building form can be determined, i.e. the dimensions at which the stresses in the columns of the structure are the smallest. The stresses in the columns of the building are the sum of the stresses caused by vertical loads and by horizontal loads acting on the structure.
In order to determine the stresses due to the horizontal loads, the moment of inertia of the building's cross-section has to be computed. Using also the overturning moment at the base of the building, the stresses in the columns can be calculated (see equation 6.18, chapter 6). Subsequently, the computed stresses $\sigma_{M;d}$ are multiplied with a factor $\gamma$ to take into account the "shear-lag effect" . This factor will

be 1.33 (see chapter 6).

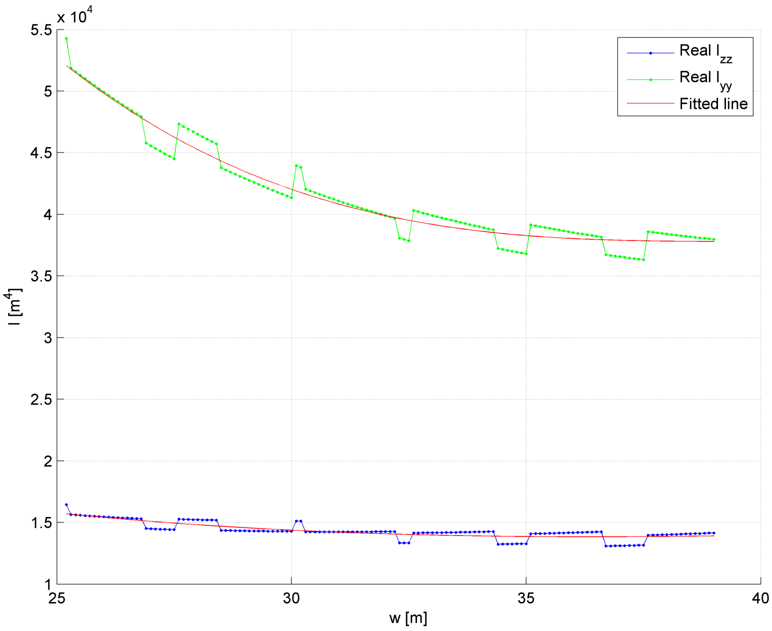
Like in chapter 6, both a concrete and a steel load-bearing structure are considered. For each structure the $I_{zz}$ and $I_{yy}$ are calculated for each step of 0.1 metre. The moments of inertia are determined by using the calculation method as described in the book of Hartsuijker [13]. Because a large amount of similar calculations have to be made successively, the computation of the moments of inertia is conducted by using MatLab computer software. An example of the MatLab input is given in appendix C.
By using MatLab it is possible to plot the changes in the cross-section's moments of inertia, when the building width w is increasing. Graph 8.6a applies to a concrete load-bearing structures and graph 8.6b applies to a steel load-bearing structure. The sudden changes in the graphs are caused by the sudden changes in the number of columns in the facades of the building. The fitted lines are third degree polynomial fittings. In this way a more smooth graph is obtained from which the trend of the graph can be read.
From the graphs can be deduced that in the y-direction the cross-section's moment of inertia decreases when the building width *w* increases. This is like what was expected. However, in the normative z-direction the cross-section's moment of inertia has almost a constant value. In this direction the decrease of parameter I is counterbalanced by the increase of the building width *w*.

In chapter 6, concrete with a characteristic compressive cylinder strength of 90 $\mathrm{N/mm^2}$ was considered. Now, a high-strength concrete mixture with a cylindrical compressive strength of 125 $\mathrm{N/mm^2}$ will be examined. The design compressive strength of such a mixture is approximately 83 $\mathrm{N/mm^2}$.
The examined steel grade is S355. This steel has a yield stress of 355 $\mathrm{N/mm^2}$. However, due to the thick steel plates which are used (150 mm), a reduced yield stress of 275 $\mathrm{N/mm^2}$ has to be taken into account.

Figure 8.7 displays the changes in the column stresses at a certain height of the skyscraper. Figure 8.7a depicts the stress changes in the columns of a concrete skyscraper at a height of 370 metres. At this height, the stresses in the columns lie around 83 $\mathrm{N/mm^2}$. This is the maximum compressive stress which can still be accommodated by the concrete. Therefore, 370 metres can be regarded as the limit of this building shape when constructed out of concrete. The graph shows as well that changing the geometry of the shape, has little effect on the stresses in the columns. This can be explained by the little decline of the cross-section's moment of inertia in the normative direction.
Figure 8.7b depicts the stress changes in the columns of a steel skyscraper at a height of 500 metres. At this height the stresses in the columns lie around 275 $\mathrm{N/mm^2}$, the stress at which the steel column will begin to yield. Therefore, 500 metres can be regarded as the height limit of this building shape, when constructed out of steel. Again, it can be seen that the shape's geometry has a little effect on the column stresses.

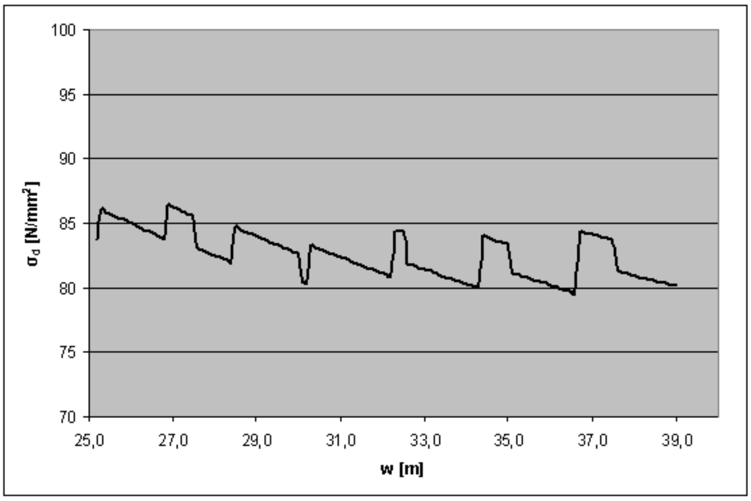



(a) Concrete

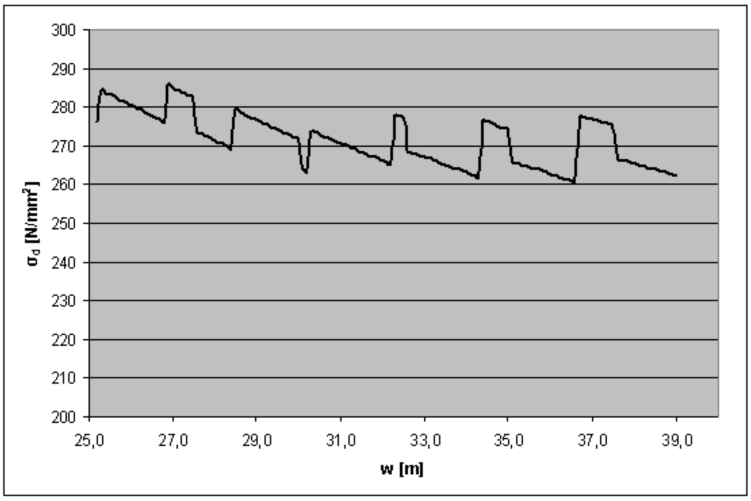



(b) Steel

**Figure 8.6:** *Moments of inertia shape B*




(a) Column stresses in a 370 metres tall concrete skyscraper




(b) Column stresses in a 500 metres tall steel skyscraper

**Figure 8.7:** *Compressive stress in columns of building shape B*

### Shape C

The cross-section of this shape has a I-shape. The geometry of this shape is given in figure 8.8.

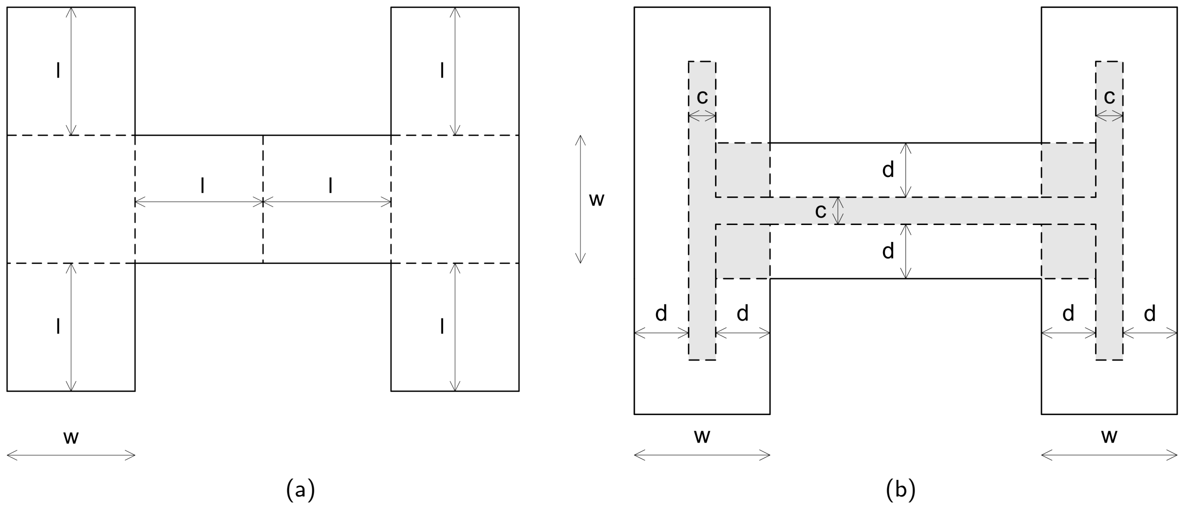
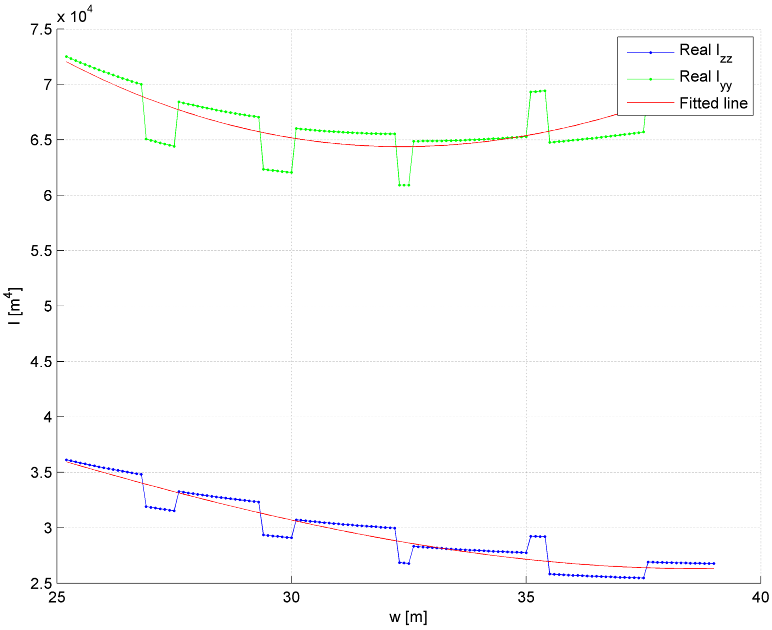


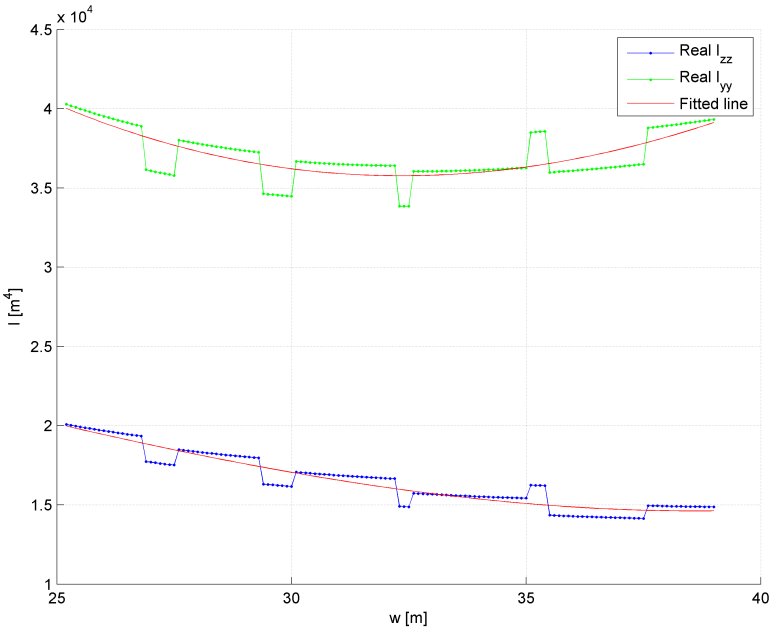

**Figure 8.8:** *Geometry of shape C*

Again the building width *w* is slowly increased with steps of 0.1 metres. By using MatLab the changes in the moments of inertia of the skyscraper's cross-section can be plotted. The result of the MatLab calculations is given in the graphs of figure 8.9. The upper graph applies to a skyscraper with a concrete structure while the lower graph shows the moments of inertia for a steel structure.
In the cross-section's normative z-direction, the moment of inertia decreases when the building width *w* is increased.

Next, the compressive stresses in the columns can be computed for both the concrete and the steel structure. The changes in the compressive column stresses are depicted in figure 8.10. The best result is obtained when the variable *l* has a maximum value and *w* has a minimum value of twice the storey depth *d*. Afterwards, the increase of *w* results in an increase of the compressive stresses in the columns. Figure 8.10a depicts the compressive stresses in the concrete columns at a height of 390 metres. At the minimum value of w, the stress in the columns is equal to 83 $\mathrm{N/mm^2}$. 83 $\mathrm{N/mm^2}$ is the maximum allowable compressive stress. Therefore, 390 metres can be regarded as the height limit of this building shape, when constructed out of concrete. When constructed out of steel, this limit lies at a height of approximately 535 metres.

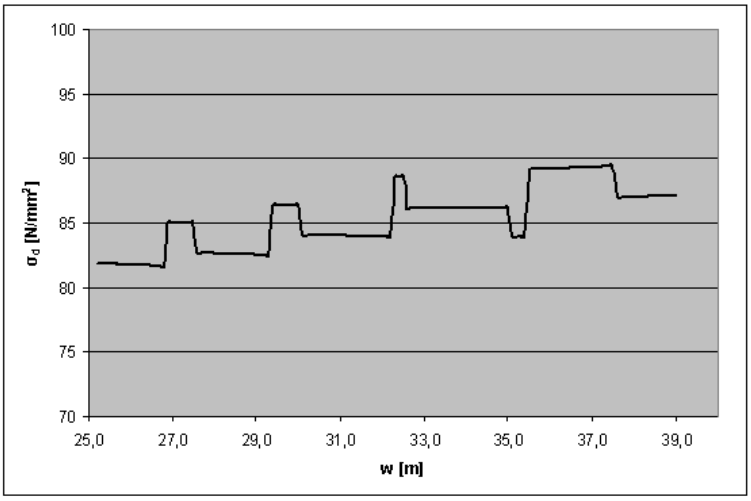



(a) Concrete

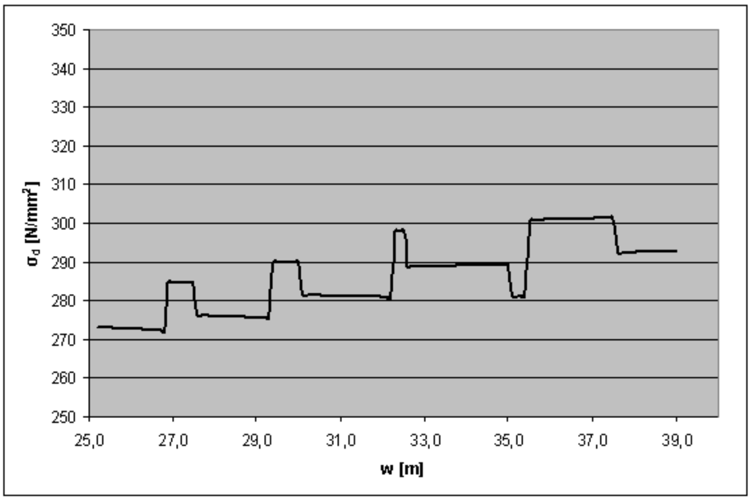



(b) Steel

**Figure 8.9:** *Moments of inertia shape C*




(a) Column stresses in a 390 metres tall concrete skyscraper




(b) Column stresses in a 535 metres tall steel skyscraper

**Figure 8.10:** *Compressive stress in columns of building shape C*

**Shape D**
Cross-section shape D, has a cross-shape. The configuration of this cross-section is given in figure 8.11.

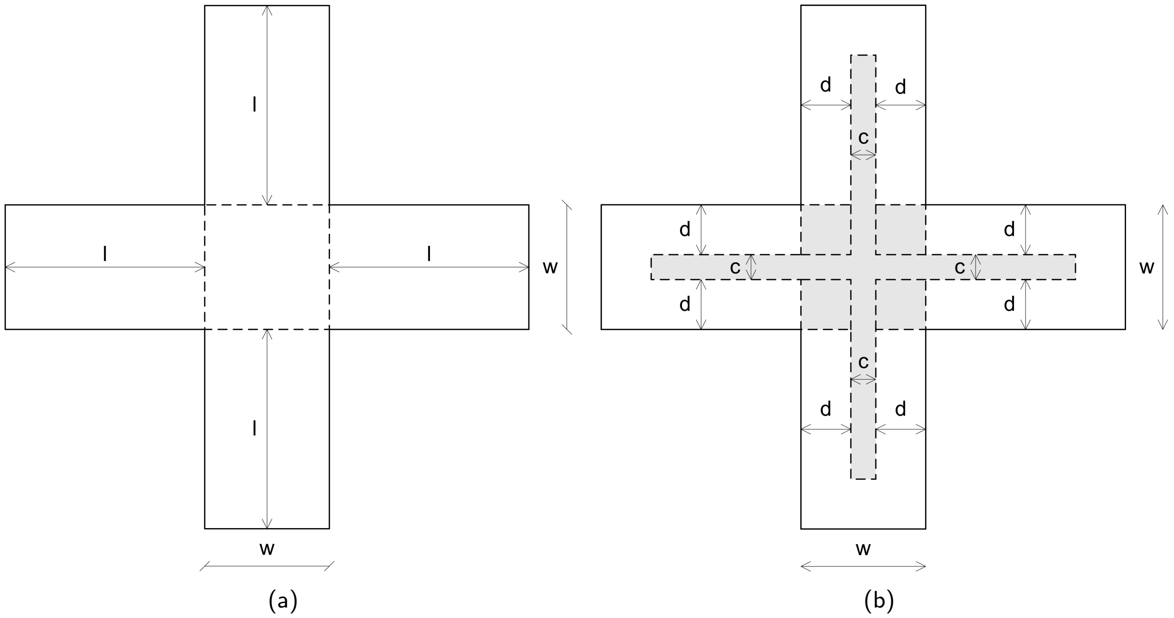
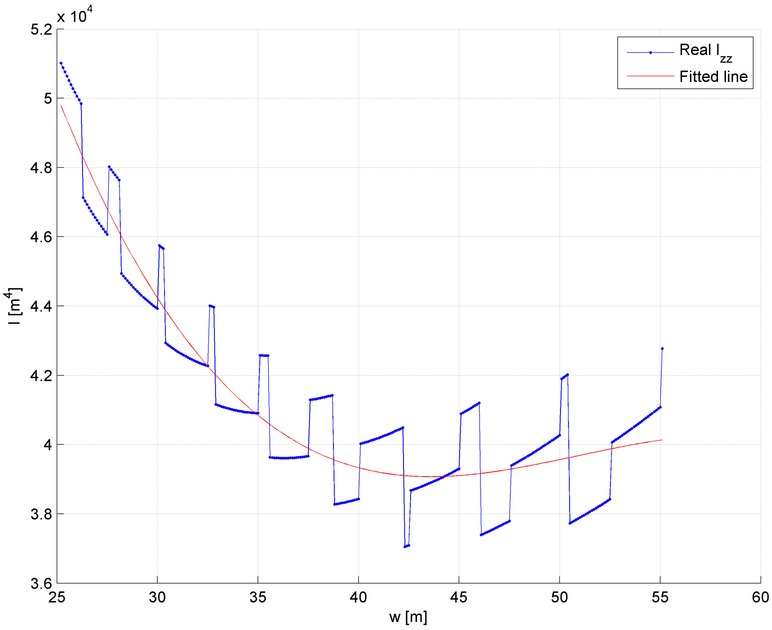


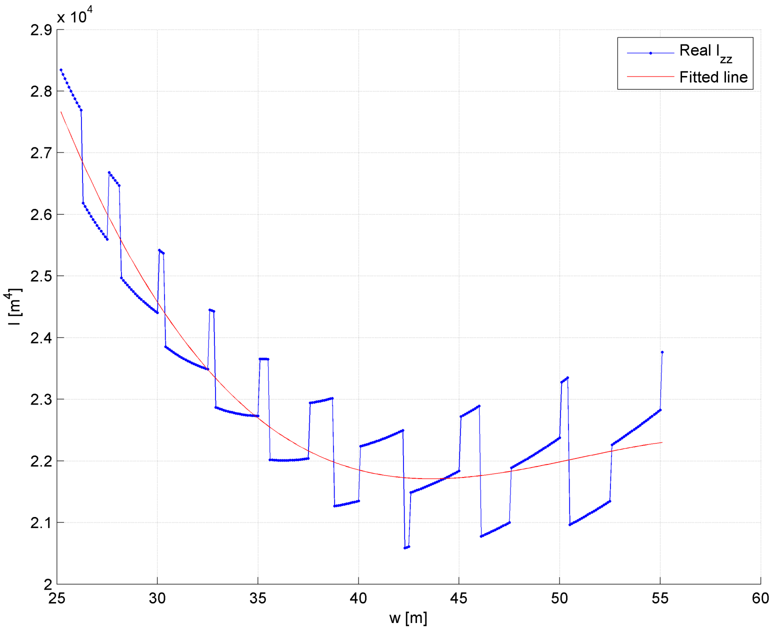

**Figure 8.11:** *Geometry of shape D*

For this cross-section shape, the moments of inertia are equal in both directions, i.e. $I_{zz} = I_{yy}$. Like building shape C, the cross-section's moments of inertia decrease when variable w is increasing (see figure 8.12). This is caused by the decreasing length of variable l.

When we look at the graphs in figure 8.13, we notice something remarkable. Despite the cross-section's decreasing moments of inertia, the compressive stresses in the columns of the building decrease as well. One would expect that the stresses in the columns increase. However, the graphs in figure 8.13 tell us otherwise. This phenomenon can be explained by the fact that the favourable effect of the larger moment of inertia, is counteracted by the larger lateral forces acting on the structure due to a larger facade surface which is subjected to the wind. The compressive stresses in the columns are the smallest when variable *w* has the maximum value of 55.2 metres and variable l equals to zero, in other words, when the building's cross-section has a rectangular shape. The cross-shaped footprint has no structural advantage. Therefore, the maximum height which can be achieved with this building shape is equal to that of a "normal" rectangular shaped building: 420 metres for a concrete structure and 585 metres when the building is constructed out of steel.

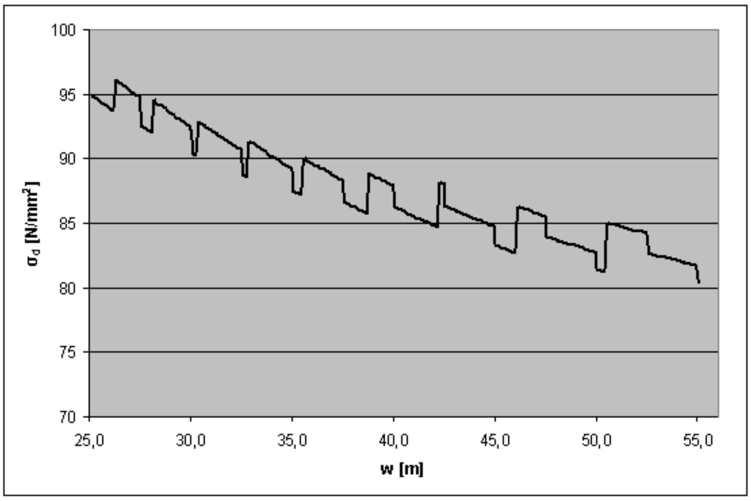



(a) Concrete

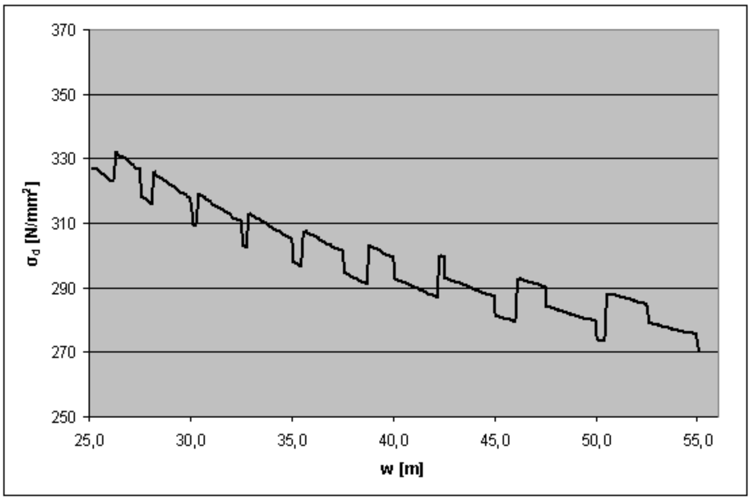



(b) Steel

**Figure 8.12:** *Moments of inertia shape D*




(a) Column stresses in a 420 metres tall concrete skyscraper




(b) Column stresses in a 585 metres tall steel skyscraper

**Figure 8.13:** *Compressive stress in columns of building shape D*

**Shape E**

Cross-section shape E, has a T-shape. The configuration of this cross-section is given in figure 8.14.

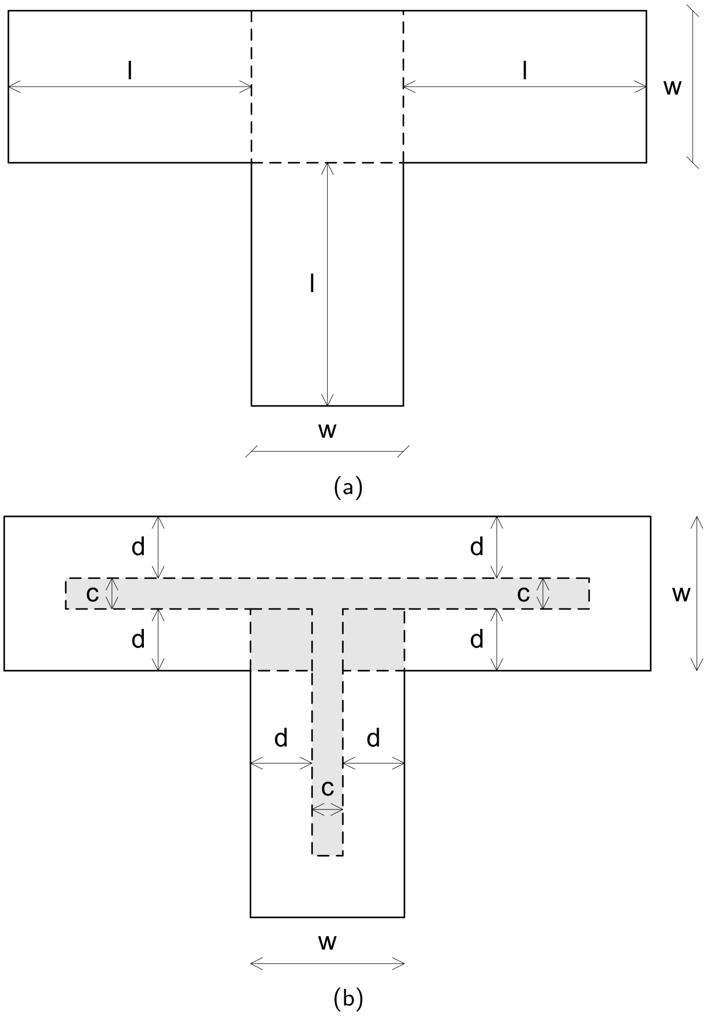
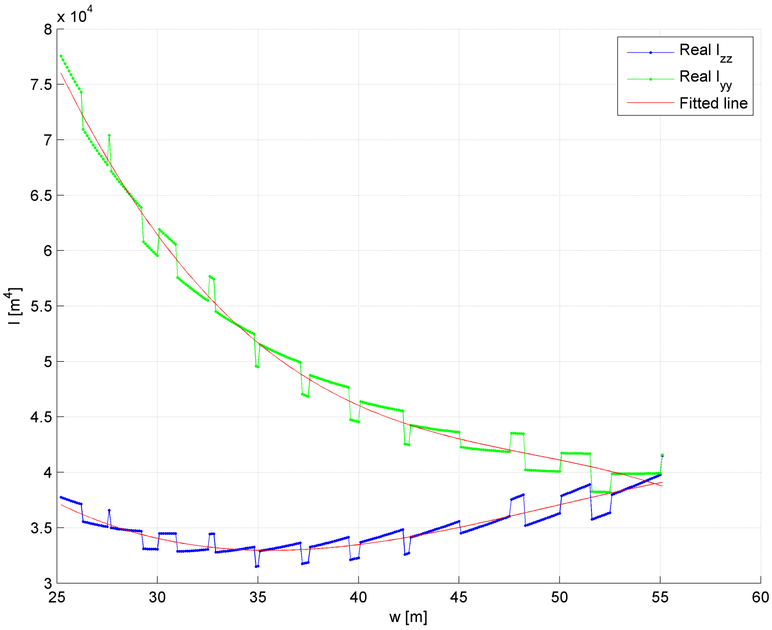


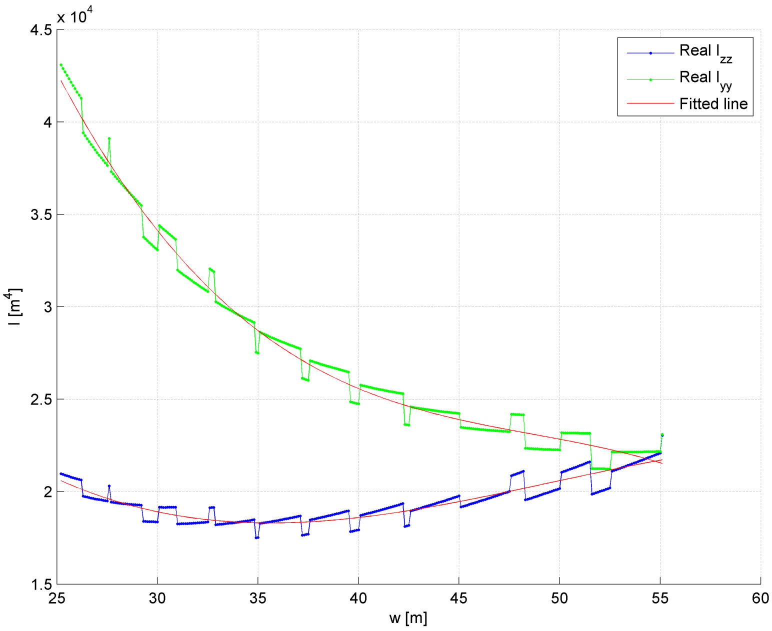

**Figure 8.14:** *Geometry of shape E*

Figure 8.15 shows the change in the cross-section's moments of inertia. When w has reached its maximum value of 55.2 metres, the skyscraper's cross-section has a rectangular shape. This means that the computed values for $\mathrm{I_{zz}}$ and $\mathrm{I_{yy}}$ are equal. This is confirmed by the graphs in figure 8.15. From these curves can also be read that the cross-section's moment of inertia in the normative direction has the largest value when parameter *l* equals to zero.

From figure 8.16 can be learnt that, similar to shape D, the lowest compressive stresses in the columns of the building are obtained when length *l* equals to zero and variable *w* has the maximum value of 55.2 metres. Like the cross-shape building form, the T-shape building form does not give any structural advantage. Therefore, its limits are similar to those of a rectangular shaped skyscraper; 420 metres for a concrete building and 585 metres for a steel building.

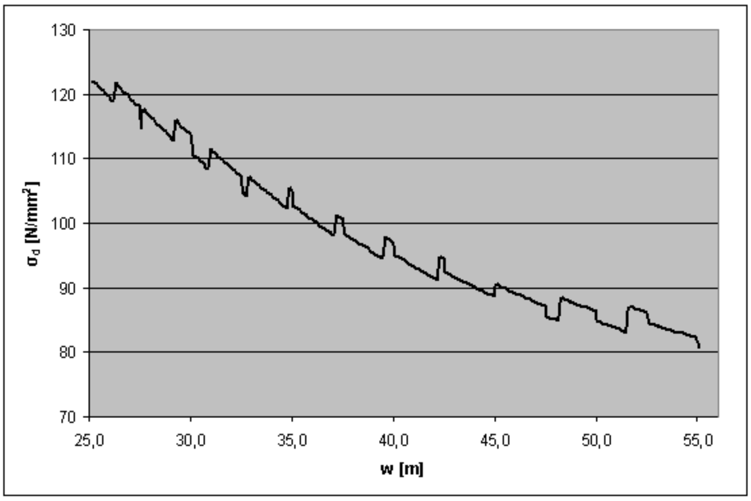



(a) Concrete

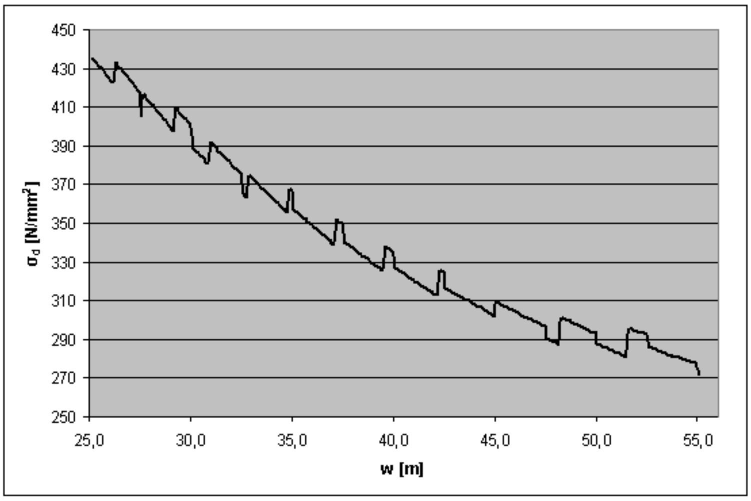



(b) Steel

**Figure 8.15:** *Moments of inertia shape E*




(a) Column stresses in a 420 metres tall concrete skyscraper




(b) Column stresses in a 585 metres tall steel skyscraper

**Figure 8.16:** *Compressive stress in columns of building shape E*

**Shape F**

This cross-section has a Y-shape. The configuration of this cross-section is given in figure 8.17.

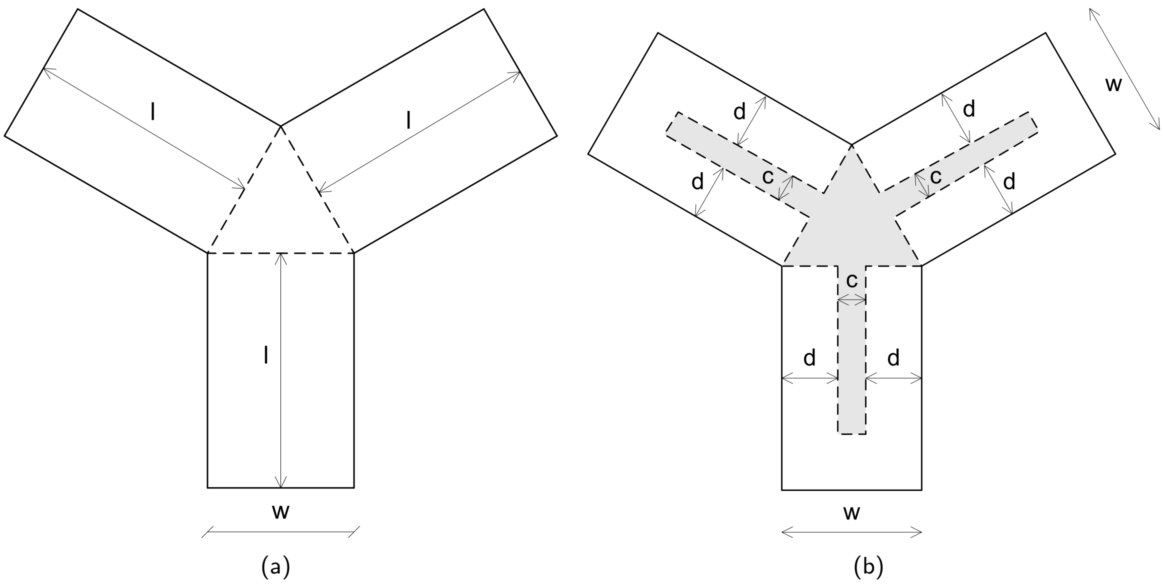



**Figure 8.17:** *Shape F*

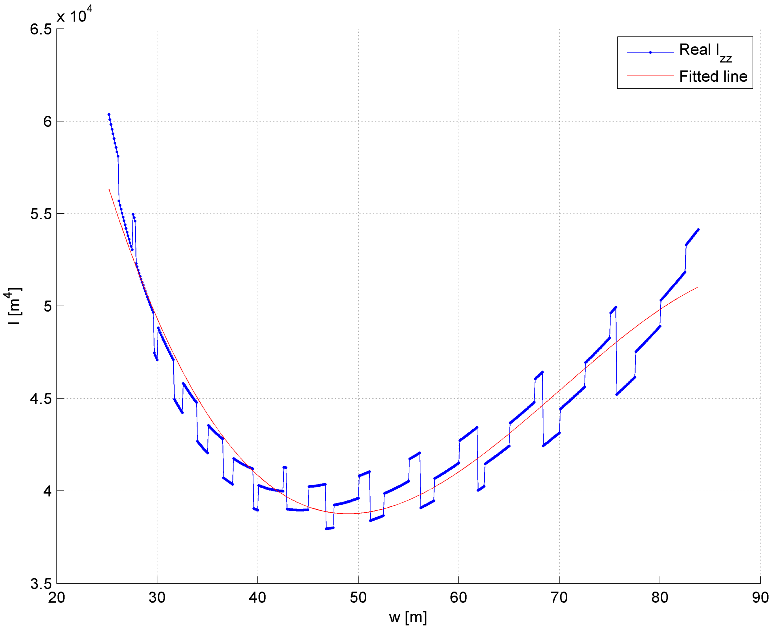
Figure 8.18 depicts the changes in the cross-section's moments of inertia. Due to its Y-shape, the values of $\mathrm{I_{zz}}$ and $\mathrm{I_{yy}}$ are the same. The curves show that at first the cross-section's moment of inertia decreases. However, when w continues to increase, the moment of inertia starts to increase again.

The graphs in figure 8.19 depict the changes of the compressive stresses in the columns of the building. Note that this time, the graphs contain two lines. The *bold line* depicts the compressive stresses in the lower columns of the cross-section, while the *thin line* depicts the changes in the compressive stresses in the upper columns of the cross-section. Let's recapitulate: the compressive stresses in the columns are computed by using equation 8.1.

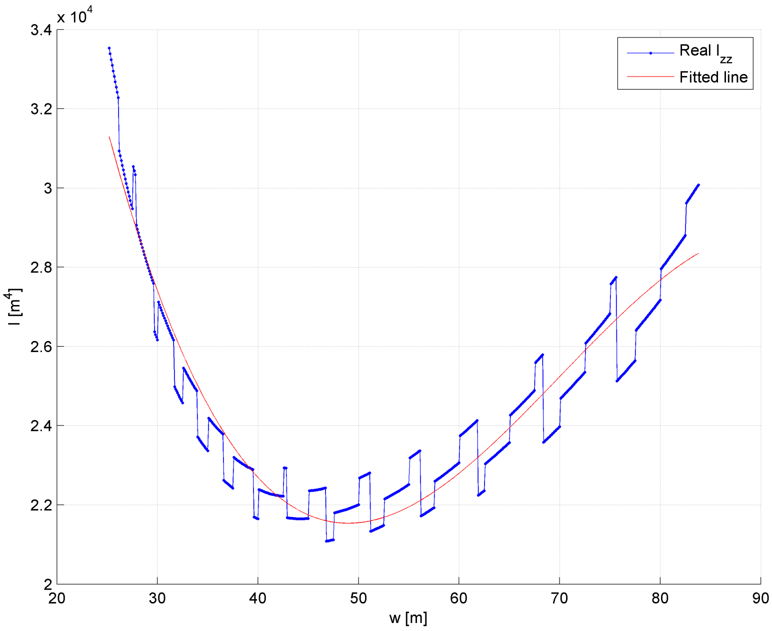
$$\sigma_{M;d} = \frac{M_d \times z_i}{I} \tag{8.1}$$

With:
$M_d$ = Overturning moment at the base of the building
$z_i$ = Distance between the median of the cross-section and the outermost fibre
$I$ = Moment of inertia of the building's cross-section

A larger value for $\mathrm{z_i}$ results in higher stresses. This means that the columns which are the furthest from the median, have to accommodate the largest compressive forces. Since this building form in asymmetrical, the lower columns of the cross-section are further away from the median than the upper columns, i.e. the lower columns have to accommodate larger compressive forces. However, when variable w is increased, the cross-section's median shifts downwards. In the most extreme case that *l* equals



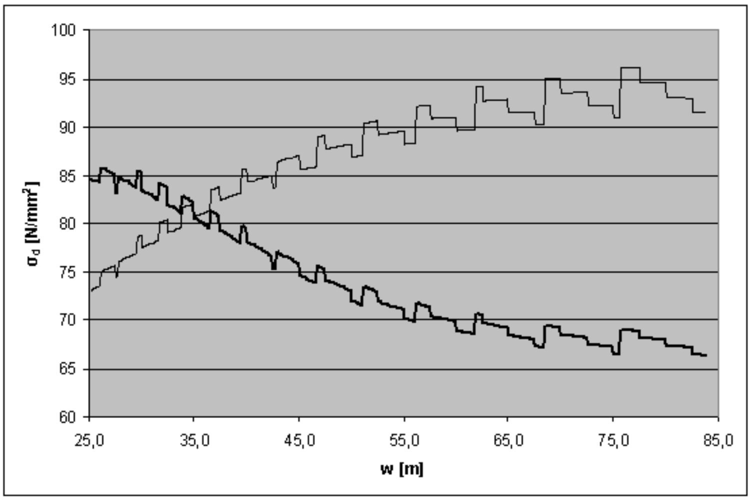

(a) Concrete



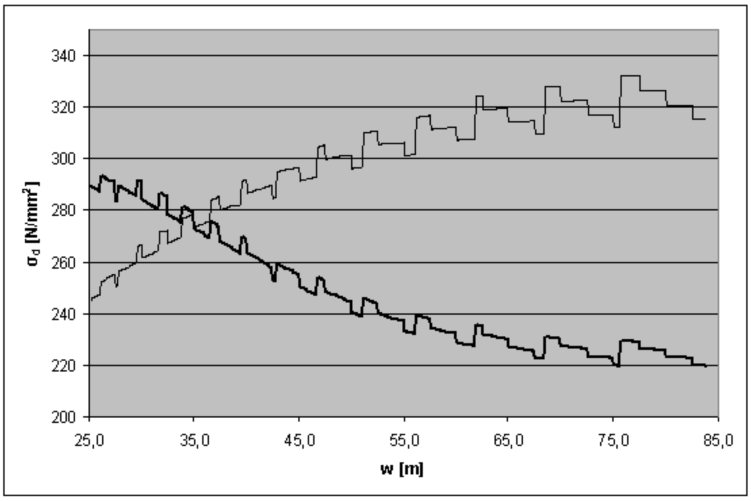

(b) Steel

**Figure 8.18:** *Moments of inertia shape F*

to zero, the lower facade columns are closer to the cross-section's median than the upper columns, i.e. exactly the opposite. So, in this case, the upper columns have to accommodate higher compressive stresses. The most efficient situation would be when the median lies exactly in the middle of the cross-section. In this case, wind loads from each side will result in similar compressive stresses in the columns. From the graphs in figure 8.19 can be read that this occurs when variable *w* equals to 35 metres (at the point of intersection).
Now that the cross-section's most efficient geometry has been established, the limits to the Y-shape building form can be determined. These are: 370 metres for a concrete structure and 510 metres for a steel structures. At these heights the maximum allowable compressive stress is reached for both materials.




(a) Column stresses in a 370 metres tall concrete skyscraper




(b) Column stresses in a 510 metres tall steel skyscraper

**Figure 8.19:** *Compressive stress in columns of building shape F*

**Shape G**

Shape G has a shifted rectangular shape. The configuration of this cross-section is given in figure 8.20.

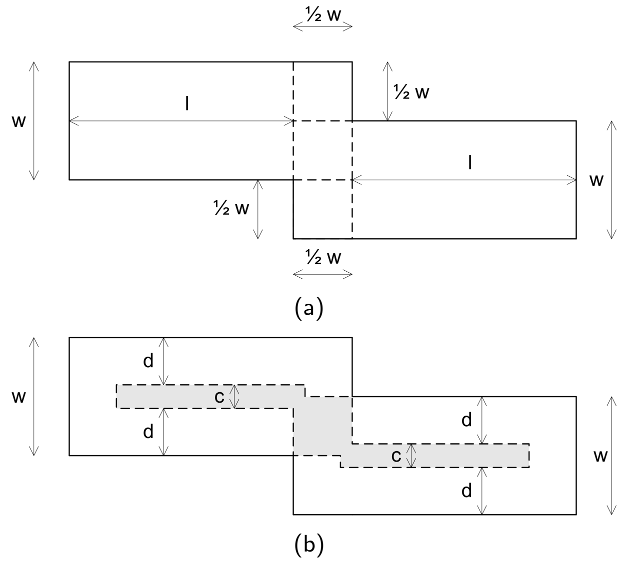
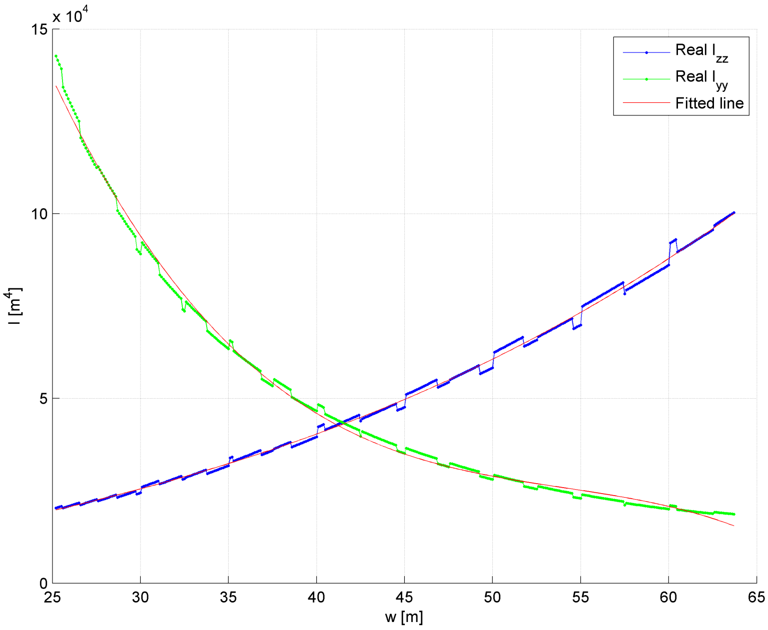


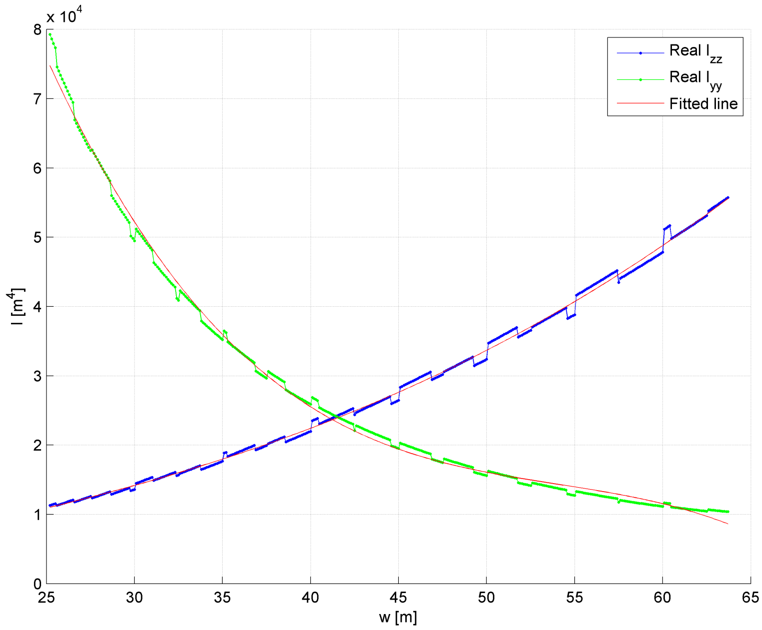

**Figure 8.20:** *Shape G*

The changes in the moments of inertia of this building form are given in figure 8.21. Due to the decrease of variable *l*, the form's moment of inertia in the y-direction will decrease. By contrast, due to the increase of variable *w*, the moment of inertia in the z-direction will increase. When variable *w* has a value of approximately 41.5 metres, the values for $\mathrm{I_{zz}}$ and $\mathrm{I_{yy}}$ are equal. Because the wind can blow from both directions, equal values for the cross-section's $\mathrm{I_{zz}}$ and $\mathrm{I_{yy}}$ are desirable.

Holding on to the building width *w* of 41.5 metres, we can determine the limits to the height of this building form. The limits to the concrete and steel load-bearing structure are respectively 400 and 550 metres. Figure 8.22 depicts the corresponding graphs.

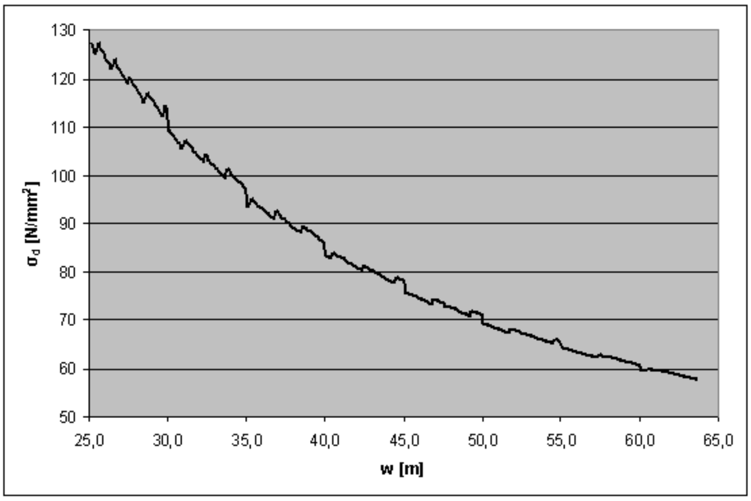



(a) Concrete

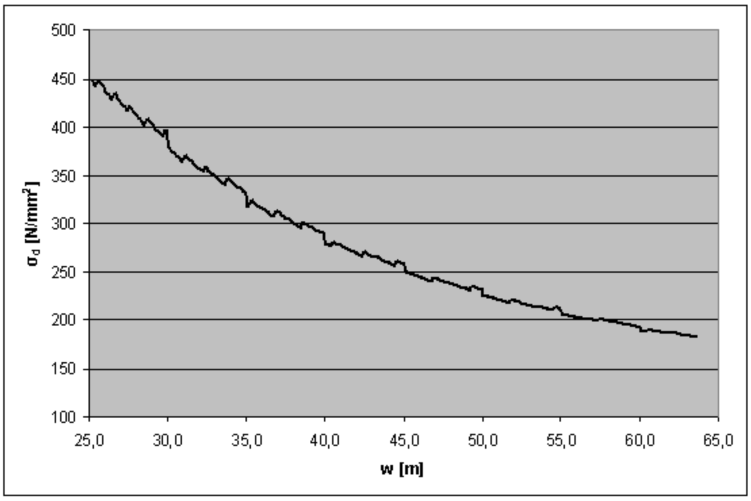



(b) Steel

**Figure 8.21:** *Moments of inertia shape G*




(a) Column stresses in a 400 metres tall concrete skyscraper




(b) Column stresses in a 550 metres tall steel skyscraper

**Figure 8.22:** *Compressive stress in columns of building shape G*

**Shape H**

A higher skyscraper can also be achieved by giving the building a round shape. The improved behaviour of this building form is not caused by its larger moment of inertia, but by its shape. Due to the round shape of the building, the wind can easily flow around it which results in a lower wind load on the structure.

The geometry of the building form is given in figure 8.23.

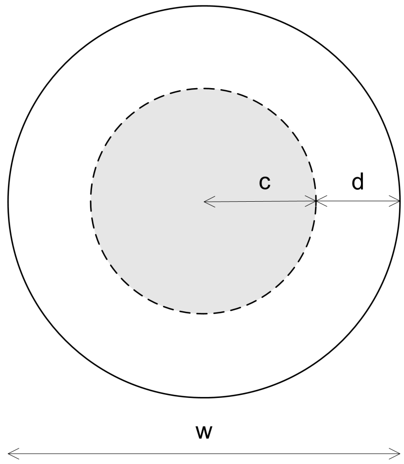



**Figure 8.23:** *Shape H*

Like in the previous cases, the gross floor area of the footprint is 3047 $\mathrm{m}^2$. This means that the diameter *w* of the building, will be 62.3 metres. Again, the centre-to-centre distance of the facade columns is 2.5 metres.

Since the diameter w can not be varied without changing the gross floor area of the footprint, the cross-section's moments of inertia are fixed. Due to the round shape of the building, the values for $\mathrm{I_{zz}}$ and $\mathrm{I_{yy}}$ are the same. For a concrete skyscraper the moment of inertia is $3.06 \times 10^4\ \mathrm{m}^4$. In case of steel columns, the building's moment of inertia will be $1.70 \times 10^4\ \mathrm{m}^4$.

For calculating the wind forces on the structure, European building code EN 1991-1-4 is used. For computing the wind forces on a round shaped building, the very same procedure as described in appendix B can be followed. The favourable effect of the round shape of the tower is taken into account by the force coefficient, $\mathrm{c_f}$. Before, this factor was calculated by using the following formula:

$$c_f = c_{f,0} \times \psi_r \times \psi_\lambda \tag{8.2}$$

With:
$c_{f,0}$ = force coefficient
$\psi_r$ = reduction factor square sections
$\psi_\lambda$ = end-effect factor

For round shaped buildings, the force coefficient is determined by formula 8.3.

$$c_f = c_{f,0} \times \psi_\lambda \tag{8.3}$$

With:
$c_{f,0}$ = force coefficient
$\psi_\lambda$ = end-effect factor

The force coefficient $c_{f,0}$ is determined by using the Reynolds numbers *Re*.
The Reynolds number is calculated by the following equation:

$$Re = \frac{b \times v(z_e)}{\nu} \tag{8.4}$$

With:
$b$ = diameter
$\nu$ = kinematic viscosity of the air ($\nu = 15 \times 10^{-6} \mathrm{m^2/s}$)
$v(z_e)$ = peak wind velocity

The peak wind velocity $v(z_e)$ is calculated by equation 8.5 below.

$$v(z_e) = \sqrt{\frac{2q_p}{\rho}} \tag{8.5}$$

With:
$q_p$ = peak velocity pressure, calculated according to appendix B
$\rho$ = air density, which is 1.25 $\mathrm{kg/m^3}$

After the Reynolds number is computed, the corresponding force coefficient $c_{f,0}$ can be read from the graph given in figure 8.24. Values for the equivalent surface roughness *k* are given in table 8.1. Parameter *b* is the diameter of the building.

In this case, the force coefficient $\mathrm{c_f}$ will be approximately 0.8. Next, the wind forces on the structure can be determined. After this, the vertical stresses in the columns due to the horizontal wind loads can be computed. After adding up the vertical stresses due to the vertical loads, the total vertical column stresses have been determined. Next, the limit to the load-bearing structure of the cylindrical skyscraper can be found. For a concrete structure this limit lies at 480 metres. For a steel structure this limit lies at a height of approximately 685 metres.

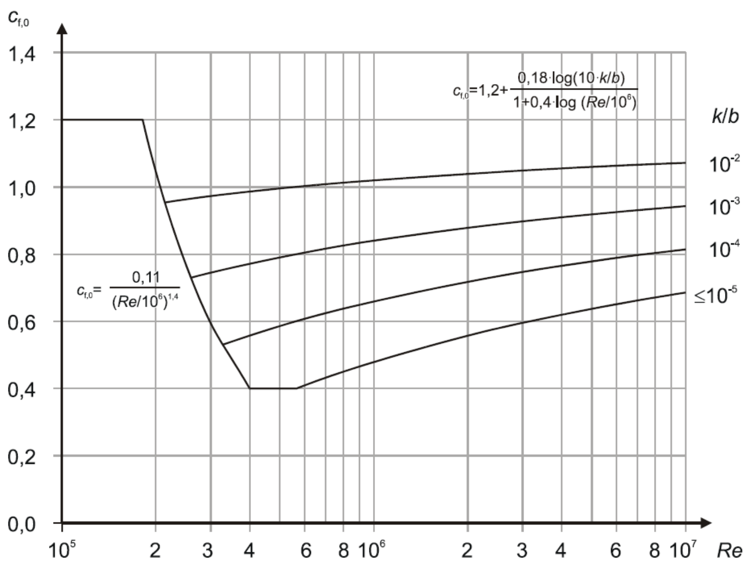



**Figure 8.24:** *Force coefficient for circular cylinders for different equivalent roughness k/b*

| Type of surface | Equivalent roughness ***k*** mm | Type of surface | Equivalent roughness ***k*** mm |
|---|---|---|---|
| glass | 0,0015 | smooth concrete | 0,2 |
| polished metal | 0,002 | planed wood | 0,5 |
| fine paint | 0,006 | rough concrete | 1,0 |
| spray paint | 0,02 | rough sawn wood | 2,0 |
| bright steel | 0,05 | rust | 2,0 |
| cast iron | 0,2 | brickwork | 3,0 |
| galvanised steel | 0,2 | | |

**Table 8.1:** *equivalent surface roughness for several materials*

**First Findings**
In table 8.2 below, an overview of the limits to the load-bearing structure of each building form is given. Apart from building shape H, all the other building shapes show a worse structural behaviour than an "ordinary" rectangular shaped skyscraper. It can be concluded that changing the shape of the building's cross-section does not improve its structural behaviour. Shape H tells us that the only way to stretch the limit to the height of a skyscraper is by changing the cross-section's form in such a way that the lateral wind forces on the structure are reduced.

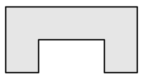
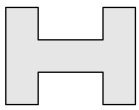
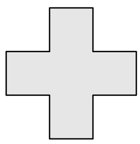
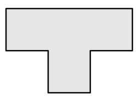
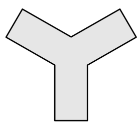
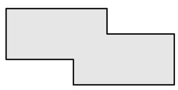
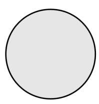
| | Concrete | Steel |
|---|---|---|
| B:  | *370 m* | *500 m* |
| C:  | *390 m* | *535 m* |
| D:  | *420 m* | *585 m* |
| E:  | *420 m* | *585 m* |
| F:  | *370 m* | *510 m* |
| G:  | *400 m* | *550 m* |
| H:  | *480 m* | *685 m* |

**Table 8.2:** *Overview of the limits to the load-bearing structure of each building form*

### 8.1.2 Comfort

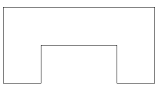
In chapter 6 the comfort demand by the skyscraper's occupants has been explained. It was concluded that the accelerations due to the across-wind are more problematic than the accelerations which occur due to along-winds. To determine the across-wind acceleration, the following equation was used:

$$a_w = f_e^2 \times g_p \left(b_m \times d_m\right)^{\frac{1}{2}} \left(\frac{a_r}{\rho g \sqrt{\beta}}\right) \tag{8.6}$$

With:
$a_w$ = peak acceleration at the top of the building
$f_e$ = eigenfrequency of the building
$g_p$ = peak factor
$b_m$ = the average width of the building
$d_m$ = the average depth of the building
$a_r$ = factor dependent on the mean wind speed at the top of the building
$\rho$ = average density of the building in kg/$\mathrm{m}^3$
$g$ = acceleration due to gravity in m/$\mathrm{s}^2$
$\beta$ = structural damping ratio

The occurring acceleration depends on the eigenfrequency of the building. This eigenfrequency is dependent on building's moment of inertia. In the previous subsection the moment of inertia for each building shape has been calculated. Using equation 8.6, the across-wind accelerations can be easily determined for each of the building shapes. Since the gross floor area of the building shapes is equal to the one of the benchmark skyscraper in chapter 6, it is assumed that the mass of the building remains the same as well; $11.7 \times 10^3 \mathrm{kN/m}$.

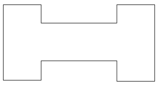
**Shape B**



Changing the geometry of building shape B had no clear structural advantage. However, the larger the moment of inertia of the building, the smaller the accelerations will be at the top of the building. Therefore the geometry with the largest moment of inertia is chosen, i.e. when variable l has its maximum value of 17.6 metres.
With this geometry, the moment of inertia for the concrete cross-section is $2.12 \times 10^4 \mathrm{m}^4$ and the moment of inertia for the steel cross-section is $1.17 \times 10^4 \mathrm{m}^4$. These values are including the factor $\gamma$, for taking into account the shear-lag effect. Like in chapter 6, this factor is assumed to be 0.75. The values for $\mathrm{b_m}$ and $\mathrm{d_m}$ are respectively 85.6 metres and 42.8 metres.
Although we consider a stronger concrete mixture than in chapter 6, we assume that the Young's modulus of the cracked concrete remains 13,200 $\mathrm{N/mm}^2$. The Young's modulus of steel is 210,000 $\mathrm{N/mm}^2$.

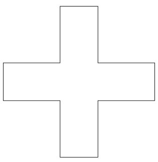
Following the calculation procedure given in chapter 6, leads to a limit of 380 metres for a concrete building and 540 metres for a steel building.

**Shape C**



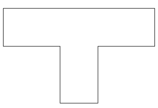
Structurally, the best result was obtained when variable l has a maximum value. The corresponding moments of inertia are $2.70 \times 10^4 \mathrm{m}^4$ for a concrete load-bearing structure and $1.50 \times 10^4 \mathrm{m}^4$ for a steel load-bearing structure. These values are again including factor $\gamma$ for the shear-lag effect. The values for $\mathrm{b_m}$ and $\mathrm{d_m}$ are respectively 73.9 metres and 48.7 metres.
This results in limits of 400 metres for a concrete building and 570 metres for a steel building.

**Shape D**



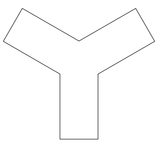
The structural limit of building form D is obtained when variable w has its maximum value, i.e. when the cross-section of the building has a rectangular shape. In this case, the values for $\mathrm{b_m}$ and $\mathrm{d_m}$ are both 55.2 metres. The limits are equal to those already found in chapter 6 of this report; 380 metres for a concrete load-bearing structure and 520 metres for a steel load-bearing structure.

**Shape E**



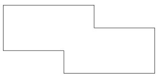
Similar to shape D, structurally, best results were obtained when the building's cross-section has a rectangular shape. Therefore, the same limits with regard to the comfort criterion are found; 380 metres for a concrete load-bearing structure and 520 metres for a steel load-bearing structure.

**Shape F**



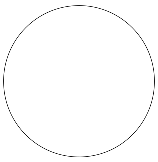
Structurally, the best result was obtained when variable *w* equals to 35.0 metres. The corresponding moments of inertia are $3.15 \times 10^4 \mathrm{m}^4$ for a concrete load-bearing structure and $1.75 \times 10^4 \mathrm{m}^4$ for a steel load-bearing structure (including $\gamma$). $\mathrm{b_m}$ and $\mathrm{d_m}$ are both 76.5 metres.
The corresponding limits which are found are: 510 metres for a concrete load-bearing structure and 740 for a steel load-bearing structure.

**Shape G**



The best structural result was obtained when variable *w* equals to 41.5 metres. Taking into account the shear-lag effect, the moments of inertia are $3.27 \times 10^4 \mathrm{m}^4$ for a concrete structure and $1.82 \times 10^4 \mathrm{m}^4$ for a steel structure. $\mathrm{b_m}$ and $\mathrm{d_m}$ are both 63.0 metres.
The limits which are found are: 440 metres for a concrete load-bearing structure and 620 for a steel load-bearing structure.

**Shape H**



The moments of inertia (including the shear-lag effect) for the round shaped cross-section are $2.30 \times 10^4 \mathrm{m}^4$ for a concrete structure and $1.28 \times 10^4 \mathrm{m}^4$ for a steel structure. $\mathrm{b_m}$ and $\mathrm{d_m}$ are both 62.3 metres.
The limits which are found are: 400 metres for a concrete load-bearing structure and 570 metres for a steel load-bearing structure.
It has to be noted that the round shape of the building can have a favourable effect on reducing the accelerations at the top of the skyscraper. However no formula's can be found in the literature to confirm this thought.

**First Findings**
Table 8.3 gives an overview of the computed limits with regard to the comfort criterion.
Remarkable is that higher altitudes seem to be achievable by applying a steel load-bearing structure. However, when we look at the building practice, we see that steel structures often need dampers to dampen the accelerations of the building, while concrete buildings of similar height do not need such measures. This means that the results found here, differ from reality. This discrepancy can be explained by the following reason: In chapter 6 the dimensions for the concrete and steel columns were assumed. However, the strength of the steel column is much higher than the strength of the concrete column. So despite the steel column's smaller moments of inertia, it shows a stiffer behaviour than a concrete column. It is also not clear whether equation 8.6 can be fully relied on when comparing both materials. The outcome of the equation is dependent on the applied structural damping ratio $\beta$. Establishing the value of this ratio is unclear.

In table 8.4 the limits found for the load-bearing structure are subtracted from the limits with regard to the comfort criterion. So, a positive figure means that the comfort limit lies higher than the limit found for the load-bearing structure of the skyscraper. From the table can be concluded that changing the building's footprint has a favourable effect on the skyscraper's limit with regard to the comfort criterion. The cross-section's larger moment of inertia and the larger width of the building are the causes of this. Besides shape H, also the building shapes D and E show negative figures. However, we call to mind that these limits were achieved when these shapes have a rectangular form.

It was mentioned before that it is likely that the comfort limits to shape H are underestimated because the favourable effect of the round form is not taken into account.

From table 8.4 can be concluded that the Y-shape proves to be the most rigid building shape, because it shows the largest positive value. There is still plenty of room for extending the height of the building without the necessity to apply dampers to dampen the building's accelerations.

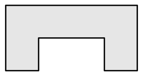
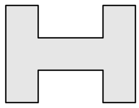
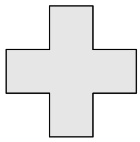
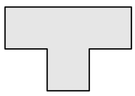
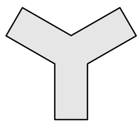
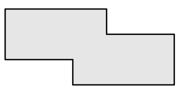
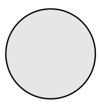
| | Concrete | Steel |
|---|---|---|
| B:  | *380 m* | *540 m* |
| C:  | *400 m* | *570 m* |
| D:  | *380 m* | *520 m* |
| E:  | *380 m* | *520 m* |
| F:  | *510 m* | *740 m* |
| G:  | *440 m* | *620 m* |
| H:  | *400 m* | *570 m* |

**Table 8.3:** *Overview of the limits of each building form with regard to the comfort criterion*

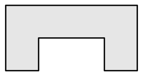
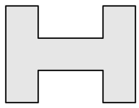
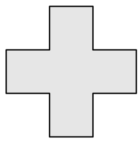
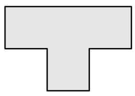
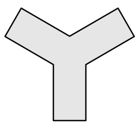
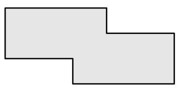
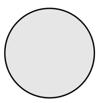
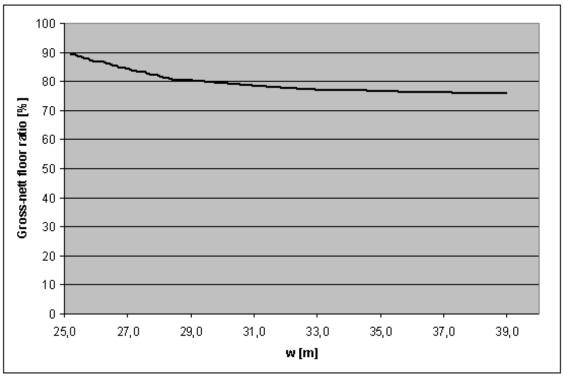
| | Concrete | Steel |
|---|---|---|
| B:  | *+10 m* | *+40 m* |
| C:  | *+10 m* | *+35 m* |
| D:  | *-40 m* | *-65 m* |
| E:  | *-40 m* | *-65 m* |
| F:  | *+140 m* | *+230 m* |
| G:  | *+40 m* | *+70 m* |
| H:  | *-80 m* | *-115 m* |

**Table 8.4:** *Comparison between the limits to the load-bearing structure and the limits with regard the to comfort criterion*

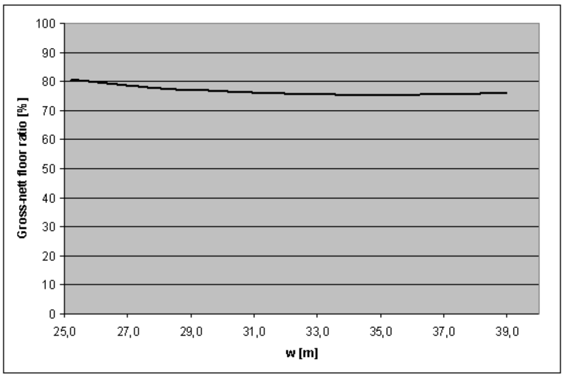
### 8.1.3 Economical Feasibility

The last aspect which is influenced by changing the building's footprint, is the economical feasibility of the building. Previously has been decided that floor space is considered lettable, when it is sunlit. By changing the shape of the building's cross-section, the floor surface which is not lit by sun can be decreased. This will result in a more favourable gross-nett floor ratio. In figure 8.25, six graphs are given which depict the changes in the gross-nett floor ratio of each building shape, when variable $w$ is increased.
All the graphs indicate that the best nett-gross floor ratios are obtained when the value of variable $w$ is minimum. However, except for building shape C, minimum values for w have proven to be structurally undesirable for the other building shapes. For building shape B, the best structural result is obtained when $w$ equals to 29 metres. From the graph in figure 8.25a can be read that the corresponding gross-nett floor ratio is approximately 80%.
For a minimum value of w, shape C has a gross-nett floor ratio of approximately 80% (figure 8.25b).
For building shapes D and E, the best structural result is obtained when the footprint of the building has a rectangular shape. In this situation $w$ has its maximum value of 55.2 metres and the gross-nett floor ratios of the building shapes are equal to the ratio of the benchmark skyscraper; 70%. This is confirmed by the graphs in figures 8.25c and 8.25d.
A building width $w$ of 35 metres gives the best structural results for building shape F. From the graph in figure 8.25e can be read that the corresponding gross-nett floor ratio is approximately 70%.
For building shape G, the best structural result is obtained when w equals to 41.5 metres. From the graph in figure 8.25f can be read that the corresponding gross-nett floor ratio is again approximately 70%.
The gross-nett floor ratio of building shape H is 64.5%.
An overview of the found gross-nett floor ratios is given in table 8.5.

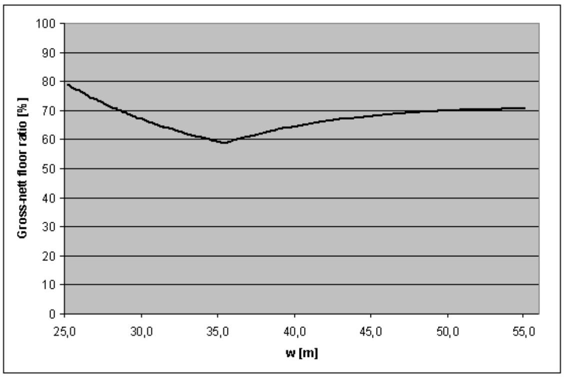
**First Findings**
Except for shapes B and C, the gross-nett floor ratios of the other building shapes are all approximately 70%, similar to the floor ratio of the benchmark skyscraper. The conclusion is that changing the form of the building's footprint proves to be less favourable for the building's gross-nett floor ratio than what was originally assumed. This is because the structural optimum of the building forms does not coincide with the optimum found for the gross-nett floor ratios of the shapes. Only with building shapes B and C, an improvement of 10% can be realised.



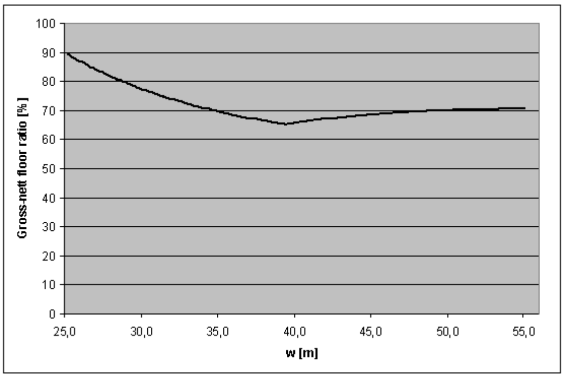

(a) Shape B



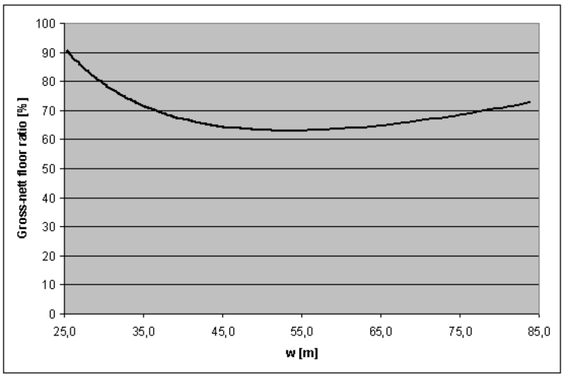

(b) Shape C



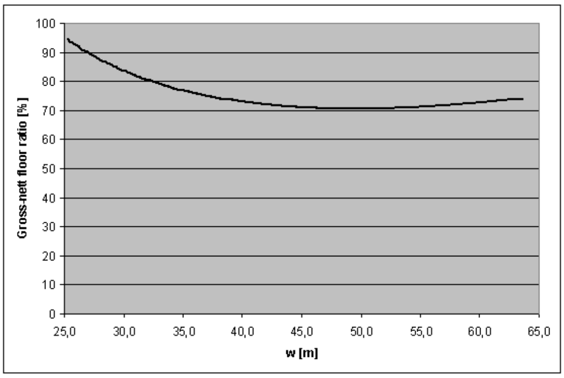

(c) Shape D




(d) Shape E




(e) Shape F




(f) Shape G

**Figure 8.25:** *Changes in the gross-nett floor ratio*

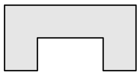
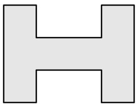
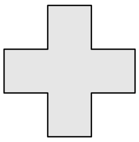
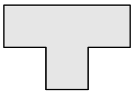
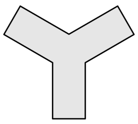
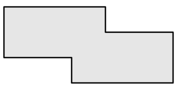
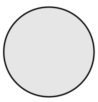
| | Gross-nett floor ratio |
|---|---|
| B:  | ≈ 80 % |
| C:  | ≈ 80 % |
| D:  | ≈ 70 % |
| E:  | ≈ 70 % |
| F:  | ≈ 70 % |
| G:  | ≈ 70 % |
| H:  | ≈ 65 % |

**Table 8.5:** *Overview of the gross-nett floor ratios of each building shape*

## 8.2 Give the Building a Tapering Shape

By giving the skyscraper a tapering shape, the limits with regard to the the load-bearing structure of the building and the comfort criterion, can be stretched (figure 8.26).

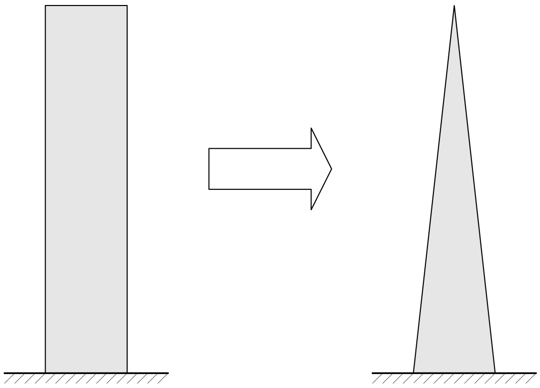


**Figure 8.26:** *Giving the skyscraper a tapering shape*

In the previous section has been discovered that a Y-shape (shape F) behaves rigid and offers the opportunity of constructing taller skyscrapers without the necessity to apply dampers to dampen the building's accelerations. Due to these promising results, shape F will be used for investigating the advantages of giving a skyscraper a tapering shape.

### 8.2.1 Load-bearing structure

By giving the building a tapering shape, the surface exposed to the wind is smaller in comparison to an equally tall straight building. Due to this, the horizontal forces which are acting on the structure are reduced which results in a smaller overturning moment at the base of the structure (see figure 8.27). This means that the vertical column stresses due to the horizontal loads are decreasing, which makes a taller structure feasible.

Due to the tapering shape of the building, the dimensions of the floors are decreasing together with the height of the building. This means that the total floor load per storey is decreasing together with the skyscraper's height.
The total area of facade per storey is decreasing as well. This brings about a reduction in the deadweight of the facade per floor. Since the dimensions of floors are declining, fewer columns are required to support the floors. This will also contribute to a reduction of the skyscraper's deadweight. Of two buildings with a similar height, the vertical loads of the tapering building will be less than those of the straight skyscraper (figure 8.28).

To estimate to what extent the vertical forces are reduced, two simplifications are made. A rectangular block is compared to a cone with a rectangular footprint. From the formulas given in figure 8.29 follows that the volume V of the cone is one-third of the volume of the rectangular block. This is used to

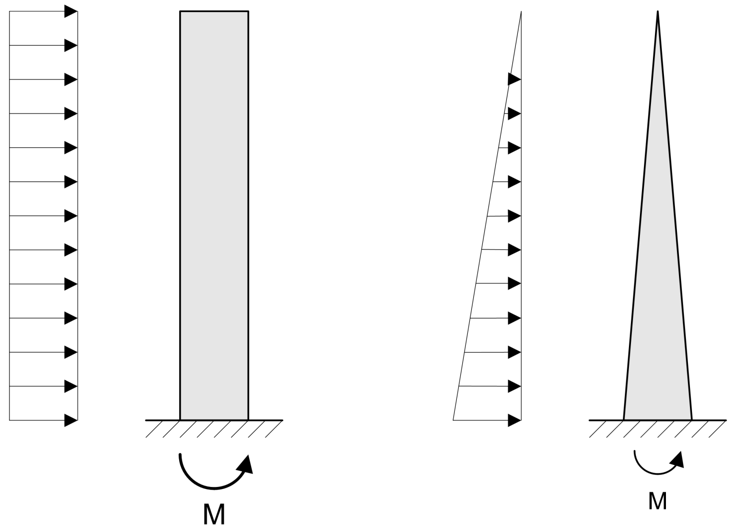



**Figure 8.27:** *Smaller horizontal forces on a tapering skyscraper*

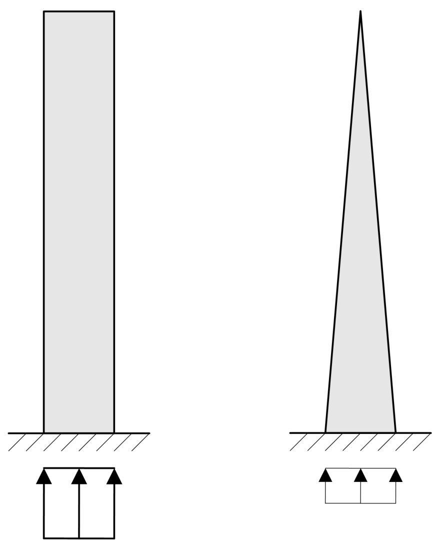
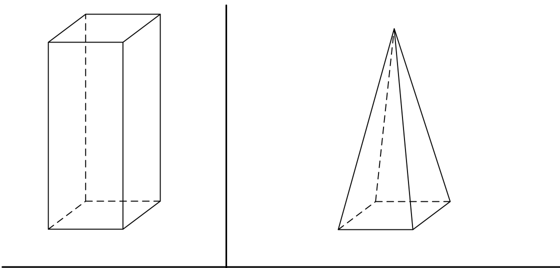


**Figure 8.28:** *Smaller vertical forces*

determine the total floor load of the tapering tower. It is assumed that the floor loads of the tapering tower are one-third of the floor loads of an equally tall straight skyscraper.
In figure 8.29 the formulas for the outer surface S of the geometrical figures are given. When the slenderness ratio h:b lies around 8–10, the outer surface of the cone is half of the outer surface of the rectangular block. Following from this, it is assumed that the deadweight of the facade and the columns of the tapering skyscraper is half of the deadweight of an equally tall straight skyscraper.



| | |
|---|---|
| $V = b^2 \times h$ | $V = \frac{1}{3} \times \left(b^2 h\right)$ |
| $S = 4bh + 2b^2$ | $S = 2\sqrt{h^2 + \frac{1}{4}b^2}\ b + b^2$ |

**Figure 8.29:** *Smaller vertical forces*

In the previous section was computed that the Y-shape behaves most stiff and thus offers the best opportunity to stretch the height of the building. Therefore, we will only look at this building shape, shape F, when considering the effects of the tapering skyscraper. The very same calculation procedure as before is used to determine the limit to the load-bearing structure, however, this time the reduced horizontal and vertical loads are considered.
The following results are obtained from the calculation procedure: a limit of *670 metres* when the skyscraper is constructed out of concrete and *900 metres* when the building is constructed out of steel. These limits are considerably higher than the ones found for the non-tapering skyscraper. However, a point of special interest are tensile forces in the building's columns. In order to determine the limit to the load-bearing structure of the building forms, the compressive stress in the leeward columns of the the tube structure are considered. These compressive stresses consist of compressive stresses caused by the vertical and the lateral loads acting on the structure. On the opposite windward side, the lateral loads introduce tensile stresses into the columns of the structure. In the previous considered structures these tensile stresses were always smaller than the compressive stresses caused by the vertical loads acting on the structure. This meant that there was always a resultant compressive stress present in the columns. It is desirable to prevent that columns have to be able to accommodate tensile forces, since this requires special connections. It also means that this tensile force has to be transferred onto the foundation of the building, requiring a different kind of foundation.
In this case, the height of the building has become so large that the tensile stresses in the windward columns exceed the compressive stresses in the columns. In case of the concrete structure a resultant tensile stress of 25.4 $\mathrm{N/mm^2}$ will be present in the windward columns. This resultant tensile stress

will be 103.3 $\mathrm{N/mm^2}$ when the structure is constructed out of steel. Both columns are capable of accommodating these tensile stresses, provided that the concrete columns are fitted with reinforcement. However, as said before, introducing tensile forces into the columns of a building is highly undesirable. A possible solution could be to add more vertical loads to the columns in order to increase the compressive stresses, however this does coincide with the goal of building the tallest skyscraper.

### 8.2.2 Comfort

We concluded before that the deadweight of a tapering skyscraper is smaller than the deadweight of an equally tall straight skyscraper. This was an advantage for the building's load-bearing structure. However, due to the reduced deadweight of the building, the building's eigenfrequency will increase. The straight Y-shaped concrete skyscraper had an eigenfrequency of 0.040 Hz. When we take the reduced deadweight of the tapering tower into account, the eigenfrequency is more than doubled: 0.096 Hz. A larger eigenfrequency leads to more strict limits regarding the building's accelerations. Additionally, due to the decrease in the building's mass, less damping will occur, resulting in higher accelerations at the top. Due to its tapering shape the limits with regard to the comfort criterion of the Y-shaped building drop to 420 metres for a concrete structure and to 660 metres for a steel load-bearing structure. Both limits are lower than the limits found for the straight Y-shaped building, respectively 510 and 740 metres.

However, it is probable that a tapering building shape reduces the building's peak accelerations. This thought is supported by the article of Melbourne and Cheung [29] in which they conclude that a tapering building shape causes reductions in the "Cross-Wind Force Spectrum" of the building. An other clue is the design of the Burj Dubai (see chapter 4). The tapering shape of the building is partly chosen for its favourable effect on reducing the cross-wind accelerations of the tower. This was the outcome of extensive wind tunnel tests which were conducted when designing the Burj. In the existing literature no clear formulas can be found which describe the favourable effect of a tapered building shape. Therefore, a further study into this topic is highly recommended.

### 8.2.3 Economical Feasibility

The tapering shape of the skyscraper will result in a more unfavourable gross-nett floor ratio. However, this loss can be limited by giving the core of the building a tapering shape as well. A tapering core is possible since the smaller floors need less lifts to service them. A smaller building core at higher altitudes is also structurally permissable. A tapering building core is also used in the Burj Dubai. The dimensions of the building core are reduced in three steps as the height of the tower increases. Nevertheless, a tapering building shape will always lead to a small worsening of the skyscraper's gross-nett floor ratio.

## 8.3 Create a Compound Structure

Creating a compound structure can positively influence the limits which are found to the building's load-bearing structure, the comfort criterion and the economical feasibility. When multiple slender towers are

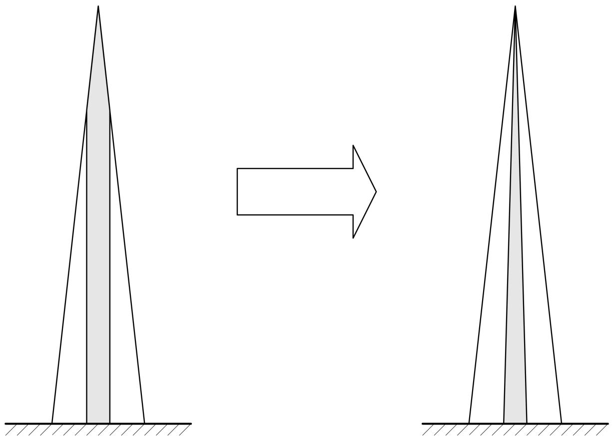


**Figure 8.30:** *Giving the building core a tapering shape*

interconnected they form one structural entity. The internal lever arm of the combined towers is much larger than the the internal lever arm of the towers separate. This means that the compound structure is more efficient in accommodating the lateral forces. This means that by creating a compound structure, the limit imposed by the load-bearing structure can be stretched. Due to the cross-section's increased moment of inertia, the limit with regard to the comfort criterion can be expected to increase as well.

### 8.3.1 Load-bearing structure

The compound structure which will be evaluated is a structure which consists out of four cylindrical towers which are positioned in a rectangular configuration (see figure 8.31). Earlier in this chapter has already been concluded that by giving a building a circular footprint, a higher structure is possible due to a decrease in the wind forces acting on the skyscraper. The centre-to-centre distance between the towers is given by variable $L$. The normative wind direction is when the wind blows from an angle of 45 degrees on the structure (figure 8.32).

First, we assume again that the gross floor surface of each storey is 3047 $m^2$. This means that each of the four towers will have a gross floor surface of approximately 762 $m^2$, which means that the towers have a diameter of 31.1 metres. It is assumed that the clearance between two towers is equal to half the diameter of a tower. This means that the centre-to-centre distance $L$ equals to 1.5 times the diameter of the towers. This ratio is also applied in figure 8.31. This openness of the structure is regarded to be sufficient to let daylight enter in between the towers.

By analogy with the shear-lag effect for Tube Structures, letting the entire cross-section of a compound structure to work as one entity, is virtually impossible to achieve. Due to inevitable deformations of the structures connecting the four towers, the cross-section's moment of inertia is reduced. So, as with the shear-lag effect, the moment of inertia is reduced with a factor $\gamma_{compound}$. Similar to the shear-lag effect, it is assumed that this factor equals to 0.75. By means of Matlab computer software, the moment of inertia of the compound structure can be computed. The computed moment of inertia, including the

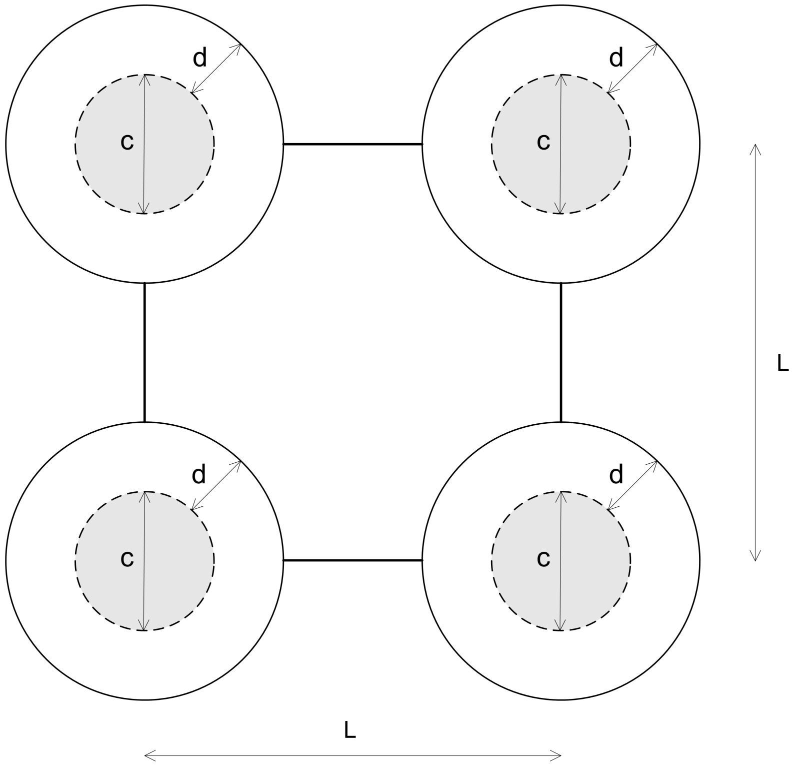



**Figure 8.31:** *Geometry compound structure*

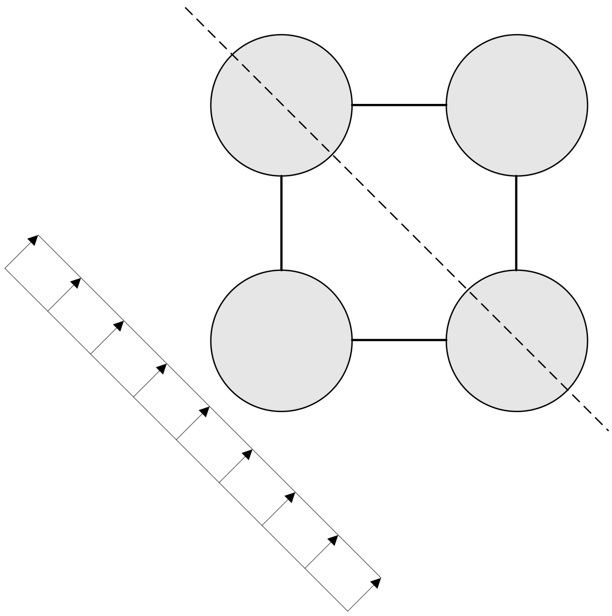


**Figure 8.32:** *The normative wind direction for the compound structure*

reduction factor, is $6.6 \times 10^4$ m$^4$ when the structure is constructed out of concrete and $3.7 \times 10^4$ m$^4$ when it is constructed out of steel.
Using the same calculation procedure as before, the limits to the load-bearing structure of the building can be determined. These limits are: 560 metres for a concrete load-bearing structure and 765 metres for a steel load-bearing structure. When we compare the computed limits with the ones found for shape H (see table 8.2) we see that, like expected, the compound structure gives a structural advantage.

Because the cross-section of this compound structure consists of multiple slender towers, the gross-nett floor ratio, determined by the entry of daylight into the building, is high. This means that there is still a possibility to widen the towers. This will worsen the gross-nett floor ratio, but due to the increased moment of inertia of the cross-section, it is expected that a higher load-bearing structure will be feasible. When each tower has a diameter of 55.3 metres, the gross-nett floor ratio is approximately 70%. The limits to the load-bearing structure of the building are now 600 metres for a concrete and 845 metres for a steel structure. Although these limits are higher than the limits which were found for the compound structure with towers with a diameter of 31.1 metres, the computed limits are lower than the ones found earlier for the tapering skyscraper. However, there is one major advantage which favours a compound structure above a tapering structure. At all times, the compressive column stresses due to the vertical loads acting on the structure, are larger than the tensile stresses caused by the lateral loads. This means that no tensile forces have to be accommodated by the building's columns. This can be considered as a major advantage.

In order to try to approach the limits which were found for the tapering skyscraper, there are two possible options. The *first* option is to lower the gross-nett floor ratio of the building to for example 60%. The *second* option is to increase the centre-to-centre distance in between the towers.
When the gross-nett floor ratio of the building is decreased to 60%, the diameter of the towers is approximately 67.7 metres. The corresponding limits to the load-bearing structure are: 625 metres for a concrete load-bearing structure and 880 metres for a steel load-bearing structure. When the centre-to-centre distance between the towers is increased to twice the diameter instead of 1.5, slightly larger gains are obtained: 635 and 895 metres.
By changing the gross-nett floor ratio of the building or by increasing the centre-to-centre distance in between the towers, the limits to the load-bearing structure can be stretched and approach those found for the tapering skyscraper. However, the structural gain is minimal. This can be explained by the fact that changing the skyscraper's gross-nett floor ratio and the centre-to-centre distance of the towers, will only influence the cross-section's moment of inertia. This will solely result in a reduction of the column stresses caused by the lateral loads acting on the structure. Since the major part of the vertical stresses in the columns is caused by the vertical loads, the contribution to increasing the overall height of the skyscraper is limited. This leads to the following conclusion: the effect of changing the building's shape has worn off and will only result in minor gains in the height of the building. Significant results can only be obtained when the dimensions of the columns themselves are increased.

### 8.3.2 Comfort

The comfort criterion will be evaluated in a similar way as for the single building shapes. First, the eigenfrequency of the compound structure is determined by means of the cross-section's deadweight

and moment of inertia. Next, the across-wind accelerations at the top of the building are computed by using equation 8.6.
In chapter 6, the mass of a building with a gross floor area of 3047 $\mathrm{m}^2$ was estimated to be $11.7 \times 10^3 \mathrm{kN/m}$. The towers in the compound structure have a gross floor area of 2400 $\mathrm{m}^2$. The mass of each tower is estimated at:

$$q_g = \frac{2400}{3047} \times 11.7 \times 10^3 = 9.2 \times 10^3 \mathrm{kN/m} \tag{8.7}$$

This means that the total mass of the four towers combined equals to: $4 \times 9200 = 36.8 \times 10^3 \mathrm{kN/m}$.
The moments of inertia for the concrete and steel load-bearing structures are respectively $3.51 \times 10^5$ $\mathrm{m}^4$ and $1.82 \times 10^5$ $\mathrm{m}^4$. For the average width and depth $b_m$ and $d_m$ of the building, twice the diameter of the tower is taken: 110.6 m.
The limits which are computed are: 900 metres for a concrete load-bearing structure and 1300 metres for a steel load-bearing structure.

Please note that this calculation is purely an indication that the larger values for the cross-section's moments of inertia lead to higher limits with respect to the comfort criterion. It is highly uncertain that the limits found here correspond with the real limits. This because it is not known whether equation 8.6 from the NBCC is also applicable on compound structures. Furthermore, like mentioned before, the cylindrical shape of the towers can have a favourable effect on the occurring accelerations at the top of the building, leading to even higher limits.

### 8.3.3 Economical Feasibility

When the limits to the load-bearing structure of the compound skyscraper were computed, it was concluded that structures with a very favourable gross-nett floor ratio are possible. However, the vertical transportation of people, goods and building services requires space as well. The space required for the vertical transportation increases together with the height of the building. This means that gross-nett floor ratio's above 80% can be considered to be unfeasible. Possibly, the appearance of new lift technologies can make higher nett-gross floor ratios possible in the near future.

## 8.4 Conclusions

Form this chapter a large number of conclusions can be derived.
Changing the shape of the building's footprint gives no structural advantage:
When the building width *w* of shape B (a U-shape) is increased, the cross-section's moment of inertia in the normative direction remains practically constant. Because this moment of inertia is smaller than the one of a regular rectangular cross-section, no structural advantage is achieved.
For shape C (a I-shape) the best results are obtained when the cross-section has a minimum building width *w*. However, the corresponding moment of inertia is smaller than the one of a regular rectangular cross-section, so no structural advantage is achieved.
For shape D (a cross-shape) the moments of inertia are the largest when parameter *w* has its minimum

value of twice the storey depth $d$. However, due to the cross-section's large width the favourable effect of the larger moment of inertia is counterbalanced by the increased lateral forces on the structure. When the compressive forces in the columns are calculated, we see that the smallest stresses are obtained when variable $w$ has the maximum value of 55.2 metres and variable $l$ equals to zero, in other words, when the building's cross-section has a rectangular shape. So no structural advantage is achieved.
The very same reasoning applies to shape E (a T-shape). The large building width means that no structural advantage is achieved despite the cross-section's larger moment of inertia.
The building width $w$ of building shape F (a Y-shape) is established by the need to locate the cross-section's median in the middle of the cross-section. In this case, wind loads from each direction will result in the same compressive stresses in the columns. This results in slightly larger moments of inertia for the cross-section when compared to a regular rectangular shape. However, the larger building width leads again to larger compressive stresses in the building's columns.
The building width of shape G is established by the need to have similar moments of inertia in both directions. This leads to minor gains in the moments of inertia of the cross-section in comparison to a rectangular footprint. And due to the larger width, this building shape shows a worse structural performance.
The only way in which changing the shape of the building's footprint can stretch the limit of the load-bearing structure, is by reducing the wind forces acting on the building. This can for example be achieved by giving the building's footprint a circular shape (shape H).
When considering the comfort criterion of the building, the Y-shaped structure (shape F) showed the best performance. This structure behaves most stiff, which results in smaller accelerations at the top of the building.
Changing the form of the building's footprint has a minor effect on the gross-nett floor ratio of the building. This because the dimensions of the building's footprint are already determined by the building's structural behaviour.

Giving the skyscraper a tapering shape gives us a desirable result; a taller load-bearing structure is feasible. However, there is one major disadvantage: The height of the building has become so large that the tensile stresses in the windward columns of the cross-section, exceed the compressive column stresses due to the vertical loads. This results in a tensile force in the columns on the windward side. Such a tensile force is highly undesirable.
Due to the building's tapering shape, the eigenfrequency of the building increases. A larger eigenfrequency leads to more strict limits regarding the building's accelerations. Additionally, due to the decrease in the building's mass, less damping will occur, resulting in higher accelerations at the top. This means that the limits with regard to the comfort criterion of the Y-shaped building decrease. However, there is reason to believe that a tapering building shape reduces the building's peak accelerations at the top. However, in the existing literature no clear formulas can be found which describe this favourable effect. Therefore, a further study into this topic is highly recommended.
There is also no improvement obtained with regard to the economical feasibility of the skyscraper. The gross-nett floor ratio of the building can even deteriorate when there is no tapering building core applied inside the building.

Creating a compound structure leads to limits to the load-bearing structure of the skyscraper which are slightly lower than the limits which were found for the tapering skyscraper. By changing the building's gross-nett floor ratio or the centre-to-centre distance between the towers, these limits can be approached.

A major advantage of the compound structure in comparison with the tapered structure is, that there will be no tensile forces in the windward columns of the building. To further increase the height of the load-bearing structure, the dimensions of the columns themselves have to be increased.
With regard to the comfort criterion, creating a compound structure results to higher limits. This due the cross-section's larger moments of inertia. It is uncertain whether the used formula from the NBCC for estimating the building's across-accelerations, is also applicable on compound structures.
Potentially, the gross-nett floor ratio of a compound skyscraper can be very high. However, it is questionable whether this is feasible, since the vertical transportation of people, goods and building services requires already a considerable amount of space.

# Chapter 9

# Remaining Challenges

In chapter 5 of this report, a list of six challenges has been drawn up. All these challenges influence the skyscraper's height.

1. Load-bearing structure
2. Comfort
3. Economical feasibility
4. Evacuating the building
5. Foundation
6. Vertical transportation

In the previous chapter three of these challenges have been considered in the pursuit of finding the ultimate limit to the height of a skyscraper. These three challenges were: the load-bearing structure of the building, the comfort criterion and the building's economical feasibility. So, three challenges out of the list of six have already been considered, but what about the three challenges remaining? Is it, considering these remaining three challenges, possible to achieve the in the previous chapter computed building heights? In chapter 6 has been concluded that especially the evacuation of the building could cause serious problems. In this chapter each of the three remaining challenges is considered and it is evaluated whether the in the previous chapter determined building heights, are feasible. The measures found in chapter 7 of this report are expected to offer adequate solutions.

## 9.1 Evacuating the Building

In chapter 6 a limit of 154 metres was found with regard to the evacuation of high-rise buildings. This height is much lower than the height limits found in the previous chapter. This means that a number of measures have to be taken to make a skyscraper of this height possible. Adequate solutions for stretching the building's height with regard to the evacuation of a skyscraper have been given in chapter 7. In this chapter, measures like extending the evacuation time, utilising new evacuation methods and raising the emergency exit floors of the skyscraper have been considered. We will now consider how these solutions can help in achieving the in the previous chapter computed building heights.

We assume the in the previous chapter described compound structure of four towers. Each of these towers has 265 floors and the towers are connected to each other by giant steel trusses. Besides their structural function, these trusses will also function as skybridges, allowing people to travel from one building to an other. The floors at which these links are connecting the towers, are equipped as fire rescue floors. Fire-rescue floors are floors where the building's occupants can await their rescue or wait till the fire is extinguished so they can escape safely from the skyscraper (see chapter 3). These Fire-rescue floors are at most 25 storeys apart.

Now we assume that a fire breaks out on the 200th floor in one of the towers. Within minutes a large part of the floor is in flames. The fire causes a thick smoke to spread over the entire floor and the two adjacent floors, obstructing the building's occupants who want to pass these floors in order to escape. However, due to vertical fire compartments, the smoke is not able to spread any further. A sprinkler system prevents the fire from spreading.
In a traditional high-rise, the building's occupants above the burning floors would be trapped. They can only hope that the fire can be extinguished before the load-bearing structure of the building fails. However the building's skybridges enable the occupants to escape to one of the adjacent towers. In these towers the lift systems are still fully operational and can be used without any risk.
The people occupying the floors underneath the floors which are on fire, can escape from the building in the conventional way, i.e. by using the staircases of the building. However, evacuating a 200 storey building still takes a long time. Utilising the building's lifts can offer a solution. As soon as the emergency starts, the lifts are put in an emergency mode. When switched to this mode, the lift cabs accelerate and decelerate much faster allowing them to transport more people in the same amount of time. Although these accelerations do not meet to the comfort demands any longer, the functioning of the system is not compromised. In the emergency mode, the lifts will only halt at the fire-rescue floors. Calls made from other floors are disregarded. The building's occupants can reach these fire-rescue floors via the building's staircases, which are kept free from smoke due to pressurisation. On the fire-rescue floors people can safely wait for the next lift car. The dispatching of the lift cabs is done fully automatic by means of a computer system. An automatic dispatching system is far more efficient than when it is manually operated by the emergency personnel. To make lifts suitable for evacuating people, hardened lift cabs have to be applied, i.e. no smoke may enter into the lift cars, the electronics should be waterproof, and a back-up power supply should be available.
Heat and smoke detectors inside the lift shafts and lift machine rooms monitor constantly whether the use of the lift is still without any danger. As soon as there is a threat, the lift cab will halt at the nearest safe landing zone and will shut itself down. For the assumed emergency situation, this means that the lifts travelling through the burning storeys are shut down within minutes after the occurrence of the emergency. However, lifts departing from skylobbies underneath the burning floors, can still be used for the evacuation of people. Keeping the building's lifts operational as long as possible, will speed up the evacuation considerably.

By applying these kind of solutions, ultra-tall buildings can still be evacuated safely and quickly. Therefore, evacuating a building with a height like presented in the previous chapter, will not cause any problems. However, please note that the solutions presented here, lie still in the future. Although most of the required technologies are available, they still need extensive testing to convince the authorities that they are safe and that they comply with existing building regulations. Thereafter, it will take an additional amount of time to capture these new measures in new regulations and building codes.

## 9.2 Foundation

The measures presented in chapter 7 of this report offer good points of departure to stretch the height of a skyscraper further than the in chapter 6 established limit of 344 metres. Especially the measure of constructing a heavier foundation for the building seems a promising solution. In chapter 6 has been assumed that the smallest allowable centre-to-centre distance between the bored piles is 2.2 times the diameter of the piles. This was based on the regulations which can be found in the Dutch building code *NEN 6743*. However, when we compare this with the designs for the foundation of super-tall buildings like the Burj Dubai and the Al Burj, we see that the applied centre-to-centre distances of the foundation piles are much smaller than 2.2 times the diameter of the piles. This means that foundations with a much larger load-bearing capacity can be constructed.
Also the construction of a building plinth at the base of the skyscraper can bring along large results. Even if the vertical loads from the columns are spread out over a slightly larger area, so a few more piles can be added to the foundation, this can dramatically increase the height of the skyscraper.
It can be concluded that there is sufficient room for stretching the building's length with regard to its foundation. This means that there is enough reason to believe that the building's foundation will not impose a limit to the height of a skyscraper.

## 9.3 Vertical transportation

From chapter 6 followed that the vertical transportation inside a skyscraper is not the most restricting factor to the building's height. In chapter 6 a limit of 720 metres was found with regard to the vertical transportation. This limit is imposed by the national building codes. Technically, heights up to a 1000 metres are feasible.
Unfortunately, the limitation to the maximum possible vertical travel, is not the only problem which is encountered when building an ultra-tall skyscraper. The waiting times and the amount of space which is occupied by the lifts, are also major issues. To keep the occupant's waiting times acceptable, more lifts have to be deployed. However, this will worsen the gross-nett floor ratio of the building. Faster lifts are the solution to this problem. Recent technological developments have made faster travels possible, but it is questionable whether this will entirely solve the problem. An extensive logistical study should be conducted to judge whether it is possible to service an ultra-tall building without compromising the user-friendliness of the lift system and keeping the space which is occupied by the system within limits. However, such a logistical study lies beyond the scope of this Master's thesis.
The development of the lift has showed us that the term "Technological fix" may be applicable. This means that each time logistical problems were solved by introducing new and improved lift technologies,

which made it possible to build ever higher skyscrapers. Yet again, a new technology is presenting itself: the Electro-magnetic lift (see chapter 7). This technology promises to make longer and faster travels possible. And, because the lift cars are not any longer connected to hoist ropes, multiple cars can operate in the same shaft. An advanced computer system makes sure that all the lift cabs are deployed in the most efficient way. Similar computer dispatching systems are already in use for dispatching conventional lift systems, but are not yet fully perfected.
It is unlikely that the vertical transportation inside a skyscraper, will eventually limit the height of the skyscraper.

## 9.4 Conclusions

Based on this chapter we can conclude that there is enough reason to believe that the three remaining challenges, evacuating the building, foundation and vertical transportation, offer sufficient starting points to stretch their limits further then the limits which were established in chapter 6 of this report. It can be expected that the limits to these three challenges can match those of the other three challenges which were considered in the previous chapter. However, in order to prove this indisputably, further research may be necessary.

# Chapter 10

# Conclusions & Recommendations

In the first chapter of this report, the goal of the thesis was given and summarised into a single sentence:
*"To gain a good insight into the challenges which will be encountered when designing and constructing an ultra-tall skyscraper, with the aim to find the ultimate limit to the height of the skyscraper."*
This goal consisted out of two parts. In the first part of thesis a good insight was gained into the thesis' subject. In the second part of the thesis, this obtained knowledge was used in order to explore the limits to high-rise. In this final chapter I will conclude whether these two goals are achieved and what the recommendations are which follow from these conclusions.

## 10.1 Conclusions

### 10.1.1 First Part of the Thesis

Through chapters 2, 3 and 4 a proper insight has been gained into the challenges which occur when designing and constructing a skyscraper. A total of 14 challenges were identified. Each of these challenges have been carefully examined and described. The result of this is a complete reference work which forms a valuable source of information for other students which are conducting a thesis with a subject related to high-rise structures. We can conclude that the first part of the thesis' goal, gaining a good insight into the subject, has been successfully achieved.

### 10.1.2 Second Part of the Thesis

No absolute limit to the height of a skyscraper has been found. This due to the fact that it is difficult, if not impossible, to mention an absolute limit to high-rise, since there are too many boundary conditions. The limits which are found, are largely dependent on the assumptions which are made in the beginning. Nevertheless, this report contains valuable information and fulfils the thesis' goal of exploring the ultimate limits to high-rise.

In chapter 5 the thesis' problem has been converged into a more manageable problem through three consecutive steps. This led to a reduced list of six challenges which are all influencing the height of the skyscraper. Selecting these challenges has been done precisely and in consultation with the graduation committee. Nevertheless, selecting these challenges is a subjective process, which means that someone else can have the opinion that other challenges should have been taken into account.
The description of the benchmark skyscraper is based on a large number of assumptions and simplifications. However, the assumed benchmark skyscraper served the goal of the thesis well. It has proven to be a useful tool for exploring the ultimate limits to high-rise.

In chapter 6 the limits to the benchmark skyscraper have been computed for each of the six remaining challenges. This chapter gives a valuable overview of the building regulations and building codes which are valid in The Netherlands. In order to keep the calculations in this chapter clear and simple, a large number of assumptions and simplifications have been made. This has been done carefully. The outcome of these calculations proved to be useful to determine the further course of the thesis. Of the computed limits, none clearly stood out. Therefore is was decided that all of the remaining challenges should be taken into the next phase of the thesis. It was also concluded that the computed limits were lower than what has already been achieved in the building practice. This is primarily due to the made assumptions and the made description of the benchmark skyscraper.

Elaborating on the findings of chapter 6, chapter 7 listed a large number of measures for stretching the limits to each of the challenges. These measures were orderly arranged into tree diagrams. Sometimes, the same solution was applicable to multiple challenges. To visualise these correlations, the tree diagrams were combined into one web diagram. From this web diagram was concluded that changing the shape of the skyscraper was one of the most promising solutions for stretching the limits to high-rise. It was expected that altering the skyscraper's form stretches the limits with regard to the building's load-bearing structure, comfort criterion and economical feasibility.

In chapter 8 the consequences of changing the shape of the skyscraper have been examined more in detail. A couple of interesting conclusions were drawn in this chapter. An unexpected result was that changing the footprint of the building had no clear structural advantage in comparison to the regular rectangular footprint. This was caused by the fact that the dimensions of the building's footprint were predetermined by the requirement that the skyscraper should show a similar behaviour for wind coming from all directions. This limited the increase of the cross-section's moment of inertia. Together with an increase of the wind load on the structure (due to an increased building width), this resulted in larger compressive stresses in the building's columns. The only way in which a large structural gain was realised, was by giving the footprint a circular shape in order to decrease the wind loads on the structure. A Y-shaped footprint showed the most favourable performance with regard to the comfort criterion of the building. This was caused by the large moment of inertia. However, it is expected that a round building shape will also have a favourable effect on reducing the accelerations at the top of the skyscraper. Further research should confirm this thought. Furthermore it was proven that, due to the predetermined dimensions of the cross-section, no gains in the nett-gross floor ratio of the building shapes could be achieved.
By giving the skyscraper a tapering shape, a significant gain with regard to the limit of the load-bearing structure was achieved. This was caused by the decreased lateral and vertical forces acting on the structure. However, there was one major structural disadvantage: The height of the building has

become so large that the tensile stresses in the windward columns of the cross-section exceeded the compressive column stresses due to the vertical loads. This results in a tensile force in the columns on the windward side of the building. Such a tensile force is highly undesirable. The limits with regard to the comfort criterion dropped. There was also no improvement obtained with regard to the gross-nett floor ratio of the skyscraper. The gross-nett floor ratio of the building can even deteriorate when there is no tapering building core applied inside the skyscraper.
The final option that was examined, was the idea of structurally interconnecting multiple slender towers. This way of changing the building's shape proved to be a good solution with good results all along the line. Due to the large moments of inertia of the cross-section, significant gains in the limit of the load-bearing structure could be achieved: 635 metres for a concrete structure and 895 metres for a steel structure. These limits approached those found for the tapering skyscraper. However a major advantage of the compound structure in comparison to the tapering structure is, that there are no tensile forces present in the windward columns of the building. Also with regard to the comfort criterion and the building's gross-nett floor ratio, the compound building form performs well.

Because only three out of the six challenges had been considered so far, chapter 9 examined whether the in chapter 8 established limits could also be achieved by the other three challenges. The measures found in chapter 7 were expected to offer adequate solutions. Indeed, it was concluded that the limits to these three challenges could match those of the other three challenges. However, it was also noted that in order to prove this indisputably, further research may be necessary.

The limits which were computed in this report are not very astonishing. The computed limits correspond to skyscrapers found in the real world, but do not show large improvements. This can be explained by the fact that the earlier made assumptions and simplifications are too much on the "safe side". Although this report gives no absolute limit to high-rise, some interesting conclusions can be drawn. It is concluded that the compound structure offers great opportunities for stretching the limits to all of the six challenges. We can conclude that this report gives a well-founded reasoning that compound structures are the most promising option when we want to build ultra-tall skyscrapers in the near future.

## 10.2 Recommendations

It was concluded that no absolute limit to the height of a skyscraper can be found. However this report indicates which path should be chosen in order to make the construction of an ultra-tall building possible. The extensive literature review described in this thesis provides the needed background knowledge. This report can serve as a starting point for the continuation of this thesis' subject. However, there is one major recommendation. In this report, the following approach was chosen: At first all of the building's boundary conditions were determined. Based on these, the limits to the skyscraper were established. However, when one wants to find the ultimate limit to high-rise, a different approach will lead to better results. Instead, it would be better to predetermine the height of the skyscraper. Based on this target one can deduce which structure is needed to achieve this height and which concessions have to be made with regard to the building's serviceability and safety. By following this alternative approach, the thesis' subject is much more confined and easier to manage. This will also push the researcher to be creative and to think beyond conventional solutions. Perhaps this method does not lead to the absolute limit, but it will be a better way to approach this.

Additionally, there are also some minor recommendations which need to be addressed:
It is needed to revise the assumptions and recommendations made. Some of these may have been chosen too much on the "safe side". Examples of this are the rather high design floor loads of 14.3 kN/m$^2$, the storey height of only 3.5 metres and the assumed Young's modulus of cracked concrete. A change in the assumptions which were made at the beginning, can lead to higher limits to the challenges.
Additionally, some further research has to be conducted in the effects of the shear-lag effect. In the report this effect was taken into account by introducing a factor $\gamma$. It was assumed that this factor had a value of 0.75. Further research has to prove whether this assumption was correct. The very same applies to the factor which was used to describe to what extend the multiple towers of the compound structure work together.
With regard to the building's comfort criterion it was decided to apply the most strict building regulations. Having the goal of this thesis in mind, one could ask whether this is correct. If we want to build an ultra-tall skyscraper we have to examine which ultimates are still possible within the limits of the law. The regulations should be pushed. This does not only apply to the comfort criterion, but also to the other challenges which were examined during this thesis.
An other recommendation is that more knowledge has to be acquired with regard to the cross-wind accelerations of a skyscrapers. In this thesis a formula from the Canadian Building code, the NBCC, was used. However it is unclear whether this formula is also valid for skyscrapers with a circular footprint or for compound structures.
After the challenges "Load-bearing structure", "Comfort" and "Economical Feasibility" were examined in chapter 8, the three remaining challenges were considered in chapter 9. It was concluded that the limits to these challenges could match to those which had been found in chapter 8. However, in order to prove this indisputably, further research is needed. In case of the challenges "Evacuating the building" and "Vertical Transportation", the use of computer models could be a solution. By building a computer model one can model how individuals move through a building. This can help to determine the number of lifts which are needed inside the skyscraper. A computer model that models the evacuation of a skyscraper can be used to determine the evacuation time of the building. This kind of computer software is already available on the market. These programmes are even capable of modelling the spread of smoke through a building.
Finally, there are two last recommendations which need to be addressed. Ultra-tall buildings have a very high slenderness ratio. Due to this high slenderness, second order effects may occur. These effects were not taken into account in this report. It should be examined whether these second order effects play an important role. The other recommendation is about assessing the economical feasibility of the building. In the report this has been done by looking at the gross-nett floor ratio of the building. Since the building's facade is one of the most expensive components, it is advisable to take as well into account the ratio between the facade surface and the gross floor surface of the building. In this way a better founded judgement can be made with regard to the building's economical feasibility.

# Appendix A

# Overcasting

Although the influence of the building on its surroundings has not been taken into account in this thesis, it is still worthwhile of making a little sidestep to one of the aspects of this challenge.

In the fourth chapter of this report has been mentioned that the entry of daylight into a building is considered to be an important factor in the design. Therefore the depth of each floor is limited. Bearing this in mind, one of the worst influences a building can have on its surroundings, is overcasting adjacent buildings and therefore preventing daylight form entering into these buildings.

However, like in many national codes, there is no criterium recorded in the Dutch building codes which regulates the intermediate distance of buildings. Such a regulation can be found in the British Standard, the national building code of the United Kingdom. This regulation states that a vertical sky component equal to 27% at 2 metres above the ground, is sufficient to illuminate the interior of a building (Asimakopoulos et al. [3]). The total vertical sky dome is $180^{\mathrm{o}}$. This means that a angle of $180^o \times 0.27 = 48.6^o$ should be free from obstructions.

This requirement is illustrated by figure A.1.

The height of the building (indicated in figure A.1 by *y*) is, with a fixed angle of $48.6^{\mathrm{o}}$, restricted by the distance *x* in between the buildings. For x we assume a 6-lane street divided by a 6 metres wide strip and with pavements on both sides. This results in an intermediate distance of 30 metres (see figure A.2).
From a simple calculation follows the maximum height y of the building:

$$y = \frac{x}{tan\alpha} = \frac{30}{\tan 48.6^{\mathrm{o}}} = 26.5\mathrm{m} \tag{A.1}$$

Since the regulation applies for two metres above the ground floor, the limit to the height of the building is *28.5 metres*.
This means that almost all the high-rise buildings in the world do not meet this requirement. Therefore this can be considered as a "soft" requirement, a target which can be aimed at, but a requirement which will not limit the height of a building. Please note that this formula is only considering the vertical sky component. The free angle in the horizontal direction is not taken into account.

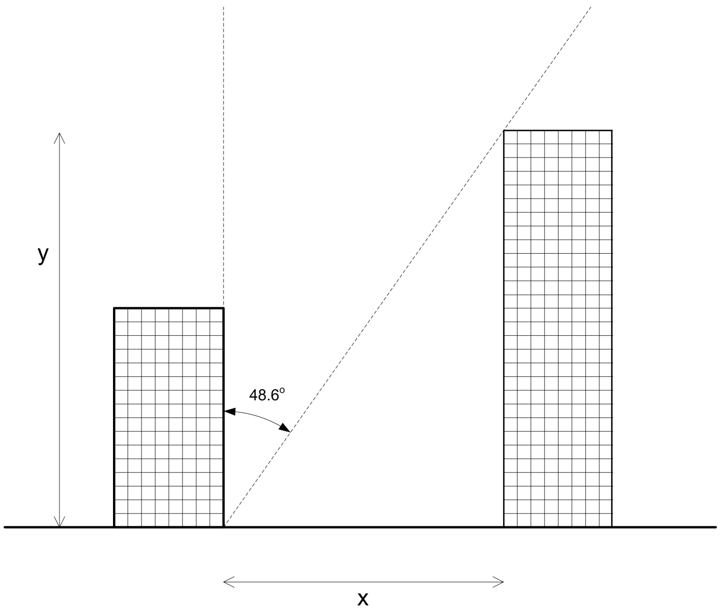



**Figure A.1:** *The overcast of a building on adjacent buildings*

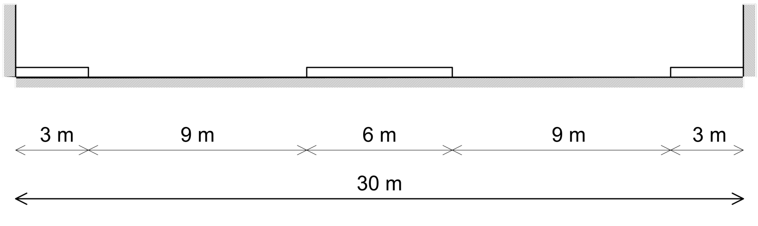



**Figure A.2:** *Cross-section of a city street*

# Appendix B

# Wind Force

The wind force which is acting on a structure is determined by equation B.1. This appendix explains how the factors in this equation are calculated.

$$q_w = c_s c_d \times c_f \times q_p(z) \times b \tag{B.1}$$

With:
$c_s c_d$ = structural factor, which has a minimum value of 0.85
$c_f$ = force coefficient
$q_p(z)$ = peak velocity pressure at reference height $z$
$b$ = width of the building

## B.1 Peak Velocity Pressure, $q_p(z)$

The peak velocity pressure is determined by the following formula:

$$q_p(z) = (1 + 7 l_v(z)) \times \frac{1}{2} \times \rho \times v_m^2(z) \tag{B.2}$$

With:
$l_v(z)$ = turbulence intensity
$\rho$ = air density, for which 1.25 kg/m$^3$ is given in the national annex
$v_m^2(z)$ = mean wind velocity

### B.1.1 Turbulence Intensity, $l_v(z)$

The turbulence intensity is described by the following formula:

$$l_v(z) = \frac{k_l}{c_0(z) \times ln\left(\frac{z}{z_0}\right)} \tag{B.3}$$

With:
$k_l$ = turbulence factor, for which 1.0 is given in the national annex
$c_0$ = orography factor, taken as 1.0 unless otherwise specified
$z_0$ = roughness length, which is equal to 0.2 according to the national annex

### B.1.2 Mean Wind Velocity, $v_m^2(z)$

The mean wind velocity is described by equation B.4:

$$v_m(z) = c_r(z) \times c_0(z) \times v_b \tag{B.4}$$

With:
$c_r(z)$ = roughness factor
$c_0(z)$ = orography factor, taken as 1.0 unless otherwise specified
$v_b$ = basic wind velocity

The roughness factor, $c_r(z)$, depends on the terrain type around the structure and on the height of the building:

$$c_r(z) = k_r \times ln\left(\frac{z}{z_0}\right) \tag{B.5}$$

With:
$k_r$ = terrain factor depending on the roughness length of the terrain
$z_0$ = roughness length, which is equal to 0.2 according to the national annex

The terrain factor, $k_r$, is described by the following formula:

$$k_r = 0.19 \times \left(\frac{z_0}{0.05}\right)^{0.07} \tag{B.6}$$

The basic wind velocity is determined by equation B.7:

$$v_b = c_{dir} \times c_{season} \times v_{b,0} \tag{B.7}$$

With:
$c_{dir}$ = directional factor, 1.0 recommended
$c_{season}$ = season factor, for the Netherlands equal to 1.0
$v_{b,0}$ = fundamental value of basic wind velocity

In the national annex of the Netherlands is determined that the fundamental value of basic wind velocity is equal to 27 m/s.

## B.2 Force Coefficient, $c_f$

The force coefficient is determined, using the following formula:

$$c_f = c_{f,0} \times \psi_r \times \psi_\lambda \tag{B.8}$$

With:
$c_{f,0}$ = force coefficient, which is given in figure B.1
$\psi_r$ = reduction factor square sections, which is given in figure B.2
$\psi_\lambda$ = end-effect factor, which is given in figure B.3

From the figure B.1 follows that for a building with a rectangular footprint, $c_{f,0}$ equals to 2.1.
From figures B.2 and B.3 follows that for the rectangular shape of the bench-mark skyscraper, $\psi_r$ and $\psi_\lambda$ have values of respectively 1.0 and 0.69.

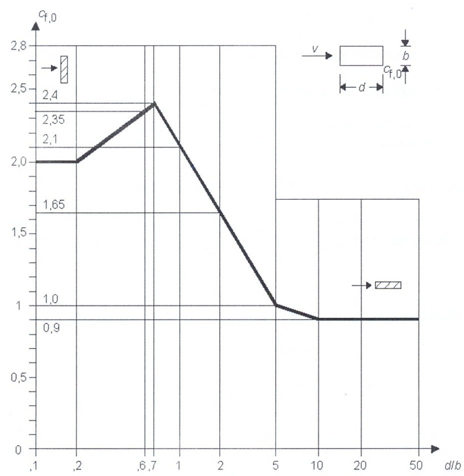



**Figure B.1:** *Force coefficient according to the EN 1991-1-4*

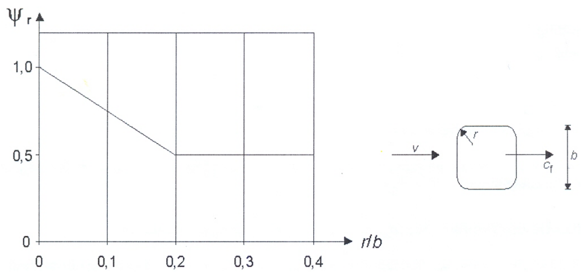



**Figure B.2:** *Reduction factor according to the EN 1991-1-4*

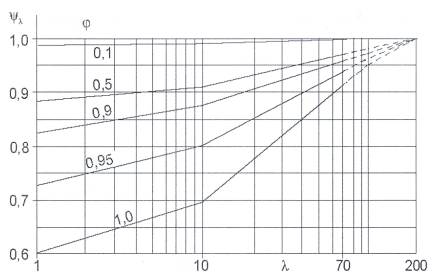



**Figure B.3:** *End-effect factor according to the EN 1991-1-4*

## B.3 Structural Factor, $\mathrm{c_s c_d}$

The structural factor is calculated by using the following formula:

$$c_s c_d = \frac{1 + 2 \times k_p \times l_v(z_s) \times \sqrt{B^2 + R^2}}{1 + 7 \times l_v(z_s)} \tag{B.9}$$

With:
$z_s =$ the reference height for determining the structural factor
$k_p =$ peak factor defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation
$l_v =$ turbulence intensity
$B^2 =$ background factor, allowing for the lack of full correlation of the pressure on the structure surface
$R^2 =$ resonance response factor, allowing for turbulence in resonance with the vibration mode

### B.3.1 Reference Height, $\mathrm{z_s}$

The reference height for determining the structural factor is given in figure B.4. This figure is given in Eurocode EN 1991-1-4.

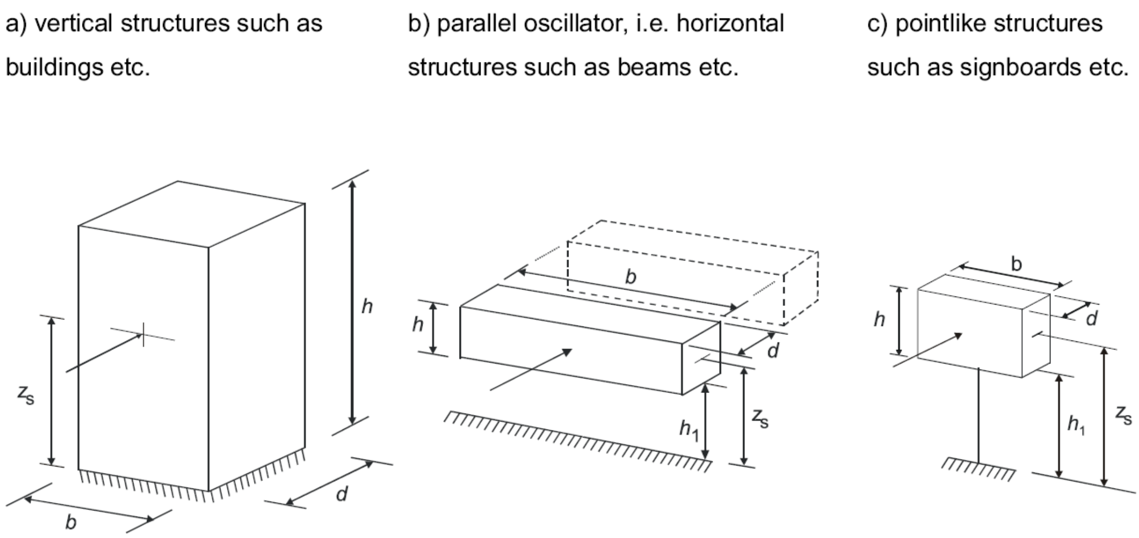



**Figure B.4:** *reference height $z_s$; (a)$z_s = 0.6 \times h$ (b)$z_s = h_1 + \frac{h}{2}$ (c)$z_s = h_1 + \frac{h}{2}$*

From figure B.4 follows that the reference height $\mathrm{z_s}$ for the benchmark skyscraper is $0.6 \times h$.

### B.3.2 Peak Factor, $\mathrm{k_p}$

The peak factor is the largest value the following two values:

1. $$k_p = 3 \tag{B.10}$$

2. $$k_p = \sqrt{2 \times ln\left(2 \times ln(\nu \times T)\right)} + \frac{0.6}{\sqrt{2 \times ln(\nu \times T)}} \tag{B.11}$$

   With:
   $T$ = averaging time for mean wind velocity, T = 600 seconds
   $\nu$ = up-crossing frequency

The up-crossing frequency is determined by using equation B.12

$$\nu = n_1 \times \sqrt{\frac{R^2}{B^2 + R^2}}; \nu \geq 0.08Hz \tag{B.12}$$

With:
$n_1$ = eigenfrequency of the building (see equation B.17)
$B^2$ = background factor (see equation B.13)
$R^2$ = resonance response factor (see equation B.15)

### B.3.3 Turbulence Factor, $\mathrm{l_v}$

The formula for calculating the turbulence factor $l_v$ has already been given in chapter 6 of this report. Instead of using the total height of the building, z, we now use the reference height, $\mathrm{z_s}$.

### B.3.4 Background Factor, $\mathrm{B}^2$

The background factor is determined via the following formula:

$$B^2 = \frac{1}{1 + \frac{3}{2} \times \sqrt{\left(\frac{b}{L(z_s)}\right)^2 + \left(\frac{h}{L(z_s)}\right)^2 + \left(\frac{b}{L(z_s)} \times \frac{h}{L(z_s)}\right)^2}} \tag{B.13}$$

With:
$b$ = width of the building
$h$ = height of the building
$L(z_s)$ = turbulent length scale at reference height $\mathrm{z_s}$

The turbulent length scale is given by the following formula:

$$L(z_s) = L_t \times \left(\frac{z_s}{z_t}\right)^{\alpha} \tag{B.14}$$

With:
$z_t$ = reference height of 200 metres
$L_t$ = reference length scale of 300 metres
$\alpha = 0.67 + 0.05ln(z_0)$ in which $z_0$ is the roughness length which is equal to 0.2 metres

### B.3.5 Resonance Response Factor, $\mathrm{R}^2$

The resonance response factor is given by the formula:

$$R^2 = \frac{\pi^2}{2 \times \delta} \times S_L(z_s, n_1) \times K_s(n_1) \tag{B.15}$$

With:
$\delta$ = total logarithmic decrement of damping
$S_L$ = wind power spectral density function
$n_1$ = natural frequency of the structure in Hertz
$K_s$ = size reduction factor

The total logarithmic decrement of damping is estimated by the following expression:

$$\delta = \delta_s + \delta_a + \delta_d \tag{B.16}$$

With:
$\delta_s$ = logarithmic decrement of structural damping
$\delta_a$ = logarithmic decrement of aerodynamic damping for the fundamental mode
$\delta_d$ = logarithmic decrement of damping due to special devices

For now, we will only take the structural damping into consideration. The values for this factor are given in a table in Eurocode EN 1991-1-4 (table B.1).

According to the Eurocode, for buildings with a height of more than 50 metres, the natural frequency, $n_1$, can be estimated using the following formula:

$$n_1 = \frac{46}{h} \tag{B.17}$$

With:
$h$ = height of the building

The wind power spectral density function is determined by using the following formula:

$$S_L(z_s, n_1) = \frac{6.8 \times f_L(z_s, n_1)}{(1 + 10.2 \times f_L(z_s, n_1))^{\frac{5}{3}}} \tag{B.18}$$

With:
$f_L(z_s, n_1)$ = non-dimensional frequency

This non-dimensional frequency can be determined by using expression B.19.

$$f_L(z_s, n_1) = \frac{n_1 \times L(z_s)}{v_m(z_s)} \tag{B.19}$$

| **Structural type** | | **structural damping, $\delta_s$** |
|---|---|---|
| reinforced concrete buildings | | 0,10 |
| steel buildings | | 0,05 |
| mixed structures concrete + steel | | 0,08 |
| reinforced concrete towers and chimneys | | 0,03 |
| unlined welded steel stacks without external thermal insulation | | 0,012 |
| unlined welded steel stack with external thermal insulation | | 0,020 |
| steel stack with one liner with external thermal insulation[a] | $h/b < 18$ | 0,020 |
| | $20 \leq h/b < 24$ | 0,040 |
| | $h/b \geq 26$ | 0,014 |
| steel stack with two or more liners with external thermal insulation [a] | $h/b < 18$ | 0,020 |
| | $20 \leq h/b < 24$ | 0,040 |
| | $h/b \geq 26$ | 0,025 |
| steel stack with internal brick liner | | 0,070 |
| steel stack with internal gunite | | 0,030 |
| coupled stacks without liner | | 0,015 |
| guyed steel stack without liner | | 0,04 |
| steel bridges + lattice steel towers | welded | 0,02 |
| | high resistance bolts | 0,03 |
| | ordinary bolts | 0,05 |
| composite bridges | | 0,04 |
| concrete bridges | prestressed without cracks | 0,04 |
| | with cracks | 0,10 |
| Timber bridges | | 0,06 - 0,12 |
| Bridges, aluminium alloys | | 0,02 |
| Bridges, glass or fibre reinforced plastic | | 0,04 - 0,08 |
| cables | parallel cables | 0,006 |
| | spiral cables | 0,020 |

NOTE 1 The values for timber and plastic composites are indicative only. In cases where aerodynamic effects are found to be significant in the design, more refinded figures are needed through specialist advice (agreed if appropriate with the competent Authority.

NOTE 2 For cable supported bridges the values given in Table F.2 need to be factored by 0,75

[a] For intermediate values of $h/b$, linear interpolation may be used

**Table B.1:** *Approximate values of logarithmic decrement of structural damping,* $\delta_s$

With:
$n_1$ = natural frequency of the building (see equation B.17)
$L(z_s)$ = turbulent length scale (see equation B.14)
$v_m(z_s)$ = Reference mean wind velocity at reference height$\mathrm{z_s}$

The size reduction factor is given by the following formula:

$$K_s(n) = \frac{1}{1 + \sqrt{(G_y \times \phi_y)^2 + (G_z \times \phi_z)^2 + \left(\frac{2}{\pi} \times G_y \times \phi_y \times G_z \times \phi_z\right)^2}} \tag{B.20}$$

With:

$$\phi_y = \frac{c_y \times b \times n_1}{v_m(z_s)} \tag{B.21}$$

$$\phi_z = \frac{c_z \times h \times n_1}{v_m(z_s)} \tag{B.22}$$

The decay constants $\mathrm{c_y}$ and $\mathrm{c_z}$ are both equal to 11.5. The shape constants $\mathrm{G_y}$ and $\mathrm{G_z}$ are given in table C.1 in the Eurocode (table B.2).

| Mode shape | Uniform | Linear | Parabolic | Sinusoidal |
|---|---|---|---|---|
| *G*: | 1/2 | 3/8 | 5/18 | $4/\pi^2$ |
| *K*: | 1 | 3/2 | 5/3 | $4/\pi$ |

NOTE 1 For buildings with a uniform horizontal mode shape variation and a linear vertical mode shape variation $\Phi(y,z) = z/h$, $G_y = 1/2$, $G_z = 3/8$, $K_y = 1$ and $K_z = 3/2$.

NOTE 2 For chimneys with a uniform horizontal mode shape variation and a parabolic vertical mode shape variation $\Phi(y,z) = z^2/h^2$, $G_y = 1/2$, $G_z = 5/18$, $K_y = 1$ and $K_z = 5/3$.

NOTE 3 For bridges with a sinusoidal horizontal mode shape variation $\Phi(y,z) = \sin(\pi \cdot y/b)$, $G_y = 4/\pi^2$, $G_z = 1/2$, $K_y = 4/\pi$ and $K_z = 1$.

**Table B.2:** *G as a function of mode shape*

## Appendix C

# Example MatLab Input

```matlab
% % BUILDING FORM F (Material: Concrete)
clear all; close all;


%% Initialisation of variables
A = 3047.0;          % Area (m2)
d = 12.6;            % Floor depth (m)
w_max  = sqrt((4*A)/(tand(60)));   % (m)
w_min  = 2*d;
w_step = 0.1;
d_ctc  = 2.5;        % Desired center-to-center distance (m)
col_w = 0.9;         % Column width (m)
col_d = 0.9;         % Column depth (m)
col_A = col_w*col_d;% Column area  (m2)


%% Computation
% Building size
w = [w_min:w_step:w_max];   % width (m)
l = 1/3*((A - ((1/4) .* w.^2 .* tand(60)))./w);   % length(m)

% number of case of study
max_case = size(w,2);

% amount of subspaces between columns
amount_subspace_w = ceil(w./d_ctc);  % number of subspaces between column on w
amount_subspace_l = ceil(l./d_ctc);  % number of subspaces between column on l

% width of subspaces (m)
width_subspace_w = w./amount_subspace_w;        % width of subpaces between
columns on w (m)
width_subspace_l = l./amount_subspace_l;        % width of subpaces between
columns on l (m)


%% Number of columns on each walls.
% wall 1 = left upper wall, then clockwise.
col_wall_1 = amount_subspace_w + 1;
col_wall_2 = amount_subspace_l;
col_wall_3 = amount_subspace_l - 1;
col_wall_4 = amount_subspace_w + 1;
col_wall_5 = amount_subspace_l - 1;
col_wall_6 = amount_subspace_l;
col_wall_7 = amount_subspace_w + 1;
col_wall_8 = amount_subspace_l;
col_wall_9 = amount_subspace_l - 1;

col_wall_tot = (col_wall_1 + col_wall_2 + col_wall_3 + col_wall_4 + col_wall_5
+ col_wall_6 + col_wall_7 + col_wall_8 + col_wall_9);


%% Median (m)

% i_z
% Walls 1 and 4
```

```
for i = 1:max_case
    iz1 = [1:col_wall_1(i) - 1].* width_subspace_w(i) .* sind(60);
    i_z_wall_1(i) = sum((iz1).*col_A);
end

% Walls 2
for i = 1:max_case
    iz2 = [1:col_wall_2(i)].* width_subspace_l(i) .* sind(30);
    i_z_wall_2(i) = sum((iz2).*col_A);
end

% Walls 3
for i = 1:max_case
    iz3 = [1:col_wall_3(i)].* width_subspace_l(i) .* sind(30);
    i_z_wall_3(i) = sum((iz3).*col_A);
end

% Walls 5 and 9
for i = 1:max_case
    iz5 = ((1/2).*w(i).*tand(60) + l(i).*sind(30)) -
([1:col_wall_5(i)].*width_subspace_l(i) .* sind(30));
    i_z_wall_5(i) = sum((iz5).*col_A);
end

% Walls 6 and 8
for i = 1:max_case
    iz6 = ((1/2).*w(i).*tand(60) + l(i).*sind(30)) + ([1:col_wall_6(i)-
1].*width_subspace_l(i));
    i_z_wall_6(i) = sum((iz6).*col_A) + (((1/2).*w(i).*tand(60) +
l(i).*sind(30)).*col_A);
end

% All the walls summed up
i_z_wall_tot = ( 2.*(i_z_wall_1) + (i_z_wall_2) + (i_z_wall_3) +
2.*(i_z_wall_5) + 2.*(i_z_wall_6) + col_wall_7.*(((1/2).*w(i).*tand(60) +
l(i).*sind(30) + l(i)).* col_A) );

i_z_upper = (i_z_wall_tot)./(col_wall_tot .* col_A);
i_z_lower = (1/2).*w.*tand(60) + l.*sind(30) + l - i_z_upper;

%Transverse i_z for copying to Exel
i_z_upper_transv = i_z_upper';
i_z_lower_transv = i_z_lower';


%% Steiner (m4)

% I_zz_steiner
% Walls 1 and 4
for i = 1:max_case
    ncw1 = i_z_upper(i) - ([1:col_wall_1(i) - 1].* width_subspace_w(i) .*
sind(60));
    I_zz_steiner_wall_1(i) = sum((ncw1.^2).*col_A) +
((i_z_upper(i)^2)*col_A);
end
```

```
% Walls 2
for i = 1:max_case
    ncw2 = i_z_upper(i) - ([1:col_wall_2(i)].* width_subspace_l(i) .*
sind(30));
    I_zz_steiner_wall_2(i) = sum((ncw2.^2).*col_A);
end

% Walls 3
for i = 1:max_case
    ncw3 = i_z_upper(i) - ([1:col_wall_3(i)].* width_subspace_l(i) .*
sind(30));
    I_zz_steiner_wall_3(i) = sum((ncw3.^2).*col_A);
end

% Walls 5 and 9
for i = 1:max_case
    ncw5 = i_z_upper(i) - ([1:col_wall_5(i)].*(width_subspace_l(i) .*
sind(30)) + (w(i).*sind(60)));
    I_zz_steiner_wall_5(i) = sum((ncw5.^2).*col_A);
end

% Walls 6 and 8
for i = 1:max_case
    ncw6 = ([1:col_wall_6(i) - 1].*(width_subspace_l(i)) +
((1/2).*w(i).*tand(60) + l(i).*sind(30))) - i_z_upper(i);
    I_zz_steiner_wall_6(i) = sum((ncw6.^2).*col_A) +
(((((1/2).*w(i).*tand(60) + l(i).*sind(30)) - i_z_upper(i)).^2)*col_A);
end


%% Moment of inertia (m4)

% I_eigen
I_zz_eigen = 1/12*col_w*col_d^3;

% I_zz
I_zz_wall_1 = (col_wall_1 .* I_zz_eigen) + (I_zz_steiner_wall_1);
I_zz_wall_2 = (col_wall_2 .* I_zz_eigen) + (I_zz_steiner_wall_2);
I_zz_wall_3 = (col_wall_3 .* I_zz_eigen) + (I_zz_steiner_wall_3);
I_zz_wall_4 = I_zz_wall_1;
I_zz_wall_5 = (col_wall_5 .* I_zz_eigen) + (I_zz_steiner_wall_5);
I_zz_wall_6 = (col_wall_6 .* I_zz_eigen) + (I_zz_steiner_wall_6);
I_zz_wall_7 = col_wall_7 .* ( I_zz_eigen + (col_A.*((1/2).*w.*tand(60) +
l.*sind(30) + l - i_z_upper).^2) );
I_zz_wall_8 = I_zz_wall_6;
I_zz_wall_9 = I_zz_wall_5;

I_zz = (I_zz_wall_1 + I_zz_wall_2 + I_zz_wall_3 + I_zz_wall_4 + I_zz_wall_5
+ I_zz_wall_6 + ...
    I_zz_wall_7 + I_zz_wall_8 + I_zz_wall_9);

I_zz_transv = I_zz';        %Transverse I_zz for copying to Exel
```

```
%% Figures

figure(1);hold on;
p1 = polyfit(w,I_zz,3);  % fit a polynomial to the points (deg 3) using 'w'
and 'I_zz'
f1 = polyval(p1,w);       % reconstruct the polynomial line using the 'w'
measurements

plot(w,I_zz,'b.-');     % plot the real data
plot(w,f1,'r-');         % plot the fitted polynomial line

grid on;hold off;
legend('Real I_{zz}','Fitted line')
xlabel('w [m]')
ylabel('I [m^4]')

print('-f1', '-r400', '-dpng','MomentsInertiaF_Concrete.png')
```

# List of Figures

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# Bibliography


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