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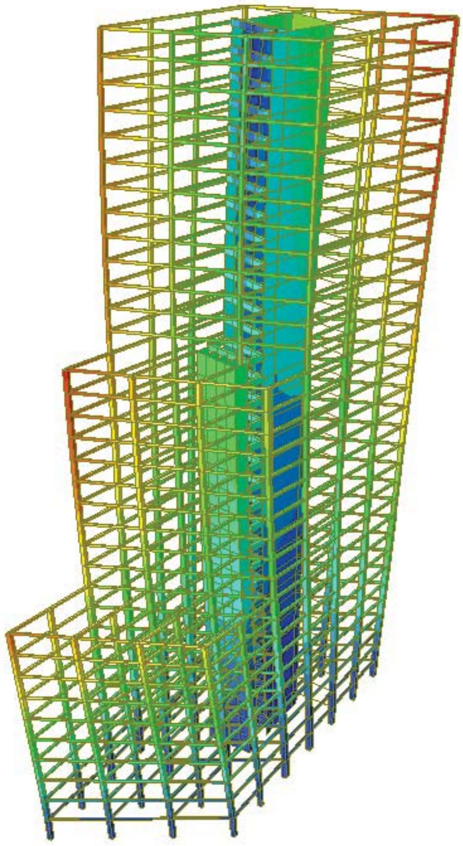

METHODOLOGY FOR PRELIMINARY DESIGN OF HIGH-RISE BUILDINGS

VIKTOR CASTLEN RIST and STEFAN SVENSSON

Structural Mechanics

Master's Dissertation

DEPARTMENT OF CONSTRUCTION SCIENCES

DIVISION OF STRUCTURAL MECHANICS

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MASTER'S DISSERTATION

METHODOLOGY FOR PRELIMINARY DESIGN OF HIGH-RISE BUILDINGS

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Abstract

The construction of high-rise buildings has previously been limited in Sweden. Changes in society in recent years, related to urbanisation, economics and architectural standards have however resulted in a greater interest for the construction of high-rise buildings. When designing high-rise buildings, challenges are faced which often can be disregarded when designing lower buildings.

In this dissertation, a methodology which can be used in the preliminary design process of high-rise buildings is developed. The methodology is based on idealised calculation models and idealised finite element models, especially focused on the dynamical properties, natural frequencies and accelerations of the building. The report will describe how these calculations can be used to make the preliminary design of high-rise building and still obtain reasonable results. The work is focused on buildings with a structural system consisting of a concrete core with the possibility of adding outriggers. The dissertation also highlights the different phenomena related to high-rise buildings that needs to be considered in the preliminary stage, or later, in the design process. This includes, among other things, comfort requirements and wind induced accelerations. The stiffness is of great importance when designing high-rise buildings. For this reason, different ways to change the stiffness of the building are also analysed.

To evaluate how reliable the idealised calculations and models are, a case study on an ongoing project is performed. The calculations are performed on Gothenburg City Gate, a 120 meter tall building that is intended to be built in Gothenburg. The case study follows the methodology developed, in order to show an example of how the idealised models can be used in the preliminary design. The results of the idealised calculations are compared with the results of the finite element analysis to evaluate the accuracy of the calculations.

The analysis shows that an idealised beam model of the building with varying stiffness will give sufficient results for the preliminary design. Also the shape function provided by Eurocode can be used with good results. The idealised calculations of the natural frequencies gives results that correspond with the more advanced FE-models, the results differed 3-13% in the first modes. This is however only true for translation modes and torsional modes are not accurate. Adding an outrigger to the building drastically increased the first natural frequencies of the building, by up to 50% for Gothenburg City Gate.

Keywords:
High-Rise, Building, Eurocode, Wind-induced acceleration, Preliminary Design, Gothenburg City Gate, Comfort Requirements, Methodology

Sammanfattning

Tidigare har byggandet av höga hus inte varit vanligt i Sverige, men på senare år har förändringar i samhället skett vilket lett till att höga hus blivit mer intressant. Vid dimensioneringen av höga hus uppkommer svårigheter och fenomen som inte behöver beaktas vid dimensioneringen av lägre hus.

I detta examensarbetet tas en arbetsmetodik fram, vilken kan följas vid den preliminära dimensioneringen av höga hus. Arbetsmetodiken baseras på idealiserade beräkningar och idealiserade FE-modeller och är framförallt inriktad på byggnadens dynamiska egenskaper, såsom egenfrekvenser och accelerationer. Arbetet beskriver hur dessa beräkningar kan användas vid den preliminära dimensioneringen av höga hus och fortfarande erhålla tillförlitliga resultat. Uppsatsen belyser även de specifika problemen som behöver beaktas vid konstruktion av höga hus vid den preliminära dimensioneringen eller i ett senare stadie. Detta omfattar bland annat komfortkrav och vindinducerad acceleration. Styvheten av byggnaden är mycket viktig för höga hus. Av denna anledning diskuteras och analyseras olika metoder på hur byggnadens styvhet kan justeras.

För att utvärdera hur pålitliga de förenklade beräkningarna och modellerna är används dessa på ett pågående projekt, Göteborg City Gate, ett 120 meter högt hus som planeras att byggas i Göteborg. Fallstudien följer den framtagna arbetsmetodiken för att visa hur de förenklade modellerna kan användas vid den preliminära dimensioneringen. Resultat från de förenklade beräkningarna jämförs med resultat från finita element modeller för att utvärdera tillförlitligheten.

Analyserna visar att en enkel balkmodell med varierande styvhet kan användas för att erhålla resultat som går att använda i den preliminära dimensioneringen. Även formfunktionen som är angiven i Eurocode kan användas med ett bra resultat. Resultaten som fås från de idealiserade beräkningsmodellerna, vilka används för att beräkna byggnadens egenfrekvens, stämmer bra överens med resultaten då hela byggnaden modellerades. Skillnader på 3-13% noterades i de första moderna som kontrollerades. Detta stämmer dock bara för egenfrekvenser i translation och inte för egenfrekvenser vid vridning. Att lägga till en outrigger ökar drastiskt de lägsta egenfrekvenserna, med upp till 50% för Göteborg City Gate.

Nyckelord:
Höga hus, Preliminär dimensionering, Eurocode, Acceleration, Egenfrekvens, Design, Göteborg city gate, Komfortkrav, Arbetsmetodik

This master's dissertation ends our five years as Civil Engineering students at the Faculty of Engineering, LTH, at Lund University. This dissertation was carried out as a project in collaboration between the Division of Structural Mechanics at LTH and Skanska Teknik AB in Malmö during the spring of 2016.

We would like to thank our supervisor Henrik Danielsson at LTH. We would also like to thank Carl Jonsson and Carl Larsson at Skanska Teknik AB for their guidance and support during the project.

Lund, June 2016

Viktor Castlen Rist

Stefan Svensson

Nomenclature

Greek letters

α	Power coefficient	-
α_A	Reduction factor for vertical loads due to large floor area	-
α_n	Reduction factor for vertical loads due to multiple stories	-
β	Critical damping ratio	-
$\ddot{X}$	Acceleration	m/s^2
Δ	Maximum wind induced lateral displacement	m
δ_a	Aerodynamic logarithmic decrement of damping	-
δ_s	Structural logarithmic decrement of damping	-
γ	Specific weight	N/m^3
ω	Angular frequency	rad/s
ϕ	Shape function	-
ψ_λ	Reduction factor of force coefficient for structural elements with end-effects	-
ψ_r	Reduction factor of force coefficient for square sections with rounded corners	-
ρ	Density	kg/m^3
$\sigma_{\ddot{X}}$	Standard deviation of acceleration, rms-acceleration	m/s^2

Latin letters

a_D	Acceleration in along-wind direction (also referred to as $\ddot{X}$)	m/s^2
a_G	Factor of galloping instability	-
A_{ref}	Reference area	m^2
a_{tot}	Total acceleration from combining a_w and a_D	m/s^2
a_w	Acceleration in cross-wind direction	m/s^2
B	Background response factor	-

b	Width of building	m
c_0	Orography factor	-
C_e	Exposure factor	-
c_f	Force coefficient	-
C_g	Gust response factor	-
c_{pe}	Shape factor for external wind load	-
c_r	Roughness factor	-
$c_s c_d$	Factor considering non-simultaneous occurrence of peak wind pressures on the surface and the effect of the vibration of the structure due to turbulence	-
D	Depth of building (also referred to as d)	m
d	Depth of building	m
E_{cm}	Average Young's modulus for concrete	Pa
F	Gust energy ratio	-
f	Frequency	Hz
f_0	Fundamental frequency	Hz
g	Gravitational acceleration	m/s^2
g_p	Peak factor (also referred to as k_p)	-
h	Height of building	m
I_v	Turbulence intensity	-
I_x	Moment of inertia in x-direction	m^4
I_y	Moment of inertia in y-direction	m^4
$\widetilde{k}$	Generalised stiffness	N/m
K	Factor related to the surface roughness coefficient	-
k_p	Peak factor	-
k_r	Terrain factor	-
$\widetilde{m}$	Generalised mass	kg
m	Mass per unit length	kg/m
m_e	Equivalent mass per unit length	kg/m
n_0	Fundamental frequency (also referred to as f_0)	Hz

$n_{1,x}$	Fundamental frequency of along-wind vibration	Hz
$n_{1,y}$	Fundamental frequency of cross-wind vibration	Hz
n_D	Buildings frequency in along-wind direction (also referred to as n_x)	Hz
n_W	Buildings frequency in cross-wind direction (also referred to as n_y)	Hz
q_b	Reference mean (basic) velocity pressure	kN/m^2
q_m	Mean velocity pressure	kN/m^2
q_p	Peak velocity pressure	kN/m^2
R	Resonant response factor	-
s	Size reduction factor	-
Sc	Scruton number	-
St	Strouhal number	-
T	Average time for reference wind velocity	s
$\bar{V}$	Reference wind speed at 10 m (also referred to as v_b)	m/s
v_b	Basic wind velocity	m/s
v_{CG}	Onset wind velocity for galloping	m/s
v_{cr}	Critical wind velocity for vortex shedding	m/s
V_H	Mean wind speed at the top of the building	m/s
v_m	Mean wind velocity	m/s
W	Width of building (also referred to as b)	m
z	Height of interest	m
z_0	Roughness length	m
z_s	Reference height	m

Contents

Chapter 1

Introduction

1.1 Background

The construction of high-rise buildings has previously been limited in Sweden, the only notable exception to this being the Turning Torso in Malmö. There have been several reasons for this, including architectural standards and lack of economic incentives. Changes in society in recent years, related to urbanisation, economics and architectural standards, have resulted in a greater interest for the construction of high-rise buildings.

When designing high-rise buildings, challenges are faced which often can be disregarded when designing lower buildings. These may include torsion of the building and swaying due to dynamic loads. Due to the lack of high-rise buildings in Sweden, the knowledge of how to deal with these problems is limited. This dissertation will cover the steps to make in the preliminary design, when different options for structural systems are considered and rough dimensions are decided.

1.2 Aims

The aim of this master's dissertation is to provide knowledge relevant for preliminary design of high-rise buildings. The methodology developed can be used in an early stage to simplify the preliminary design of the building. Different model simplifications and idealisations are studied, with the aim of finding appropriate and simple models and thus reducing the need of advanced FE-models. If the structural system and its general properties can be determined in a reliable and appropriate way in an early stage, the construction process and size of the usable surface can be estimated better. This leads to a more reliable economy for the project which will benefit both the entrepreneur and the developer. Better knowledge of the structural system in an early stage of the project also simplifies upcoming design calculations. The idealised models will mainly be adopted for a structural system with a concrete core and outrigger.

1.3 Objectives

The main objective is to develop a simplified methodology which can be used in an early stage of the design process of high-rise buildings. The methodology is based on idealised calculations and idealised FE-models, the step-by-step process can be seen in figure 4.1. The calculation methods will be limited to cover structural systems with a concrete core, with the possibility of adding an outrigger for increased stiffness. The dissertation will also highlight the different phenomena related to high-rise buildings that need to be considered, during the preliminary stage or later in the design process. The stiffness is of great importance when designing high-rise buildings. For this reason different ways to change the stiffness of the building are analysed. The dissertation will bring up the following issues:

- What are the specific challenges and problems related to high-rise buildings?
- How does Eurocode handle these challenges?
- What steps should be made in the preliminary design?
- How can a building be idealised and still give results that are reliable?
- When should a wind tunnel test be performed and what kind of results can be expected?

1.4 Methods

A literature study is performed, which among other things cover different structural systems for high-rise buildings, specific challenges related to high-rise buildings, dynamic loads and what rules Eurocode has for high-rise buildings.

The literature study is followed by a case study on an ongoing project, Gothenburg City Gate, a 120 meter tall building that is intended to be built in Gothenburg. The project is provided by Skanska which is also the project developer. The case study follows the methodology developed, this is in order to show an example of how the idealised models can be used in the preliminary design. The idealised calculations are compared with FE-models to evaluate the accuracy of the calculations.

1.5 Outline of the report

In chapter 2, high-rise buildings are discussed. The history, the definition and the different structural systems that can be used are shown. Chapter 3 covers the specific challenges and phenomena related to high-rise buildings as well as information about wind tunnel tests. Chapter 4 explains design and calculation methods. Here the calculation methods for the shape functions of a building, the natural frequencies and accelerations due to dynamic loads are presented. Chapter 5 contains the case study of Gothenburg City Gate, where the methodology in section 4.1 is followed to perform the calculations in the preliminary design. Different simplifications of the building are made to examine if they give reasonable results. The calculations are performed by hand, and by the use of the software programs Strusoft Frame Analysis, Strusoft FEM-Design and Midas GEN. Chapter 6 contains comparisons between FE-models, wind tunnel tests and measurements on real buildings, which is made to evaluate the reliability of the different methods of analysis. Chapter 7 summarizes the results and contains the conclusions and the discussion of the dissertation.

Chapter 2

High-rise buildings

This chapter begins with an explanation of the construction process to clarify the scope of the report. This is followed by a short section about the definition and history of high-rise structures. Finally, different structural systems used in high-rise buildings are briefly explained and illustrated.

2.1 Structural design process

The construction process can be divided into four phases, each ending with a different set of documents. The first phase (Prestudy) ends with concept documents, the second phase (Planning and project development) with a programme document, the third (Schematic and building permit design) with outline documents and technical systems and the fourth phase (Detailed design) with construction documents. The design process laid out in this dissertation takes place before the fourth phase where detailed design is performed and the construction documents are made.

The first phase includes, among other things, choosing where the building should be located, the size of the building, the type of load bearing system and what kind of facades and foundation that should be used. This is followed by the second phase which includes setting rules and requirements for the building, performing preliminary structural design and applying for construction permits. The third phase includes more structural design on idealised models, making economic assessments and possible wind tunnel tests if it is necessary. The fourth and last phase includes the detailed design that leads up to the construction documents.

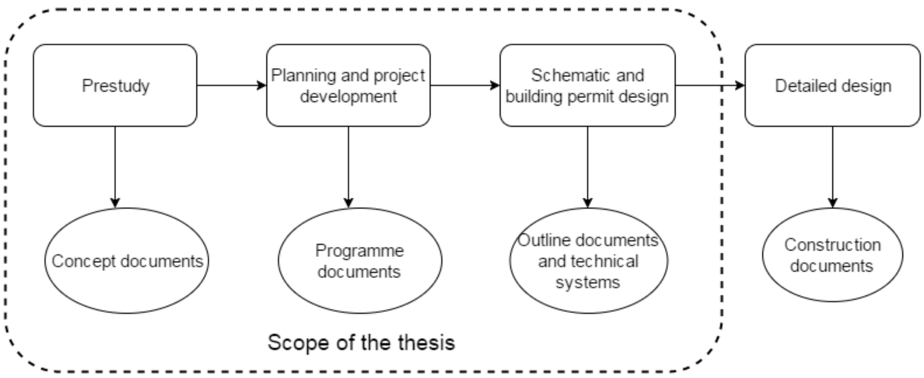


Figure 2.1: Building process.

2.2 Definition

There are many different ways to define what a high-rise building is. An architect or a city planner will often define it as a building that clearly protrudes above the surrounding buildings. If the height of the building has a big impact on the evacuation or if the height of the building is greater than the maximum reach of firefighting equipment it can be called a high-rise building because of fire regulations [27]. In the United States, a building taller than 23 meters is called a high-rise building according to the National Fire Protection Association while the Emporis standard defines a high-rise building as a building with a height of at least 35 meters [18].

For a structural engineer, the definition of a high-rise building lies with the problems that are associated with the design of the building. A building is then most often considered a high-rise building when dynamic loads becomes relevant. In general terms, a flexible building can be assumed to be affected by dynamic loads while a rigid building is assumed not to. The accepted criteria for a rigid building is when the fundamental frequency of the building is less than 1 Hz [64]. However, tall rigid buildings can also have issues related to their design because of their height, this could for example be the need for construction stage analysis or special construction techniques.

2.3 History

Tall buildings can be seen all over the world, they can be used to show of wealth and power, religious beliefs or to push the boundaries of engineering. The ancient Egyptians built the pyramids nearly 5000 years ago as tombs for their Pharaohs and their consorts, and to this day they are still standing as some of the oldest high-rise structures in the world. However, they are not considered buildings since they are not inhabitable. The Gothic cathedrals of Europe were the skyscrapers of medieval time, rising far above anything else in Europe. All the early high-rise structures have one thing in common, they have a structural system made of masonry which limited the building height due to the high self-weight of the material. When reinforced concrete and steel were introduced as building materials, taller buildings could be built [27].

For a long time Chicago was the leading city in high-rise building design. It was here that the first steel high-rise building was built, the Home Insurance Building, a 10-story building finished in 1885. This building marked the start of high-rise building design. In the beginning of the 20th century tall buildings started to appear in New York. In 1903 the Flatiron Building (22-stories) was finished, in 1909 the Metropolitan Life Insurance Building (50-stories) and in 1913 the Woolworth Building (57-stories) [27].

For many years, the United States was the leading country when it came to high-rise buildings, both in the amount of buildings being built and the maximum height. Since the end of the last century, Asia and the Arabic region have constructed a large amount and the tallest high-rises in the world. These have included the Petronas Towers in Malaysia, Taipei 101 in Taiwan and Burj Khalifa in Dubai. High-rise buildings can today be found on all continents and there is always a new record setting building being built [27].

2.4 Structural systems

There are a variety of different load bearing systems for high-rise buildings, which one to use depends on the height of the building, where the building is located and the architectural design. The higher a building is, the more material is needed to resist lateral loads. At approximately 50 floors the material costs for resisting lateral loads in a rigid frame becomes greater than those for the vertical load bearing system. This is why an appropriate load bearing system is required [64]. Some of the most common structural systems used in high-rise buildings are explained below.

2.4.1 Rigid frames

For buildings with a fairly low height, a rigid frame can be used. A rigid frame consists of columns and girders with moment resisting connections. It resists lateral loads with the bending resistance of the columns and beams. When designing buildings with moment resisting frames, the size of the columns and beams are often controlled by the bending stiffness and not by the load capacity. The high bending stiffness is needed to limit the drift due to lateral loads. Furthermore the behaviour of the building depends on the design of the connections, if a big rotation between the beam and column is allowed, the lateral sway of the building will increase rapidly and cause problem with the comfort in the building [29].

Steel or concrete can be used for this type of system. For steel, the maximum appropriate height is about 30 stories and for concrete about 20 stories. For buildings over 30 stories there is a risk of large lateral swaying from wind and earthquakes, the connections between the beams and the columns also become too complicated and too expensive in order to withstand the large moments, especially for steel frames [29].

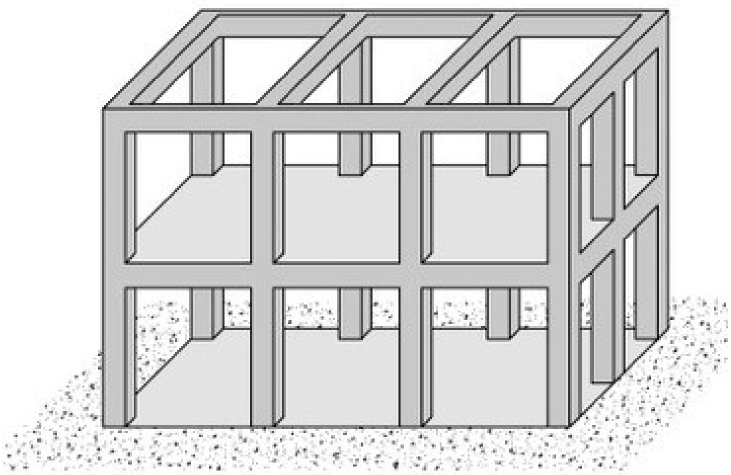


Figure 2.2: Illustration of a rigid frame [25].

2.4.2 Shear walls

Shear walls have a high resistance in their own plane and are used to resist lateral loads. Shear walls can resist overturning moments, shear forces and also torsion if they are properly placed in the building.

Shear walls can be used in different ways. One way is to use a system of columns with a flat slab and shear walls, which will extend the effective height up to about 20 stories compared to 10 stories for a similar system with just columns and slabs [29]. A shear wall system is shown in figure 2.3. By connecting shear walls a coupled shear wall is obtained. The walls are coupled by placing beams between the shear walls as shown in figure 2.4. This is an effective way to greatly increase the lateral stiffness of a building. This is often done to accommodate holes for windows and doors, this type of system is effective up to 40 stories [64].

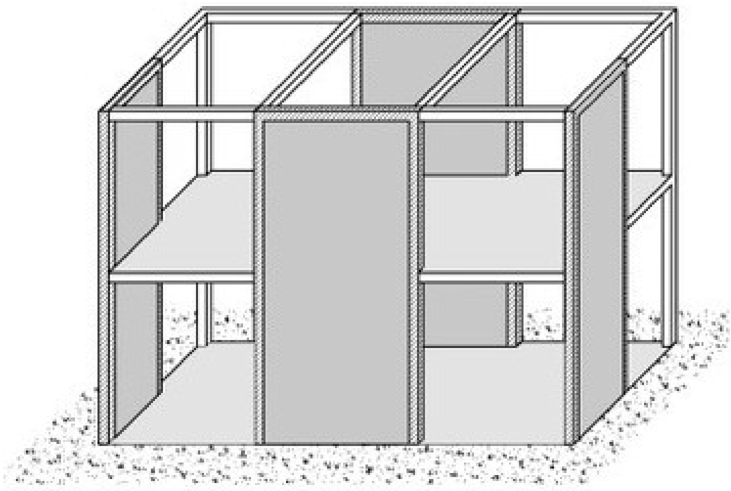


Figure 2.3: Illustration of shear walls [25].

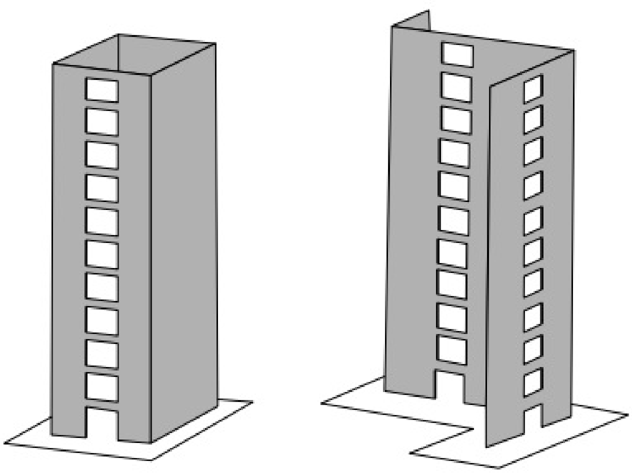


Figure 2.4: Illustration of coupled shear walls [64].

2.4.3 Core and outrigger systems

A very common way of using shear walls is to use them in a core supported structural system, which means that shear walls are cast around elevator and stair shafts to create a core. The core can resist lateral, vertical and torsional loads. This system is effective up to 45 floors. Additional vertical loads are taken by columns [64].

A combination of shear walls and rigid frames can be used to create buildings up to 60 floors. The shear walls are often placed around elevator and staircases to create a core while the frames are on the exterior of the building which allows deep beams to be placed on the outside of the building. This type of structural system reduces the overturning moment and the risk for uplift at the core [64].

A core can be complemented with an outrigger structure to greatly increase its bending stiffness. The outrigger itself consists of stiff floors high up in the building. The stiffness can for example be generated with walls, one or two-stories high. These outriggers are connected to columns that stretch along the perimeter of the building down to the ground. When the structure is subjected to lateral loads they are resisted with axial forces in the exterior columns and the moment in the core is decreased. Belt walls are used to resist the rotation of the outriggers and to engage all columns in the exterior. Belt walls consist of walls or trusses placed on the perimeter of the outrigger floor. Outriggers will reduce the lateral displacements of the building due to bending, however they do not increase the shear or torsional resistance of the building which still must be resisted by the core. Outriggers and belt walls can be used for very high buildings, up to 150 floors [64].

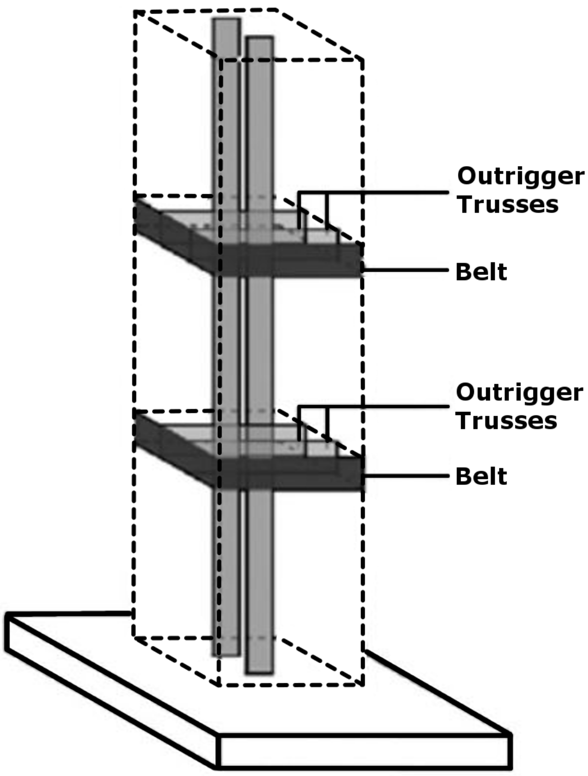


Figure 2.5: Illustration of an outrigger system [19].

2.4.4 Braced frames and shear trusses

Diagonal braces can be a supplement to a rigid frame in order to create a more rigid building. Braced systems reduce the large shear racking deformations by decreasing bending of girders and columns. Diagonal members are placed inside the frames which carry lateral loads and therefor reduces bending of beams and columns. Braced frame systems are often more economical than moment resisting frames. There are however several disadvantages, for example reduced flexibility in floor plan layout, space planning and electrical routing. The braced frames are often placed in the core of the building. Depending on the size of the core, the torsional resistance may be the controlling design parameter. The braced frame system is used in steel buildings and is effective up to 40 floors. There are a wide variety of different bracing systems which can be used [64]. There are two types of braced frame systems, concentric braced frames (CDF) or eccentric braced frames (EDF). In the concentric braced frames, many of the members intersect in a common point. This is not a requirement when using eccentric braced frames. Concentric braced frames are very strong and stiff which does not make them ideal in seismic zones due to their poor inelastic behaviour. In seismic zones it is better to use the eccentric braced frames, since this bracing type combines the strength and stiffness of a braced frame with the inelastic energy dissipation characteristics of a moment resisting frame. This type of system is effective up to 25-30 stories.

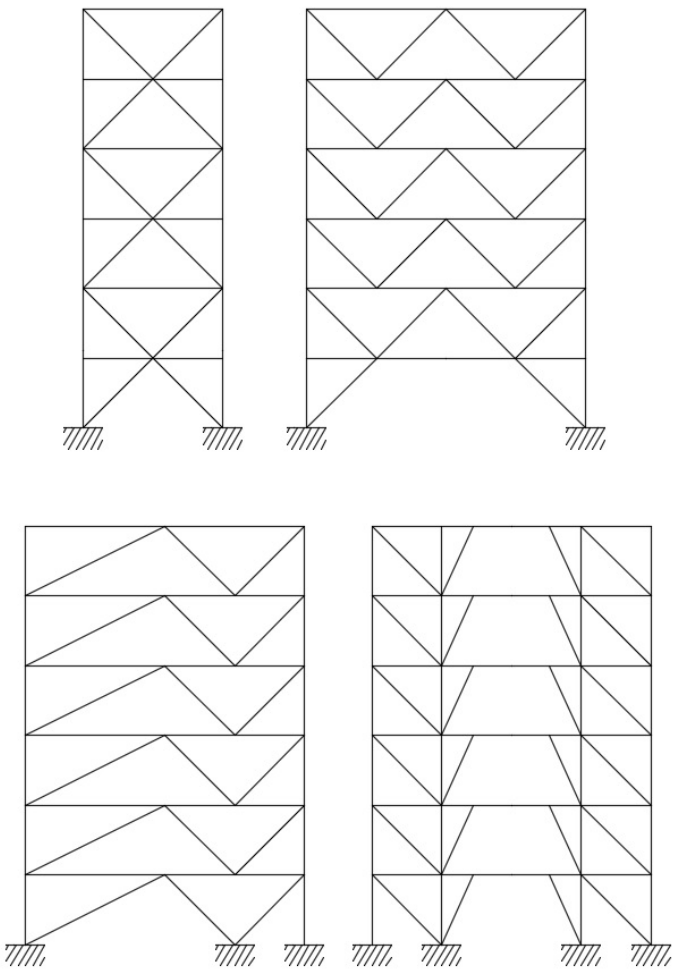


Figure 2.6: Illustration of concentric braced frames [64].

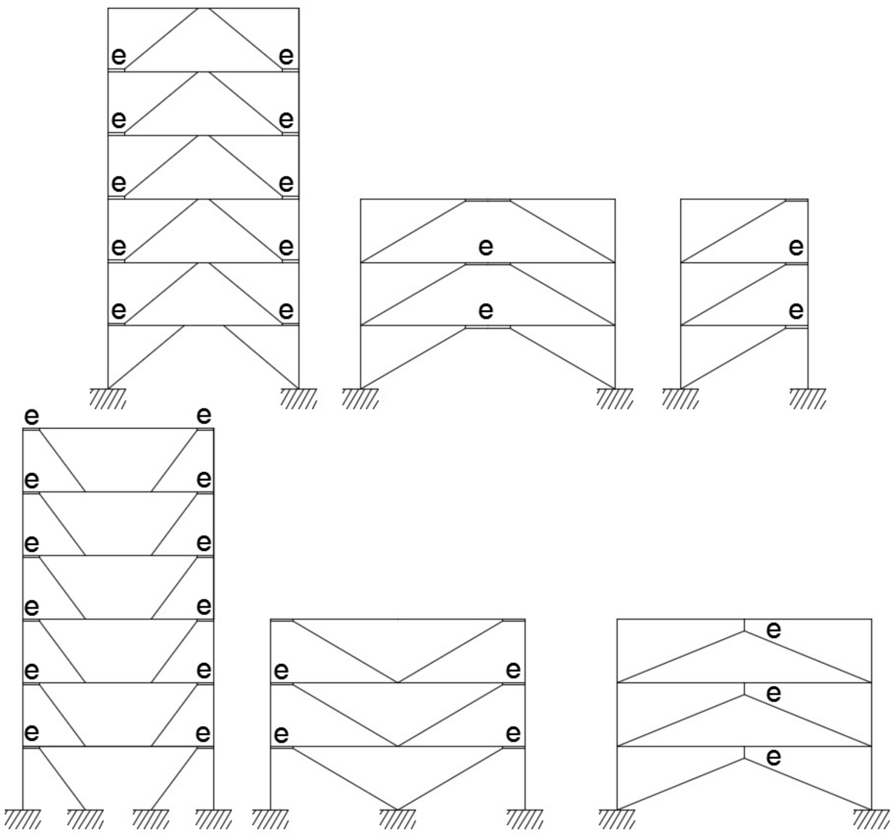


Figure 2.7: Illustration of eccentric braced frames [64].

Another type of truss system is the staggered truss system. This bracing system was developed for residential buildings that are fairly long and narrow. Normally the system can be used for heights up to 25 stories. In this system, trusses that are one floor high are placed in an alternating pattern on each floor, the floor transfers the lateral loads to the trusses which means that the columns does not receive any bending action. Since the truss system should not block the passage through the building, some diagonal members of the truss must be removed. This is normally done in the centre of the building. Since the diagonals are removed the shear is instead carried by a stiff moment frame, which is added around the opening in the truss system. Openings in the truss system should be avoided since it is expensive to implement [64].

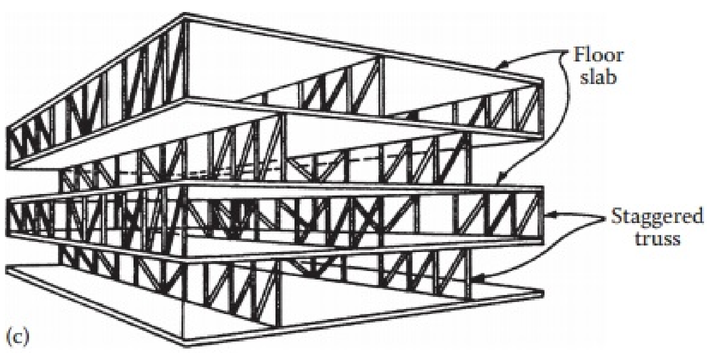


Figure 2.8: Illustration of a staggered truss system [64].

2.4.5 Tubular systems

There are many different types of tubular systems. Even if they are partly different, they use approximately the same technique to carry loads. Most of the tall buildings in the world are designed with some kind of tubular system [59]. The framed tube system is used for buildings up to 60 stories. The load bearing capacity and stiffness of the structural system is provided by the moment resisting frames that form a tube around the edge of the building. The tubes around the perimeter of the building engages the entire building to resist lateral loads. To create the frame for the tube systems, columns are placed closely together around the buildings exterior. A basic tubular system is shown in figure 2.9 [64].

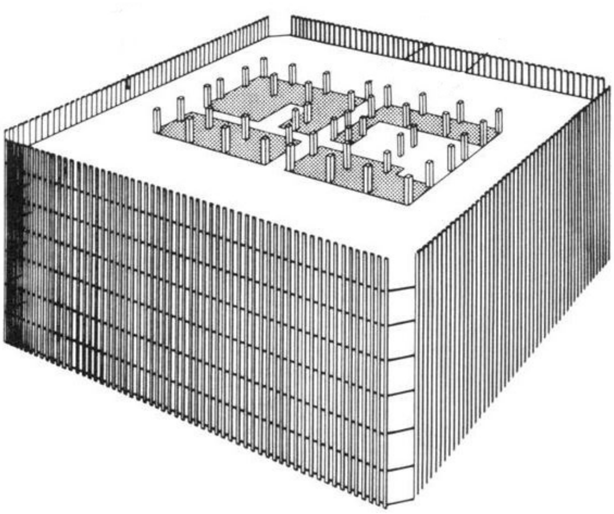


Figure 2.9: Illustration of tube system [26].

To obtain an even better structural system, additional bracing can be mounted on the exterior of the building. This type of system is called an exterior diagonal tube system and is one of the most used system. The exterior diagonal tube system can be used in buildings up to 100 stories [64]. In figure 2.10 the exterior diagonal tube system is illustrated.

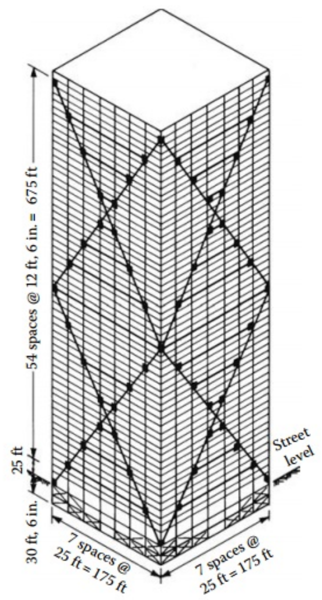


Figure 2.10: Illustration of an exterior diagonal tube system.

By connecting individual tubes, a bundled tube system is obtained. The tubes working together results in a very strong structural system, this means that the columns can be placed at an even greater distance which allows big openings for windows [64]. An illustration of the bundled tube system is shown in figure 2.11

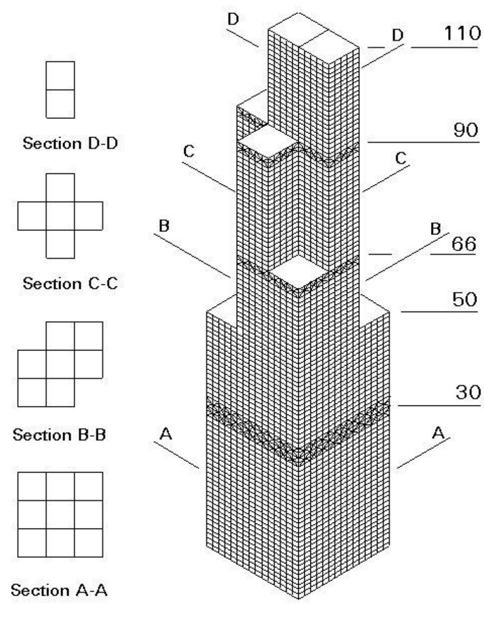


Figure 2.11: Illustration of a bundled tube system [55].

By using a structural core inside the framed tube system, another type of tubular system is obtained; the tube-in-tube system. It uses the advantages achieved by a central core and combines it with very efficient framed tube system. The central core often contains the elevator shaft and service shaft [64].

Chapter 3

Specific challenges related to high-rise buildings

When designing high-rise buildings, phenomena arise that could be disregarded when designing lower buildings. In this chapter, some of the different challenges specifically related to high-rise structures are explained. The chapter ends with some information about wind tunnel tests.

3.1 Eurocode rules for high-rise buildings

In addition to the requirements that needs to be checked for low-rise buildings, the lateral deformations and accelerations also need to be checked for high-rise buildings. According to Eurocode 1991-1-4 [33] chapter 6.3.2, the maximum along-wind displacement and the standard deviation of the acceleration should be evaluated. To calculate the maximum displacement, a static equivalent wind load can be used. There is no recommendation for how to calculate the cross-wind acceleration in Eurocode. There is however a method to use in the national building code of Canada (NBCC). Because of this, the NBCC will be used in this dissertation when calculating the cross-wind acceleration.

For slender buildings, with height to depth ratio $h/d > 4$, in grouped arrangement, the effect of turbulence around the base of nearby structures must be taken into account. However, the interference can be neglected if the distance between the buildings is more than 25 times the cross dimension of the upstream building or if the fundamental frequency of the downstream building is higher than 1 Hz. Eurocode recommends wind tunnel tests or consulting a specialist if interference should be considered [33].

The Swedish national annex (EKS10) [5] gives recommendations for how to calculate the wind load when the deformations and accelerations should be calculated. A wind load with a return period of 5-years has been chosen in EKS, this is in accordance with ISO 6897 [23] where criteria for horizontal movement of structures in a frequency range of 0,063 to 1 Hz are listed [5].

3.2 Natural frequencies

All structures have specific frequencies for when the structure begins to resonate, these frequencies are called the natural frequencies. The lowest natural frequency is known as the fundamental frequency. The natural frequencies are important when doing the dynamic analysis of a structure. Dynamic loads, for example wind and earthquake loads can cause structures to sway drastically and in worst case cause a collapse. One of the most famous example of this is the Tacoma narrows bridge which collapsed in 1940 due to wind-induced oscillation. The natural frequencies are described with modes, each mode is described by a natural frequency and a shape. The three first natural frequencies for a building are normally the sway in both directions (x- and y-direction) and the torsional sway (around the z-axis). The fundamental frequency in each direction is normally the most important one, an explanation for this is given in chapter 3.2.1. Normally, calculation of the natural frequencies is made by a computer program which can handle the eigenvalue analysis [65].

Some important aspects to take into account when calculating the natural frequency are the mass distribution and the stiffness. The mass and stiffness at each floor are required. Normally, the mass includes all dead loads plus 10-30% of the live load. There is no rule for how much of the live load that should be included and the number is based on what the building is used for and the opinion of the structural engineer. It is important to include all mass since it will have a great effect on the natural frequency. The moment of inertia is taken around an axis in the centre of gravity of the building. The mass distribution along the building height is needed to determine the natural frequencies of the building. Furthermore, the displacements and natural frequencies can be used to calculate the acceleration at the top of the building according to Eurocode. The damping ratio describes how the oscillation of the building decays after it has been disturbed. Currently there is no way of computing the damping ratio of a building, a value is chosen based on experience [65].

3.2.1 Effective mass participation factor

An infinite number of natural frequencies can be found for any structure, however only the first couple of natural frequencies are of importance. This is due to the effective mass participation factor, i.e. how much of the buildings mass participates in each natural frequency. The fundamental frequency in each direction generally has a value of around 60-80%, while the second natural frequency has around 10-20%. The sum of the effective mass participation factors is called the cumulative effective mass participation and will, if summed over all frequencies, total 100%. When deciding how many natural frequencies to consider, the natural frequencies should have a cumulative effective mass participation of 90% [64].

3.3 Wind

Unlike seismic loads, wind loads need to be considered no matter where the building is located. Wind is caused by pressure differences in the atmosphere which causes the air to move. The movement of air is fairly undisturbed high off the ground, but close to the ground, the surface of the earth and man-made structures cause turbulence and eddies.

Eddies are vortexes that arise when the wind is flowing past an obstacle, like a building or rough terrain [48].

The wind is a very complex load since it is a dynamic load that varies with height, location and over short time spans. In addition to the direct load effect, there are two other types of wind effects that must be taken into account according to Eurocode when designing a high-rise building. These are vortex shedding and galloping. Other effects caused by the wind include flutter, buffeting and ovalling [48]. In this dissertation, only the effects listed in Eurocode, vortex shedding and galloping, will be discussed and analysed. Short explanations of the effects listed in Eurocode are given in section 3.3.1 and section 3.3.2.

3.3.1 Galloping

Galloping is a self-induced movement of the building in the cross-wind direction that occurs due to aerodynamic forces. The phenomenon starts at a specific wind velocity that depends on the stiffness, mass, type and shape of the building. The amplitude of the galloping increases when the wind speed increases. Galloping is often an issue for bridges, but in some cases it can also be a problem for buildings [33].

3.3.2 Vortex shedding

Vortex shedding is a phenomenon that occurs when the wind blows on a structure and vortexes are shed alternately on the opposite sides of the structure. This will cause a shifting load on the structure perpendicular to the wind direction. If the frequency of the alternating load matches the natural frequencies of the building, it will induce large swaying and vibration of the structure. Once a building starts to sway the frequency of the vortex shedding will no longer depend on the wind velocity or the shape of the building but on the natural frequency of the building, this is known as the lock-in effect. Due to the lock-in effect, a building can resonate with the vortex shedding even though the vortex shedding is calculated at a frequency that differ from the buildings natural frequency [33].

3.3.3 Wind on pedestrians

When wind hits a building it will take the easiest way to get past, going around rather than over is the easiest for any relatively slender building. Wind going around buildings can give rise to strong vortexes causing discomfort for pedestrians near the base of the building.

Analytically it is almost impossible to estimate the effects the wind will have on pedestrians due to the large amount of factors involved. Wind tunnel tests can be used to obtain reliable estimates of wind conditions around the base of a building. Experience is a key factor in determining the comfort for pedestrians [64]. Also CFD analyses have been used to assess the wind conditions for pedestrians. However, CFD-analysis will not be included in this dissertation. Wind tunnels are discussed in section 3.12.

3.4 Comfort requirements

One of the most important aspects to consider during the design process of high-rise buildings is the serviceability limit state. The design for comfort is performed with two main factors in mind, the horizontal deflection and the motion of the building. The lateral movements of the building includes the maximum deflection of the building and the story drift. The story drift is the difference in deflection between consecutive floors and is of interest due to the possibility of damages to non-structural elements such as cladding. The horizontal deflection is calculated with equivalent static loads and the limit for horizontal deflection is generally set to $H/450 - H/500$. When looking at the motions of the building the acceleration is the factor that is evaluated. Comfort due to motion will be discussed further due to its complicated nature [20].

Sway and accelerations are effects that can cause major discomfort for occupants if not dealt with correctly. Careful study of the response to dynamic loads such as wind loads and seismic loads must hence be performed. Wind tunnel tests are always performed on complex high-rise buildings in order to evaluate their response to winds and their interactions with their surroundings.

Accelerations are often quantified either by the peak value or by the root-mean-square (rms) value. The former assumes that humans are mainly affected by peak accelerations during a certain time frame and ignore the smaller accelerations. The latter assumes that it is the average value of a number of cycles in the same time frame that determine the effect on comfort. A peak acceleration can be converted into a rms-value by dividing it with a peak factor. This factor depends on the type of oscillation, however for a pure sinusoidal oscillation it is $\sqrt{2}$ and for most buildings it can be set to approximately 3.5. Boggs [4] concluded that the rms-value was a better criteria than the peak acceleration due to a couple of reasons. There is more experience with using the rms-measurements, rms-values are easier to obtain and more consistent and they work better than peak accelerations based on the small amount of evidence that exists [4].

3.4.1 The effect of acceleration on humans

The human tolerance for motion is a very complex field and depends on many different factors like gender, age and individual sensitivity [64]. Humans perceive motion through the vestibular organs, proprioceptive sensations, auditory cues and visual cues. The combination of these determine a humans sensitive to motion [7]. Buildings have the ability to vibrate in translation and in rotational direction around its vertical axis. The acceleration in translation is referred to as linear acceleration and in rotation it is called angular acceleration or yaw. The angular acceleration also causes a linear acceleration that increases with the radius from the centre of rotation. Linear acceleration are primarily perceived by the vestibular system of the body whilst the angular acceleration are more detectable with visual cues [36]. The angular acceleration is measured in rad/s^2. Table 3.1 gives a basic overview of how humans perceive different accelerations.

Table 3.1: Human perception of acceleration [60].

Acceleration [m/s^2]	Effect
<0.05	Humans cannot perceive motion.
0.05-0.1	a) Sensitive people can perceive motion. b) Hanging objects may move slightly.
0.1-0.25	a) Majority of people will perceive motion. b) Level of motion may effect desk-work. c) Long term exposure may produce motion sickness.
0.25-0.4	a) Desk-work becomes difficult or almost impossible. b) Ambulation still possible.
0.4-0.5	a) People strongly perceive motion. b) Difficult to walk naturally. c) Standing people may lose balance.
0.5-0.6	Most people cannot tolerate the motion and are unable to walk naturally.
0.6-0.7	People cannot tolerate the motion.
>0.85	Objects begin to fall and people may be injured.

Research has been conducted in Japan to determine humans response to acceleration [4]. The result of this research can be seen in figure 3.1. The research listed the peak accelerations and matched them with human comfort, this laid the basis for the comfort requirements in Japan. Since the research was performed with sinusoidal oscillation the peak acceleration can be converted into rms values by dividing the peak value by $\sqrt{2}$, these rms values are listed on the right side of the plot. However the peak value for most buildings is generally around 3.5, so the relation between peak acceleration and rms acceleration in figure 3.1 is not true for buildings [4].

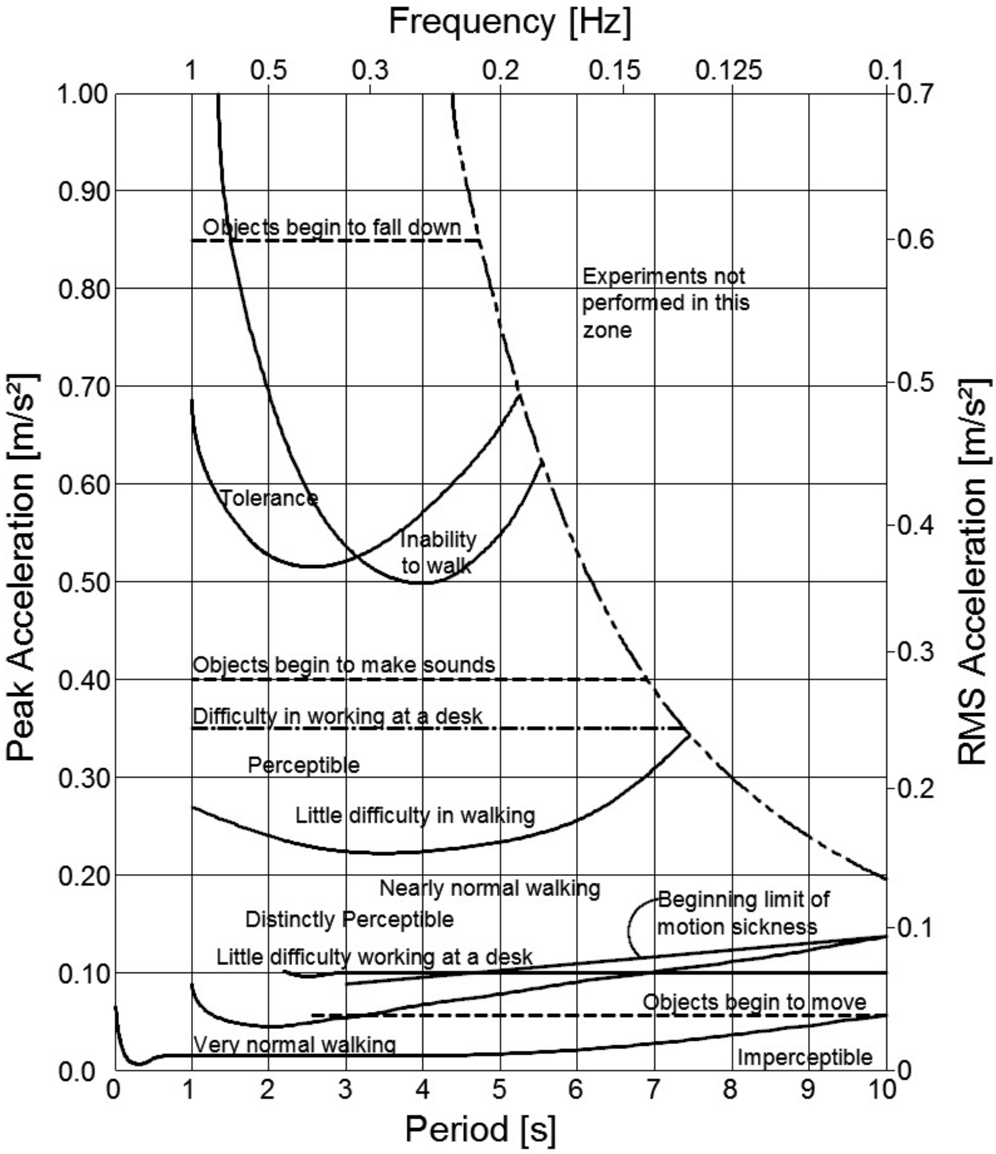
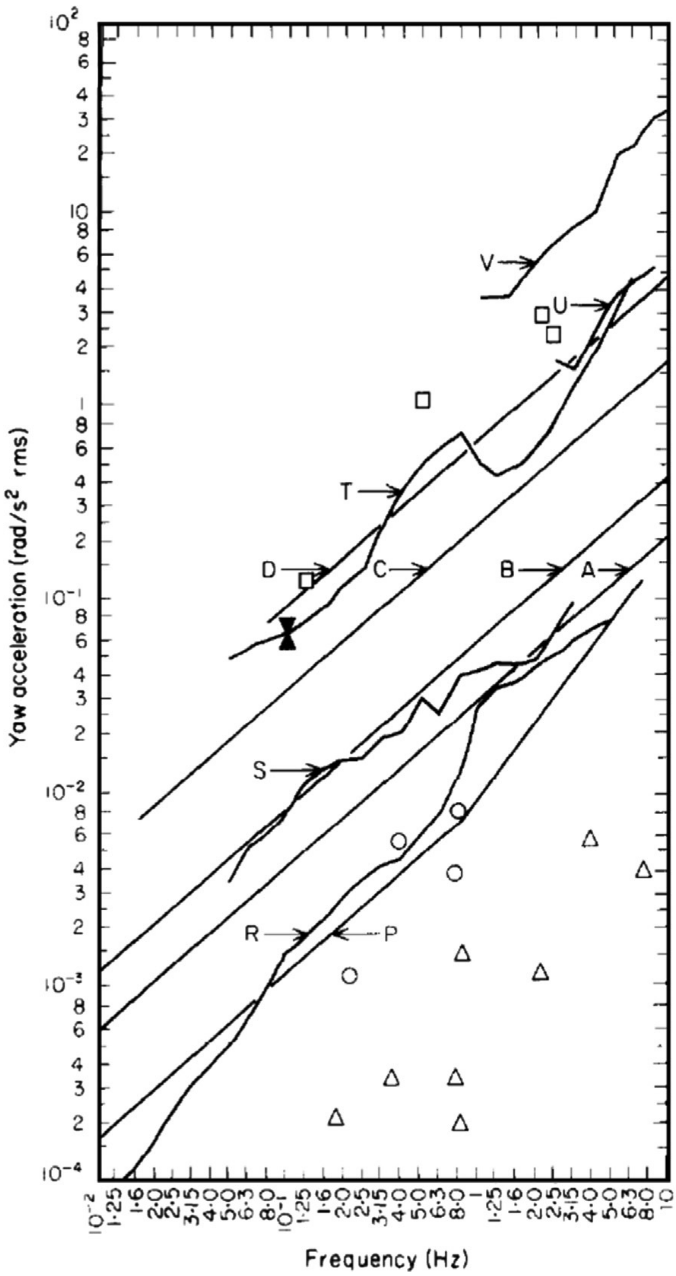


Figure 3.1: Human response to different acceleration levels at different frequencies [4].

The frequency and duration of the accelerations affect the way that humans perceive motion. Humans can tolerate higher accelerations at lower frequencies. At around 1-2 Hz humans are the most sensitive to acceleration and above that humans are less sensitive. Several studies have been performed with moving rooms to assess human comfort. All studies concluded that humans tolerate the least amount of accelerations around 1-2 Hz. Buildings with the same level of acceleration can be perceived differently if the accelerations last longer in one of the buildings. This can be seen in a study on airport control towers where a tower in Sydney received more complaints than one in Brisbane, despite both having acceptable levels of acceleration [14]. The return period also determines the acceptable levels of accelerations, a longer return period will naturally allow for higher accelerations. Further research has been conducted on the subject of angular acceleration. Based on the results of these studies, recommendation curves for angular accelerations were made, these are shown in figure 3.2. As can be seen in the figure, people can tolerate a higher acceleration with no visual cues (curve S) compared with if the people have visual cues (curve R).



Subtitle:

- Line A: minimum adverse comment (10 min maximum average for 1 year return period);
- Line B: minimum adverse comment (10 min maximum average 5 year return period);
- Line C: trained personnel, skilled task or decision (10 min maximum average 5 year return period);
- Line D: harness or restraint required to work on the building;
- Curve P: base perception of yaw vibration curve;
- Curve R: yaw acceleration mean perception curve, with visual cues;
- Curve S: yaw acceleration mean perception curve, no visual cues;
- Curve T: mean equal sensation of yaw acceleration contour;
- Curve U: yaw acceleration curve matched to the 1h fatigue-decreased proficiency ISO 2631 (vertical axis), tests on an open rig with an F105 seat;
- Curve V: yaw acceleration equal sensation contour for 10 Hz, 0.8 m/s² rms vertical reference acceleration.

Figure 3.2: Human response and recommendations to yaw acceleration [37].

3.4.2 Comfort requirements in standards

Eurocode does not include any information regarding the comfort requirements for acceleration due to wind loads. There are however other standards that include recommendations that can be used. The most well known criteria for accelerations are given by ISO 6897 [23], ISO 10137 [24], the National Building Code of Canada (NBCC) [54], The Architectural Institute of Japan Guidelines for the Evaluation of Habitability to Building Vibration (AIJ-GEH-2004) [1] and the Council on Tall Buildings and Urban Habitat (CTBUH) [21]. The requirements in these standards are based on measured values, below the origins for some of these standards are given.

Hansen et al. [30] performed one of the first studies of its kind in the early 1970's when two buildings were examined. From these tests a suggested level of comfort for acceleration was set to 0.005 g, in rms, for a wind load with a return period of 6-years. This was expected to result in 2% of the occupants complaining [30].

Irwin [38] suggested that established curves for comfort should be calibrated with the help of the measurements of Hansen et al. to create a new comfort criteria. This curve, which is shown in figure 3.3, was adapted into the international standard ISO 6897(1984) with the modification of having a 5-year return period for the wind instead of a 6-year return period.

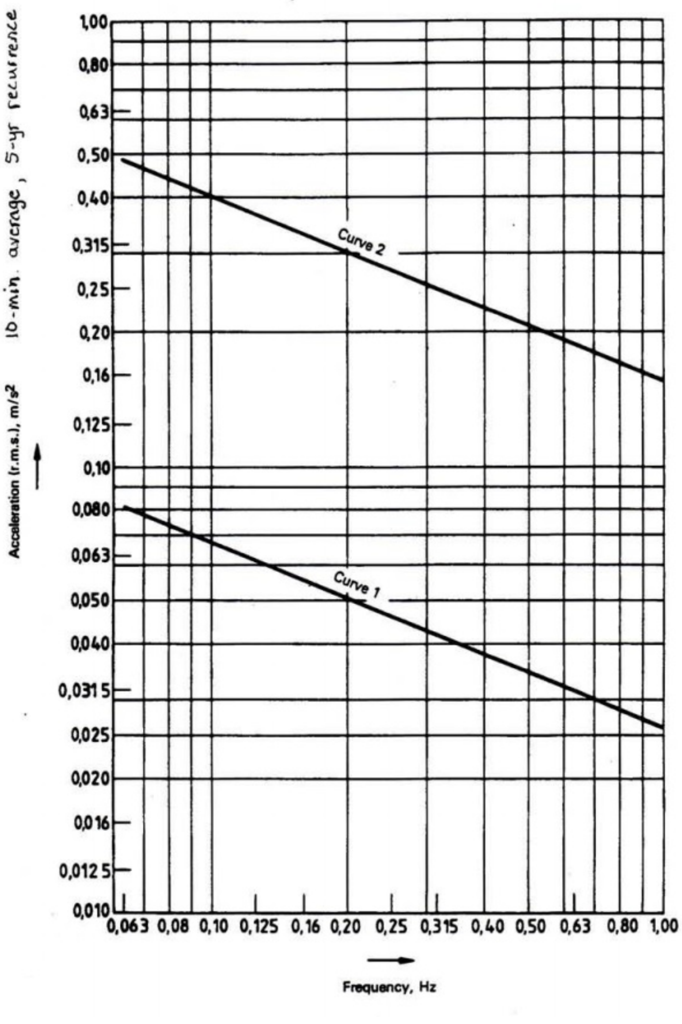


Figure 3.3: Comfort requirements according to Irwin [37].

After Irwin established the curves, Davenport [12] proposed two curves for comfort based on 2% and 10% of the occupants complaining. The curves were drawn to correlate with Hansen's criteria and with the known perception curves for weekly occurring winds. These curves were however based on peak values which meant that Hansen's values had to be converted into rms values, this was done by dividing the values with a peak factor of 3.5. Davenports curves were eventually taken into use in the Boundary Layer Wind Tunnel Laboratory (BLWTL) criteria of 0.020 g peak acceleration for a 10-year wind [4].

In figure 3.4 the values listed by some of these current standards are shown together. Some values have been recalculated to get a peak acceleration for a 1-year return period. The AIJ-GEH-2004 lines states how many per cent of the occupants that will percieve the vibrations. In figure 3.5 the requirements given by NBCC and CTBUH are shown. Figure 3.6 shows Irwin's suggested rms acceleration requirements that were later adopted into ISO 6897. Also ISO 10137 gives recommendations for peak accelerations, these are given in figure 3.7.

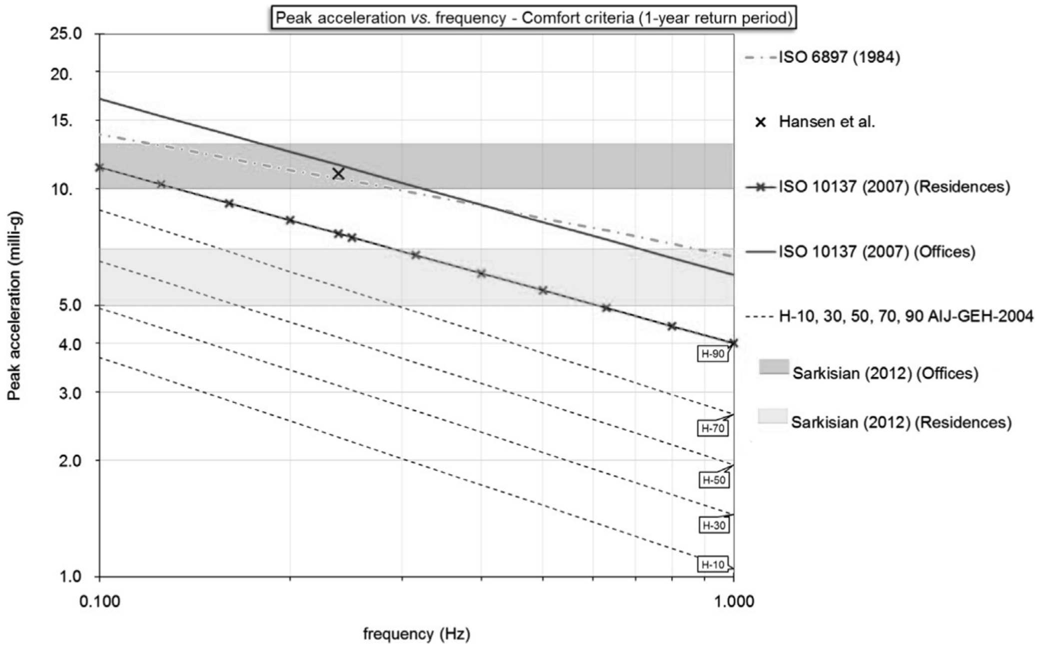


Figure 3.4: Peak acceleration values for comfort requirements according to different standards [20].

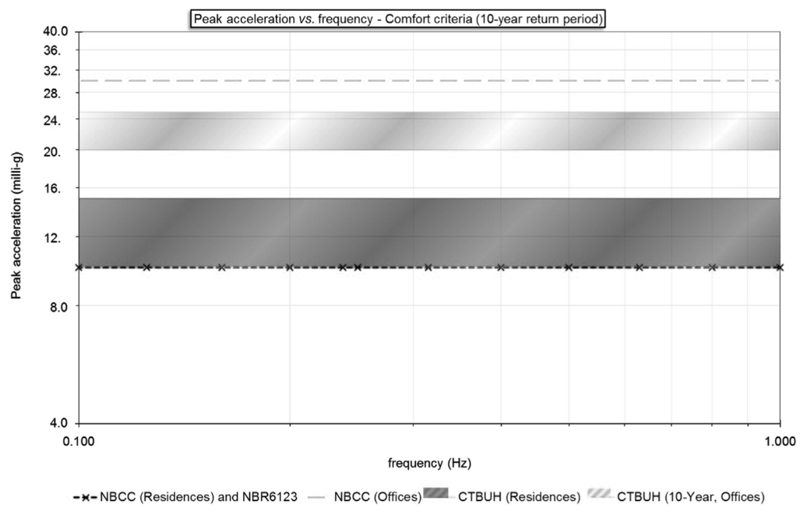


Figure 3.5: Peak acceleration values for comfort requirements according to NBCC and CTBUH [20].

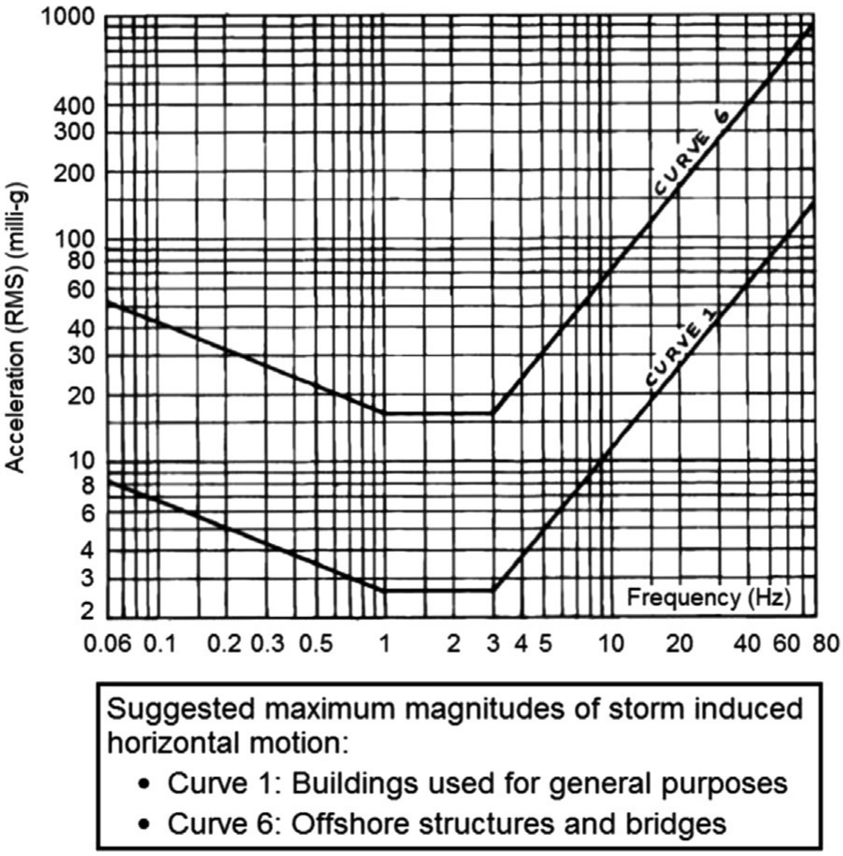


Figure 3.6: Suggested maximum rms acceleration by Irwin, later adopted into ISO 6897 [36].

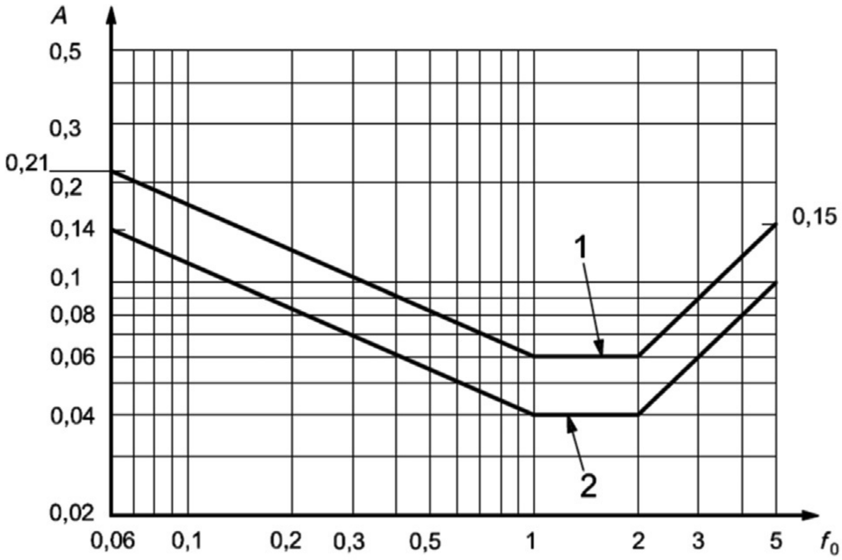


Figure 3.7: Peak acceleration (m/s^2) for human comfort according to ISO 10137. Curve 1 is for offices and curve 2 for residences [24].

3.5 Accidental loads and progressive collapse

Progressive collapse is defined as a collapse of a large part of a structure initiated by a smaller failure of a load bearing element. One of the most known examples of a progressive collapse is the Ronan Point building, where a small gas explosion on the 18:th floor knocked out a load bearing concrete panel. This caused the floors above to collapse leading to the entire side of the building collapsing [51], see figure 3.8.

Figure 3.8: Progressive collapse of Ronan Point building [51].

Since the collapse of the Ronan Point building in 1968, many building codes have addressed this type of failure. According to Eurocode 1990 [34], buildings are to be designed and executed to not take a disproportionate amount of damage from explosions, impacts or the consequences of human error, with regard to the severity of the load. What a building is supposed to withstand is decided for each individual project with the client and the authorities. Furthermore, a building should be able to withstand limited damage without the entire building collapsing.

The maximum size of a local failure, due to a collapse of a structural element should be limited to the smallest of 15% of the floor area or 100 m^2, in each of the two adjacent floors [5]. If the collapse of a structural element creates a local failure larger than what is listed above the structural element should be classified as a key element. According to A.8 in Eurocode 1991-1-7 [35], a key element of the structure should be able to resist a load A_d in vertical or horizontal direction. The value for A_d should be 34 kN/m^2 [35].

To take progressive collapse into consideration, buildings are put into three different consequence classes based on the type of the building. Class one is a low risk group which includes buildings where people rarely are located and class three is a high risk group. All buildings above 15 floors and buildings where a large amount of people are located are placed into consequence class three. If a building is in class three, a risk assessment should be done for the building, appendix B in Eurocode 1991-1-7 [35] gives an overview of how a risk assessment could be performed.

3.6 Seismic design

Sweden does not have any earthquake zones and the only buildings that are required to be designed for earthquakes are facilities that deal with radioactive materials. For very large structures such as bridges and high-rise buildings it is possible that clients want the structures to be able to withstand earthquakes. This is something that needs to be decided for each individual project. Seismic design is very important in seismic regions, however it will not be dealt with in this dissertation.

3.7 P-delta effect

In high-rise buildings, the P-delta effect can have a big impact on the overturning moment. The P-delta effect is a second order effect and is caused by the axial force and the displacement of the structure. When designing high-rise buildings in seismic zones this effect is of even greater importance due to the swaying of the buildings. If the swaying of the building is large and therefor creates a large displacement, the overturning moment could cause damage to the building [13]. To avoid damage or collapse due to the P-delta effect, the lateral stiffness or the strength of the building must be increased. Since this phenomenon is most common in seismic zones, calculations on the P-delta effect will not be included in this dissertation.

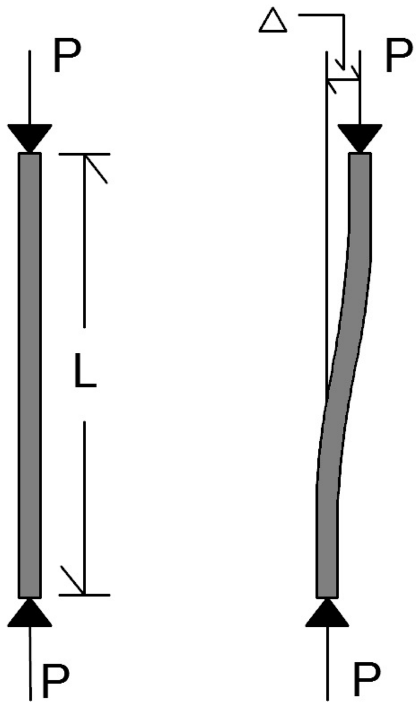


Figure 3.9: Illustration of P-delta effect [58].

3.8 Construction sequence analysis

Many buildings are only designed with respect to their completed form and loads during construction are largely ignored. The analysis of the structure during the construction stage for high-rises is equally important as for the completed building. This is because the vertical members can have a total shortening of several hundred millimeters and stresses during construction can exceed those after the building has been completed.

The construction sequence analysis can be divided into two parts, the first part covers the order in which loads are applied and the effects of the loads on the unfinished structure. During construction the loads in the structure might cause different stresses than they would if they were placed on a finished building. An analysis should then be performed after each story has been built, advanced FEM software can perform these analyses. The second part involves the effects of the time dependent effects creep and shrinkage [28].

3.9 Differential shortening

The absolute shortening of a column or a wall is the total deformation the element will receive between the time it has been cast and after a long time. This includes elastic deformation, creep and shrinkage. If a building is designed without taking shortening into consideration problems may arise with elevators, cracking of finishes and damages to pipes among other things [39].

When two elements receive different absolute shortening, a differential shortening arises. If the structure is statically indeterminate, internal forces arise that the beam might not have been intended to withstand. Differential shortening may also cause floors to be uneven. The higher up in a building, the more differential shortening there will be [50].

The effects of shortening are always present, no matter what the buildings height is. However, the differential shortening does not become large enough for it to be taken into considerations until the buildings height reaches around 40 floors [50]. The time spent on construction affects the amount of shortening. A slower construction will allow concrete to harden more and experience less shortening. The grade of concrete does not have a significant effect on differential shortening [39].

The most efficient way to deal with differential shortening is to let all vertical load bearing elements take the same stress. All deformations will then be the same assuming that the climate conditions are the same for all elements. This is however rarely possible because lateral loads are also carried by vertical members, which creates non-constant stresses. In tubular systems, with closely spaced perimeter columns and widely spaced interior columns, the interior columns have a fairly constant stress from gravity loads. The perimeter columns on the other hand carry a smaller load from gravity and provide stiffness to the building. Due to the stiffness generally being the designing factor of the structure, the perimeter columns will be larger than the interior columns. The vertical deformations will then be greater in the interior columns due to receiving larger stresses. For building with cores the effect is the opposite, the exterior columns will deform more than the core [63].

3.10 Soft story collapse

High-rise buildings often have less walls in the bottom floors to create more open spaces, and hence lower stiffness in comparison to higher up in the building. Lateral loads can cause the first story to collapse while the rest of the building resists the loads and remains standing [64]. In figure 3.10 an illustration of a soft story collapse is shown.

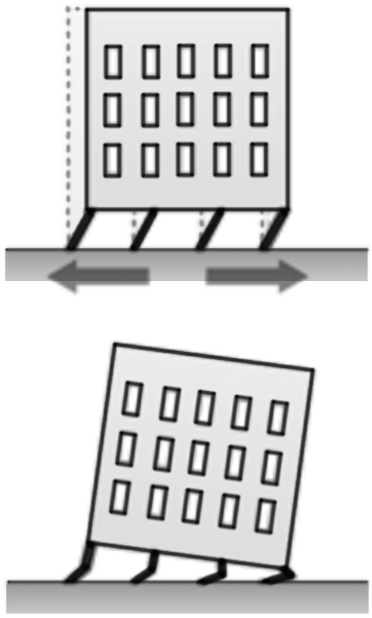


Figure 3.10: Illustration of a soft story collapse [56].

3.11 Damping

In order for a building to stop swaying after being excited, the vibration energy needs to be converted into thermal energy which is achieved with damping. All structures have a natural structural damping that will eventually stop all vibrations. Structural damping can be referred to as rate-independent damping, due to the damping being equally large no matter what the frequency of the vibration. The development of new high strength building materials and optimized structures has led to much lighter buildings than before. A lighter structure can lead to motion problems in the building, primarily due to the wind load. If the dynamic loads on the structure cause too large movements in the building, additional damping may need to be installed. Damping systems can be divided into passive and active systems. Passive systems are not as effective as the active systems, however they are cheaper and in general more reliable [44].

A system can, depending on the amount of damping, be classified as underdamped, critically damped or overdamped. In figure 3.11 three curves have been plotted to show the behaviour of the different systems after an initial displacement. A critically damped system will come to a stop without oscillating in the shortest time possible. An overdamped system will not oscillate, but will come to a stop slower than a critically damped system. An underdamped system will oscillate before coming to a stop. All buildings are underdamped and the structural damping is mainly related to the building material of the structural system, the structural damping is generally $\zeta = 0.1$ or below [10].

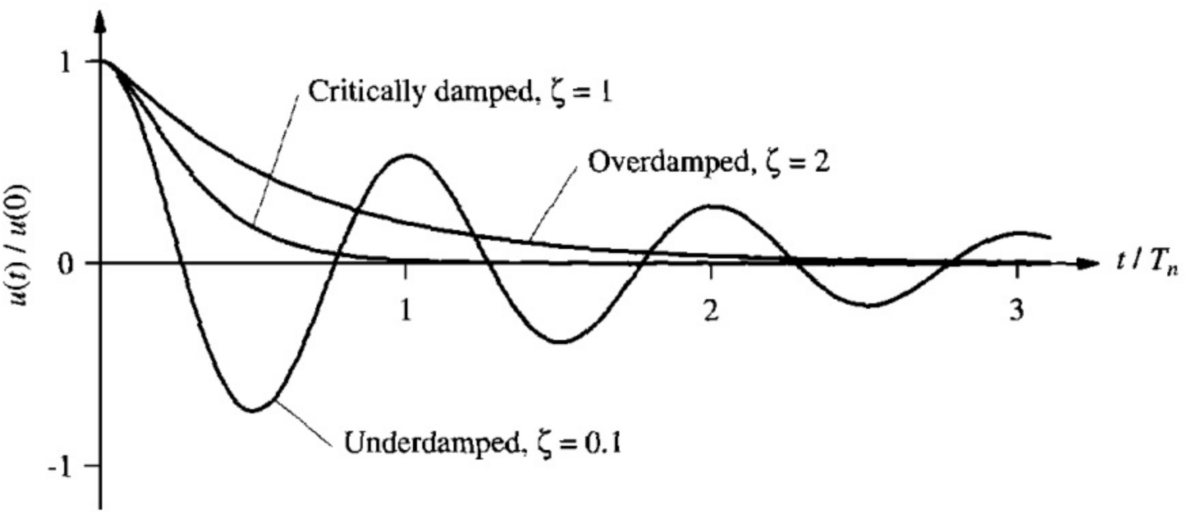


Figure 3.11: Free vibration of underdamped, critically damped and overdamped systems [10].

3.11.1 Passive systems

There are two types of damping systems, passive systems and active systems. The passive systems works either by dissipating energy or by using mass systems to generate counteracting inertia forces.

Tuned mass dampers consists of a mass at the top of the building, connected to the structural system with a spring and a damper. The damper is tuned to be out of phase with the natural frequency of the building and thereby dissipating energy when the building starts to move. The mass can be placed on bearings or a material with low friction. The springs and dampers are then placed between the mass and the vertical supports to the sides. The mass can also be placed on a pendulum which is suspended from the ceiling.

Instead of a solid mass, a liquid can be used. Tuned liquid dampers use water to create a counteracting inertia force. This system can be designed to use the water already in a building, such as a pool or a water tank. A tuned sloshing damper is a type of liquid damper that is made up of a tank that has a geometric design which enables the water waves to match the buildings fundamental frequency. Placing screens and baffles into the water enables energy dissipation. Figure 3.12 shows an illustration of a tuned sloshing damper.

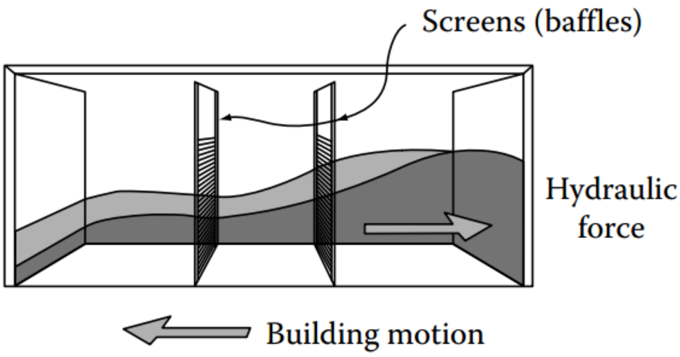


Figure 3.12: Tuned sloshing damper [64].

Another liquid based damping system is the tuned liquid column damper. The water is put in a tank with two columns and a horizontal passage connected in a U-shape. The horizontal passage contains obstacles that dissipate energy. The fundamental frequency of the tank is entirely based on the geometry of the tank. A tuned liquid column damper can be seen in figure 3.13.

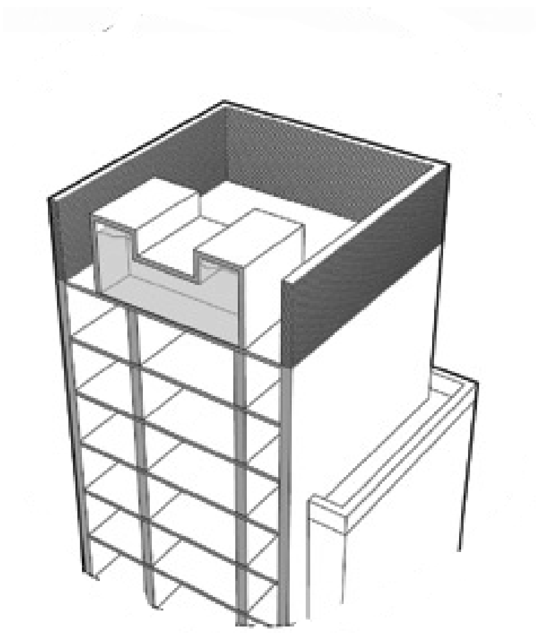


Figure 3.13: Tuned liquid column damper [62].

Viscous or viscoelastic dampers work by dissipating energy, they are installed into the primary structural system, e.g. by placing polymers between steel plates. The polymers work by dissipating energy when they are exposed to shear forces, in figure 3.14, an illustration of this type of damper is shown [44]. Unlike rate-independent damping, or structural damping, the viscous damper provides a higher damping for higher frequencies of motion [10].

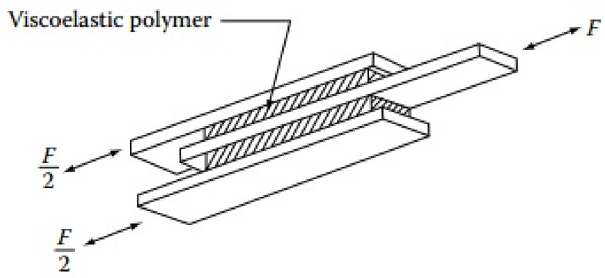


Figure 3.14: Illustration of viscoelastic damper [64].

Lately some research on the possibility to use electro-magnetic dampers in buildings has been performed. This is a type of damper that has been used a lot in the vehicle industry but not for buildings [2].

3.11.2 Active systems

With an active mass damper, the vibration of the building is picked up by a sensor and a computer then determines the optimal way to move a mass damper to reduce motions. Another system is an active variable stiffness device, this system adjusts the stiffness of the building to keep the frequency of the building away from that of the wind or seismic load. This type of system is very expensive and the reliability limits the use of active systems [44].

3.12 Wind tunnel tests

3.12.1 Introduction

Wind tunnel tests are common to perform when designing tall buildings, especially when designing complex structures and when the surrounding terrain results in complex wind flows. Eurocode deals with the local terrain effects on the building with the help of terrain factors. This method will be sufficient for lower buildings, but not for high-rise buildings. The terrain type depends, among other things, on the distance to the ocean and how much of the surrounding land that is settled, this means that the terrain type can change over the years. Performing wind tunnel test will lead to a more optimized structural system that will reduce costs [38].

The wind causes torsional loads and loads in the cross-wind direction, this is also something that some analytical models does not take into account. By replicating the surroundings and making a scale model of the building, good results for torsional movements and loads can be achieved with wind tunnel tests [38].

3.12.2 How wind tunnel test are performed

Wind tunnel tests are performed by blowing air from different angels on a scale model of the building, terrain and surrounding structures. The model normally consists of the building that should be measured and the objects within a radius of 300-1000 meters, depending on the size of the building and how complex the surrounding terrain is [3]. Normally models in scales between 1:200 to 1:400 are used. A typical wind tunnel is made of several components, including fan, flow straightener, roughness blocks and a turntable. See figure 3.15 for an illustration of a wind tunnel.

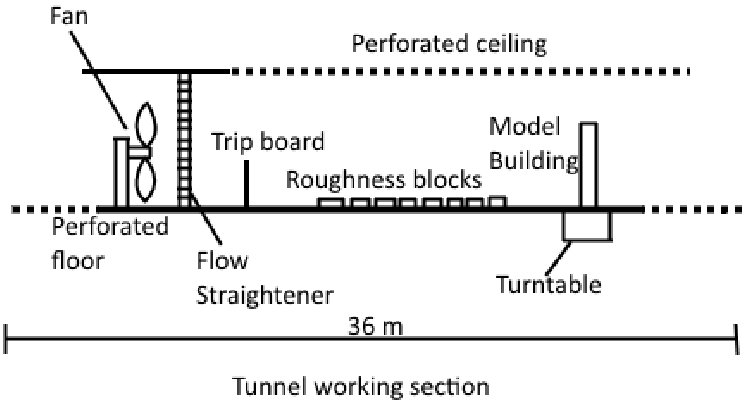


Figure 3.15: Illustration of wind tunnel tests [64].

A wind tunnel test used to find out the wind loads on the building normally consists of the following steps [3]:

- Replicate the real wind environment in the tunnel, this includes the wind speed and turbulence caused by both nearby buildings and the shape of the terrain.
- Create the scale model of the building and place it in the simulated environment, measure the response of the building.
- Analyse the results of the wind speeds and the dynamical response to obtain static-equivalent loads.
- Refine and optimize the structure.

3.12.3 Guidelines for when to perform wind tunnel tests

There are no rules on when or on what types of buildings it is necessary to perform a wind tunnel test. There are however some general guidelines on when it is recommended to perform a wind tunnel test [38]:

- The height of the building is over 120 meters.
- The height of the building is four times its average width.
- The lowest natural frequency of the building is less than 0.25 Hz.
- The reduced velocity is greater than five, $U/(f_0 \cdot b_{av})$ Where U is the mean wind velocity at the top of the building, f_0 is the lowest natural frequency and b_{av} is the average width.

- The building have a shape that is significantly different from the standard building shapes given in Eurocode 1991-1-4, chapter 7.
- The building is located in a complex environment which causes interaction effects between buildings or environment.

3.12.4 Input data for the wind tunnel tests

Before the wind tunnel tests can be performed, there are several important factors that must be considered. It is not only the building that needs to be in scale, it is also important that the flow of air in the wind tunnel represents the wind in full-scale which means that the scaled model must create the turbulence characteristics and wind loads corresponding to the real situation. Before the test can be performed some dynamic properties must be known [64]:

- Natural frequencies for the first six modes of vibration.
- Mode shapes for the first six modes of vibration.
- Mass distribution and stiffness, given for each floor.
- Damping ratio.
- Information such as floor height and overturning moments.

The wind conditions at the site of the buildings are also important to know in order to use wind tunnel results in a good way.

3.12.5 Different methods and output results

There are three different ways to determine the loads and responses on tall buildings with wind tunnel tests. The high-frequency-force-balance and the high-frequency-pressure-integration method rely on using a light and rigid model while the aeroelastic model method relies on using a model that accurately portrays the real buildings stiffness, mass distribution and geometry.

High frequency force balance (HFFB) is sometimes also referred to as the High-frequency-balance (HFB) or the High frequency base balance (HFBB) method. This method is based on measuring the force on the base of the building caused by the wind. To use this type of analysis, the scaled model must be light and stiff so the model does not start do vibrate by itself, since the model should only reflect the oscillations of the wind. This method is mainly used to obtain dynamic properties of the building [16].

High frequency pressure integration (HFPI) is based on measurements of pressure at several locations on the building. The main purpose of using the HFPI method is to obtain local pressures on the building in order to design cladding. However with sufficient pressure measurements the pressures can be integrated to obtain forces. Integrating the pressure over the entire building should yield the same results as the HFFB method. The pressure taps are primarily located around areas of the building where large pressure gradients arise, for example around corners and at the roof top. The HFPI tests give a fairly accurate result but is labor intensive and cannot be used on very slim buildings due to installation of pressure taps [16].

Aeroelastic model method uses a scaled model that matches the properties of the full scale building. This means that the mass, stiffness and damping must be modelled correctly to get a good result. This type of test is performed on buildings were the results, reaction forces, displacements and acceleration are also in scale and can be measured directly on the model. The aerodynamic damping can also be obtained from this test [3].

Aeroelastic models are more complicated and expensive than HFFB and HFPI tests and should only be performed when they are absolutely necessary. The following list contains some criteria for when an aeroelastic model might be needed [64].

- The slenderness is greater than 5-8.
- There is a likelihood of significant cross-wind response according to calculations.
- The structure is light, 1.2-1.6 kN/m^3.
- The structure has very low inherent damping, such as a building with welded steel construction.
- The structural system consists of a central concrete core which provides low torsional flexibility.
- The calculated natural frequency of the building is low, under 0.13 Hz.
- There are nearby buildings that could create strong torsional loads and strong buffeting action.
- The wind primarily blows in the direction in which the building is the most sensitive to wind.
- The building contains apartments or hotel rooms, these residents are more sensitive than office workers to dynamic effects.

Depending on what kind of test that is performed, a variety of results can be obtained. Below is a summary of the results that can be obtained from the tests.

- Forces and overturning moments.
- Detailed results of the pressure acting on the building and the pressure on an isolated part of the building.
- Displacements and accelerations of the building.

Chapter 4

Design and calculation methods

In this chapter the calculation methods used to analyse a high-rise building are explained. The chapter begins with the presentation of the calculation methodology for designing high-rise buildings in the preliminary stage. After that, the methods of calculating wind velocities, wind pressures and vertical loads according to Eurocode are explained. This is followed by how an idealised model of the building can be used to calculate the natural frequency. In the next part, the equations used to calculate the acceleration according to Eurocode and NBCC are shown.

4.1 Calculation methodology

One of the goals in this dissertation is to develop a methodology for the preliminary design of high-rise buildings. This has been done by literature studies, with the help of engineers from Skanska and by performing simulations on Gothenburg City Gate. In figure 4.1, the methodology is shown. The calculations used in the methodology are presented in chapter 4 and are used in the case study of Gothenburg City Gate in chapter 5.

1. Architectural drawings: The preliminary design starts with the architectural drawings, these drawings represent the base for the rest of the work.

2. Assessment of architectural drawings: The first step is to verify that it is possible to build what the architect has in mind. A load bearing system is determined based on economics and architecture of the building.

3. Vertical loads: Calculate the vertical loads in the building to obtain an approximation of the size and location of the load carrying components.

4. Idealised model: Based on the drawings, an idealised model of the building is made. This can for instance be a simple beam model with a varying stiffness.

5. Calculate the mass distribution and stiffness for the idealised model. The model should have the same mass distribution as the entire building, but the stiffness from only the main structural system can be used.

6. 2D-model: The next step is to make a 2D-model based on the idealised model. Strusoft Frame Analysis or similar program can be used to obtain good results. A

standard shape function according to Eurocode can be chosen, the 2D-model does not need to be made in that case.

7. The 2D-model is used to calculate the lateral displacement of the building, a unit load is therefore applied. When the lateral displacement is known a shape function can be adapted to the displacement.

8. Natural frequency: When the shape function for the building is known it can be used to calculate the natural frequency of the building. Once the natural frequency is known the design procedure can continue to the accelerations.

9. Acceleration: The along-wind acceleration of the building can be calculated according to Eurocode. The calculated accelerations must be evaluated and compared with comfort requirements, if they are within the limits the design procedure can continue to the FE-model.

10. FE-model: In this step, a 3D model of the building is created in Midas GEN or a similar program. Here more detailed results of the natural frequencies can be obtained. The results from the program should be used to evaluate the reliability of idealised calculations and to obtain the torsional natural frequencies that cannot be calculated with the idealised 2D-model.

11. Wind load: When the wind effect is considered it must be decided if wind tunnel tests should be performed or if it is enough to use Eurocode recommendations. High-rise buildings are normally tested in wind tunnels. If the results from the wind tunnel test are satisfactory the detailed design can begin.

12. Detailed design: When the preliminary design on the building is completed the detailed design can begin. This step is not included in this dissertation.

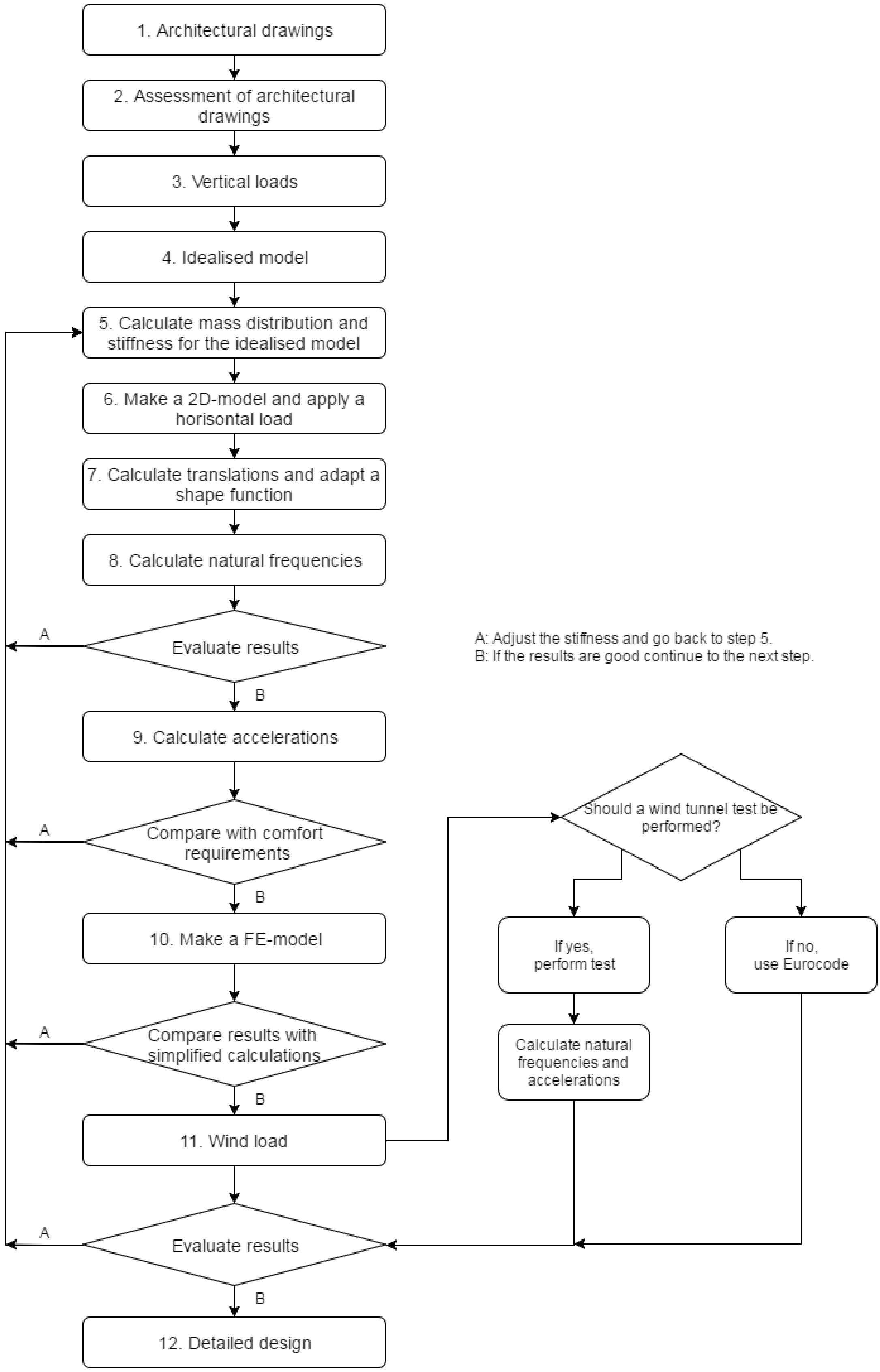


Figure 4.1: Calculation flowchart to follow when doing the preliminary design.

4.2 Wind velocities and wind pressures

The wind load can be described by a static part and a dynamic part. The static part is an average value and acts as a distributed load on the structure. For taller buildings, a dynamic part is needed to consider the dynamic nature of the wind. Most design codes use a dynamic amplification factor to take gust effects into account. In Eurocode, the reference peak velocity pressure, q_p, includes both the static and dynamic part of the wind load by using the turbulence intensity I_v. The factor c_d takes the dynamic response of the building into account when calculating equivalent static loads. EC 1991-1-4 deals with wind loads but is only designed for buildings up to heights of 200 meter.

The equation given in Eurocode to calculate the basic velocity pressure q_b is

$$q_b = \frac{1}{2} \cdot \rho_{air} \cdot v_b^2 \tag{4.1}$$

where v_b is the basic wind velocity and ρ_{air} is the air density (normally set to 1.25 kg/m^3).

The peak velocity pressure is calculated according to

$$q_p(z) = [1 + 6 \cdot I_v(z)] \cdot \left[k_r \cdot \ln\left(\frac{z}{z_0}\right) \cdot c_0(z)\right]^2 \cdot q_b \tag{4.2}$$

where I_v is the turbulence intensity given by equation 4.23. $c_0(z)$ is the orography factor, this needs to be considered if the terrain (e.g. hills or cliffs) increases the wind velocity by more than 5%. In other cases it can be set to 1.0. The value of k_r is calculated with equation 4.6 and z_0 is the roughness length, obtained from figure A.5 in appendix A.1. According to EC 1991-1-4 chapter 4.3.3 the effects of orography can be neglected if the slope of the upwind terrain is less than 3°. If it needs to be considered, EC 1991-1-4 A.3 gives direction for how it should be done. If the orography factor is set to 1.0, equation 4.2 can be simplified to the following expression

$$q_p(z) = c_e(z) \cdot q_b \tag{4.3}$$

The exposure factor $c_e(z)$ can be retrieved from figure A.7 in appendix A.1.

The mean wind velocity $v_m(z)$ at the height z above the terrain can be calculated with

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b \tag{4.4}$$

where v_b is the basic wind velocity, obtained from figure C-4 in EKS [5]. If calculations for comfort requirements are done, this value can be multiplied with 0.855 to get a load that is equivalent to a 5 year load according to EKS10 [5]. $c_r(z)$ is the roughness factor and is calculated with

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \tag{4.5}$$

where k_r is calculated according to

$$k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \tag{4.6}$$

where $z_{0,II}$ is 0.05 and z_0 is obtained from table A.5 in appendix A.1 based on the terrain type.

To obtain the mean pressure value, equation 4.1 is used but with the average wind velocity, v_m, instead of the basic wind velocity.

$$q_m = \frac{1}{2} \cdot \rho_{air} \cdot v_m^2 \tag{4.7}$$

4.3 Loads

There are many different kinds of actions on a building. The structural system must resist both vertical and lateral loads. Different loads are listed below.

- Permanent loads: Self weight from structural members, non-structural members and self weight from installations.
- Imposed loads: Loads from occupants, furniture and snow.
- Horizontal load: Wind load
- Other loads: Seismic loads and accidental loads.

When choosing the characteristic values of the vertical live loads, table 6.1 in EC 1991-1-1 should be used [32]. When designing high rise buildings there are some specific requirements listed in Eurocode, these are discussed in the section below.

4.3.1 Vertical loads

According to EC 1991-1-1 chapter 6.2.2(2), the live load from a building with multiple stories can be reduced with a factor α_n according to 6.3.1.2(11).

$$\alpha_n = \frac{2 + (n-2)\,\psi_0}{n} \tag{4.8}$$

where n is the number of stories with the same load type above the loaded structural part and ψ_0 is a factor according to figure A.9 in appendix A.1.

A reduction factor can also be added based on the size of the floor according to Eurocode 1991-1-1 chapter 6.3.1.2(10) [32]. The reduction factor, α_A, is calculated with

$$\alpha_A = \frac{5}{7} \cdot \psi_0 + \frac{A_0}{A} \leq 1.0 \tag{4.9}$$

where A_0 is a constant set to 10 m^2 and A is the loaded area.

According to the newer EKS, these two factors can be combined when counting on specific load sets. All load sets are listed in chapter A1.3 in Eurocode 1990 and in EKS. The reduction factors can be combined for set B when calculating loads for category A and B (living area and office space) while using load combination 6.10b. They can also be combined for set C under category A and B for load combination 6.10 [5].

4.3.2 Wind loads

The force acting on a structure or structural component can be calculated with the following equations

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{4.10}$$

where c_f is the force coefficient, $q_p(z_e)$ is the peak velocity pressure according to equation 4.3 and A_{ref} is the reference area. c_f can be obtained from section 7 and 8 in EC 1991-1-4.

The factor $c_s c_d$ takes the non-simultaneous occurrence of peak wind pressures on the surface (c_s) and the effect of the vibration of the structure due to turbulence (c_d) into account, c_s and c_d should not be separated. The combined value of the two variables can be calculated according to the equation below from EKS.

$$c_s c_d = \frac{1 + 2 \cdot k_p \cdot I_v\left(z_s\right) \cdot \sqrt{B^2 + R^2}}{1 + 6 \cdot I_v\left(z_s\right)} \tag{4.11}$$

where k_p, B and R are variables calculated according to equation 4.24, 4.26 and 4.27.

For a building with a height of less than 15 m, $c_s c_d$ is set to 1. Annex D in EC 1991-1-4 gives graphs for determining $c_s c_d$ for multistory buildings, however EKS does not allow this annex to be used in Sweden [33].

4.4 Shape functions

When lateral loads act on a building it will deflect and shape functions can be used to describe the deflection of the building. Shape functions can be used to calculate the translation of the building, the generalised stiffness and generalised mass of the building. The generalised stiffness and the generalised mass can then be used to calculate natural frequencies if no detailed 3D-model of the building has been made. Since the shape function is dependent on the load, it can be difficult to find a shape function that is appropriate for both along-wind exponential load, cross-wind uniform wind load and seismic loads [10].

According to Eurocode, the shape function of a building can be assumed to be

$$\phi(z) = \left(\frac{z}{h}\right)^{\zeta} \tag{4.12}$$

where the value of ζ is chosen based on the type of structural system in the building, see table 4.1

Table 4.1: ζ-value according to Eurocode.

ζ	Description
0.6	Slender frame structures with non-load-sharing walling or cladding.
1.0	Buildings with a central core plus peripheral columns or larger columns plus shear bracing.
1.5	Slender cantilever buildings and buildings supported by central reinforced concrete cores.
2.0	Towers and chimneys.
2.5	Lattice steel towers.

The second derivative of the shape function in equation 4.12 is used in the calculation of the natural frequency and is

$$\phi(z)'' = \frac{(\zeta^2 - \zeta)}{h^2} \left(\frac{z}{h}\right)^{(\zeta-2)} \tag{4.13}$$

4.5 Natural frequencies

The shape function can be used to calculate the natural frequency of the building if using a simple beam model. The first natural frequency of a single degree of freedom system can be calculated as described below. To calculate higher modes, computers are commonly used. The building is here idealised as a cantilever beam, see figure 4.2.

For a building with a varying stiffness the method below should be used.

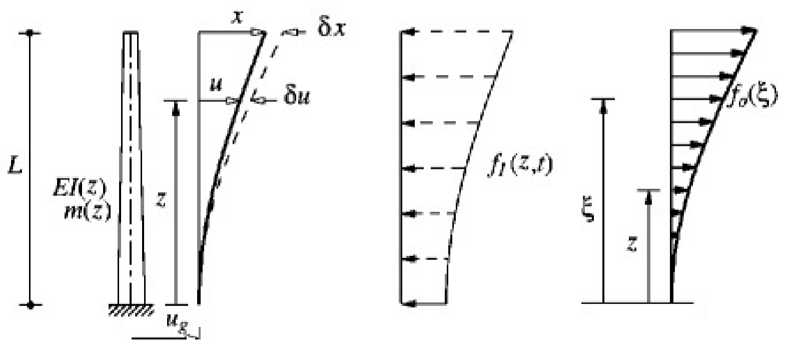


Figure 4.2: Displacement in building when subjected to a uniformly distributed load [10].

The first natural frequency can generally be calculated with equation 4.18 where $\widetilde{k}$ is the generalised stiffness and $\widetilde{m}$ is the generalised mass. The equation of motion can be used to calculate the generalised stiffness and generalised mass [10]

$$\widetilde{m} = \int_0^L m(x)[\phi(x)]^2 dx \tag{4.14}$$

$$\widetilde{k} = \int_0^L EI[\phi''(x)]^2 dx \tag{4.15}$$

where ϕ is the shape function of the structure. The shape function must satisfy the boundary conditions, for the idealised model used in this case the boundary conditions for the base of the building are $\phi(0) = 0$ and $\phi'(0) = 0$ [10].

When the generalised mass and generalised stiffness are known the natural frequency can be calculated. The angular frequency can be calculated with

$$\omega = \sqrt{\frac{\widetilde{k}}{\widetilde{m}}} \tag{4.16}$$

and the natural frequency can then be calculated with

$$f = \frac{\omega}{2\pi} \tag{4.17}$$

Normally equation 4.16 and 4.17 can be written together as

$$f = \frac{1}{2\pi}\sqrt{\frac{\widetilde{k}}{\widetilde{m}}} \tag{4.18}$$

The first natural frequency can generally be calculated with equation 4.19, where ρ is the buildings mass per unit of length, EI is the stiffness and L is the height of the building. According to Nicoreac and Hoenderkamp [53], this equation is generally accepted for calculating the natural frequency. This applies for a building similar to a cantilever girder, with a constant stiffness and mass. This is not true for high-rise buildings and another model that takes the varying stiffness into account should provide a more reliable result [53].

$$\omega = 1.875^2\sqrt{\frac{EI}{\rho L^4}} \tag{4.19}$$

4.6 Accelerations

To calculate the acceleration in the top of the building due to the dynamic loads, Eurocode can be used, however only the acceleration for the first mode can be calculated. Eurocode only includes the along-wind sway and not the motion in the cross-wind direction. To calculate the cross-wind acceleration the Canadian building code can be used. The equations are empirical and based on wind tunnel tests.

4.6.1 Eurocode

After the natural frequency is known, the acceleration can be calculated with Eurocode. Eurocode includes two separate methods to calculate the acceleration. In the Swedish national annex there is a method that is to be used in Sweden, the calculation method below follows this method [5].

The maximum acceleration $\ddot{X}_{\max}(z)$ can be calculated according to

$$\ddot{X}_{\max}(z) = k_p \cdot \sigma_{\ddot{X}}(z) \tag{4.20}$$

where k_p is the peak factor calculated with equation 4.24 and $\sigma_{\ddot{X}}(z)$ is the standard deviation of the acceleration, or rms, given by

$$\sigma_{\ddot{X}}(z) = \frac{3 \cdot I_v(h) \cdot R \cdot q_m(h) \cdot b \cdot c_f \cdot \phi_{1,x}(z)}{m} \tag{4.21}$$

where R is the resonant response and is calculated with equation 4.27, $q_m(h)$ is the reference mean (basic) velocity pressure at height h and is calculated with equation 4.7, m is the mass per unit length, b is the width of the building and $\phi_{1,x}(z)$ is the fundamental along-wind modal shape, see equation 4.12. c_f is the force coefficient, for a rectangular section the equation is given by

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda \tag{4.22}$$

where $c_{f,0}$ is the force coefficient, ψ_r is the reduction for rounded corners and ψ_λ is the end-effect factor. See figure A.1, A.2 and A.3 in Appendix A.1.

I_v is the turbulence intensity and is calculated with

$$I_v = \frac{k_l}{c_0(z) \cdot \ln(z/z_0)} \tag{4.23}$$

where k_l is the turbulence factor and is set to 1.0 according to the Swedish national appendix, $c_0(z)$ is the orography factor according to Appendix A.3 in Eurocode 1991-1-4 [33], it takes into account the increase in wind speed over isolated hills and escarpments. Normally the orography factor can be set to 1.0 for the purpose of high-rises. z_0 is the roughness length, see figure A.5 in Appendix A.1.

To calculate the peak acceleration, the peak factor k_p in equation 4.20 must be known. It is calculated with

$$k_p = \sqrt{2 \cdot \ln(v \cdot T)} + \frac{0.6}{\sqrt{2 \cdot \ln(v \cdot T)}} \tag{4.24}$$

where v is the average fluctuation rate, calculated with equation 4.25 and T is the average time for reference wind velocity, normally 600 s is used according to Eurocode. v is calculated with

$$v = n_{1,x} \frac{R}{\sqrt{B^2 + R^2}} \tag{4.25}$$

where $n_{1,x}$ is the fundamental frequency of along wind vibration. B is the background response factor and B^2 is calculated according to

$$B^2 = \exp\left[-0.05\left(\frac{h}{h_{ref}}\right) + \left(1 - \frac{b}{h}\right)\left(0.04 + 0.01\left(\frac{h}{h_{ref}}\right)\right)\right] \tag{4.26}$$

where h is the height of the building and h_{ref} is the reference height of the building. h_{ref} is given by figure A.6 in Appendix A.1.

R is the resonant response factor and R^2 is given by

$$R^2 = \frac{2 \cdot \pi \cdot F \cdot \phi_b \cdot \phi_h}{\delta_s + \delta_a} \tag{4.27}$$

where δ_s is the structural logarithmic decrement of damping and δ_a is the aerodynamic logarithmic decrement of damping. δ_s is selected according to A.8 in Appendix A.1 and δ_a is calculated according to equation 4.28. ϕ_b and ϕ_h are calculated with equation 4.31 and 4.32.

The aerodynamic damping is calculated with equation.

$$\delta_a = \frac{c_f \cdot \rho \cdot b \cdot v_m(z_s)}{2 \cdot n_1 \cdot m_e} \tag{4.28}$$

where c_f is calculated with equation 4.22, ρ is the air density, b is the width of the building, $v_m(z_s)$ is the mean wind velocity at the reference height, n_1 is the lowest fundamental frequency and m_e is the equivalent mass per unit length. A cantilevered structure with a varying mass distribution m_e may be approximated by the average value of m over the upper third of the structure [33].

F is calculated with

$$F = \frac{4 \cdot y_C}{(1 + 70.8 \cdot y_C^2)^{\frac{5}{6}}} \tag{4.29}$$

where y_C is calculated with

$$y_C = \frac{150 \cdot n_{1,x}}{v_m(h)} \tag{4.30}$$

where $n_{1,x}$ is the fundamental frequency in along wind direction and $v_m(h)$ is the average wind velocity.

$$\phi_b = \frac{1}{1 + \frac{3.2 \cdot n_{1,x} \cdot b}{v_m(h)}} \tag{4.31}$$

$$\phi_h = \frac{1}{1 + \frac{2 \cdot n_{1,x} \cdot h}{v_m(h)}} \tag{4.32}$$

4.6.2 NBCC

To calculate the acceleration in cross wind direction, the Canadian building code NBCC [54] is used. First the value of a_r is calculated according to

$$a_r = 78.5 \cdot 10^{-3} \left[\frac{V_H}{n_W \sqrt{W \cdot D}} \right]^{3.3} \tag{4.33}$$

where n_W is the buildings frequency perpendicular to the wind, W is the width of the building in meter (perpendicular to wind) and D is the depth of the building in meter (parallel to wind). V_H is the mean wind speed at the top of the structure given by

$$V_H = \bar{V} \sqrt{C_{eH}} \tag{4.34}$$

where $\bar{V}$ is the reference wind speed at 10 m and C_{eH} is the exposure factor at the top of the building according to figure B.1 in Appendix B.1

The acceleration of the building in the cross-wind direction is calculated according to

$$a_w = n_W^2 \cdot g_p \sqrt{W \cdot D} \left(\frac{a_r}{\rho_B \cdot g \sqrt{\beta_W}} \right) \tag{4.35}$$

where ρ_B is building density, g is the gravitational acceleration, β is the critical damping ratio with commonly used values of 0.01 for steel, 0.015 for composite and 0.02 for cast-in place concrete buildings. g_p is the peak factor which can be calculated with equation 4.36 or with figure B.2 in Appendix B.1.

$$g_p = \sqrt{2 \cdot \ln(v \cdot T)} + \frac{0.577}{\sqrt{2 \cdot \ln(v \cdot T)}} \tag{4.36}$$

T is 3600 s and v is the average fluctuation rate, cycles/s. NBCC calculates v with the following equation.

$$v = n_0 \sqrt{\frac{s \cdot F}{s \cdot F + \beta \cdot B}} \tag{4.37}$$

where n_0 is the fundamental frequency, F is the gust energy ratio from figure B.5, B is the background response factor from figure B.3 and s is the size reduction factor from figure B.4. All figures can be found in Appendix B.1

The along-wind acceleration can be calculated with

$$a_D = 4 \cdot \pi^2 \cdot n_D^2 \cdot g_p \sqrt{\frac{K \cdot s \cdot F}{C_e \cdot \beta_D}} \cdot \frac{\Delta}{C_g} \tag{4.38}$$

where n_D is the fundamental frequency parallel to the wind direction, g_p is the peak factor according to equation 4.36, K is a factor related to the surface roughness coefficient of terrain, 0.08 for exposure A, 0.10 for B and 0.14 for C. s is the size reduction factor, F is the guest energy ratio, Δ is the maximum wind-induced lateral displacement in the along-wind direction, typically obtained from computer analysis. A value of H/450 can be used in preliminary analysis. C_e is the exposure factor, referred to as C_{CH} earlier, β_D is the critical damping ratio in along wind direction and C_g is the gust response factor which can calculated with

$$C_g = 1 + g_p \left(\frac{\sigma}{\mu} \right) \tag{4.39}$$

where $\frac{\sigma}{\mu}$ can be calculated with

$$\frac{\sigma}{\mu} = \sqrt{\frac{K}{C_{eH}} \left(B + \frac{s \cdot F}{\beta} \right)} \tag{4.40}$$

C_{eH} is the exposure factor for the top of the building.

Another equation can be used if the maximum deflection is set to be related to the fundamental frequency of the building with a linear modal representation of the building motion. This equation is

$$\frac{a_D}{g} = g_p \sqrt{\frac{K \cdot s \cdot F}{C_e \cdot \beta_D}} \left(\frac{3.9}{2+\alpha}\right) \left(\frac{C_e \cdot q}{D \cdot \rho_B}\right) \tag{4.41}$$

where α is a power coefficient related to C_e, $\alpha = 0.28$ for exposure A, $\alpha = 0.50$ for exposure B and $\alpha = 0.72$ for exposure C. D is the depth parallel to the building, ρ_B is the mass density of the building and q is the reference wind pressure, $q = 650 \cdot 10^{-6} \cdot \bar{V}^2$ with $\bar{V}$ in m/s.

The resulting peak acceleration can be calculated by combining the along wind and cross wind acceleration according to

$$a_{tot} = \sqrt{(a_w^2 + a_D^2)} \tag{4.42}$$

4.7 Wind effects considered by Eurocode

Wind loads play an integral part in the design of high-rise buildings and careful consideration needs to be taken for both the static and the dynamic nature of the wind load.

4.7.1 Galloping

The galloping phenomenon starts at a specific velocity, depending on the size, type, stiffness and mass of the building. The amplitude of the galloping increases when the wind speed increases. The wind speed at which the galloping starts can according to Eurocode 1991-1-4 [33] be calculated with

$$v_{CG} = \frac{2 \cdot Sc}{a_G} \cdot n_{1,y} \cdot b \tag{4.43}$$

where a_G is the factor of galloping instability, see figure A.10 in Appendix A.1. $n_{1,y}$ is the cross wind fundamental frequency, b is the width of the building and Sc is Scrutons number and can be calculated with

$$Sc = \frac{2 \cdot \delta_s \cdot m_{i,e}}{\rho \cdot b^2} \tag{4.44}$$

where δ_s is the structural logarithmic decrement of damping, see figure A.8 in Appendix A.1. $m_{i,e}$ is the equivalent mass m_e per unit length for mode i as defined in Eurocode 1991-1-4 annex F.4(1) [33]. For a cantilever structure with a distributed mass, the value can be set as the average value of m over the upper third of the structure. ρ is the density of air.

It should be ensured that

$$v_{CG} \leq 1.25 \cdot v_m(z) \tag{4.45}$$

to avoid risk of galloping.

The height z is the height at which the galloping excitation is expected, most likely the top of the building.

4.7.2 Vortex shedding

Eurocode 1991-1-4 appendix E.1 deals with vortex shedding, however this is not to be applied in Sweden. Instead the standard BSV 97 *Snö och vindlast* [52] should be used. The frequency of the vortex shedding for the building can be calculated according to

$$f_v = St \cdot \frac{v}{d} \tag{4.46}$$

where St is the Strouhal number, a proportionality constant between the velocity of the wind and the frequency of the vortex shedding which can be chosen from figure A.11 in Appendix A.2. v is the wind speed and d is the width of the structure. Vortex shedding will occur if the frequency, f_v, matches the fundamental frequency of the building, f_0.

A critical wind load can be calculated based on the fundamental frequency of the building. If the critical wind load is greater than the characteristic wind load, vortex shedding does not need to be considered according to BSV 97. The critical wind velocity is

$$v_{cr} = \frac{f_0 \cdot d}{St} \tag{4.47}$$

If vortex shedding is to be considered, a load can be calculated according to BSV 97. Furthermore, if the critical velocity for vortex shedding, v_{cr}, and the onset wind velocity for the galloping, v_{CG}, are near each other interacting effects are likely to occur. If that is the case, Eurocode recommends getting a specialist advice. To see if the two wind velocities are too close the following limits are used

$$0.7 < \frac{v_{CG}}{v_{cr}} < 1.5 \tag{4.48}$$

According to Eurocode, the effect of vortex shedding does not need to be taken into account when

$$v_{crit,j} > 1.25 \cdot v_m \tag{4.49}$$

where

$$v_{crit,j} = \frac{b \cdot n_{i,y}}{St} \tag{4.50}$$

where b is the width of the building, $n_{i,y}$ is the cross wind fundamental frequency and St is Strouhals number.

4.8 Outriggers

How the structural system of outriggers works is described in section 2.4.3. In the following chapter calculation methods that can be used for outriggers are laid out.

4.8.1 Optimal vertical placement of outriggers

The vertical placement of the outrigger is of great importance to how it functions. A uniformly distributed load will cause a rotation around the horizontal plane of the structure. The exterior columns will provide stiffness and a counteracting rotation to the structure, leading to lower deflections and smaller moments at the base of the building. The columns can be seen as springs providing a higher stiffness the more they are compressed or elongated [64].

The size of the counteracting rotation depends on two factors, the stiffness of the equivalent spring and the magnitude of the rotation of the cantilever at the outrigger location due to lateral loads. The spring stiffness grows larger the shorter the columns are under the outrigger, so to provide the maximum amount of spring stiffness the outrigger should be placed at the bottom of the building. However, the magnitude of the rotation is largest at the top of the building and the smallest at the bottom. So the optimal location to place the outrigger should be somewhere in between. It turns out that the optimum location is $0.455H$ from the top of the building if certain assumptions are made, such as an even distributed load, a prismatic core, the outrigger is infinitely rigid etc. If two outriggers are placed in a building they should be placed at $1/3H$ and $2/3H$ for an optimal function. The same logic follows for more outriggers [64].

The placement of outriggers is however often restricted by the architectural requirements of the building. The outriggers are very intrusive on the floor they are placed on and are therefore often put on floors that are dedicated to elevators and ventilation [64].

4.8.2 Deflection, moment and stiffness with uniform core and columns

Smith and Coull [60] laid out the basic calculation method for outrigger design for a building with a uniform core and uniform columns. With this method, a deflection at the top can be calculated, an equivalent stiffness for the entire building and the moment that the outrigger adds to the structure. The wind load is assumed to act as a uniformly distributed load.

The moment applied from the outrigger at the location of the outrigger can be calculated with

$$M = \frac{w(H^3 - x^3)}{6EI_{core}} \left[\left(\frac{1}{EI} + \frac{2}{EA_{column} \cdot d^2} \right) \cdot (H - x) + \left(\frac{d}{12EI_{outrigger}} \right) \right]^{-1} \tag{4.51}$$

where w is the evenly distributed load, H is the height of the building, EI_{core} is the bending stiffness of the core, EI is the stiffness of the entire building, $EI_{outrigger}$ is the stiffness of the outrigger, A_{column} is the area of the columns that are active in the outrigger system and d is the distance between the outer columns.

The deflection at the top can be calculated as the deflection of a cantilever girder minus the reduced deflection from the moment that the outrigger provides according to

$$y_{top} = \frac{wH^4}{8EI_{core}} - \frac{M(H^2 - x^2)}{2EI_{core}} \tag{4.52}$$

A stiffness for the core combined with the outrigger can be calculated with

$$(EI)_{building} = EI + \frac{d^2(EA)_{column}}{2} \tag{4.53}$$

If, as is most common, the core and the columns are not prismatic the calculation method complicates significantly. A calculation method for such a case has been investigated by Cheok, Er and Lam [9].

Chapter 5

Preliminary design calculations

In this chapter a study of an ongoing project is performed. Some preliminary design calculations are carried out which follow the working methodology displayed in chapter 4.1. In addition to the calculations, analyses for different idealisations and stiffness adjustments are performed.

5.1 Purpose

The purpose of the case study is to show the idealised models and calculations that can be used in preliminary design of a high-rise building. The chapter follows the calculation methodology described in section 4.1. After the calculations are performed, FE-analysis using the commercial softwares Strusoft FEM-design and Midas GEN are carried out and the results are compared with the results of the idealised models.

5.2 Description of building

This section represents the first three steps of the methodology, see figure 5.1

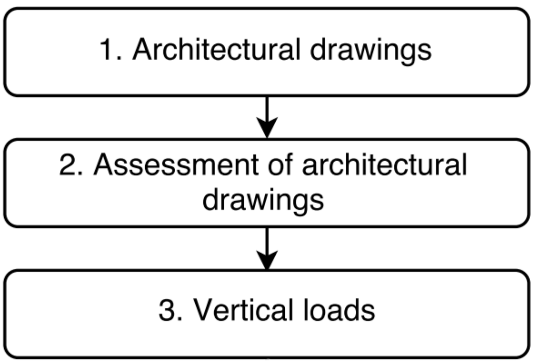


Figure 5.1: First three steps of methodology.

To evaluate and analyse the effects described in chapter 4, a study of an ongoing project was carried out. The building in question is Gothenburg City Gate, a 34 stories, 120 meter high office building. The construction has not started yet, but the project is expected to be completed in 2019. The structural system of the building consists of a concrete core, concrete columns and steel beams. The core withstands vertical, torsional and lateral loads while the columns at the perimeter of the building only carry vertical loads. The steel beams connects the columns at the perimeter to the central core. Skanska provided architectural drawings of the building and preliminary dimensions of structural elements were estimated based on previous similar projects. Based on these estimations, preliminary design calculations according to chapter 4 were carried out. In addition to the 120 meter tall tower there is a lower ten stories tall building connected to the higher building. In figure 5.2 an illustration of the building is shown.

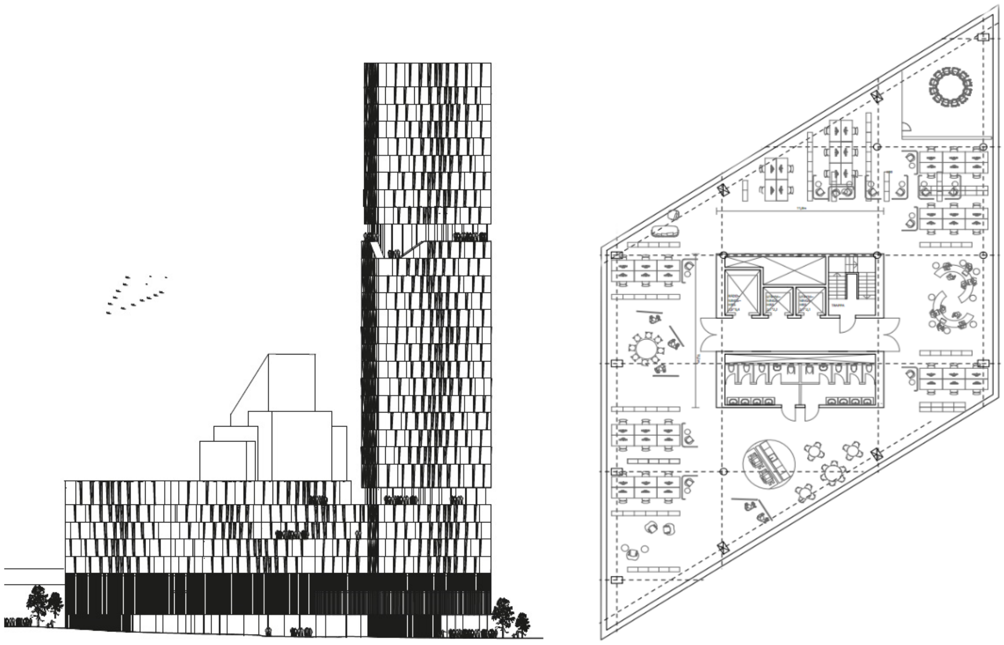


Figure 5.2: Illustration of Gothenburg city gate with typical floor plan.

5.2.1 Building properties

The different properties of the building needed for the calculations are listed below:

- Height: 120 meter
- Number of stories: 34
- Story height: 3.6 meter
- Structural system: Concrete core around elevator and stair shaft, concrete columns around the perimeter of the building and steel beams connecting the columns to the core.
- Floor: HDF 120/27
- Height of side building: 10 stories
- The column dimension on the bottom section (floor 0-10) is 900x900 mm, the middle section (floor 11-21) 700x700mm and the top section (floor 22-34) 500x500 mm.

5.2.2 Loads and material properties

The different material properties and loads are listed below.

- Concrete: C50, $E_{cm} = 37$ GPa
- $\gamma_{concrete} = 24$ kN/m^3
- Reinforcement: $E = 210$ GPa
- Office load: 2.5 kN/m^2
- Load from parking garage: 2.5 kN/m^2

5.3 Idealised calculations

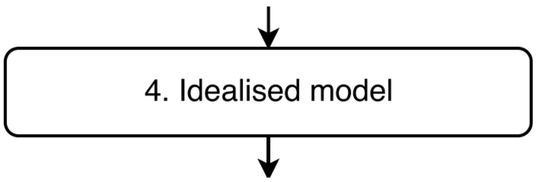


Figure 5.3: Fourth step of the methodology.

In the fourth step the building is idealised. In this case the entire building was modelled as three beam elements with different stiffnesses. The stiffness was being reduced with the height of the building, stories 0-10 have one stiffness, 11-21 have a lower stiffness and 22-34 have the lowest stiffness. In figure 5.4 the idealised model is illustrated. In the first part of the calculations the lower 10-story building is ignored and only the stiffness from the tall building is used.

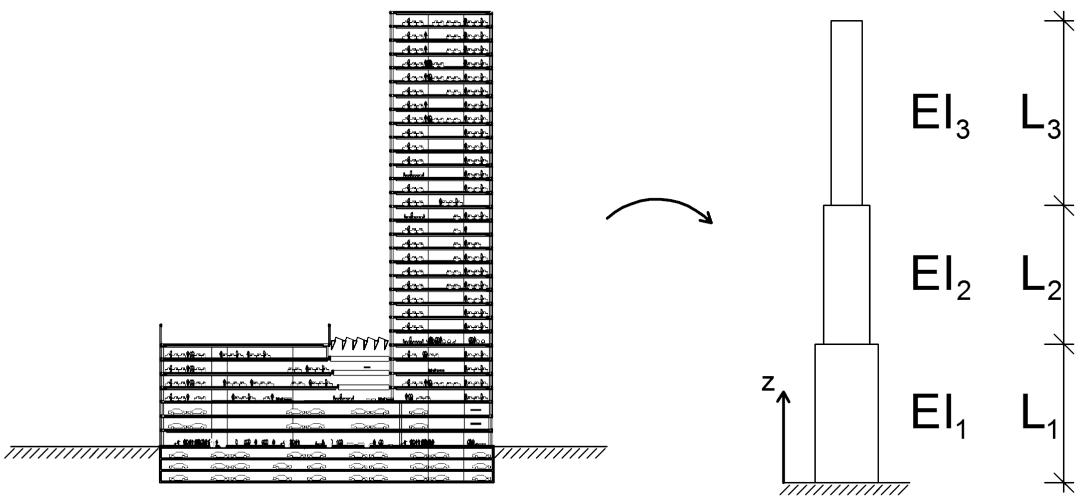


Figure 5.4: Idealised model of the building.

After the building has been idealised, step five in the methodology can be initiated. Here the mass distribution and the stiffness are calculated.

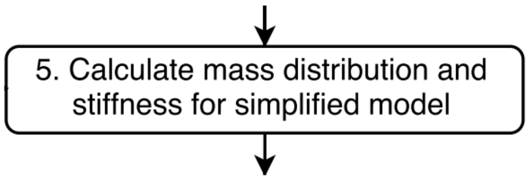


Figure 5.5: Fifth step of the methodology.

The mass distribution of the building must be known, this in order to be able to calculate the natural frequency in a later step of the process. In this case it was assumed that the mass was evenly distributed over the height of the building. This can be seen as a valid assumption since the load is mainly added from each story. Each story was only placed with a distance of 3.6 m and over 120 m this becomes a nearly evenly distributed load. With the conditions specified in section 5.2.1, the mass distribution is 200000 kg/m. The calculations are shown in Appendix C.

Furthermore, the stiffness of the three sections, the Young's modulus, E, and moment of inertia, I, must be known. Autodesk AutoCAD was used to calculate the moment of inertia of the different sections. The three different core-types are shown in figure 5.6 below. The first core is the stiffest with 550 mm thick walls, the second one has a wall thickness of 450 mm and the third has a wall thickness of 350 mm. Part of the elevator shaft ends at the 21st floor and as a result the stiffness of the top beam element is significantly lower than that of the bottom sections.

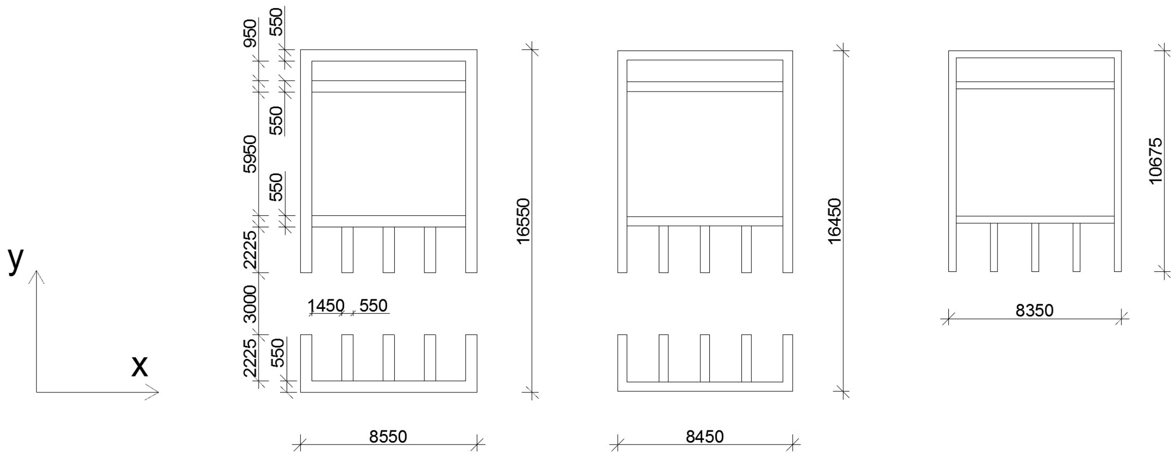


Figure 5.6: Cores of the building.

In table 5.1 the moments of inertia and the areas of concrete are shown. I_x is the moment of inertia in the stiff direction and I_y is the moment of inertia in the weak direction. The stiffness relations between the different sections are also displayed in the table. Worth noting is that the lower section is seven times stiffer than the top section in the stiff direction but only two times stiffer in the weak direction.

Table 5.1: Properties of concrete core

Core type	Floor	I_x [mm^4]	I_y [mm^4]	A [m^2]	L [m]	I_{xi}/I_{x3}	I_{yi}/I_{y3}
1	0-10	$8.77 \cdot 10^{14}$	$2.78 \cdot 10^{14}$	23.1	36	7.1	2.1
2	11-21	$7.18 \cdot 10^{14}$	$2.26 \cdot 10^{14}$	18.9	36	5.8	1.7
3	22-34	$1.24 \cdot 10^{14}$	$1.33 \cdot 10^{14}$	10.2	48	1.0	1.0

The weighted value of the Young's modulus was calculated to take the reinforcement into account. According to Eurocode a minimum reinforcement in concrete walls is $A_{min} = 0.002 \cdot A_c$ and the maximum amount of reinforcement in concrete walls is $A_{max} = 0.04 \cdot A_c$ where A_c is the area of the concrete, this means that the value of the Young's modulus can vary between two values calculated below.

$E_{min} = 0.002 \cdot 210 + 0.998 \cdot 37 = 37.3$ GPa
$E_{max} = 0.04 \cdot 210 + 0.96 \cdot 37 = 43.9$ GPa

The amount of reinforcement was assumed to be five times the minimum requirement, which meant that 1% of the total area of the cross section consisted of reinforcement. The Young's modulus became

$E_{comp} = 0.01 \cdot 210 + 0.99 \cdot 37 = 38.7$ GPa

5.3.1 2D-model and shape functions

The calculations continue to the sixth and seventh step of the methodology where the 2D model is made and the shape functions are determined.

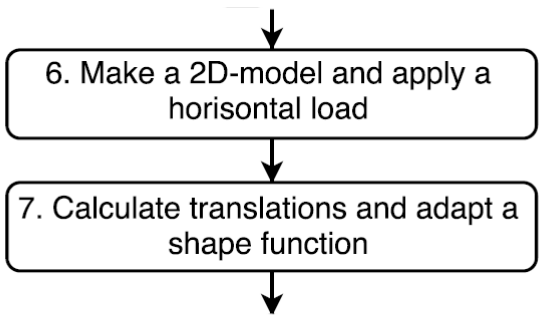


Figure 5.7: Sixth and seventh step of the methodology.

With the assumed properties shown above, a simple model of the structure was made in Strusoft Frame Analysis. Two types of loads were added to the side of the building, one uniform horizontal load of 10 kN/m and one nonuniform load. Different simulations were performed for the different loads and the loads were not acting at the same time. The reason for testing both load types was to see if there were any major differences in the results obtained. Using a uniformly distributed load provides easier calculations. The values in the nonuniformly load were calculated based on Eurocode, and extrapolations were performed were Eurocode did not give values. The deflection is calculated and exported into Microsoft Excel where a shape function is adapted to the normalised displacement. Figure 5.8 shows the model, load and the displacement of the building after the analysis in Frame Analysis was made. The displacement in the figure comes from the uniformly distributed load.

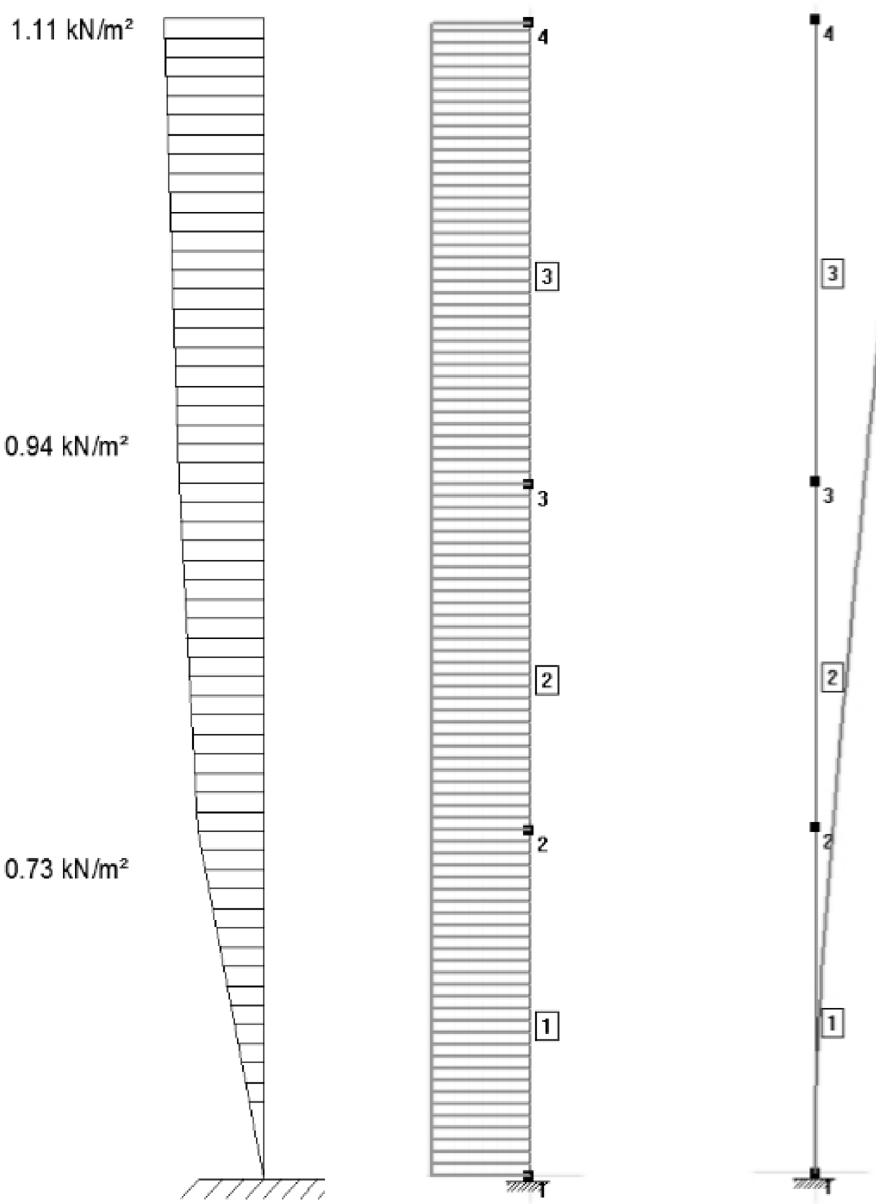


Figure 5.8: Non-uniform load, uniform load and displacement in Strusoft Frame Analysis.

The shape function proposed in Eurocode with different values of ζ, see equation 4.12, were tested to see if they fit the normalised displacement. In figure 5.9 and figure 5.10 the adapted shape functions are shown. The same analysis was performed with a non-uniform load. This result can be seen in figure 5.11 and in figure 5.12

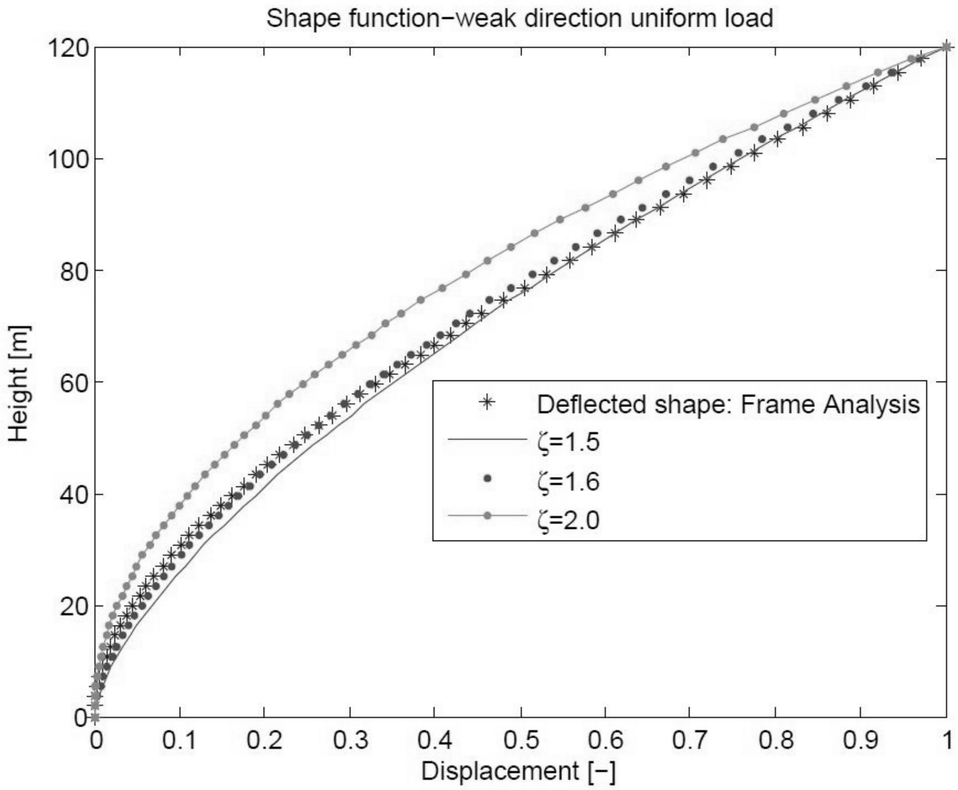


Figure 5.9: Shape function with uniform load in weak direction.

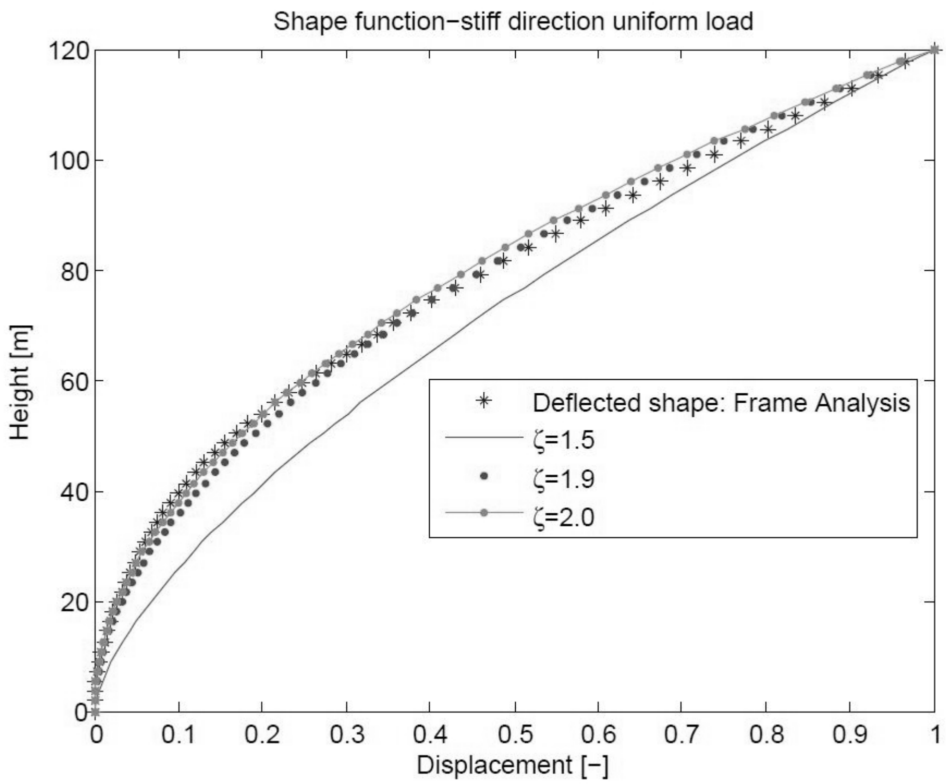


Figure 5.10: Shape function with uniform load in stiff direction.

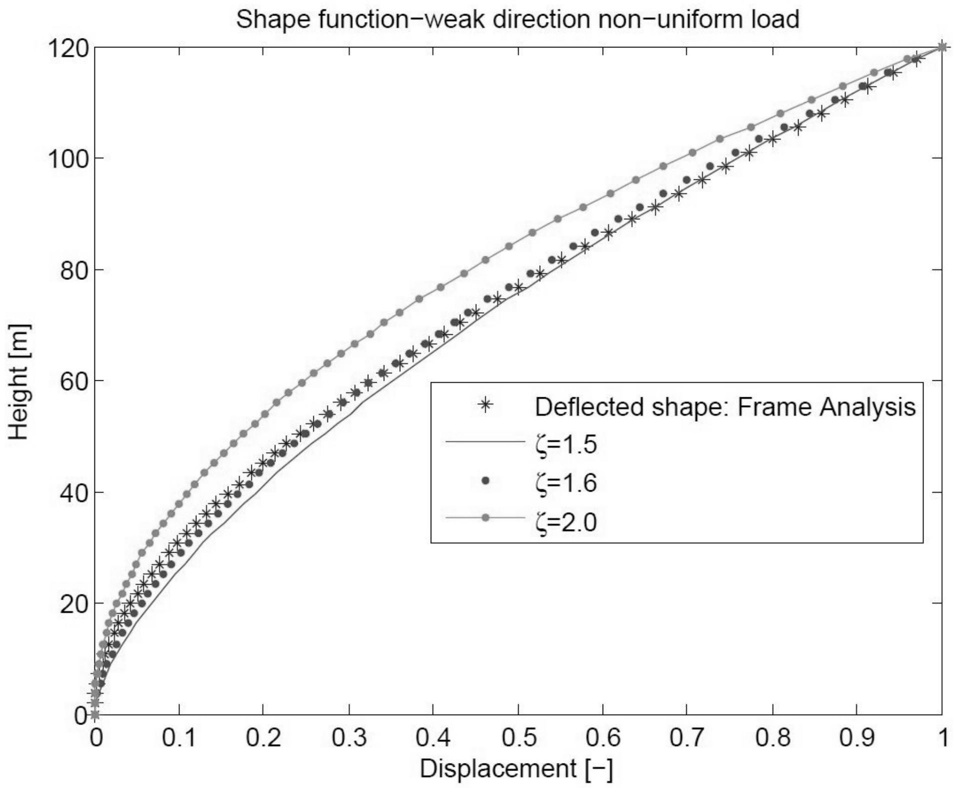


Figure 5.11: Shape function with non-uniform load in weak direction.

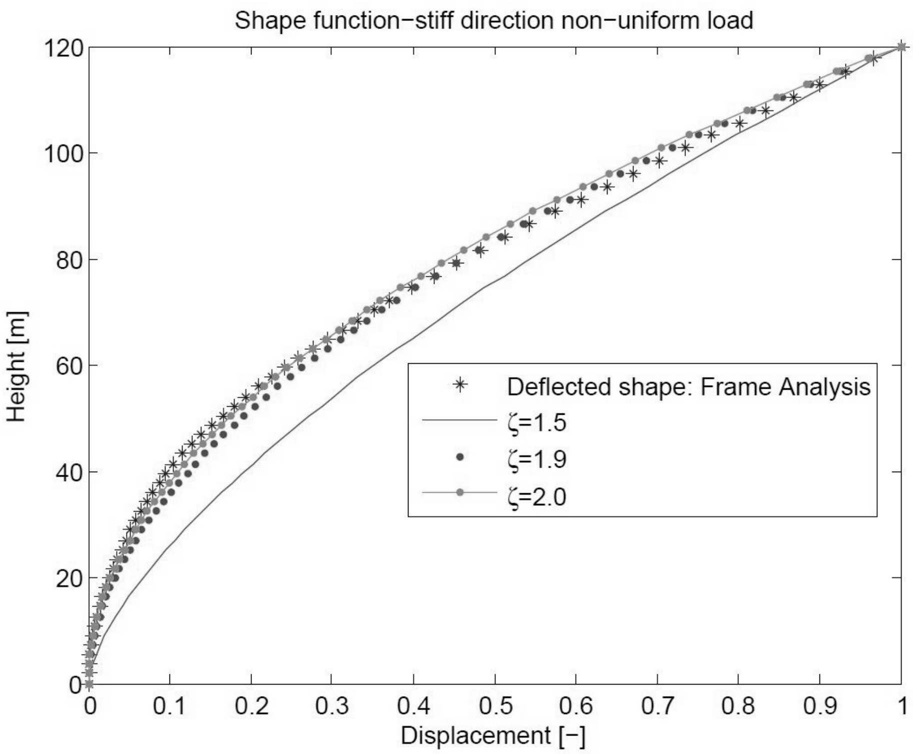


Figure 5.12: Shape function with non-uniform load in stiff direction.

In figures 5.9-5.12 it can be seen that the shape function given by Eurocode does not fit perfectly when using the recommended ζ-value of 1.5. By changing it, the curve can be shifted to match the calculated displacement. In the weak direction the value of 1.5 could be used but in the stiff direction it was better to use the higher value, 2.0. The shape of function seemed to depend on the ratio of the stiffnesses relations between the three sections in the height direction of the building. To test this, the building was modeled with different stiffnesses to see what ζ-value was the most suitable.

To evaluate how to choose the ζ-value, a parametric study was performed. The study was performed by changing the relation between the stiffness $(EI)_1$, $(EI)_2$ and $(EI)_3$ and then adapting the shape function from Eurocode by changing the ζ-value. The deflection in the stiff direction was used during the study.

In table 5.2, the results of the study are shown. The second column shows how the stiffness in the three different sections relate to one another, as shown the section closest to the ground was seven times stiffer than the top one. The remaining columns in the table shows the different tests that have been made. In the first test the stiffness of the building was kept constant in all three sections, in the second test the section closest to the ground was seven times stiffer than the two above and the tests in the remaining columns is performed in the same way. On the bottom row the best matching ζ-value is shown. The best match was determined by plotting the displacements from Strusoft Frame Analysis and comparing the values with those of the shape function.

Table 5.2: Parametric study of shape function.

	Stiffness relation, real case.	Test 1	Test 2	Test 3	Test 4	Test 5
I_1	7.1	1	7	1	7	1/7
I_2	5.8	1	1	7	7	1/7
I_3	1	1	1	1	1	1
ζ-value	2.0	1.5	2.1	1.4	1.7	1.5

It can be seen in the table that the correct value of the ζ-factor depends on the relation between the stiffness in the building. The recommended value of 1.5 can be used when the stiffness of the building is kept constant or if the highest part of the building is stiffer than the lower parts.

5.3.2 Natural frequencies

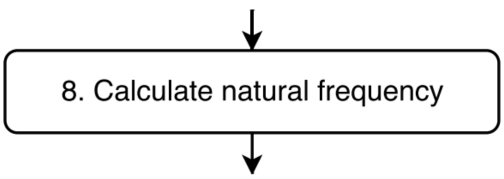


Figure 5.13: Eighth step of methodology.

In this step the fundamental frequencies in the x-direction and the y-direction of the building were calculated. The frequencies for the weak direction when the ζ-value was chosen to 1.6 are shown. The same calculations were performed several times with different values on ζ, the results are shown in table 5.3.

Using the shape function, the stiffness and the mass of the building, the fundamental frequencies can be calculated. The generalised mass and generalised stiffness of the building is calculated with equation 4.14 and equation 4.15.

The mass is assumed to be evenly distributed over the height of the building resulting in

$$\widetilde{m} = \int_0^L m(z)[\phi(z)]^2 dz \;=\; 200000 \int_0^{120} \left[\left(\frac{z}{120}\right)^{1.6}\right]^2 dz = 5.7 \cdot 10^6 \text{ kg} \tag{5.1}$$

Furthermore the stiffness of the building changes three times in the building, the generalized stiffness can therefore be calculated with

$$\widetilde{k_x} = \int_0^L EI_x[\phi''(z)]^2 dz \Rightarrow \tag{5.2}$$

$$\widetilde{k_x} = E\left(I_1 \int_0^{36} [\phi'']^2\, dz + I_2 \int_{36}^{72} [\phi'']^2\, dz + I_3 \int_{72}^{120} [\phi'']^2\, dz\right) = 21.3 \cdot 10^6 \text{ N/m} \tag{5.3}$$

The second derivative of the shape function is shown in equation 4.13.

After the generalized mass and stiffness are known, the natural frequency of the building can be calculated

$$f = \frac{1}{2\pi}\sqrt{\frac{\widetilde{k}}{\widetilde{m}}} = \frac{1}{2\pi}\sqrt{\frac{21.3 \cdot 10^6}{5.0 \cdot 10^6}} = 0.33 \text{ Hz} \tag{5.4}$$

In table 5.3 the calculated natural frequencies with different values of ζ are shown for both the weak and stiff direction of the building. The ζ-values are the same that are used in the plots in figure 5.9 and 5.10.

Table 5.3: Calculated natural frequencies in the stiff and weak direction depending on the ζ-value.

ζ	$\widetilde{m}$ [kg/m]	$\widetilde{k}$ [N/m]	f [Hz]
	Stiff direction		
1.5	$6.0 \cdot 10^6$	$85.3 \cdot 10^6$	0.60
1.9	$5.0 \cdot 10^6$	$47.6 \cdot 10^6$	0.49
2.0	$4.8 \cdot 10^6$	$47.1 \cdot 10^6$	0.50
	Weak direction		
1.5	$6.0 \cdot 10^6$	$27.6 \cdot 10^6$	0.34
1.6	$5.7 \cdot 10^6$	$21.3 \cdot 10^6$	0.31
2.0	$4.8 \cdot 10^6$	$18.2 \cdot 10^6$	0.31

Equation 4.19 is also tested to see what the results are. Since only one stiffness is used in this equation three frequencies are calculated in each direction using the three different stiffnesses.

Table 5.4: Calculated natural frequencies in the stiff and weak direction using equation 4.19.

I [m^4]	f [Hz]
Stiff direction, I_y	
877	0.54
718	0.49
124	0.20
Weak direction, I_x	
278	0.30
226	0.27
133	0.21

It can be seen that using the bottom stiffness gives a value that correlates fairly well with the values calculated using the more complicated method.

5.3.3 Accelerations

After the natural frequencies are calculated the process can continue to step nine where the accelerations are calculated.

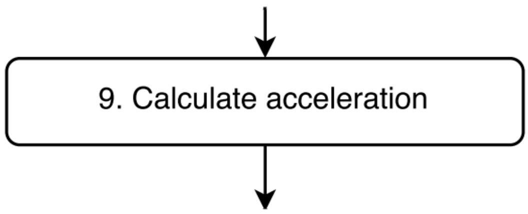


Figure 5.14: Ninth step of the methodology.

When the frequencies are calculated, predictions for the accelerations at the top of the building are calculated based on the equations from section 4.6. The calculations are performed in Microsoft Excel, the spreadsheet is shown in appendix C. In table 5.5 the accelerations for the different values of the natural frequencies are shown. a_D is the acceleration in the along-wind direction and a_w is the acceleration in the cross-wind direction. Along-wind acceleration is calculated according to both EC and NBCC, the cross-wind acceleration is calculated according to NBCC.

Table 5.5: Peak accelerations at the top of the building in the along-wind direction, a_D, and in the cross-wind, a_w, direction according to Eurocode and NBCC.

ζ	f [Hz]	a_D, EC [m/s^2]	a_D, NBCC [m/s^2]	a_w, NBCC [m/s^2]
Wind in stiff direction				
1.5	0.64	0.048	0.044	0.052
1.9	0.52	0.059	0.049	0.058
2.0	0.53	0.058	0.049	0.058
Wind in weak direction				
1.5	0.36	0.089	0.059	0.024
1.6	0.33	0.097	0.065	0.032
2.0	0.33	0.097	0.065	0.031

If the calculated accelerations are compared with the values given in figure 3.1 it can be seen that the accelerations are low compared to recommendations and would probably not cause any problems to the tenants.

5.4 Computer analysis of idealised model

In the previous sections, the natural frequencies are calculated with the help of shape functions. In this section, the cores are modeled in Strusoft FEM-Design and the natural frequencies and accelerations were again calculated and compared to the results in the previous sections.

The building is modelled with the three different cores shown in figure 5.6. Since the model should be used for calculating the natural frequencies of the building, it is important to use the total mass for the entire building and not only the mass for the core. In figure 5.15 the model in Strusoft FEM-design is shown.

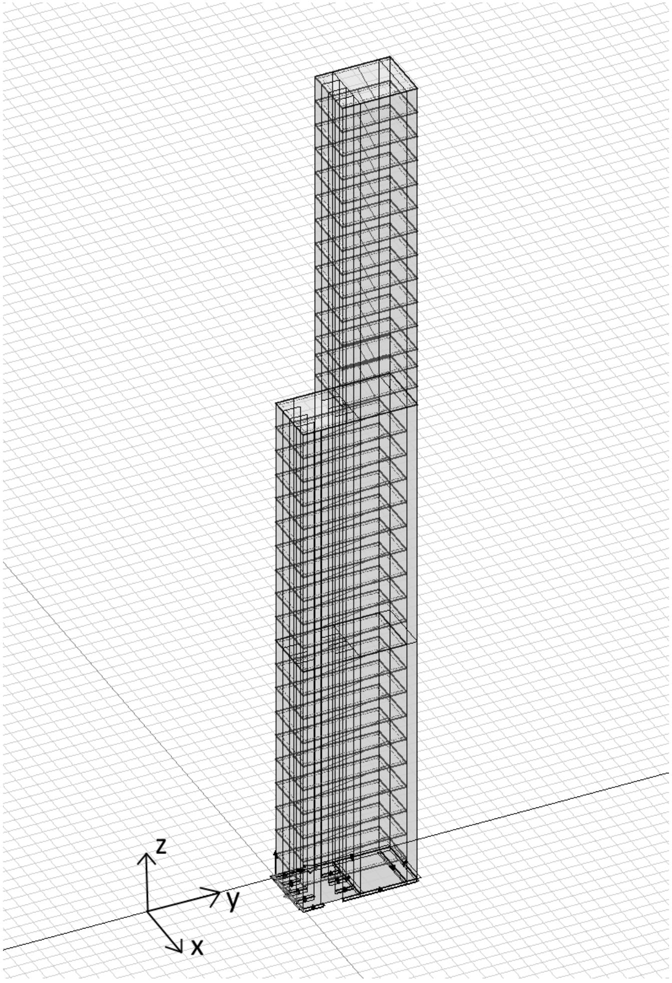


Figure 5.15: Building modeled in Strusoft FEM-design.

5.4.1 Natural frequencies and accelerations

The five first natural frequencies according to the FE-analysis in Strusoft FEM-Design are shown in table 5.6. In table 5.7, the accelerations for the first mode in the weak and stiff directions are shown. The accelerations are calculated according to section 4.6 with the natural frequencies in table 5.6. Only the accelerations for the first modes are shown since Eurocode and NBCC only provides calculation methods for these types of accelerations. In figure 5.16 the displacements of the core is shown. Comparing the values in table 5.6 and table 5.7 with those in table 5.4 and table 5.5 shows that the difference between the idealised FE-model and the idealised calculations are very small.

Table 5.6: Natural frequencies according to Strusoft FEM-design.

	f [Hz]
1. First mode in weak direction	0.35
2. First mode in stiff direction	0.56
3. Second mode in weak direction	1.54
4. Second mode in stiff direction	1.91
5. First mode in torsional direction	2.38

Table 5.7: Accelerations from Strusoft FEM-design frequencies.

	f [Hz]	a_D, EC [m/s^2]	a_D, NBCC [m/s^2]	a_w, NBCC [m/s^2]
Wind in weak direction	0.35	0.091	0.061	0.029
Wind in stiff direction	0.56	0.055	0.046	0.054

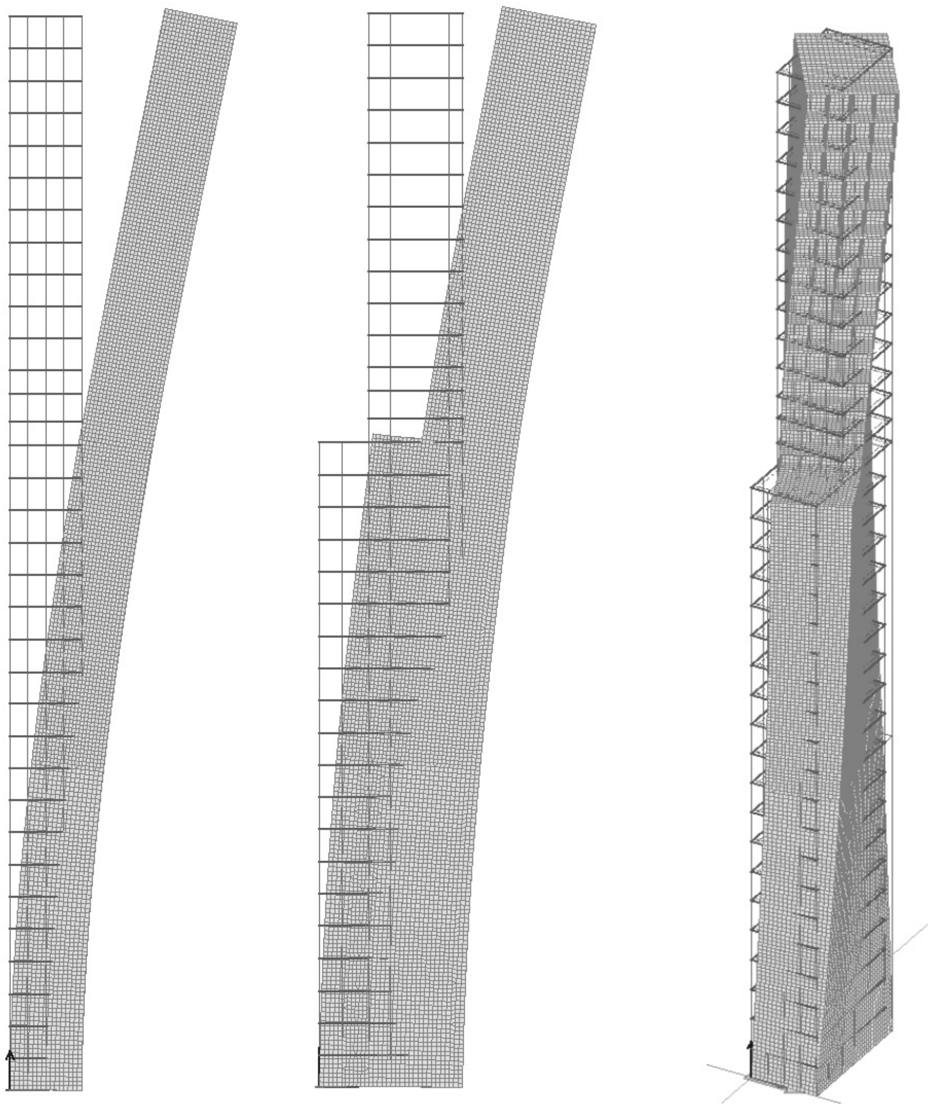


Figure 5.16: Displacements of core.

5.5 Simulations of detailed building

When the idealised analyses are done, a more detailed analyses of the building should be performed. A more accurate FE-model is made to verify natural frequencies and to be able to consider the torsional modes. In this section a model of the entire building with all the components of the structural systems is made. The natural frequencies and accelerations are evaluated.

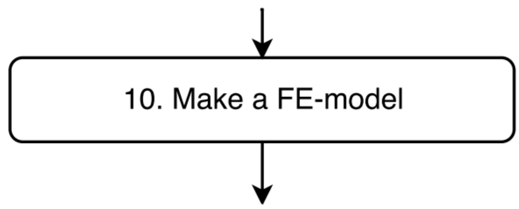


Figure 5.17: Tenth step of the methodology.

To be able to see if the idealised model of the building can be used, simulations of the entire building were made in Midas GEN. Since only architectural drawings were available, dimensions and placement of columns and beams were determined based on experience. In the list below the dimensions of the structural components are shown. All the columns are made of concrete (C50) and the beams of steel (s450), the dimensions of the core is the same as previously used, see section 5.3.

- Columns bottom section: 900x900 mm
- Columns middle section: 700x700 mm
- Columns top section: 500x500 mm
- Beams for HDF-floor: THQ-beams
- The two cores, the small one with elevators and the big one with the stair shaft, are connected with concrete beams at each floor. The width of the beams are the same as the walls.

Some assumptions were made when modelling the building to prevent overly complex models. The concrete slabs are not modelled, instead rigid diaphragms are used and a load representing the self weight of the floor and live load are added. The rigid diaphragms are fully rigid in plane but have no flexural stiffness, the effects of using rigid diaphragms are discussed in section 6.2. Furthermore, no non-structural elements are modelled.

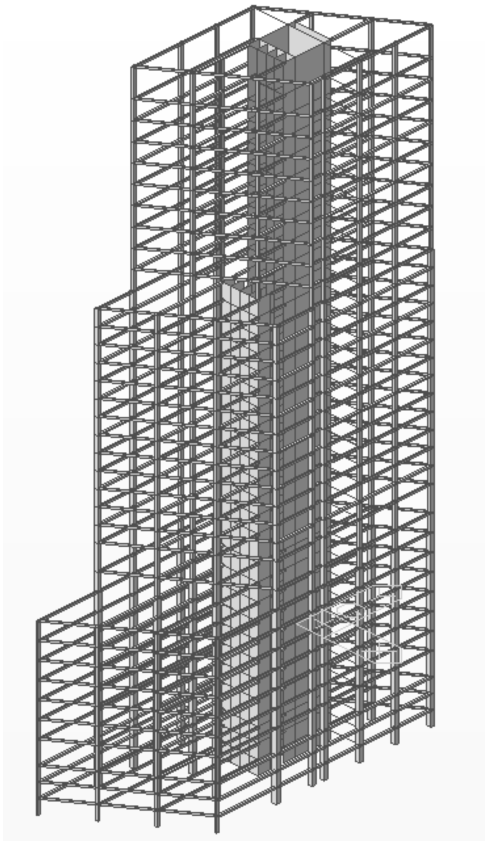


Figure 5.18: Model of the building in Midas GEN.

The natural frequencies from the simulations are shown in table 5.8 and the calculated accelerations are shown in table 5.9. The accelerations are calculated according to section 4.6 with the natural frequencies in table 5.8. Only the accelerations for the first modes are shown since Eurocode and NBCC only provides calculation methods for these types of accelerations.

Table 5.8: Natural frequencies according to Midas GEN.

	f [Hz]
1. First mode in weak direction	0.39
2. First mode in stiff direction	0.57
3. First mode in torsional direction	1.16
4. Second mode in weak direction	1.90
5. Second mode in stiff direction	2.11

Table 5.9: Accelerations from Midas GEN frequencies.

	f [Hz]	a_D, EC [m/s^2]	a_D, NBCC [m/s^2]	a_w, NBCC [m/s^2]
Wind in weak direction	0.39	0.082	0.054	0.028
Wind in stiff direction	0.57	0.054	0.040	0.047

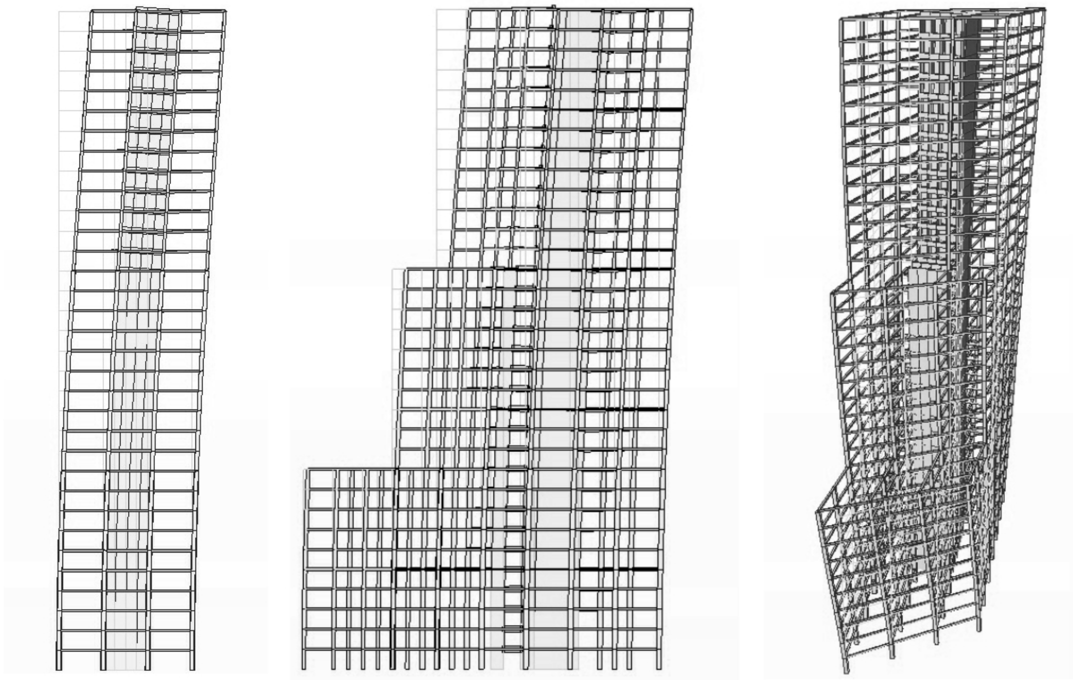


Figure 5.19: Deformation of the building.

The wind load has thus far only been based on Eurocodes standard wind load. For a high-rise building wind tunnel test should be performed, or more accurate load should be applied with the help of experience and more advanced analytical methods. This is the tenth step of the methodology in this dissertation and is not covered here. When the results are evaluated the detailed design should be made, this step is not shown in this dissertation.

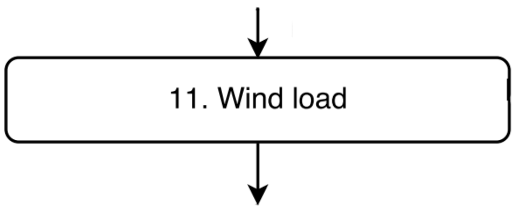


Figure 5.20: Eleventh step of the methodology.

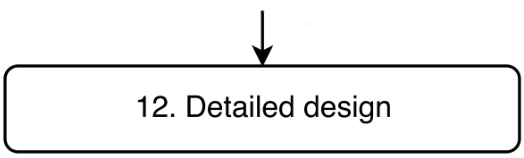


Figure 5.21: Twelfth step of the methodology.

5.6 Adjusting the stiffness of the building

If the stiffness of the building needs to be adjusted, noted ”B” in the methodology, there are several ways to do this. In the sections below some methods are tested and evaluated.

5.6.1 Connecting main building to core of lower side building

To increase the stiffness of the building, the ten-story side-building was connected to the main-building. No hand calculations were performed, only simulations in Strusoft FEM-design. Two tests were performed, one with the side building placed perpendicular to the main-building and one with the side-building placed in an angle of 57 degrees in relation to the main-building, see figure 5.22 for a clarification. The case were the angle was 57 degrees represents the real scenario and the case were the buildings were perpendicular was a simplification. In table 5.10 the results of the 57 degree simulation are shown together with percentage change. In table 5.11 the corresponding results for the case where the buildings were placed perpendicular are shown. To simulate the two cores being coupled to each other, fictitious bars with a very high stiffness were used. Test were also made by connecting the buildings with walls and large concrete beams. All three different connections gave similar results as the fictitious bar. The fictitious bars do however not add any mass to the building or generate additional natural frequencies. Figure 5.23 shows the displacement in the weak direction.

By connecting the two buildings, the stiffness can be raised and as a result the natural frequencies will be higher. The biggest change of the natural frequency was 14% for the 57 degree case and 21% for the simplified case.

Table 5.10: Natural frequencies according to Strusoft FEM-design, 57 degrees between buildings.

	No side building [Hz]	With side building [Hz]	Change [%]
First mode, weak direction	0.35	0.40	14
First mode, stiff direction	0.56	0.57	2
Second mode, weak direction	1.54	1.63	6
Second mode, stiff direction	1.91	1.92	1
First mode, torsional direction	2.38	2.62	10

Table 5.11: Natural frequencies according to Strusoft FEM-design, 90 degrees between buildings.

	No side building [Hz]	With side building [Hz]	Change [%]
First mode, weak direction	0.35	0.41	17
First mode, stiff direction	0.56	0.57	2
Second mode, weak direction	1.54	1.66	8
Second mode, stiff direction	1.91	1.93	1
First mode, torsional direction	2.38	2.87	21

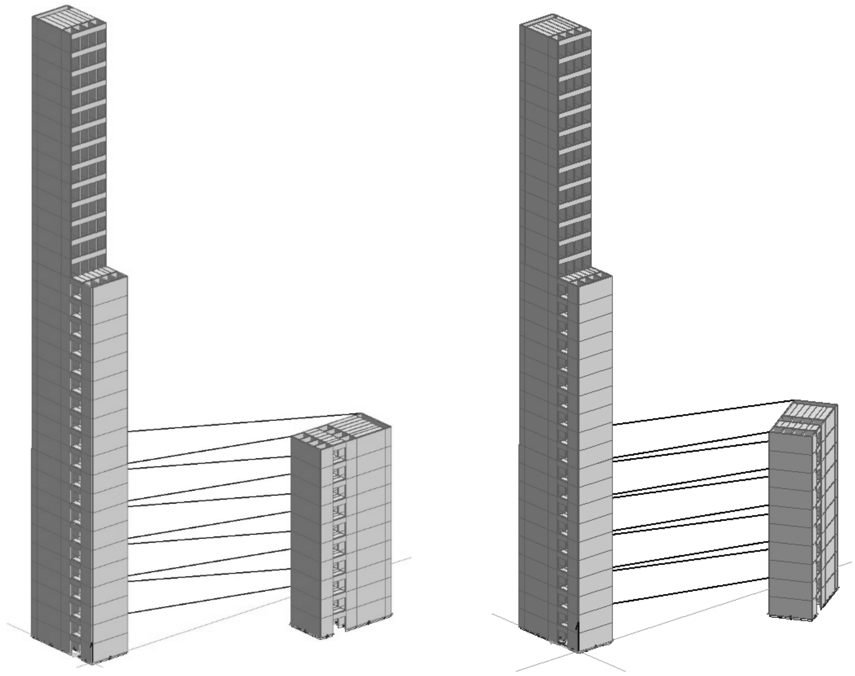


Figure 5.22: Illustration of the two cores.

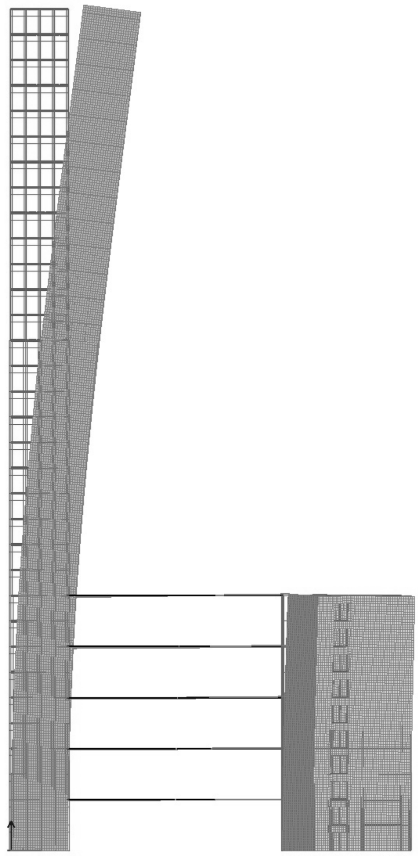


Figure 5.23: Illustration of the displacement of the coupled cores.

5.6.2 Outriggers

A common way to increase the stiffness of a building is to add an outrigger system, see section 4.8 for an explanation of what an outrigger is. The analyses concerning outriggers will be divided into a couple of different parts. Chapter 5.6.2.1 and chapter 5.6.2.2 will perform analyses on an idealised model with a prismatic core. This is to examine the equations laid out in chapter 4.8.2. Idealised models in Strusoft FEM-Design and Midas GEN are made and compared to the equations. There is no simple method of calculating natural frequencies without using FEM software for an outrigger system.

In section 5.6.2.3, analyses are run for the complete building in the case study to examine the effect that outriggers have on natural frequencies.

5.6.2.1 Idealised calculations

The equations used for calculating deflections and moments in the building are presented in section 4.8. An idealised model of the building will be made due to the restraint of the core needing to be uniform for the entire building. The building will be assumed to have a non-varying moment of inertia of 278 m^4 the entire way to the top. An outrigger will be placed on a height of 60 m. The outrigger consists of two concrete walls on each side, each wall has a thickness of 550 mm and a height of 3.6 m. A uniform distributed load, 52.5 kN/m, is applied in the weak direction of the building. The load is based on the design wind load according to Eurocode and the width of the building. The moment at the bottom is calculated by taking the moment at the bottom for the core without an outrigger and then subtracting the value applied by the outrigger. In the calculations below the moment in the outrigger and the displacement is shown.

The moment applied from the outrigger at the location of the outrigger is calculated with

$$M_{outrigger} = \frac{w(H^3 - x^3)}{6EI_{core}} \cdot \left[\left(\frac{1}{EI} + \frac{2}{EA_{column} \cdot d^2} \right) \cdot (H - x) + \left(\frac{d}{12EI_{outrigger}} \right) \right]^{-1} \tag{5.5}$$

$= 53.8$ MNm

and the moment at bottom of the building without the outrigger can be calculated with

$$M = \frac{ql^2}{2} = 378 \text{ MNm} \tag{5.6}$$

The moment in the bottom with the outrigger is

$$M_{bot} = 378 - 53.8 = 324.2 \text{ MNm} \tag{5.7}$$

The deflection at the top is calculated as the deflection of a cantilever girder minus the reduced deflection from the moment that the outrigger provides according to

$$y_{top} = \frac{wH^4}{8EI_{core}} - \frac{M(H^2 - x^2)}{2EI_{core}} = 0.099 \text{ mm} \tag{5.8}$$

5.6.2.2 Idealised models in Strusoft Frame Analysis and Midas GEN

Two models were made of the building in Strusoft Frame Analysis, these are shown in figure 5.24. The first is a more complicated model where the outer columns have been modeled to the ground and they have been connected to the core with beams representing the floors. The outer columns have a total dimension of 3.6 m^2 and the beams representing the floor are determined to have a moment of inertia of $0.02 m^4$. All connections, except for those to the outrigger and supports, are pinned.

The second model has springs to represent the rest of the structure under the outrigger, this provides a great simplification in performing the modelling. The stiffness of the spring is calculated based on the dimensions of the outer columns with the formula $k = \frac{EA}{L}$. This gives a spring stiffness of $2.3 \cdot 10^9$ N/m.

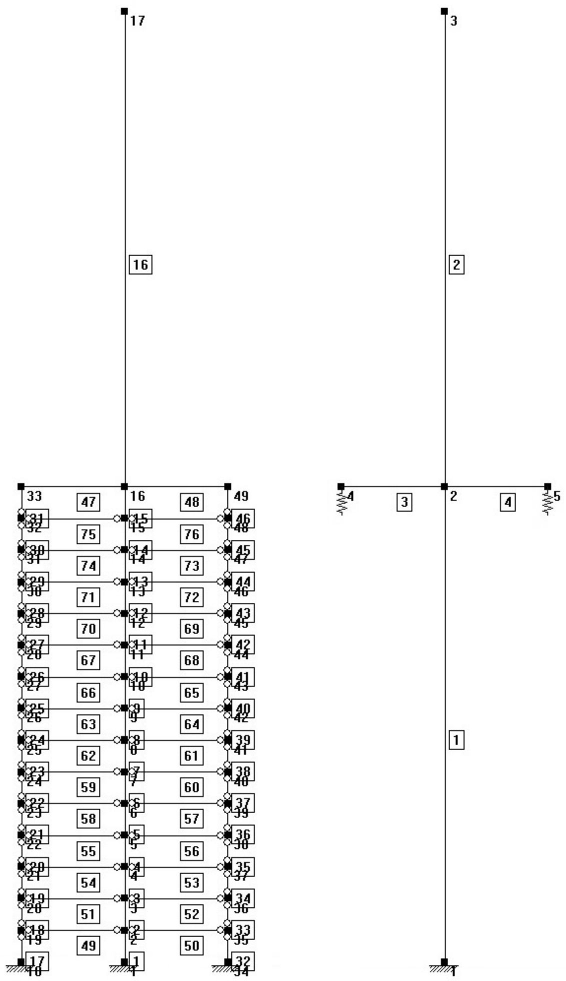


Figure 5.24: Illustration of the two Strusoft Frame Analysis models, model 1 on the left and model 2 on the right.

Results from the two models are shown in table 5.12, it can be seen that the two models give very similar results. The results are also very close to those calculated with idealised calculation in chapter 5.6.2.1.

Three models were made in Midas GEN, one idealised with only the core and columns and two full Midas GEN models. The connections in the model were pinned. The difference between the two full Midas GEN models was the belt wall, in the first model it consisted of a truss made up of hollow steel girders, 35x35x8 mm, and the second model had a belt wall made out of a 550 mm thick concrete wall. The three models are shown in figure 5.25.

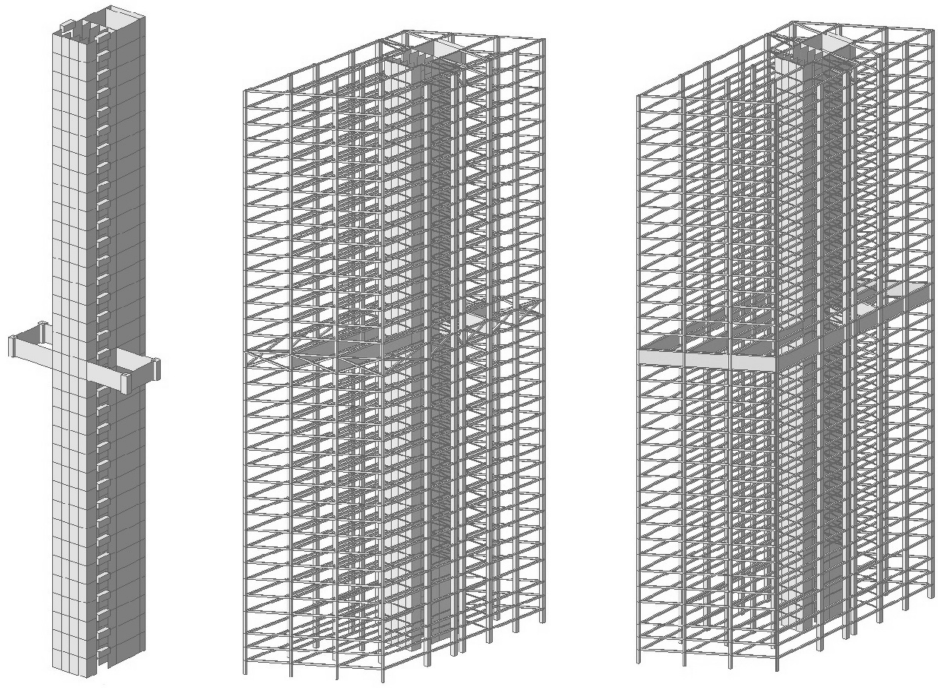


Figure 5.25: Illustration of the three Midas GEN models.

All Midas GEN models were considerably stiffer and resulted in smaller deflections than the Strusoft Frame Analysis models. Having a concrete wall as a belt wall also had noticeable effect on the buildings deflections.

Table 5.12: Results from Strusoft Frame Analysis for outrigger models, M_{out} is the moment applied by the outrigger to the core.

	y_{top} [m]	M_{out} [MNm]	M_{bot} [MNm]
No outrigger	0.126	-	-
Idealised calculations	0.099	53.8	324.2
Strusoft Frame Analysis model 1	0.093	54.5	310.7
Strusoft Frame Analysis model 2	0.098	57.0	321.0
Idealised Midas GEN model	0.049	-	-
Full Midas GEN model, truss belt wall	0.065	-	-
Full Midas GEN model, concrete belt wall	0.057	-	-

5.6.2.3 Natural frequencies in Midas GEN

In the Midas GEN model, the outrigger system was placed on different floors to see the how the placement of the outrigger system would affect the natural frequencies of the building. The outrigger system consisted of concrete walls that connected the core with the beam and column system around the perimeter of the building. A truss system of diagonal braces around the building made up a belt wall, see figure 5.27.

The first simulation was made with an outrigger system placed in the middle of the building, see figure 5.26. An analysis of the natural frequencies was made, the results are shown in table 5.13.

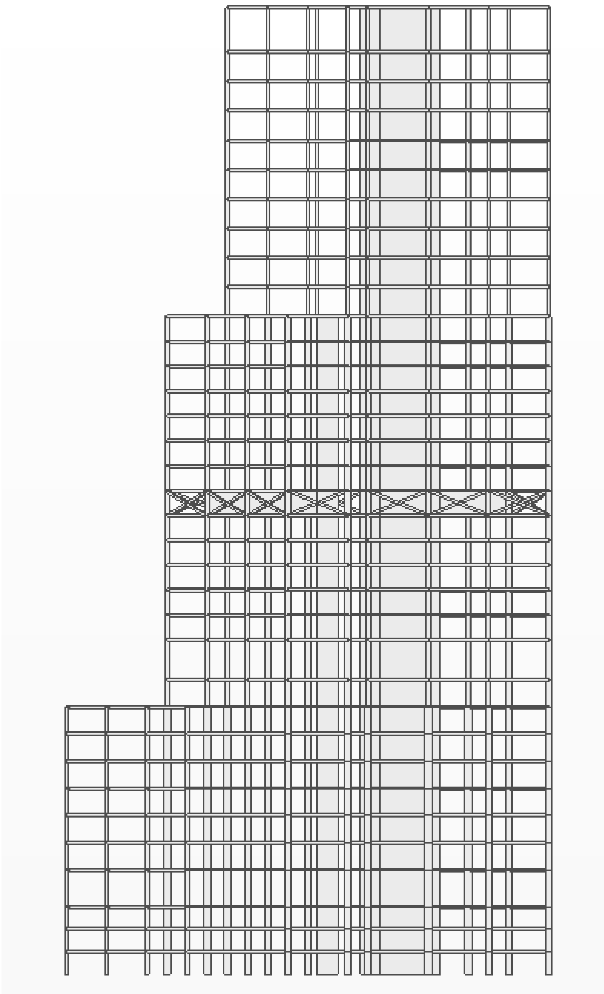


Figure 5.26: Illustration of building with outrigger.

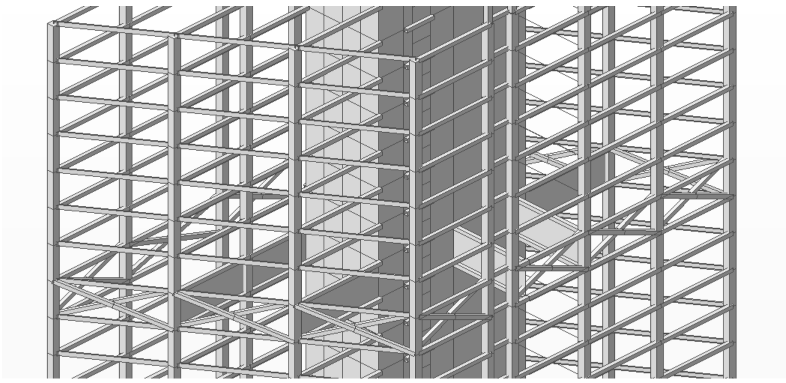


Figure 5.27: Illustration of outrigger.

Table 5.13: Natural frequencies with outrigger system placed on the middle floor.

	Without outrigger [Hz]	With outrigger [Hz]	Change [%]
First mode, weak direction	0.39	0.51	31
First mode, stiff direction	0.57	0.70	23
First mode, torsional direction	1.16	1.15	-1
Second mode, weak direction	1.90	1.88	-1
Second mode, stiff direction	2.11	2.08	-1

In the results it can be seen that the natural frequencies increase for the first modes in the weak and stiff direction, however for the torsional direction and for the second modes the natural frequencies are basically unchanged. An additional simulation is made where two outriggers are used. The outrigger systems are placed where the stiffness of the core is changed, see figure 5.28. In table 5.14 the results are shown.

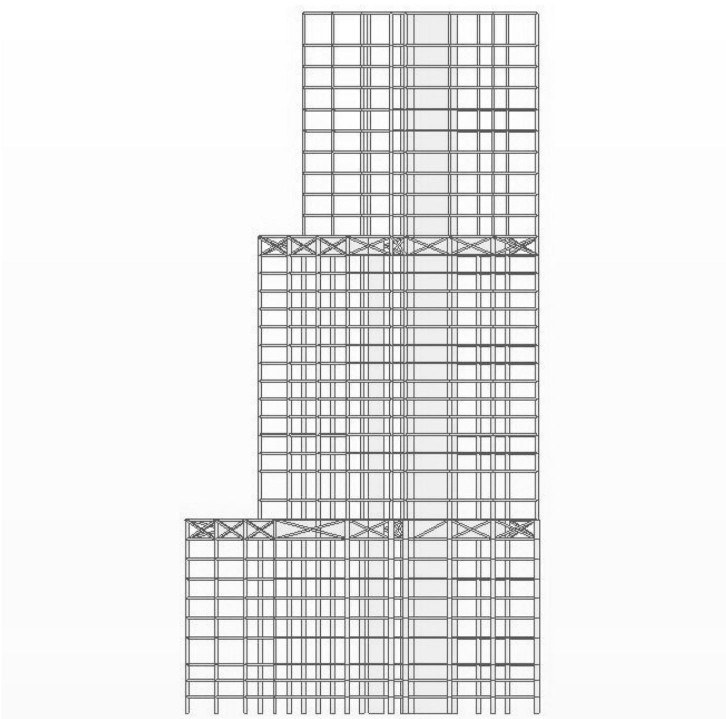


Figure 5.28: Outrigger placed on two floors.

Table 5.14: Natural frequencies with outrigger system placed on two floors.

	Without outrigger [Hz]	With outrigger [Hz]	Change [%]
First mode, weak direction	0.39	0.58	49
First mode, stiff direction	0.57	0.82	44
First mode, torsional direction	1.16	1.16	0
Second mode, weak direction	1.90	2.16	14
Second mode, stiff direction	2.11	2.46	17

5.6.3 Increasing core stiffness

Increasing the dimensions of the existing structural system will clearly increase the stiffness, but might not always be the best option, especially if a lot of stiffness needs to be added to the structure. One way to increase the overall stiffness of the building is to make the entire core stiffer. This can be done either by changing the thickness or Young's modulus of the walls. Young's modulus can be changed either by increasing the amount of reinforcement or by changing the quality of the concrete. A study has been made to see how the properties of the core affects the natural frequencies of the building. The first study examines the change of the natural frequencies due to the change of wall thickness and the second study examines the change of natural frequencies due to the change of Young's modulus. The simulations were performed in Midas GEN on the idealised model of the building, this means that only the core is analysed.

The thickness of the walls were changed in two steps. The first simulation was made with the original thickness of the walls (550 mm, 450 mm and 350 mm) with the thickest walls on the lowest section, see figure 5.6. In the next step, the thickness of the walls were increased by 20%, this means that the corresponding walls obtain the thickness 660 mm, 540 mm and 420 mm. In the last step, the thickness of the walls were increased by 40%, this means that the walls obtains the thickness 770 mm, 630 mm and 490 mm. The results of the study are shown in table 5.15.

Table 5.15: How the natural frequencies change when the wall thicknesses are varied.

	Original size [Hz]	20% increase [Hz]	Change [%]	40% increase [Hz]	Change [%]
First mode, weak direction	0.38	0.38	2	0.39	4
First mode, stiff direction	0.60	0.61	1	0.62	2
First mode, torsional direction	1.67	1.70	2	1.73	3
Second mode, weak direction	2.06	2.08	1	2.09	1
Second mode, stiff direction	2.47	2.47	0	2.48	0

As can be seen in the table it is not very efficient to increase the wall thickness of the core to obtain a higher stiffness of the building. A 40% increase of the wall thickness only resulted in a 4% change of the natural frequency in the first mode.

The study of how the Young's modulus affects the stiffness was performed in a similar way as for the wall thickness. The Young's modulus was increased in three steps, each step involved a change of 10%. The starting value of the Young's modulus was 35.2 GPa, this represents the value of concrete in strength class C40/50. The different steps are shown below and the results of the study are shown in table 5.16.

- Step 1: 35.2 GPa
- Step 2: 38.7 GPa (10% increase)
- Step 3: 42.2 GPa (20% increase)
- Step 4: 45.8 GPa (30% increase)

Table 5.16: How the natural frequencies changes when Young's modulus is varied.

	Original size [Hz]	10% increase [Hz]	Change [%]	20% increase [Hz]	Change [%]	30% increase [Hz]	Change [%]
First mode, weak direction	0.38	0.40	5	0.41	10	0.43	14
First mode, stiff direction	0.60	0.64	5	0.66	10	0.69	14
Second mode, weak direction	1.67	1.76	5	1.83	10	1.91	14
Second mode, stiff direction	2.06	2.17	5	2.26	10	2.35	14
First mode, torsional direction	2.47	2.60	5	2.70	10	2.81	14

In the results it can be seen that the change of Young's modulus results in a linear change of the natural frequency. To increase the modulus of elasticity could be a useful way to change the overall stiffness of the building if a small change is required. The Young's modulus can be varied by either changing the amount of reinforcement or by changing the concrete strength class. If the amount of reinforcement should be changed it cannot exceed the requirement of maximum amount of reinforcement according to Eurocode. To change the amount of reinforcement or change the concrete strength class could be very expensive due to the amount needed to cast the entire core.

A change in the Young's modulus will only change the stiffness of the building, this will change the value of $\widetilde{k}$ in equation 5.9. By increasing the Young's modulus by 10%, $\widetilde{k}$ changes by $\sqrt{1.1} = 1.05 \Rightarrow 5\%$. The same applies to the 20% and the 30% increase.

$$f = \frac{1}{2\pi}\sqrt{\frac{\widetilde{k}}{\widetilde{m}}} \tag{5.9}$$

Chapter 6

Reliability of calculations

When analyses are performed the results can vary drastically depending on different assumptions and analyses methods. To be able to establish if results obtained from calculations, FE-simulation and wind tunnel tests are correct, there is a need to compare the results with full scale measurements. This chapter looks at studies that have compared results from analyses and measurements of full scale buildings.

6.1 Performing measurements on finished buildings

There are two standard methods to perform dynamic testing for buildings, these are the forced vibration test (FVT) and the ambient vibration test (AVT). The forced vibration test relies on adding vibrations to the structure with vibration generators. This is not a common method today due to the cost of the machines required and the need to shut the building down during the time of the measurements. The ambient vibration test works by measuring the response of the building to wind excitations. This is a cheaper method that is easier to perform and is therefore the most common one [6].

6.2 FE-Models

Making FE-models is today standard when designing any large structure. Making a complete and accurate model is complicated, time consuming and requires powerful computers. To make the process shorter, some simplifications are normally done that affect the results of the model.

Table 6.1 has a summary of results from studies that have looked at FE-models and measured values of buildings. It can be seen that the steel buildings have natural frequencies that correlate well with the values obtained from the FE-models. However, for the reinforced concrete the difference can be up-to 70%.

Table 6.1: Summary of FE-models and real measurements [42].

Material and type of structure	Height [m]	Mode	Natural frequency Measured [Hz]	FEM [Hz]	Difference [%]	Included
RC Core and concretre filled tube column [6]	280	1st	0.19	0.19	0.0	Bare frame, core wall opening and thin wall in core
		2nd	0.20	0.22	8.0	
		3rd	0.57	0.61	7.0	
Steel tube [40]	-	1st	8.50	8.00	-5.9	Bare frame
		2nd	8.30	7.90	-4.8	
		3rd	4.40	4.70	6.8	
Steel frame [49]	63.3	1st	0.76	0.76	0.0	Bare frame and composite beams
		2nd	0.87	0.86	-2.2	
		3rd	1.15	1.11	-3.5	
Steel frame [66]	108	1st	0.57	0.57	0.0	Bare frame
		2nd	2.18	2.15	-1.4	
		3rd	4.58	4.48	-2.2	
RC frame and core walls [47]	200	1st	0.37	0.30	-18.5	-
		2nd	1.40	1.11	-20.5	
		3rd	2.98	2.45	-17.7	
RC shear wall [31]	218	1st	0.31	0.18	-41.9	Bare frame
		2nd	0.31	0.18	-41.9	
		3rd	0.53	0.16	-69.8	
RC shear wall [8]	206	1st	0.36	0.18	-50.0	-
		2nd	0.37	0.19	-49.0	
		3rd	0.58	0.15	-74.0	
RC frame and core walls [8]	51.3	1st	1.67	0.82	-50.8	-
		2nd	1.75	0.63	-64.2	
		3rd	2.38	1.03	-56.7	
RC frame and core walls [61]	52.8	1st	1.79	0.53	-70.2	Bare frame
		2nd	1.72	0.66	-61.6	
		3rd	2.63	0.62	-76.4	
RC shear wall [61]	112.7	1st	0.78	0.38	-51.0	Bare frame
		2nd	0.72	0.36	-49.8	
		3rd	0.65	0.38	-41.2	
RC frame and core walls [41]	148	1st	0.35	0.23	-36.2	Bare frame
		2nd	0.38	0.28	-26.0	
		3rd	0.74	0.46	-37.8	
Steel frame and RC structure [47]	420.5	1st	0.15	0.16	10.2	-
		2nd	0.38	0.45	-17.1	
		3rd	0.58	0.66	-15.3	

FE-models can often contain a lot of simplifications on the building, this in order be able to create the model faster and make a model that is not to complicated. These simplifications can however have an impact of the results of the analysis. Below are some factors that are often simplified, but can have an impact on FE-models [42].

6.2.1 Non-structural components

The stiffness of high-rise buildings comes from the main structural bearing system. This includes the core, shear walls, moment resisting frames etc. When making FE-models, these systems are modelled and non-load bearing elements like partition walls, external walls, stairs etc. are ignored. These systems do however contribute to the overall stiffness of the building. The size of the difference is in large part due to what the non-structural components consists of [61].

6.2.2 Flexural stiffness of floor slabs

Diaphragm floors are used to model the floors in some commercial FE-programs. The diaphragm floors are completely rigid in-plane and have no flexural stiffness unlike plate elements. This will result in a significantly shorter analysis time than if the floor was to be modelled using plate elements. The diaphragm floors will completely ignore any stiffness between the outer columns and the inner core from the flexural stiffness of the floor, which will reduce the stiffness of the building [46]. This assumption can be valid for moment frames, however, when shear walls are used it is not as valid. The more floors a building has, the larger the error will become from using floor diaphragms while having shear walls [45]. The error will be in the range of 5-15% [42].

6.2.3 Beam-end-offset

In FE-models, connections are modelled in nodal points but in reality the connection take up a space. To compensate for this an offset to the connection can be used. In studies this effect has been shown to change the natural frequency by up to 5% [42].

6.2.4 Young's modulus for concrete

Concrete is most often stronger and stiffer than what is expected. Using a stiffness that is closer to the actual concrete will make the building stiffer and increase the natural frequency [42]. The Young's modulus of the reinforcement bars is also considerably higher than that of concrete, if more reinforcement is used than what was expected the material will be stiffer. Cracks can also form in the concrete which will reduce the stiffness of the structural elements.

6.2.5 Wall openings

Most shear walls in high-rise buildings contain holes for doors, corridors etc. How openings are modelled have a large affect on the results of the analysis.

Brownjohn et. al [6] made FE-models of the Republic Plaza building in Singapore and compared the results with measured values. The building is 280 m tall and has a concrete core and a steel frame. The core has a large opening on one side from floor 37 and up. There are two outriggers in the building located at floor 28 and 47. A FE-model with only vertical beam elements similar to the model in figure 5.8 gave good results for the fundamental frequencies in the translational directions. The model also contained the torsional stiffness which gave torsional frequencies. These were overestimated, probably due to the fact that wall openings were not included. A FE-model with the entire core modelled but without any openings gave reasonable values for the translational natural frequencies however gave almost twice as high values for the torsional frequencies. When the major opening of the core from floor 37 and up was modelled, the lateral frequencies reduced but not significantly. The torsional frequencies were also reduced but they were still overestimated by about 50%. When minor openings for doors etc. were modelled the torsional frequencies were reduced significantly, they were instead underestimated by up to 20%. Adding the steel frame and outriggers to the building gave good results for all values.

The conclusion that was made was that the openings of the walls have a significant affect on the torsional rigidity of the building however not so much on the translational rigidity [6].

6.3 Wind tunnel reliability

To validate the results of wind tunnel tests, researchers have made measurements on full scale buildings and compared it with the results from the wind tunnel tests. Studies carried out by Dalgliesh [11] in the 80s showed that the accuracy of mean pressure was satisfactory, but there was a big difference when looking at pressure cause by vortex shedding.

Low rise buildings show a significantly larger discrepancy between measured value in wind tunnels and measured values in finished buildings than high-rise buildings [17].

Studies comparing acceleration in completed buildings with those obtained from wind tunnel tests show good accuracy. The wind tunnels will in general overestimate the accelerations.

6.4 Accelerations according to NBCC and Eurocode

The acceleration distribution for a tall building is said to follow a Gaussian distribution. According to measurement shown in figure 6.1 and figure 6.2, this fits well for most of the curve, however the extreme accelerations happen more frequently than expected.

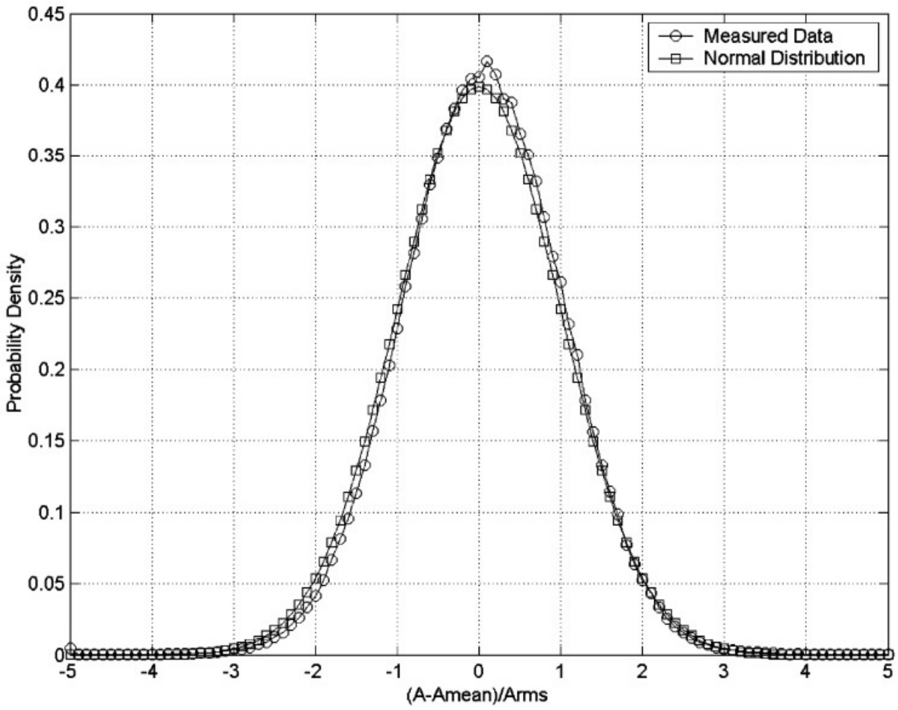


Figure 6.1: Measured acceleration during 3-hours compared to a gaussian distribution [57].

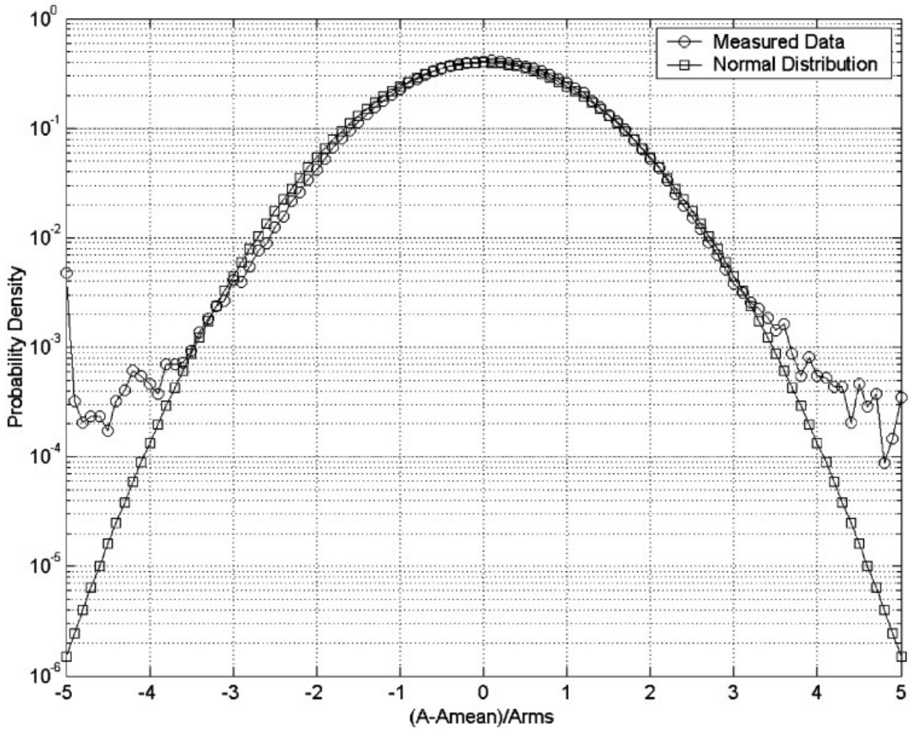


Figure 6.2: Measured acceleration during 3-hours compared to a gaussian distribution with a logarithmic probability axis [57].

6.4.1 NBCC

In figure 6.3, the results of a study [22] looking at accelerations in 48 buildings are shown. The accelerations were calculated according to NBCC and compared to accelerations obtained from wind tunnels. It can be seen that the calculated values are generally higher than those measured. Overestimating the accelerations is to be preferred rather than underestimating the accelerations. However, having a large spread will cause uncertainties if the calculated values are actually overestimated. The spread is partly due to simplifications in the theoretical models that have been adopted into NBCC. The simplifications include approximating the cross sectional area of the building with the width and the depth of the building, only including the correlation length of the wind by approximations in the size reduction factor S and omitting aerodynamic damping. The same study presented a calculation method able to get more accurate results. When comparing different types of buildings it was found that the equations in NBCC are most accurate for tall buildings, this is due to the assumption in the calculations that the resonant part of the response dominates. Lower buildings are more dependent on the background response which is mainly connected to the details of the specific site. For tall buildings, the resonant part of the response dominates and the natural frequencies of the building play a larger part than the site details. This is however in general of minor importance, since accelerations rarely present problem for low buildings [22].

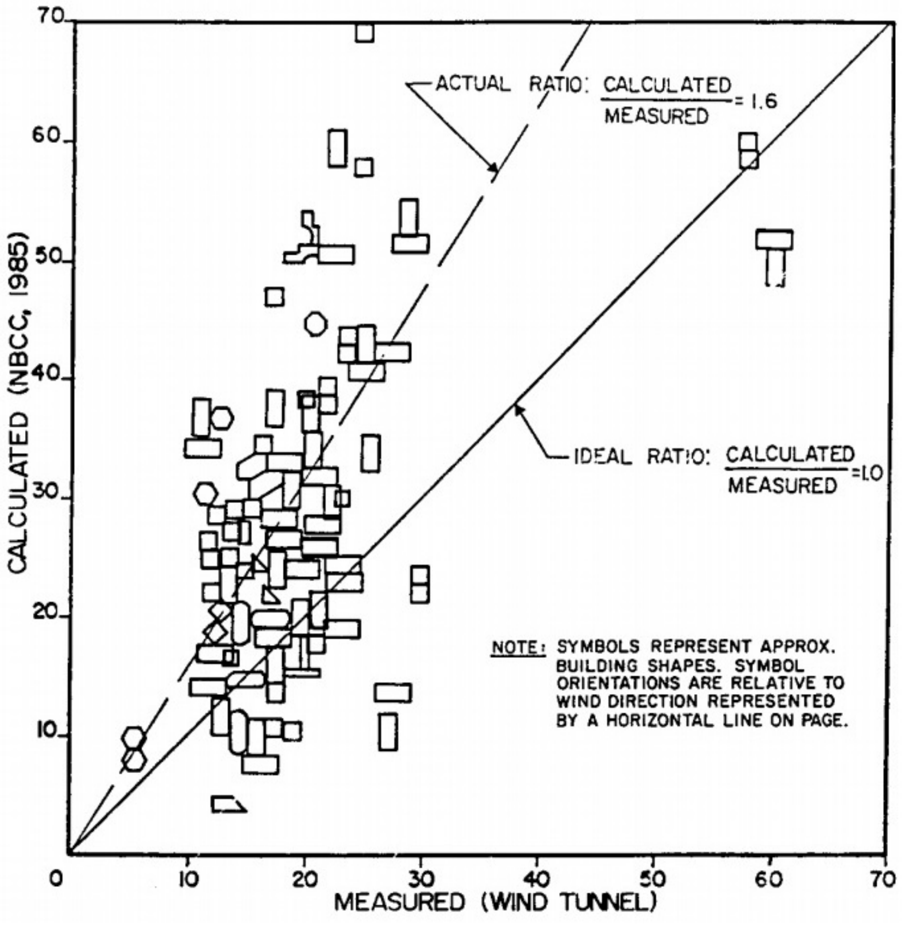


Figure 6.3: Calculated accelerations according to NBCC compared to results from wind tunnel tests [22].

6.4.2 Eurocode

No comparisons between results from Eurocode's calculation method and measured results on wind tunnel models, or real buildings, have been found by the authors. The Swedish national annex also has a different method of calculating accelerations than Eurocode does, the reliability of this method is unknown. However, a comparative study between major wind codes came to the conclusion that the along-wind response is fairly consistent between different standards, the cross-wind response was however more scattered. This should imply that Eurocode's calculation method will also generally overestimate the acceleration [43].

Chapter 7

Conclusion

In this chapter the results from the dissertation are summarized and discussed. Some suggestions on further research are also presented.

7.1 Summary of results

- The methodology presented in the dissertation shows a very simplified method that can be used in the preliminary design process.
- To make an idealised beam model of the building with varying stiffness will give sufficient results for the preliminary design.
- The shape function given in Eurocode can be used with good results if the correct ζ-value is chosen.
- The correct value of ζ-factor depends on the stiffness ratio in the building, and not just on the structural system of the building which Eurocode states.
- To calculate deflection shapes, there is a negligible difference between using a uniformly distributed load and a non-uniform load according to Eurocode.
- The idealised calculations of the natural frequencies give results that correspond well to the more advanced FE-analysis.
- The idealised equation 4.19 can be used to estimate the natural frequency of the building.
- Eurocode does not include information about comfort requirements and accelerations, if this is necessary other building codes can be used.
- Calculating accelerations can give varying results depending on what standard is used.
- To connect the main building to a lower side-building will make a noticeable difference on the stiffness, however the placement of the side-building will affect the stiffness. In this case the side-building changed the stiffness 21% if it was placed perpendicular to the main building.

- Idealised calculations on outriggers will underestimate the stiffness of the building. There is no simple way to calculate the natural frequencies of an outrigger system without a FE-analysis.
- By adding an outrigger the fundamental frequency of the specific building studied here can be increased by 31%, by adding two outriggers it is increased by 49%.
- Increasing the thickness of the walls in the core or changing the Young's modulus is not an effective way to reduce the accelerations or increase the natural frequencies of the building.
- FE-models of buildings can give results that differ a lot from measured results.
- Calculations will generally overestimate the accelerations in a building.

In table 7.1, and regarding the specific building studied in chapter 5, a comparison between natural frequencies of the idealised model and the complete model is shown. In table 7.2 the accelerations in the top of the building are summarised, both for the idealised model and the completed model. In table 7.3 the difference between the accelerations on the idealised model and completed model are shown.

Table 7.1: Comparison between idealised and completed model.

	Idealised model [Hz]	Complete model [Hz]	Change [%]
First mode, weak direction	0.35	0.39	11
First mode, stiff direction	0.56	0.57	2
Second mode, weak direction	1.54	1.90	23
Second mode, stiff direction	1.91	2.11	11
First mode, torsional direction	2.38	1.16	105

Table 7.2: Peak accelerations in the top of the building in the along-wind direction and in the cross-wind direction according to Eurocode and NBCC.

	f [Hz]	a_D, EC [m/s^2]	a_D, NBCC [m/s^2]	a_w, NBCC [m/s^2]
Idealised model				
Wind in weak direction	0.35	0.091	0.061	0.029
Wind in stiff direction	0.56	0.055	0.046	0.054
Completed model				
Wind in weak direction	0.39	0.082	0.054	0.028
Wind in stiff direction	0.57	0.054	0.040	0.047

Table 7.3: Difference of the acceleration between the idealised model and the completed model.

	a_D, EC [%]	a_D, NBCC [%]	a_w, NBCC [%]
Wind in weak direction	11	13	4
Wind in stiff direction	2	15	15

7.2 Discussion

The methodology presented in the beginning of chapter 4 is developed with help of the analyses performed and with help of engineers from Skanska. The methodology is specifically focused on the dynamic wind loads on the building. Seismic design is not included since it is not a requirement when designing buildings in Sweden. It is important to note that the results from the analyses performed in this dissertation are based on the Gothenburg City Gate project, which has a concrete core structural system. If another building is to be designed and a different structural system is used, the results may differ.

In the beginning of the calculations there are some assumptions made that can affect the upcoming calculations. The first assumption is that the mass is evenly distributed along the height of the building. If the size of the building is constant along the height, this is a reasonable assumption. In this case the shape of the building is changed in the upper third of the building which will affect the mass distribution. If the mass distribution is changed it will have an effect on the natural frequencies and the accelerations. Furthermore, an assumption is made that only 20% of the live load is used in the mass-calculations. How much of the live load that should be included is not obvious and depends on what kind of activities the building is used for.

A high-rise building can be idealised with beam elements to obtain reasonable results for the preliminary design. This idealisation will underestimate the natural frequencies, and hence, overestimate the accelerations. For a preliminary design this is desirable.

When calculating the natural frequencies the shape function of the building must be known. It turns out that the shape function provided by Eurocode is a good fit for the deflected form of a building. The value of ζ does however vary depending on the stiffness distribution along the height of the building. The recommended value of 1.5 can be used when the stiffness of the building is kept constant or if the highest part of the building is stiffer than the lower parts. To build a high-rise building that has a constant stiffness or is stiffer at the top is not a realistic scenario. If the stiffness should be kept constant the core of the building must have the same dimension at the top as it has in the bottom, this will create large vertical loads due to the amount of concrete used and it will also increase the cost of the building. This means that the value of ζ must be chosen depending on how the stiffness of the building changes and not just on the structural system which Eurocode states. Using a uniform or nonuniform load will not lead to a significant change in the deflection shape of a building. For this reason it is easier to use uniform loads for preliminary design. The reason for this is that the wind load increases fast close the ground and will quickly become similar to a uniform load. Adding additional wind load at the bottom of the building will also not have a significant affect on the building due to the small leverage arm to the base of the building.

In table 7.1, it can be seen that the difference between the natural frequencies for the idealised model and the completed model are not very large. This is only true for the translational modes and not for the torsional modes. Differences can be due to the mass of the building being located in the core of the building in the idealised model. In the completed model, much of the mass is located outside the core and the loads are carried by the columns on the perimeter of the building.

In table 7.3 the differences of the accelerations between the idealised model and the completed model are shown. The difference between the models is up to 15%, depending on if Eurocode or NBCC is used. The only reason that a difference exists is due to the difference of the natural frequencies. If the acceleration in the torsional direction should be evaluated the building code from Japan can be used. When calculating accelerations with different standards there will be smaller differences between them. The equations to calculate accelerations are derived from wind tunnel tests, to make them usable they are then simplified. Depending on what simplifications are made the equations will give varying results depending on the input data.

There are a few ways to increase the stiffness of the building. To connect the main building to a lower side-building will increase the stiffness of the building and is a very common way to build high-rise buildings. How the side-building is oriented will affect contribution of stiffness to the high-rise building. The simulations in this dissertation were made by connecting the two buildings with fictitious bars. These bars had an infinite stiffness which may cause some errors in the results. In reality it can be difficult to connect different buildings due to the high stiffness needed to connect the buildings.

An efficient way to increase the stiffness of a building is to add an outrigger system. The disadvantage of using an outrigger system is that it limits the usage of one or a couple of floors where the outrigger walls must be placed. There is however normally floors dedicated to installations that can be used for the outrigger system which means that no usable space must be sacrificed.

Hand calculations were performed for outriggers and FE-models were also performed in Strusoft Frame Analysis and Midas GEN. The results of the hand calculations were in good agreement with the 2D-models in Strusoft Frame Analysis. The 3D-models in Midas GEN did however differ a lot from the hand calculations and the 2D-models. The 2D-models had a significantly lower stiffness than the 3D-models, so much that it is an unreasonable underestimation, even for the preliminary design. When making the idealised models in 2D and 3D, the columns can be replaced with spring supports to make the modelling easier. This will not have an affect on the results.

Calculating natural frequencies in a building with an outrigger system is very complicated, if this is needed a FE-model should be used. The calculation methods that exists for outriggers in non-prismatic buildings are also very complicated. When evaluating outriggers full FE-models should be used.

Increasing the thickness of the walls or changing the Young's modulus is not a very efficient way to increase the stiffness of the building. If the wall thickness is increased 40%, it will only result in 4% change of the natural frequencies. This is because an increase in the wall thickness will both increase mass and stiffness, meaning that the change for natural frequencies is very low. Increasing the wall thickness will however reduce stresses and reduce deflection of the building. In the results it can be seen that the change of Young's modulus results in a linear change of the natural frequency. The Young's modulus can be varied either by changing the amount of reinforcement or changing the concrete strength class. If the amount of reinforcement should be changed it cannot exceed the requirement of maximum amount of reinforcement according to Eurocode. Changing the amount of reinforcement or changing the concrete strength class could be very expensive due to the high quality of concrete needed. There are more efficient ways to increase the overall stiffness of the building than changing the thickness of the walls or the Young's modulus.

When comparing measured values to those obtained by FE-models there are large differences, especially for concrete core buildings. The buildings turn out stiffer than calculated which is an error on the safe side. However it can cause large extra costs due to extra material and less usable space in the building. Calculating accelerations according to Eurocode and NBCC will generally overestimate the accelerations compared to measured values.

Eurocode currently lacks a lot of information needed for the design of high-rise buildings. Eurocode's wind loads are only intended to be used up to 200 meter in height which creates limitations for very tall buildings. Furthermore there is nearly no information about how wind loads should be calculated on high-rise buildings and there is no information at all regarding torsional forces. These factors limits the ability to design high-rise buildings in Sweden and the rest of Europe.

To be able to evaluate comfort requirements in high-rise buildings, there is a need to include methods for calculating cross-wind accelerations and torsional accelerations in Eurocode. The Japanese standards include methods to calculate these accelerations that are based on wind tunnel test and experimental measurements. Implementing these methods into Eurocode and adjusting for European circumstances would probably be the best option to bring needed calculation methods to Eurocode. Eurocode also lacks recommended acceleration requirements, today the client needs to make decisions based on international standards.

In future versions of Eurocode there are goals to add more information regarding wind, specifically dynamic response, vortex-induced vibrations, response of structures and force and pressure coefficients. Size effects concerning large structures and dynamic loads for foundations are also planned to be added [15].

7.3 Further research

This dissertation has only considered the use of a core supported building with the possibility of adding outriggers for added stiffness. This structural system is regarded to be the one most likely to be used in Sweden at the current time. Further work could be carried out for other structural systems. Further research could be done on how Swedish laws, construction practices and financial aspects could affect the design and construction of high-rise structures in Sweden.

The Japanese wind code covers torsional vibrations, something that neither NBCC or Eurocode does. It would be of interest to study the Japanese wind codes for this reason.

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Appendix A

Eurocode

A.1 Eurocode

Below are the charts used to calculate the acceleration.

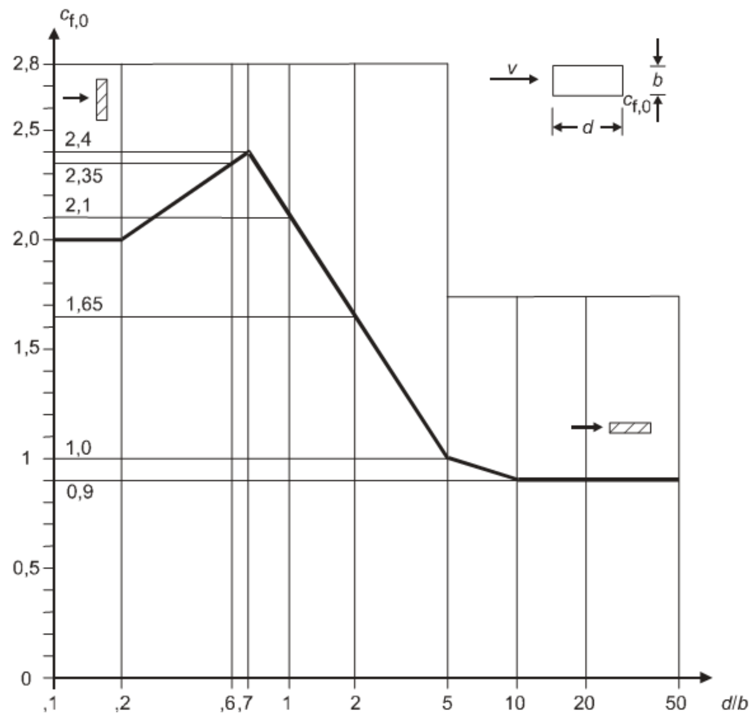


Figure 7.23 — Force coefficients $c_{f,0}$ of rectangular sections with sharp corners and without free end flow

Figure A.1: Force coefficients [33].

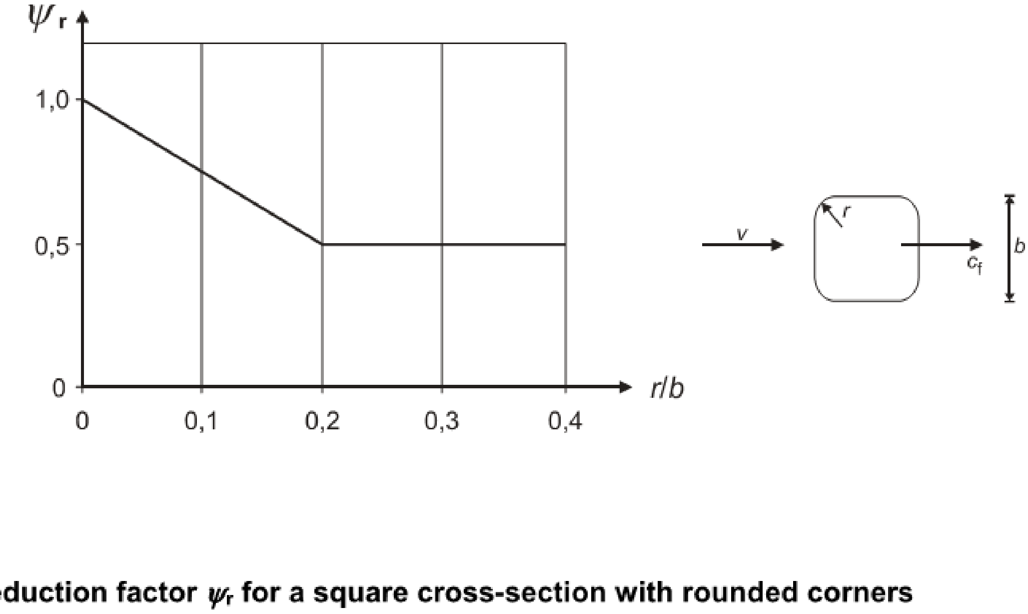


Figure A.2: Reduction factor [33].

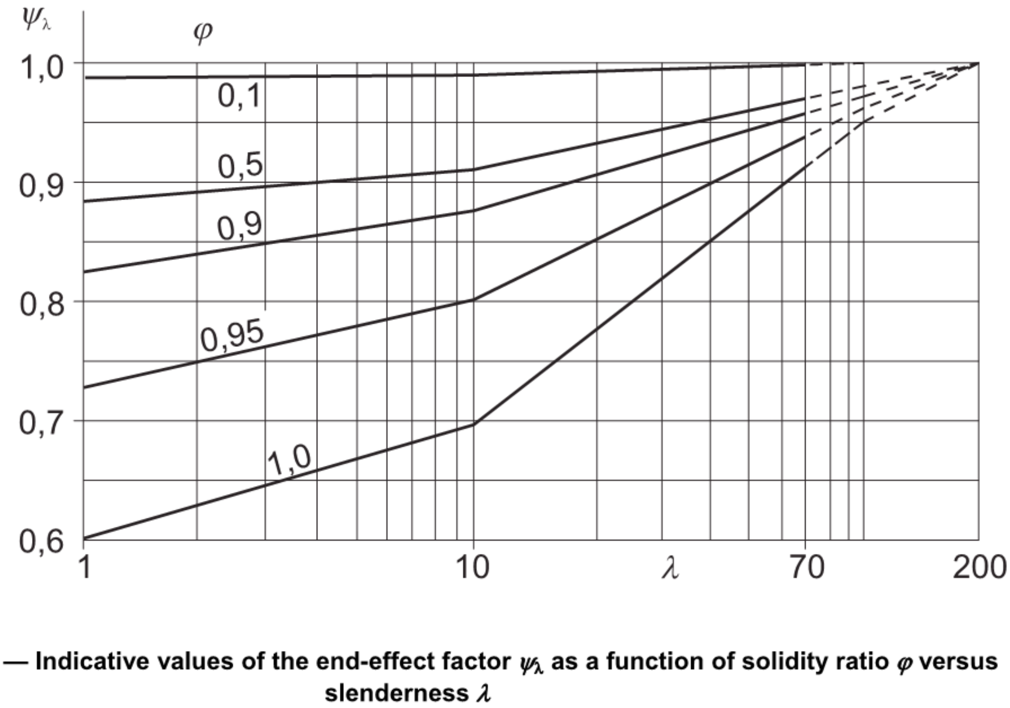


Figure A.3: End effect factor [33].

Table 7.16 — Recommended values of λ for cylinders, polygonal sections, rectangular sections, sharp edged structural sections and lattice structures

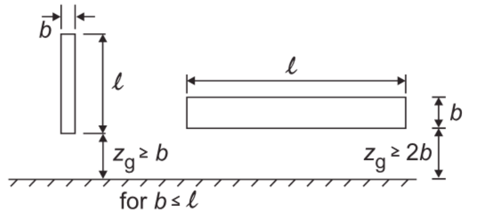
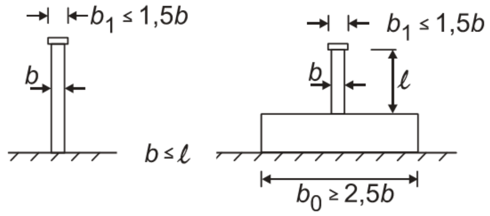
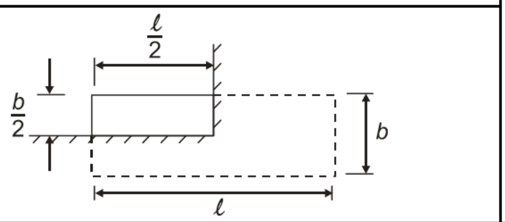
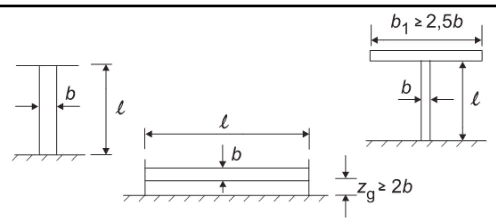
No.	Position of the structure, wind normal to the plane of the page	Effective slenderness λ
1		For polygonal, rectangular and sharp edged sections and lattice structures: for $\ell \geq 50$ m, $\lambda = 1{,}4\ \ell/b$ or $\lambda = 70$, whichever is smaller for $\ell < 15$ m, $\lambda = 2\ \ell/b$ or $\lambda = 70$, whichever is smaller For circular cylinders: for $\ell \geq 50$, $\lambda = 0{,}7\ \ell/b$ or $\lambda = 70$, whichever is smaller for $\ell < 15$ m, $\lambda = \ell/b$ or $\lambda = 70$, whichever is smaller For intermediate values of ℓ, linear interpolation should be used
2		
3		
4		for $\ell \geq 50$ m, $\lambda = 0{,}7\ \ell/b$ or $\lambda = 70$, whichever is larger for $\ell < 15$ m, $\lambda = \ell/b$ or $\lambda = 70$, whichever is larger For intermediate values of ℓ, linear interpolation should be used

Figure A.4: Slenderness [33].

Table 4.1 — Terrain categories and terrain parameters

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

(2) The terrain roughness to be used for a given wind direction depends on the ground roughness and the distance with uniform terrain roughness in an angular sector around the wind direction. Small areas (less than 10% of the area under consideration) with deviating roughness may be ignored. See Figure 4.1.

Figure A.5: Roughness length [33].

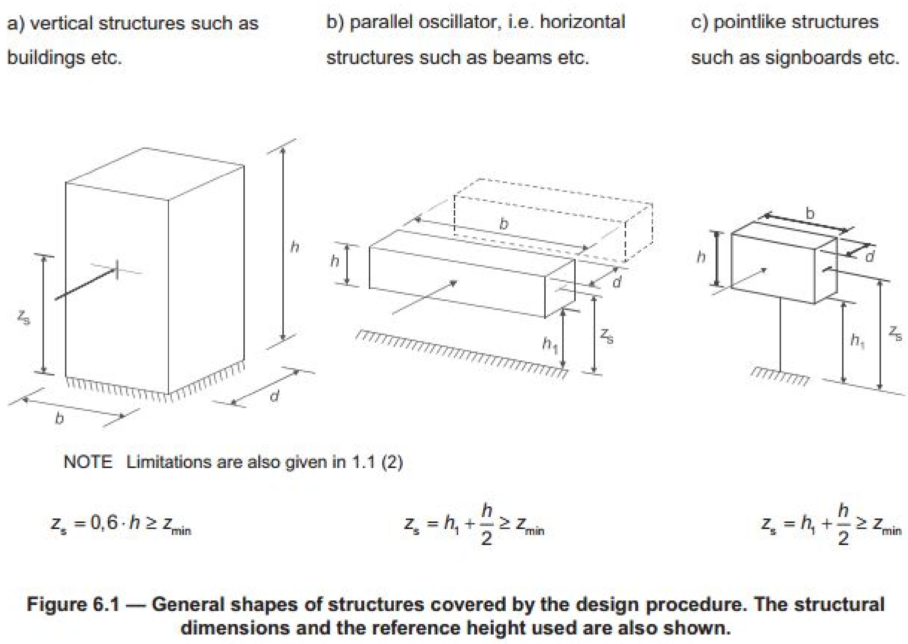


Figure A.6: Reference height [33].

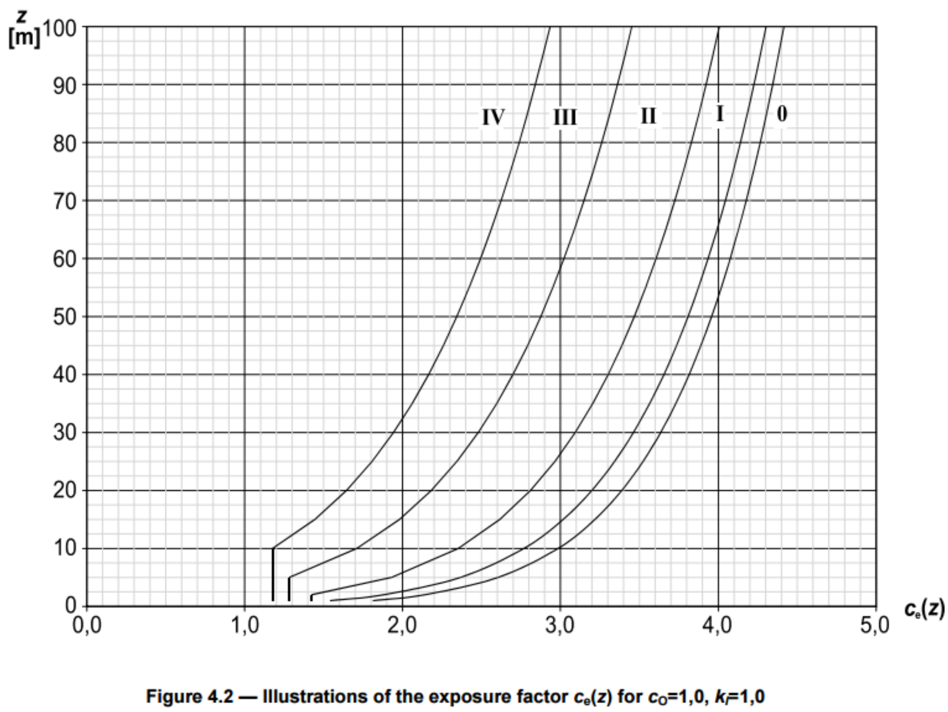


Figure A.7: Exposure factor [33].

Table F.2 —Approximate values of logarithmic decrement of structural damping in the fundamental mode, δ_s

Structural type		structural damping, δ_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

Figure A.8: Structural damping [33].

Tabell A1.1 – Rekommenderade värden för ψ-faktorer för byggnader

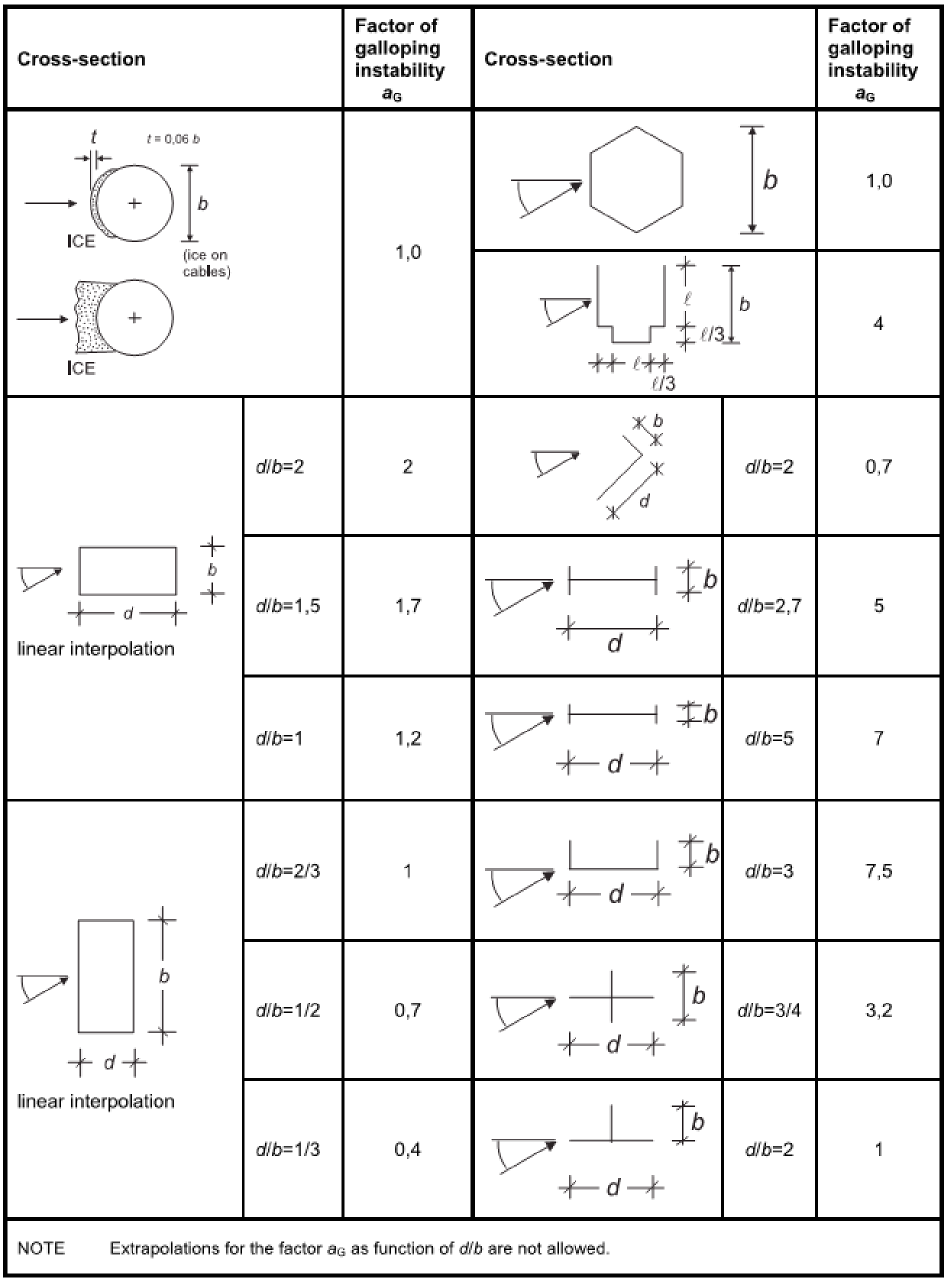
Last	ψ_0	ψ_1	ψ_2
Nyttig last i byggnader, kategori (se EN 1991-1-1)			
Kategori A: rum och utrymmen i bostäder	0,7	0,5	0,3
Kategori B: kontorslokaler	0,7	0,5	0,3
Kategori C: samlingslokaler	0,7	0,7	0,6
Kategori D: affärslokaler	0,7	0,7	0,6
Kategori E: lagerutrymmen	1,0	0,9	0,8
Kategori F: utrymmen med fordonstrafik, fordonstyngd ≤ 30kN	0,7	0,7	0,6
Kategori G: utrymmen med fordonstrafik, 30kN < fordonstyngd ≤ 160kN	0,7	0,5	0,3
Kategori H: yttertak	0	0	0
Snölast på byggnader (se EN 1991-1-3)*			
Finland, Island, Norge, Sverige	0,70	0,50	0,20
CENs övriga medlemsländer, för platser högre belägna än H > 1000 m.ö.h.	0,70	0,50	0,20
CENs övriga medlemsländer, för platser belägna lägre än H ≤ 1000 m.ö.h.	0,50	0,20	0
Vindlaster på byggnader (se EN 1991-1-4)	0,6	0,2	0
Temperaturlast (ej brand) i byggnader (se EN 1991-1-5)	0,6	0,5	0

ANM. ψ-värden kan fastställas i den nationella bilagan.

* För länder som inte uppräknas nedan, se aktuella lokala förhållanden.

Figure A.9: ψ factor [34].

Table E.7 — Factor of galloping instability a_G

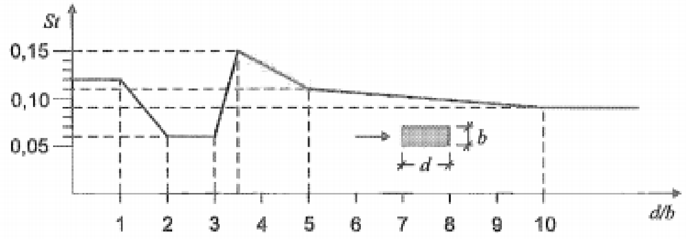


Cross-section		Factor of galloping instability a_G	Cross-section		Factor of galloping instability a_G
$t = 0{,}06\,b$; ICE (ice on cables); ICE		1,0			1,0
					4
linear interpolation	$d/b=2$	2		$d/b=2$	0,7
	$d/b=1{,}5$	1,7		$d/b=2{,}7$	5
	$d/b=1$	1,2		$d/b=5$	7
linear interpolation	$d/b=2/3$	1		$d/b=3$	7,5
	$d/b=1/2$	0,7		$d/b=3/4$	3,2
	$d/b=1/3$	0,4		$d/b=2$	1

NOTE Extrapolations for the factor a_G as function of d/b are not allowed.

Figure A.10: Galloping instability [33].

A.2 BSV 97



Figur 3:41c
Strouals tal *St* för rektangulär sektion med skarpkantade hörn.

Tabell 3:41a
Strouhals tal *St* och formfaktorn μ_{tr} för olika tvärsektioner

Tvärsektion		St	μ_{tr}
d	för alla Re-tal	0,2	se fig 3:41b
d, b	$0{,}5 \le d/b \le 2$	se fig. 3:41c	1,1
d, b	$d/b=1$	0,11	0,8
	$d/b=1{,}5$	0,10	1,2
linjär interpolation	$d/b=2$	0,14	0,3
d, b	$d/b=1$	0,13	1,6
linjär interpolation	$d/b=2$	0,08	2,3
d, b	$d/b=1$	0,16	1,4
linjär interpolation	$d/b=2$	0,12	1,1
d, b	$d/b=1{,}3$	0,11	0,8
linjär interpolation	$d/b=2{,}0$	0,07	1,0

Figure A.11: Strouhals number [52].

Appendix B

NBCC

B.1 NBCC

Below are the charts used to calculate the acceleration according to NBCC.

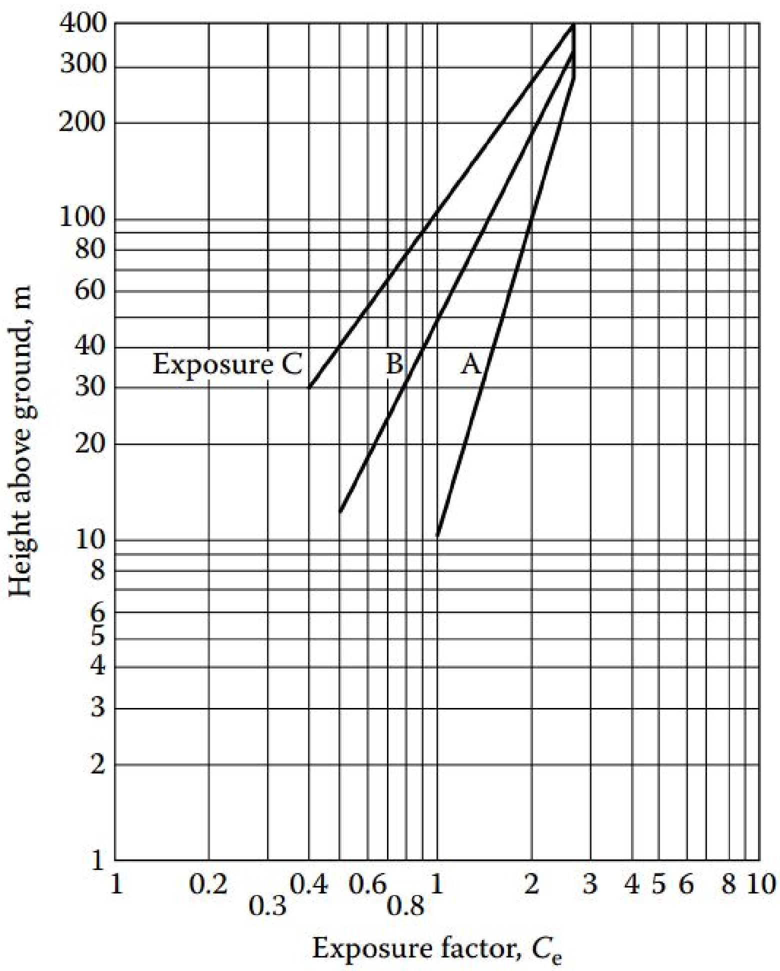


Figure B.1: Exposure factor [64].

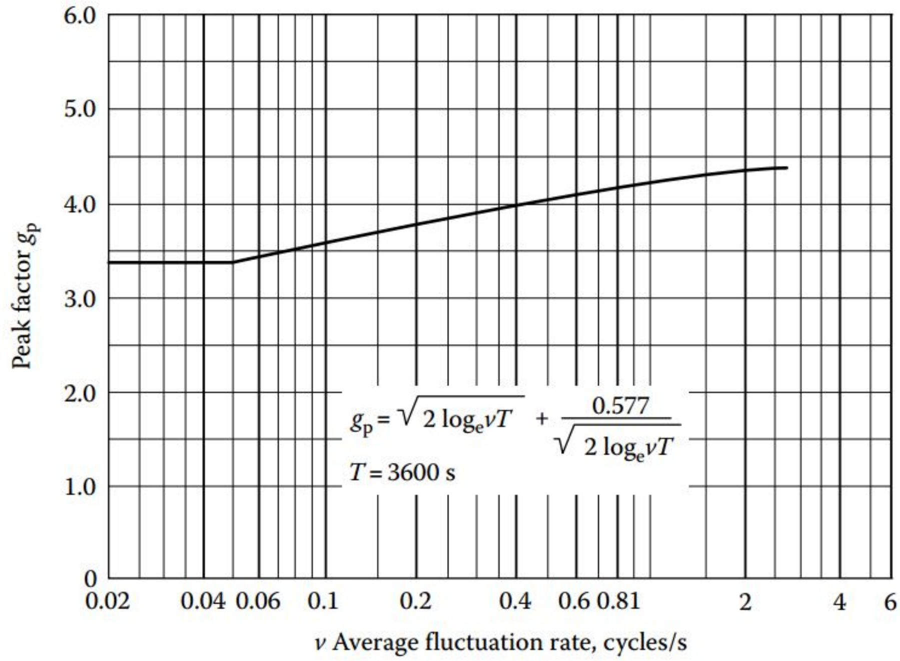


Figure B.2: Peak factor [64].

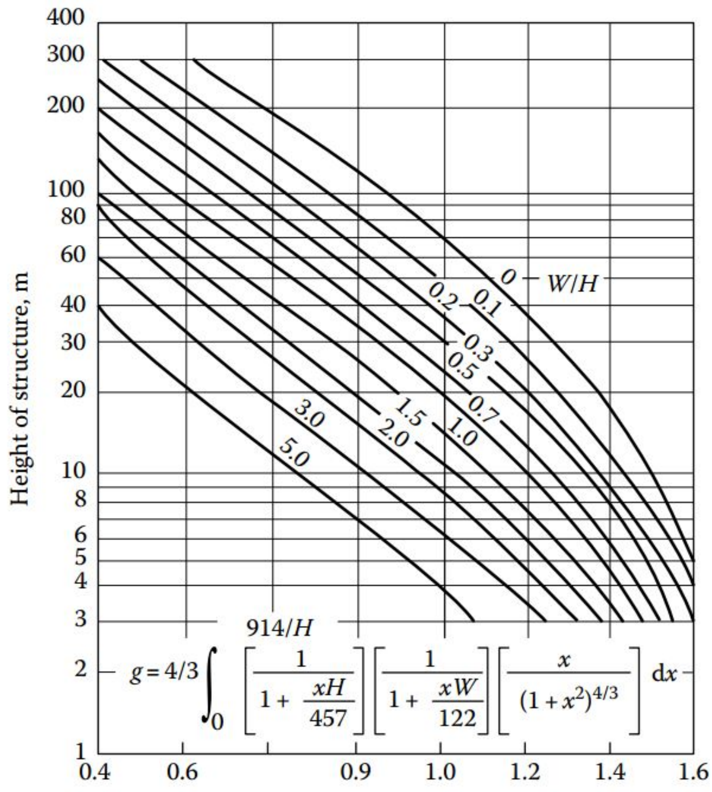


Figure B.3: Background turbulence factor [64].

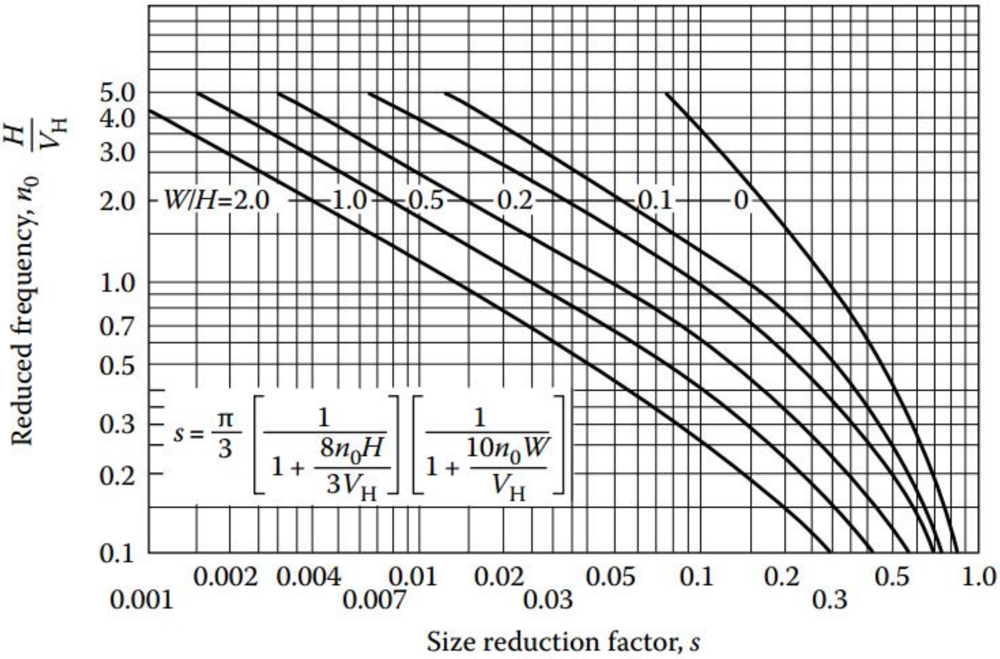


Figure B.4: Size reduction factor [64].

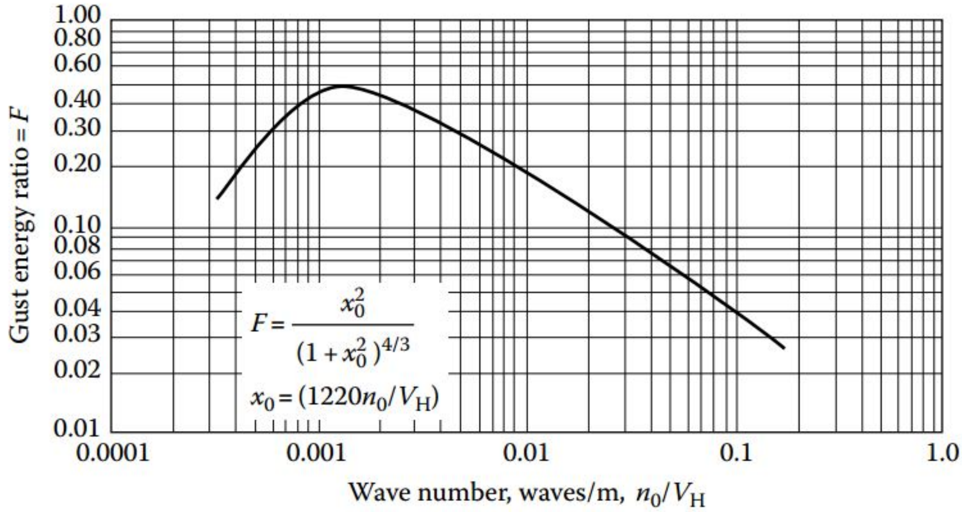


Figure B.5: Guest energy ratio [64].

Appendix C

Calculations

C.1 Mass calculation

In this section some of the calculations are shown.

Mass calculation GCG

Input data

Building parameters		
Number of floors	34	
Office floors	31	
Number of floors, office type 1	7	
Number of floors, office type 2	12	
Number of floors, office type 3	12	
Garage floors	3	
Floor height	3,6	m
Total height	120	m
Number of columns per floor	16	

Loads		
Office load*	0,5	kN/m2
Parking load*	0,5	kN/m2
Self weight concrete	25	kN/m3
Concrete on hollow core slab	0,03	mm
Installations	0,5	kN/m2

*20 % of live load

Variables

Elevator schaft type 1		
Wall thickness	550	mm
m^2 concrete wall	23,1	m2
Elevator schaft type 2		
Wall thickness	450	mm
m^2 concrete wall	18,9	m2
Elevator schaft type 3		
Wall thickness	350	mm
m^2 concrete wall	10,12	m2

m^2 office type 1	822	m2
m^2 office type 2	822	m2
m^2 office type 3	822	m2
Antal kvd parkering/plan	822	m2

Column dimensions	0,49	m2
Slab thickness	0,206	mm

Results

Live loads		
Office load	12741	kN
Parking load	1233	kN
Dead loads		
Elevator schaft	45894,6	kN
Columns	23990,4	kN
Slab	143931,85	kN
Installations	13974	kN

Total	241764,85	kN
	24176485	kg
	24176	ton
	201471	kg/m

Figure C.1: Mass distribution in building.

C.2 Acceleration

In the dissertation the acceleration is calculated with Microsoft Excel, in the figures below the an example of the calculations are shown.

C.2.1 Acceleration according to Eurocode

Acceleration according to Eurocode

Variables								
Calculated values								
Reduced wind speed according to EKS	**v_b**	**26**	v_Ta	22,23				
Roughness factor	**co**	**1**	**kr**	0,234	cr	1,122	v_m(h)	24,939
Orography factor	**z_0**	**1**			cr(z_s)	1,002	v_m(z_s)	22,278
	z_0,11	0,05						
Fundamental frequency of the building	**n_1,x**	**0,33**	ϕ_b	0,371				
Average wind velocity of top of building	v_m(h)	24,939	ϕ_h	0,239				
Width of building	**b**	**40**						
Height of building	**h**	**120**						
	n_1,x	0,33						
	v_m(h)	24,939	y_C	1,985	F	0,073	R^2	0,365
Force coefficient	**c_f**	**1,188**						
Reference height, 0,6*h for high buildings	z_s	72						
Density of air	ρ	1,2	δ_a	0,011				
Average wind velocity at reference height	v_m(z_s)	22,278						
Equivalent mass per unit of length	**m_e**	**175689**						
	n_1	0,33						
	b	40						
0,1 for concrete building och 0,05 for steel buildings	δ_s	0,1						
	b	40						
	h	120						
	h_ref	72	B^2	0,955				
	B^2	0,955						
	R	0,604						
	n_1,x	0,33	v	0,174				
	v	0,174						
Time when acceleration is calculated, 600 s according to EC	T	600	k_p	3,245				
Orography factor according to appendix A.3, Eurocode	**c_0(z)**	**1**	I_v(z_s)	0,234				
Height of interest	z	120	I_v(h)	0,209				
Roughness length from table 4.1, Eurocode	z_0	1						
	I_v(h)	0,209						
	R	0,604						
Reference mean (basic) velocity pressure at height h	q_m(h)	373,162						
	b	40	σ_X	0,038				
	c_f	1,188						
1 in the top	ϕ_1,x(z)	1						
Mass per unit of length	**m**	**175689**						
	k_p	3,245	Acceleration	0,124	m/s^2			
	σ_X	0,038		1,264	% of gravity (2% is the acceptable level)			

Figure C.2: Calculation of acceleration according to Eurocode.

C.2.2 Acceleration according to NBCC

Acceleration according to NBCC

Variables								
Calculated values								
Reference wind speed at 10 m	**V-**	**22,23**					Used to calculate F	
Exposure factor at the top ef the building	**C_CH**	**1,3**	v_H	25,346			x_0	17,328
Fundamental frequency in the cross wind direction of the building	**n_w**	**0,59**	a_r	0,223				
Width of building	**b**	**40**						
Depth of building	**d**	**24**						
Height of building	**H**	**120**						
Fundamental frequency, minimum of cross wind and along wind vibration.	**n_0**	**0,36**						
Size reduction factor	s	0,028	v	0,173	g_p	3,754		
Gust energy ratio, can be calulated or selected from figure	F	0,149						
Background turbulence factor,	**B**	**0,7**						
Critical damping ratio. 0,01 for steel, 0,015 for composite and 0,02 for concrete	**β**	**0,02**						
	T	**3600**						
	n_b	0,59						
	g_p	3,754						
	b	40						
	d	24						
	a_r	0,223						
Density of building	**ρ_B**	**186**						
	β	0,02						
Gravity	g	9,81						
Surface roughness of the terrain. 0,08 for A, 0,10 for B and 0,14 for C	**K**	**0,14**						
	C_eH	1,3	σ/μ	0,313				
	B	0,7						
	s	0,028						
	F	0,149						
	β	0,02						
	g_p	3,754	C_g	2,175				
	σ/μ	0,313						
	C_g	2,175						
Maximum deflection, can be set to H/450 for preliminary design	**Δ**	**0,267**						
Fundamental frequency in along wind direction	**n_d**	**0,36**						
	g_p	3,754						
	K	0,14						
	s	0,028						
Exposure factor	F	0,149						
	C_e	1,3						
	β	0,02						
Power coefficient related to C_e, 0,28 for A, 0,50 for B and 0,72 for C	**alpha**	**0,72**						
	q	0,321						
	a_w	**0,035**	m/s^2 (cross wind acceleration)					
		0,356	% of gravity (2% is the acceptable level)					
	a_d2	**0,008**	m/s^2 (along wind acceleration)					
		0,772	% of gravity (2% is the acceptable level)					

Figure C.3: Calculation of acceleration according to NBCC.