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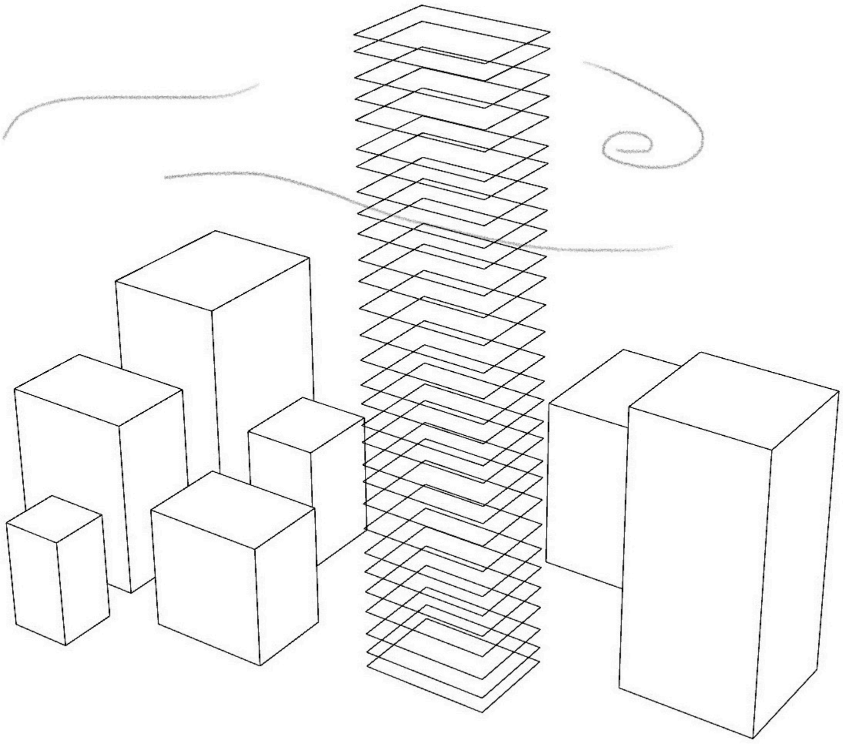

Early estimation of a structural systems' stiffness in high-rise buildings

A development of guidelines in design for wind

Master's thesis in Structural Engineering and Building Technology

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Master's thesis 2023

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Abstract

As our buildings grow higher so should the knowledge in the design of high-rise buildings. Tall structures require specialised understanding due to the great impact of wind load and subsequently the importance of stability. In the conceptual phase of design, it is essential to make a good estimation of the required stiffness. Today's building standards generally provide a conservative geometry with overestimated loads. A development of guidelines for the design of high-rise buildings in an early stage are desirable and consequently the ambition of this thesis.

The aim of the thesis is to examine current methods of design in a comprehensive analysis of high-rise buildings. By evaluating and comparing the design method by building codes with the result from provided wind tunnel testing, new guidelines for early assessment of a high-rise buildings' structural system will be reached.

To accomplish this research different methods of analysis is carried out. To incorporate current design approaches, interviews with structural engineers specialized in tall buildings are conducted. An analytical analysis is conducted by current building codes for design of high-rise buildings followed by a FEM analysis to investigate required stiffness. The structural system response is evaluated by comfort criteria which set the base for the development of new guidelines.

The research showed a distinctly difference of wind loads predicted by building standards and wind tunnel testing. This resulted in a discrepancy between required stiffness for different configurations of the structural system. Observing the results, a development of guidelines for early estimation of a structural systems' stiffness in high-rise buildings were reached.

Keywords: High-rise buildings, wind, building standard, wind tunnel test, FEM-modelling, stiffness, early estimation, building design.

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Sammanfattning

I samma takt som våra byggnader växer sig högre, bör också kunskapen i design av höga hus. Höga konstruktioner kräver specialiserad förståelse på grund av vindens kraftiga inverkan och därmed betydelsen av stabilitet. I en konceptuell fas av desig är det viktigt att göra en bra uppskattning av erforderlig styvhet för höga hus. Dagens byggnormer ger generellt en konservativ geometri med överdimensionerade laster. Utveckling av riktlinjer för design av höghus i ett tidigt skede är önskvärt och följaktligen ambitionen med denna forskning.

Syftet med avhandlingen är att undersöka aktuella designmetoder i en omfattande analys av höga hus. Genom att utvärdera och jämföra design metoder i byggnormer med resultat från vindtunneltest, kommer nya riktlinjer för tidig uppskattning av höga hus och dess bärverk att utvecklas.

För att genomföra denna forskning utförs olika analyser inom höga hus. För att integrera nuvarande metoder av design genomförs intervjuer med byggnadskonstruktörer specialiserade på höga hus. En analytisk undersökning genomförs av nuvarande byggrekommendationer för projektering av höga hus följt av en FEM-analys för att undersöka erforderlig styvhet. Bärverkets egenskaper utvärderas mot komfortkrav som skapar möjlighet till utveckling av nya riktlinjer.

Forskning visade en distinkt skillnad mellan vindlaster som uppskattats av byggstandarder och vindtunneltest. Detta resulterade i en diskrepans mellan erforderlig styvhet för olika konfigurationer av bärverk. Genom observervation av dessa resultat utvecklades riktlinjer för tidig uppskattning av styvhet i bärverk för höga hus.

Keywords: Höghus, vind, byggnadsstandard, vindtunneltest, FEM-modellering, styvhet, tidig uppskattning, byggnadsdesign.

Acknowledgements

This Masters' thesis focuses on the development of guidelines in design directed towards an early estimation of high-rise buildings. The work of this thesis was performed during spring 2023 as a finalization of the Masters' program Structural Engineering and Building technology. The thesis is a collaboration between the Building Construction Department at Sweco Sverige AB in Gothenburg and the Department of Architecture and Civil Engineering at Chalmers University of Technology. I would like to show my gratitude to my supervisor at Sweco, Per Langefors for his contributory guidance and advise throughout the work of this thesis. I would also like to address a thanks to my examiner and supervisor from Chalmers, Ignasi Fernandez for his valuable consultation and encouragement for this research. Last but not least I would like to send thanks to the structural engineers Andreas Lindelöf, Marco Binfaré and Gustav Söderlund for their engagement and contribution to my interview study.

Hanna Josefsson, Gothenburg, June 2023

List of Acronyms

Below is the list of acronyms that have been used throughout this thesis listed in alphabetical order:

CFD	Computional fluid dynamics
DR	Drift ratio
EC	EuroCode
EKS	EUropeiska konstruktionsstandarder
FEM	Finite element method
HFFB	High-frequency force balance
HFPI	High-frequency pressure integration
ISO	International Organization for Standardization
NBCC	National Building Code of Canada
SLS	Service limit state
SS	Svensk standard
SR	Slenderness ratio
ULS	Ultimate limit state
WTT	Wind tunnel test

Nomenclature

Below is the nomenclature of parameters and variables that have been used throughout this thesis.

Roman upper case letters

A_{ref}	Reference area
B^2	Background factor
D	Along-wind building plan dimension
$(EI)_{WTT}$	Stiffness based on wind tunnel test wind load
$(EI)_{EN}$	Stiffness based on building standard wind load
K_x	Dimensionless coefficient
L	Turbulence length
R^2	Resonance response factor
Re	Reynolds number
S_L	non-dimensional power spectral density function
T	Mean wind velocity time
W	Across-wind building plan dimension
$\ddot{X}_{max}$	Along-wind acceleration

Roman lower case letters

b	Building width
$b_{average}$	Average building width
c_{dir}	Directional factor
c_e	Exposure factor
c_f	Force coefficient
$c_{f,0}$	Fundamental force coefficient

c_o	Orography factor
c_{pe}	External pressure coefficient
c_{pi}	Internal pressure coefficient
c_r	Roughness factor
c_{season}	Season factor
$c_s c_d$	Structural factor
f_L	Non-dimensional frequency
h	Building height
h_{strip}	Segment of building height
k_l	Turbulence factor
k_p	Peak factor
k_r	Terrain factor
l_v	Turbulence intensity
m_e	Equivalent mass per unit length
$n_{1.x}$	Fundamental frequency
q_b	Basic velocity pressure
q_p	Peak velocity pressure
r	Radius
v_b	Basic wind velocity
$v_{b,0}$	Fundamental basic wind velocity
v_c	Peak wind velocity circular section
v_m	Mean wind velocity
vT_a	Average wind velocity
v_{50}	Characteristic fundamental wind velocity
w	Building minimum width
w_e	External wind load
z	Observed building height
z_{max}	Maximum height
z_{min}	Minimum height
z_s	Reference height
z_0	Roughness length
$z_{0,II}$	Roughness length for terrain factor II

Greek letters

δ	
δ_a	Logarithmic decrement for aerodynamic damping
δ_s	Logarithmic decrement for structural damping
Δ	Transverse deflection
λ	Slenderness
ρ	Air density
ν	Mean up-crossing frequency
ν_k	Kinematic viscosity air
ψ_r	Reduction factor round corners
ψ_λ	End-effect factor free-end flow
$\sigma_{\ddot{x}}$	Acceleration standard deviation
φ	Solidity ratio
$\phi_{1,x}$	Mode shape

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1

Introduction

This Masters' thesis is a research of high-rise buildings and their structural systems with the ambition to advance current guidelines of design. This chapter cover an background description along with the aim and scope of this thesis.

1.1 Background

Throughout history, high-rise buildings became a symbol for modernization and prosperity. Along with new building technology and the costly and limited space in the cities, high-rise buildings started to qualify as an economic and functional solution. In the beginning of the 20th century, a building of 16 floors was appointed the most profitable height, weighing income against investment cost (Hellemar, 2017). Today, the definition of a high-rise building is distinctly higher. Growing up to over 100 floors, the structure is seen more as a technical difficulty rather than an economic advantage (Caldenby, 1990).

Due to its height, the design of high-rise buildings is very complex. Current building codes and guidelines on wind loads are oriented to low-rise buildings and subsequently often result in conservative and uncertain geometries of the structural system in high-rise buildings. Further on, building standards generally do not cover important wind effects, for instance aerodynamics and crosswind excitation. A reliable solution to obtain a more realistic load condition due to wind action is to use wind tunnel tests (Irwin, Denoon, & Scott, 2013). However, wind tunnel test are costly and time consuming and consequently only performed in the end phase of the design. A wind tunnel test is generally used in a detailed end design to reduce overestimated material as well as being a confirmation of the earlier design choices.

To make an assessment of a high-rise buildings' structural system in early stage, FEM modelling is commonly utilized. This method brings the results substantially closer to the wind tunnel test outcome in comparison to static calculations by building standards. FEM modelling of high-rise buildings can be performed in various modelling programs and in diverse level of detail.

The development of structural assessment and building design have over time been the result of pushing the limits and striving towards the impossible (Isaksson, Mårtensson, & Thelandersson, 2020). In early planning of high-rise buildings, it is crucial to make a good estimation of how much space the stabilizing structural system will take in order to design the floor plan. To facilitate the design process of high-rise buildings, particularly in early stages, an improvement of current building standards is desirable and is the ambition and motivation for this thesis.

1.2 Aim

This Masters' thesis aims to be incorporated in new guidelines to use in early estimations of high-rise buildings in order to make better approximations of how much space the structural system will take. By critically identify and evaluate differences in current design techniques, new understanding that is crucial for the improvement of the design process of high-rise buildings will be reached.

1.2.1 Objectives

The aim of the thesis will be achieved by the accomplishment of the following objectives:

- Identification of different methods for design of high-rise buildings by performing a literature study.
- Interviews with structural engineers specialized in the design of tall buildings.
- Implementation of principal calculations for high-rise buildings by building standards including comparison with wind tunnel test result.
- FEM analysis of three reference buildings, conducting an eigenfrequency investigation to use in comfort criteria evaluation.
- Development of methods on early estimation of a structural systems' stiffness based on an assessment of previous conclusions.

1.3 Method

The initial phase of the thesis consists of a comprehensive literature study on the design methods of structural systems for high-rise buildings. The literature study is performed to give the thesis a foundation and to be able to integrate obtained knowledge in the proceeding development of the work.

To gather, document and possibly incorporate current design approaches within the industry in this research work, interviews with high-rise building designers are carried out. The goal with these interviews is to assimilate different views of the current techniques of designing high-rise buildings, identify what is missing and highlight what is in need of development.

Three different high-rise buildings will be used as case study for the development of the thesis. Following, a comparison is made for each building where the wind loads is calculated analytically by building standards respectively extracted from provided wind tunnel testing. This will allow to identify the gap between the different design approaches for tall buildings. To analyze the buildings' response and subsequently extract required stiffness of the structural system, finite element programs are used.

By critically evaluating and comparing existing knowledge together with results gathered from the FEM analysis, the desired development of methods is to be established.

1.4 Scope and limitations

This thesis is an investigation of high-rise buildings. The thesis is provided with three different reference buildings that are observed and utilized for developing new guidelines for deign. The buildings observed are each of different height and configuration. This allow for a comprehensive study of high-rise buildings of various designs. The analysis will be limited to these reference buildings and the final conclusions are based on these results. Wind tunnel tests and architectural drawings relative to each reference building are provided by Sweco Sverige AB.

Different buildings code standards appurtenant to different regulations regarding wind loads and comfort demands. Although a diversity of methods would present a broader perspective, this thesis is limited to Eurocode European standards (EN-1991-1-4, 2005) together with additional standards by International Organization for Standardization (ISO:10137, 2008) and Building construction recommendations EKS:12 (BFS:2022:4, 2022). Acceleration and frequency calculations are only performed for along-wind accelerations, as described in EN-1991-1-4. The Eurocode does not consider across-wind acceleration and neither does this thesis.

FEM modeling are conducted in the finite element program RFEM and the models are designed simplified where the main structural system alone is assumed to carry all the load. The slabs are modelled with an simplified geometry as the loading are manually concentrated on the core walls. The core walls are modelled with fewer openings than in reality which needs to be considered in the end results. Furthermore, the material of the structures is assumed to be entirely made out of concrete and behave linear elastic. Although different concrete qualities are used in each building, the structural systems are modelled using an average concrete class of C40/50. The stiffness is collected by a section generator in the program FEM-Design where the simplified cross sections are sketched.

In design of high-rise buildings the influence of piles are crucial, especially for structures mounted at ground conditions with geotechnical difficulties. Wind loads acting on the building are transferred down to the piling foundation. Subsequently, the stiffness of the piles are of great importance in design. However, the design of piles are of a completely different investigation and hence not discussed in this thesis.

Tall structures are designed based on the impact of wind and seismic activity. The buildings considered in this thesis are based in Sweden where seismic activity is not an issue. Of this reason, seismic consequences are not considered. If development of guidelines for the design of high-rise buildings are to be conducted in a location where seismic activity is crucial, this needs to be addressed.

1.5 Outline of report

The thesis is divided into several chapters which added together provides for the development of guidelines for design.

Chapter 2	Literature study of high-rise buildings and how to design for wind.
Chapter 3	Interview study with designers specialized in high-rise building design.
Chapter 4	A case study of three different high-rise buildings.
Chapter 5	The theory of building standards are presented and explained.
Chapter 6	Description of FEM analysis conducted for each case building.
Chapter 7	Presentation of results from analytical analysis and FEM modelling.
Chapter 8	Discussion of results.
Chapter 9	Final remarks with conclusion and proposal for further investigations.

2

High-rise buildings

2.1 Why we build tall

As the world grow, so does our buildings. Throughout history, there have always been an interest in building towards the sky. Building tall advocate for wealth and power and have been a tool for promoting prosperity. In present time, we build tall to make money. High-rise buildings that generate further floor area while utilizing the simple, repeating building design that is a natural choice for skyscrapers would enlarge revenue. However, the costly space together with construction and maintenance expenses needs to be compensated by the rent income. Subsequently, the buildings must be suitable for people meant to use them. The most profitable design to make money would be to maximize the using space inside whilst keeping the construction and architecture cost at a minimum (Ascher, 2011).

In decision of building height, there is an economic height considered which give the highest return of investment cost. A buildings' efficient height is strictly depending on the design of the structural system. Even if a taller building contribute to extra floor space, higher buildings also require a stronger core, additional bracing and larger mechanical systems that all occupy an extensive part of the buildings' space area. Since the commercial viability is as vital as the structural design itself, the buildings' height are rarely adapted to its efficient height (Ascher, 2011). From an economic perspective, a minimum net floor area of 75% is needed for a building to be regarded as efficient in its use (Sarkisian, 2016).

The definition of a high-rise building is diverse. One interpretation is that a building can be labeled a high-rise building when the height exceeds 23 meters (Sarkisian, 2016). Another characterization is to regard a building as tall when the aspect ratio exceeds 5:1. All together it can be stated that a building is considered a high-rise when its height affect the design substantially (Truby, 2014).

2.1.1 Architecture and design

Building aesthetics are often a secondary consideration due to the importance of other aspects such as construction and safety. However, the design of high-rise buildings is an iterative process between structural engineering and architecture (Ascher, 2011). Skyscrapers has a vital impact on its surroundings and its design needs to match its location and purpose. Due to economical reasons, the structure of high-rise buildings often becomes the architecture itself (Ascher, 2011).

The initial phase of designing a high-rise building is to select a core. The location of the core can vary and together with its composition it will have a huge impact on the floor layout. The core act as the main structural component which provide the building with required stiffness to resist lateral forces (Ascher, 2011).

A valuable part of high-rise buildings is simplicity. By designing for purity, the structure will response in harmony with the architecture (Sarkisian, 2016).
Often in high-rise buildings, the structural design can be translated by looking at the architecture. By designing the structure to follow the natural flow of force, the building becomes the most efficient in its use. The force always takes the shortest and easiest way through the structure (Sarkisian, 2016).

2.1.2 Societal consideration

Super-tall buildings today are of multi-use; residential, office space, hotels, gyms etc., making each skyscraper an individual "vertical city". In toady's society, people spend approximately 90% of their day inside, independently on activity. Consequently, the buildings we spend our lives in affect our health (Ascher, 2011).

The wind load acting in the building, do not only bring technical issues of deign, but the sway induced by wind becomes a problem for the users of the building. The movements generated by the wind could potentially cause motion sickness and discomfort for the users. In the US, the limit for sway is 1/500th of the building height (Ascher, 2011). While in Europe, there is only recommendations for maximum top deflection limit.

The floor plan of the building needs to be design in line with its purpose. For high-rise buildings the usage is typically of either residential, hotel, commercial or retail use. Although every individual type of use needs to be addressed through design with differences in height, acoustic measures, and spacing of walls, the main dominating discrepancy is the occupancy rate. For residential use of a building with an area of 1000m2 the occupancy rate would land between 20-30 people whilst hotel use would be 35-40 people and 80-120 people for office use (Truby, 2014).

Since the human is more sensitive to motion when laying down, the limit of acceleration is lower for buildings of residential use rather than office or commercial based spaces. For one year horizontal acceleration return wind period, the criteria for an office space is 10-13 milli-g's compared to a residential of 5-7 milli-g's. These limits are based on a damping ratio of 1,5% for concrete and a maximum predicted acceleration from wind impact (Sarkisian, 2016).

2.2 High-rise buildings and its structural systems

When designing a high-rise building, the same design approach as for low-rise buildings is made, but with some additional considerations. Specialists in different professions frequently needs to be involved in the process such as wind, geotechnical and fire specialists. A significant difference from designing low-rise buildings is the impact of wind and the lateral loading due to this natural phenomenon causes. Since the wind load is very dominant for high-rise buildings, this load case usually governs the design and choice of structural system, especially the selection of main stabilizing units such as core, columns and walls (Truby, 2014).

In this regard sway of the building becomes an important requirement difficult to meet. To prevent excessive swaying of the building it is fundamental to provide the structure with a bracing system. The lateral bracing of the building gives the structure its required stiffness and restrict movements induced by wind loads and other lateral loads (Ascher, 2011). Furthermore, the structural framing system will not only provide the building with its needed strength and stiffness but also determine the structures' responses to drift and accelerations (Truby, 2014).

Besides the structural system itself, dampers are commonly used to shift weight around in order to prevent heavy sways caused by wind (Ascher, 2011). To obtain a high degree of efficiency of the structures' stiffness, the structural elements should be placed at the buildings perimeter. To acquire the most efficient stiffness in a high-rise building, a squared shaped structure with large columns in each corner generates the best result in this manner. While for a circular shaped structure with columns at its edges the efficient stiffness is reduced to 50% in comparison due to increased dynamic behaviour (Sarkisian, 2016).

2.2.1 The core

The core is the fundamental part of a buildings' structural system. It will accommodate all vertical movement such as elevators and stairs. Most importantly it will be the dominant component resisting lateral loading (Truby, 2014).

Due to its high importance, the core needs to be designed in a precise and optimized way. An ineffective design will increase the final cost since it will affect both the usage of floor area and the quality of the structural system, and therefore aggravate the construction process (Truby, 2014).

Using core shear walls for stability, efficiency is found when locating these walls symmetrically in relation to the centre lines of the structure. Increased lengths of the core walls will also provide efficiency to the structure. Often, the thickness of the core wall is situated between 350mm to 600mm when observing buildings up to 200m in height. Due to this considerable thickness, it is crucial to make detailed approximations in initial phase of design in order to develop the floor plan in collaboration with architects (Truby, 2014).

2.2.2 Building loads

In early design it is vital to design for service limit state, SLS, which is often the governing design aspect. However, ultimate limit state, ULS, also needs to be analysed. In high-rise buildings, wind loads are governing for the design of high-rise buildings (Truby, 2014). Seismic loads is not considered in Sweden due to its non-critical seismic ground.

The structural system is designed based on the loads acting on the building. There are sustained loads (permanent load) from the buildings self weight and transient load (variable load) from forces acting on the building. The permanent load is of maximum certainly whilst the transient loads depends on the behaviour of the system. When designing for high-rise buildings, load combinations of transient and sustained load is accommodated by reduction factors, weighing up for the probability that the worst loads will not occur simultaneously (Sarkisian, 2016).

In careful design, the gravity loads from the structure can be favorable and used to resist overturning moments caused by imposed loading (Sarkisian, 2016). In-situ reinforced concrete is often used when building tall. Since concrete have a large amount of self-weight, the structure alone helps with the stability (Truby, 2014). A rough estimation is that as the height of the building is doubled, so is the total gravity load (Ascher, 2011). The material have also natural damping that resist sway. The natural damping of a material is defined in how well the material is on disperse energy (Truby, 2014).

The dominant load affecting the design of high buildings are the lateral load caused by wind (Ascher, 2011). The load from wind acting on the building facade, increases with the height of the building, exponentially (Sarkisian, 2016). When doubling the height of the building, the lateral load caused by wind forced are increased with a factor of four (Ascher, 2011).

Wind will give rise to a dynamic effect of the building since the load is not static. The load is added in different degrees and unpredictable times, even though one can predict an roughly repeating cycle of wind motions (Truby, 2014). The effect of wind loads could be damaging of the structure. It can also create discomfort to the buildings' users. The building can also cause unpleasant wind motions at street level.

2.3 Design for wind

Wind is a complicated phenomenon that acts in three dimensions and needs to be considered over time. When designing for wind, the wind load prediction is mainly gained from observational studies of the current wind climate on site. The phenomena wind is both depending on the orography and temperature differences at the surface (Hughes, 2014).

Wind load is a variable load that is mainly affecting the external surfaces of a building, but also create an internal pressure. Measured in force per surface area, the wind load is assumed to be directed perpendicular to the buildings' facade. Generally, structures today possess enough mass and natural damping to resist momentary loading of the dynamic wind force (Isaksson et al., 2020). The wind load pressure and correspondingly the wind velocity is increased gradually with height of the building facade. Above some point, depending o the building design ad location, the wind speed is assumed constant (Hughes, 2014).

The orientation and shape of the structure is of great importance when designing a high-rise building for wind movement (Sarkisian, 2016). Wind pressure acting on a building depends on different aspects, wind direction and speed as well as the shape of the building. Wind is seldom uniform, the speed increased at higher height but is more predictable. To reduce the lateral pressure on the structure, the building can be designed with holes in it (Ascher, 2011).

Wind acting directly on the windward side of the building has the greatest impact on the structure. The negative suction pressure that occurs on the leeward side and adds on to the loading together with drag effects parallel to the wind direction. The wind direction, magnitude and the topography is different for all sites and contribute to change in designing for wind. Fundamental observation in design in the strength of the structure along with its serviceability. High-rise buildings are most commonly designed looking at across-wind rather than along-wind. This is due to the phenomena vortex shedding that generate normal forces to the perpendicular side of the buildings (Sarkisian, 2016).

The wind impact on the building is strongly influenced by the topography of the location. The resulting wind pressure is substantial higher in an open area in comparison to a building dense area (Sarkisian, 2016).

2.3.1 Slenderness

When designing the building for sway the slenderness of the structure is of high importance. Especially at the initial phase of design, the slenderness ratio can interpret how active the structural system will be (Truby, 2014). The aspect ratio, or slenderness ratio (SR) defines the relation between height and minimum width of the structure, see Equation 2.1 and is desired to fall between 6/1 or 7/1. Yet it is not uncommon for high-rise buildings to obtain value of 8/1. In such cases, damping is of great importance for comfort reasons (Ascher, 2011). When the slenderness ratio exceeds 8/1 the dynamic performance of the structure will dominate the design. However, the slenderness ratio should only be used in the beginning of design for approximation. For final design, widths of the structural system will be the governing dimensions (Truby, 2014). Slenderness dimensions are illustrated in Figure 2.1.

$$SR = h/w \tag{2.1}$$

where

h	building height $[m]$
w	building minimum width $[m]$

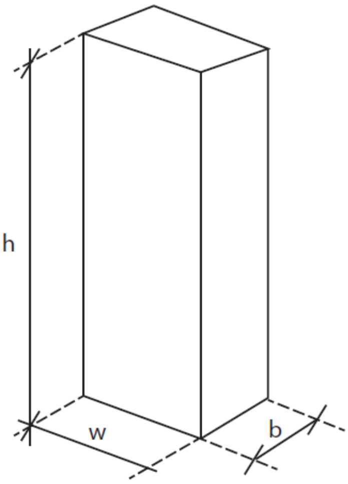


Figure 2.1: *Slenderness dimensions of a building.*

2.3.2 Drift

Wind acting on the building will give rise to a lateral displacement of the building called drift. The drift is generally defined as the difference in displacement at the very top of the structure but can also be measured locally between floors (Sarkisian, 2016). If the lateral displacement is too high for each floor, this could lead to poor consequences for the design of the facade where cladding and internal partitions could be affected (Truby, 2014).

The limitation for drift ratio (DR) in design are often set to 1/500 of the structure height when analyzing the elastic response and is calculated by Equation 2.2. Even though, some buildings with a drift of 1/400 has been constructed (Sarkisian, 2016). The inter-storey drift is approximately between 1/500 and 1/200 (Truby, 2014).

$$DR = \Delta/h \tag{2.2}$$

where

Δ	transverse deflection $[m]$
h	building height $[m]$

When the building is objected to wind that generates lateral forces, second order effects needs to be considered. The consequence of second order effects could raise the drift with as high as 10% (Sarkisian, 2016).

2.3.3 Building dynamics

When building tall, considerations regarding the buildings dynamic performance needs to be incorporated. Wind loads of different frequencies will affect the building in diverse ways. Along with the initial damping in construction materials, the structure will be impacted by its natural frequency. If the frequency generated by wind is approaching the natural frequency of the structure, there is a possibility of consequence regarding heavy loading and displacement (Truby, 2014).

Tall buildings in general conduct low natural frequencies which makes the structure more susceptible to be influenced by wind (Kwok, Hitchcock, & Burton, 2009). An approximation of a buildings fundamental frequency is generated by taking the number of floors divided by 10. To analyze the fundamental frequency period more closely, each mode needs to be observed (Sarkisian, 2016).

2.3.3.1 Accelerations

Heavy winds generates movements of the structure. Acceleration is generated by both along- and across-wind motions. For very slender buildings such as skyscrapers, across-wind accelerations tends to dominate the choice of design. The biggest impact affecting the acceleration is the shape of the building along with its height and location (Sarkisian, 2016).

To confirm the dominance of across-wind acceleration over along-wind, Equation 2.3 needs to be fulfilled. The different wind directions are illustrated in Figure 2.2.

$$\sqrt{W \cdot D}/h < 1/3 \tag{2.3}$$

where

W	cross-wind building plan dimension $[m]$
D	along-wind building plan dimension $[m]$
h	along-wind building plan dimension $[m]$

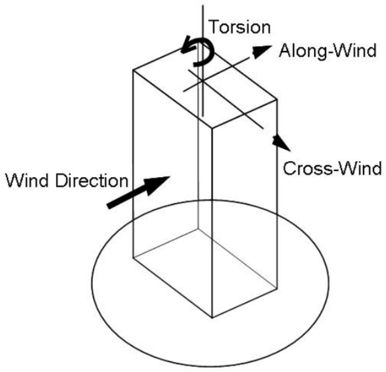


Figure 2.2: *Wind direction illustration.*

2.3.3.2 Human perception of motion

Maximum acceleration due to resonant response generated by wind load occur at the top of the building and can cause an unpleasant perception of motion at the uppermost floors. As humans, we can easy feel a discomfort when experiencing vibration responses in a building. However, the sensitivity to vibrations are different for everyone and depends on several aspects such as the height of the building, acoustics and visuals. From investigations regarding the perception of motions, majority of building users would feel the dynamic response of a building subjected to an acceleration greater than 0.1 m/s^2. Table presents an idea of human perception to different magnitudes of acceleration. (Abu-Zidan, Mendis, Gunawardena, Mohotti, & Fernando, 2022).

Table 2.1: Human perception to building acceleration (Abu-Zidan et al., 2022).

Level	Acceleration [m/s^2]	Human perception
1	< 0.05	No perception of motion
2	0.05 - 0.1	Sensitive users perceive motion
3	0.1 - 0.25	All users generally perceive motion
4	0.25 - 0.4	Office work impossible
5	0.4 - 0.5	Difficulties when walking naturally
6	0.5 - 0.6	Walking naturally is impracticable
7	0.6 - 0.7	Impossible to walk
8	> 0.85	Falling objects in the building

After repeatedly exposure to high vibrations, users of the building could experience several issues such as nausea, dizziness and headache (Kwok et al., 2009). Human sense of motion is investigated by looking at the buildings' behavior, stiffness and damping (Sarkisian, 2016). Since the users comfort of the building is a crucial consideration, accelerations and top deflections of a high-rise building are generally dominating the design choices (Abu-Zidan et al., 2022).

2.3.3.3 Damping

To regulate the acceleration movement of the building, damping is used. Damping of the structure is due to the material used and in relation to the return period for wind load. For concrete the damping ratio of a 50 year return period is 3% (Sarkisian, 2016).

Damping reduce the lateral acceleration. This is important for the users perception of building motion. To reduce the lateral acceleration, natural damping from use of material can be adjusted by changing the structure design, shape, adjusting aerodynamic response (Truby, 2014).

Damping correspond to magnitude, hence with stronger winds the displacements will be larger. Which damping value that should be chosen in design is not set, and the opinion differ from engineer to engineer. Total damping values between 1.5-3.0 % of critical have earlier been used for design (Truby, 2014). states that the upper limit is probably an over-estimation of damping value when looking at ultimate-limit state.

2.3.4 Building aeroynamics

Non-aerodynamic shapes as squares give rise to a wind phenomena called Vortex shedding. When wind meets the face of the building, the pressure difference at the different sides of the building creates vortices that that pressure the building perpendicular to the wind direction. Can be a problem at high frequencies. To minimize vortex shedding, the building can be placed so that the long facade is located parallel with the wind direction. There is also design measures to prevent this phenomena, different shapes such as rounded corners, twists and holes create a more aerodynamic structure (Ascher, 2011).

2.3.4.1 Vortex shedding

As previously mentioned, across-wind is governing for high and slender buildings. This wind motion perpendicular to the wind direction generates a phenomenon called vortex shedding (Sarkisian, 2016). Vortex shedding is a wind motion of flow that generates asymmetric pressure distribution acting on the across-wind facade of the building, illustrated in Figure 2.3. If the frequencies induced from vortex shedding occur spontaneously as the natural frequency, resonance would develop (Abu-Zidan et al., 2022). The effect of vortex shedding very much depends on the shape of the building. A circular shaped high-rise building will give rise to a higher cross-wind motion due to vortex shedding than a rectangular one. Further development that reduces vortex shedding is to integrate holes in the building structure. Implementing the vortex shedding frequency along with the building shape and wind speed one can define a so called strohaul number that characterizes the oscillating flow mechanism (Sarkisian, 2016).

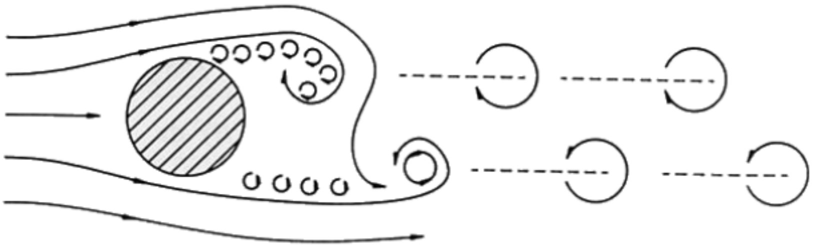


Figure 2.3: *Illustrative explanation of vortex formation.*

2.4 Building standards

Building standards translate the complex wind movement into simplified loads that can be more easily understood (Sarkisian, 2016). Building codes are today very conservative since they are based on general wind conditions and with a simplified building shape in consideration.Design approaches of wind based on current standards is established from historic climate data of the specific area (Sarkisian, 2016). The codes do not account for the aerodynamics of the building (Ascher, 2011). Neither do the majority of building standards of today not perpetrate to the common design criteria for drift (Sarkisian, 2016).

With respect for differences in different codes, the majority of building standards is applicable to low-rise buildings and often only generates wind loads in along-wind direction. This is a very conservative demarcation since other aspects from wind loads needs to be considered in design, such as crosswind and torsional loading (Irwin et al., 2013). In its nature, the motion of wind is dynamic and needs to be considered in the design of buildings. If a building acquire great stiffness and damping, the structure can be seen as static and subsequently designed in a simplified manner (Boverket, 1997).

2.4.1 EN-1991-1-4:2005

The Eurocode Eurpoean standards includes methods used for verification of a buildings' capacity and strength. For guide on determining wind loads in design of buildings, Eurocode EN 1991-1-4:2005 is used. The Eurocode states to issue methods for design of buildings and structures of both traditional and innovative character, however the standard is inadequate for more unusual conditions and the code refer to adopt specific investigations. The code also refers to further investigations of the design using wind tunnel tests or detailed numerical design methods (EN-1991-1-4, 2005).

The Eurocode is applicable to buildings not exceeding 200 m in height and gives guidance for natural wind loads against every observed surface of the building, including attached components (EN-1991-1-4, 2005). It is fundamental to specify predicted wind loads for each mean recurrence interval along with its critical issues. These wind loads along with partial factors accounting for the unpredictable events are achieved using existing building codes. Although building codes may be an adequate way to get a rough estimation of the design of high-rise buildings, the standards lack quality in including wind phenomena as crosswind excitation, aerodynamics and the effect of neighbouring buildings. Absence of these may not only lead to an inefficient design with higher costs but also malpractice of building motions that could affect the users of the building (Irwin et al., 2013).

The current Eurocode do not incorporate wind maps and is therefore meant to be adapted together with a national annex compatible to the wind climate in question. Presented method and equations in the Eurocode generate characteristic values of wind loads that apply directly to a service limit state analysis. To validate the building in ultimate limit state, additional partial factors would be needed (Hughes, 2014).

2.5 Wind tunnel test

Wind tunnel tests have been a part of the building industry since the rise of Word Trade Center in New York in the 1960s and have been developed ever since. To analyse the structural loads generated by wind and the buildings' response to these, wind tunnel test is carried out. Building code calculations are inadequate to specify the details needed for designing high-rise buildings and consequently wind tunnel testing is needed (Irwin et al., 2013). According to Sarkisian (2016), buildings higher than 40 floors should be checked in a wind tunnel test.

According to Irwin et al. (2013), a wind tunnel test would be favorable for a building that satisfy any of following criteria:

i. The height of the building h, exceeding 120 meters.

ii. The height of the building h, is greater than four times the average width $b_{average}$.

iii. Minimum natural frequency is smaller than 0.25 Hz.

iv. The abbreviated wind velocity $v/(f_1 b_{average})$ at ultimate limit state is larger than five.

Subsequently, the need of a wind tunnel test is depending on many aspects. Each building is different in its design with varying slenderness, location and structure, which all will determine the obligation of a wind specialist involvement (Irwin et al., 2013).

Although wind tunnel tests are frequently conducted in the design of high-rise buildings, the engineers implementing the results rarely have enough knowledge to understand it. If correct used, the wind loads gathered from wind specialists and wind tunnel test can minimize the cost of deign significantly. It is vital that the designer using these wind loads generated by wind specialist with a critical eye. Similarly to the possibility of different interpretation of results by the engineers, different wind consultants will generate diverse wind loads. Subsequently is is important as engineer to have knowledge in possible cause of uncertainties that could arise in a wind tunnel test (Irwin et al., 2013).

To be able to implement the wind loads generated by a wind tunnel test in an accurate manner, five steps should be conducted. This method is described by Alan G. Davenport as the "chain", see Figure 2.4. We have all heard of the expression "the chain is never stronger than its weakest link", and the same apply to a wind tunnel test implementation. The chain can be used both for an analytical method as well as for a wind tunnel test procedure. The most important requirement of using the chain is to remember to consider the two different chains separately. Similarly as the process will be weakened if one link of the chain is neglected, the result will not be accurate if one is to combine an analytical method together with results from a wind tunnel test. The two different approaches is only to be compared and evaluated against each other in the end (Irwin et al., 2013).

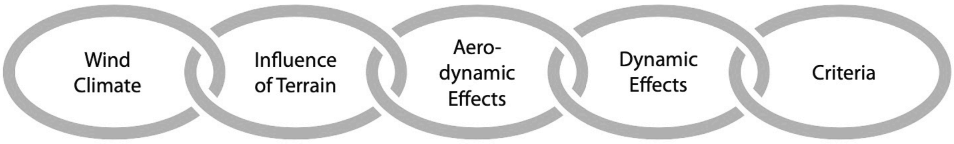


Figure 2.4: *The Alan G. Davenport chain explanation of wind loading.*

The first part of the chain is the wind climate where the direction and velocity of the wind at the specific location is required. Secondly, the influence of terrain is considered. This part of the chain includes the topography together with the surface roughness. Aerodynamic effects is the next link which incorporate the local effects from neighboring buildings. The dynamic effects of the chain speak for the building movement generated by wind, also incorporating aeroelastic consequences. The last link of the chain is represented by the importance of criteria, to estimate the building under the effect of wind (Irwin et al., 2013).

When choosing a building design, the structure needs to be checked against prevailing wind at its intended location. This is made by using wind-tunnel tests where the building design is iterated several times to find and efficient solution. In the wind-tunnel test, the strength and performance of the building in relation to the wind impact is analyzed (Ascher, 2011).

When creating a wind-tunnel test, all buildings within a radius of approximately 0.8 km is constructed in a miniature version, usually on a scale of 1:400. The model is put on a rotational platform, see Figure. The wind is simulated by fans with a speed up to 100 km/hour. To see the vortices formation, smoke is used (Ascher, 2011).

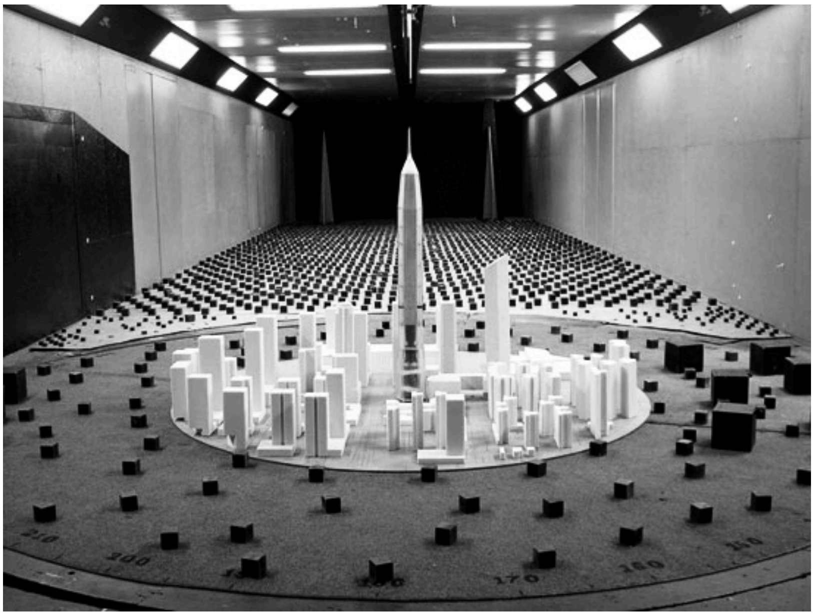


Figure 2.5: *Example of a wind tunnel test model.*

According to Sarkisian (2016), a wind-tunnel test should consist of following investigations:

I. Wind model simulation of the building in question together with surrounding construction within a radius if 0.8 km based on climate data.

II. Exterior walls modelled with pressure taps to provide for pressure measurements.

III. Wind analysis for pedestrian comfort.

IV. Force-balance simulation.

V. Aero-elastic simulation for height at minimum 300 m.

When creating a wind tunnel test, the specific topography is modelled with neighbouring buildings and relevant landscaping affecting the wind motion. Wind tunnel testing is of two different types, high frequency .. and cladding pressure design, where the first mentioned is the most common performed, generating wind loads needed to design the stabilizing system of the structure. The inferior one is where local pressure is analysed on different parts of the buildings' face in order to design the facade (Irwin et al., 2013). Both types of wind tunnel testing is presented in following chapter.

2.5.0.1 High-frequency force balance (HFFB) wind tunnel test

In a high-frequency force balance (HFFB) wind tunnel test, a rigid model of the structure is connected to a balance that rotates the building model for specific angles and wind speeds. The wind tunnel test measure reactions in the base that is used to generate corresponding wind loading on each floor as well as accelerations and deflections (Abu-Zidan et al., 2022).

2.5.0.2 High-frequency pressure integration (HFPI) wind tunnel test

To optimize the building facade against wind loads, the wind tunnel test high-frequency pressure integration (HFPI) is utilized. The model is covered in an amount of pressure taps, recommended at least one tap per 120 m^2 of the facade. This method observe instantaneous pressures at local building areas which are used to design customized facade elements (Abu-Zidan et al., 2022). Subsequently, the cost of design could be minimized considerable.

2.6 Computational Fluid Dynamics

Computational fluid dynamics (CFD) is a new developed method for analyze of the building response induced by wind. It is specifically adopted to predict ground-level wind velocity (EN-1991-1-4, 2005). This numerical method simulate fluid flow based on the fundamental equations of Navier-Stokes and is a favorable parametric tool that is cheaper and less time consuming compared to a wind tunnel test. Using CFD, more advanced architectural designs can be evaluated (Abu-Zidan et al., 2022). An advantage of using CFD is that the technique includes the effects of temperature which are not conducted in building codes. In comparison to a wind tunnel test, a CFD model measures infinitely amount of fluctuations on the building while a wind tunnel test is narrowed down to the amount of pressure taps mounted on the building model (Thordal, Bennetsen, & Koss, 2019). Unfortunately, this method is not yet competent enough to generate quantitative results of detail for wind loads (EN-1991-1-4, 2005). The reason for this uncertainty is that the CFD model demand the designer to set several parameters which subsequently is a risk due to possible miscalculations. However, CFD is a advantageous method to use in an early conceptual stage of design for high-rise buildings (Abu-Zidan et al., 2022).

3

Interview study

To incorporate current design approaches in the design of high-rise buildings, a few interviews with structural engineers specialised in tall buildings are conducted. Each interview and subsequently designer is connected to a specific reference building that puts the interview in a narrower context of high-rise buildings.

3.1 Interview guide

The interviews are performed following an interview guide accommodating general questions in the design of high-rise buildings. The interview guide contains the following aspects:

I. How is the design process in early stage for high-rise buildings carried out?
II. Is the assessment of design in early stages based on analytical calculations or FEM modelling?
III. What is the general time span of design in early stage?
IV. How is the workflow between architect and building designer?
V. What guidelines or industry practise are implemented in design of high-rise buildings?
VI. Are frequency and acceleration calculated to evaluate a structural systems' stiffness and stability? Are there other parameters observed in assessment of stability?
VII. What wind load models are used in early design of high-rise buildings?
VIII. What is absent or in need of development in current codes for early design of high-rise buildings?
IX. In development of this thesis and its research, what is relevant to examine and what is desired to gain in a new method of early deign of high-rise buildings?

To personalize and modify each interview to what is relevant for each specific designer, the questions are only used as a initial guidance rather than a strict scheme. Subsequently, gathered material from each interviews is summarized in relevant subheading and not each question from the interview guide.

3.2 Interview I - Andreas Lindelöf

The first interview is with Andreas Lindelöf who is a strcutural engineer at VBK Konsulterande ingenjörer AB in Gothenburg. Reference building used in this interview is Karlatornet at Lindholmen, Gothenburg, see Figure 3.1. After its final construction, Karlatornet will be a 246 meter high skyscraper located next to the Gothenburg harbor (serneke.se, 230515). The interview with Andreas was performed at VBK's office in Gothenburg.

Figure 3.1: *Final design of Karlatornet at Lindholmen, Gothenburg.*

3.2.1 Design of high-rise buildings

The design of high-rise buildings at VBK is a parameterized based work where a FEM model is created in an early stage. The FEM program used is ETABS with appurtenant script in Excel and Visual Basic. High-rise buildings are often repetitive in its design and hence easy to model. Andreas describes that the early models usually takes one or two days to create, depending on the amount of set-ups along with its mesh size. The model is a pretty rough estimation of the structural system with some simplifications, but still more detailed than general FEM models in the building industry. When analysing the model, different thicknesses and geometries of the core are observed. Several parameters are measured in these iterations; frequency, acceleration, total horizontal displacement at the top of the building and concrete weight. The results are visualised in tables and diagrams that makes it easy to quickly appoint a favorable solution.

The design of high-rise buildings is an iterative process together with the architects. Except for a design including outriggers, the columns in the buildings' structural system has no impact on the stability and are only positioned in the FEM model as a representation in early design. The floor layouts with their structural systems are chosen together with the architects. Several FEM models are analysed before taking an informative decision based on the resulting data set.

Based on experience, the wind loads considered in design are set pretty low, varying depending on project. Andreas describes that the wind loads usually are adjusted to terrain category IV based on the Eurocode European standards and that they tend to be even more offensive regarding the exposing factors. This method is not based on a strict scientific rule, but implemented to reduce the wind loads.

The natural frequency is one of the important measures that are observed in the initial models. The period time and how it changes is observed in each analysis. This parameter can be calculated straight from building standards but the calculations will be on a highly advanced level when including the piling design. An important feature when designing buildings here in Gothenburg is to incorporate the piling deign. The piles has an important role with their stiffness and are modelled together with the structural system in ETABS.

When designing a high-rise building, the stiffness is usually the parameter deciding the final design. But when we start to construct higher buildings, as the Karlatornet, exceeding the 200m mark, the acceleration will instead govern the design of the structural system, Adreas declares. The FEM model is created with predicted stiffness, for beams connecting the core and the bottom slab, stiffness values along with the degree of concrete cracking are based on experience.

When a structural system is decided upon, VBK is quick to incorporate a wind consult into the process. The wind consults will analyse the building proposal and generate an early assessment of the wind loads based on their knowledge and experience. These wind loads received from an early assessment are used in further design of the building until a wind tunnel test is performed.

Regarding the skyscraper Karlatornet, the exposing factors were not decreased as offensive as in other projects since the building is placed considerable close to the ocean where stronger winds occur compared to other buildings constructed in the city. The structural system of Karlatornet include outriggers that also were analysed in an early FEM model to decide upon at which height to put them and how it will affect the frequency. Before a final wind tunnel test were performed of Karlatornet, a workshop in wind tunnel design was conducted. In this workshop, which is a huge investment for the design of a high-rise building, several iterations of the building design were made and analysed. Different locations, angles, designs, heights etc. were tested in the workshop before a final solution were chosen.

A final wind tunnel test generate wind loads in horizontal x- and y-direction as well as the rotation. These loads are specified for each individual floor and then combined in 24 different load combinations. The wind loads are based on the current topography as well as future additional structures predicted to surround the building. The wind loads are observed in both service limit state and ultimate limit state. The natural frequency will vary depending on observed limit state. In ultimate limit state the concrete is cracked and subsequently weaker. At the same time the damping will be increased at ultimate limit state, compared to service limit state, which also affects the wind load. To sum up, the design is based on 96 different wind loads; current topography, future topography, SLS and ULS combined in 24 load combinations.

Besides the most common wind tunnel test HFFB, Karlatornet were tested for cladding pressure, HFPI, where the local pressure on the facade is measured by adding several taps on the model. Andreas explains that by a rough estimation, the facade cover an equal large investment cost as the core itself. Subsequently it is possible to lower the cost considerably by observing the local wind pressure at the facade and design accordingly. In Karlatornet, the local wind pressure is greater at the twist as well as at building corners and top, depending on building side observed. Adapting the facade depending on local wind pressure saved the project a large amount of investment cost.

In the project of Karlatornet, there have been a detailed consideration regarding different directions of wind velocity. The wind tunnel test is based on directional dependent wind data where the wind load is higher from southwest to northwest. The resulting wind loads for each direction are displayed in a so called wind rose and the building is designed accordingly.

Gathered results from wind tunnel tests shows significant decrease in wind loads compared to prediction based on building standards. Andreas speaks of how it is a complex task to conclude any acquaintances between a wind tunnel test and wind loads by building codes to define any guidelines to use in future projects.

3.2.2 A new method for design

The cost of a wind consulting early assessment is rather low, around 50 000 SEK. It is a rather cheap investment in relation to its favorable outcome, Andreas express. Of this reason, a new method for early design may not be worth its time. Wind consults are considerable competent and exceptional knowledge would be required to reach their results for predicting wind loads in an early stage of high-rise building design. However, further guidelines for the design of high-rise buildings that would bring the engineer even closer to the "true" solution would certainly strengthen the design concept, especially in an early stage.

Some buildings are of complex manners, for example Region City here in Gothenburg. The buildings are really close to each other which will create different wind movement and pressure on the structures. Additionally, the buildings share foundation which will challenge the design further since the wind loads are taken care of at the foundation. Even though one can predict that heavy wind gusts will not occur at the same time and with the same direction on the buildings, it is still a complicated task to perform without a wind consult in charge. In a case like this, the design will be based on different terrain categories for various directions facing the buildings' all sides as well as a complex wind movement between the buildings. Andreas explains that this kind of analysis is way beyond engineering practise and needs to be addressed by a wind specialist.

One disadvantage of including a wind consult in an early phase of the project is the requirement of making a suitable choice of wind consult company to work with. Even though the main idea with the actual wind tunnel test is to confirm and refine the results from the early assessment, the project is not closed to working with the same wind consultant as in the early stage. The choice of wind design team is based on degree of proficient but also important, a matter of investment cost. Subsequently, changing company could be misleading when comparing wind tunnel tests to an early assessment of wind loads.

An interesting observation Andreas proposed was to bring forth more clear guidelines regarding the total displacement at the top of the building. Today, there are only recommendations exsisting without any context to relevance for the design of high-rise buildings in particular. Since the slenderness of the building have an substantial affect on the predicted wind load, a more slender design will generate higher wind loads, the drift is an vital feature in design. In reality the wind loads will be lower at parts of the building where the stiffness is higher, but in design by building standards, these dynamic wind effects is kept constant. The total horizontal deformation at the top of the building is measured against different ratios in the early FEM analysis and often chosen to match a recommendation of h/500. If one were to be more comfortable of choosing a higher displacement, like h/400, this would generate a whole other building design, Andreas expresses. Of course this kind of guidelines must include the consequences of accepting a higher displacement value such as shear deformations that affects the design of the facade.

It would be interesting to gather results from wind tunnel tests for buildings in different ranges of height, for example 50-100m, 100-150m, 150-200m etc and compare relevant reduction factors with the Eurocode building standards, Andreas declares. Connecting building height with relevant reduction factors would be a profitable development of current guidelines in design. To bring even more knowledge in a method like this, the frequency could be an additional parameter to observe.

3.3 Interview II - Marco Binfaré

The second interview is with Marco Binfaré who is a building designer at WSP in Sundsvall. Reference buildings for this interview is Norra Tornen, located in Vasastaden, Stockholm, see Figure 3.2. Norra Tornen consists of two buildings, Helix of 111 meters in height and Innovationen of 120 meters in height. The buildings' primary use is residential. The interview with Marco was performed via a Teams meeting.

Figure 3.2: *Norra tornen in Vasastaden, Stockholm.*

3.3.1 Design of high-rise buildings

The design of high-rise buildings at WSP is an iterative process together with the architects for a certain project. For each architectural proposal, a simple analyze of the buildings' stability is conducted. In this investigation the buildings' slenderness is observed in relation to the location of the core. The hand-made calculations are made in a rough and general matter, sometimes random detail checks are made, but no detailed calculations is conducted in this early stage. When a final layout of the structural system is chosen together with the architects, a more detailed analysis of the building is conducted using FEM modelling. The whole development of high-rise building design is an iterative process, where changes can be necessary even in a later stage. Marco explains how the iteration of FEM models often results in a variety of approximately 10 models with different properties. The most common FEM-program used at WSP is FEM-Design but Marco also talks about the advantage of using the FEM program ETABS particularly for the design of high-rise buildings.

In the beginning of the design process, wind loads are conducted using the Eurocode European standards since they give more conservative and higher values. The load cases are observed in both service limit state as well as ultimate limit state. One of the most important parameter for the design of high-rise buildings is the natural frequency, Marco explains. A desirable value of the natural frequency is approximately 0.35 Hz. An important matter is to avoid to get the torsion mode as first mode, favorable would be to get the x- and y-component as first two modes and subsequently torsion as last and third mode. This is due to the difficulty to control the torsion of the building. Subsequently looking at the natural frequency is a useful tool to analyze the buildings' design. This will tell if the structural system is in need of further core walls for stability. Demands of the natural frequency is missing in today's building codes, there is only a few recommendations to be found. The natural frequency of the building will affect the acceleration at the top of the building and is therefore an vital parameter to observe. Demands on acceleration for high-rise buildings are also neglected in today's standards, although ISO standards do have some inputs, there is a lack of specified demands. The acceleration is observed after a chosen layout is set and will determine whether the building will be suitable for residential or office use. Another feature to observe in the design is the slenderness of the building. Marco explains that there is a lot of studies about this conducted in the US, but Eurocode lack the same level of guidelines. Often in design, the choice of total horizontal deflection at the top of the building is based on experience from previous projects.

In later design of high-rise buildings, a wind tunnel test is performed. There is no early assessment conducted before the actual wind tunnel test. If there is a need to investigate a special type of facade that is not covered in the Eurocode, a CFD analysis can be implemented before the wind tunnel test. In general, one wind tunnel test is performed per project. Although, if there are some uncertainties of the resulting wind loads, for exapmle if there is a significant decrease of wind loads compared to the analysis by building standrads, a second opinion could be conducted. The resulting wind loads from the wind tunnel test are approximately 15-20% lower than the wind loads conducted by building standards. Depending on the type of core used in design it will induce different consequences. For a core made of steel, a 20% decrease in wind loads will have a huge impact, while the same reduction of wind loads for a concrete core may not give a significant difference. Although, the reduction of wind loads will nevertheless save material usage and investment cost together with possible improvements of frequency and acceleration.

From the wind tunnel test, the dominant wind directions are displayed by a wind rose. It demonstrate the most critical load case of the side of the building facing the dominant wind direction, which is dependent on the unique design of the building. With the dominant wind direction in mind, 40-50 different load cases in 24 load combinations are conducted and combined to find the most critical case.

For the project of Norra Tornen, a cladding wind tunnel test, HFPI, was performed since the buildings' facade are of a special design. In this analysis there is an detailed observation of attachments and fine technicalities. The cladding analysis that generates the local pressures on the buildings' facade resulted in an partition of segments that could be designed with different wind loads. By implementing this analysis, the structure reaches its optimization. Although, one do not want do optimize to the maximum, Marco explains, especially for a project like this where the core is made out of prefabricated concrete elements due to possible mounting difficulties.

A challenge in the project of Norra Tornen, Marco explains, was the non-centred core. The core was instead located markedly to the right, which gave rise to a rotational difficulty. The two buildings had some disparity to each other. The Helix building was in need of an additional core wall in the outer part of the floor layout since it is more slender than the other building. This was a case where the engineer got involved later than the architects and the design resulted in a need of additional stability. Favorably, the engineers should be involved in the early phase to minimize these kinds of late changes in design. Furthermore, the project Norra Tornen brought challenges with its prefabricated core which gave rise to problematic joints and an additional demands in transferring the loads through the structure to gain stability. The buildings are designed with overhangs of the facade in three different directions with each a length of five meters each whereupon the structure rise even 10 floors higher. Marco describe the project as a fun but long process.

3.3.2 A new method for design

There is a big lack of guidelines and methods in today's European building codes regarding the design of high-rise buildings. There is a lot of literature to gain from the US, Canada and UK regarding high-rise buildings. Marco explains that the reason for this inadequacy in the European standards could be of the simple reason that there has not been a need for designing high-rise buildings, until now. Especially here in Sweden, there is not a lot of tall buildings and especially not close to the height of international skyscrapers.

According to Marco, guidelines or a manual for designing high-rise buildings together with a detailed literature collection would be desirable. To develop a method in early design to assess if the core of the building is adequate or not, would benefit the design process. A method or guidelines of this kind could include principally key numbers to follow and include an amount of three or four bullet points to be verified.

Another important aspect for the design of high-rise buildings that Marco declare is to consider the consequence of carbon dioxide. This kinds of tall buildings that could amount approximately 400 concrete walls with high concentration of cement are not to be ignored. This is a component in the design of high-rise buildings that can and should be optimized for the environments' sake. It should be a part of the early design where depending on the span and slab, an analyse of the amount of carbon dioxide emission is conducted. The construction of tall buildings has only begun and there will be more, Marco express. There is an opportunity to optimize the use of concrete in high-rise buildings, especially when observing the slabs. At WSP the consideration of carbon dioxide emissions in an early stage has not been conducted in the past. There is a current development of method regarding net zero at WSP in the UK. In this method, one can adjust the amount of floors, material, spans and variation of slab to observe the expected carbon dioxide emission. There is a lot to gain by this kind of analysis, Marco explains.

3.4 Interview III - Gustav Söderlund

This interview is performed with Gustav Söderlund who is a building designer at PE Teknik & Arkitektur in Gothenburg. Gustav is one of the designers behind Hotel Draken which is a high-rise building of 100 meters in height, located in Gothenburg, see Figure 3.3. The building which is a hotel was constructed between the years 2020 to 2023. The interview with Gustav was performed at PE's office in Gothenburg.

Figure 3.3: *Draken Live at Järntorget, Gothenburg.*

3.4.1 Design of high-rise buildings

At PE Teknik & Arkitektur, the design of high-rise buildings is dominated by implementing FE modelling in an early stage. Analysing the building by using FEM modelling is an easy and quick way of design compared to using hand calculations, Gustav explains. The FEM program that Gustav prefers for high-rise buildings is ETABS. ETABS is a program with more options compared to other FEM programs such as boundary conditions and several functions favorable for high-rise buildings. It is not that important to go into details in an early stage of modelling, it is more about analysing approximately how large the core needs to be. In the beginning the model is a general layout that is copied and added on each floor of the building to get an overview of the structural systems' response.

In the early stages of design, the location of the columns is not crucial for the stability. The main task is to find a structural system that falls between the core walls that has been conducted together with an architectural idea of layout. It is an iterative process between the architectural design, demands on accessibility and technical spaces and the structural stability. The structural system is adjusted repeatedly by either increasing the thickness of the core walls or by modifying the geometry. In some special cases, there could also be a need of integrating an outrigger in design, especially for higher and more slender buildings.

The wind loads are included in the model by using Eurocode European standards. Gustav disclose that when using the Eurocode standards there can be a huge difference to the wind loads gained from a wind tunnel test. In an early project stage, there is an early assessment conducted by wind specialists. After adjusting the design to match their recommendations, an actual wind tunnel test is performed. Observing the results from the wind tunnel test, the wind loads in the project Draken were decreased with approximately 20% compared to the loads generated from calculations by building standards. Additionally, there is a dynamic factor that needs to be included in the design of high-rise buildings. This dynamic factor tends to be of a high value when generated by hand calculations, while the wind experts can decrease this value based on their experience of the buildings' structure, location and the known behaviour of the wind. At PE, Gustav explains that they tend to lower the value a bit in the beginning, but always lean against the word of the hired wind specialists to get a professional opinion. At PE Teknik & Arkitektur the wind tunnel test used for projects of high-rise buildings has been the high-frequency force balance (HFFB) kind of wind tunnel test.

Except for the wind loads itself, accelerations are a crucial issue for high-rise buildings. The acceleration needs to be limited to a certain level suitable for the buildings' intended use. In this matter, Gustav points out that we as engineers are fixed to the current building standards, while the wind consults possess greater experience regarding building dynamics. The acceleration is depending on the buildings' mass and subsequently its inertia. For a lightweight building, accelerations at the top could be a problem. Gustav explains that his interpretation of acceleration for buildings is that it starts to be a problem when exceeding 100m in height and probably escalates from there.

When looking at the acceleration demands, these differ between the Eurocode European standards and the EKS, where the values retrieved from the Eurocode is lower. Even though this is a one parameter variety, the result will differ with approximately 30%. Gustav then describes that is has been a challenge to compare these accelerations generated by building standards to the accelerations gained from the wind consults. Looking at the results generated by wind specialists, the damping has been set to 15% which is not the case when following current building standards.

In the project Draken, Gustav talks about the challenge in integrating acceleration together with piling design. When the piles are deforming, it creates a weakness of the structural system. Normally, some piles are placed diagonally in the ground to be able to take horizontal loading. In this project, the piles were vertically placed, which created another demanding task. The limitation of acceleration chosen for the project Draken was a value between the recommendations of a residential and office used building.

Although service limit state dominates the design, ultimate limit state is investigated for the matter of tension. If any adjustment is needed in regards to the ultimate limit state, it is normally solved by changing to a better concrete quality or adapting the reinforcement. Issues in design for ultimate limit state is not a geometrical concern as it is for the service limit state verification. As of now, the concrete weight is not being analyzed in projects at PE Teknik & Arkitektur, Gustav declares. Regarding the total deflection at the top of the building, Gustav explains that the most common choice is by following the recommendation of h/500.

Another challenge in the project of Draken was the prefabricated elements and how to handle the joints and connections that occur. The idea were to make the core act as a whole homogeneous structure regardless its segmented division. To make this happen it was important that the connections were stiff enough. This is an issue that do not occur when constructing in-situ, where the task is more or less to only reinforce correctly.

3.4.2 A new method for design

Looking at the 10% difference between wind loads gained from calculations based on building standards and wind loads received from wind tunnel tests, there is definitely room for development of guidelines for a early design assessment of high-rise buildings. But at the same time, wind is a complex phenomena, Gustav declares, and require a lot of information to analyse properly.

It would be useful to create a method to secure demands on accelerations for high-rise buildings, Gustav express. Both Eurocode European standards and the EKS have methods for calculating the acceleration where the Eurocode measurements usually generate a lower value of acceleration. When following the EKS guide, a return period of five years is implemented while the return period in Eurocode is one year. Gustav explains that the factors received for different types of return periods are different depending on source. As Gustav further mentions, the accelerations received from the wind consultants are sometimes difficult to match against the values gained from calculations by norm. There is no easy way to find a common approach to use in future projects before including a wind specialist.

Final words about the importance of acceleration, Gustav talks about how it is crucial to solve this issue early. There is no way to improve the concrete quality for increasing the bending stiffness. In today's building industry there are more and more common to construct timber buildings with lower self weight compared to concrete or steel structures. It will then be even more vital to handle the accelerations since it is mainly the mass and inertia that decreases the acceleration. When observing concrete buildings, a structure of 100 meters usually do not endure problems regarding accelerations. The difficulty starts beyond this, when there is a need of outriggers and specialized wind analyses.

4

Case study

A case study is implemented in this thesis to support the development of guidelines for design of high-rise buildings in an early stage. To conduct a comparison between building standards and wind tunnel tests, this thesis has been provided with three different reference buildings with appurtenant data. With respect to the building designers, the three different buildings are presented anonymously with titles; Building A-C. Due to this anonymous commitment, limited information of the buildings are presented in this chapter. The data provided focus on the buildings' structural systems along with wind properties and wind tunnel testing results. The buildings analysed are of different heights and configuration to bring a variety of analysis.

4.1 Building A

Building A is a 111 meters high building with 34 floors. The buildings length is decreased with its height where the largest length is of 48 meters while the smallest section is only 22 meters. Architectural drawings of Building A is shown in Figure 4.1 to 4.3. The building are mainly of residential usage and is located where a fundamental wind velocity of 24 m/s is assumed. Structural properties of Building A along with wind properties are presented in Table 4.1 and 4.2.

Table 4.1: *Structural properties for Building A.*

Building height	111 m
Building minimum width	15 m
Building length	22-48 m
Slenderness ratio	7.4/1
Across-wind dominance	$0.164 < 1/3$
Shear wall thickness	400 mm
Slab thickness	200 mm

Table 4.2: *Wind load properties for Building A.*

Terrain category	II
Basic wind velocity	24 m/s
Peak wind velocity pressure	1.37 kN/m^2
Equivalent mass m_e	193332 kg

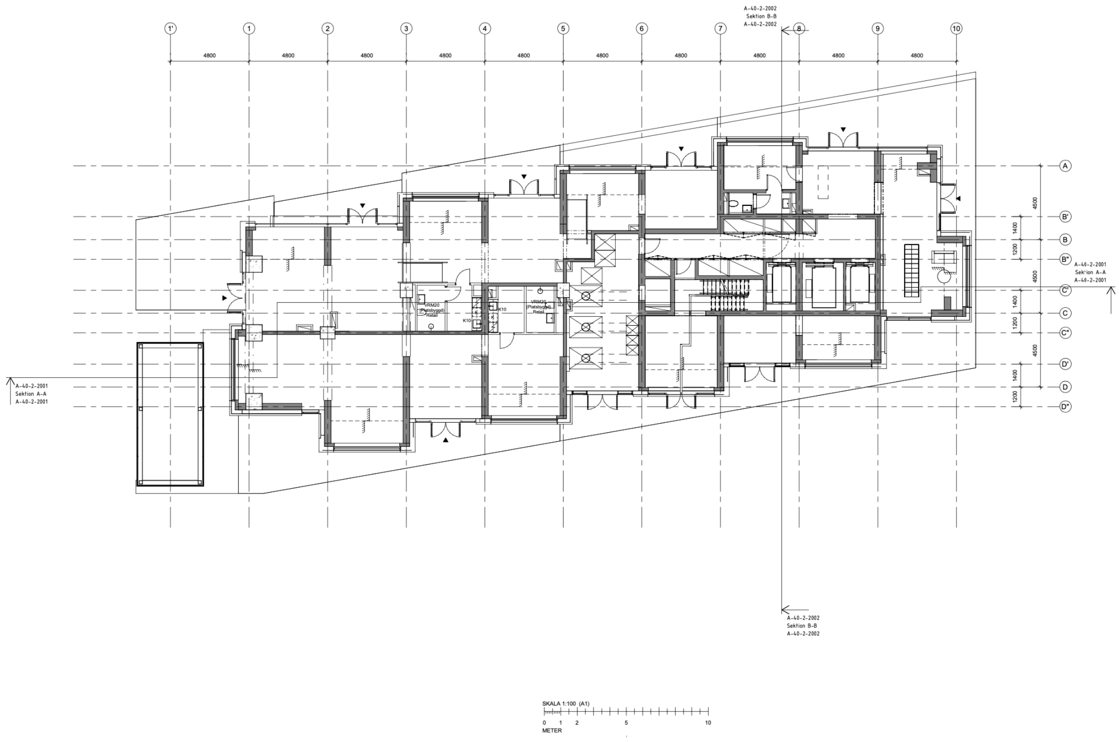


Figure 4.1: *Entrance floor plan of Building A.*

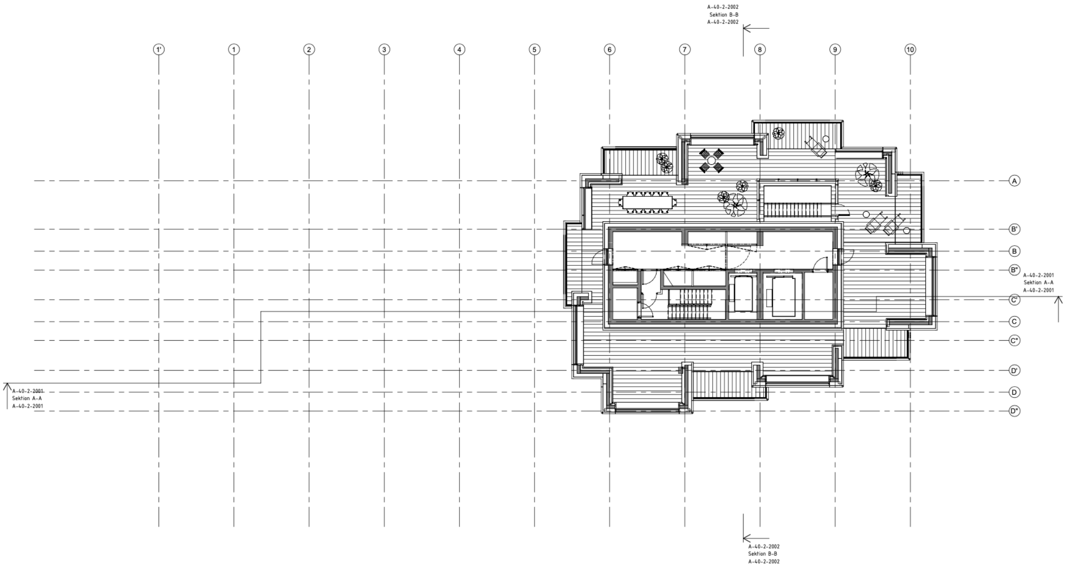


Figure 4.2: *Rooftop floor plan of Building A.*

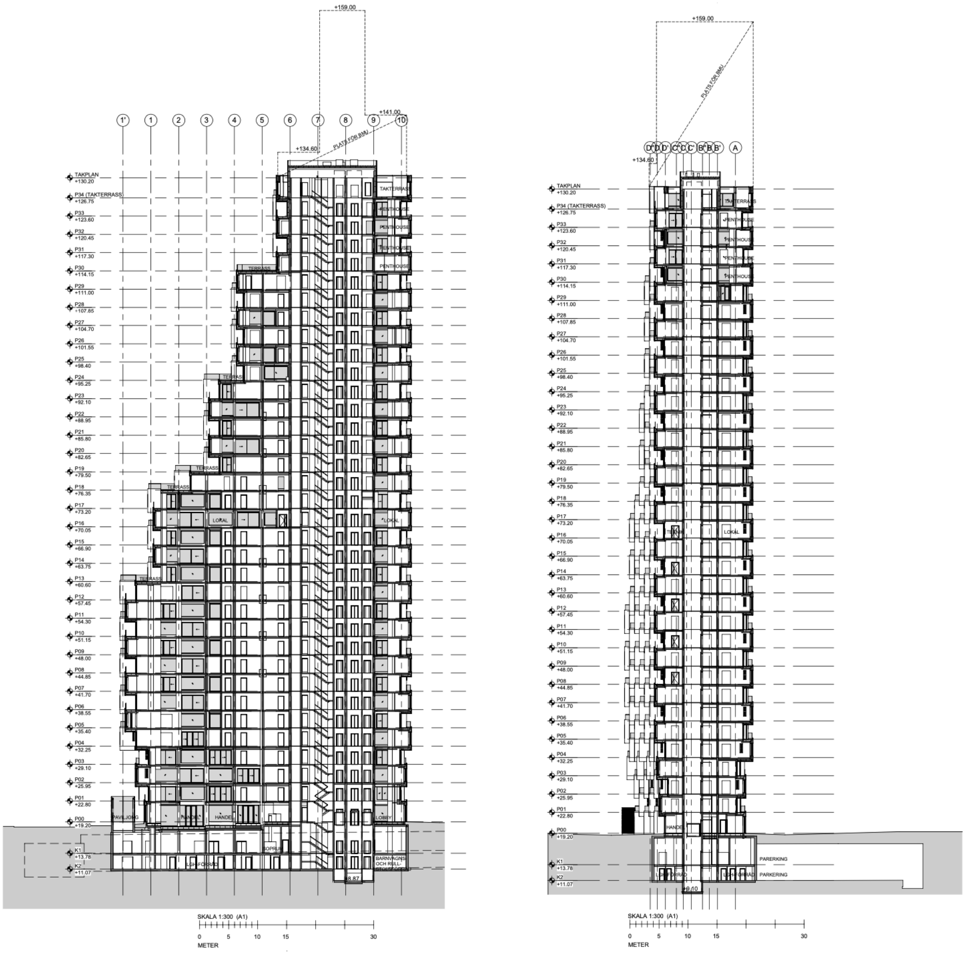


Figure 4.3: *Section A-A respectively section B-B of Building A.*

To design Building A against strong winds and optimize its structural system, a wind tunnel test was performed for both structural and cladding design. Gust-induced vibrations as well as along- and across-wind accelerations were investigated. Natural frequencies for mode 1 to 3 from wind tunnel testing are presented in Table 4.3.

The wind tunnel test was conducted with 36 different wind directions in steps of 10 degrees (0-350). The building was modelled both as solitary configuration as well as with close buildings within a radius of 250 m. The model in the wind tunnel test was on a scale model of 1:250 with approximately 3200 pressure taps covering the facades. In this thesis, the dominant wind direction is analysed and subsequently compared to wind tunnel test results on the same side.

Table 4.3: *Frequency for mode 1 to 3 for Building A.*

Mode	Frequency
Mode 1	0.319 Hz
Mode 2	0.470 Hz
Mode 3	0.550 Hz

4.2 Building B

Building B is a 94.7 meters high circular shaped building with 29 floors. The building is of residential usage and located in a terrain that is characterized as type II. The building is designed with a 24 cornered facade, which in building code regulations is designed as a circular structure. The circular geometry induce wind issues such as increased formation of vortex shedding.

Further on, the fundamental wind velocity is assumed to be 24 m/s. Structural- and wind load properties of Building B is presented in Table 4.4 and 4.5 followed by architectural drawings in Figure 4.4 to 4.6.

Table 4.4: *Structural properties Building B.*

Building height	94,7 m
Building minimum width	25 m
Building length	51 m
Slenderness ratio	3.8/1
Along-wind dominance	$0.54 > 1/3$
Shear wall thickness	400 mm
Slab thickness	200 mm

Table 4.5: *Wind load properties Building B.*

Terrain category	II
Basic wind velocity	24 m/s
Peak wind velocity pressure	1.33 kN/m^2
Equivalent mass m_e	246000 kg

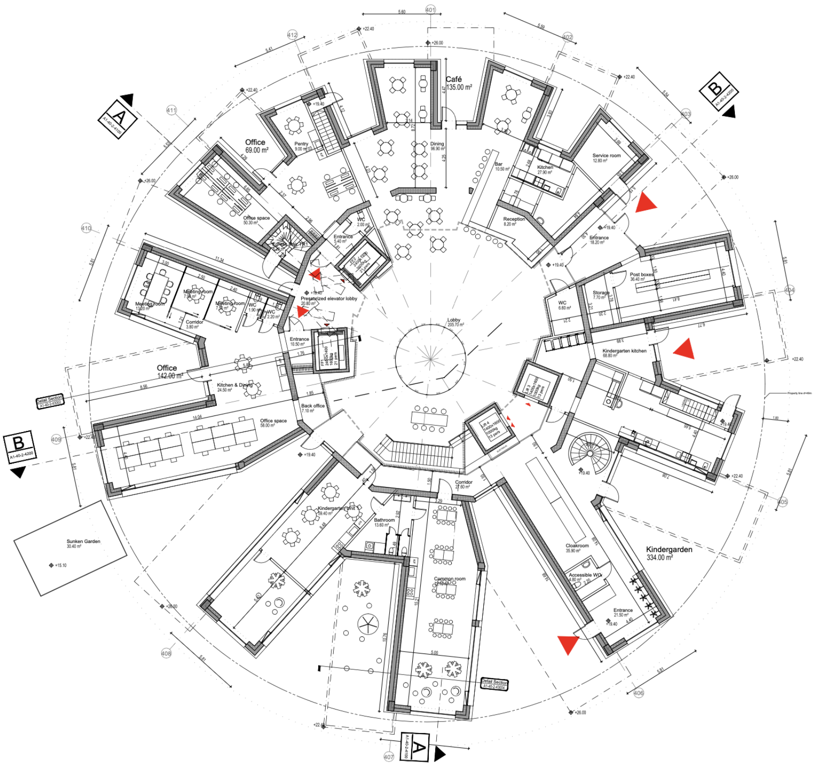


Figure 4.4: *Entrance floor plan of Building B.*

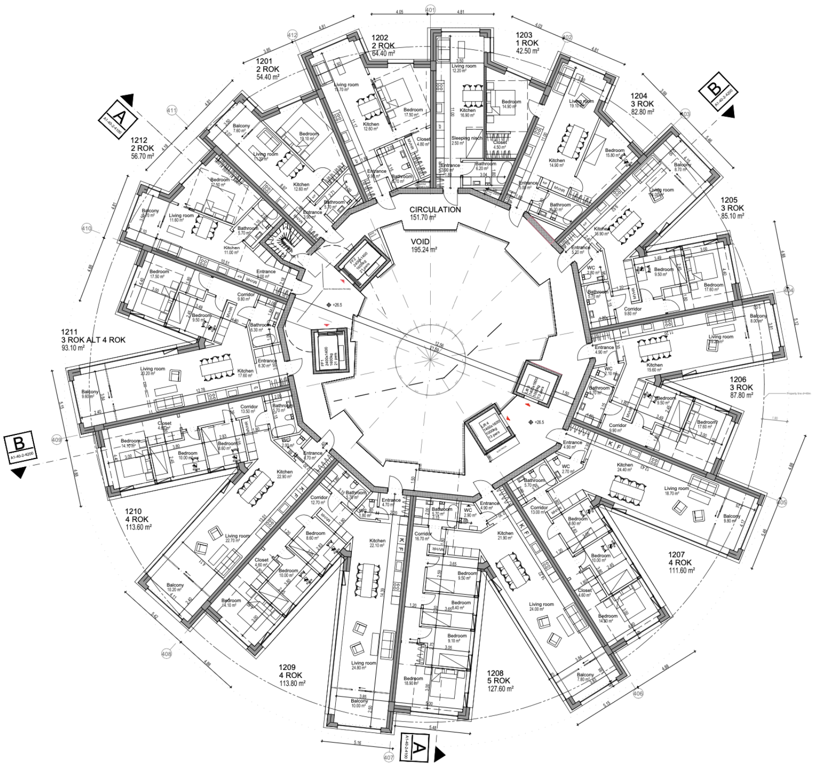


Figure 4.5: *Typical apartment floor plan of Building B.*

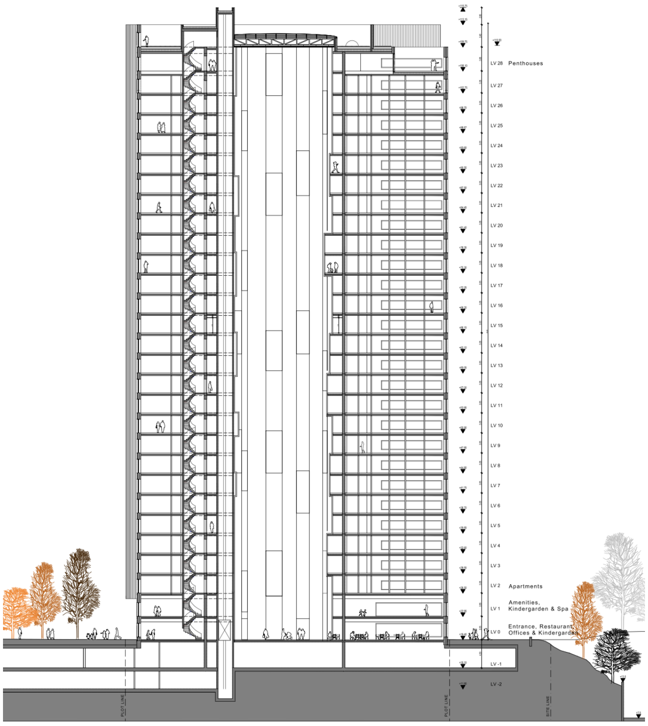


Figure 4.6: *Section A-A of Building B.*

The wind tunnel test for this building consisted of 36 different wind directions in steps of 10 degrees (0-350 degrees). There were two different variety of test where one is with the building alone and the other one with neighboring buildings and relevant structures within a radius of 300 meters. The model is of scale 1:300 with approximately 250 pressure taps mounted on the building model. For this building, both a structural and cladding design wind tunnel test were conducted. The structure were investigated against gust-induced vibrations as well as wind induced across- and along accelerations. Natural frequencies for mode 1 to 3 from wind tunnel testing are presented in Table 4.6.

The wind loads conducted from the wind tunnel test do not include any safety factors. Further, the wind tunnel test result do not incorporate the construction phase of the building and how this affect the need for building strength. Neither does this wind tunnel test cover the impact of future neighboring structures close to the building and possible consequences of this.

Table 4.6: *Frequency for mode 1 to 3 for Building B.*

Mode	Frequency
Mode 1	0.327 Hz
Mode 2	0.565 Hz
Mode 3	0.583 Hz

4.3 Building C

Building C is a 70 meters high building with 20 floors. The building is of mainly residential usage with some public areas in the bottom floor. The building is located with an assumed terrain type II and fundamental wind velocity of 24 m/s. The buildings' stabilizing system consists of a core around the stairway and elevators along with several concrete core walls and steel bracing crosses.

Table 4.7: *Structural properties Building C.*

Building height	70 m
Building minimum width	34 m
Building length	21 m
Slenderness ratio	3.3/1
Along-wind dominance	$0.38 > 1/3$
Shear wall thickness	400 mm
Slab thickness	220 mm

Table 4.8: *Wind load properties Building C.*

Terrain category	II
Basic wind velocity	24 m/s
Peak wind velocity pressure	1.25 kN/m^2
Equivalent mass m_e	239457 kg

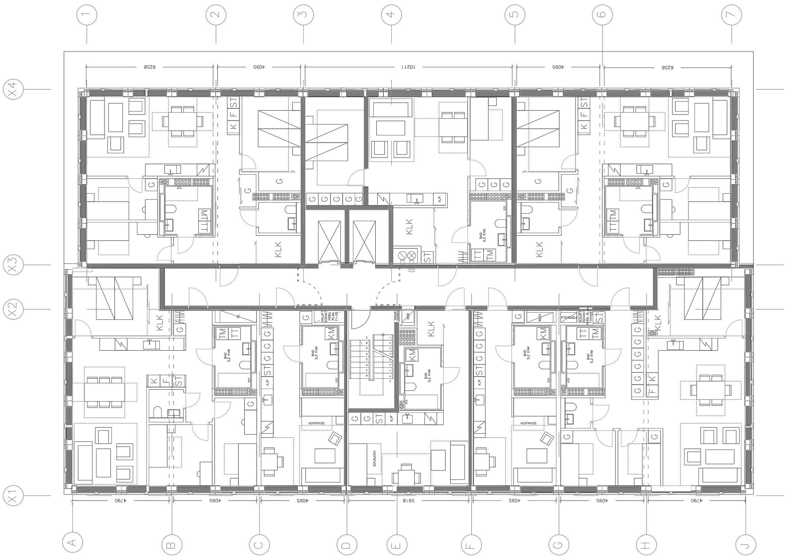


Figure 4.7: *Typical apartment floor plan of Building C.*

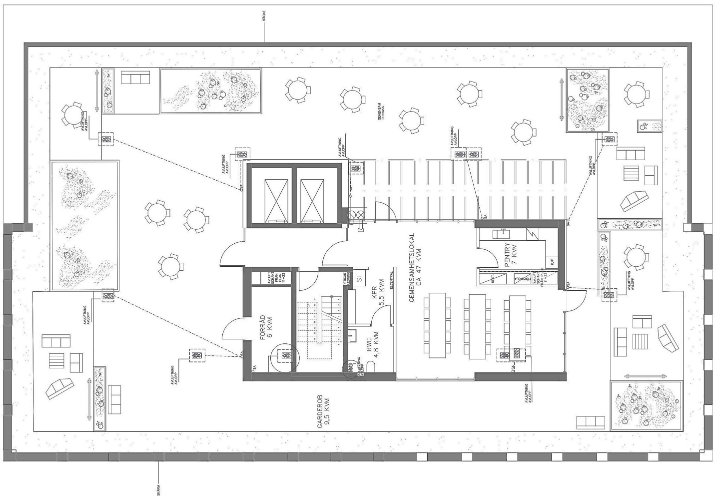


Figure 4.8: *Roof top floor plan of Building C.*

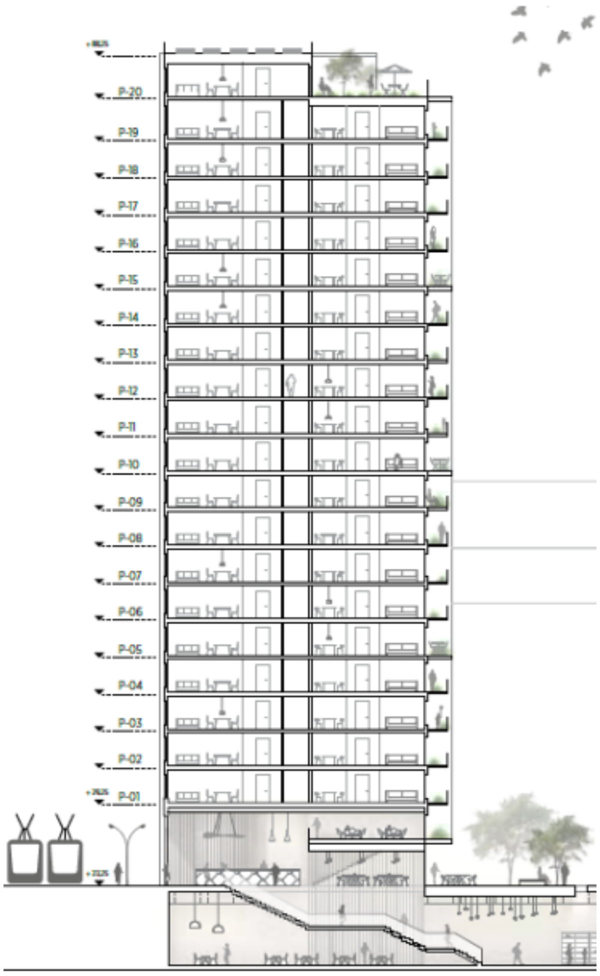


Figure 4.9: *Section A-A of Building C.*

The wind tunnel test consisted of a structural as well as a cladding analysis with 256 pressure taps. The wind tunnel model was tested in 36 wind directions with 10 degrees apart. Further on, a pedestrian comfort analysis was performed at relevant pedestrian areas such as the balconies. Natural frequencies for mode 1 to 3 from wind tunnel testing are presented in Table 4.9.

Table 4.9: *Frequency for mode 1 to 3 for Building C.*

Mode	Frequency
Mode 1	0.615 Hz
Mode 2	0.846 Hz
Mode 3	0.988 Hz

4.4 Structural system simplification

Each reference building are to analysed with analytical building code methods as well as a FEM analysis. The aim is to develop guidelines for the design on high-rise buildings in an early conceptual phase of design. Subsequently, the structural systems of the buildings are simplified to minimize time consuming work and provide a quick assessment of stiffness. The core walls are designed with few openings. In reality, the core walls consists of a lot of irregularities. The slabs are designed with a simple geometry related to each building configuration. Modified structural systems for Building A - Building c is presented in Figure 4.10 to 4.12.

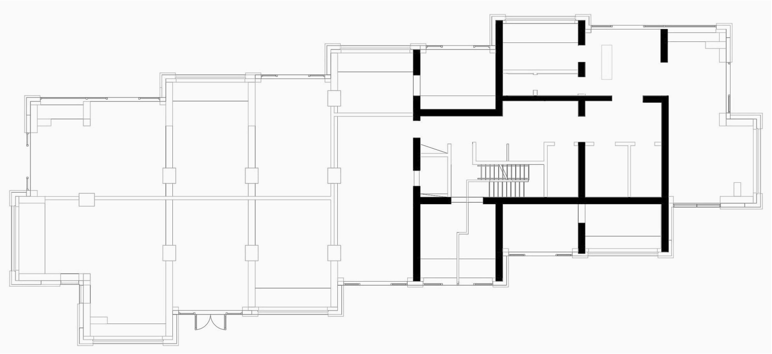


Figure 4.10: *Structural system of Building A with highlighted core walls.*

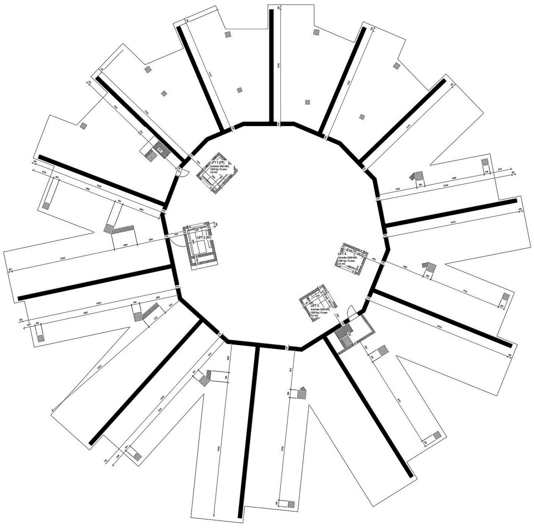


Figure 4.11: *Structural system of Building B with highlighted core walls.*

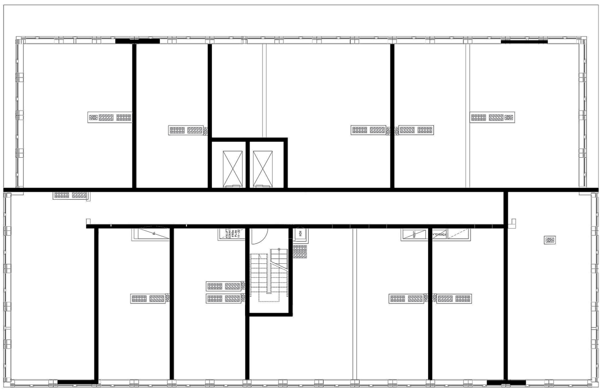


Figure 4.12: *Structural system of Building C with highlighted core walls.*

5

Analytical analysis

The goal with this thesis are to conduct a simple and easy method to use in early assessment of a high-rise building based on existing building standards. This chapter is an presentation of wind load design together with along-wind acceleration by Eurocode European standard edition (EN-1991-1-4, 2005). Some minor adjustments has been made to ease the understanding of the equations in a pedagogic way. The methods presented in the Eurocode is supported by new developed equations by the building standard recommendations EKS 12 (BFS:2022:4, 2022). To evaluate the vibration response of the building the International Organization for Standardization (ISO:10137, 2008) is used. A theoretical presentation of methods and equations used to evaluate the buildings presented in the case study.

5.1 Wind load

The building standard EN-1991-1-4:2005 is a part of the Eurocode European standards. In this building code, the wind load is considered as a variable and bound load. By following this method, a characteristic wind load assessment is obtained.

The basic wind velocity is obtained by implementing Equation 5.1 and is modified with regards to the direction of the wind as well as season of construction. The recommended value for c_{dir} and c_{season} is 1.0.

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} \tag{5.1}$$

where

c_{dir}	directional factor $[-]$
c_{season}	season factor $[-]$
$v_{b,0}$	fundamental basic wind velocity $[m/s]$

The fundamental basic wind velocity $v_{b,0}$ describes the wind climate for each specific location and are obtained from Figure 5.1. The velocity is defined as the average wind speed under a 10 minutes period on height 10 meters in an open terrain with small obstacles.

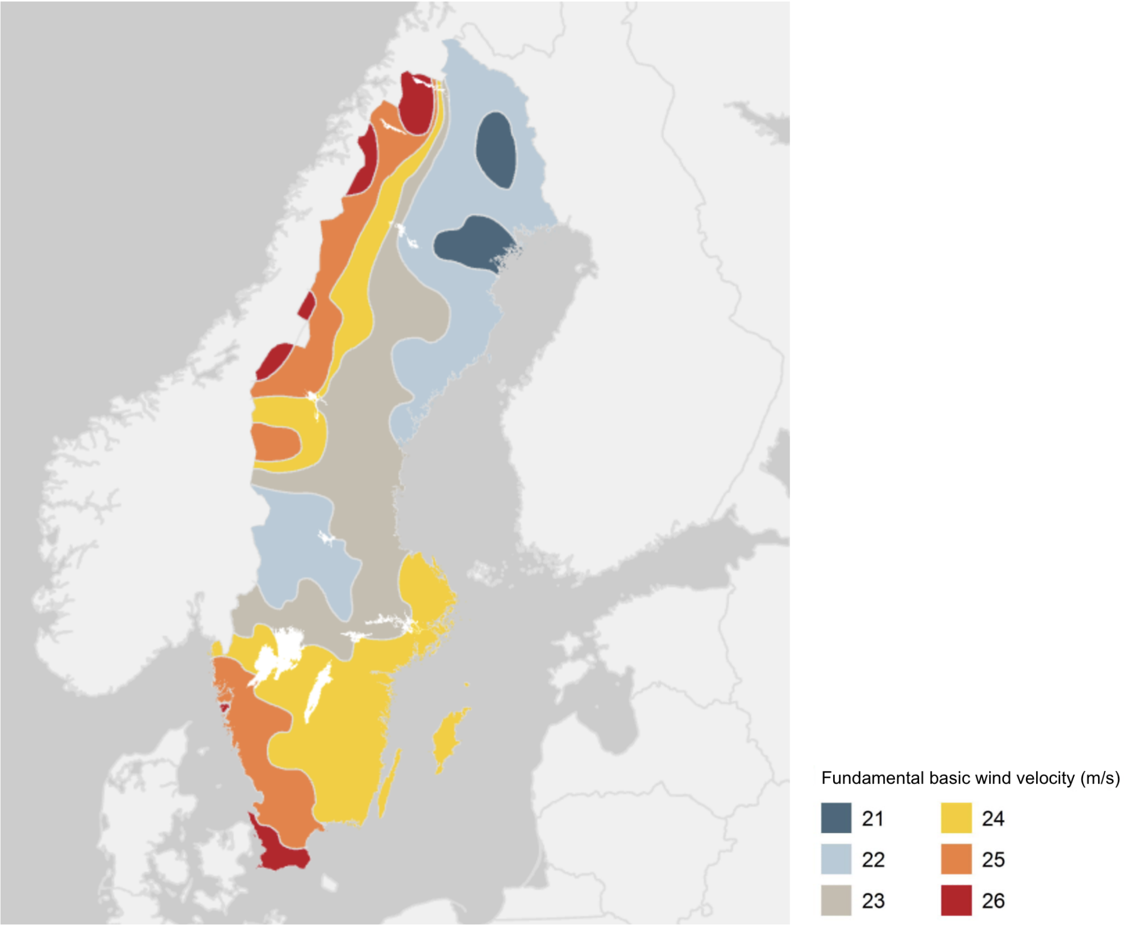


Figure 5.1: *Map of wind loads zones for fundamental basic wind velocity* v_{b0}.

From the fundamental basic wind velocity, the mean wind velocity $v_m(z)$ can be obtained by utilizing Equation 5.2. The mean wind velocity is modified with regards to the terrain roughness ans topography.

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \tag{5.2}$$

where

$c_r(z)$ roughness factor $[-]$
$c_o(z)$ orography factor $[-]$

The ortography factor is considering the effects of hills in the orography that increase the wind speed more than 5%. If ortography not specified or not affecting the wind velocity, the recommended value for the orotrophy factor $c_o(z)$ is 1.0. The roughness factor $c_r(z)$ is depending on the building height by a logarithmic profile and are obtained by applying either Equation 5.3 or 5.4.

$$c_r(z) = k_r \cdot ln(\frac{z}{z_0}) \qquad for \qquad z_{min} \leq z_{max} \tag{5.3}$$

$$c_r(z) = c_r(z_{min}) \qquad for \qquad z \leq z_{min} \tag{5.4}$$

where

k_r	terrain factor $[-]$
z	building height $[m]$
z_0	roughness length $[m]$
z_{min}	minimum height $[m]$
z_{max}	maximum height, 200 m $[m]$

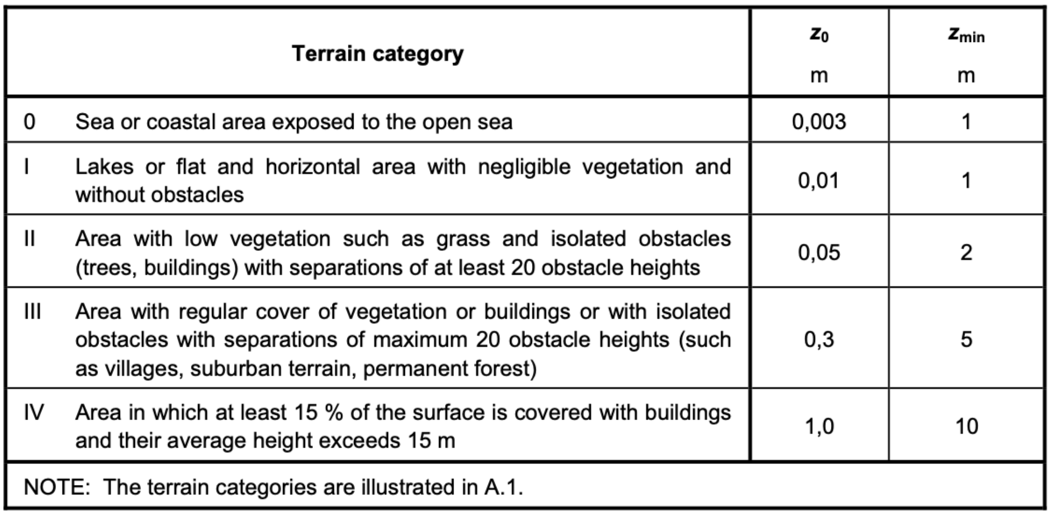
and

$$k_r = 0.19 \cdot (\frac{z_0}{z_{0,II}})^2 \tag{5.5}$$

where

$z_{0,II}$	roughness length for terrain category II, 0,05 m $[m]$

The parameters z_0 and z_{min} is dependent on the terrain category for each specific building and is to be obtain from Table 5.2.



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.			

Figure 5.2: *Terrain categories and terrain parameters z_0 and z_{min}.*

The different terrain categories are depending on the current topography of the specific location. Illustrative explanation of terrain categories is seen in Figure 5.3 and 5.3.

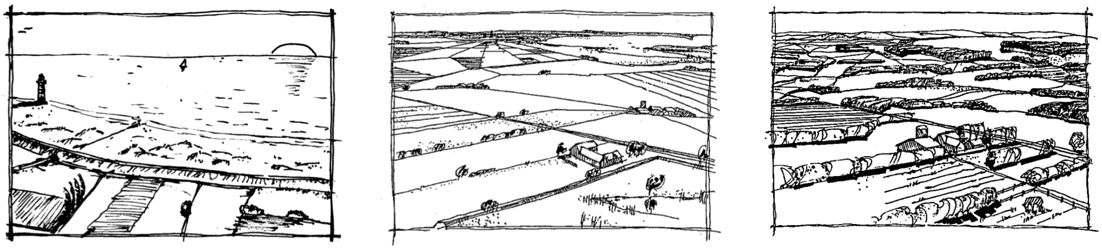


Figure 5.3: *Illustrations of terrain category 0, I, II.*

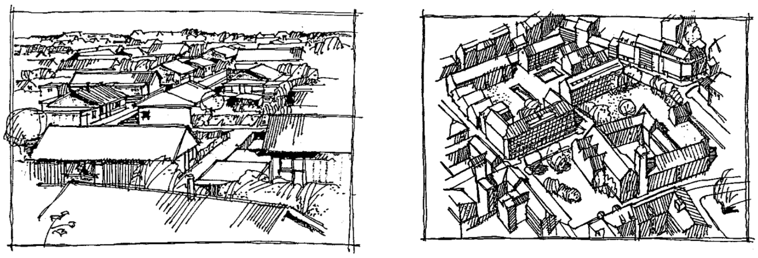


Figure 5.4: *Illustrations of terrain category III and IV.*

The peak velocity pressure is calculated using either Equation 5.6 or Equation 5.7. The air density ρ is set to 1.25 kg/m^3.

$$q_p(z) = [1 + 7 \cdot l_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) \tag{5.6}$$

or

$$q_p(z) = c_e(z) \cdot q_b \tag{5.7}$$

where

$l_v(z)$	turbulence intensity $[-]$
ρ	air density $[kg/m^2]$
$c_e(z)$	exposure factor $[-]$
q_b	basic velocity pressure by Equation 5.10 $[kg/ms^2]$

and

$$l_v(z) = \frac{k_l}{c_o(z) \cdot ln(z/z_0)} \qquad for \qquad z_{min} \leq z_{max} \tag{5.8}$$

$$l_v(z) = l_v(z_{min}) \qquad for \qquad z \leq z_{min} \tag{5.9}$$

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \tag{5.10}$$

where

k_l turbulence factor $[-]$

Recommended value for the turbulence factor k_l is 1.0. The exposure factor $c_e(z)$ is gained for a flat terrain by implementing the diagram in Figure 5.5 that relates the exposure factor to the height of the building z and the terrain category. Observing the diagram, it is clear that the values of $c_e(z)$ do not exists for a building higher that 100 meters.

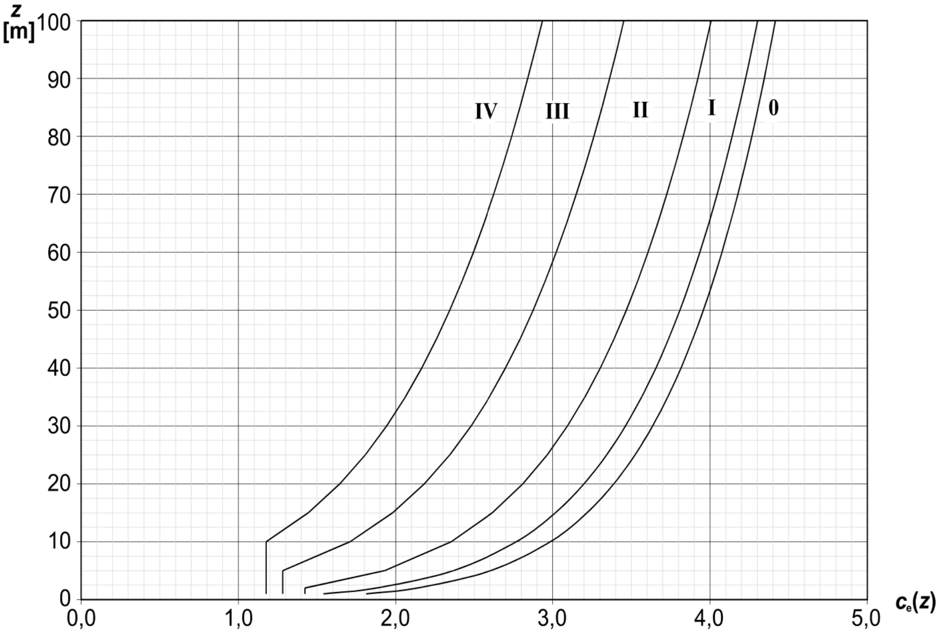


Figure 5.5: *Exposure factor $c_e(z)$ for orography factor $c_o(z)$=1.0 and turbulence factor k_l=1.0.*

The external wind load w_e is calculated using Equation

$$w_e = q_p(z) \cdot c_{pe} \tag{5.11}$$

where

c_{pe} external pressure coefficient $[-]$

The wind force acting on the buildings' each floor element is calculated using Equation 5.12. To simplify the calculations, the buildings' facade can be divided into segments instead of looking at each floor and Equation 5.13 can be utilized.

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{5.12}$$

or

$$F_w = c_s c_d \sum_{elements} c_f \cdot q_p(z_e) \cdot A_{ref} \tag{5.13}$$

where

$c_s c_d$	structural factor $[-]$
c_f	force coefficient $[-]$
A_{ref}	reference area $[m^2]$

The structural factor $c_s c_d$ contains of the size factor c_s and the dynamic factor c_d. The size factor consider an decreased effect on the wind load regarding the non-simultaneity of occurrence of the peak wind pressures. The dynamic factor instead include the escalated effect on wind load from vibrations by cause of turbulence in resonance with the building. In an analytical analysis which only consider the static response of the building, the dynamic factor c_d is set to 1.0 while the size factor c_s is calculated by Equation 5.14.

$$c_s c_d = \frac{1 + 2 \cdot k_p \cdot l_v(z_s)\sqrt{B^2 + R^2}}{1 + 6 l_v(z_s)} \tag{5.14}$$

where

k_p	peak factor $[-]$
$l_v(z_s)$	turbulence intensity for reference height z_s $[-]$
B^2	background factor $[-]$
R^2	resonance response factor $[-]$

The peak factor k_p is calculated using Equation 5.15. For static analysis of structures, k_p=3.0 should be used.

$$k_p = \sqrt{2 \cdot ln(\nu T)} + \frac{0.6}{\sqrt{2 \cdot ln(\nu T)}} \tag{5.15}$$

where

ν	mean up-crossing frequency $[Hz]$
T	mean wind velocity time, T=600 $[s]$

The mean up-crossing frequency ν, presented by Equation 5.16, should be limited to a lower boundary of 0.08 Hz which corresponds to a peak factor k_p of 3.0.

$$\nu = n_{1,x}\sqrt{\frac{R^2}{B^2+R^2}} \qquad ; \qquad \nu \geq 0.08Hz \tag{5.16}$$

where

$n_{1,x}$	fundamental flexural frequency of the structure $[Hz]$

The lowest fundamental flexural frequency of a building higher than 50 meters can be estimated using Equation 5.17.

$$n_{1,x} = \frac{46}{h} \tag{5.17}$$

To consider reduction of the effective wind pressure due to decreasing correlation at increasing area of loading, the background factor is implemented. The resonance response factor is applied to regard the impact of resonance between the turbulence generated by wind and the natural oscillation of the structure. The background factor B^2 and the resonance response factor R^2 is calculated by Equation 5.18 and 5.19 respectively.

$$B^2 = exp[-0.05(\frac{h}{z_s}) + (1-\frac{b}{h})(0.04+0.01(\frac{h}{z_s}))] \tag{5.18}$$

$$R^2 = \frac{2\pi \cdot F \cdot \phi_b \cdot \phi_h}{\delta_s + \delta_a} \tag{5.19}$$

where

z_s	reference height of the structure $[m]$
δ_s	logarithmic decrement for structural damping $[-]$
δ_a	logarithmic decrement for aerodynamic damping $[-]$

and

$$F = \frac{4 \cdot y_c}{(1+70,8 \cdot y_c^2)^{5/6}} \tag{5.20}$$

$$y_c = \frac{150 \cdot n_{1,x}}{v_m(h)} \tag{5.21}$$

$$\phi_b = \frac{1}{1+\frac{3.2 \cdot n_{1,x} \cdot b}{v_m(h)}} \tag{5.22}$$

$$\phi_h = \frac{1}{1 + \frac{2 \cdot n_{1,x} \cdot h}{v_m(h)}} \tag{5.23}$$

The reference height z_s for buildings is obtained in the Eurocode by utilizing Equation 5.24, see related Figure for illustrative explanation.

$$z_s = 0.6 \cdot h \geq z_{min} \tag{5.24}$$

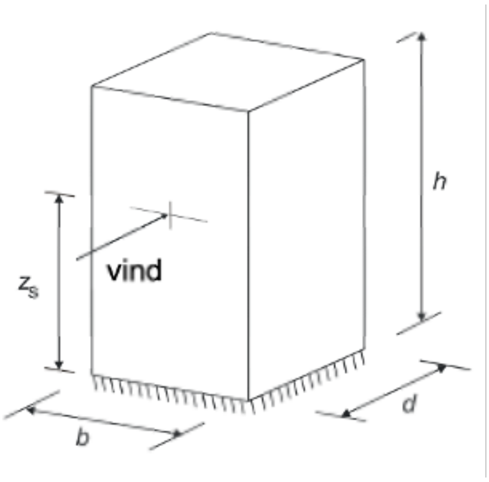


Figure 5.6: *Illustrative explanation of reference height z_s as per Eurocode.*

General the modal deflections is constant along the building height and the logarithmic decrement for aerodynamic damping δ_a can be calculated using Equation 5.25. The logarithmic decrement for structural damping is extracted from a table in the Eurocode depending on the building material, whereas for reinforced concrete $\delta_s = 0.10$.

$$\delta_a = \frac{c_f \cdot \rho \cdot b \cdot v_m(z_s)}{2 \cdot n_{1,x} \cdot m_e} \tag{5.25}$$

where

m_e	equivalent mass per unit length $[m]$

When observing the velocity pressure $q_p(z)$ of the windward side of a building, the reference height z_e is used together with a partition of segments depending on the relation of the aspect ratio h/b. The sectioning of velocity pressure profile is categorized by the three following cases and illustrated in Figure 5.7.

- The building should be considered as one whole part when the height of the building h is less than the width b.

- The building should be considered as two parts when the height of the building h is greater than the width b but smaller or equal than $2b$. The lower segment of the building division should in height be equal to the width b whilst the upper segments' height subsequently should be equal to $h - b$.

- The building should be considered as several parts when the height of the building h is greater than $2b$. Both the lower and uppermost segment of the building division should in height be equal to the width b whilst the middle part should be divided into equal segments of h_{strip}.

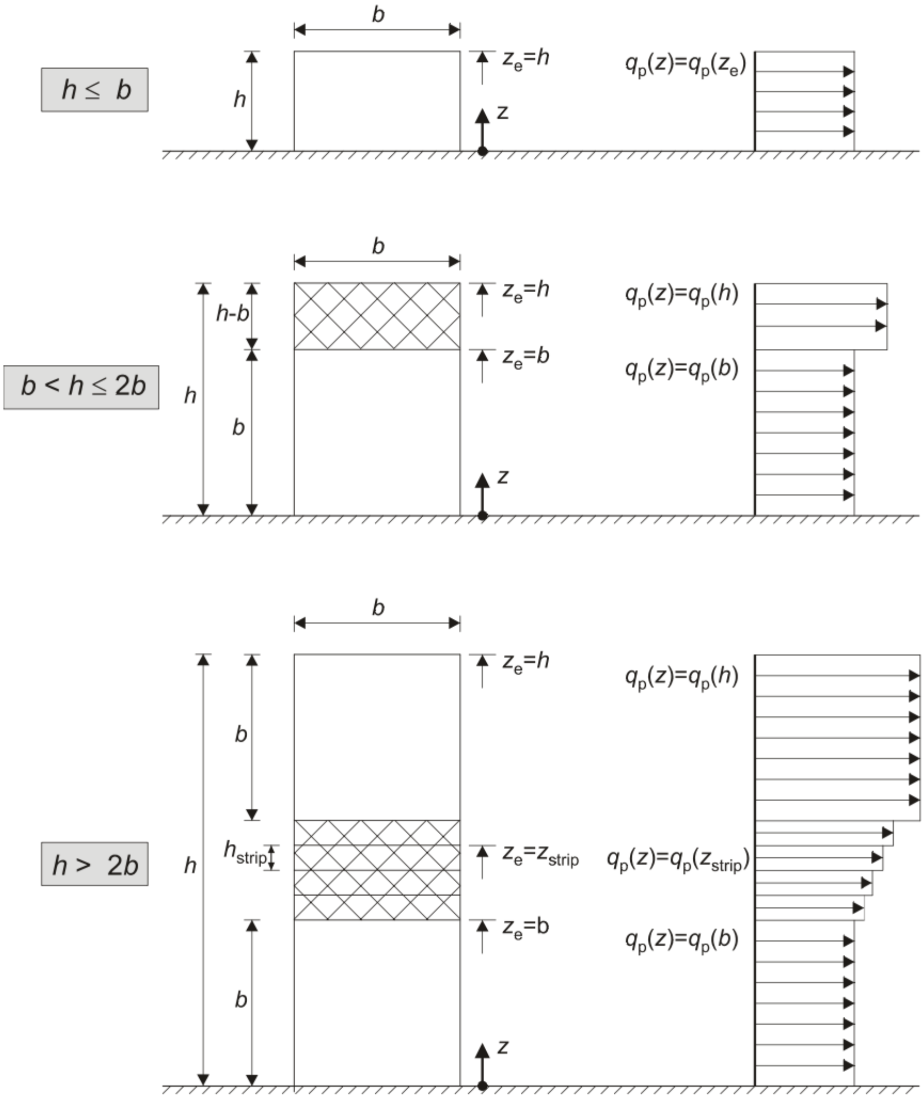


Figure 5.7: *Velocity pressure profile for each case of aspect ratio h/b.*

To provide for an pedagogical presentation of the force coefficient c_f, it will be divided into two different sub chapters where chapter introduce the force coefficient for a rectangular structural element while chapter consider a circular structural element.

5.1.1 External pressure coefficient for rectangular sections

The pressure coefficient for external wind load represent the impact of wind on the buildings' external surfaces. The coefficients for the external wind load are dividend into local and global coefficients where $c_{pe,10}$ are applicable for surfaces area equal or larger than 10 m^2 and $c_{pe,1}$ are utilized for an surface area of 1 m$\hat{2}$ or smaller.

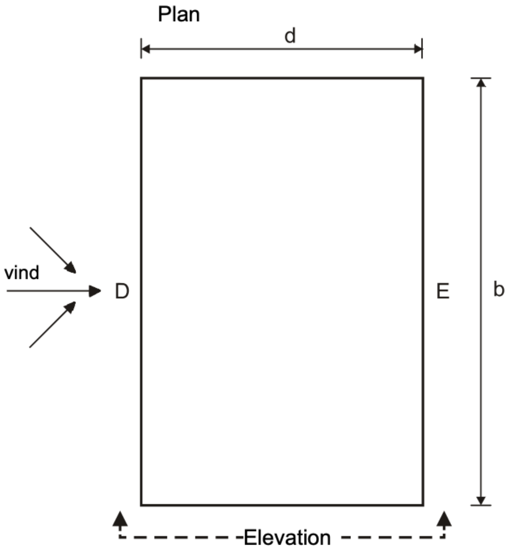
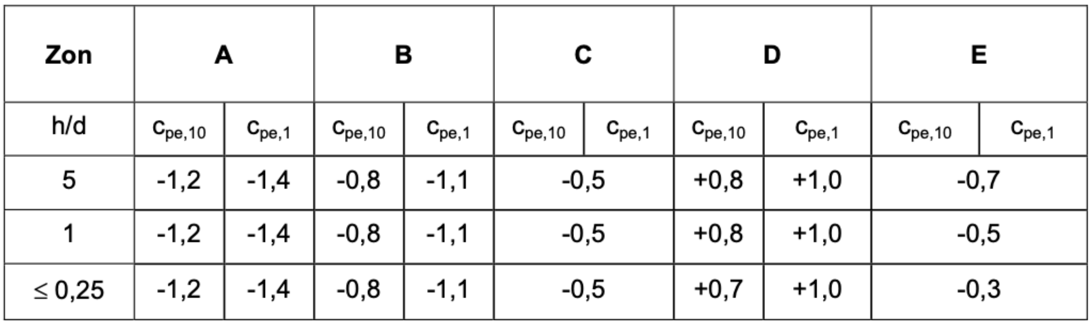


Figure 5.8: *Zones of external wind pressure on a rectangular building.*

Recommended values of the external pressure coefficient c_{pe} is presented in Figure 5.9. If the ratio h/d do not match any of the valus in the table, interpolation can be made. The pressure coefficients for side D and E should be used together. To make up for deficient correlation, the total force should be multiplied with 1.0 for a building where $h/d \geq 5$ and with 0.85 for a building with $h/d \leq 1$. For an aspect ratio between 5 to 1, interpolation ca be utilized.



Zon	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Figure 5.9: *Recommended value for external pressure coefficients*

5.1.2 Force coefficient for rectangular and slender sections

For a building where $h/d > 5$, the external pressure coefficient can be replaced by a force coefficient c_f.

For a rectangular element the force coefficient is obtained by implementing Equation 5.26.

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda \tag{5.26}$$

where

$c_{f,0}$	force coefficient for a section with sharp corners and no free-end flow [−]
ψ_r	reduction factor for sections with round corners [−]
ψ_λ	end-effect factor for sections with free-end flow [−]

The force coefficient $c_{f,0}$ that is established for a rectangular sectioned element with sharp corners and no free-end flow is obtained by observing the ratio of d/b together with related diagram as per Figure 5.10.

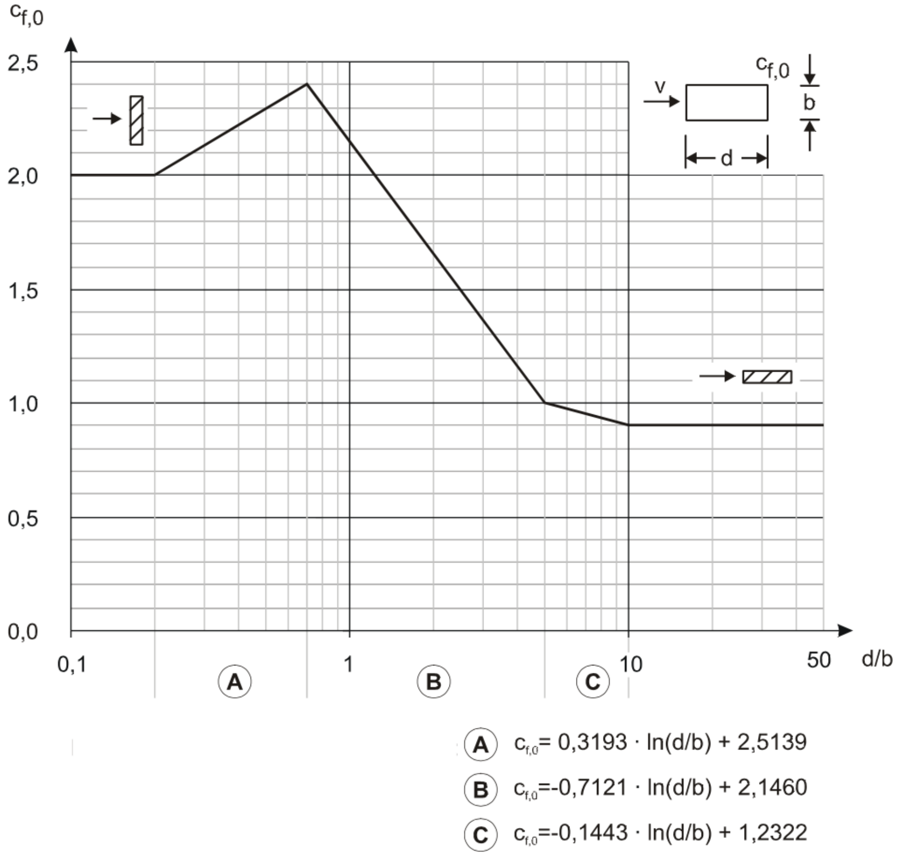


Figure 5.10: *Force coefficient $c_{f,0}$ for a rectangular section with sharp corners and no free-end flow.*

This diagram of conducting the reduction factor ψ_r can also be used for buildings with $h/d > 5.0$.

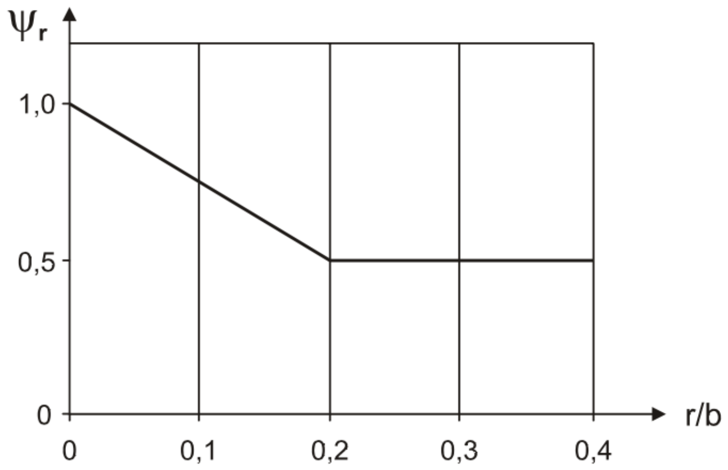


Figure 5.11: *Reduction factor ψ_r for a section with rounded corners.*

For a rectangular building of height greater or equal than 50 meters, the effective slenderness λ should be set to the smallest of either $\lambda = 1.4 \cdot h/b$ or $\lambda = 70$. Relating the effective slenderness with the solidity ratio φ, the end-effect factor ψ_λ is obtained by diagram presented in Figure 5.12. For a solid section the solidity ratio is set to 1.0.

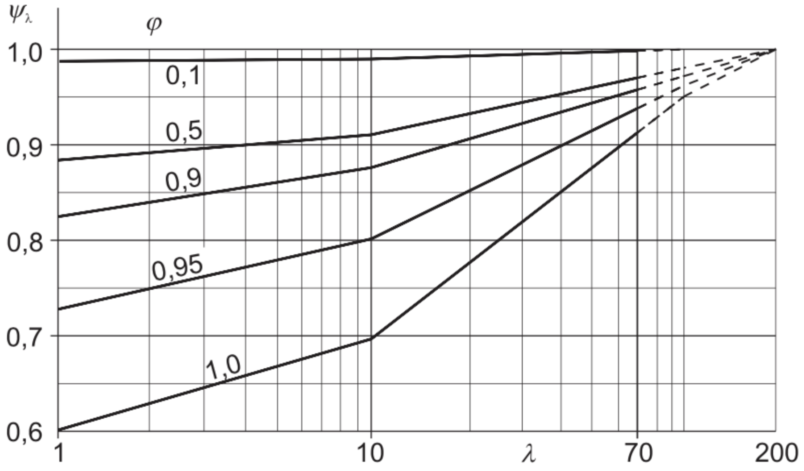


Figure 5.12: *End-effect factor ψ_λ in relation to the solidity ratio φ and the effective slenderness λ.*

5.1.3 Force coefficient for circular sections

The force coefficient used for a circular section is calculated by Equation 5.27.

$$c_f = c_{f,0} \cdot \psi_\lambda \tag{5.27}$$

where

$c_{f,0}$ force coefficient for a circular section with no free-end flow $[-]$
ψ_λ end-effect factor $[-]$

The force coefficient for a circular section is dependent on the equivalent roughness k/b and is obtained by using diagram presented in Figure 5.13. The equivalent surface roughness k are set to 100.

Reynolds number *Re* is calculated by implementing Equation 5.28 and 5.29. Reynolds number represent the ratio between inertial and viscous forces in fluid flow. It is a non-dimensional value which influence the wind loads impact on a building.

$$Re = \frac{b \cdot v_c}{\nu_k} \tag{5.28}$$

$$v_c = \sqrt{\frac{2 \cdot q_p}{\rho}} \tag{5.29}$$

where

v_c peak wind velocity at reference height z_e $[-]$
ν_k kinematic viscosity air, 15e-6 $[m^2/s]$

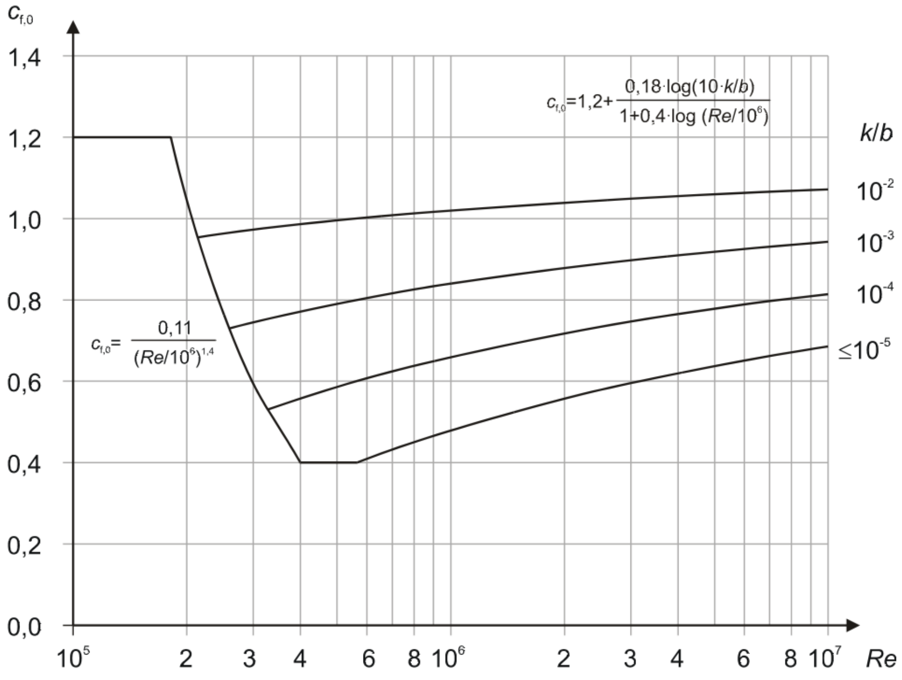


Figure 5.13: *Force coefficient $c_{f,0}$ for a circular section with no free-end flow.*

For a circular building of height greater or equal than 50 meters, the effective slenderness λ should be set to the smallest of either $\lambda = 0.7 \cdot h/b$ or $\lambda = 70$. The end-effective factor ψ_λ is as for an rectangular section obtained by diagram presented in Figure 5.12, Chapter 5.1.3.

5.1.4 Development

Since the diagram relating the exposure factor with the height of the building is limited to a building height of 100 meters, the graph has been developed to apply for even higher buildings. The new developed diagram for the exposure factor is seen in Figure and is applicable for buildings with a height up to 120 meters. The limitation of 120 meters is due to the case studies in this thesis where the highest reference building reach 111 meters in height. Further development of the diagram can be made for even higher buildings, but as observed intuitively, the relation between the building height z and the exposure factor $c_e(z)$ will start to converge when analyzing taller structures.

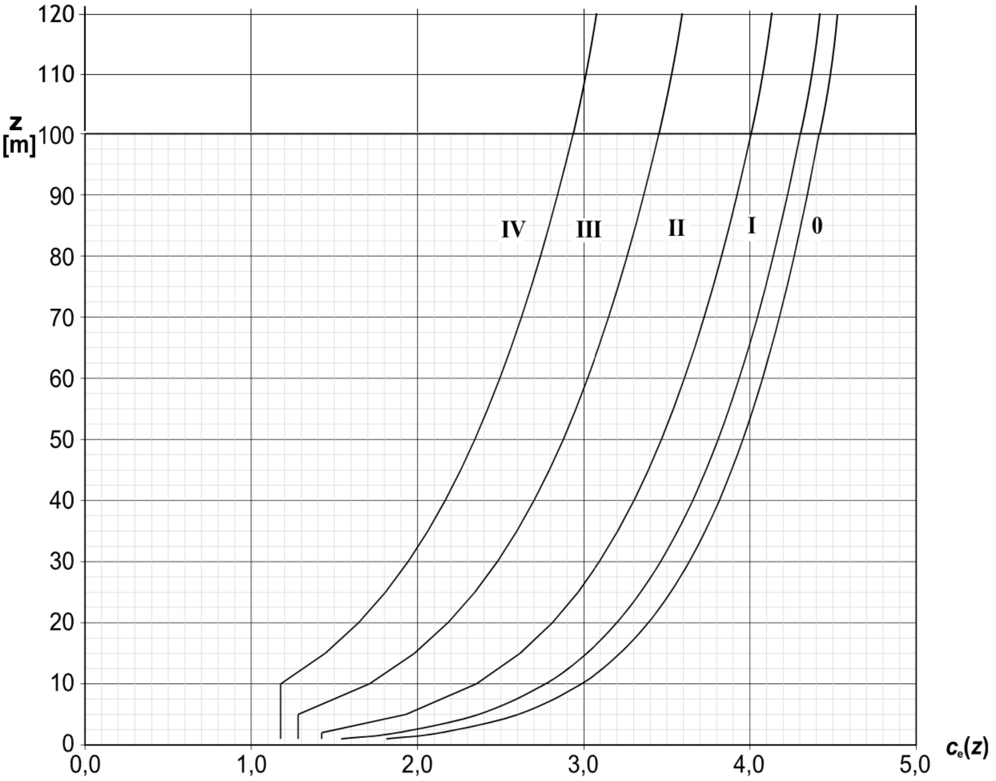


Figure 5.14: *Development of relation between exposure factor $c_e(z)$ and height z for orography factor $c_o(z)$=1.0 and turbulence factor k_l=1.0.*

5.2 Acceleration

The along-wind acceleration can be calculated per Eurocode standards using Equation 5.30.

$$\ddot{X}_{max}(z) = k_p \cdot \sigma_{\dot{x}}(z) \tag{5.30}$$

where

k_p	peak factor $[-]$
$\sigma_{\dot{x}}(z)$	acceleration standard deviation $[m/s^2]$

The standard deviation for characteristic along-wind acceleration should be calculated as per Equation 5.31.

$$\sigma_{\dot{x}}(z) = \frac{c_f \cdot \rho \cdot b \cdot l_v(h) \cdot vT_a}{m_e} \cdot R \cdot K_x \cdot \phi_{1,x}(z) \tag{5.31}$$

where

c_f	force coefficient $[-]$
ρ	air density, $\rho = 1.25$ $[-kg/m^3]$
b	building width $[m]$
$l_v(h)$	intensity factor $[-]$
vT_a	average wind velocity $[m/s]$
m_e	equivalent mass $[kg/m]$
R	resonance response factor $[-]$
K_x	dimensionless coefficient $[-]$
$\phi_{1,x}(z)$	Mode shape $[-]$

and

$$\phi_{1,x}(z) = (\frac{z}{h})^{\xi} \tag{5.32}$$

where

ξ	1.5 for buildings of reinforced concrete $[-]$

The non-dimensional power spectral density function is calculated by Equation 5.33. $S_L(z, n_{1,x})$ is the non-dimensional power spectral density function and cover the wind distribution over frequencies, calculated by Equation 5.33.

$$S_L(z, n_{1,x}) = \frac{6.8 \cdot f_L(z, n_{1,x})}{(1 + 10.2 \cdot f_L(z, n_{1,x}))^{5/3}} \tag{5.33}$$

The non-dimensional frequency is obtained by Equation 5.34.

$$f_L = \frac{n_{1,x} \cdot L(z)}{vTa} \tag{5.34}$$

The characteristic turbulence length for a building with $z \geq z_{min}$ is calculated using Equation 5.35 with L_t is equal to 300 m and z_t equal to 200 m. The turbulence length represent the gust wind effect.

$$L(z) = L_t \cdot (\frac{z}{z_t})^{\alpha} \tag{5.35}$$

where

$$\alpha = 0.67 + 0.05 ln(z_0) \tag{5.36}$$

Assuming that the mode function is expressed per Equation 5.32, the non-dimensional coefficient K_x is calculated as follows;

$$K_x = \frac{(2 \cdot \xi + 1) \cdot ((\xi + 1) \cdot [ln(\frac{z_s}{z_0}) + 0.5] - 1)}{(\xi + 1)^2 \cdot ln(\frac{z_s}{z_0})} \tag{5.37}$$

Assessment of comfort evaluation is based on a wind velocity for a return period for an average of once every five years due to criteria for "responses of people to horizontal motion of structures in the frequency range 0.063 to 1 Hz (ISO 6897). For a return period T_a, the wind velocity can be calculated per Equation 5.38.

$$vT_a = 0.75 \cdot v_{50} \cdot \sqrt{[1 - 0.2 ln(-ln(1 - \frac{1}{T_a}))]} \tag{5.38}$$

where

T_a	return period in years $[-]$
v_{50}	characteristic fundamental wind velocity $[-]$

The characteristic fundamental wind velocity is assumed to exceed its value with 2 % during one year which corresponds to a return period of average 50 years (BFS 2015:6).

5.2.1 Comfort criteria

To evaluate the vibration response acceptability of a building, ISO Standard SS-ISO-10137:2008 is implemented. For a one-year return period, the relation between the peak acceleration A and the first natural frequency f_0 is evaluated by the diagram in Figure 5.15. The relation between this two parameters should not exceed the curve where curve 1 represents offices and curve 2 residential. The method for calculating the along-wind acceleration according to Eurocode include Equation 5.38 which if analysed for one year return period, will go towards infinity. Hence an alteration needs to be made when using this equation together with the occupant comfort criteria curve.

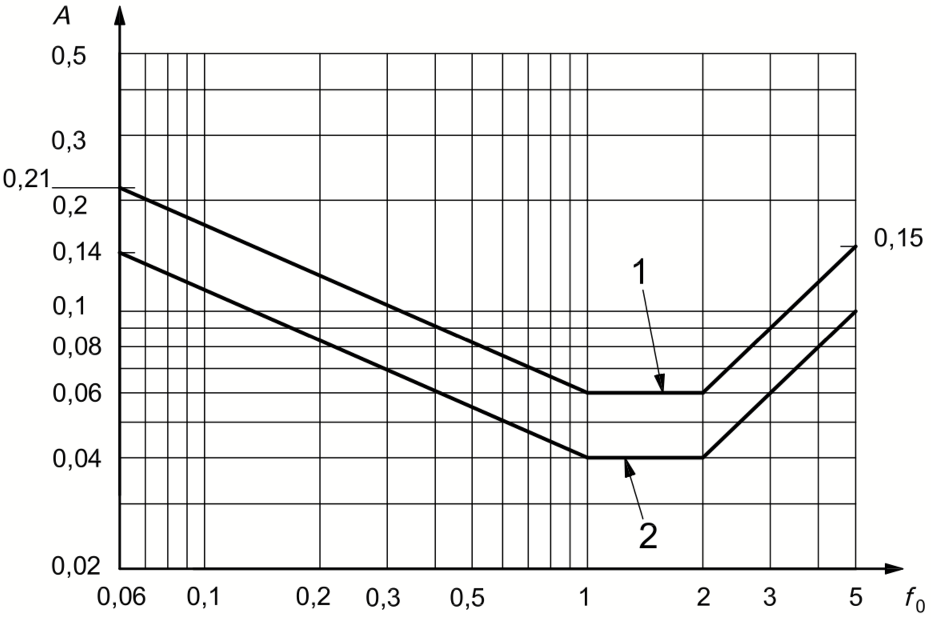


Figure 5.15: *Occupant comfort criteria for vibration acceptability in a building for one-year return period.*

6

FEM analysis

The FEM analysis is used to conclude differences between the analytical calculations and the wind tunnel test results. The FEM modelling is conducted in the FEM program RFEM. A model for each building is design in a simplified manner, explained in detail in following sub-chapters. An eigenfrequency analysis is conducted where the natural vibrations generated from self-weight of the building is observed. Resulting natural frequencies are calculated for each configuration of model where an iterative investigation of core wall thicknesses is conducted. This chapter covers an introduction if the program RFEM along with the modelling process in general and individual for each building from the case study.

6.1 RFEM modelling

RFEM is a finite element method program that focus on structural analysis and design software which is developed by Dlubal. Licence in RFEM is provided for the cause of this masters' thesis by Sweco Sverige AB.

The models are designed in a simplified manner with core walls and slabs. The slabs are designed without any weight. In this way, there is no need to design columns in the model which either way do not contribute to the global stability of the structure. The self-weight of the slabs together are instead included as a line load at the core walls with a magnitude of 25 kN/m, assumed that each core wall take 5 meters of slab weight.

The FEM-analyses are conducted with EN 1990 as standard together with Svensk Standard SS as National annex. The models are analysed in characteristic service limit state, hence all partial factors are set to 1.0. The analyse is conducted with consequence class 3, high consequence for serious injury of people.

Two different load combinations are defined for simplicity. The first load combination are with the lateral loads from permanent self-weight of the concrete core walls as well as the concrete slabs together with dividing walls. The second load combination is the wind. The analysis of the natural frequency is based on the mass case of the self-weight of the whole building. The concrete weight is assumed to 25 kg/m^3 for the whole structure.

To analyse the dynamic behaviour of the structures, the add-on module RF-DYNAM PRO is used in RFEM. Conducting this analysis generates the natural frequencies for wanted number of lowest eigenvalues. To compare to the provided wind tunnel tests, three modes are investigated. The method used in RFEM for solving the eigenvalue problem is the Lanczos method. Mass matrix being used is a diagonal matrix with transnational degrees of freedoms. No additional stiffness modification is made.

6.2 Building A

Building A is modelled in RFEM by designing the layout of core walls together with simplified slabs. The slabs are modelled as rectangle solids without any irregularities. Each floor height is building accordingly to the provided architectural drawings. Both core walls and concrete slabs are of concrete class C40/50. FEM model of Building A is to be seen in Figure 6.1.

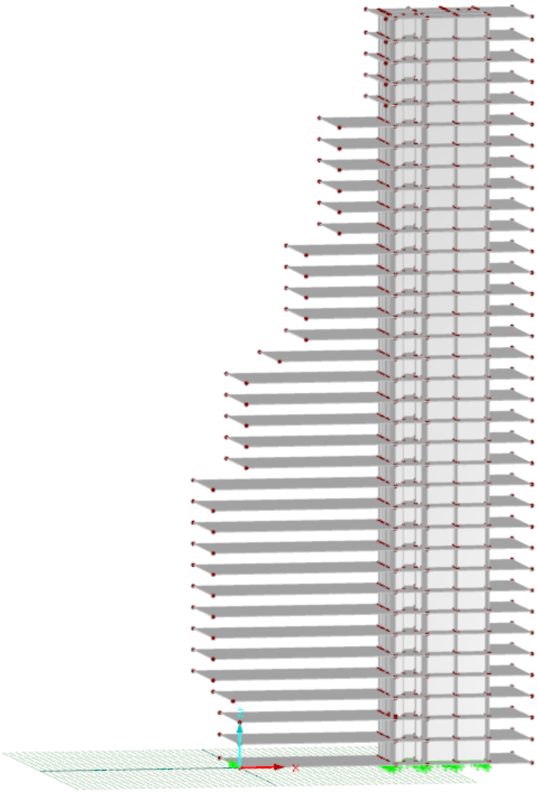


Figure 6.1: *FEM model of the structural system Building A.*

Building A is at first modeled with 400 millimeters core wall thickness. This corresponds to an average core wall self-weight of 30 kN/m. The slab is 200 mm in thickness which translates to a load of 25 kN/m that is added as a line load on the core walls.

The core wall thickness was adjusted in several iterations to fulfill the comfort criteria. The final model consisted of core walls of 700 millimeters in thickness. This corresponds to a core wall self-weight of 52.5 kN/m. The slabs remains unchanged and subsequently its self-weight applied to the core walls. Summarized loading is presented in Table 6.1.

Table 6.1: *Iteration configurations with corresponding loading.*

Iteration configuration	Core wall self-weight	Slab self-weight
400 mm core walls, 200 mm slabs	30 kN/m	25 kN/m
700 mm core walls, 200 mm slabs	52.5 kN/m	25 kN/m

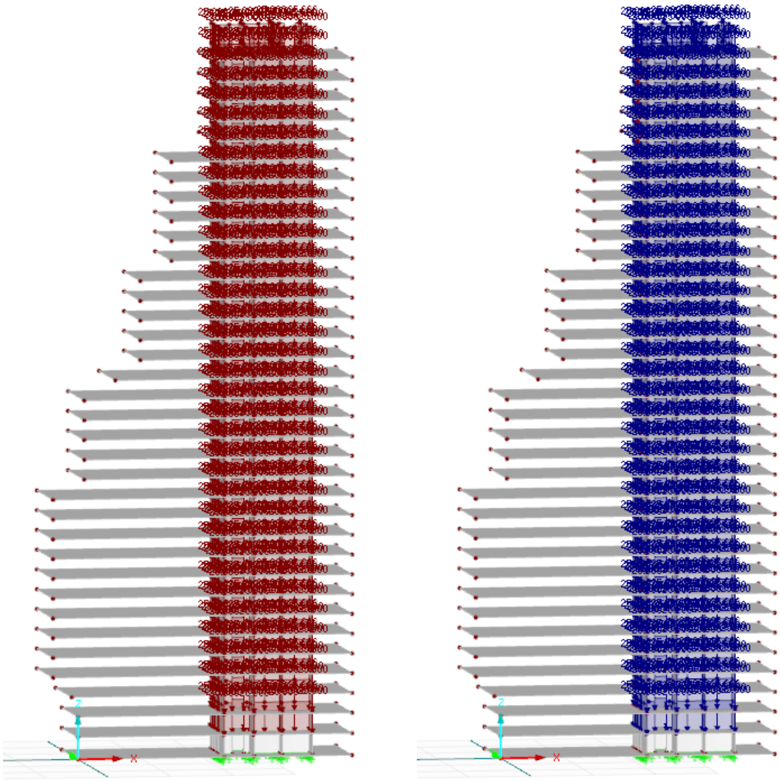


Figure 6.2: *Load models with self-weight and additional vertical loading, respectively.*

The dynamic analysis of the structural system generates the natural frequency for each mode investigated. Appurtenant with the natural frequency, the program also presents the eigenvalues, angular frequency and natural period for the dynamic response. A illustration of the response of vibration for the building is showed in Figure 6.3 and 6.4.

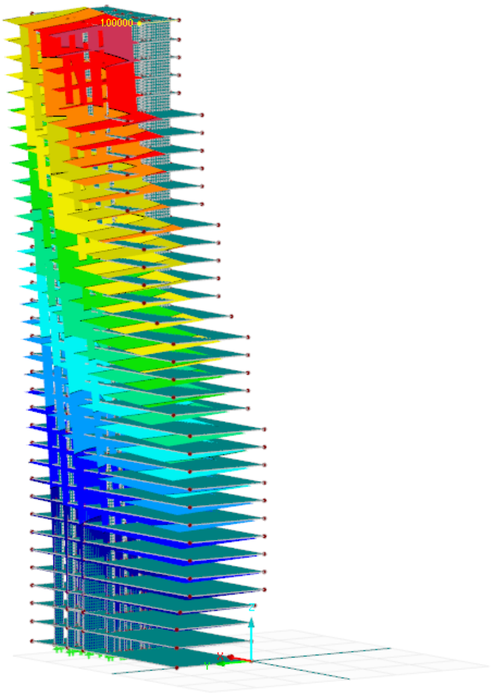


Figure 6.3: *Dynamic analysis model Building A.*

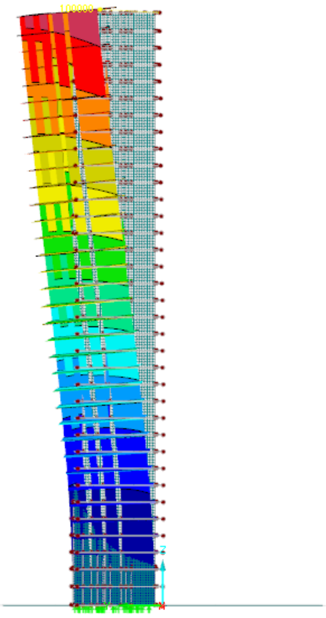


Figure 6.4: *Dynamic analysis model Building A.*

6.3 Building B

When creating a FEM model for Building B, simplifications have been made for the design of the slab as well as the core walls. The slab are simplified as a 24 cornered shape that follows the core walls positions. Both core walls and concrete slabs are of concrete class C40/50. The core walls do not include any openings as in reality, there is a lot of. This is considered later when observing the result. FEM model of Building B is to be seen in Figure 6.5.

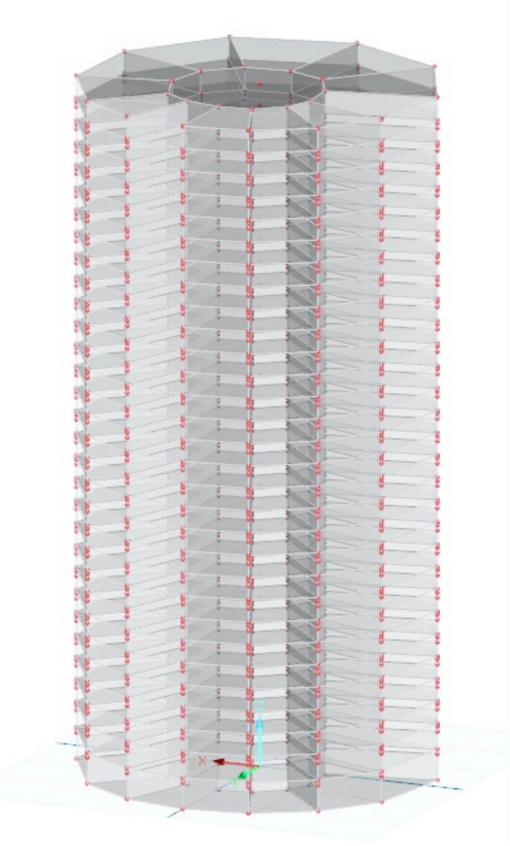


Figure 6.5: *FEM model of the structural system Building B.*

First iteration is made with 400 millimeter core wall thickness and 200 millimeter slab thickness. Subsequently, the self-weights remain the same for core walls and slabs as for Building A. Loading models are to be seen in Figure 6.6.

In a later iteration the core wall thickness is increased to a value of 600 millimeters. This results in a self-weight loading of 45 kN/m. The slabs are as previous case remained the same. See summarized loading is presented in Table 6.2.

Table 6.2: *Iteration configurations with corresponding loading.*

Iteration configuration	Core wall self-weight	Slab self-weight
400 mm core walls, 200 mm slabs	30 kN/m	25 kN/m
600 mm core walls, 200 mm slabs	45 kN/m	25 kN/m

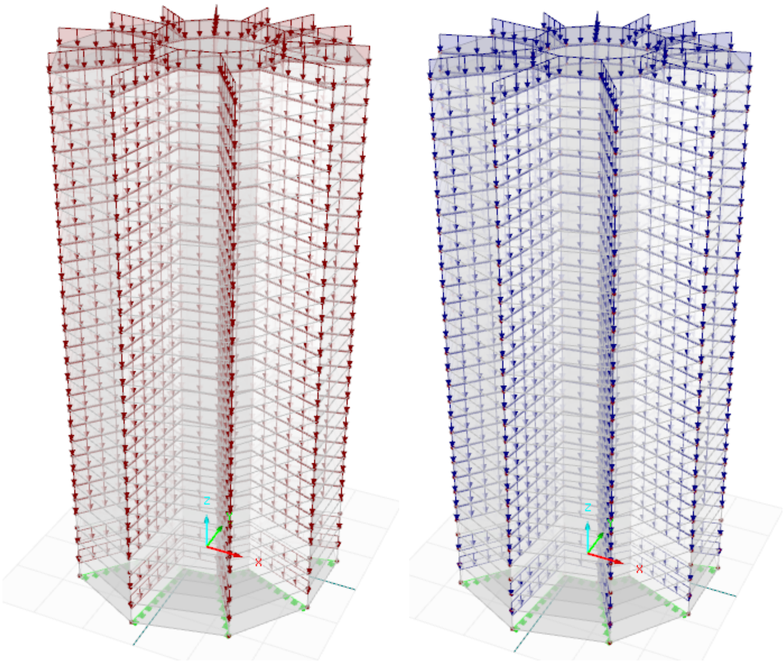


Figure 6.6: *Load models with self-weight and additional vertical loading, respectively.*

The dynamic response of the structure is analysed and provide desired natural frequencies for further investigation. Dynamic behaviour of Building B is shown in Figure 6.7.

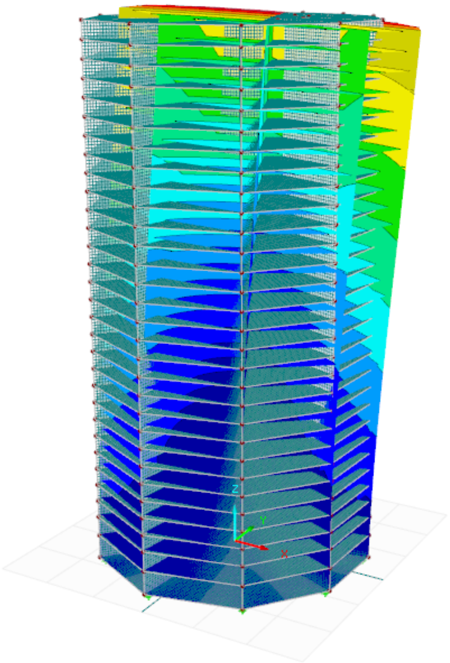


Figure 6.7: *FEM model of Building B after dynamic analysis.*

6.4 Building C

For Building C, both core walls and concrete slabs are of concrete class C40/50 as previous building models. The core walls are modelled as continuous walls as a simplification. In reality, there are a lot of openings in the core walls at each floors. In this building, the slabs are of 220 millimeters thickness. FEM model of Building C is to be seen in Figure 6.8.

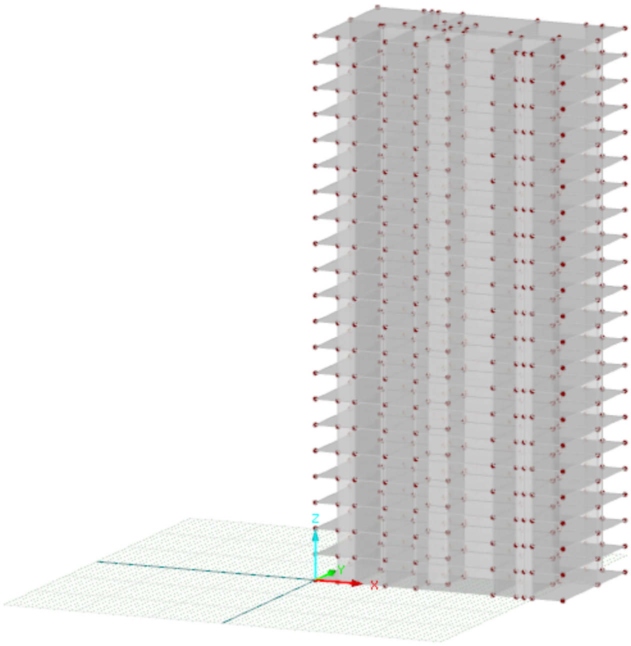


Figure 6.8: *FEM model of the structural system Building C.*

As previous building models, the first iteration is made with 400 millimeter core wall thickness. In Building C, the slabs are of 220 millimeters thickness and subsequently generate a higher self-weight load applied on the concrete core walls. The assumption that each wall takes 5 meters of slab weight still stands which results in a self-weight load of 27.5 kN/m.

To accomplish the comfort criteria, the core wall thickness is increased to 500 millimeters which generate a self-weight load of 37.5 kN/m. The slab thickness is once agian remained the same. Summarized loading in Table 6.3.

Table 6.3: *Iteration configurations with corresponding loading.*

Iteration configuration	Core wall self-weight	Slab self-weight
400 mm core walls, 220 mm slabs	30 kN/m	27.5 kN/m
500 mm core walls, 220 mm slabs	37.5 kN/m	27.5 kN/m

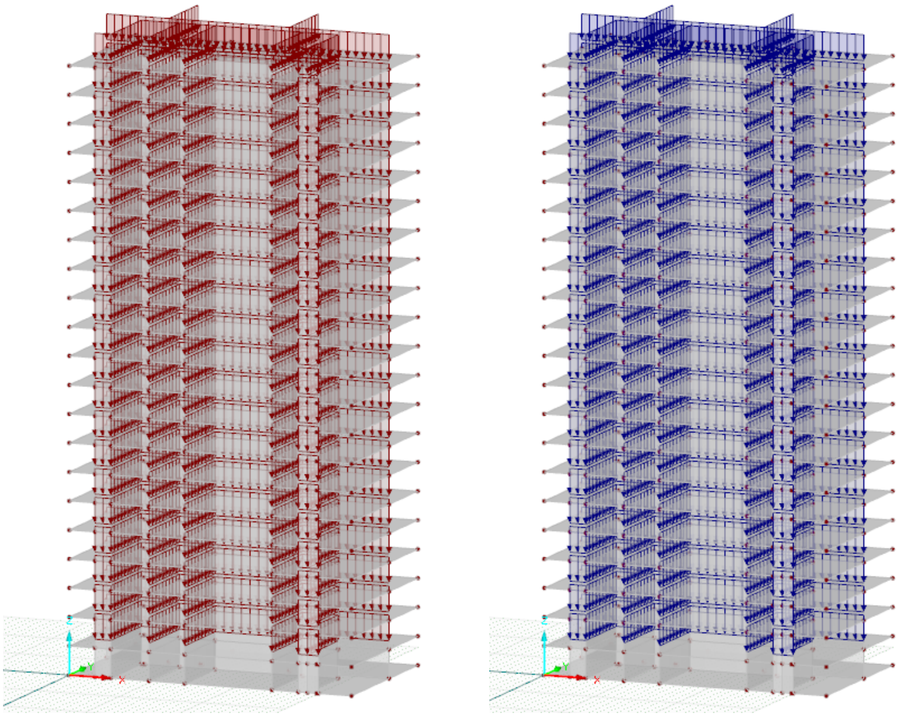


Figure 6.9: *Load models with self-weight and additional vertical loading, respectively.*

The dynamic analysis of the structural system generates the natural frequency for each mode investigated as for previous buildings. A illustration of the response of vibration for the building is showed in Figure 6.10.

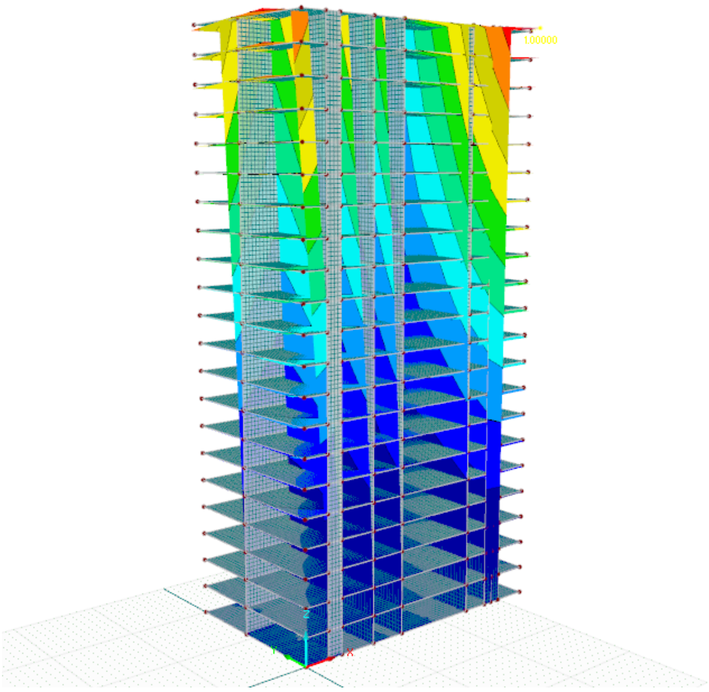


Figure 6.10: *FEM model of Building C after dynamic analysis.*

7

Results

Results gathered from the analytical calculations based on current building standards and FEM modelling analysis is presented in this chapter. The results are presented together with relevant information from the wind tunnel test as a comparison.

Each reference building is observed and analysed by first following current building standards to predict the wind loads acting on the buildings' dominant wind direction facade. This analysis is divided into four different methods as per following;

EN-1991-1-4 1A	The peak velocity pressure is assumed by utilizing Equation 5.6. The wind force is calculated by implementing Equation 5.13 and Figure 5.7 for segmented division of building surface.
EN-1991-1-4 2A	The peak velocity pressure is assumed by utilizing Equation 5.7 and appurtenant developed method for the exposure factor. The wind force is calculated by implementing Equation 5.13 and Figure 5.7 for segmented division of building surface.
EN-1991-1-4 1B	The peak velocity pressure is assumed by utilizing Equation 5.6. The wind force is calculated by implementing Equation 5.12 and is analysed for each floor of the building.
EN-1991-1-4 2B	The peak velocity pressure is assumed by utilizing Equation 5.7 and appurtenant developed method for the exposure factor. The wind force is calculated by implementing Equation 5.12 and is analysed for each floor of the building.

The FEM analysis showed the dynamic response for each building and generated desired natural frequencies, presented in following sub-chapters. The natural frequency is implemented in calculation to predict the maximum acceleration at the top of the building. By looking at the relation between frequency and acceleration, comfort criteria can be evaluated. In the stiffness calculations, a modulus of elasticity for concrete class C40/50 of 25 GPa is used for all reference buildings.

7.1 Building A

Building A is a rectangular shaped building were the building length is decreased along its height. As presented in Chapter 4 and Table 4.1, the building is very slender with a slenderness ratio of 7.4/1. Subsequently, the force factor implemented in the analytical analysis are calculated by practice of the theory in Chapter 5.1.2, addressing slender structures.

7.1.1 Wind loads

When observing Building A, the dominant wind direction is the long facade, see Figure 7.1. The analysis consider the decreased length along its height.

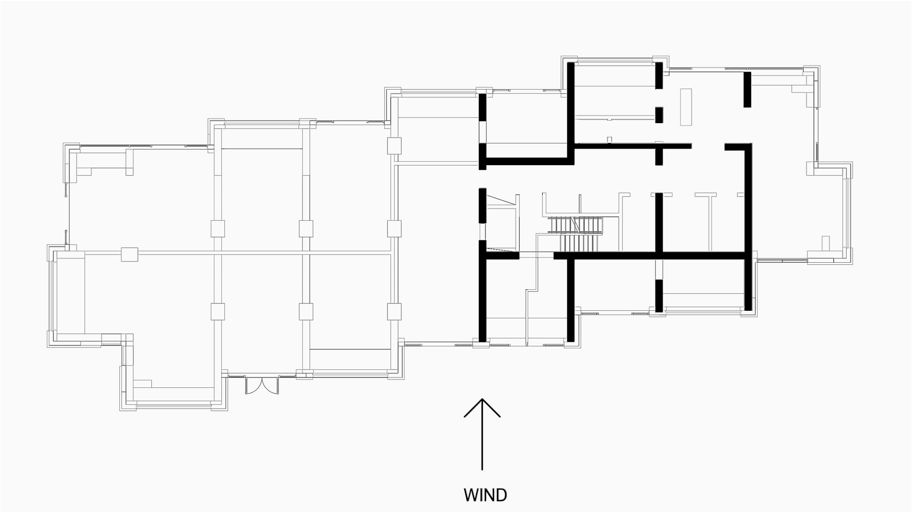


Figure 7.1: *Structural system of Building A with dominant wind direction.*

The peak velocity pressure from the analytical calculation is obtained by looking at the top of the building. Calculations based on Eurocode building standards result in a peak velocity pressure of 1.47 kN/m^2. Provided wind tunnel test for Building A show a peak velocity pressure of 1.37 kN/m^2.

Table 7.1: *Peak wind velocity pressure at the top of the building.*

Method	$\Sigma q_p(h)$
EN-1991-1-4	1.47 kN/m^2
Wind tunnel test	1.37 kN/m^2

Resulting total force on the long facade of the building is presented together with wind tunnel test result in Table 7.2. The comparison shows both at difference between the building standard methods as well as a discrepancy from the wind tunnel test. The total wind force generated by the wind tunnel test is lower than the forces obtained from the analytical analysis. Observing the building code methods, it is shown that the wind load assessment is closer to the wind tunnel test when investigating the wind force on each individual floor. Complete calculation results for each floor and segment are presented in Appendix A.

Table 7.2: *Total wind load and comparison to wind tunnel test.*

Method	ΣF_y	Difference to WTT
Wind tunnel test	6755 kN	
EN-1991-1-4 1A	9266 kN	41 %
EN-1991-1-4 2A	8131 kN	24 %
EN-1991-1-4 1B	7613 kN	16 %
EN-1991-1-4 2B	7597 kN	16 %

7.1.2 Frequency and acceleration

The FEM analysis in RFEM is initially completed by looking at the structural system of Building A with a core wall thickness of 400 millimeters. The dynamic analysis generate natural frequencies for mode 1-3, see Table 7.3. The frequencies obtained from this eigenfrequency analysis is showed to be of lower value than the wind tunnel test.

Table 7.3: *Natural frequency from FEM analysis and wind tunnel test.*

Mode	FEM analysis	Wind tunnel test
Mode 1	0.247 Hz	0.293 Hz
Mode 2	0.340 Hz	0.456 Hz
Mode 3	0.477 Hz	0.487 Hz

After several iterations, a configuration of Building A with a core wall thickness of 700 millimeters are conducted. This results in a first mode natural frequency of 0.318 Hz. Table 7.4 present frequencies for all different configurations along with calculated top acceleration for along-wind motion. Calculations for acceleration are based on the mean wind velocity at the top of the building obtained from building code assessment respectively wind tunnel test results.

Table 7.4: *Frequency and acceleration for each configuration of Building A.*

Configuration	$n_{1,x}$	$\ddot{X}_{max}(z)$
Wind tunnel test, 400mm	0.247 Hz	0.056 m/s^2
EN-1991-1-4, 400mm	0.247 Hz	0.086 m/s^2
EN-1991-1-4, 700mm	0.318 Hz	0.067 m/s^2
EN-1991-1-4, $n_{1,x}$=46/h	0.410 Hz	0.050 m/s^2

The relation between the frequency and the top acceleration is evaluated in the comfort criteria per ISO:10137:2008. The different results of frequency and acceleration are inserted in the evaluation diagram, Figure 7.2. As observed, the wind tunnel test results achieve an approved level of comfort since it falls within the limit of residential usage, curve number 2. For the frequency and acceleration obtained from wind load calculations based on Eurocode EN-1991-1-4, the configuration of Building A with a core wall thickness of 400 millimeters do not accomplish desired demands of comfort. First at a increased thickness of 700 millimeters core wall, the demands of comfort for residential use is achieved. As for the natural frequency assumed by theoretical methods, the comfort demand is fulfilled.

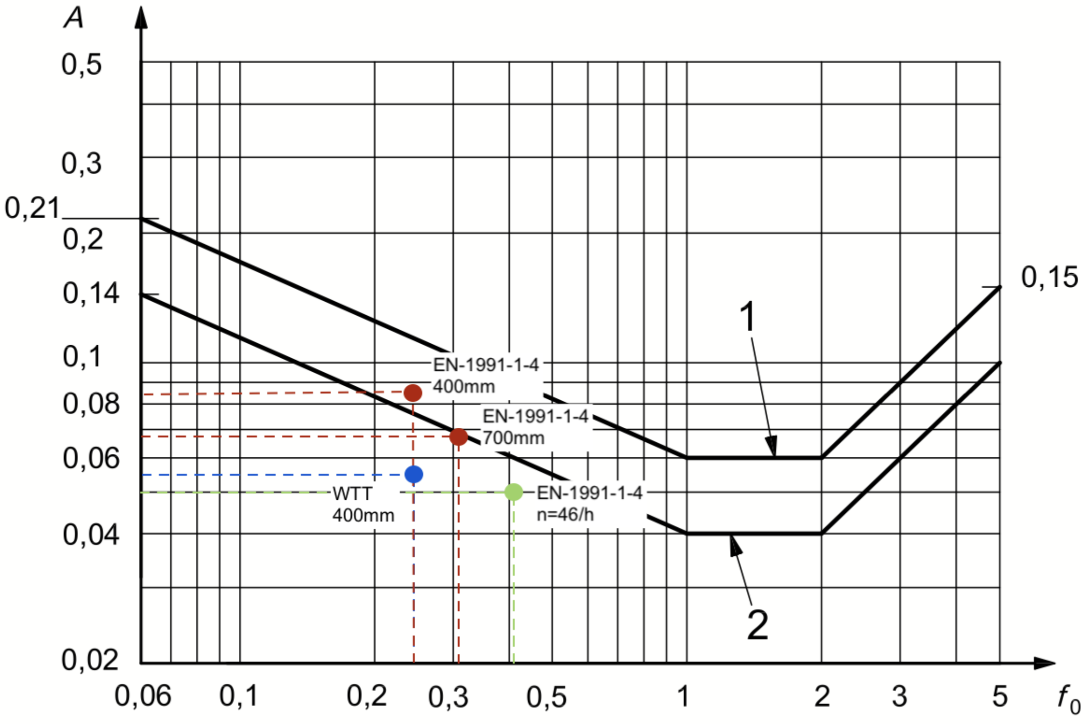


Figure 7.2: *Evaluation curve for comfort demands.*

7.1.3 Stiffness

The cross section of the structural system for Building A is sketched in the FEM program FEM-Design to obtain the second moment of inertia. This analysis is carried out for both configurations of core wall thickness. The stiffness is calculated for each configuration where $(EI)_{WTT}$ represents the core wall thickness of 400 millimeter, the needed thickness for fulfillment of comfort by wind load from wind tunnel test results. The stiffness $(EI)_{EN}$ represents the configuration of core walls of 700 millimeters that showed to be required for obtaining an approved comfort based on wind load by EN-1991-1-4.

The ratio between these two stiffness configurations are showed below in Equation 7.1. It is observed that the stiffness required when using a wind load assessment based on building standards is 41% higher.

$$\frac{(EI)_{WTT}}{(EI)_{EN}} = 0.59 \tag{7.1}$$

where

$(EI)_{WTT}$	required stiffness based on wind load from wind tunnel test [−]
$(EI)_{EN}$	required stiffness based on wind load from EN-1991-1-4 [−]

7.2 Building B

Building B is a circular shaped building of approximately 95 meters with 29 floors. Due to its circular silhouette, the force coefficient used in the analytical analysis is obtained by following the methods in Chapter 5.1.3. In the FEM analysis, the concrete slabs are modelled with a 24 cornered geometry without consideration of irregularities. The core walls are designed without any openings in them.

7.2.1 Wind loads

The main wind direction is from the side of the building, see Figure 7.3. In the analytical analysis, viscous forces are considered due to its circular like shape.

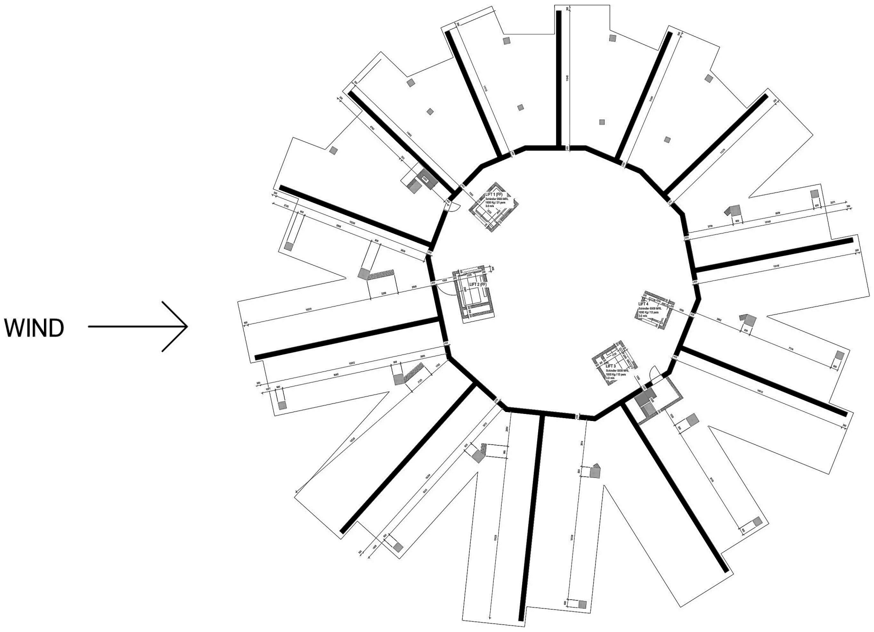


Figure 7.3: *Floor plan of Building B with dominating wind direction.*

When observing the pear wind velocity pressure at the top of the building, it is shown that the analytical analysis generate a higher assumption than the wind tunnel test, see Table 7.5.

Table 7.5: *Peak wind velocity pressure at the top of the building.*

Method	$\Sigma q_p(h)$
EN-1991-1-4	1.43 kN/m^2
Wind tunnel test	1.33 kN/m^2

The analytical results are for Building B closer to the wind tunnel test results as per Table 7.6. Once again the analytical methods that investigate the wind force on each individual floor of the building result in a value closer to the wind tunnel test result.

Table 7.6: *Total wind load and comparison to wind tunnel test.*

Method	ΣF_y	Difference to WTT
Wind tunnel test	5088 kN	
EN-1991-1-4 1A	6880 kN	35 %
EN-1991-1-4 2A	6281 kN	23 %
EN-1991-1-4 1B	5514 kN	8 %
EN-1991-1-4 2B	5506 kN	8 %

7.2.2 Frequency and acceleration

The FEM analysis generated natural frequencies for mode 1-3, presented in Table 7.7. The obtained frequencies are lower than the ones predicted in the wind tunnel test.

Table 7.7: *Natural frequency from FEM analysis and wind tunnel test.*

Mode	FEM analysis	Wind tunnel test
Mode 1	0.222 Hz	0.327 Hz
Mode 2	0.347 Hz	0.565 Hz
Mode 3	0.428 Hz	0.583 Hz

The core wall thickness for Building B were iterated repeatedly to achieve comfort criteria demands. The end configuration resulted in a core wall thickness of 600 millimeters and hence a natural frequency of 0.265 Hz. Load case and configuration for the final iterations are shown in Table 7.8.

Table 7.8: *Frequency and acceleration for each configuration.*

Configuration	$n_{1,x}$	$\ddot{X}_{max}(z)$
Wind tunnel test, 400mm	0.222 Hz	0.067 m/s^2
EN-1991-1-4, 400mm	0.222 Hz	0.084 m/s^2
EN-1991-1-4, 600mm	0.265 Hz	0.071 m/s^2
EN-1991-1-4, $n_{1,x}$=46/h	0.490 Hz	0.036 m/s^2

When evaluating the comfort criteria, it is shown that the wind tunnel test configuration fulfill the demands with a core wall thickness of 400 millimeters while the design based on building standard wind load, the core wall thickness was required to be 600 millimeters. The building code assessment of the natural frequency 46/h fulfill the limitation of comfort but results in values of frequency and acceleration far from both wind tunnel and FEM analysis results. Comfort criteria for Building B is to be seen in Figure 7.4.

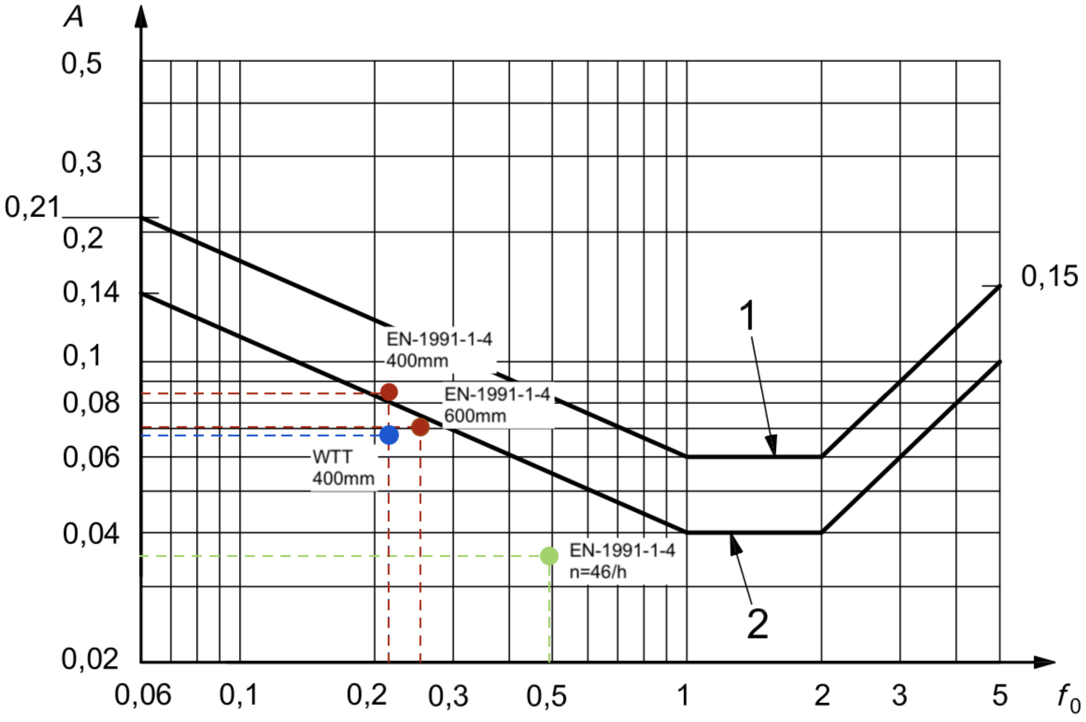


Figure 7.4: *Evaluation curve for comfort demands.*

7.2.3 Stiffness

By utilizing the section provider in FEM-Design, the second moment of inertia for both configurations of Building B is obtained. The stiffness is calculated for each case where $(EI)_{WTT}$ represents the core wall thickness of 400 millimeter, the needed thickness for fulfillment of comfort by wind load from wind tunnel test results. The stiffness $(EI)_{EN}$ represents the configuration of core walls of 600 millimeters that showed to be required for obtaining an approved comfort based on wind load by EN-1991-1-4.

The ratio between these two stiffness configurations are showed below in Equation 7.2. It is observed that the stiffness required when using a wind load assessment based on building standards is 31% higher.

$$\frac{(EI)_{WTT}}{(EI)_{EN}} = 0.69 \tag{7.2}$$

where

$(EI)_{WTT}$	required stiffness based on wind load from wind tunnel test $[-]$
$(EI)_{EN}$	required stiffness based on wind load from EN-1991-1-4 $[-]$

7.3 Building C

Building C is a 70 meters high building with 21 floors. By the Eurocode guidelines, this building is not considered as a high slender building since the slenderness ratio is below 5/1. Of this reason the building is design based on the method explained in Chapter 5.1.1. When conducting the FEM analysis, the core walls are designed with less openings than in reality. Specifically does the entrance floor really consist of significantly fewer core walls.

7.3.1 Wind loads

The dominant wind direction is on the long facade as pictured in Figure 7.5. The structural system illustrated in the figure is used for every floor in the FEM model.

Figure 7.5: *Floor plan of Building C with dominating wind direction.*

Comparing the peak velocity pressure at the top of the building from wind tunnel test against the building code result, there is a significant difference. The building generate an assumption of 1.34 kN/m^2 while the wind tunnel test provide a peak pressure of only 1.25 kN/m^2.

Table 7.9: *Peak wind velocity pressure at the top of the building.*

	$\Sigma q_p(h)$
EN-1991-1-4	1.34 kN/m^2
Wind tunnel test	1.25 kN/m^2

Observing the results of total wind force acting on the building facade, there is a significant difference between the methods by building codes and the wind tunnel testing. Even the analytical methods that investigate each individual floor differ a lot from the wind tunnel test result. The total wind force for each method conducted is presented in Table 7.10.

Table 7.10: *Result and comparison between wind tunnel test and analytical methods.*

Method	ΣF_y	Difference to WTT
Wind tunnel test	3158 kN	
EN-1991-1-4 1A	4709 kN	49 %
EN-1991-1-4 2A	4399 kN	39 %
EN-1991-1-4 1B	3878 kN	23 %
EN-1991-1-4 2B	4052 kN	28 %

7.3.2 Frequency and acceleration

The first FEM analysis of Building C is conducted with a core wall thickness of 400 millimeters. The dynamic analysis generate natural frequencies for mode 1-3, see Table 7.11. The frequencies obtained from this eigenfrequency analysis is observed to be of lower value than the frequencies presented in the wind tunnel test report.

Table 7.11: *Natural frequency from FEM analysis and wind tunnel test.*

Mode	FEM analysis	Wind tunnel test
Mode 1	0.356 Hz	0.615 Hz
Mode 2	0.392 Hz	0.846 Hz
Mode 3	0.423 Hz	0.988 Hz

After iteration of the core wall thickness for Building C, a final configuration of 500 millimeter thickness was reached. Table 7.12 present frequencies for all different configurations along with calculated top acceleration for along-wind motion. Calculations for acceleration are based on the mean wind velocity at the top of the building obtained from building code assessment respectively wind tunnel test results.

Table 7.12: *Natural frequency and acceleration for each configuration.*

Configuration	$n_{1,x}$	$\ddot{X}_{max}(z)$
Wind tunnel test, 400mm	0.356 Hz	0.052 m/s^2
EN-1991-1-4, 400mm	0.356 Hz	0.067 m/s^2
EN-1991-1-4, 500mm	0.407 Hz	0.058 m/s^2
EN-1991-1-4, $n_{1,x}$=46/h Hz	0.660 Hz	0.033 m/s^2

Per ISO:10137:2008 comfort criteria, the relation between the frequency and the top acceleration is evaluated. The different results of frequency and acceleration are inserted in the evaluation diagram, Figure 7.6. As observed, the wind tunnel test results achieve an approved level of comfort since it falls within the limit of residential usage, curve number 2. For the frequency and acceleration obtained from wind load calculations based on Eurocode EN-1991-1-4, the configuration with a core wall thickness of 400 millimeters do not accomplish desired demands of comfort. First at a core wall thickness of 500 millimeters, the demands of comfort for residential use is achieved. As for the natural frequency assumed by theoretical methods, the comfort demand is fulfilled.

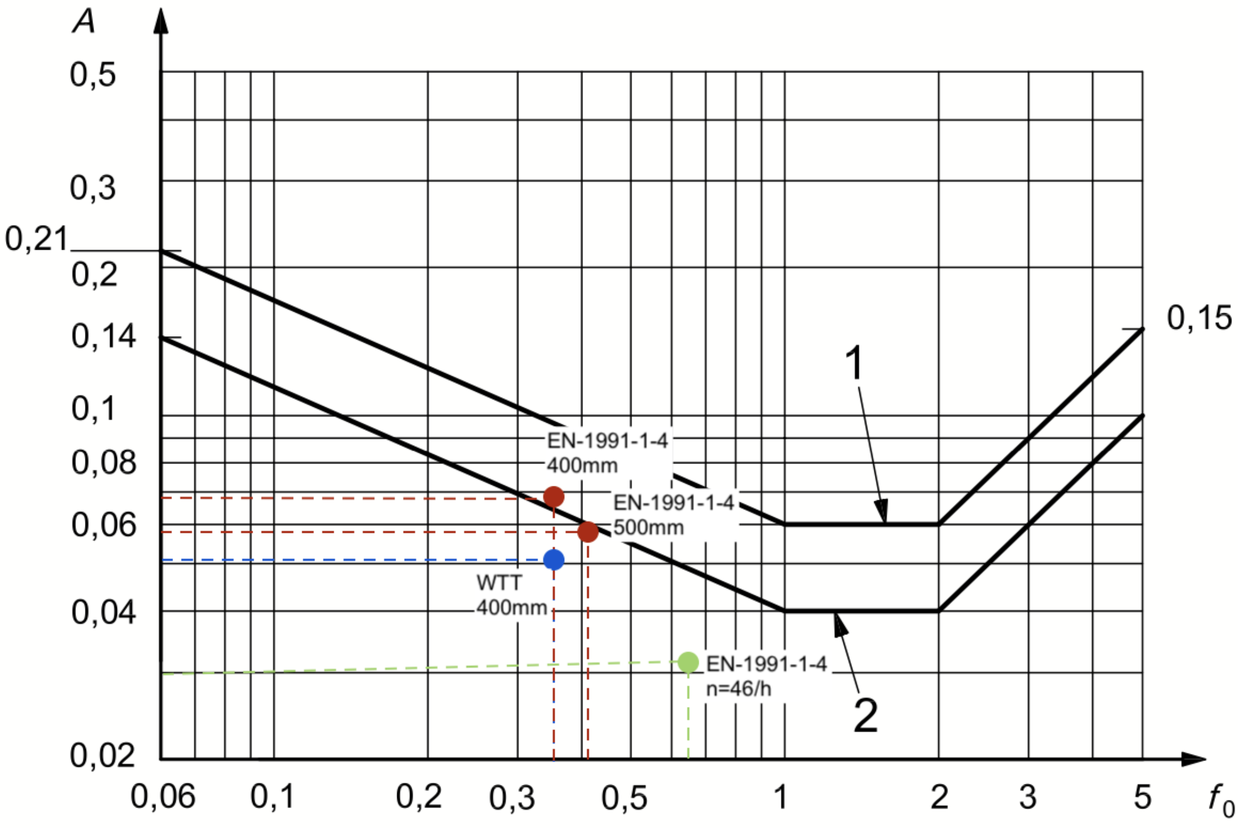


Figure 7.6: *Evaluation curve for comfort demands.*

7.3.3 Stiffness

Once again, the second moment of inertia is generated by the section editor in the program FEM-Design. The stiffness is calculated for each case where $(EI)_{WTT}$ represents the core wall thickness of 400 millimeter, the needed thickness for fulfillment of comfort by wind load from wind tunnel test results. The stiffness $(EI)_{EN}$ represents the configuration of core walls of 500 millimeters that showed to be required for obtaining an approved comfort based on wind load by EN-1991-1-4.

The ratio between these two stiffness configurations are showed below in Equation 7.3. It is observed that the stiffness required when using a wind load assessment based on building standards is 20% higher.

$$\frac{(EI)_{WTT}}{(EI)_{EN}} = 0.80 \tag{7.3}$$

where

$(EI)_{WTT}$	required stiffness based on wind load from wind tunnel test $[-]$
$(EI)_{EN}$	required stiffness based on wind load from EN-1991-1-4 $[-]$

7.4 Guidelines for design of high-rise buildings

Summarizing results from the analytical- and FEM based analysis, there is a pronounced difference between assumed wind load by building standard and wind tunnel tests. It can subsequently be stated as a recommendation to decrease the wind loads significantly after an early assessment of high-rise buildings based on building standards. Stiffness reduction factor for each building height is presented in Table 7.13 and plotted in Figure 7.7.

Table 7.13: *Stiffness reduction factor.*

Building height[m]	Reduction factor
70	0.80
94.7	0.69
111	0.59

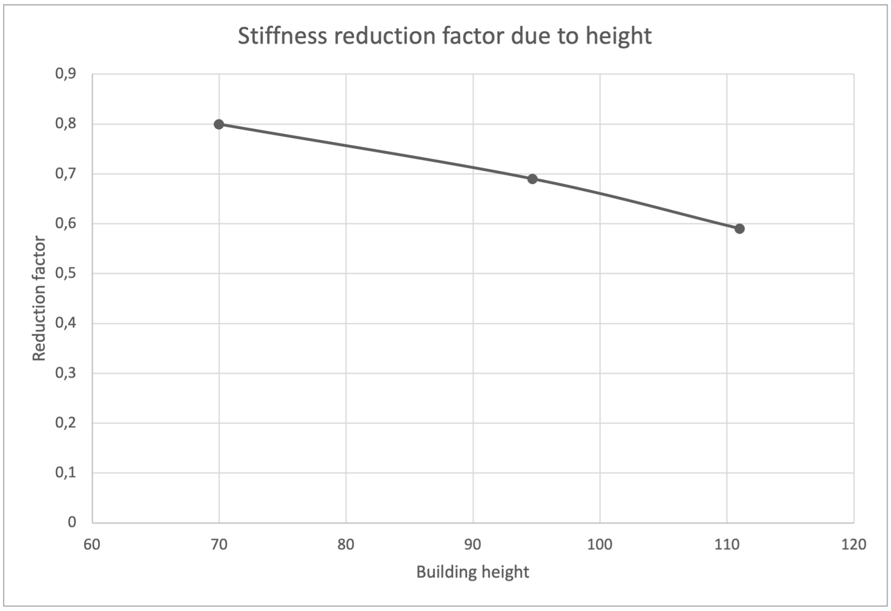


Figure 7.7: *Stiffness reduction factor in relation to building height.*

The stiffness of a tall buildings' structural system, required for comfort criteria demands, shows to be strongly related to the building height. By observing the result of analysis conducted in this thesis, based on three reference buildings of various configurations and design, following guidelines for an early estimation of a structural systems' stiffness in high-rise buildings can be stated;

- For a building of 110 meters and high slenderness ratio, a decrease of the structural systems' stiffness up to 40% can be utilized in design by building standards.

- For a building of 95 meters and high slenderness ratio, a decrease of the structural systems' stiffness up to 30% can be utilized in design by building standards.

- For a building of 70 meters and high slenderness ratio, a decrease of the structural systems' stiffness up to 20% can be utilized in design by building standards.

Based on the three reference buildings, there is shown to be a strong relation between required stiffness reduction and height. By following this trend, predictions of stiffness reduction factors for higher buildings are obtained, see Table 7.14 and Figure 7.8. These predictions are collected by looking at the three reference buildings of this thesis. These stiffness reduction factors are not to be used in design as buildings of these heights needs to be observed in future investigations for development of guidelines. These predictions are gathered to discussed the significant overestimation in the design of high-rise buildings by current building standards.

Table 7.14: *Stiffness reduction factor prediction.*

Building height[m]	Reduction factor
70	0.80
95	0.69
110	0.59
125	0.51
150	0.37
175	0.23
200	0.09

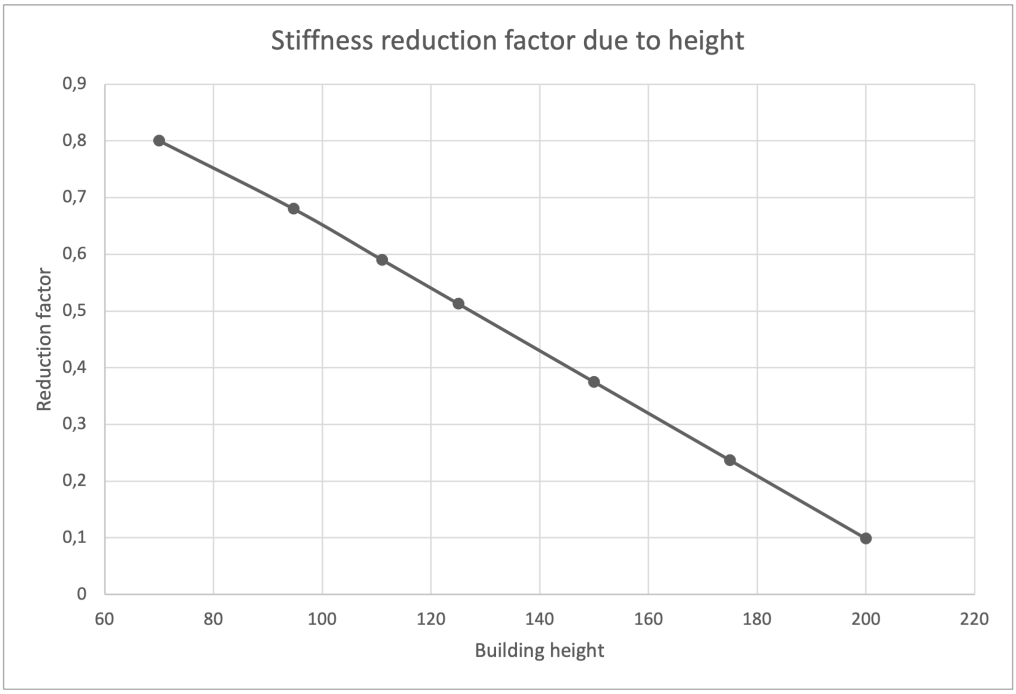


Figure 7.8: *Stiffness reduction factor in relation to building height.*

8

Discussion

In this chapter, the results obtained in this thesis is discussed and evaluated.

8.1 Interview study

The interview study consisted of three interviews with structural engineers specialized in tall buildings. It resulted in a valuable insight of the current approaches for designing high-rise buildings along with clarification of what is actually missing in today's building standards.

Wind is a very difficult phenomenon to predict without using wind tunnel test and since every model is different, so is the resulting wind loads. Since the acceleration primarily becomes a problem for buildings higher than 100m with a high slenderness, there has been a low need for developed guidelines earlier since there is few buildings constructed at this height in Europe. If the building industry keep growing with higher buildings of significant slenderness, current building standards would be in a need of a development. Accelerations governs the design of high-rise buildings and will have an even greater impact on the design for light-weight buildings which is more and more common.

There is a clear mismatch in current building standards regarding the acceleration. The comfort criteria by ISO standard are created for a one-year return period whilst acceleration calculations by the Eurocode European building standards are unable to calculate for one year. It is evident that there is a need of improvements in the interplay between acceleration calculations and comfort criteria evaluation.

The interviews shows that there are a lot of uncertainties in current building code guidelines for the design of high-rise buildings and are in an urgent need of further development. There is still a definite difference between companies methods for design and they are often based on experience as an addition to national building codes. It is showed that, based on these three interviews, the early assessment of high-rise buildings are based on FEM modelling. This method proves to be more efficient and quick rather than prediction based on hand-calculations. Although, it seems that each engineering company sticks to their own scripts and practices in the design of buildings. It would be useful to collect these different approaches and bring forth a general method for an early assessment of high-rise buildings.

8.2 Analytical analysis

The analytical analysis in this thesis was based on the Eurocode European standard (EN-1991-1-4, 2005) together with building recommendations (BFS:2022:4, 2022) and comfort criteria (ISO:10137, 2008). These are methods and guidelines that regularly are developed to match the expanding building industry. The building codes are stated to be at the safe side of design, but often result in a significant overestimation of design. This result in consequences of wasted material along with limitations on what can be built. Observing three reference buildings by following theoretical methods of design by building codes, it is obvious that the current standards result in a conservative outcome in the design of high-rise buildings.

When observing the difference between an analytical assessment and wind tunnel testing results, there is a significant discrepancy in wind load prediction. All calculations on wind loads in this thesis based on analytical methods, showed to be greater than wind loads provided by wind tunnel testing. For the peak wind velocity pressure, building codes results in an estimation of approximately 10 kN/m^2 in every reference case. The building codes estimates a more parabolic shape of wind load profile where the wind loads are increased along the building height. The wind tunnel test show a more equal distribution of wind loads in comparison.

In a comparison between analytical methods, it is shown that wind loads calculations for each floor of the building (method 1B and 2B) are closer to a realistic distribution than methods were the facade is divided into segments (method 1A and 2A). Although, the methods require distinctly different levels of time commitment and detailed analysis. A more simplified guide will, based on the results of this thesis, generate a higher prediction of wind loads and would need to be reduced considerably if used in design.

Observing the acceleration analysis conducted by utilizing the natural frequency assumed by Eurocode methods, the comfort criteria for all reference buildings were fulfilled. Consequently this simplified estimation could be used as a quick estimation of comfort criteria demands.

Not only do the methods stated by the building codes result in an overestimated design. The equations are not tailored to the desired analysis for comfort criteria evaluation. When calculation the along-wind acceleration by Eurocode standards, the equation includes the natural logarithm. When analysing the acceleration for a return period of one year, this equation will subsequently go towards infinity. This goes against to the comfort criteria by ISO standards, which graph are customized for a one-year return period. In this thesis, the return period was chosen to 1.5 years to come as close as possible to a more realistic solution. This alteration bring some deficiency of result but is still efficient for an early assessment of design.

8.3 FEM analysis

FEM analysis is a favorable tool for the design of high-rise buildings. Depending on simplifications made, different considerations needs to be regarded in design. The FEM analysis in this thesis is conducted using simplified interpretation of the structural systems' of provided reference buildings. The outcome would most likely be different if a more detailed translation of the structural system was made.

A simplification of the loads implemented in the FEM modelling was made in an early process of this thesis where the self-weight of the concrete slabs where assumed to have an impact on the core within a five meter distance. Analysing the frequency, a total load impact from the slabs should be used for an accurate prediction. This simplification could the be reason behind the high values of frequency for reference building A since the core are not located in the center of the slabs. Still, the total mass of the buildings are in the same range of values as in the wind tunnel tests and therefore judged as an approved assumption for an early assessment of a buildings stiffness.

The evaluation of the comfort criteria for buildings are not depending on the wind load profile on the structure, but only the mean wind velocity at the reference height. This reference height is assumed to be subjected to the highest wind load. For a more detailed maximum wind load prediction, the total wind profile should be observed.

Three references buildings were investigated in this work. Each buildings with a different height and configuration. The difference made room for observing various methods in design by building standard regulation and brought a comprehensive study to this thesis. In the end, it was still shown that the stiffness prediction depended mainly on the buildings' height.

8.4 Development of guidelines for design

As the mass of the building expands, the natural frequency is lowered while the acceleration is increased. On the other hand, with increased mass, the inertia of the building also grows larger which in its turn decreases the acceleration. Of this reason, the relation between these important parameters can be hard to predict in an early deign phase without conducting several analyses with several iterations. This thesis present the use of stiffness estimation as a guide towards a better approximation in early design of tall buildings.

Developed guidelines for an early assessment of a structural systems' stiffness in high-rise buildings in design for wind shows a distinct connection between building height and estimated stiffness. This thesis provide the recommendation that depending on height of the building and slenderness ratio, reduce the stiffness with appurtenant percent. This method could be a favorable estimation used in the conceptual phase in design of tall buildings where it is important to quickly propose a layout of the floor plan and how much space the stabilizing system will take. By optimizing a buildings structural system in early design a more favorable layout can be developed that strengthen the buildings' construction along with saving material use and time consuming work.

The relation between stiffness reduction factor and building height for slender buildings shows a clear trend. By following this trend for even higher buildings, prediction of stiffness reduction factors where presented. This shows that the higher the building, the bigger reduction of the stiffness can be utilized in design by building codes. This prediction is not meant to be used in design for higher structures since buildings of these heights will behave in different manner than the reference buildings studied in this thesis. The prediction is rather to be used as an understanding of the overestimation that comes with design by building codes for high ans slender structures. To define guidelines for higher and more slender buildings, a more detailed investigation of several buildings should be conducted.

9

Final remarks

9.1 Conclusions

This thesis studied the current design approaches for estimating required stiffness of a high-rise building in an early stage. In current building codes, there are few guidelines to use in the design of a tall structures. Methods presented often result in an overestimation with simple geometries of the building. The aim of this thesis was to develop guidelines to use as an early estimation of structural systems' stiffness for high-rise buildings.

Results of this thesis showed a significant difference between wind loads predicted using building codes and conducted wind tunnel tests. This required an increase i stiffness for the structural system in order to fulfill comfort demands for residential use. By comparing stiffness ratios needed to fulfill each configuration of core wall thickness between wind loads estimated by building standards and wind tunnel test, guidelines for the design of high-rise buildings were propsed.

These new guidelines for an early estimation of a structural systems' stiffness in tall buildings can be used as an advice in the conceptual phase of design. Having a better understanding regarding allowable decrease of stiffness in conceptual design phase, the design of high-rise buildings can be improved significantly with less time-consuming work and better approximations.

9.2 Further investigations

The variety of reference buildings brought a comprehensive investigation to this thesis. In further development of guidelines in design, it would be profitable to narrow down the discrepancy to get a more detailed outcome. To observe more slender buildings of greater heights against each other would push the recommendations further and bring more relevance to guidelines in the design of tall structures.

The Eurocode is limited to buildings below 200 meters in height. As this thesis highest reference building was 111 meter high, further investigation would be recommended to include higher structures. There is an urgency in developing guidelines for tall structures, especially for a height up and exceeding 200 meters as consequences of acceleration will have a greater impact on these heights.

In the development of guidelines for design of tall buildings, further investigations should include the analysis of across-wind acceleration. Especially for slender structures, this type of acceleration is generally dominating the final design choices. Methods for calculating across-wind acceleration would be a desirable extension of current European building standards.

References

Abu-Zidan, Y., Mendis, P., Gunawardena, T., Mohotti, D., & Fernando, S. (2022). Wind Design of Tall Buildings: The State of the Art. *Electronic Journal of Structural Engineering*, *22*(1), 53–71. doi: 10.56748/ejse.2233101

Ascher, K. (2011). *The Heights - Anatomy of a skyscraper* . Penguin Books.

BFS:2022:4. (2022). *Boverkets föreskrifter och allmänna råd (2011:10) om tillämpning av europeiska konstruktionsstandarder (eurokoder)* (Tech. Rep.).

Boverket. (1997). *Boverkets handbok om snö- och vindlast, utgåva 2* (2nd ed., Vol. 1:1; K. Nero & S. Åkerlund, Eds.). Karlskrona: Boverket, byggavdelningen.

Caldenby, C. (1990). *Höga Hus i Göteborg; Varför, var och hur samt inte minst om.* Göteborg: Stadsbyggnadskontoret.

EN-1991-1-4. (2005). Eurocode 1: Actions on structures -Part 1-4: General actions -Wind actions. *European Committee for Standardization*, *4*(2005).

Hughes, A. F. (2014). Wind actions to bs EN 1991-1-4. (SCI P394).

Irwin, P., Denoon, R., & Scott, D. (2013). *Wind Tunnel Testing of High-Rise Buildings* (Tech. Rep.). Retrieved from `http://www.ctbuh.org`

Isaksson, T., Mårtensson, A., & Thelandersson, S. (2020). *Byggkonstruktion* (4:1 ed.). Lund: Studentlitteratur AB.

ISO:10137. (2008). *SVENSK STANDARD Grundläggande dimensioneringsregler för bärverk-Byggnaders samt gång-och cykelbroars brukbarhet med hänsyn till svängningar och vibrationer (ISO 10137:2007, IDT) Bases for design of structures-Serviceability of buildings and walkways against vibration (ISO 10137:2007, IDT)* (Tech. Rep.). Retrieved from `www.sis.se`

Kwok, K. C., Hitchcock, P. A., & Burton, M. D. (2009). Perception of vibration and occupant comfort in wind-excited tall buildings. *Journal of Wind Engineering and Industrial Aerodynamics*, *97*(7-8). doi: 10.1016/j.jweia.2009.05.006

Mendis, P., Ngo, T., Haritos, N., Hira, A., Samali, B., & Cheung, J. (2007). Wind loading on tall buildings. *Electronic Journal of Structural Engineering*, *7*, 41–54. doi: 10.56748/ejse.641

Sarkisian, M. (2016). *Designing Tall Buildings.* doi: 10.4324/9781315714639

serneke.se. (230515). *https://www.serneke.se/projekten/karlatornet/.*

Thordal, M. S., Bennetsen, J. C., & Koss, H. H. H. (2019). Review for practical application of CFD for the determination of wind load on high-rise buildings. *Journal of Wind Engineering and Industrial Aerodynamics*, *186*. doi: 10.1016/j.jweia.2018.12.019

Truby, A. (2014). *Tall buildings : structural design of concrete buildings up to 300m tall.*

References of figures

Figure 2.1: $https://www.facebook.com/BKNDesignCenter/photos/a.418792-908195367/3737972859610672/?type=3$

Figure 2.2: Mendis et al. (2007)

Figure 2.3: Mendis et al. (2007)

Figure 2.4: Irwin et al. (2013)

Figure 2.5: $https://mcgrawimages.buildingmedia.com/CE/CE_images/2013/Sep_GSedit-10.jpg$

Figure 3.1: $https://www.sernekebostad.se/hitta-bostad/-vara-omraden/karlastaden-goteborg/karlatornet/?gc-lid=Cj0KCQiAx6ugBhCcARIsAGNmMbiSMyUYy9NdjdZXuI0TUl7B7ygDJ6hjXjU1FvYrkX4hEooyOj8Z44aAou5EALw_wcBgallery$

Figure 3.2: $https://www.svd.se/a/Albqpj/spana-in-utsikten-fran-vaning-33-i-norra-tornen$

Figure 3.3: $https://www.youtube.com/watch?v=FD9R-Usb_GM$

Figure 4.1-4.9: Provided by Sweco Sverige AB

Figure 4.10-4.12: Provided by Sweco Sverige AB with modifications

Figure 5.1-5.13: EN-1991-1-4 (2005)

Figure 5.14: EN-1991-1-4 (2005) with modifications

Figure 5.15: ISO:10137 (2008)

Figure 6.1-6.10: FEM-analysis in RFEM

Figure 7.1: Provided by Sweco Sverige AB with modifications

Figure 7.2: ISO:10137 (2008) with modifications

Figure 7.3: Provided by Sweco Sverige AB with modifications

Figure 7.4: ISO:10137 (2008) with modifications

Figure 7.5: Provided by Sweco Sverige AB with modifications

Figure 7.6: ISO:10137 (2008) with modifications

A

Appendix Building A

A.1 Wind loads

Table A.1: *Properties of Building A.*

Building height	111 m
Building width	15 m
Building length	22-48 m
c_{dir}	1.0
c_{season}	1.0
$v_{b,0}$	24 m/s
v_b	24 m/s
q_b	360 kg/ms^2
z_0	0.05 m
z_{min}	2 m
k_r	0.19
k_l	1.0
m_e	193332 kg
ϕ	1.0
λ	3.24
ψ_λ	0.65
B^2	0.95
R^2	0.41
$c_s c_d$	1.13
$n_{1,x}$	0.41 Hz
ν	0.23
F	0.08
ϕ_b	0.36
ϕ_h	0.36
δ_s	0.10
δ_a	0.02
y_c	1.17
k_p	3.33
T	600 s

Table A.2: *Wind properties at height z, Building A.*

z	111 m
$c_0(z)$	1.0
$c_r(z)$	1.46
$v_m(z)$	35.14 m/s
$\sigma_v(z)$	4.56 m/s
$l_v(z)$	0.13
$q_p(z)$	1.47 kPa

Table A.3: *Wind properties at reference height* $z_s, BuildingA$.

z_s	66.6 m
$c_0(z_s)$	1.0
$c_r(z_s)$	1.37
$v_m(z_s)$	32.81 m/s
$\sigma_v(z_s)$	4.56 m/s
$l_v(z_s)$	0.14
$q_p(z_s)$	1.33 kPa

Table A.4: *Wind loads for segmented division per EN-1991-1-4 method 1A, Building A.*

Level [m]	$F_w(z)$ [kN] EN 1A
111.00	2594.18
60.30	290.45
57.15	323.95
54.00	322.66
50.85	290.77
47.70	5442.48
$\sum$	9264.48

Table A.5: *Wind loads for segmented division per EN-1991-1-4 method 2A, Building A.*

Level [m]	$F_w(z)$ [kN] EN 2A
111.00	2587.57
60.30	247.60
57.15	274.18
54.00	271.10
50.85	242.49
47.70	4506.53
$\sum$	8129.46

Table A.6: *Wind loads on each floor per EN-1991-1-4 method 1B and wind tunnel test, Building A.*

Floor	Level [m]	$F_w(z)$ [kN] EN 1B	$F_w(z)$ [kN] WTT
Roof	111.00		
P34	107.55	167.80	119
P33	104.40	159.23	109
P32	101.25	158.26	109
P31	98.10	157.26	109
P30	94.95	180.38	109
P29	91.80	203.14	157
P28	88.65	201.71	157
P27	85.50	200.23	157
P26	82.35	198.70	160
P25	79.20	197.12	160
P24	76.05	211.77	160
P23	72.90	226.07	192
P22	69.75	224.01	192
P21	66.60	221.86	192
P20	63.45	219.61	192
P19	60.30	232.78	192
P18	57.15	260.19	192
P17	54.00	271.58	215
P16	50.85	268.08	215
P15	47.70	264.39	215
P14	44.55	260.46	215
P13	41.40	271.18	215
P12	38.25	281.08	230
P11	35.10	275.69	230
P10	31.95	269.85	230
P09	28.80	263.47	230
P08	25.65	256.43	230
P07	22.50	248.56	230
P06	19.35	239.63	230
P05	16.20	229.30	230
P04	13.05	216.99	230
P03	9.00	195.38	230
P02	6.75	168.03	230
P01	3.60	147.00	230
P00	0.00	64.10	263
$\sum$		8129.46	6554

Table A.7: *Wind loads on each floor per EN-1991-1-4 method 2B and wind tunnel test, Building A.*

Floor	Level [m]	$F_w(z)$ [kN] EN 2B	$F_w(z)$ [kN] WTT
Roof	111.00		
P34	107.55	167.60	119
P33	104.40	159.19	109
P32	101.25	158.40	109
P31	98.10	157.61	109
P30	94.95	180.61	109
P29	91.80	203.24	157
P28	88.65	201.69	157
P27	85.50	200.14	157
P26	82.35	198.08	160
P25	79.20	197.05	160
P24	76.05	211.79	160
P23	72.90	225.67	192
P22	69.75	223.87	192
P21	66.60	222.06	192
P20	63.45	219.66	192
P19	60.30	232.12	192
P18	57.15	259.51	192
P17	54.00	271.10	215
P16	50.85	268.02	215
P15	47.70	264.93	215
P14	44.55	260.31	215
P13	41.40	271.37	215
P12	38.25	281.13	230
P11	35.10	275.11	230
P10	31.95	268.23	230
P09	28.80	263.93	230
P08	25.65	256.19	230
P07	22.50	245.88	230
P06	19.35	239.00	230
P05	16.20	227.82	230
P04	13.05	215.79	230
P03	9.00	193.22	230
P02	6.75	166.58	230
P01	3.60	145.23	230
P00	0.00	63.95	263
$\sum$		7596.08	6554

A.2 Acceleration

Table A.8: *Acceleration data for wind load based on EN-1991-1-4, Building A.*

EN-1991-1-4	$n_{1,x} = 0.247$	$n_{1,x} = 0.318$	$n_{1,x} = 0.410$
v_{50}	32.81 m/s	32.81 m/s	32.81
T_a	1.5	1.5	1.5
vTa	24.375 m/s	24.375 m/s	24.375 m/s
ξ	1.5	1.5	1.5
K_x	1.622	1.622	1.622
L(z)	169.31 m	169.31 m	169.31 m
$S_L(z, n_{1,x})$	0.09	0.078	0.067
ν	0.134 $1/s$	0.141 $1/s$	0.143 $1/s$
f_L	1.716	2.209	2.848
δ_d	0.00	0.00	0.00
δ_a	0.021	0.017	0.013
δ_s	0.10	0.10	0.10
δ	0.121	0.117	0.113
B^2	0.536	0.536	0.536
η_h	5.174	6.661	8.589
η_b	2.237	2.881	3.714
R_h	0.175	0.139	0.11
R_b	0.348	0.287	0.233
R^2	0.223	0.131	0.075
R	0.472	0.362	0.273
$\phi_{1,x}$	1.00	1.00	1.00
k_p	3.164	3.181	3.186
σ_a	0.027 m/s^2	0.021 m/s^2	0.016 m/s^2
X_a	0.086 m/s^2	0.067 m/s^2	0.05 m/s^2

Table A.9: *Acceleration data for wind load based on wind tunnel test, Building A.*

Wind tunnel test	$n_{1,x} = 0.247$
v_{50}	32.81 m/s
T_a	1.5
vTa	24.375 m/s
ξ	1.5
K_x	1.622
L(z)	169.31 m
$S_L(z, n_{1,x})$	0.09
ν	0.134 $1/s$
f_L	1.716
δ_d	0.00
δ_a	0.021
δ_s	0.10
δ	0.121
B^2	0.536
η_h	5.174
η_b	2.237
R_h	0.175
R_b	0.348
R^2	0.223
R	0.472
$\phi_{1,x}$	1.00
k_p	3.164
σ_a	0.027 m/s^2
X_a	0.086 m/s^2

A.3 Stiffness

Table A.10: *Stiffness properties Building A.*

E	25 GPa
I_{400mm}	8.31e14 mm^4
I_{700mm}	1.42e15 mm^4
EI_{400mm}	2.08e13 Nm^2
EI_{700mm}	3.55e13 Nm^2

B

Appendix Building B

B.1 Wind loads

Table B.1: *Properties of Building B.*

Building height	94.7 m
Building width	51 m
Building length	51 m
c_{dir}	1.0
c_{season}	1.0
$v_{b,0}$	24 m/s
v_b	24 m/s
q_b	360 kg/ms^2
z_0	0.05 m
z_{min}	2 m
k_r	0.19
k_l	1.0
m_e	246000 kg
ϕ	1.0
λ	1.3
ψ_λ	0.63
B^2	0.94
R^2	0.33
$c_s c_d$	1.12
$n_{1,x}$	0.49 Hz
ν	0.25
F	0.78
ϕ_b	0.36
ϕ_h	0.27
δ_s	0.10
δ_a	0.01
y_c	2.12
k_p	3.35
T	600 s
ν_k	1.5e-05 m^2/s
v_c	1.51 m/s
Re	5.14e06
k	100 mm

Table B.2: *Wind properties at height z, Building B.*

z	94.7 m
$c_0(z)$	1.0
$c_r(z)$	1.43
$v_m(z)$	34.41 m/s
$\sigma_v(z)$	4.56 m/s
$l_v(z)$	0.13
$q_p(z)$	1.43 kPa

Table B.3: *Wind properties at reference height* $z_s, Building B.$

z_s	56.82 m
$c_0(z_s)$	1.0
$c_r(z_s)$	1.34
$v_m(z_s)$	32.08 m/s
$\sigma_v(z_s)$	4.56 m/s
$l_v(z_s)$	0.14
$q_p(z_s)$	1.28 kPa

Table B.4: *Wind loads for segmented division per EN-1991-1-4 method 1A, Building B.*

Level [m]	$F_w(z)$ [kN] EN 1A
94.70	3103.15
51.00	3777.22
$\sum$	6880.37

Table B.5: *Wind loads for segmented division per EN-1991-1-4 method 2A, Building B.*

Level [m]	$F_w(z)$ [kN] EN 1A
94.70	3100.93
51.00	3180.27
$\sum$	6281.20

Table B.6: *Wind loads on each floor per EN-1991-1-4 method 1B, Building B.*

Floor	Level [m]	$F_w(z)$ [kN] EN 1B
Roof	94.70	110.07
P29	91.60	218.66
P28	88.50	217.14
P27	85.40	215.57
P26	82.30	213.95
P25	79.20	212.28
P24	76.10	210.54
P23	73.00	208.74
P22	69.90	206.86
P21	66.80	204.91
P20	63.70	202.88
P19	60.60	200.75
P18	57.50	198.53
P17	54.40	196.19
P16	51.30	193.73
P15	48.20	191.13
P14	45.10	188.37
P13	42.00	185.44
P12	38.90	182.30
P11	35.80	178.93
P10	32.70	175.29
P09	29.60	171.32
P08	26.50	166.96
P07	23.40	162.12
P06	20.30	156.66
P05	17.20	150.39
P04	14.10	143.03
P03	11.00	134.05
P02	7.90	132.34
P01	4.30	130.37
P00	0.00	54.84
$\sum$		5514.34

Table B.7: *Wind loads on each floor per EN-1991-1-4 method 2B, Building B.*

Floor	Level [m]	$F_w(z)$ [kN] EN 1B
Roof	94.70	131.28
P29	91.60	246.48
P28	88.50	224.20
P27	85.40	212.05
P26	82.30	210.41
P25	79.20	208.77
P24	76.10	207.14
P23	73.00	204.95
P22	69.90	203.31
P21	66.80	201.67
P20	63.70	198.94
P19	60.60	197.30
P18	57.50	195.11
P17	54.40	192.38
P16	51.30	190.19
P15	48.20	188.01
P14	45.10	185.27
P13	42.00	181.99
P12	38.90	178.72
P11	35.80	176.53
P10	32.70	172.16
P09	29.60	169.42
P08	26.50	163.96
P07	23.40	158.49
P06	20.30	154.67
P05	17.20	148.11
P04	14.10	141.01
P03	11.00	143.10
P02	7.90	139.95
P01	4.30	126.70
P00	0.00	54.71
$\sum$		5506.97

B.2 Acceleration

Table B.8: *Acceleration data for wind load based on EN-1991-1-4, Building B.*

EN-1991-1-4	$n_{1,x} = 0.222$	$n_{1,x} = 0.265$	$n_{1,x} = 0.490$
v_{50}	32.08 m/s	32.08 m/s	32.08
T_a	1.5	1.5	1.5
vTa	23.833 m/s	23.833 m/s	23.833 m/s
ξ	1.5	1.5	1.5
K_x	1.623	1.623	1.623
L(z)	155.89 m	155.89 m	155.89 m
$S_L(z, n_{1,x})$	0.099	0.090	0.062
ν	0.135 $1/s$	0.143 $1/s$	0.149 $1/s$
f_L	1.452	1.733	3.205
δ_d	0.00	0.00	0.00
δ_a	0.019	0.016	0.011
δ_s	0.10	0.10	0.10
δ	0.119	0.116	0.109
B^2	0.537	0.537	0.537
η_h	5.174	4.844	8.956
η_b	4.058	2.609	4.823
R_h	0.216	0.185	0.105
R_b	0.354	0.310	0.186
R^2	0.313	0.218	0.055
R	0.560	0.467	0.235
$\phi_{1,x}$	1.00	1.00	1.00
k_p	3.166	3.184	3.199
σ_a	0.027 m/s^2	0.022 m/s^2	0.011 m/s^2
X_a	0.084 m/s^2	0.071 m/s^2	0.036 m/s^2

Table B.9: *Acceleration data for wind load based on wind tunnel test, Building B.*

Wind tunnel test	$n_{1,x} = 0.222$
v_{50}	28.64 m/s
T_a	1.5
vTa	21.277 m/s
ξ	1.5
K_x	1.623
L(z)	155.887 m
$S_L(z, n_{1,x})$	0.093
ν	0.125 $1/s$
f_L	1.626
δ_d	0.00
δ_a	0.017
δ_s	0.10
δ	0.117
B^2	0.537
η_h	4.545
η_b	2.448
R_h	0.196
R_b	0.326
R^2	0.249
R	0.499
$\phi_{1,x}$	1.00
k_p	3.143
σ_a	0.021 m/s^2
X_a	0.067 m/s^2

B.3 Stiffness

Table B.10: *Stiffness properties Building B.*

E	25 GPa
I_{400mm}	1.32e16 mm^4
I_{600mm}	1.92e16 mm^4
EI_{400mm}	3.29e14 Nm^2
EI_{600mm}	4.79e14 Nm^2

C

Appendix Building C

C.1 Wind loads

Table C.1: *Properties of Building C.*

Building height	70 m
Building width	21 m
Building length	34 m
c_{dir}	1.0
c_{season}	1.0
$v_{b,0}$	24 m/s
v_b	24 m/s
q_b	360 kg/ms^2
z_0	0.05 m
z_{min}	2 m
k_r	0.19
k_l	1.0
m_e	239457 kg
$c_{pe,D}$	0.8
$c_{pe,E}$	-0.62
$c_{pe,reduce}$	0.94
B^2	0.95
R^2	0.28
$c_s c_d$	1.12
$n_{1,x}$	0.66 Hz
ν	0.31
F	0.06
ϕ_b	0.32
ϕ_h	0.26
δ_s	0.10
δ_a	0.01
y_c	2.98
k_p	3.42
T	600 s

Table C.2: *Wind properties at height z, Building C.*

z	70 m
$\mathrm{c}_0(z)$	1.0
$\mathrm{c}_r(z)$	1.38
$\mathrm{v}_m(z)$	33.03 m/s
$\sigma_v(z)$	4.56 m/s
$\mathrm{l}_v(z)$	0.14
$\mathrm{q}_p(z)$	1.34 kPa

Table C.3: *Wind properties at reference height* $z_s, Building C.$

z_s	42 m
$\mathrm{c}_0(z_s)$	1.0
$\mathrm{c}_r(z_s)$	1.28
$\mathrm{v}_m(z_s)$	30.70 m/s
$\sigma_v(z_s)$	4.56 m/s
$\mathrm{l}_v(z_s)$	0.15
$\mathrm{q}_p(z_s)$	1.20 kPa

Table C.4: *Wind loads on each floor per EN-1991-1-4 method 1A and wind tunnel test, Building C.*

Level [m]	$F_w(z)$ [kN] EN 1A
70.00	2311.39
36.63	140.76
33.30	2256.97
$\sum$	4709.12

Table C.5: *Wind loads on each floor per EN-1991-1-4 method 2A and wind tunnel test, Building C.*

Level [m]	$F_w(z)$ [kN] EN 2A
70.00	2308.22
36.63	117.89
33.30	1973.16
$\sum$	4399.27

Table C.6: *Wind loads on each floor per EN-1991-1-4 method 1B and wind tunnel test, Building C.*

Floor	Level [m]	$F_w(z)$ [kN] EN 1B	$F_w(z)$ [kN] WTT
Roof	70.00	115.57	100.779
P20	66.60	226.39	195.5708
P19	63.27	221.64	199.2173
P18	59.94	219.12	188.4294
P17	56.61	216.47	168.3109
P16	53.28	213.68	191.574
P15	49.95	210.72	180.0747
P14	46.62	207.58	168.2743
P13	43.29	204.23	132.5711
P12	39.96	200.64	156.8014
P11	36.63	196.77	152.218
P10	33.30	192.57	149.296
P09	29.97	187.97	130.2978
P08	26.64	182.88	134.8137
P07	23.31	177.19	141.8272
P06	19.98	170.72	136.7584
P05	16.65	163.20	117.6434
P04	13.32	154.20	132.2449
P03	9.99	142.91	115.72
P02	6.66	127.62	117.0094
P01	3.33	103.15	141.6616
P00	0.00	43.25	7.1727
$\sum$		3878.49	3158.266

Table C.7: *Wind loads on each floor per EN-1991-1-4 method 2B and wind tunnel test, Building C.*

Floor	Level [m]	$F_w(z)$ [kN] EN 2B	$F_w(z)$ [kN] WTT
Roof	70.00	115.41	100.779
P20	66.60	410.22	195.5708
P19	63.27	221.21	199.2173
P18	59.94	218.78	188.4294
P17	56.61	216.95	168.3109
P16	53.28	213.31	191.574
P15	49.95	211.48	180.0747
P14	46.62	206.62	168.2743
P13	43.29	203.58	132.5711
P12	39.96	200.55	156.8014
P11	36.63	196.90	152.218
P10	33.30	192.04	149.296
P09	29.97	187.78	130.2978
P08	26.64	183.53	134.8137
P07	23.31	176.24	141.8272
P06	19.98	170.77	136.7584
P05	16.65	161.65	117.6434
P04	13.32	153.14	132.2449
P03	9.99	143.42	115.72
P02	6.66	124.58	117.0094
P01	3.33	100.88	141.6616
P00	0.00	43.15	7.1727
$\sum$		4052.20	3158.266

C.2 Acceleration

Table C.8: *Acceleration data for wind load based on EN-1991-1-4, Building B.*

EN-1991-1-4	$n_{1,x} = 0.356$	$n_{1,x} = 0.407$	$n_{1,x} = 0.660$
v_{50}	30.81 m/s	30.81 m/s	30.81
T_a	1.5	1.5	1.5
vTa	22.889 m/s	22.889 m/s	22.889 m/s
ξ	1.5	1.5	1.5
K_x	1.624	1.624	1.624
L(z)	133.208 m	133.208 m	133.208 m
$S_L(z, n_{1,x})$	0.075	0.090	0.055
ν	0.185 $1/s$	0.190 $1/s$	0.191 $1/s$
f_L	2.072	2.369	3.841
δ_d	0.00	0.00	0.00
δ_a	0.011	0.011	0.011
δ_s	0.10	0.10	0.10
δ	0.111	0.111	0.106
B^2	0.565	0.565	0.565
η_h	5.008	5.726	9.285
η_b	2.433	2.781	4.510
R_h	0.180	0.159	0.102
R_b	0.327	0.295	0.197
R^2	0.210	0.157	0.052
R	0.459	0.397	0.228
$\phi_{1,x}$	1.00	1.00	1.00
k_p	3.265	3.273	3.275
σ_a	0.020 m/s^2	0.018 m/s^2	0.010 m/s^2
X_a	0.067 m/s^2	0.058 m/s^2	0.033 m/s^2

Table C.9: *Acceleration data for wind load based on wind tunnel test, Building C.*

Wind tunnel test	$n_{1,x} = 0.365$
v_{50}	$27.53\ m/s$
T_a	1.5
vTa	$20.452\ m/s$
ξ	1.5
K_x	1.624
L(z)	$133.208\ m$
$S_L(z, n_{1,x})$	0.076
ν	$0.169\ 1/s$
f_L	2.319
δ_d	0.00
δ_a	0.011
δ_s	0.10
δ	0.110
B^2	0.565
η_h	5.605
η_b	2.722
R_h	0.163
R_b	0.300
R^2	0.165
R	0.406
$\phi_{1,x}$	1.00
k_p	3.237
σ_a	$0.016\ m/s^2$
X_a	$0.052\ m/s^2$

C.3 Stiffness

Table C.10: *Stiffness properties Building A.*

E	$25\ GPa$
I_{400mm}	$4.21e15\ mm^4$
I_{6500mm}	$5.28e15\ mm^4$
EI_{400mm}	$1.05e14\ Nm^2$
EI_{500mm}	$1.32e14\ Nm^2$

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