The influence of changes in the applied wind load since TGB 1955 on existing buildings for additional layers

Master thesis

J. Kloet

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Julia Kloet

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Thesis committee:	Ir. S. Pasterkamp (chair)	Delft University of Technology
	Dr.ir. P. van der Male	Delft University of Technology
	Ir. H. Alkisaei	Delft University of Technology
	Ing. R. Alting MSEng RC	Pieters Bouwtechniek Delft
	Ir. S. van Eck	Pieters Bouwtechniek Delft
	Prof.dr.ir. R.D.J.M Steenbergen	TNO & Ghent University

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This master thesis marks the final step towards completion of my Master of Science in Civil Engineering at the faculty of Civil Engineering and Geosciences at Delft University of Technology. It has been written in collaboration with Pieters Bouwtechniek; an engineering consultancy in construction advice. The topic of this thesis originates from my passion about existing buildings and how these can be transformed to contribute to solve the current housing crisis. Over the past decades, design codes for wind load design have undergone significant revisions. My research aims to obtain insight the influence of those developments on existing buildings when adding levels and to provide practical recommendations for engineers and architects.

I would like to express my gratitude towards the members of my thesis committee; From Delft university: Pim van der Male, Hoessein Alkisaei and especially Sander Pasterkamp, who was also the chair of my committee. I would like to thank them for their enthusiasm and counsel throughout the course of my graduation project. I would also like to thank Raphaël Steenbergen from TNO, especially on his guidance regarding the probabilistic part of my thesis.

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I hope this thesis serves as a valuable contribution to the field of civil engineering and inspires future research into this topic.

Julia Kloet
Delft, 2024

EXECUTIVE SUMMARY

The Netherlands is currently experiencing a housing crisis, with projections indicating that the housing shortage will continue to increase. Some of the causes area urbanisation and the lack of available land to build. An interesting alternative is adding building levels to existing buildings to increase the number of dwellings. A challenge in modifying existing buildings this way is ensuring the structural stability in this transformed building, since the existing part is often designed using older design codes and the new top level must be designed with the newest Eurocode. In practice, this concept of adding levels is already applied for many cases. However, an overview of the exact changes in wind loads and what kind of influence this has on adding new levels to an existing building is missing. Additionally, it is unknown what the exact gap is between the building or location specific wind load and the wind load according to the design codes. The objective of this research project is therefore to provide insight into the changes in the design code for wind loads and what kind of influence these changes have on the application of additional levels on existing buildings. This objective leads to the following question: "What is the influence of changes in the applied wind load since the TGB 1955 building design code on existing buildings when applying additional levels and how representative is the wind load that is prescribed by these design codes in the view of optimal designing for adding building levels?"

First, the development of the design codes, TGB 1955 to the Eurocode, is analysed. This provides background information and identifies the key parameters that influence the wind load. Furthermore, the location and building specific wind load is compared to the design codes. To do so, the wind load is divided into the wind pressure and pressure coefficients. The location specific wind pressure is determined by analysing measurement data by the KNMI. The pressure coefficients for specific building types are obtained using computational fluid dynamics (CFD) simulations. Furthermore, the influence of the changed wind load on the bearing structure is evaluated for the existing building and its new top level. This is followed by a case study on the SCYE010 building. This is a former office building that is turned into apartments and a new optimized top level is designed for it. The building is located at Schiedam, which is in the province of Zuid-Holland.

Overall, it can be concluded that the design codes for wind loads have increased in complexity over the years. The prescribed wind pressure did increase for most cases as well. Additionally, the number of coefficients increased, as well as the intricacy of the formulas to determine them. The wind areas have been analysed and critical locations for building transformations are determined.

The research on the building and location specific wind speed in comparison with the design codes, resulted into two main findings:

1) Overall, the wind speeds and wind pressures that resulted from the analysis of the KNMI measurements are in line with the prescribed values from the design codes. However, two weather stations deviate from the expected outcome: Hoek van Holland and Vlissingen. Both are located along the coast in wind area II, but showed wind speeds that were higher than De Kooy, which is located in wind area I and is on the coast line as well. Therefore, a revision of the wind area map is suggested.

2) The pressure coefficients in the design codes are assumed to be constant. However, the outcome of the CFD models and validation with existing wind tunnel experiments showed that the pressure coefficients are not constant over the surfaces of the building subjected to wind load. Furthermore, the width of the building seems to have significant influence on the distribution of the pressure coefficient over this surfaces. However, in the current Eurocode, only the height/depth ratio is taken into account.

By using the research findings, the general influence on the bearing structure when adding a level is determined. Stability systems and vertical load bearing systems are analysed, including the foundation. The general conclusion from the research is that the total wind loads will increase, due to both the increase in height and due to the increase in wind load that is prescribed by the design codes. For the top level a new bearing structure need to be designed that is connected to the existing building. By assessing the building, the overcapacity in the existing building has to be defined and if necessary, strengthening can be applied. The goal is to optimally use the substructure of the building to support the new top levels.

Finally, in the case study, all of the research findings are combined to design an optimized top level. Two optimized designs are developed; The first one with the original layout of the top level, but with an optimized location. The second one has an optimized volume of the top level. This second design resulted in the lowest additional loads on the existing structure and is therefore chosen to be the best fit. This was determined by evaluating the forces on the foundation. The capacity of the foundation piles is based on the information that is provided by the archive drawings and calculations. An interesting additional finding from this case study was the influence of the geometrical deviation of the top level compared to the existing building. In this case, a set back was applied and this resulted in a lower wind pressure on the top level. This set back was optimized to minimize the loads on the top level. Furthermore, for the new top level, an new stability system is designed.

All of these findings provide an improved understanding of the influence of the development of the design codes for wind load, especially for the situation where level(s) are being added to an existing building. The influence is built up from many elements and this research has highlighted the developments that can be critical for designing additional top levels for existing buildings, but the opportunities as well.

CONTENTS

ACRONYMS

CC	Consequence Class
CDF	Cumulative Density function
CFD	Computational Fluid Dynamics
FEA	Finite Element Analysis
FEM	Finite Element Model / Finite Element Method
KNMI	Koninklijk Nederlands Meteorologisch Instituut (EN: Royal Dutch Meteorological Institute)
LES	Large Eddy simulations
LSM	Least squares method
MLM	Maximum Likelyhood Method
MOM	Method of Moments
NEN	NEderlandse Norm (EN: Dutch design code)
POT	Peak Over Threshold
RANS	Reynolds-Averaged Navier-Stokes
SI	Système International d’unités (EN: International System of Units)
TGB	Technische Grondslagen voor Bouwvoorschriften (EN: Technical base for building regulations)
UC	Unity Check

LIST OF SYMBOLS

Topic	Symbol	Explanation	Unit
Dimensions	h	Height	[m]
	b	Breadth = width	[m]
	d	Depth	[m]
Wind load	q_w	Wind load	[kN/m^2]
	q_p	Wind pressure (NL: stuwdruk)	[kN/m^2]
	ρ	Density	[kg/m^3]
	$I(z)$	Turbulence intensity at height z	[-]
Coefficients	c_p (=c_{pe})	External pressure coefficient	[-]
	c_{fr}	Friction coefficient	[-]
	c_{pi} (=c_o)	Internal pressure coefficient	[-]
	c_sc_d	Structural factor	[-]
	c_{dim}	Dimension factor	[-]
	c_t	Combination of coefficients	[-]
	ϕ	Dynamic factor	[-]
	p	Pressure	[kN/m^2]
TGB 1955	v	Wind speed	[m/s]
TGB 1972	g	Gust influence	[-]
	r	Roughness factor	[-]
	v_u	Wind speed	[m/s]
	f	Variation coefficient	[-]
	T_b	Influence of a wind gust	[-]
	T_r	-	[-]
	a	Distance to the coast (for determining the wind area)	[m]
	v_{u10}	Base wind speed	[m/s]
TGB 1990	v_w	Wind speed	[m/s]
	u_*	Friction speed	[m/s]
	z_0	Roughness length	[m]
	d	Displacement height	[m]
	h_m	Average building height	[m]
	α	Building density	[-]
	B	-	[-]
	E	-	[-]
	a	Vibration acceleration	[m/s^2]
	κ	Von Karman constant = 0.4	[-]
Eurocode	v_m	Mean wind speed	[m/s]
	k_I	Turbulence factor	[-]
	c_o (z)	Orography factor	[-]
	c_r (z)	Roughness factor	[-]
	v_b	Base wind speed	[m/s]
	z_{min}	Minimal height	[m]
	k_r	Terrain factor	[-]
	z_s	60% of total building height	[m]
	R	Resonance factor	[-]

Topic	Symbol	Explanation	Unit
Statistics	$F(Y)$	Cumulative chance Y is smaller than y	[-]
	μ	Location parameter (Gumbel)	
	μ	Mean of a dataset	
	σ	Scale parameter (Gumbel)	
	σ	Standard deviation of a dataset	
Statistics	γ	Euler's constant = 0.577… (Havil, 2010)	
	$\bar{y}$	Mean of the dataset	
	k	Shape parameter (Weibull)	
	λ	Scale parameter (Weibull)	
	R	Repetition time	[nr of years]
General	z	Height	[m]
	h	Height	[m]
	b	Width of the building	[m]
	d	Depth of the building	[m]
	f_e	Eigen frequence	[Hz]
	δ	Displacement	[m]

1 INTRODUCTION

In this chapter, the introduction to the research is presented. First, the research context and problem is discussed. This is followed by the research objective and the scope of the research. Furthermore, the research questions are defined. And finally, the reading guide of the research is presented.

1.1 Research context

The Netherlands is currently facing a significant housing problem. According to data from last summer (July, 2023) the shortage is 390.000 dwellings and the expectancy is that this will exceed 400.000 in 2024 (NOS, 2023). Known causes of this shortage are urbanization and limited available land. Due to this predicament interesting developments in the building industry take place. One of the promising alternatives is that existing buildings get a new function and will be altered in order to fit this new function. Partly with the goal of increasing the profit of the project, often additional layers are added to the existing building. One of the main challenges for this alternation to the existing building is to obtain sufficient stability in this new, combined structure, because the bottom part, the ‘old’ part, is designed according to older codes and the new layers are designed using the most recent Eurocode. Additionally, a topped up, and therefore higher building simply catches more wind.

Over the last few years, reuse of existing buildings has become more common and is considered necessary at some locations in the Netherlands, since there is limited space to build new dwellings. Those existing buildings get a new function and research on how to approach this new way of designing is limited. There are design codes (the NEN 8700 series) which provide guidelines for renovations, but specifically for adding extra levels to an existing building, the research is in short supply. In practice, often levels are added to existing buildings, so decisions are being made on what design codes to use when combining existing structures with new ones. However, an overview of the exact changes in wind loads and what kind of influence this has on adding new structural elements (levels) to an existing building is missing. Furthermore, the exact gap between the location or building specific wind on a building and the wind load according to the design codes is unknown.

1.2 Research objective

The objective of this research project is to provide insight into the changes in the design code for wind loads and what kind of influence these changes have on existing buildings with the application of additional levels. The wind load according to the design codes will be compared to the building and location specific wind load as well. To do so, starting from the TGB 1955, the design codes will be evaluated and the fundamental changes will be determined for wind loads.

1.3 Research scope

Design codes from 1955 until now (2024) will be discussed. This includes the TGB 1955 (NEN 1055), TGB 1972 (NEN 3850), TGB 1990 (NEN 6700) and the Eurocode (NEN-EN 1990 to 1999). The Eurocode has been introduced in the Netherlands in 2012 (NEN & Lurvink, 2022). The general design code as well as the Dutch National Annex will be evaluated. Since the TGBs are specifically for the Netherlands, only buildings in the Netherlands are within the scope of this research. Buildings with a maximum height of 70 meters will be discussed, since the Netherlands does not have that many high rise buildings, so a high percentage of the Dutch buildings will fall within this range. The main focus will be on characteristic loads, since partial safety factors were introduced with the TGB 1990. Before 1990, safety factors where solely applied on the material side. Since this thesis focusses on the side of the loads, a comparison of the design loads would be unfair.

1.4 Research questions

Together, the research problem, objective and scope lead to the main research question:

> "What is the influence of changes in the applied wind load since the TGB 1955 building design code on existing buildings when applying additional levels and how representative is the wind load that is prescribed by these design codes in the view of optimal designing for adding building levels?"

To answer this, sub-questions have been derived from the question. Together they will provide an answer to the main research question:

1. What is the difference between the wind loads defined in the TGB 1955, TGB 1972, TGB 1990 and Eurocode?
   1.1. What parameters are included in the design codes for wind load and how are they determined?
   1.2. How did the design codes develop between 1955 and now?

2. How does a building or location specific wind load relate to wind load defined by the design codes?
   2.1. What method can be used to determine the building or location specific wind load?
   2.2. What is the difference between the building or location specific wind load versus the wind load according to the design codes?
      2.2.1. How representative is the wind load?
      2.2.2. What is the development of wind speed over the years?

3. Adding layers to an existing building influences the wind load in a twofold manner: extra surface subjected to wind load and a difference in design codes for wind load between the now and past. What influence does this changed wind load have on stability systems and vertical load bearing structures of the existing structure when extending an existing building?

4. Are there any general recommendations for enhanced assessment of wind load on existing buildings when applying additional layers and what are they?

1.5 Methodology

The methodology is based on the process of wind loads, presented in Figure 1.1 (C. Geurts, 1996). All of the steps will be compared for the different design codes. This is the first part of the research. A literature study will provide the information to do so. The last step of the process of wind loads, the criteria for safety, is strictly not part of the scope of this thesis, as mostly characteristic values will be compared. This is due to the fact that partial safety factors were not applied at the loads until the TGB 1990. Higher allowed stresses on the material side provided the necessary safety in the design. Throughout the thesis, some of these criteria will be highlighted and explained further if this is required in order to make a proper comparison.

Furthermore, the building and location specific wind loads will be compared to the wind loads defined by the design codes. The building and location specific wind load is assumed to be based on the wind speed and pressure coefficient. These are the main input parameters to determine the wind load. By multiplying these values, the wind load is obtained. As presented in the figure, the wind climate determines the wind speed in a certain area. The winds speed and terrain combined, gives the wind pressure on a building. The location and building specific wind speed will be determined by combining data from the KNMI database and the pressure coefficients will be obtained by analysing a Computational Fluid Dynamics (CFD) model.

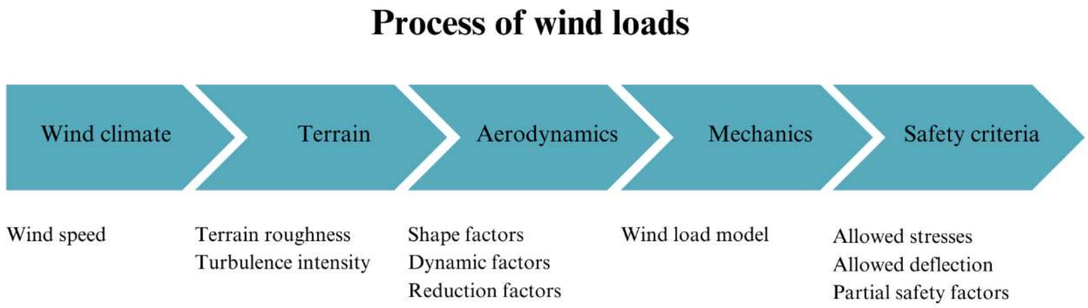


Figure 1.1 Process of wind loads

Next, the influence of the change in wind load and of the additional levels on the bearing structure of the existing building will be discussed. This consists of three parts; the lateral load bearing structure, which is the stability system, the vertical load bearing structure and the foundation.

1.6 Reading guide

This thesis consists of three main parts; (1) the research , (2) the case study and (3) the concluding summary.

The research is divided into three sections; first, in chapter 2, the design codes (TGB 1955, TGB 1972, TGB 1990 and Eurocode) for wind load are compared to each other. The wind load is broken down into wind pressure and coefficients. These are all analysed and the developments are displayed. In the second part of the research, chapter 3, the location or building specific wind load is researched. And finally, the third part, evaluates the influence of the changed wind load on the bearing structure. This is chapter 4.

The case study in chapter 5 entails SCYE010, a former 8-level office in the city of Schiedam. The office is built in 1978 and transformed into dwellings in 2021. The original calculations were therefore performed with the TGB 1972 and the new ones with the Eurocode. The building already had a top level which did not have the same layout the lower part of the building. The aim of this case study is to optimize the top level on the building and to apply the findings from the research for this optimization.

The last part, the concluding summary consists of the discussion (chapter 6), conclusion and recommendation of the master thesis (chapter 7).

PART I
RESEARCH

2 COMPARISON DESIGN CODES FOR WIND LOAD

To start the research on the influence of changes in the applied wind load for additional layers on existing buildings, the first step is to look at wind load that is specified in the design codes. To calculate the wind load on a building, the wind pressure is combined with several parameters in order to determine a value which represents the wind on a building in an appropriate way. Since TGB 1955, the code has developed a lot. The number of parameters that need to be taken into account has increased significantly to create a more realistic model. An overview of these parameters is given in section 2.1. The wind pressure is the first parameters that is discussed. This evaluation is in section 2.2. It will be followed by a talk through of the coefficients in section 2.3. Furthermore, a wind comparison tool is set up in Excel to easily compare the prescribed values by the design codes. It also gives the possibility to get insight into the consequences of adding levels to an existing building. This will be explained thoroughly in section 2.4. To conclude, the overall development of the design codes for wind loads is discussed in section 2.5.

2.1 Overview

In order to give a clear description of the codes, the chapter is started with an overview of the formulas for wind load and which code implements which parameters. This is presented in Table 2.1. Roughly, the formulas consists of two parts; wind pressure (q_p) and coefficients (c). This explains the division that is made in this chapter.

Important to note is that over the years the acronyms of the parameters have changed, plus they went from Dutch to English over the years. In this thesis the acronyms from the Eurocode are used in order to provide clear comparisons and prevent confusion.

Table 2.1 Overview formulas and parameters for all wind design codes

	TGB 1955[1]	TGB 1972[2]	TGB 1990[3]	Eurocode[4]
Formula	$q_w = q_p * c$	$q_w = q_p * c$	$q_w = c_{dim} * c_{index} * c_{eq} * \phi_1 * q_p$	$q_w = c_s c_d * c_f * q_p(z_e) * c_p$
External pressure/suction	c_p	c_p	$c_{index} = c_{pe}$	$c_p = c_{pe,10}$
Friction		c_{fr}	$c_{index} = c_{fr}$	c_{fr}
Internal pressure/ suction	c_o	c_o	$c_{index} = c_{pi}$	$c_p = c_{pi}$
Local pressure/suction			$c_{index} = c_{pe,loc}$	$c_p = c_{pe,1}$
Combination of coefficients		c_t	$c_{index} = c_t$	$c_p = c_f$
Dynamic effects		ϕ_1	ϕ_1	c_d [5]
Dimensions	$reduction\ factor$		c_{dim}	c_s [5]

[1] (Koninklijk Instituut van Ingenieurs, 1955)
[2] (Nederlands Normalisatie Instituut, 1972)
[3] (Nederlands Normalisatie Instituut, 1990)
[4] (Stichting Koninklijk Normalisatie Instituut, 2023)
[5] Parameter $c_s c_d$, which is used in the Eurocode, is usually considered as one parameter. However, in order to compare it to the other design codes, it has been separated. This is because the two parts, c_s and c_d, show similarities with parameters which are implemented in previous design codes. The minimum value is 0.85.

2.2 *Wind pressure*

The wind pressure is the first parameter to evaluate. This parameters forms the base for the total wind pressure value. In all of the codes, the value of the wind pressure is depending on the height of the building and the wind speed. In this part of the chapter, first, the wind pressure formulas from each design code are given in section 2.2.1. Next in section 2.2.2, the wind pressure is further explained for each design code. This part focusses more on the application of the wind pressure, rather than the background of the formulas. In all formulas, the roughness of the terrain has a relatively important role. Therefore, this part is evaluated separately in section 2.2.3.

2.2.1 Wind pressure formulas and the background of wind speed

Per design code, the wind pressure formulas will be shown and explained shortly. It will be clarified what the changes are in the formula compared to the formula from the previously published design code. The formula of wind pressure is based on the formula for kinetic energy, where the traveling mass is multiplied by ½ and the squared speed. To determine dynamic pressure, which is the case for wind pressure, the mass is replaced by the mass density. This result is the formula: $q_p = \frac{1}{2} * \rho * v^2$ (Benson, 2021). For wind, the mass density of air is used. In real life this value can vary over the height, because it is depending on temperature and air pressure. However, in the calculation for wind loads, a fixed value of 1.25 kg/m^3 is used (AMS, 2017). This value is given in all of the design codes as a fixed value and is only applicable on land, so for not offshore wind load calculations. Furthermore, some background information will be given regarding the determination of the wind speeds that are applied in the design codes.

TGB 1955

TGB 1955 uses a relatively simple system to determine the wind pressure on a building. There are three increments in height as is presented in Table 2.2. Below a height of 20 m and above a height of 40 m, the wind pressures are constant. In between, linear interpolation is applied. Furthermore, there is a division between moderate and high wind pressure and there are three different areas to choose from (Koninklijk Instituut van Ingenieurs, 1955). When to apply which type, will be explained in paragraph 2.2.2.

Table 2.2 Moderate and high wind pressure TGB 1955 (Koninklijk Instituut van Ingenieurs, 1955)

Height	Moderate wind pressure $[kN/m^2]$	High wind pressure $[kN/m^2]$
$0 < h \leq 20\ m$	$q_{p,moderate}$	$q_{p,high}$
$20 < h \leq 40\ m$	$q_{p,moderate} + 0.01 * (h - 20)$	$q_{p,high} + 0.015 * (h - 20)$
$h > 40\ m$	$q_{p,moderate} + 0.01 * 20$	$q_{p,high} + 0.015 * 20$

Table 2.3 Base wind pressure per area TGB 1955 (Koninklijk Instituut van Ingenieurs, 1955)

	$q_{p,moderate}$ $[kN/m^2]$	$q_{p,high}$ $[kN/m^2]$
Coast line	0.60	1.00
Coast	0.50	0.85
Inland	0.40	0.70

The wind pressure in TGB 1955 is based on formula (2. 1). The wind pressure formulas of all the reviewed design codes are based on the formula $q = \frac{1}{2}\rho v^2$.

$$\mathrm{q_p} = \frac{\mathrm{v}^2}{16}\ [kg/m^2]\left(= \frac{\mathrm{v}^2}{1600}\right)\ [kN/m^2] \tag{2. 1}$$

$$v = Wind\ speed\ [m/s]$$

It requires some explanation on how to obtain formula (2. 1), as the base formula cannot directly be recognised in equation (2. 1). The TGB 1955 refers to a background article by Schoemaker and Wouters (1932). It starts with calculating the density:

$$\rho = \frac{m}{a} = \frac{1.23\ [kg/m^3]}{9.81\ [m/s^2]} \approx \frac{1}{8}\ kg * s^2/m^4$$

This value is filled into the base formula and in order to obtain the correct units, it is divided by 100 (when assuming 10 m/s^2 for the gravitational acceleration):

$$q_p = \frac{1}{2}\rho v^2 = \frac{1}{2} * \frac{1}{8} * v^2 = \frac{v^2}{16} \rightarrow\ units\text{: } kg * s^2/m^4 * (m/s)^2 = kg/m^2$$

However, this method does not align with the way density is defined in the design codes published after the TGB 1955. To start, in the later design codes a different value is applied for the density of air; 1.25 kg/m^3, where TGB 1955 applies 1.23 kg/m^3. Furthermore, to obtain the density, this mass of air is then divided by the gravitational acceleration. This results in a incorrect unit for density; $kg*s^2/m^4$ instead of kg/m^3, because density is calculated by dividing the mass by the volume. However, this unit for wind pressure does result in the standard unit for the wind pressure for that time period; kg/m^2. To convert it to kN/m2, it can simply be divided by 100.

Limited information is provided regarding the value the wind speed. The article by Schoemaker and Wouters (1932) implies that measured wind speeds can be applied directly into the formula for wind pressure. First of all, it states that the wind pressures are based on measurements at weather station De Bilt. Secondly, it says that during the most powerful storms the maximum wind speeds are 29 m/s. This wind speed results in a wind pressure of $29^2/1600 = 0.53\ kN/m^2$. The article states that this value is then rounded up to 0.6 kN/m^2. Since the design code is based on this article, it can be assumed that for extra safety the Dutch Royal Institute of Engineers (NL: Koninklijk Instituut van Ingenieurs) increased the value to 0.7 kN/m^2 for the inland region. Schoemaker and Wouters assume an even higher load of 0.8 kN/m^2. The coast and coastline region are based on the value obtained at De Bilt and the article prescribes an increase of 25% for the coast region and 50% for the coastline region, resulting in a value of 1.0 kN/m^2 and 1.2 kN/m^2. This is in line with the Prussian guidelines, that were often referred to at that time. However, the Dutch Royal Institute of Engineers applied an increase of 21% and 39%, which results in 0.85 kN/m^2 for the coast region and 1.0 kN/m^2 for the coastline. Even though these values are lower than the suggested values in the articles, the method is probably accepted due to the fact that this still results in significant safety. Important to note is that this is for the high wind loads and not the moderate.

Buildings with a height which is less than 16 m get exemption from stability calculations, unless the ratio between height and width gives reason to do otherwise. There are no guidelines for which ratios are assumed to be safe or not, so this relies on the judgement of the engineer.

Furthermore, important to note is that in the TGB 1955, the safety is coming from the material side, where a factor is applied. The loads do not get a safety factor, so the values that are given can be applied directly.

TGB 1972

TGB 1972 has elaborated enormously on the complexity of the calculation of extreme wind pressures. Instead of a stepwise increase of pressure, the formula has become logarithmic and multiple additional parameters have been incorporated (Nederlands Normalisatie Instituut, 1972). Formula (2. 2) is the formula for wind pressure in the TGB 1972.

$$q_p = \frac{1}{2}\rho v^2 \left(= \frac{v_u^2}{1.6}\left(1 + gr\sqrt{T_b}\right)\right) \tag{2. 2}$$

$$\rho = 1.25\, kg/m^3 = average\ density\ of\ air$$

$$v = v_u(1 + gf) = wind\ speed\ [m/s] \tag{2. 3}$$

$$v_u = Average\ wind\ speed\ during\ one\ hour$$

$$g = peak\ factor \approx 4\ (for\ gust\ of\ 1\ to\ 30\ seconds)$$

$$f = \frac{1}{2} r\sqrt{T_b} \tag{2. 4}$$

$$r(h) = Terrain\ roughness\ factor$$

$$T_b(h) = Influence\ of\ windspeed\ distribution\ during\ a\ gust$$

The wind speed v_u is the average hourly wind speed which has the chance of 50% to be exceeded during a period of 5 years. Formula (2. 3) gives the equation for the wind speed. The value of the average wind speed has been deducted based on years of measurements by the KNMI and according to the design code it shows a normal distribution. The parameters of the formula for the average wind speed are the height at which the speeds needs to be determined, the base wind speed (v_{u10}) and parameter α, which both depend on the location. This base wind speed is assumed to be measured at 10 m height. Formula (2. 6) presents the equation for the wind speed v_u and Table 2.4 presents this base wind speed..

$$v_u = v_{u10}\left(\frac{h}{10}\right)^{\alpha} \tag{2. 5}$$

$$\alpha = 0.13\ (coast)$$
$$= 0.19\ (inland)$$

Table 2.4 Wind speed per wind area

Wind area	v_{u10} [m/s]
Coast line	26.0
Inland	20.5

Using the known distribution of wind from certain directions, it can be concluded that at a maximum of 20% of the time, the wind direction will be the same as the wind direction that was used for the calculation. The code states that this means that there is 40% chance that the given wind pressure will be exceeded once per 25 years (Nederlands Normalisatie Instituut, 1972).

TGB 1990

Similar to the other design codes, the TGB 1990 uses the formula for dynamic pressure for wind pressure. In addition to this base, a new parameter is incorporated into the formula: the turbulence intensity. This parameter is depending on the height of the structure and roughness length z_0. Alike the formula of the TGB 1972, the wind pressure formula is a logarithmic function (Nederlands Normalisatie Instituut, 1990).

$$q_p = \left(1 + 7I(z)\right) * \frac{1}{2} * \rho * v_w^2(z) \tag{2. 6}$$

$$I(z) = \frac{k}{ln\left(\frac{z-d}{z_0}\right)} \tag{2. 7}$$

$$v_w(z) = 2.5 * u_* * ln\left(\frac{z-d}{z_0}\right) \tag{2. 8}$$

With: $I(z) = Turbulence\ intensity\ at\ height\ z$

$\rho = 1.25\ kg/m^3\ mass\ of\ air$

$z_0 = Roughness\ length\ [m]$

$d = Displacement\ height\ [m]$

$u_* = Friction\ speed\ [m/s]$

$k = Factor\ (= 1.0\ for\ rural, = 0.9\ for\ urban)$

The friction speed is not directly measured by the KNMI weather station. Several steps are required to obtain this value form the measured data. In the background paper on the wind loads of TGB 1990 by Van Staalduinen (1992), these start values are called potential wind speed ($u_p(10)$). The TGB 1990 prescribes a potential wind speed for each wind area. Next, the average wind speed at 60 m height is calculated, using a roughness length z_o of 0.03 m and a displacement height d of 0 m:

$$u(60) = \frac{ln\left(\frac{60-d}{z_o}\right)}{ln\left(\frac{10-d}{z_o}\right)} * u_p(10) = 1.308 * u_p(10) \tag{2. 9}$$

Using the average wind speed at 60 m height, the hourly mean wind speed at height z can be calculated. In this case, the 'real' roughness length (z_{or}) has to be used. These are presented in Table 2.7 in section 2.2.3. In a rural terrain, d is 0 m.

$$u(z) = \frac{ln\left(\frac{z-d}{z_{or}}\right)}{ln\left(\frac{60}{z_{or}}\right)} * u(60) \tag{2. 10}$$

The final step is to calculate the friction speed u_*. A formula, which is very similar to the formula for the wind speed $v_w(z)$, is used (Van Staalduinen, 1992):

$$u(z) = \frac{u_*}{\kappa} * ln\left(\frac{z-d}{z_{or}}\right) \rightarrow u_* = u(z) * \frac{\kappa}{ln\left(\frac{z-d}{z_{or}}\right)} \tag{2. 11}$$

$$u_* = Friction\ speed\ [m/s]$$

$$\kappa = Von\ Karman\ constant = 0.4$$

$$d = Displacement\ height\ [m]$$

$$z_{or} = Roughness\ length\ [m]$$

Table 2.5 presents the wind speeds and friction speeds that are used per wind area by the TGB 1990.

Table 2.5 Wind speed and friction speed TGB 1990

Wind area	v [m/s] (=$u_p(10)$)	u_* [m/s]
I	27.4	2.25
II	25.0	2.30
III	22.5	2.25

Eurocode

The general formula for wind pressure given in the Eurocode is exactly the same as in TGB 1990. However, some of the parameters have a different formula themselves, such as the turbulence intensity and the wind speed. For the turbulence intensity, displacement height is not part of the formula anymore and z_{min} is introduced. The wind speed is not a logarithmic function anymore.

$$q_p(z) = \left(1 + 7I(z)\right) * \frac{1}{2} * \rho * v_m^2(z) \tag{2. 12}$$

$$I(z) = \frac{k_I}{c_o ln\left(\frac{z}{z_0}\right)} \qquad for\ z_{min} \leq z \leq z_{max} \tag{2. 13}$$

$$I(z) = I(z_{min}) \qquad for\ z \leq z_{min} \tag{2. 14}$$

$$v_m = c_r(z) * c_o(z) * v_b\ (Average\ wind\ speed) \tag{2. 15}$$

With: $I(z) = Turbulence\ intensity\ at\ height\ z$

$k_I = 1.0 = Turbulence\ factor$

$$z_0 = Roughness\ length\ [m] \rightarrow depends\ on\ terrain\ category$$

$$z_{min} = Minimal\ height\ [m] \rightarrow depends\ on\ terrain\ category$$

$$c_o(z) = 1.0 = Orography\ factor$$

$$c_r(z) = Roughness\ factor$$ See (2. 22)

$$v_b = v_{b,o} = \ Base\ wind\ speed\ (according\ to\ national\ annex)$$

The wind pressures that are given in the design code have a return period of 50 years. They are the characteristic values, based on standard values of wind speed or wind pressure. Wind speed v_m it based on the base wind speed $v_{b,0.}$ This is the characteristic average wind speed over the last 10 minutes of an hour, measured at a height of 10 m above ground level (Stichting Koninklijk Normalisatie Instituut, 2023). Furthermore, the wind speed is depending on the orography and the roughness factor. The orography factor is prescribed by the national annex of the Eurocode and is equal to 1.0. The roughness factor is depending on the terrain. This will be explained further in section 2.2.3.

$$v_m = c_r(z) * c_o(z) * v_b\ (Average\ wind\ speed) \tag{3. 1}$$

$$v_b = v_{b,o} = \ Base\ wind\ speed\ (according\ to\ national\ annex)$$

The base wind speed that is used for each wind area is given in Table 2.6.

Table 2.6 Base wind speed Eurocode

Wind area	v_b [m/s]
I	29.5
II	27.0
III	24.5

2.2.2 General explanation wind pressure per design code

So far, the wind pressure formulas have been explained for each design code. This reveals some of the background of these codes. To apply the wind pressure formulas on a specific building, more input information is required. One of the main things is the location of the building, which results in a wind area. Over the years, the map of the wind areas for the Netherlands has changed. Furthermore, the building height is important. To start, this is one of the input parameters of the wind pressure formulas, but is determines the distribution of the wind pressure over the height of the building as well. This will be referred to as wind load model. For all design codes, these main factors that influence the wind pressure will be high lighted and further explained in this section.

TGB 1955

In the TGB 1955 the wind pressure on a building is depending on roughly three things; location in the Netherlands, material, building height.

Location

The location can be determined according to the map shown in Figure 2.1. This makes a division between three areas; coast line, coast and inland.

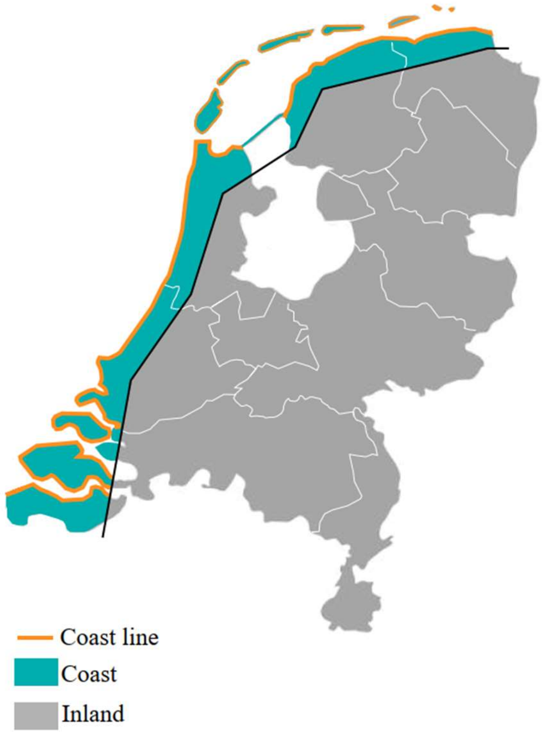


Figure 2.1 Map of the Netherlands with wind areas 1955

Material

Depending on the material, one may use high and/or moderate wind pressure (Figure 2.3). Only for steel structures moderate wind can be used, but at the same time, the combination with the high wind pressure should be checked as well. The design code states that the allowed stresses for timber and concrete are too global in combination with the material properties, to allow for the use of moderate wind pressure. The page from TGB 1955 that explains this is presented in Appendix A (section A.1) as reference.

Building height

Over the entire building height, the same wind pressure has to be applied, as is visualised in Figure 2.2. This is the maximum value on the building, which occurs at the maximum height. So the building height can be used to obtain the wind pressure from Figure 2.3.

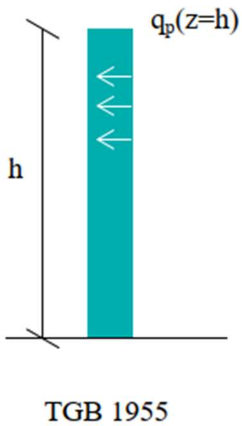


Figure 2.2 Model of wind pressure on building

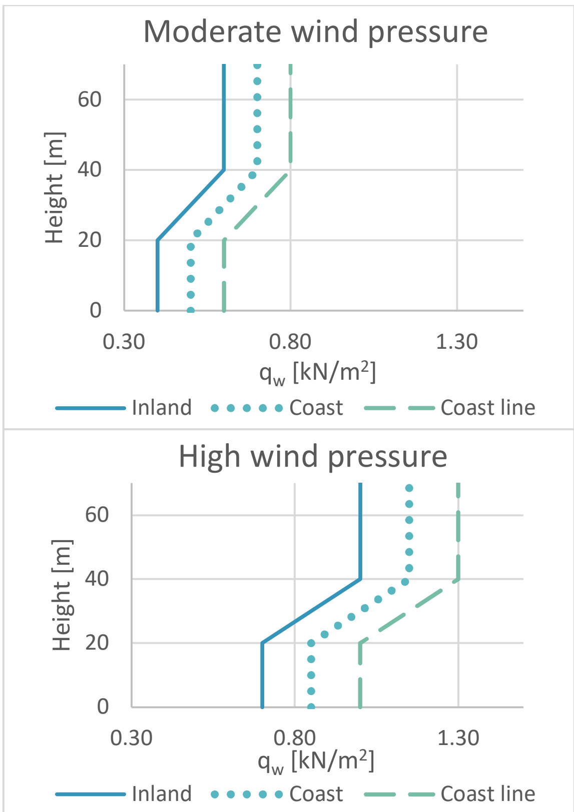


Figure 2.3 Graph of high wind pressures over height

TGB 1972

In the TGB 1972 the wind pressure is mainly depending on two things; location in the Netherlands and building height. It is not depending on material anymore.

Location

Figure 2.4 shows the wind area map of the Netherlands in 1972. It seems to have only two areas. However, the 'coast' wind area exists as well. The reason this is not visualised is because it depends on the building height (h) in combination with the distance to the coast (a). The way the wind areas are divided is as following:

Coast line	a ≤ 25 h
Coast	25 h ≤ a ≤ 50 h
Inland	a > 50 h

Figure 2.6 gives the values of wind pressure for the coast line and inland. For the coast wind area, interpolation can be applied.

Figure 2.4 Map of the Netherlands with wind areas 1972

Building height

The TGB 1972 does not specify whether the wind pressure is constant or not over the height of the building. Archival research (for example from case study SCYE010 (Groenenbeek & Poot B.V., 1975)) shows that the code was interpreted in a way that it is constant up to a height of 7 m and from that point it linearly increases over the height. Figure 2.5 visualises this. The table from TGB 1972 which shows this is added to Appendix A (section A.2)

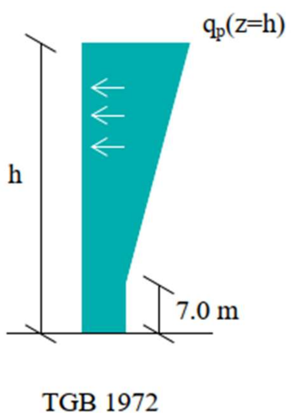


Figure 2.5 Model of wind pressure on building

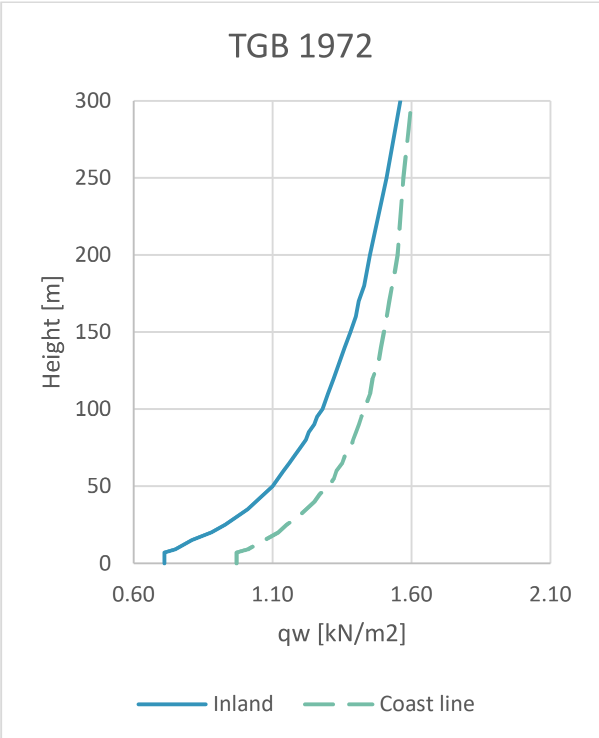


Figure 2.6 Graph of wind pressures over height

TGB 1990

In the TGB 1972 the wind pressure that needs to be applied is mainly depending on three things; location in the Netherlands, terrain and building height.

Location

Figure 2.7 shows the wind area map of the Netherlands in 1990. The division is simply made by the borders of the provinces in the Netherlands, which are shown with the white lines. Except for the province of Noord-Holland, where a division is made between the wind areas at the height of the municipality of Heemskerk.

Terrain

Next, it needs to be determined whether the building is situated in a rural or urban area. This is a simplification of the roughness of the surroundings of a building. An area with a radius of up to 1 km needs to be evaluated in order to determine whether a building is located in a rural or urban area, because of the influence on the turbulence. Figure 2.9 shows the graphs with wind pressures of both categories for the 3 wind areas.

Building height

Depending on the ratio between the height and the width (b) of the building, a different approach to the model needs to be applied. When the height is smaller than the width, the same maximum wind pressure is applied over the total height. When the height is larger than the width, the bottom part is subjected to a constant wind pressure and the part above is linearly increasing until the maximum wind pressure is reached at the top of the building.

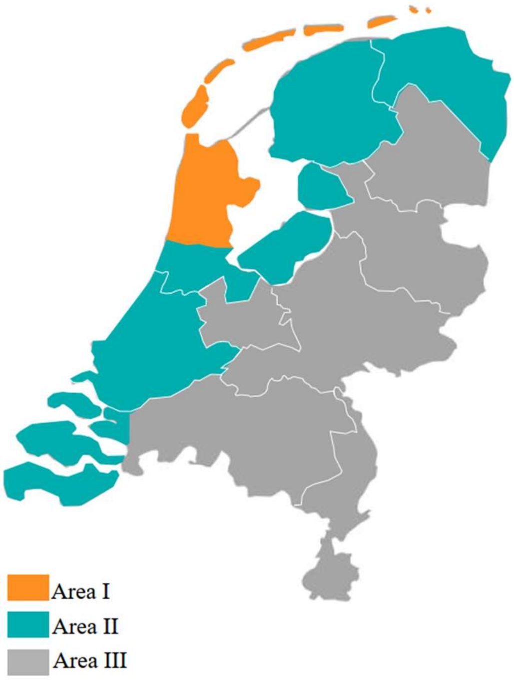


Figure 2.7 Map of the Netherlands with wind areas 1990

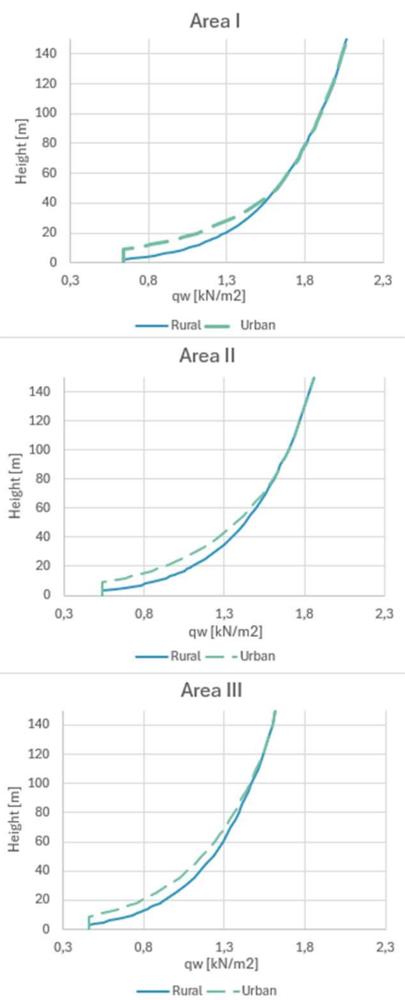


Figure 2.9 Graphs of wind pressures over height

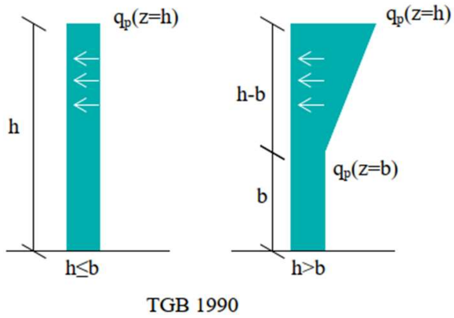


Figure 2.8 Model of wind pressure on building

Eurocode

In the Eurocode the wind pressure is depending on the same three things as in the TGB 1990; location in the Netherlands, terrain and building height.

Location

Figure 2.10 shows the wind area map of the Netherlands. This map has not changed compared to the wind area map of TGB 1990.

Terrain

In this category, the Eurocode has made changes. An extra type has been added for area I and II: coast. Figure 2.12 shows the wind pressure graphs for the three areas with the different types of terrain categories.

Building height

The model of the wind load on a building has changed as well. Same as for TGB 1990 it is depending on the slenderness ratio between the height and width of the building, but an extra step has been added for buildings with a height in between the width and 2*width. For these buildings, the wind pressure is divided into two rectangular blocks instead of a linear increase. This results in an increased total wind load, which means it is more conservative. However, this simplifies the calculation significantly. For higher buildings, a simplification of the linear increase has been applied as well, by instructing to make use of small increments for part of the height and assume equal wind pressure on the top and bottom part.

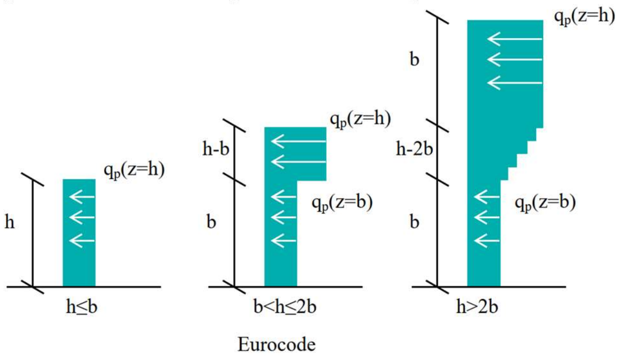


Figure 2.11 Model of wind pressure on building

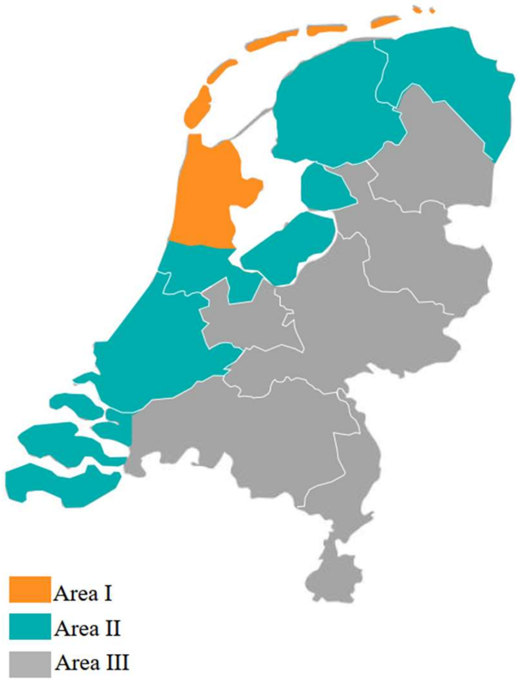


Figure 2.10 Map of the Netherlands with wind areas Eurocode

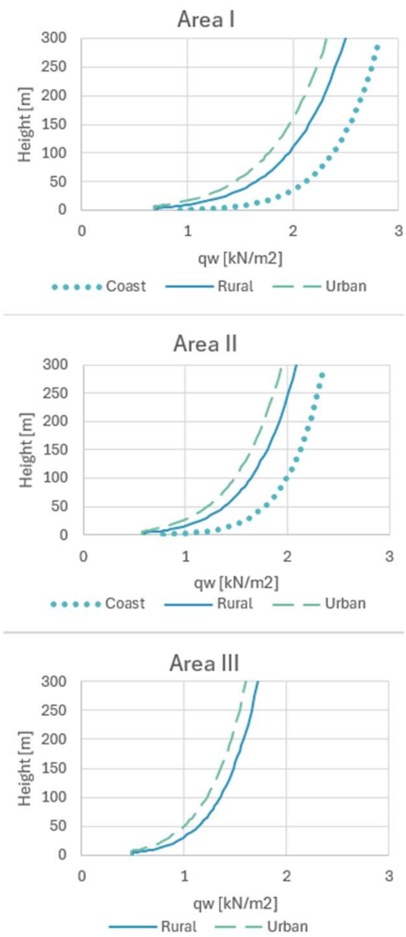


Figure 2.12 Graph of coast/rural/urban wind pressures

2.2.3 Terrain roughness

The terrain roughness is implemented into the calculation for wind pressure since the TGB 1972. The TGB 1955 does not mention terrain roughness. This section will further explore use of the terrain roughness in the wind load calculations.

TGB 1972

To recap, in section 2.2, the formula for wind pressure is given: $q_p = \frac{v_u^2}{1.6}(1 + gr\sqrt{T_b})$. In this formula, ‘r’ represents the influence of the roughness of the terrain. The values of ‘r’ are presented in Figure 2.13 and are based on measurements in the Netherlands and outside of the Netherlands. The bottom line (number 1) is the roughness factor for areas along the North Sea coast and the top line (number 2) for areas inland. The factor is presented as a function of the height of the construction. The graph shows that the factor decreases with an increase in height and that there is a logarithmic relation (Nederlands Normalisatie Instituut, 1972).

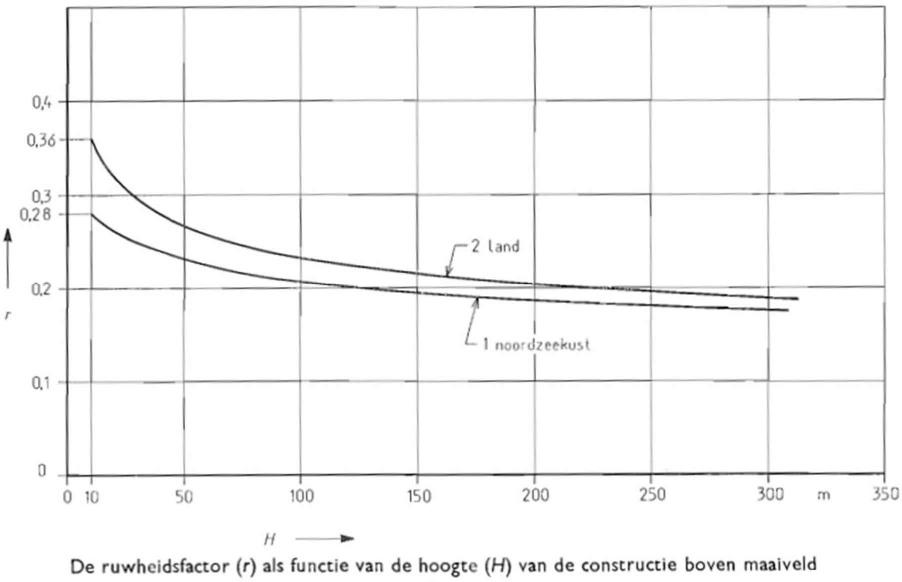


Figure 2.13 Roughness factor r as function of the height TGB 1972 (Nederlands Normalisatie Instituut, 1972)

TGB 1990

In section 2.2.1, the formula for wind pressure is given, according to TGB 1990: $q_p = (1 + 7I(z)) * \frac{1}{2} * \rho * v_w^2(z)$. In the TGB 1990 the roughness is not a parameter, but it is incorporated in the roughness length, z_0. The roughness length is integrated in the formula for turbulence intensity as well as in the formula for wind speed. Those formulas are presented again below.

$$Turbulence\ intensity \rightarrow I(z) = \frac{k}{ln\left(\frac{z-d}{z_o}\right)} \tag{2. 7}$$

$$wind\ speed \rightarrow v_w(z) = 2.5 * u_* * ln\left(\frac{z-d}{z_o}\right) \tag{2. 8}$$

The TGB 1990 makes a subdivision between eight categories for roughness length. The smallest one has a value of z_0=0.0002 m, which represents an open sea or open water. The value increases to over 2.0 m for city centres. The TGB 1990 has standardised the values based on the location of the building (area I/II/III) and the terrain categories (rural/urban). These values are presented in Table 2.7. Besides that, in case of the building being in an urban area; the roughness length can be determined with a formula as well. The area for which the buildings should be evaluated is depending on the height of the assessed building (Nederlands Normalisatie Instituut, 1990).

$$z_o = 0.5 * \alpha * h_m \tag{2. 16}$$

$$Average\ building\ height \rightarrow\ h_m = \sum h_i \frac{A_i}{\sum A_i}\ [m] \tag{2. 17}$$

$$Building\ density \rightarrow\ \alpha = \frac{A_{buildings}}{A_{no\ buildings}}\ [-] \tag{2. 18}$$

Table 2.7 Roughness length TGB 1990 (Nederlands Normalisatie Instituut, 1990)

z_0 [m] **Area**	**I**	**II**	**III**
Rural	0.1	0.2	0.3
Urban	0.7	0.7	0.7

Eurocode

The Eurocode has a very similar formula to determine wind pressure compared to TGB 1990. The formula is: $q_p(z) = \big(1 + 7I(z)\big) * \frac{1}{2} * \rho * v_m^2(z)$. However, it consists of some essential changes for the parameters that are influenced by the terrain. For example, the logarithmic part of the turbulence intensity is not dependent on the displacement height (=d) anymore. Besides that, the wind speed is now depending on a roughness factor as well, which is a logarithmic function (Nederlands Normalisatie Instituut, 2011b).

$$Turbulence\ intensity \rightarrow\quad I(z) = \frac{k_I}{c_o ln\left(\frac{z}{z_o}\right)} \qquad for\ z_{min} \leq z \leq z_{max} \tag{2. 19}$$

$$I(z) = I(z_{min}) \qquad for\ z \leq z_{min} \tag{2. 20}$$

$$Turbulence\ factor \rightarrow\quad k_I = 1.0$$

$$Average\ wind\ speed \rightarrow\quad v_m = c_r(z) * c_o(z) * v_b \tag{2. 21}$$

$$Roughness\ factor \rightarrow\quad c_r(z) = k_r\ ln\left(\frac{z}{z_0}\right) \qquad for\ z_{min} \leq z \leq 200\ m \tag{2. 22}$$

$$c_r(z) = c_r(z_{min}) \qquad for\ z \leq z_{min} \tag{2. 23}$$

$$Terrain\ factor \rightarrow\quad k_r = 0.19\left(\frac{z_o}{z_{0,II}}\right)^{0.07}\ , z_{0,II} = 0.05 \tag{2. 24}$$

The Eurocode describes five terrain categories, with written descriptions as well as drawings (Nederlands Normalisatie Instituut, 2011b). The National Annex for the Netherlands reduces this to three categories: sea/coast area, rural area and urban area. This is presented in Table 2.8 (Stichting Koninklijk Normalisatie Instituut, 2023). The three categories are not directly taken from the general Eurocode. The values in Table 2.8 lie in between the values for the different terrain categories from the general Eurocode. As a reference, this the table from the Eurocode is presented in Appendix A (section A.3). This is also a simplification compared to the table of TGB 1990, where a division is made for the three areas (I/II/III). Furthermore, the value for urban areas has reduced slightly from 0.7 to 0.5, which could be caused by more experimental results, that prove a reduction is possible.

Table 2.8 Terrain categories and parameters Eurocode National Annex

Terrain category	z_0 [m]	Z_{min} [m]
0 Sea/coast	0.005	1
II Rural	0.2	4
III Urban	0.5	7

It is also stated that the roughness length should be calculated using the formula below. This is the same formula as in TGB 1990 and α and h_m can be calculated the same way as in TGB 1990 (Stichting Koninklijk Normalisatie Instituut, 2023).

$$z_o = 0.5 * \alpha * h_m \tag{2. 25}$$

Other countries, such as Belgium uses all five terrain categories that are presented in the general Eurocode (Bureau de Normalisation, 2010). This is interesting, since the Netherlands has reduced it to three. Category number I and IV have been removed. Number I represents a flat and horizontal area with a negligible amount of vegetation and no obstacles. It could be argued that the Netherlands does not have areas large enough of this type of terrain to make use of it. Terrain category number IV is an area where at least 15% of the area is covered with buildings with a minimum height of 15 m. Since the Netherlands is a very densely this is a category that could be applicable. However, a closer look at the prescribed roughness length for the Eurocode and the National Annex show that the roughness lengths used for the National Annex are actually in between some of the categories from the general Eurocode. So in fact, compared to the Eurocode, the national annex applies a completely different table for terrain categories. The UK applies this same trick and combines category I and II, III and IV (BSI, 2008). Germany reduces the number of terrain categories as well, but they reduced it to four (Deutsches Institut für Normung, 2009). The overview of terrain categories of each country has been added to Appendix A (section A.3). It can be concluded that all countries have chosen an optimal amount of types for their terrain categories. However, the Netherlands uses relatively a small amount, compared to the other west-European countries.

2.3 *Coefficients*

The coefficients that are analysed in this section are the shape factors, the dynamic magnification factors and the reduction factors based on dimensions of the buildings. As has been depicted in the introduction of this chapter, not all of these parameters have been implemented in the calculation for wind design since the TGB 1955. As the codes developed, more parameters were included to be able to obtain more realistic wind loads. In this paragraph, the parameters will be discussed and the differences between the design codes will be highlighted.

2.3.1 Shape factor

The shape factor is an umbrella term for parameters that have influence on the wind load due to the shape of the building. The most important ones are the wind pressure/suction (c_{pe}), local wind pressure/suction ($c_{pe,loc}$), internal wind pressure/suction (c_{pi}), wind friction (c_{fr}) and the total shape factor (c_t).

Pressure coefficient c_{pe}

The simplest, and simultaneously the most common case for which the parameter of (external) wind pressure and suction needs to be applied, is for closed buildings with a rectangular shape. Table 2.9 gives an overview of the parameters that need to be applied for those rectangular shaped buildings.

Table 2.9 Overview of wind pressure/suction parameters

Closed building (rectangular)	Windward side		Leeward side		Parallel to wind
TGB 1955	65°<α<90°	0.9		-0.4	-0.4
	0°<α<65°	-0.4			
TGB 1972	65°<α<90°	0.8		-0.4	-0.8/-0.4
	0°<α<65°	0.4			
TGB 1990		0.8		-0.4	-0.8/-0.4
Eurocode		0.8	$h/d = 5$	-0.7	-1.2/-0.8
			$h/d \leq 1$	-0.5	

The pressure coefficients prescribed by the design codes are determined based on numerous wind tunnel tests. During those tests, the wind pressure is measured at a lot of points across the maquette of the building. Using these pressures, the pressure coefficient can be calculated. In wind tunnel research, the pressure coefficients are defined using the pressure difference over a construction (Geurts & Bouwen met Staal, 2012):

$$c_p = \frac{p_2 - p_1}{\frac{1}{2}\rho v_m^2} \qquad (2.\ 26)$$

Eurocode makes a division between two types of pressure coefficients; $c_{pe,1}$ and $c_{pe,10}$, where $c_{pe,1}$ represents a surface of 1 m^2 and $c_{pe,10}$ a surface of 10 m^2. $C_{pe,1}$ is meant to be used for smaller elements (local pressures) and $c_{pe,10}$ for the total load bearing construction (global pressures). This thesis only addresses the global pressures.

For standard calculations of wind, the pressure at the windward side is added to the suction at the leeward side. To compare the code, a short calculation is performed, which shows the development of the combined coefficient over the years. Most coefficients increase with the publication of new codes, but this one decreases for the duration of two codes and then increases with the publication of the Eurocode. However, in contrast to the older codes, the Eurocodes takes into account that the chance is very small that the maximum pressure/suction appears at the same time at the wind- and leeward side. This assumption is incorporated in the lack of correlation factor of 0.85. By applying this factor, the value of c_{pe} approaches the value given by the older design codes.

TGB 1955 $c_{pe} = 0.9 + 0.4 = 1.3$
TGB 1972 $c_{pe} = 0.8 + 0.4 = 1.2$
TGB 1990 $c_{pe} = 0.8 + 0.4 = 1.2$
Eurocode $c_{pe} = 0.8 + 0.7 = 1.5\ (*\,0.85 = 1.275)^*\ for\ h/d = 5$
$c_{pe} = 0.8 + 0.5 = 1.3\ (*\,0.85 = 1.105)^*\ for\ h/d \leq 1$

For (partly) open buildings the design codes have developed massively. Where TGB 1955 has a 2x2 table for three kinds of roofs, the Eurocode has increased to 8x10 tables, which often differ per wind direction as well. Therefore, it can be concluded that the codes have increased on level of detail. The value of the parameters have increased as well; in TGB 1955 the range was about ±0.4 to ±1.8, in TGB 1972 it was ±0.4 to ±1.5, in TGB 1990 it was ±0.4 to ±1.8 and finally in the Eurocode it has increased to ±0.4 to ±2.9.

Internal pressure coefficient c_{pi}
The internal pressure and suction parameter is known in the older codes as over- and under pressure (c_o). TGB 1955 only assumes over- or under pressure in case the building has an opening. The newer codes (TGB 1972 & 1990) state that there can be over- or under pressure in closed buildings as well. However, Eurocode does not mention this. Table 2.10 presents an overview of the internal pressure/suction parameter for the different design codes. Over the years, the parameter increases, but most importantly, it can be altered depending of other specifications of the building.

Table 2.10 Internal pressure/suction parameter

c_{pi}	*One side open building*		*Closed building*	
	Windward open	Leeward open	Pressure	Suction
TGB 1955	0.6	-0.3		
TGB 1972	0.8	-0.4	0.3	-0.3
TGB 1990	$0.3 \leq c_{pi}* \leq 0{,}8$	$-0{,}4 \leq c_{pi}** \leq -0{,}3$	0.3	-0.3
EC	See explanation Eurocode below			

$$*c_{pi} = 1{,}0 - 0{,}1 \log\left(\frac{V}{A}\right)$$
$$**c_{pi} = -\left(0{,}44 - 0{,}02 \log\left(\frac{V}{A}\right)\right)$$
$$V = gross\ volume\ building\ [m^3]$$
$$A = gross\ area\ openings\ in\ façade\ [m^2]$$

Eurocode
The Eurocode takes a different approach; it first instructs to determine whether there is a dominant side. This means that the area of the opening on one side (A1) is at least twice as

much as the sum of the areas of the openings on other sides (A2). In this case a percentage of the external pressure at that side must be applied.

$$if\ A1 \geq 2 * A2 \qquad c_{pi} = 0.75 * c_{pe}$$
$$if\ A1 \geq 3 * A2 \qquad c_{pi} = 0.90 * c_{pe}$$

In the case where there is no dominant side, the internal pressure/suction can be obtained from a graph (figure 7.13 in NEN-EN 1991-4). The value obtained from this graph is based on the ratio between A1 and A2, plus the ratio between the height and depth of the building.

Wind friction coefficient c_{fr}

Wind friction is being taken into account for surfaces parallel to the wind direction. This concept has been implemented in the design code since TGB 1972. In that code no distinction based on roughness of the surface has been made. For all types of surfaces 0.04 is applied. Since TGB 1990, a division is made for a smooth, rough and very rough surface. The values given in that code still apply in the Eurocode. Table 2.11 presents the wind friction coefficient overview.

Table 2.11 Wind friction coefficient

c_{fr}	Smooth surface	Rough surface	Very rough surface
TGB 1955		-	
TGB 1972		0.04	
TGB 1990	0.01	0.02	0.04
EC*	0.01	0.02	0.04

*In the Eurocode (article 5.3 (4)) is stated that the effect of friction on the sides may be neglected when the total area of the parallel surfaces is smaller than 4 times the total area of the perpendicular surface (Nederlands Normalisatie Instituut, 2011b).

Total shape factor c_t / force coefficient c_f

Since TGB 1972, for billboards, sign boards and other shapes that are added to buildings, a total shape factor (c_t) may be applied. This factor has a value of 2.0. For trusses a slightly lower total factor can be applied. This has a value of 1.6.

TGB 1990 has extended the shapes on which the total wind factor may be used. This code uses the total wind factor for shapes like triangles, squares, pentagons and other polygonal shapes, but also for spheres or part of spheres. This simplifies the calculation for buildings which such shapes, since just one factor can be used for the entire shape.

Eurocode does not work with the total shape factor. However, the force coefficient (c_f) applies for similar cases. The Eurocode states that the force coefficient accounts for the global effect of the wind on the construction, including friction. Eurocode also added that the force coefficient is allowed to be used for building with a ratio h/b > 5.0. The order of magnitude has remained about the same for the different codes, but the number of included cases and exceptions has increased.

2.3.2 Dynamic magnification factors

The dynamic factors have had a significant development over the years. In TGB 1955 nothing is mentioned regarding dynamics for wind load design. In TGB 1972 the first formula is derived to determine dynamic influence parallel as well as perpendicular to the wind direction. This paragraph will discuss the development of the dynamic factors per design code. TGB 1955 will be left out, since there is no relevant data to discuss. Important to note is that the dynamic factor is a magnification factor. So, whereas most other coefficients reduce the wind load, this factor increases it. The formulas of all parameters mentioned in this chapter are summarized in Appendix B.

TGB 1972
As stated in the introduction of this paragraph, TGB 1972 was the first design code to incorporate a dynamic factor for the wind load on a building. The dynamic factor does not need to be applied on all buildings. There can be two reasons for which dynamics need to be included:

1) The building has a height of 60 m or more and/or the slenderness ratio H/B > 5.
2) The building is sensitive for vibrations.

If statement 1 or 2 applies for the building under review, the dynamic factor is applied. The dynamic wind load is calculated with formula 2.26 for the direction parallel to the wind direction.

$$q_{dyn} = q_p * \varphi_1 \tag{2. 27}$$

$$\varphi_1 = \frac{1 + 4r\sqrt{T_b + T_r}}{1 + 4r\sqrt{T_b}} \tag{2. 28}$$

The roughness factor r has been discussed before in paragraph 2.2.3. The value for the roughness factor can be derived from the graph. T_b is a parameter for the influence of a wind gust. This influence is depending on the height of the building and is presented in the design code in a graph as well. The last parameter, T_r, is depending on the eigen frequency of the construction. The formula of the eigen frequency is given by formula (2. 29) and has undergone an interesting development as well, which will be discussed in the paragraph of the dynamic factor of the Eurocode.

$$f_e = \sqrt{\frac{0.25}{\delta}} \tag{2. 29}$$

For special cases, the dynamic wind load perpendicular to the wind direction needs to be calculated as well. This is for slender circular buildings or parts of buildings. This dynamic factor, φ_2, has a different formula. Although, it is roughly depending on the same parameters; roughness, wind gust and eigen frequency.

TGB 1990
In TGB 1990, the guidelines for applying a dynamic magnification factor in wind direction have changed slightly;

1) Building height is over 50 m.
2) The slenderness ratio h/b > 5.

A building needs to fulfil both criteria in order to make dynamics mandatory in the calculations for wind load design.

The formula for the dynamic magnification factor parallel to the wind direction has endured more development. Although, the setup roughly looks the same. The parameter for turbulence intensity has been added, which has been discussed shortly in paragraph 2.2.3. Furthermore, B and E have been implemented. B is depending on the height and the width of the building. Compared to the formula from TGB 1972, it could be said that B replaces parameter T_b, since they both reflect the affected surface by the wind load. E is composed of multiple parameters; the eigenfrequency, damping ability of the material and the width and height of the building, which is comparable with T_r. For the formulas of these parameters will be referred to Appendix B.

$$\phi_1 = \frac{1 + 7\, I(h)\, \sqrt{B + E}}{1 + 7\, I(h)\, \sqrt{B}} \tag{2. 30}$$

The formula for the eigen frequency has changed slightly compared to TGB 1972; the value for acceleration has increased from 0.25 to 0.384, which increases the eigen frequency.

$$f_e = \sqrt{\frac{a}{\delta}} = \sqrt{\frac{0.384}{\delta}} \tag{2. 31}$$

Perpendicular to the wind direction is a dynamic factor mandatory for buildings with a height of 100 m or more as well as for the circular buildings for which this already was required in TGB 1972.

Eurocode

The Eurocode does not use a ϕ-coefficient to account for dynamic influence of wind on a building, but it applies a combined coefficient that also represents the dynamic part; $c_s c_d$. This coefficient is called the structural factor. Most of the time, $c_s c_d$ is considered as one coefficient with one formula. However, technically it is built up from two factors, which have their own formula. c_s is the coefficient which accounts for dimensions of the building and c_d is the dynamic coefficient. Formula (2. 32) is the formula which represents the dynamic part of the coefficient.

$$c_d = \frac{1 + 2k_p * I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7I_v(z_s)\sqrt{B^2}} \tag{2. 32}$$

In general, the formula shows resemblance to the formula from TGB 1990. Both formulas incorporate the turbulence intensity (I(h) and $I_v(z_s)$). However, the formula for B has changed by implementing not only the height and width of the building as is done in TGB 1990, but also the turbulence length scale. Furthermore, the height at which the turbulence intensity is taken at height z_s, which is 60% of the total height, instead of at the full height of the building. R is the resonance factor. It captures the same domain as parameter E in the TGB 1990, but the complexity of the formula has increased. A new parameter in the formula is k_p. k_p is the peak factor. It has a minimum value of 3 and it is based on the frequency of a gust and the duration of the reference wind speed.

In the Eurocode, the same rules apply as in TGB 1990 to determine whether the calculation for the dynamic influence has to be performed:

1) Building height is over 50 m.
2) The slenderness ratio h/b > 5.

Both requirements need to be fulfilled. If this is not the case, a fixed value of $c_d = 1.05$ needs to be used. This means that for every building the dynamic influence is taken into account.

In the Eurocode, a very simplified formula for eigen frequencies is allowed (formula (2. 33)). Especially compared to the eigen frequencies according to TGB 1972 and TGB 1990. This formula is empirically determined, instead of the formula's from the TGBs, which are based on theory. Therefore, in practice, the formula from TGB 1990 is sometimes checked as well.

$$f_e = \frac{46}{h} \qquad (2.\ 33)$$

2.3.3 Reduction factors based on dimensions

The overview at the start of this chapter showed that in the formula for wind load, only TGB 1955, TGB 1990 and the Eurocode have implemented a coefficient based on the dimensions of a building. This is different than the shape factor, which is a reduction factor based on the pressure distribution on the surface.

TGB 1990 uses c_{dim} and Eurocode named it c_s. As has been mentioned, c_s is part of construction factor c_sc_d. This factor uses the combined formulas of c_s and c_d. Usually these factors are not calculated separately, but are calculated directly using the combined formula. However, for the sake of comparison, it will be reviewed individually.

TGB 1955

The TGB 1955 states that a reduction factor of 0.85 may be used if one of the main dimensions of a surface is larger than 10 m (Koninklijk Instituut van Ingenieurs, 1955).

TGB 1990

TGB 1990 uses factor c_{dim}. It can be calculated using formula (2. 34), but has the restriction that is has to be smaller or equal to 1.0, which shows that it can only be used to reduce the wind load.

$$c_{dim} = \frac{1 + 7I(h)\sqrt{B}}{1 + 7I(h)} \qquad (2.\ 34)$$

The formula includes parameter B and the turbulence intensity. The turbulence intensity has been mentioned before for TGB 1990 (paragraph 2.2.1). However, for factor c_{dim}, a different formula has been derived, which used a standardised value for the roughness length and factor k. Therefore, it is only depending on height h:

$$I(h) = \frac{1}{\ln\left(\frac{h}{0.2}\right)} \qquad (2.\ 35)$$

Eurocode

The Eurocode uses factor c_s to account for dimensions of the building. A first glance of the formula shows a lot of resemblance.

$$c_s = \frac{1 + 7I_v(z_s)\sqrt{B^2}}{1 + 7I_v(z_s)} \qquad (2.\ 36)$$

Same as for the comparison of the dynamic factor, the set up stays the same. Although the name of parameter B has stayed the same, but it has been has been squared and given a different formula. The formula for parameters B is the same formula as given in paragraph 2.3.2.

For the case where the fixed value of 1.05 is applied for dynamic factor c_d, the minimum value of c_s is 0.81. This results in a minimum combined value of $c_s c_d \geq 0.85$.

Furthermore, if a building with a framework and shear walls is lower than 100 m and the building height is less than four times the depth of the building, the structural factor $c_s c_d$ is equal to 1.0. This may also be assumed when the building height is lower than 15 m. For any other case, the factor needs to be calculated using the discussed formulas for c_s and c_d or the combined formula (Nederlands Normalisatie Instituut, 2011b).

2.4 *Wind load comparison tool*

For the purpose of easily comparing wind loads from different design codes, a wind load comparison tool is created. A special feature of the tool is that it is possible to assign extra levels to a building. The tool is created in Excel, which makes it accessible to every Microsoft Office (365)-user.

Input

The tool requires all the basic specifications of the building as input, such as number of floors, floor height, width, depth and main material properties. Furthermore, the location of the building is a very important input parameter of the wind load. As explained in the previous paragraphs, every design code calculates this differently, so this has to be filled in separately. A screenshot of the sheet with the required input is shown in Figure 2.14. All of the parameters that are not numbers, are to be chosen from a drop down menu and they are further explained if necessary when the cursor passes the Excel cell. In the example shown in Figure 2.14, the original building has 10 levels with a level height of 4.0 m and there are two new levels with a height of 5.0 m each.

To make a proper comparison between the wind loads according to the design codes, the characteristic values will be compared. The partial safety factors that are used to obtain design values instead of characteristic loads were introduced with the TGB 1990; TGB 1955 and TGB 1972 did not yet have partial safety factors and used safety on the side of the material.

Input

Specs of building		
Location	Delft	
Surroundings	Urban	
Distance to the coast (=a)	21000	m
Wind area | TGB 1955	Inland	
Wind area | TGB 1972	Inland	
Wind area | EC/TGB 1990	II	
nr of floors	10	
Floor height	4	
Height (= z_{max})	40	m
Width (=b)	20	m
Depth (=d)	20	m
Radius of corners (=r)	0	m
nr of extra floors	2	
Floor height	5	
New height (= z_{new})	50	m
Roof surface	Very rough	
Material	Concrete	
Weight	400	kg/m^3
Young's modulus (cracked)	15000	N/mm^2
Moment of inertia	4.513E+12	mm^4

Calculation methods	
TGB 1955	
Safety against tilting	No
TGB 1972 / TGB 1990	
Eigen frequency	Eurocode method
Displacement at the top	Simple calculation
Sensitive to vibrations	No
Eurocode	
Vibration shape bending y-axis	Uniform
Vibration shape bending z-axis	Linear
Vibration shape torsion y-axis	Linear
Vibration shape torsion z-axis	Linear
Stability system	Framework and shear walls

Note: For the calculation of the dynamic factor of the TGB 1972, some values need to be entered manually in the TGB 1972 worksheet

Figure 2.14 Input for the wind load comparison tool

Calculation

To calculate the wind pressure, the formulas from section 2.2.1 are used. The tables with wind pressure that are provided in most of the design codes are used to verify the calculated results. Most of the design codes have clearly specified values and formulas that can be implemented to obtain the wind pressure, except for TGB 1972. The formula for wind pressure from the TGB 1972 includes two parameters which have to be deduced from a graph; roughness factor r and gust influence factor T_b. Formulas which reflect these graphs are formulated in order to automatically calculate the wind pressure according to TGB 1972. However, slight alternations of the graphs are necessary, because the obtained values do not line up with the wind pressure values that are presented in the design code itself. In Appendix A.4, this mismatch between values is shown, including the formulated equations that are applied in the wind load comparison tool.

Output

The output is presented in two ways; the first one is on the same sheet as the input. It presents the wind pressure and coefficients corresponding to the dimensions and specifications of the building for each of the reviewed design codes. This way, it is possible to reflect quantitatively on the development of the coefficients as well. Furthermore, it gives the design values of the bending moment and horizontal force, and presents the percentual increase of those forces due to the extra levels that are added. This is visualised as well in three graphs, which is shown in Figure 2.15.

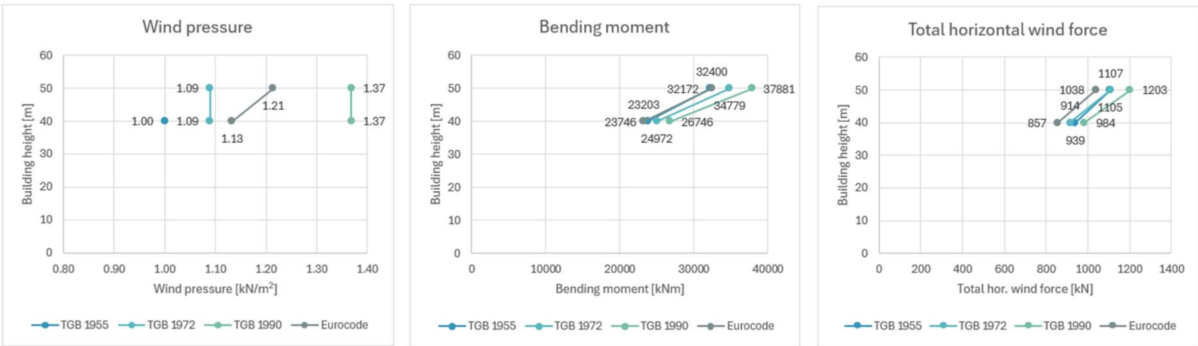


Figure 2.15 Graphs with increase of wind pressure, bending moment and horizontal force due to extra levels

The second way the output is presented is through an overview where the wind pressure, the wind load, the bending moment and horizontal force are given per level. This way, the forces can also be implemented easily in other calculation software. Figure 2.16 an example is given for a building. The extra levels are highlighted (light grey cells) and the increase of the bending moment and horizontal force a given separately in Figure 2.17. Figure 2.18 presents the way the wind loads should be applied to the building over the height. This differs per design code and is often depending on the ratio between the height and width of the building. All these rules are implemented into the tool and it automatically chooses the correct method.

					TGB 1955				TGB 1972				TGB 1990				Eurocode			
Floor nr	Floor height [m]	z [m]	Level height [m]	z to foundation [m]	q_p (z) [kN/m²]	q_k (z) [kN/m]	Bending moment M_k [kNm]	Horizontal force F_k [kN]	q_p (z) [kN/m²]	q_k (z) [kN/m]	Bending moment M_k [kNm]	Horizontal force F_k [kN]	q_p (z) [kN/m²]	q_k (z) [kN/m]	Bending moment M_k [kNm]	Horizontal force F_k [kN]	q_p (z) [kN/m²]	q_k (z) [kN/m]	Bending moment M_k [kNm]	Horizontal force F_k [kN]
0	0	0	2	4	1.00	22.1	177	44	0.71	17.0	136	34	0.93	20.3	163	41	0.90	17.5	140	34.9
1	4	4	4	8	1.00	22.1	707	88	0.71	17.0	545	68	0.93	20.3	650	81	0.90	17.5	559	69.9
2	4	8	4	12	1.00	22.1	1061	88	0.72	17.3	832	69	0.93	20.3	976	81	0.90	17.5	839	69.9
3	4	12	4	16	1.00	22.1	1414	88	0.79	19.1	1219	76	0.93	20.3	1301	81	0.90	17.5	1118	69.9
4	4	16	4	20	1.00	22.1	1768	88	0.84	20.2	1620	81	0.93	20.3	1626	81	0.90	17.5	1398	69.9
5	4	20	4	24	1.00	22.1	2122	88	0.89	21.3	2040	85	0.93	20.3	1951	81	0.90	17.5	1678	69.9
6	4	24	4	28	1.00	22.1	2475	88	0.92	22.1	2477	88	1.02	22.2	2489	89	1.13	22.1	2476	88.4
7	4	28	4	32	1.00	22.1	2829	88	0.95	22.9	2930	92	1.09	23.8	3049	95	1.13	22.1	2829	88.4
8	4	32	4	36	1.00	22.1	3182	88	0.98	23.6	3397	94	1.15	25.2	3631	101	1.13	22.1	3183	88.4
9	4	36	4	40	1.00	22.1	3536	88	1.01	24.2	3877	97	1.21	26.5	4233	106	1.13	22.1	3537	88.4
10	4	40	4.5	45	1.00	22.1	4475	99	1.03	24.8	5026	112	1.26	27.6	5582	124	1.13	22.1	4476	99.5
11	5	45	5	50	1.00	22.1	5525	111	1.06	25.5	6376	128	1.32	28.8	7205	144	1.21	24.1	6031	120.6
12	5	50	2.5	52.5	1.00	22.1	2901	55	1.09	26.1	3431	65	1.37	29.9	3930	75	1.21	24.1	3166	60.3

Figure 2.16 Wind load per level

	TGB 1955	TGB 1972	TGB 1990	Eurocode
Friction on roof $F_{w,k}$ [kN]	-	17.4	21.9	19.4
Total bending moment M_k [kNm]	32172	34779	37881	32400
Total horizontal force F_k [kN]	1105	1107	1203	1038
Consequences of extra levels				
Extra bending moment [kNm]	8426	9807	11135	9197
Extra horizontal force [kN]	166	193	219	181

Figure 2.17 Resulting forces

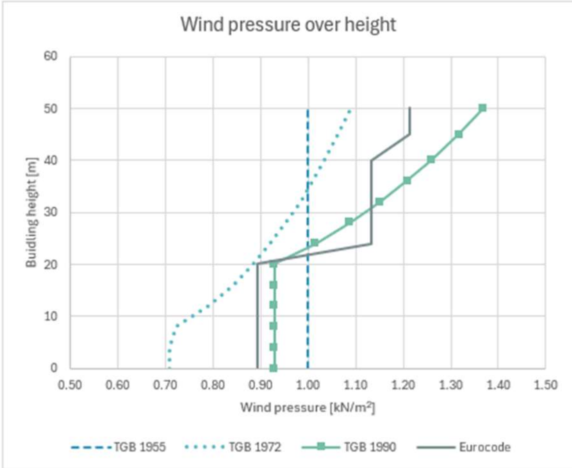


Figure 2.18 Wind pressure on the building

Advised application of tool

The wind comparison tool lends itself perfectly for early inventories regarding the possibility of adding levels to an existing building, because with limited information, this tool gives immediate results that say something on the consequences of adding levels in terms of increase in bending moment and horizontal force. This information can be used to calculate the increase of compression/tension in the foundation. The tool could also help the decision making process on the amount of added levels and their height, since the influence of these changes is visible in just a few clicks. Furthermore, the wind loads per level can be used as input for all different kinds of calculation software, which often require point loads per level instead of line loads.

Improvements

A thing that could be improved to the wind comparison tool is that the extra floors should be calculated according to another design code than the original part of the building. This could be done through a drop down menu where the design code for the original and new part can be chosen.

Another improvement would be to make functions of several parameters of the dynamic magnification factor (φ_1) of the TGB 1972. The gust-energy ratios (F_D and F_L) and the reduction coefficient (S) are now to be obtained from graphs and have to be filled in manually in the wind load comparison tool. Ideally, the tool would calculate those parameters automatically.

2.5 *Conclusion | Overall development of design code for wind design*

The formula for wind load roughly consists of two parts; the extreme wind pressure and the coefficients. The overall development will be reviewed by looking at these parts separately.

Wind pressure

The formulas for wind pressure have developed significantly. This development is easily visualised as well. Figure 2.19 and Figure 2.20 present the extreme wind pressure according to the design codes, but in order to provide a clear overview, a selection of the graphs is chosen for this combined graph. From TGB 1955 (both high and moderate) and TGB 1972, the inland graphs have been used. From TGB 1990 and Eurocode (urban) the graphs of area III have been used. These have been chosen to be compared, due to the expectation that these would be best comparable.

Wind pressure development

The first thing that is clear, is that the moderate wind pressure from the TGB 1955 is significantly lower than all of the others and therefore it is probably a good thing that this was not allowed on all types of structures. On the other hand, the high load from the TGB 1955 is very interesting; even though it has a very 'simple' shape, it is actually not that far off the values of the other codes. It roughly follows the same shape. During the period where the TGB 1955 was in use, high rise buildings were not common. At least, not in the Netherlands. This is best visualised in Figure 2.20; the larger the height of a building, the larger the differences between the TGB 1955 and the newer codes. This can be explained, because TGB 1955 states that everything above 40 m has the same value.

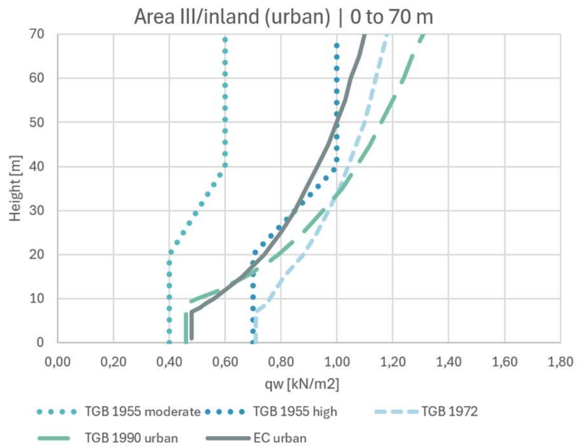


Figure 2.19 Extreme wind pressures 0 to 70m

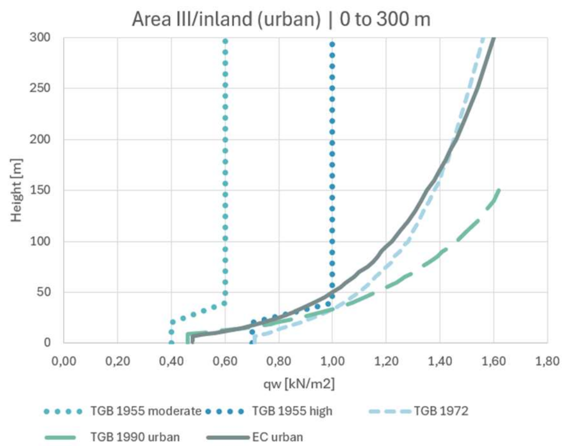


Figure 2.20 Extreme wind pressures 0 to 300m

The graph in Figure 2.19 would almost make one think that for low- to mid-rise buildings, the older codes are not such a bad fit, when just the wind pressures are compared. However, it completely depends on the comparison that is made. Figure 2.21 clearly shows that for a different area, the graphs do not align. This figure shows the coast line wind pressure values of TGB 1955 and TGB 1972, which are the highest that are defined in TGBs. For TGB 1990 and Eurocode, the values from area I are shown. And from area I, the rural and coast wind pressure is chosen. So, for all of the codes, the highest available values are summarized in this figure. The figure visualises the large differences between the design codes already at 70 m. Especially the values of the coast terrain from the Eurocode are significantly larger than the ones from the previous design codes. Figure 2.22 is the same figure, but extended to 300 m.

This clarifies the increase of wind pressure in design codes even more. To be fair, buildings of that height were not being built along the coast of the Netherlands in 1955, so it was not necessary to specify anything in the code about that, but it does show the limitations of the older design codes.

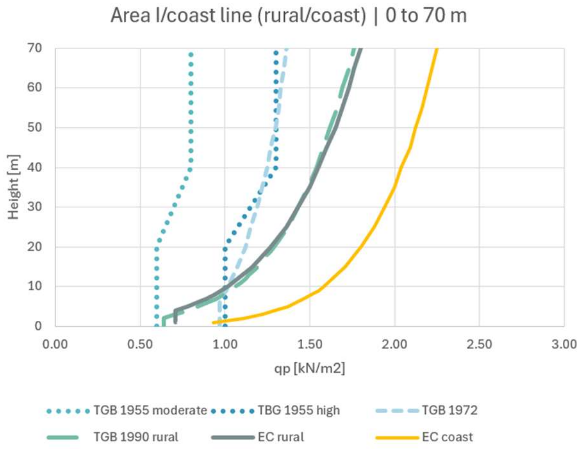


Figure 2.21 Extreme wind pressures 0 to 70 m

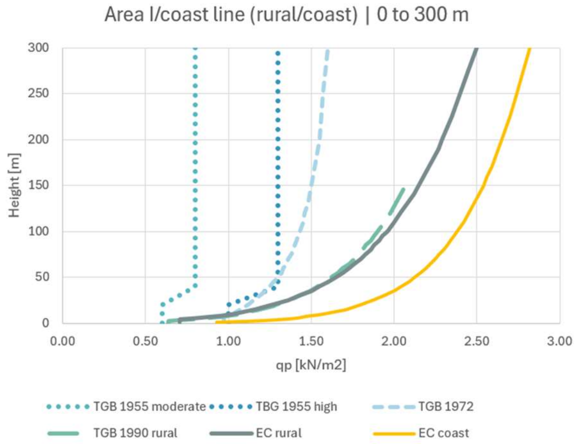


Figure 2.22 Extreme wind pressures 0 to 300 m

In Appendix C, all of the combinations of graphs have added. These comparisons clearly visualize the differences in wind pressures prescribed by the design codes.

Wind area map development
As has been mentioned, the value of the wind pressure depends on the location of the building. This wind area map has developed over the years as well. TGB 1955 had specified three areas in the Netherlands; coast line, coast and inland. The Eurocode still has three areas, but now named them wind area I, II and III. Plus, the borders of the wind areas have changed. Within those areas there is another division between rural, urban and coastal areas. This results in a better fit to the actual situation of the building.

Figure 2.23 shows the comparison between the wind areas according to the TGB 1955 and TGB 1990/Eurocode. According to the TGB 1955 and TGB 1972, the largest part of the Netherlands falls under the 'inland' area and this is still the case for the TGB 1990 and Eurocode, but now it is called wind area III. However, there are some interesting locations along the coastal regions of the Netherlands, where a noteworthy shift in type of wind area is implemented. The most interesting place to look at is the eastern part of the province of Noord-Holland, which is highlighted with yellow in Figure 2.23. As can be derived from the figure, this area falls under the inland area according to the TGB 1955 and TGB 1972, which means the lowest winds pressures that are available in these design codes may be applied in this region. However, since the publication of the TGB 1990, this region is wind area I, which is the region with the highest wind pressures of the TGB 1990 and Eurocode. This could be detrimental, when adding levels to a building with a building year before 1990, since the difference in applied wind pressure could not be larger. Therefore, extra caution is necessary. Other areas that could be critical, are the areas that where inland according to TGB 1955 and TGB 1972, but are wind area II since TGB 1990. The differences between the wind pressures of those wind areas are not as much as for the highlighted yellow zone, but can still be significant. The overview with the wind pressures that applies in this highlighted area can be found in Appendix C.2.

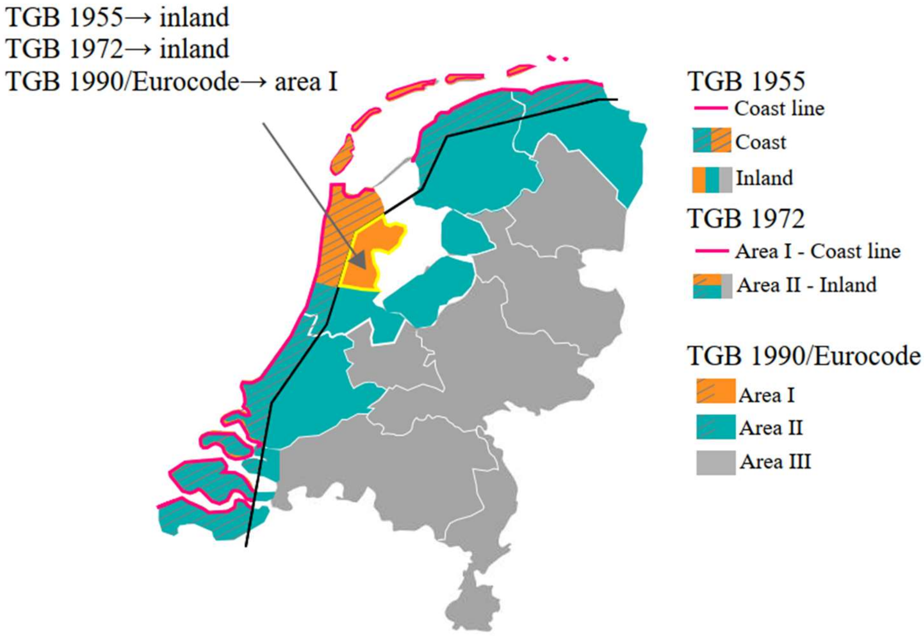


Figure 2.23 Combined map with wind areas

With the development of the design codes, the models of wind load over the height of a building have increased a bit in complexity, but result in a more realistic approach; based on the h/b ratio of the building, a model can be chosen, which results in less wind pressure on certain parts of the building, as compared to TGB 1955 where the maximum wind load had to be applied over the entire height of the building.

Terrain development
Furthermore, since TGB 1972, the roughness of the terrain is implemented in the formulas for wind pressure. This takes into account the terrain in which the building will be situated and has had significant impact on the wind pressure which has to be taken into account. In the Netherlands three kinds of terrain categories are defined in the Eurocode; coast, urban and rural. Using an urban instead of rural area for the calculation of turbulence intensity can cause a 20% difference for a building with a height of 50 m. Logically, the higher the building, the less influence the roughness length of the area has on the turbulence intensity of the building. Although, it is definitely not neglectable. For low- to mid-rise buildings, this factor can have impact on the wind pressure that should be used. In TGB 1990 the graphs for urban and rural merge at a height of 60 m, while a building of 300 m still has to be calculated as urban in Eurocode. It could be argued that the terrain on ground level has limited influence on the extreme wind pressure at 300 m height. Although it is important to note that this also depends on the presence of other high rise buildings.

Overall conclusion wind pressure
The conclusions regarding the development of wind pressure according to the design rules are important to be aware of when designing extra levels on existing buildings. The effects of the developments are location specific, but overall, the maximum wind pressure that needs to be applied increases. And, naturally, when adding extra height to the building, this maximum wind pressure increases even more. This needs to be handled with caution during the design process.

Coefficients
The coefficients are the other part of the formula. The change in these coefficients is partly depending on the number of coefficients and other parameters that are included in the calculation. The parameters incorporate an increasing amount of information about the building as well. This makes the calculations more complex, but also more realistic. A very important factor that has changed over time is the partial safety factor. This is standard in the new codes, but was not used in the TGB 1955 and TGB 1972.

Shape factors
For the shape factors some exact numbers have been given. The external pressure/suction c_{pe} has increased from $0.8 + 0.5 = 1.3$ for TGB 1955 to $0.8 + 0.7 = 1.5$ for the Eurocode for a rectangular building where $h/d = 5$. However, in the Eurocode the lack of correlation factor may be included, which causes a significant reduction. This factor is 0.85, so $0.85 * 1.5 = 1.275$, which is lower than the value from TGB 1955. Compared to the TGB 1972 and TGB 1990, the pressure coefficient did still increase, even when including the lack of correlation factor, since both codes use a value of $0.8 + 0.4 = 1.2$. For buildings where $h/d \leq 3.2$, the pressure coefficient in combination with the lack of correlation factor result in lower values than the TGB 1972 and TGB 1990 as well ((0.8 + 0.61)*0.85=1.1985). This is visualised in Figure 2.24. The internal wind pressure changed even more; the values in the Eurocode are 20-75% larger than in the TGB 1955. By using the force coefficient c_f, the difference can increase even more.

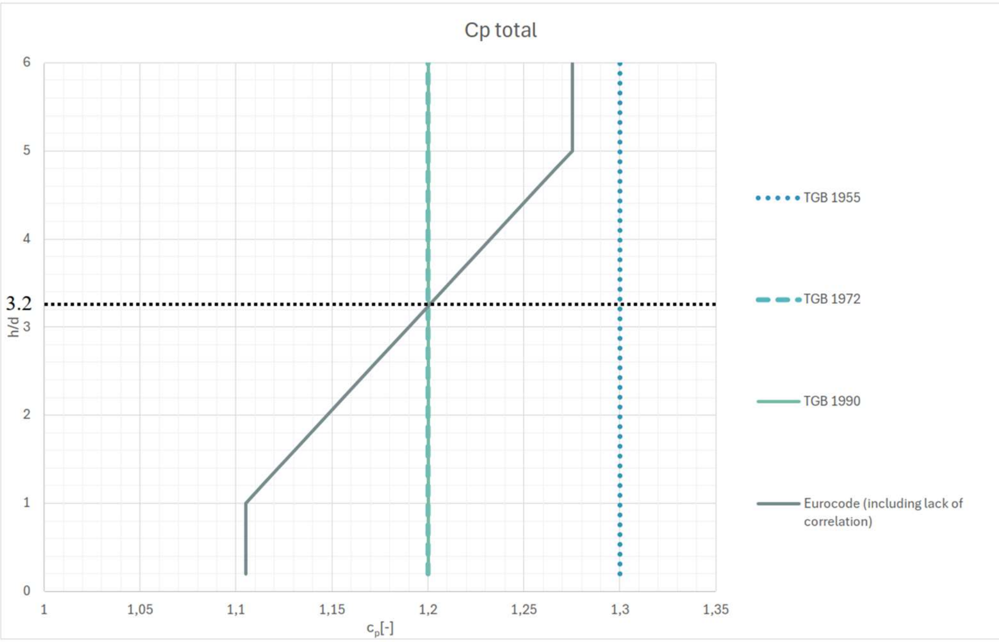


Figure 2.24 Graph of Cp for the design codes

Dynamic factors
For the dynamic factor and the dimension factor, the difference is found in the complexity of the prescribed formulas. With each published code, the complexity increased. With that development, more effects are incorporated into the formula. This makes it more accurate compared to reality. However, the ratio between accurate calculations and time that is spend on the calculations should be acceptable. The dynamic factors all depend on the eigen frequency. In paragraph 2.3.2, the formulas for the eigen frequency are given. TGB 1972,

TGB 1990 and Eurocode all have a different calculation method. The difference between the methods becomes even more clear by visualizing it. Figure 2.25 presents the eigen frequencies, calculated by using the different methods, over the building height. The formula of TGB 1972 and TGB 1990 both depend on the displacement of a building in combination with acceleration. For this example, a simple calculation of h/500 is used to determine this. Whereas, the Eurocode formula is empirical. The graphs shows that with increasing the height, the difference between the eigen frequency based on the Eurocode method and the eigen frequency based on the TGB 1972 and TGB 1990 increases as well.

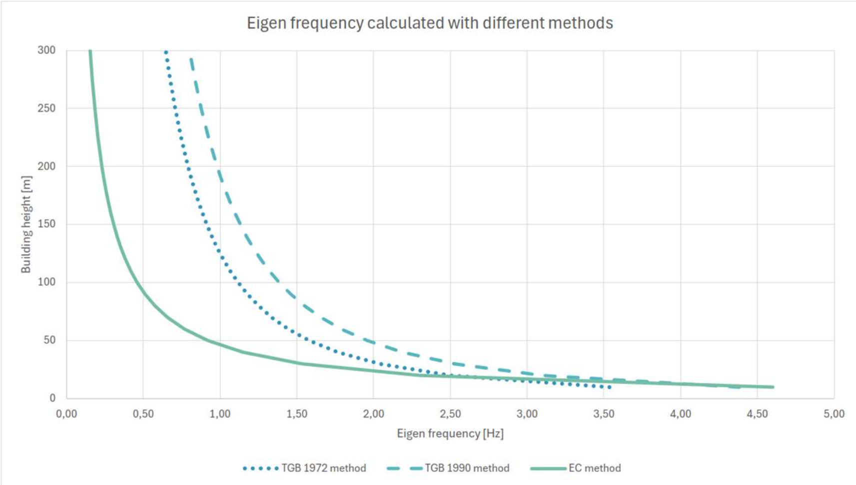


Figure 2.25 Eigen frequency over building height

This significant difference between calculation methods of the eigen frequency has considerable influence on the value of the dynamic factors as well. To express this difference, an example set up has been checked for two design codes, with all three calculation methods for the eigen frequency. In this example, a building is placed in wind area II with urban terrain with variable height and a fixed width and depth of 25 metres. This results in the graphs presented in Figure 2.26. When increasing the height of the building, the difference between the dynamic factor with different calculation methods for the eigen frequency increases simultaneously. Especially between the Eurocode method and the methods from the TGBs. This highlights the importance of carefully calculating this factor and choosing the calculation method for the eigen frequency.

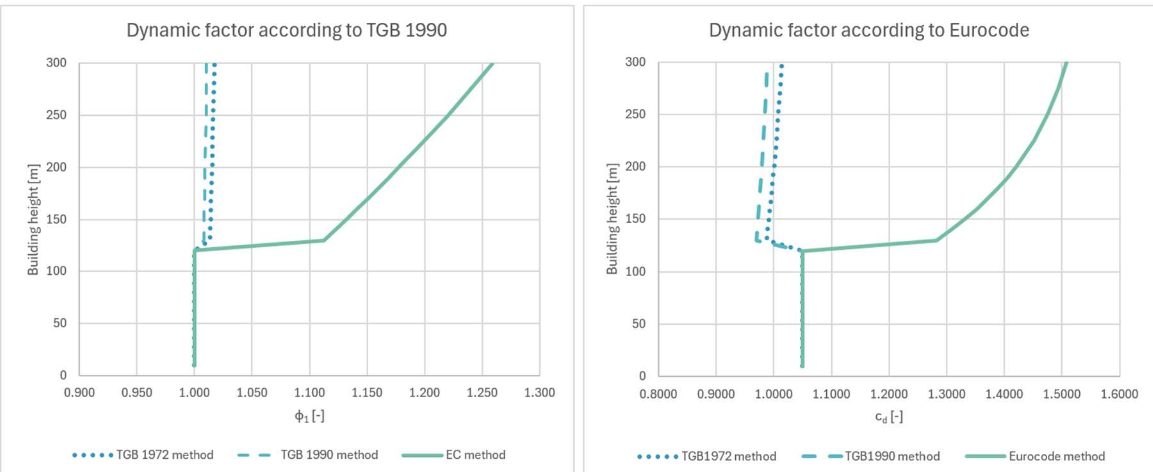


Figure 2.26 Dynamic factors according to Eurocode and TGB 1990

Apart from the difference between the methods for calculating the eigen frequency, the two graphs in Figure 2.26 show that the dynamic factor according to TGB 1990 and Eurocode differ as well. Whether the dynamic factor according to Eurocode exceeds the value according to TGB 1990, is depending on the height of the building and the calculation method of the eigen frequency. The abrupt changes in the graphs are due to the requirements for the dynamic factor calculations. When a building is over 50 m and the H/B ratio is larger than 5, the dynamic factor needs to be calculated. Otherwise the value is fixed.

Overall conclusion coefficients
In general, the number of coefficients that need to be applied increased, as well as their complexity. More aspects need to be taken into account, making the calculation more realistic and specific for a certain building. This is important to be aware of when doing renovations on an existing building, but especially when adding extra levels, since this changes the dimensions, due to the fact that the dimensions and the ratio between those dimensions is for most coefficients leading when determining the values.

3 BUILDING AND LOCATION SPECIFIC WIND LOAD VERSUS WIND LOAD DEFINED BY THE DESIGN CODES

When levels are added to an existing building, the increase in horizontal loads and bending moments can be determined using the design codes. However, it is often unknown how much actual wind load is added to the building. The actual wind load is impossible to know due to uncertainties in measurement equipment for example. Therefore, the terminology of building or location specific wind load is used. The previous chapter showed that the wind pressure increased over the years, according to the design codes, but it is unclear how much of this increase is caused by increase of wind speeds and how much is caused by an increase in safety in the calculation from wind speeds to wind pressures to wind loads.

For an existing buildings where levels are added, the overcapacity of the bearing structure is addressed to accommodate the extra loads. But it is interesting to know the overcapacity on the load side of the calculation as well. In this case the overcapacity means the difference between the wind load according to the design codes and the building or location specific wind load. A structural engineer is not allowed to take this possible extra safety in the wind loads into account when performing the calculations for wind load. Nonetheless, it is of significant importance to have a proper understanding of the representativeness of the wind load according to the design codes.

The previous chapter explored the background of the design codes. Based on the outcome of the background research, a method to determine the building or location specific wind load on a building is described. This method will be applied in the following part, where this wind load is determined. Finally, the comparison between wind load defined by design codes and the building or location specific wind load will be performed. The method to determine the wind load on a building will consist of two parts; wind speed and coefficients. Those are roughly the main parts of the formula for wind pressure. So the simple version of the wind load formula that is assumed here is $q_w = q_p * c$. Figure 3.1 visualizes this.

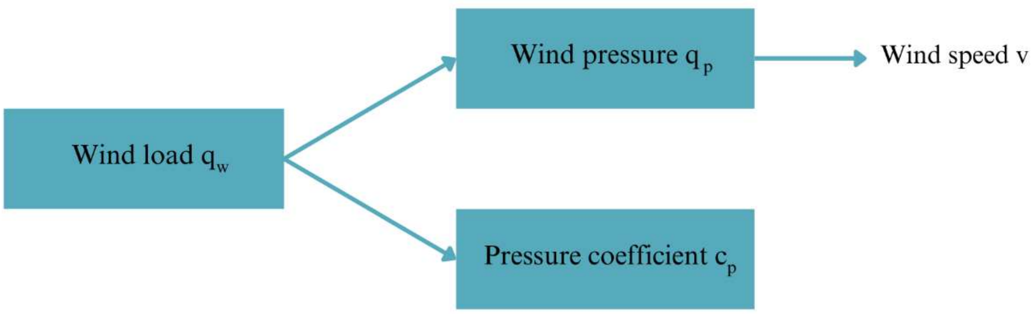


Figure 3.1 Simplification of components of wind pressure

3.1 Method to determine building or location specific wind load

Two methods need to be defined; one to determine the wind speed and one to determine the pressure coefficient. The code provides values for the wind speed for each area in the Netherlands, but for this research, these values will be obtained from data from the KNMI database. The coefficients will be opted to be obtained from software which uses computational fluid dynamics (CFD). Both parts will be explained thoroughly in this chapter.

3.1.1 Wind speed

The method to determine the wind speed is divided into several steps. The first step is the data collection. The KNMI database is used to obtain data, but more parameters need to be specified in order to subtract the correct data sets. Next, the method of data processing is discussed, which will be used to calculate the wind speeds that will be compared with prescribed values by the design codes.

Method of data collection

Weather stations have been placed all over the Netherlands during the last 180 years. The first one was placed in Den Helder in 1843. The quality and reliability of the measurements was not as good as they are right now, but it was a good start to determine the wind climate in the Netherlands. A general overview of wind speed in The Netherlands, provided by C. Braak in 1929, showed the lack of proper wind measurements, because he had to make use of data of obliquely growing trees (Braak, 1929). For a long time, the Netherlands only had five weather stations, which was considered as sufficient for this country. In the 1950s there was a large increase in the amount of weather stations. In that period, the number increased to 30 weather stations, which had better equipment as well. Until 1968, the measurements were stored on punched cards, but in that year the KNMI started digitalizing the measurements (Wieringa & Rijkoort, 1983). This short summary of the history of wind measuring is relevant to this research, because it has influenced the values for wind speed that are used in the design codes.

The measurements that the KNMI collected over the years are available in an open database. Measurements from all weather stations are available (KNMI, 2024). The database has stored four types wind speeds; daily mean wind speed, maximum hourly wind speed, minimum hourly wind speed and maximum wind gust. Eurocode has based it's wind speed on another dataset; the average wind speed over the last 10 minutes of an hour. This data is only available for the reference period of the Eurocode.

Information from 50 weather stations is available on the KNMI database. It is out of the scope of this research to review all of those, so a selection is made, based on location and reliability of the data. From each area (I/II/III) a few stations are chosen to compare. The chosen weather stations are presented in Table 3.1 and highlighted in Figure 3.2.

Table 3.1 Weather stations that will be reviewed (KNMI, 2024)

Number	Weather station	Area (according to Eurocode)	Start date	End date
235	De Kooy[1]	I	01-01-1906	Ongoing
240	Schiphol	II	01-01-1951	Ongoing
260	De Bilt [2]	II	01-01-1901	Ongoing
275	Deelen	III	01-01-1951	Ongoing
280	Eelde [3]	II	01-01-1906	Ongoing
310	Vlissingen	II	01-01-1906	Ongoing
330	Hoek van Holland	II	01-01-1971	Ongoing
380	Maastricht [4]	III	01-01-1906	Ongoing

[1] Weather station De Kooy has been moved in 1972. Measurements have been homogenized.

[2] Weather station De Bilt has been moved in 1951 and equipped with new measurement instruments. However, the weather station shows an unreliable decrease and fluctuations in wind speed measurements after this change (See Appendix D.1). Thererfore, from this weather station no data after 1951 will be used.

[3] Weather station Eelde has been moved from Groningen to Eelde in 1951. Although the data is homogenized, it is not a very reliable station, because it is now located at a house, which cannot be qualified as a rural terrain. Therefore, only data prior to 1951 is used from this weather station.

[4] Weather station Maastricht has been moved from the city to the airport. A short review of the data (Appendix D.1) shows that the data at the old location has lots of fluctuations and therefore cannot be used to draw reliable conclusions. Therefore, only data from the years after 1951 is used from this weather station (KNMI, n.d.)

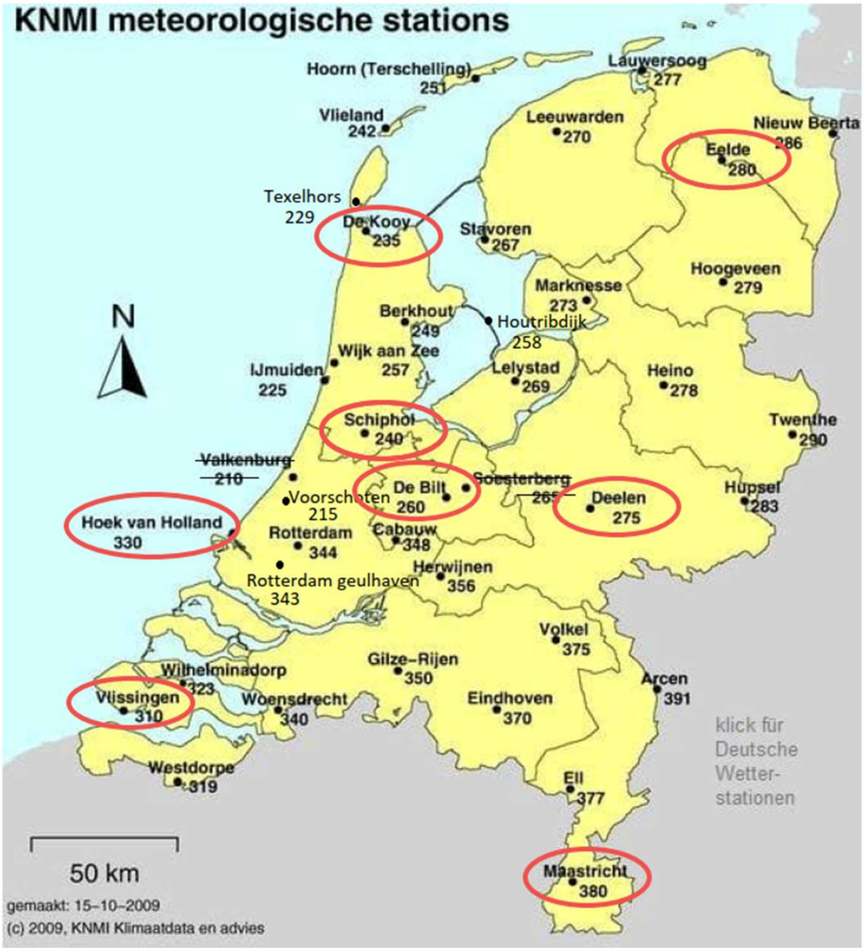


Figure 3.2 Weather stations KNMI (KNMI, 2009)

Some more practical information about the wind measurements;

- The wind is supposed to be measured at a height of 10 m. It does change a little bit for every weather station, but data has been fitted to 10 m height by the KNMI in case of large differences.
- The KNMI provides daily mean/maximum/minimum values. If they are expected to be wrong, they get removed from the database. Therefore, it is assumed that the data that the KNMI provides is correct and no further validation of the KNMI data will be performed in this research.

Method of data processing
The data that is downloaded from the KNMI database is stored as a csv file. A common way to process this kind of data is by using Python. Python has a lot of options to plot data and is relatively easy to test different distributions. That is important, because distributions are used to obtain wind speeds for a certain return period. There are no strict guidelines regarding the proposed distributions that are used for evaluation of wind. However, usually the Normal- (Gauss), Weibull- or Gumbel distribution are used (Wever, 2009).

Reference period
For each design code a reference period of 25 years will be used. Literature gives different periods that could be used in a range of 20 to 30 years, and 25 years is assumed to be acceptable for this purpose (Van Staalduinen, 1992 & Vindteknikk et al., 2013).

Type of measurements
The datasets with day observations that can be downloaded from the website of KNMI store four types of wind speed measurements; daily mean windspeed, maximum hourly mean wind speed, minimum hourly mean wind speed and maximum wind gust. Most interesting for this research will be the maximum hourly mean wind speed. KNMI provides hourly observations as well. This data has been used by the Eurocode to determine wind speeds. This is the average wind speed during the 10 minutes preceding the time of observation (KNMI, 2024).

An example of a measurement set is presented in Figure 3.3. Using this data two sets are created. The first one is the yearly maxima list. This one stores the maximum wind speed value of each year in the data set. These points are highlighted with the red dots. The other list is created using the Peak Over Threshold (POT) method. This is a function which returns all points which are higher than a specified value (the threshold) and are at least a certain amount of hours apart from each other. Both of these values need to be given as input for the function. The threshold is made depending on the dataset; the value is obtained by calculating the upper 95% quantile and this value is used as threshold. This means that it is a different value for each dataset. The threshold in Figure 3.3 is 13.9 m/s and is visualised by the upper horizontal black line. All dark blue dots of the POT maxima list are above this line, but yearly maxima values can be below this value. Therefore it is interesting to review both data sets. For this research is specified that the points of the POT list need to be at least 48 hours apart from each other. This is to make sure that not multiple high values from the same storm are processed.

For this figure it should be noted that the dots representing the POT maxima dataset have been given an offset, in order to makes both dots visible in case the POT maxima and yearly maxima have the same data point. So, the POT maxima value is not higher than the yearly maximum value.

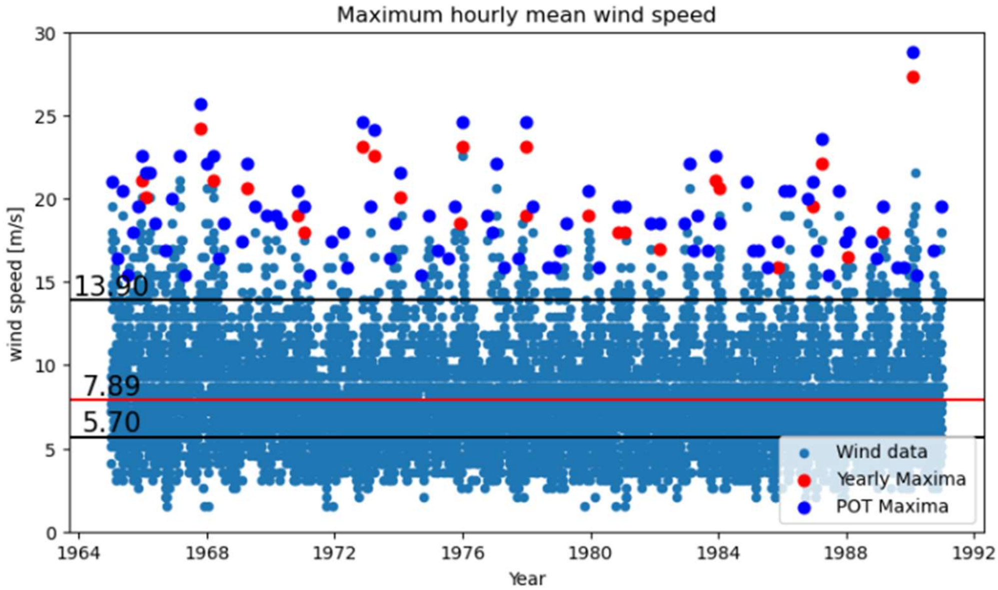


Figure 3.3 Measurement set of weather station Schiphol for period 1965-1990

Distributions

To describe the chance on extreme events, often an extreme-value distribution is used, such as the Generalized Extreme Value distribution (GEV). The GEV distribution is a common tool to use for cases of block maxima. The Gumbel distribution is a special type of the GEV distribution. The cumulative density function (CDF) for the Gumbel maximum distribution is:

$$F(y) = e^{-e^{-x}} \tag{3. 2}$$

Where x is the normalized Gumbel variable:

$$x = \frac{y-\mu}{\sigma} \tag{3. 3}$$

In this formula, y is the event that is described by the distribution. In this case y is the wind speed. The CDF calculates the cumulative chance that variable Y is equal or smaller than y. Variable μ is the location parameter and σ is the scale parameter, which must not be confused with the mean and standard deviation from a dataset, which are usually presented by the same Greek characters. These parameters can be calculated with the method of moments (MOM) (Millard & Kowarik, 2023):

$$\mu = \bar{y} - \gamma * \sigma \tag{3. 4}$$

$$\sigma = \frac{\sqrt{6}}{\pi} * s_m \tag{3. 5}$$

$$s_m^2 = \frac{1}{n}\sum_{i=1}^{n}(y_i - \bar{y})^2 \tag{3. 6}$$

$\gamma = Euler's\ constant = 0.577$ ... (Havil, 2010)

$\bar{y} = mean\ of\ the\ dataset$

The other distribution is the Weibull distribution. The CDF of the Weibull distribution is given by formula (3. 7). The formula is depending on two parameters, apart from value x for which the probability needs to be determined. Those parameters are the shape parameter k and scale parameter λ. The shape parameter describes the shape of the distribution, which can be left- or right-skewed, steadily decreasing or more like a normal distribution. The scale parameters represents the variability of the dataset.

$$F(y, \lambda, k) = 1 - e^{-\left(\frac{y}{\lambda}\right)^{k}} \tag{3. 7}$$

The values of the shape and scale parameter can be determined with several methods, such as the maximum likelihood method (MLM), method of moments (MOM) and the least squares method (LSM). The most commonly used is the MLM, which will also be applied in this research. The method gives the following formulas for determining the parameters (Mahmood et al., 2020):

$$k = \left[\frac{\sum_{i=1}^{N} x_i^k \ln(x_i)}{\sum_{i=1}^{N} x_i^k} - \frac{\sum_{i=1}^{N} \ln(x_i)}{N}\right]^{-1} \tag{3. 8}$$

$$\lambda = \left(\frac{\sum_{i=1}^{N} x_i^k}{N}\right)^{1/k} \tag{3. 9}$$

Repetition time

The wind speed that is implemented into wind load design needs to have a certain repetition time. Depending on the design code, this time can differ. The repetition time can be determined using the CDF. $F(y)$ gives the probability that the value y is lower than a certain value. For the repetition time, the probability is needed that the value is higher than a certain value. This probability can be obtained like this: $1 - F(y)$. The repetition time is inversely proportional to this obtained probability, so it can be calculated like this (Wever, 2009):

$$R(y) = \frac{1}{1 - F(y)} \tag{3. 10}$$

Where $R(y)$ is the repetition time.

To obtain the wind speed for a certain return period, formula needs to be transformed. Furthermore the inverse formula of the CDF's of Gumbel and Weibull are needed. These formulas become:

Probability of return period R: $$F = 1 - \frac{1}{R} \tag{3. 11}$$

Inverse of Gumbel's CDF $$y = \sigma * -ln(-ln(F)) + \mu \tag{3. 12}$$

Inverse of Weibull's CDF $$y = \lambda * (-ln(1 - F))^{1/k} \tag{3. 13}$$

Using these transformed formulas, a return value plot can be created. An example of this plot is shown in Figure 3.4. The extrapolated Gumbel and Weibull distributions are shown and the wind speeds are shown for a return period of 12.5 years. This can be altered in the python script, depending on the design code. The wind speeds given a certain return period can be compared to the values prescribed by the design code under evaluation.

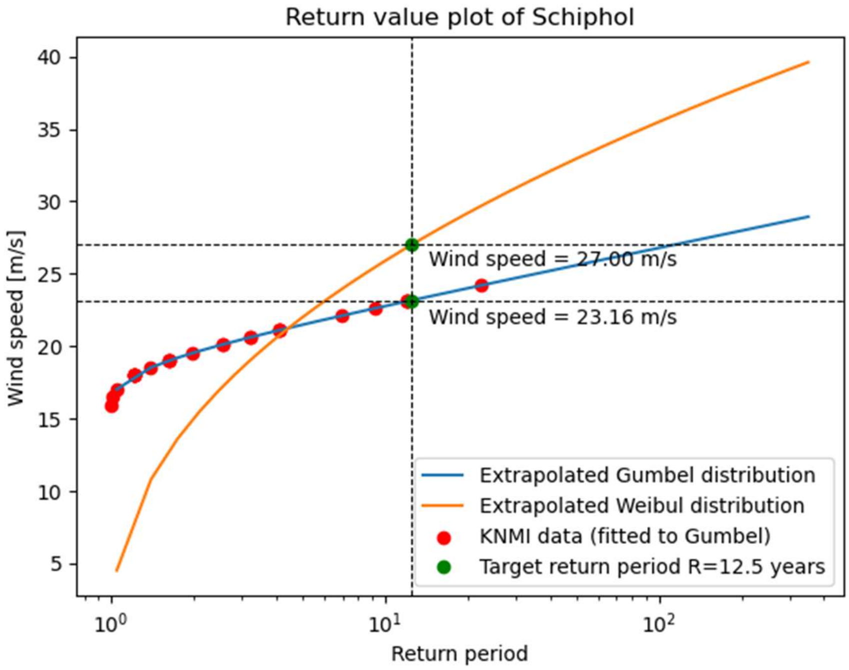


Figure 3.4 Return value plot of Schiphol for period 1965-1990

3.1.2 Pressure coefficients

The second part of the formula for wind load consists of the pressure coefficients. Several methods were considered to obtain this information, such as wind tunnel testing and Computational Fluid Dynamics (CFD). For this thesis, multiple tests need to be performed to determine the influences of change in height and/or shape. Wind tunnel testing is too time consuming for this purpose and is therefore not considered a good fit for this type of research. The other option is CFD. CFD is relatively new and therefore extensive validation is necessary to prove the quality of the outcome. Therefore, the outcomes of the CFD model will be compared with wind tunnel test of buildings with similar dimensions. In any case, the outcome can be used qualitatively.

First, some relevant background on CFD will be given, followed by an explanation of the applied software package and its parameters. Finally, the method of validation of the CFD simulation is described.

CFD

Fluids, such as water and air, can take any shape and deform under the application of forces. Fluid dynamics is a subdiscipline of fluid mechanics and describes the flow of fluids. Fluid dynamics is based on three fundamental principles:

1) Conservation of mass
2) Conservation of momentum
3) Conservation of energy

For each of these principles, equations can be derived. The total system of these equations is called the Navier-Stokes equations and CFD is the use of numerical methods to solve this set of equations. The first step is grid generation, which is also known as defining a mesh. The three main methods for solving the equations are the Finite Difference Method (FDM), the Finite Element Method (FEM) and the Finite Volume Method (FVM).

Reynolds-averaged Navier-Stokes (RANS) and Large Eddy Simulations (LES) are two turbulence models that are widely applied to account for the turbulent flows of fluids. (Blazek, 2015). For RANS k-ω and k-ε are the most popular methods, due to their robustness and because they are relatively computationally cheap. The methods are used to solve transport equations, using the kinetic energy (k) and either the specific dissipation rate (ω) or the kinetic energy dissipation rate (ε). RANS k-ε is robust and computationally cheap. However, RANS k-ω provides better results in at the near wall region. It requires a higher mesh resolution, which increases the calculation time (Dlubal Software, 2024a). The RANS k-ω Shear Stress Transport (SST) model combines k-ω and k-ε and takes it bests properties. This gives it a better performance than the standard models (Menter, 2009).

The amount of research on CFD in wind engineering has increased over the years, which is related to the improvements regarding computer technology, making it possible to run more complex models. The Eurocode states: “In supplement to calculations, wind tunnel tests and proven and/or properly validated numerical methods may be used to obtain load and response information, using appropriate models of the structure and of the natural wind.”. This implies that, in theory, a CFD may be used if it fulfils all of the other requirements, such as proper validation. In some cases it is used to determine pedestrian comfort at street level. However, in practice it is not yet an excepted method to determine wind pressure and coefficients on buildings (Fransos & Lo Giudice, 2015).

Software package & parameters
The software package that will be used is RWIND by Dlubal Software. In this program, 3D constructions are uploaded and furthermore, only a wind load description is required (Dlubal Software, 2024c). The wind load description can be filled in in either RWIND or RFEM. RFEM is another software package by Dlubal Software and is used for Finite Element Analysis (FEA). For this research, RFEM is used to construct the models and those models are then imported to RWIND to perform a wind analysis. The input parameters of the wind load description will be discussed shortly in Table 3.2.

Table 3.2 Wind analysis settings

Parameter	Applied value	Explanation
Type of flow	Steady flow	For the type of flow, either a steady or a transient flow can be applied. For the purpose of this research, a steady flow is sufficient. Besides that, a transient flow calculation increases the calculation time significantly, which is not beneficial for the progress of the research.
Air density (ρ)	1.25 kg/m^3	Fixed value given in Eurocode
Kinematic velocity	0.000015 m^2/s	Fixed value for air at 15° (standard applied value) (Cadence CFD Solutions, 2022)
Turbulence model	RANS k- ω SST	The software gives a choice between two turbulence models: RANS k-ω SST and RANS k-ε. RANS k-ω SST is the best fit for this purpose.
Numerical solver type	OpenFOAM	OpenFOAM is the standard solver applied in RWIND. It is free, open source software for CFD calculations. It uses the FVD method (OpenFOAM, 2023).
Finite volume mesh density	30 %	Standard setting is 20%, which is increased in order to obtain more detailed results.
Residual target value	0.001	This is the target value of the residual. The programs does a number of iterations (a minimum or maximum of this value can be chosen), which gives results with a residual with a minimum of this target value.
Minimum nr of iterations	300	Fixed value
Maximum nr of iteratons	800-1000	Normally, 500 iterations should be enough to reach the residual target value. However, the larger and more complex the model, the larger the number of iterations becomes. Most of the times 800-1000 iterations will be enough, but with an increased mesh density, the number of required iterations will increase as well.

To determine the pressure coefficient, RWIND uses equation (3. 14). The acronyms for the parameters in the equation are not exactly the same as the formula discussed in the background paper of the Eurocode (formula (2. 26)). However, the principle is the same.

$$c_p = \frac{p - p_\infty}{\frac{1}{2} * \rho * v_\infty^2} \tag{3. 14}$$

$p = Static\ pressure$

$p_\infty = Static\ pressure\ in\ the\ freestream$

$\rho = Fluid\ density = 1.25\ kg/m^3$

$v_\infty = Freestream\ velocity\ of\ the\ fluid$

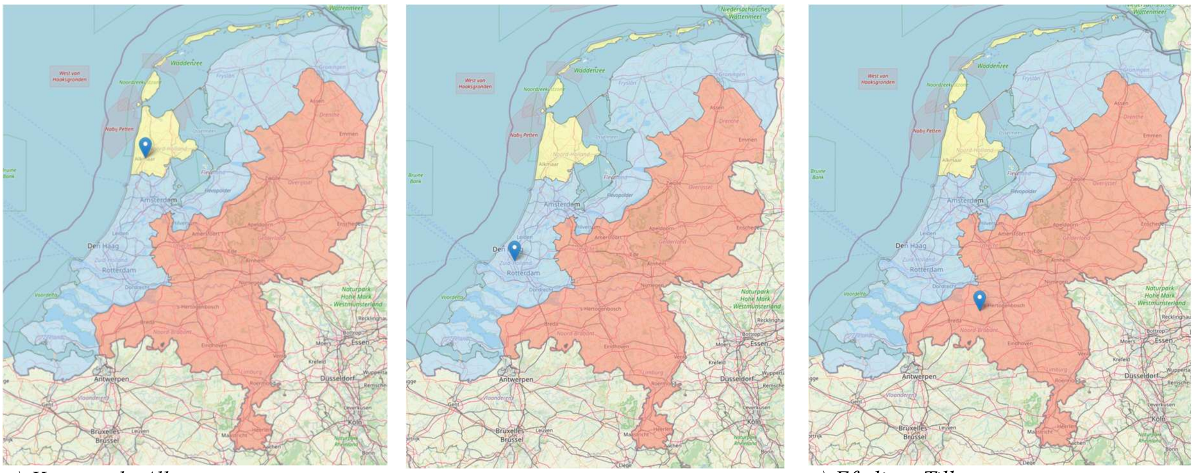
(Dlubal Software, 2024b)

Locations

Furthermore, the wind profile needs to be set up. This starts by choosing a location. The software implemented the wind areas into google maps, so by entering the address of the location, the entire wind profile is generated with the right wind velocities. The only thing that has to be changed manually the terrain category. A dropdown menu gives the choice between 0, II and III. By changing this, the terrain factor k_r is changed automatically. For this research, three locations are considered. One for every wind area in the Netherlands. These locations are presented in Table 3.3 and Figure 3.5. One of the variants will be run in all three locations, in order to prove that this does or does not influence the coefficient that applies on the building surface. Same goes for the terrain category.

Table 3.3 Locations of this research

Wind area	Location	Adress
I	Kaasmarkt, Alkmaar	Waagplein, 1811 JP, Alkmaar
II	Faculty of Civil Engineering TU Delft, Delft	Stevinweg 1, 2628 CN, Delft
III	Efteling, Kaatsheuvel	Europalaan 1, 5171 KW, Kaatsheuvel



a) Kaasmarkt Alkmaar *b) Faculty of Civil Engineering* *c) Efteling, Tilburg*

Figure 3.5 Locations on map with wind areas

Variants

To start, several variants are reviewed. Different combinations of building dimensions are checked to see how representative the wind pressure and pressure coefficient prescribed by the design codes are, since these values are depending on the height/depth ratio or none ratio. Table 3.4 presents the buildings that are used to start the comparison. Variant 1 is rectangular and variant 2 and 3 have a square plan. Variant 3 has rounded corners and the influence of increasing this will be checked by reviewing to radii.

Table 3.4 Dimensions of variants

	Variant 1			Variant 2		
b [m]	50			30		
d [m]	20			30		
h [m]	30	50	70	30	50	70

	Variant 3			
b [m]	20			
d [m]	20			
Radius [m]	0	1	3	6

Considered zones pressure coefficient
The most common zones to consider for the wind load are zone D and E, which are highlighted in Figure 3.6. Zone D is the windward side of the building and zone E is the leeward side. The values that correspond with these zones may be added up and multiplied with the wind pressure. For zone D, the fixed value is +0.8 [-]. The value of the coefficient of zone E is depending on the height/depth ratio of the building. The guidelines provided by the Eurocode for this are discussed in paragraph 2.3.1. Depending on the shape of the building, more zones should be taken into account for the calculations of wind loads. However, this thesis only considers rectangular shaped buildings with a flat roof, for which zone D and E are usually the only shape coefficients that are used, so other coefficients are outside the scope of this thesis.

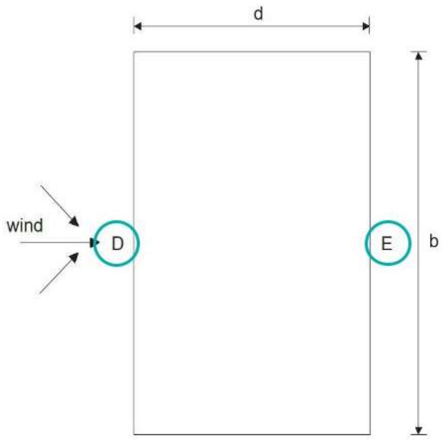


Figure 3.6 Zones for shape coefficient from Eurocode (Nederlands Normalisatie Instituut, 2011b)

Validation of results
The validation of the results from RWIND will be performed by comparing the outcome with results from wind tunnel research on similar building typologies. These wind tunnel test results are obtained from an open data base of the Tokyo Polytechnic University, who tested 22 models of buildings with different width/depth and width/height ratios (Tokyo Polytechnic University, 2003). From this data base, contour plots with values of the pressure coefficient c_p can be obtained. Figure 3.7 shows the contour plot of a building with a height:width:depth ratio of 3:2:1, which is relatively similar to variant 1 with a height of 70 m (h:b:d = 3.5 : 2.5 : 1).

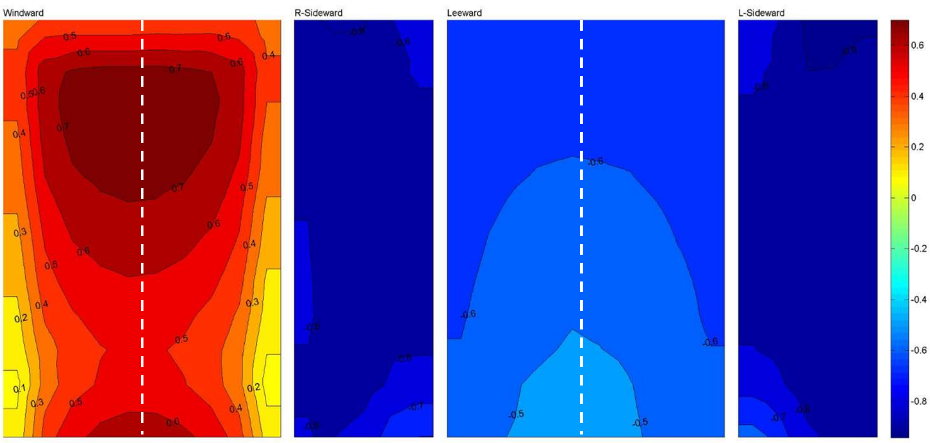


Figure 3.7 Mean wind pressure coefficient on a building with ratio h:b:d 3:2:1 (Tokyo Polytechnic University, 2003)

The contour plots present the mean values of the pressure coefficient. This standard for wind tunnel research, but is also in line with the Eurocode. By multiplying the mean value of the pressure coefficient with the maximum wind pressure, the maximum wind load is obtained (Geurts & Bouwen met Staal, 2012). However, it should be noted that by using the mean, there remains some uncertainty in the values. The root mean squared (RMS) contour plot is automatically calculated as well. This accounts for those uncertainties. However, it cannot be said with certainty where in the calculation is accounted for this uncertainty of the pressure coefficient.

The data along the white, dotted lines from Figure 3.7 can be transformed into graphs using a Python script, making it possible to compare this data with the data from RWIND and the Eurocode. This way, the results from RWIND can be validated.

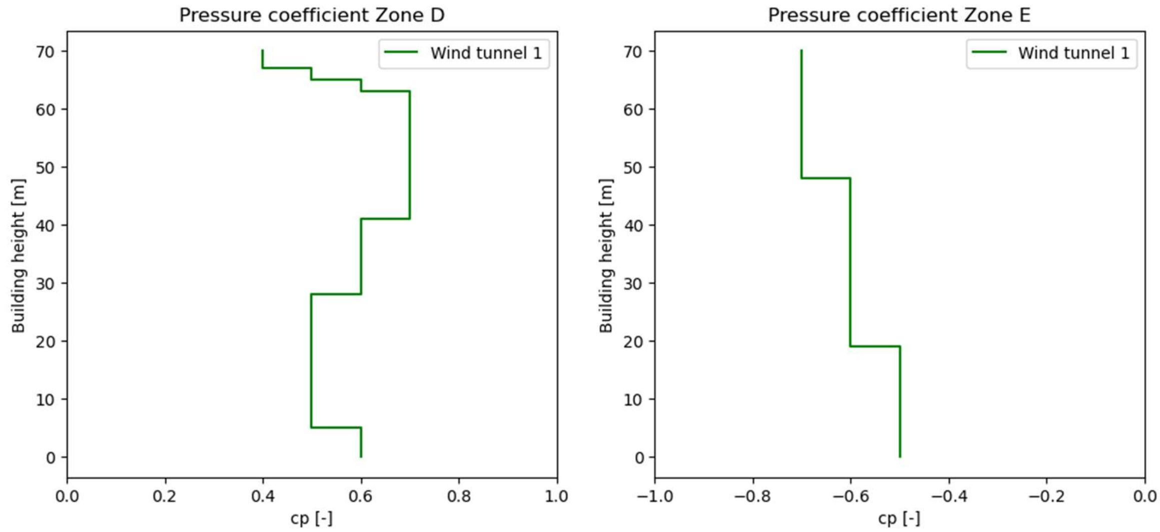


Figure 3.8 Graphs of pressure coefficient in zone D and E for wind tunnel experiment 1

Another wind tunnel experiment by the Tokyo Polytechnic University that is interesting, is one with a ratio of h:b:d = 4:3:1. Table 3.5 provides an overview of the ratios of the variant as well as of the two wind tunnel tests and the ratio of variant 1 lies in between the ratios of the two wind tunnel tests, so both of them will be compared to the pressure coefficients calculated by RWIND.

Table 3.5 Dimension ratios

	Variant 1 dimensions	Variant 1 ratios	Wind tunnel test 1	Wind tunnel test 2
h	70 m	3.5	3	4
b	50 m	2.5	2	3
d	20 m	1	1	1

The second wind tunnel test with ratio h:b:d of 4:3:1 is presented in Figure 3.9. Especially for zone D, this contour plot has some significant differences with the other wind tunnel test. The first difference is the maximum value of the pressure coefficient. This is 0.9 [-], whereas for wind tunnel test 1, the maximum value was 0.8 [-]. The second difference is even better visible in the graph of the pressure coefficient in Figure 3.10, which are the low values of the coefficient for the part below 10 m. These differences can only be caused by the shape of the building. Interestingly is that this difference in ratios has less impact on the leeward side of the building. Both buildings have a stepwise increase from around -0.4 to -0.7 [-]at the maximum building height. This is important to note, because the Eurocode prescribes a fixed and constant value for the pressure coefficient on the windward side, which is +0.8 [-] and a varying pressure coefficient for the leeward side, which is depending on the height/depth ratio of the building.

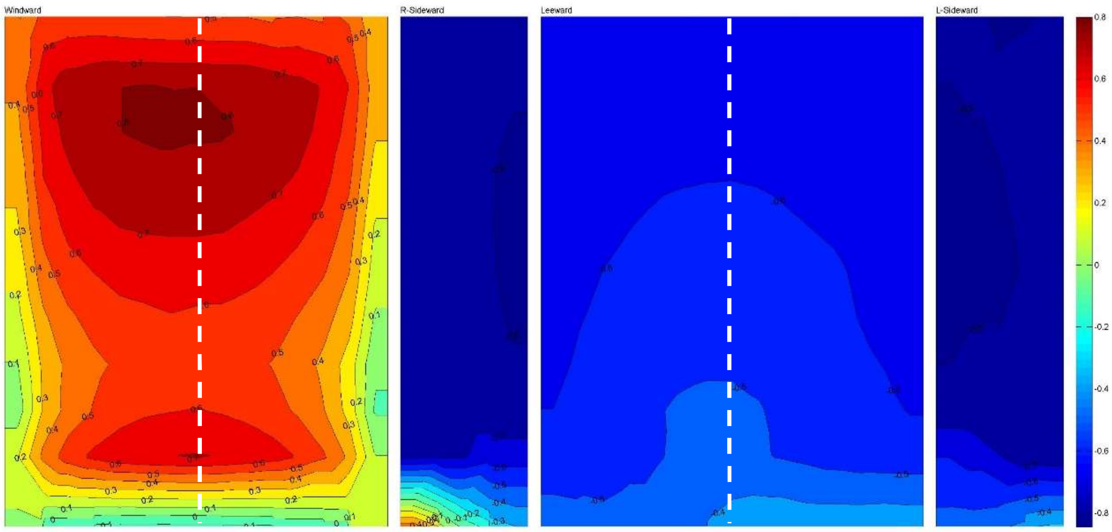


Figure 3.9 Mean wind pressure coefficient on a building with ratio h:b:d 4:3:1 (Tokyo Polytechnic University, 2003)

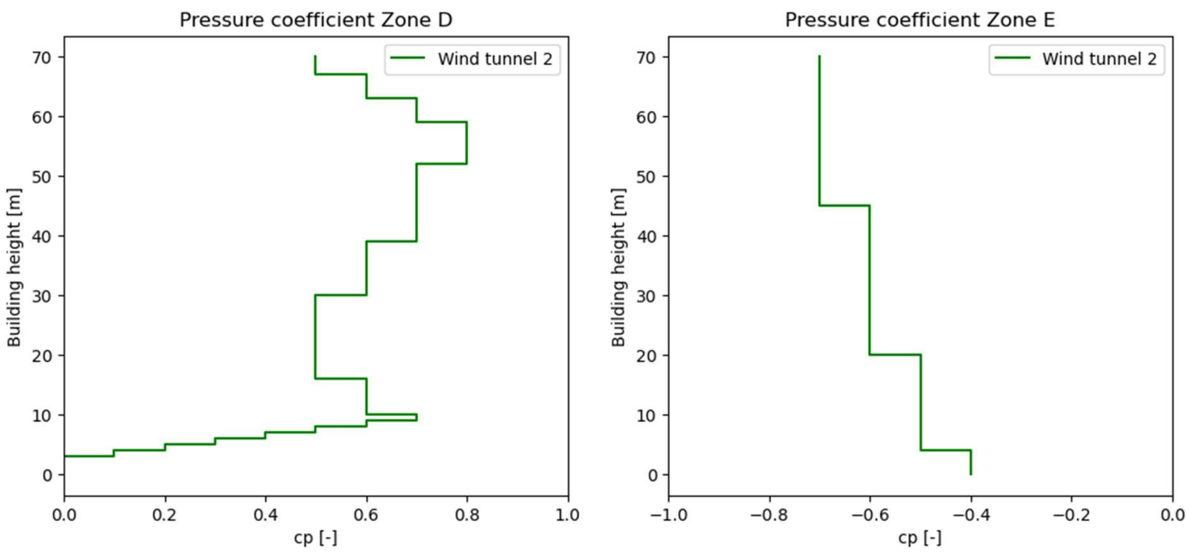


Figure 3.10 Graphs of pressure coefficient in zone D and E for wind tunnel experiment 2

3.2 *Difference between data and wind load defined by the design codes*

This subchapter will discuss the results of the processed data and compare it to the design codes. Wind speed and the wind pressure resulting from this, will be discussed first. Next, the pressure coefficients, calculated by the CFD model in RWIND, will be evaluated. Together, this will answer the question: How representative is the wind load?

3.2.1 Results

3.2.1.1 Wind speed

The data from the wind speed measurements is compared to the wind speeds that are used in the design codes. This could give insight into the conservativeness of the design code. For each design code a comparison will be made with data from the representative time period.

TGB 1955

This design code does not provide the used wind speeds. Therefore, the first step is to transform the formula for wind pressure (formula (2. 1)) so it can be used to determine the wind speeds.

$$q_p = \frac{v^2}{1600} \rightarrow v = \sqrt{1600 * q_p} \qquad (3.\ 15)$$

TGB 1955 makes a distinction between high and moderate wind pressures. Table 3.6 provides an overview of the obtained wind speeds for the different areas and for the high and moderate wind pressure that are specified in the design code.

Table 3.6 Wind speed based on wind pressure from TGB 1955

Height	Moderate wind speed $[m/s]$			High wind speed $[m/s]$		
	Coast line	Coast	Inland	Coast line	Coast	Inland
$0 < h \leq 20\ m$	30.98	28.28	25.30	40.00	36.88	33.47
$20 < h \leq 40\ m$	Interpolate			Interpolate		
$h > 40\ m$	35.78	33.47	30.98	45.61	42.90	40.00

The next step is selecting the correct weather stations and period that has to be evaluated. In the TGB 1955 is stated that data from weather station De Bilt is used. De Bilt falls under the ‘inland’ area. Two other weather stations (De Kooy and Vlissingen) are reviewed as well to compare the wind speed values of the ‘coast line’ area as well. Figure 3.11 shows the wind area map of the TGB 1955, where the considered weather stations are indicated. For all codes a reference period of 25 years of measurements is used. Therefore, for TGB 1955 the period of 1929 to 1954 is evaluated. Since the design code is based on one maximum wind speed measured at De Bilt, occurring during a powerful storm, it can be assumed that statistics did not play a role in this design code. Nonetheless, this research will make use of statistics to determine whether statistics in combination with the maximum wind speeds will result in similar values.

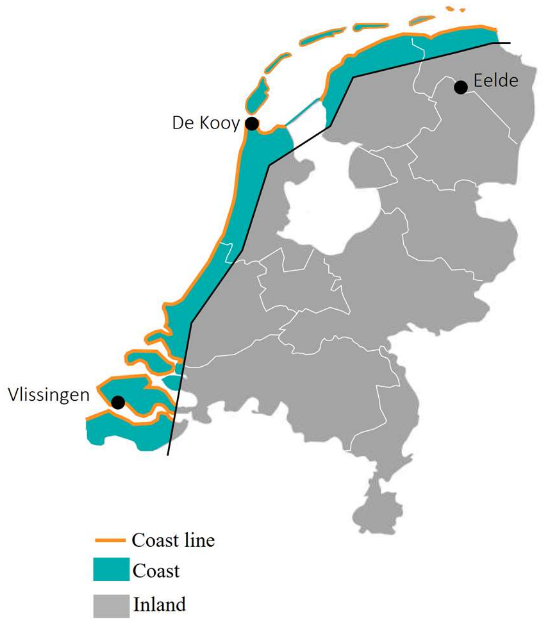


Figure 3.11 Wind area map TGB 1955 with weather stations

Since statistics have not been used to determine the wind speed for the TGB 1955, the codes does not define a return period as well. Therefore 50 years has been assumed. Standard distribution for extreme values are the Gumbel distribution and the Weibull distribution, as has been explained. It is not specified what type of distribution is used for this code and therefore, both Gumbel and Weibull distributions are reviewed. In Appendix D.2 the return plot are given for both the yearly maxima and the POT values.

First the comparison is made between the yearly maxima and POT values obtained from the maximum hourly mean wind speeds measured by the weather stations. This comparison is shown in Table 3.7, where the obtained values are compared to the high wind speeds from TGB 1955. From the TGB 1955, the high wind speeds for h ≤ 20 m are used, since the weather stations measure the wind speeds at a height of 10 m. The values generated by the Gumbel and Weibull distributions are significantly lower than the wind speeds according to TGB 1955.

Table 3.7 Comparison of wind speed values from return plot and design code values (high wind loads)

R=50y		Wind speed [m/s] from distribution:		Wind speed according to TGB 1955	Differences compared to TGB 1955			
Yearly maxima	**Wind area**	**Gumbel**	**Weibull**		Δ_{gum} [m/s]	Δ_{gum} [%]	Δ_{wei} [m/s]	Δ_{wei} [%]
De Kooy	Coast line	**28.7**	40.0	40.0	-11.3	-28	-0.0	0
De Bilt	Inland	**26.4**	25.9	33.5	-7.0	-21	-7.6	-23
Vlissingen	Coast line	**32.4**	27.9	40.0	-7.6	-19	-12.1	-30
POT								
De Kooy	Coast line	25.5	27.4	40.0	-14.5	-36	-12.6	-32
De Bilt	Inland	21.5	20.2	33.5	-12.0	-36	-13.3	-40
Vlissingen	Coast line	26.7	24.0	40.0	-13.3	-33	-16.0	-40

Wind speeds this high have not been encountered during the reviewed period. A maximum hourly mean wind speed this high did not even occur during the full life time of the weather stations. The wind speed of a Gumbel distribution with a return period of 100 years comes a little closer to the value prescribed by the code. Then it is approximately 28 m/s compared to 33.47 m/s for weather station De Bilt.

However, this is not a fair comparison, since the values that are prescribed in the TGB 1955 have been increased significantly to reach this value. The research on the background of the design code (paragraph 2.2.1) shows that the original value, on which the code is based, is 29.0 m/s. When comparing the data from De Bilt with this value, the difference seems to be acceptable. With the Gumbel distribution, the obtained value is only 2.56 m/s lower than this 29.0 m/s.

A small, but interesting side note is that the 29.0 m/s has never been measured at De Bilt. The KNMI data gives a maximum 26.8 m/s during the entire life time of weather station De Bilt. This figure is added in Appendix D.2. The two other weather stations that have been reviewed show higher values, as can be expected at the coast, but the design code and the article by Schoemaker and Wouters (1932) gives the impression that this is entirely based on measurements at De Bilt.

In conclusion, it can be said that the Gumbel distribution with the yearly maxima dataset is the best fit in comparison with the design code.

Wind speed to wind pressure

Finally, the obtained wind speeds are used to calculate the wind pressure. Formula (2. 1) is used and the results are presented in Table 3.8. The difference between the calculated wind pressure, based on the wind speeds from the KNMI data, and the wind pressure prescribed by TGB 1955 is rather large. This is well explainable with the discussion of the used wind speeds in paragraph 2.2.1, where it became clear that the TGB 1955 wind pressures are based on values that have been rounded up several times with a significant amount. The starting point was a wind speed of 29 m/s, which resulted in a wind pressure of 0.53 kN/m^2. This is more in line with the results in Table 2.8.

Table 3.8 Wind speed to wind pressure

Yearly maxima	Wind area	Wind speed [m/s]	Wind pressure [kN/m^2]	Wind pressure TGB 1955 [kN/m^2]	Difference	
					kN/m^2	%
De Kooy	Coast line	28.7	0.51	1.00	-0.49	-49
De Bilt	Inland	26.4	0.44	0.70	-0.26	-38
Vlissingen	Coast line	32.4	0.66	1.00	-0.34	-34

TGB 1972

For TGB 1972, the reference period 1946 to 1971 is used. This makes four weather stations a good fit; De Kooy, Eelde, Vlissingen and Maastricht. These are shown on the map of the wind areas for TGB 1972 in Figure 3.12. TGB 1972 gives the wind speeds that have been used to determine the wind pressure. These are presented in Table 3.9. This can be compared to data from the weather stations.

Figure 3.12 Wind area map TGB 1972 with weather stations

Table 3.9 Wind speed per wind area

Wind area	v [m/s]
Coast line	26.0
Inland	20.5

TGB 1972 does not provide any information regarding distributions. Therefore, both Gumbel and Weibull distributions are reviewed. The return period is given and is 25 years. The maximum hourly mean wind speeds are used. This results in the comparison presented in Table 3.10.

Table 3.10 Comparison of wind speed values from return plot and design code values

R=25y		Wind speed [m/s] from distribution:		Wind speed [m/s] according to TGB 1972	Differences compared to TGB 1972			
Yearly maxima	Wind area	Gumbel	Weibull		Δ_{gum} [m/s]	Δ_{gum} [%]	Δ_{wei} [m/s]	Δ_{wei} [%]
De Kooy	Coast line	27.2	37.7	26.0	1.2	5	11.7	45
Eelde	Inland	22.7	31.2	20.5	2.2	11	10.7	52
Vlissingen	Coast line	24.0	26.1	26.0	-2.0	-8	10.1	39
Maastricht[1]	Inland	18.4	29.6	20.5	-2.1	-10	9.1	44
POT								
De Kooy	Coast line	**25.2**	28.27	26.0	-0.8	-3	2.3	9
Eelde	Inland	**20.9**	22.31	20.5	0.4	2	1.8	9
Vlissingen	Coast line	**22.6**	26.33	26.0	-3.4	-13	0.3	1
Maastricht[1]	Inland	**17.1**	20.54	20.5	-3.4	-17	0.0	0

[1] Measurements starting from 1951 are used instead of 1946. See explanation at section 3.1.1.

The table shows that the values obtained through the Gumbel distribution are close to the values used in the TGB 1972. On average, the yearly maxima list provides values that are closer to the design code values that the POT values. However, they exceed the values from the design code. This makes it unlikely that this set has been used, since the values will not have been rounded down due to the safety point of view. Therefore, it can be assumed that the POT dataset with a Gumbel distribution is used. Since those values are up to 17% below the values from the TGB 1972, it can be concluded that there is some extra safety within the wind speeds.

Wind speed to wind pressure

The wind speeds that are based on the KNMI data are used to calculate the wind pressure. In Table 3.11, those wind pressures are compared to the wind pressures that are defined in TGB 1972. De Kooy and Eelde are very close to the wind pressures in those locations according to the TGB 1972. This is in line with the comparison between the wind speeds; those differences were minimal as well. The wind pressure calculated for Vlissingen is 0.21 kN/m^2 lower than the wind pressure in the TGB 1972. This is due to the relatively low wind speed for a weather station at the coast line. Maastricht also has a considerably large difference between the calculated wind pressure and the one from TGB 1972. However, for a city this far inland, it is to be expected that the wind speed and therefore wind pressure are lower than the wind pressure that is used for the entire inland region, since that covers the provinces closer to the coast as well.

Table 3.11 Wind speed to wind pressure

Yearly maxima	Wind area	Wind speed [m/s]	Wind pressure [kN/m^2]	Wind pressure TGB 1972 [kN/m^2]	Difference	
					kN/m^2	%
De Kooy	Coast line	25.2	1.00	1.02	-0.02	-2
Eelde	Inland	20.9	0.81	0.88	-0.07	-8
Vlissingen	Coast line	22.6	0.81	1.02	-0.21	-21
Maastricht	Inland	17.1	0.54	0.88	-0.34	-39

TGB 1990

The TGB 1990 and Eurocode use the same three wind areas. However, the base wind speeds differ. For TGB 1990 they are presented in Table 3.12. TGB 1990 uses the maximum hourly mean wind speed with a return period of 12.5 years and a Gumbel distribution (Van Staalduinen, 1992).

The background report of the code by Van Staalduinen (1992) provides a table with factors to calculate back to a return period of 12.5 years. So for example, if the wind speed in area I was calculated with a return period of 50 years, it can simply be divided by factor $r = 1.20$. This full table with r-factors is provided in Appendix D.4. This specific example has been checked as well, but results in values that are too much off compared to wind speeds obtained directly with a return period of 12.5 years, so those will not be used for this research.

Figure 3.13 Wind area map TGB 1990 with weather stations

Table 3.12 Wind speed TGB 1990

Wind area	v [m/s]
I	27.4
II	25.0
III	22.5

The TGB 1990 wind speed research is based on the period 1964 to 1989 and uses data from seven weather stations; De Kooy, Schiphol, Deelen, Eelde, Vlissingen, Hoek van Holland and Maastricht. These weather stations are presented on the wind area map of TGB 1990 in Figure 3.13. The outcomes of the return value plots are summarised in Table 3.13. This shows that the yearly maxima list is the best fit to the values provided by the design code. The values are 5% lower on average than the values from the design code, so it can be concluded that there is some extra safety built into the wind speeds.

Table 3.13 Comparison of wind speed values from return plot and design code values

R=12.5y Yearly maxima	Wind area	Wind speed [m/s] from distribution: Gumbel	Wind speed [m/s] according to TGB 1990	Differences compared to TGB 1990	
				m/s	%
De Kooy	I	**25.9**	27.4	-1.5	-6
Schiphol	II	**24.1**	25.0	-0.9	-4
Deelen	III	**21.5**	22.5	-1.0	-5
Eelde	III	**22.8**	22.5	0.3	1
Vlissingen	II	**27.0**	25.0	2.0	7
Hoek van Holland	II	**26.5**	25.0	1.5	6
Maastricht	III	**19.7**	22.5	-2.8	-14
POT					
De Kooy	I	23.5	27.4	-3.9	-17
Schiphol	II	21.1	25.0	-3.9	-18
Deelen	III	18.3	22.5	-4.2	-23
Eelde	III	18.3	22.5	-2.9	-15
Vlissingen	II	19.6	25.0	-1.1	-4
Hoek van Holland	II	24.0	25.0	-1.2	-5
Maastricht	III	23.8	22.5	-5.0	-29

Wind speed to wind pressure

In Table 3.14, the wind pressures calculated with the KNMI data are compared with the wind pressures from TGB 1990. Almost all of the wind speeds were lower than the wind speed prescribed for those regions in the TGB 1990, except of Hoek van Holland and Vlissingen. These weather stations are located along the coast line in wind area II. They even obtain higher wind pressures than weather station De Kooy, which is located at the coast in wind area I.

Two of the weather stations are more than -0.15 kN/m^2 off. The first one is Eelde, which is located in the province of Drenthe (wind area III). Since it is very close to the border of Groningen, which is wind area II, it could be expected that this location would have wind speeds in the upper region of wind area III and maybe even close to the one used in wind area II. Table 3.13 shows that the wind speed is indeed higher than prescribed by the TGB 1990. However, it still results in a lower wind pressure than the TGB 1990. This is due to the calculation of the friction speed, which is one of the parameters of the wind pressure equation for the TGB 1990. This calculation is added to Appendix D.4. The other weather station that is further off from the prescribed wind pressure is Maastricht. Same as for the TGB 1972, this can be explained with the location of the weather station, which is more inland than the other weather stations in this wind area.

Table 3.14 Wind speed to wind pressure for rural terrain for z=10m

Yearly maxima	Wind area	Wind speed [m/s]	Wind pressure [kN/m^2]	Wind pressure TGB 1990 [kN/m^2]	Difference	
					kN/m^2	%
De Kooy	I	25.9	0.94	1.06	-0.12	-11
Schiphol	II	24.1	0.81	0.88	-0.07	-7
Deelen	III	21.5	0.65	0.73	-0.08	-11
Eelde	III	22.8	0.58	0.73	-0.15	-20
Vlissingen	II	27.0	1.02	0.88	0.14	16
Hoek van Holland	II	26.5	0.98	0.88	0.10	12
Maastricht	III	19.7	0.54	0.73	-0.19	-26

Eurocode

The wind speeds that the Eurocode uses as a base are increased compared to the ones provided by TGB 1990. This difference is around 2.0 m/s. This can be partly explained by the used return period; Eurocode uses a period of 50 years and the TGB 1955 12.5 years, which is a significant difference. Table 3.15 presents the wind speeds used by the Eurocode.

Figure 3.14 Wind area map Eurocode with weather stations

Table 3.15 Wind speed Eurocode

Wind area	v [m/s]
I	29.5
II	27.0
III	24.5

Another difference between TGB 1990 and Eurocode is that the Eurocode uses the average wind speed over the last 10 minutes of an hour, whereas the wind speed in TGB 1990 is based on the maximum hourly mean wind speed on a day. Four weather stations are reviewed; De Kooy, Schiphol, Hoek van Holland and Deelen. Where these weather stations are located is indicated on the wind area map of the Eurocode in Figure 3.14. Even though the data is hourly data, this set is often referred to as the 10-min data set. Appendix DAppendix D.5 presents the overview of the return value plots. The reference period that is used is 25 years and is from 1986 to 2011.

The wind speeds that the Eurocode prescribes are determined using the Rijkoort-Weibull model in combination with the yearly maxima wind speeds. However, due to the complexity of the model and the lack of detailed reports, it is impossible to reconstruct the model, according to the team of the KNMI-HYDRA project. Apart from that, the team of the KNMI-HYDRA has found some critical points at which the Rijkoort-Weibull model was lacking. Therefore, they developed a new model based on a conditional Weibull model and a generalized Pareto model (Verkaik et al., 2003). This thesis aims to compare measurement data with the wind speeds used in the design codes. Therefore, the year maxima wind speeds will be used in combination with a Weibull distribution.

Table 3.16 presents the wind speeds obtained using the Weibull distribution and compares it to the wind speeds according to the values prescribed by the Eurocode. Some things stand out. For example, that the wind speed at weather station Hoek van Holland is higher than the value that is prescribed by the Eurocode for the wind area where Hoek van Holland lies in; wind area II. It should be noted that the weather station is located close to the sea, which can explain the high wind speeds. On the side of the design code, this fact is taken later into account during the calculation of the wind pressure, where the terrain influence is implemented. Interestingly, the wind speed at Hoek van Holland is even higher than at the weather station of De Kooy. De Kooy is located in wind area I, which means the wind pressure prescribed by the code is higher than for Hoek van Holland. This difference was also noted in the comparison for TGB 1990. Another explanation for the high wind speeds at Hoek van Holland is that there are flaws in the measurements.

For the other weather station, the wind speeds obtained through the Weibull distribution seem to be a good fit with the wind speeds according to the Eurocode; The values for wind speed are lower than the Eurocode and the differences are acceptable.

Table 3.16 Comparison of wind speed values from return plot and design code values

R=50 y Yearly maxima	Wind area	Wind speed [m/s] from distribution: Weibull	Wind speed according to Eurocode	Differences compared to Eurocode	
				m/s	%
De Kooy	I	26.2	29.5	-3.3	-11
Schiphol	II	21.5	27.0	-5.5	-21
Hoek van Holland	III	28.8	27.0	1.8	7
Deelen	II	20.0	24.5	-4.5	-18

Wind speed to wind pressure

The wind pressure that are calculated with the wind speeds from KNMI measurements are presented in Table 3.17 and compared with the wind pressures in the Eurocode. Since the wind speed is squared in the formula for wind pressure, the difference between the value that is calculated and the value from the Eurocode becomes larger when transforming the wind speed to wind pressure. Especially at Schiphol, this difference becomes significant.

Table 3.17 Wind speed to wind pressure

Yearly maxima	Wind area	Wind speed [m/s]	Wind pressure [kN/m^2]	Wind pressure Eurocode [kN/m^2]	Difference	
					kN/m^2	%
De Kooy	I	26.2	0.80	1.02	-0.22	-22
Schiphol	II	21.5	0.54	0.85	-0.31	-37
Hoek van Holland	III	28.8	1.50	1.32	0.18	14
Deelen	II	20.0	0.47	0.70	-0.23	-33

3.2.1.2 Pressure coefficients

As previously explained, the method to obtain the coefficients that apply on a building subjected to wind load, is to work with RWIND, which can perform CFD analyses. This section will discuss and present the results of the CFD analysis. Variant 1, with a building height of 70 m, will be used to showcase the steps of the method and type of results that are obtained in this research. The other variants will be discussed more briefly. The full overview of the results will be presented in Appendix E. Furthermore, a comparison is made between the pressure coefficients on buildings for all the different wind areas and terrain categories in order to verify whether these aspects influence the pressure coefficient.

Evaluation of variants

Variant 1

Figure 3.15 and Figure 3.16 presents results of variant 1 with a building height of 70 m. The left figure presents the wind pressure on the building and the right figure the pressure coefficient (c_p). Apart from the values, the figures have the exact same colour distribution. This is interesting, because the design codes, TGB 1955 to Eurocode, all assume a constant value for the pressure coefficient.

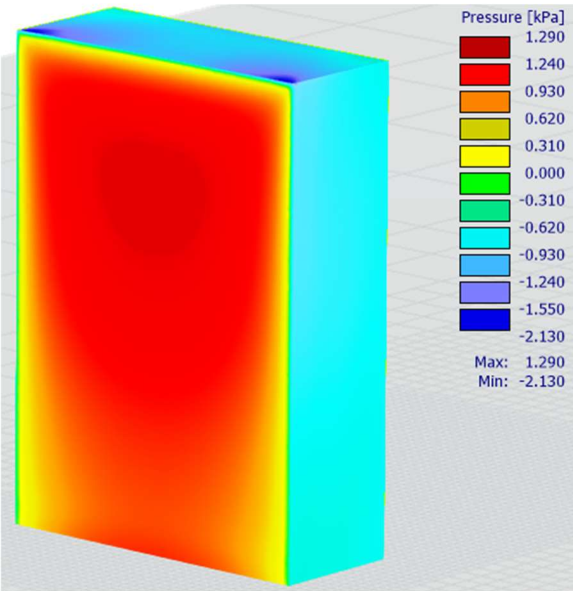


Figure 3.15 RWIND wind pressure results variant 1 - 70 m

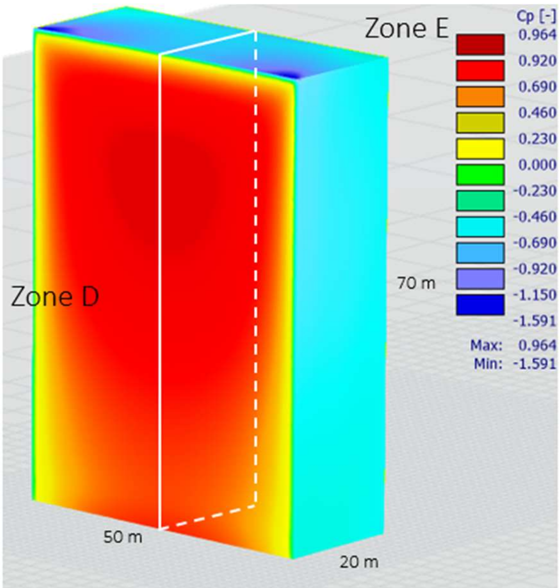


Figure 3.16 RWIND pressure coefficients results variant 1 - 70 m

From these graphs, a plot over the height can be obtained from RWIND. For this case, the pressure coefficients along a line in the middle is evaluated, as is indicated in Figure 3.16 (white line). This data is processed using a Python script and presented together with the values prescribed by the Eurocode in Figure 3.17 and Figure 3.18. This shows that the shape of the coefficient over the height of the building is indeed the same as the wind pressure. Even though it was not the goal to use RWIND for determining the wind pressure on the building, it is interesting to compare as well as it is useful to determine the goodness of fit. Because, for the pressure coefficient c_{pe}, it is known that the Eurocode value is constant and the pressure coefficient is not. Therefore it is harder to determine the goodness of fit. Besides that, the pressure coefficient is linked with the wind pressure through the equation for the pressure coefficient, which explains the similar shapes.

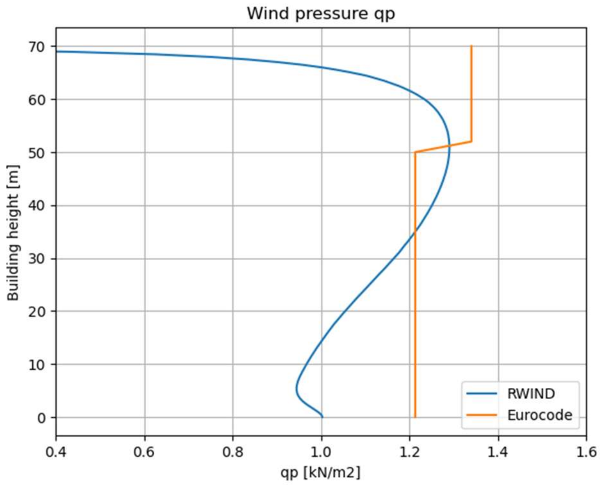


Figure 3.17 Wind pressure over the height variant 1 - 70 m

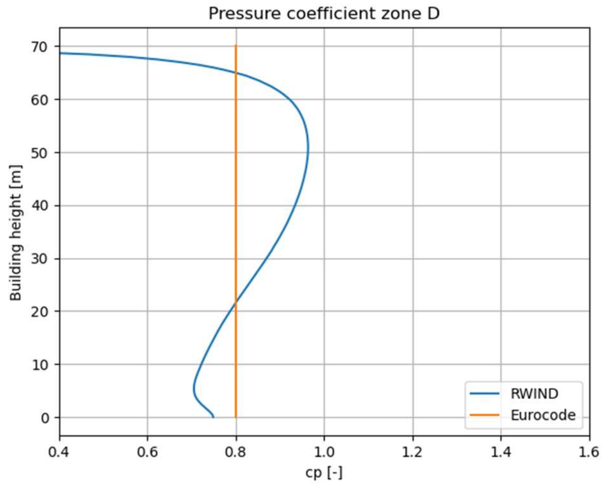


Figure 3.18 Pressure coefficient zone D for variant 1 - 70 m

In Figure 3.19, the pressure coefficient in zone E is compared to the value of the Eurocode. According to the Eurocode, this value is depending on the height/depth ratio of the building, resulting in a value of -0.63 [-] constant over the height of the building. Even though the values are off for both zone D and E, the total pressure coefficient value comes relatively close to the prescribed value by the Eurocode. As is clearly visible in Figure 3.20, the distributions over the height do absolutely not align. However, it was already previously argued that the Eurocode uses a simplification to represent the pressure coefficient.

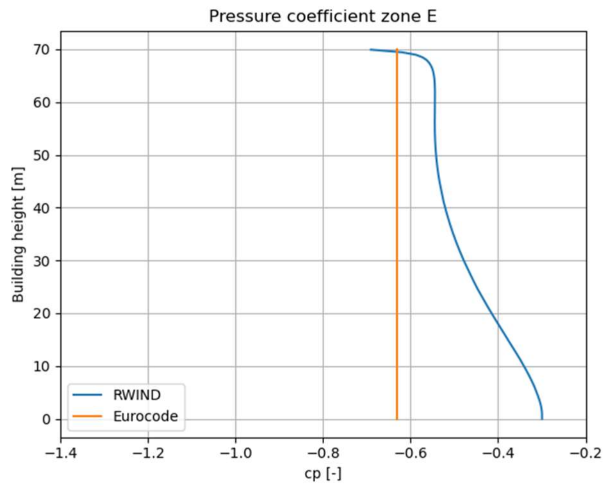


Figure 3.19 Pressure coefficient zone E for variant 1 - 70 m

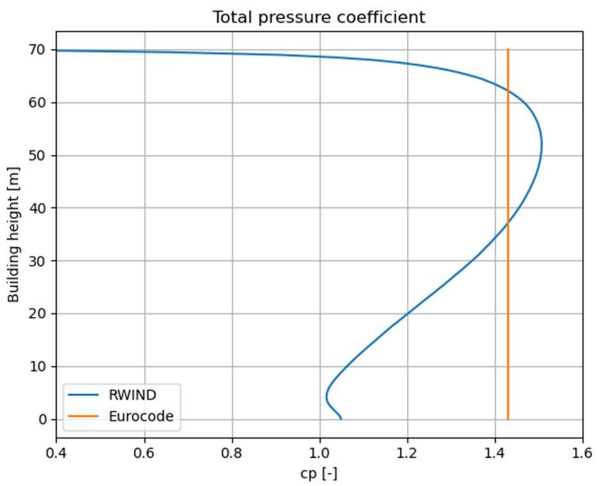


Figure 3.20 Total pressure coefficient for variant 1 - 70 m

<u>Intermezzo - Comparison with TGB 1972</u>

Another interesting comparison can be made with the wind pressure prescribed by the TGB 1972. The graph in Figure 3.21 shows that the shape of the wind pressure according to the TGB 1972 has a similar shape as the RWIND results. In the TGB 1972, the first 7.0 m are constant, which is a simplification of the shape of the actual wind pressure for that height, which reduces first before it starts to increase. The difference between the two graphs is 0.1 to 0.15 kN/m^2 until around ⅔ of the height of the building is reached. This difference can be explained by the wind speed that has been used as input. TGB 1972 uses 20.5 m/s as base wind speed and Eurocode used 27.5 m/s for this wind area. Then the RWIND output shows that the wind pressure rapidly decreases and the according to the TGB 1972 the wind pressure continues to increase. Literature does not say whether this is due to safety reasons or for a more simplistic calculation method.

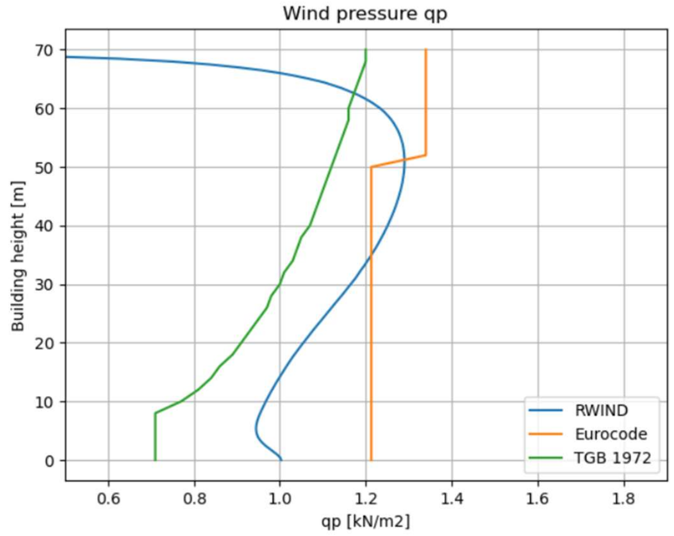


Figure 3.21 Wind pressure over height variant 1 including TGB 1972

<u>Validation</u>
To validate the outcomes of the CFD model, the comparison with wind tunnel research output is made. Two wind tunnel models are compared with the CFD output and the prescribed values from the Eurocode. In the Eurocode +0.8 [-] is standard for the windward side (zone D) and by linear interpolation -0.63 [-] is found for the leeward side (zone E) of variant 1 with a height of 70 m. All of this data is combined in Figure 3.22, Figure 3.23 and Figure 3.24. In all figures, the similarities in shape can be found, where the wind tunnel experiments are simplifications of the curved graph resulting from the RWIND model. This can be explained by the contour plots (Figure 3.7 and Figure 3.9).

For zone D (Figure 3.22), the pressure coefficients from the wind tunnel tests are lower than the ones from RWIND. The maximum pressure coefficient RWIND gives is even higher than the Eurocode value. The maximum value of wind tunnel test 2 is exactly the same as the Eurocode.

In zone E (Figure 3.23), the pressure coefficients from the wind tunnel tests are lower than the CFD output as well. However, this brings the total pressure coefficient closer together, since the absolute values are added and the suction in zone E results in a negative pressure coefficient. This can be concluded from Figure 3.24, where the maximum total values of wind tunnel test 2 and RWIND are almost the same.

In conclusion, it can be said that qualitatively, the RWIND wind tunnel simulation results in a representative model of the wind pressure coefficient. Quantitively, the values are a bit off from the measurements from the wind tunnel tests and from the prescribed values by the Eurocode. This gives reason to believe that the RWIND values are too high, since the Eurocode would not use values that are rounded down. Interestingly is that the combination of the pressure coefficients from zone D and E, which is used in practice for the global wind load calculations, results in similar values as one of the wind tunnel test. It even approaches the value of the Eurocode better; the difference is 5% for the maximum value of the total pressures coefficient. For zone D and E this differences were 20% and 10%.

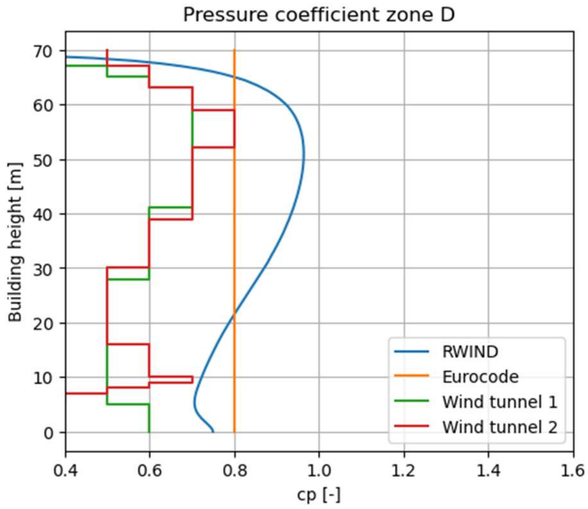


Figure 3.22 Pressure coefficient zone D

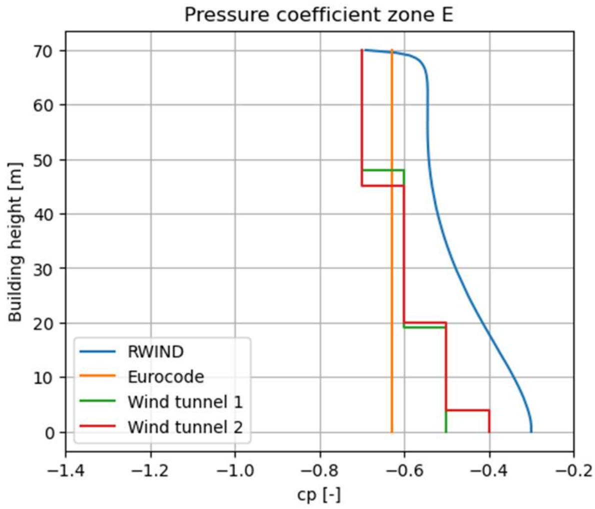


Figure 3.23 Pressure coefficient zone E

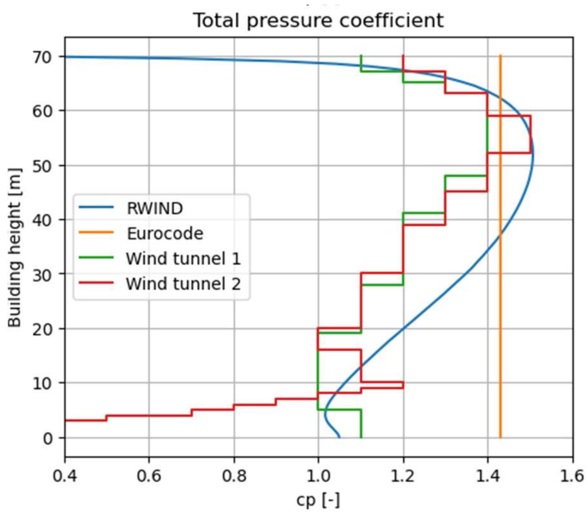


Figure 3.24 Total pressure coefficient

<u>Other heights for variant 1</u>

In Appendix E (Figure E. 3), an complete overview of variant 1 with different building heights is given. In this overview, the graphs of the different models are plotted on top of each other. By increasing the height, the graphs keep a similar shape and at first glance, the maximum and minimum values do not seem to increase significantly. A closer look is necessary to evaluate this properly. For zone D, the maximum value increases from 0.92 [-] to 0.96 [-] with increase of the building height from 30 m to 70 m. The depth of the building stays the same, so this means the height/depth ratio increases from 1.5 to 3.5. Eurocode assumes the value of the pressure coefficient in zone D is independent of this ratio. As is visible in Figure 3.25, this is not the case according to the RWIND simulation. Earlier it was pointed out that the values that are obtained through RWIND are on the high side and should not be used to make conclusions. However, these are interesting graphs that show that the increase of the pressure coefficient does not only happen on the leeward side of the building when the height depth ratio increases. Literature does not say how the Eurocode values are obtained exactly, but it could be possible that the windward side is chosen constant and the leeward more flexible in order to simplify the calculations.

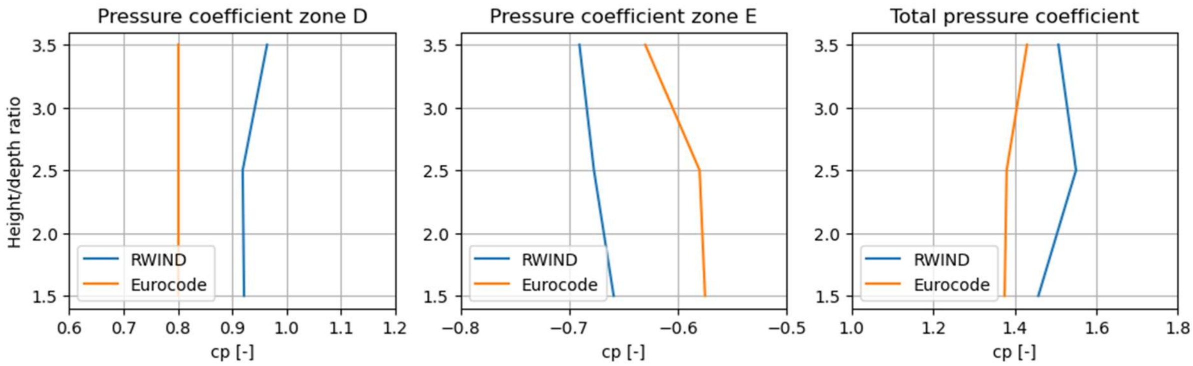


Figure 3.25 Overview of pressure coefficients against height/depth ratio

Variant 2

The plots of variant 2 for all heights are stored in Appendix E.2. The plot of the pressure coefficient in zone D shows something notable compared to this same graph for variant 1. For variant 1, the maximum values of the pressure coefficient show a small dependence on the height of the structure. However, for variant 2, the maximum value of the pressure coefficient increases significantly with increase of the building height. This is interesting, because the height/depth ratio of this variant is exactly the same as for variant 1. The only difference is that variant 2 has a smaller width than variant 1. On the other hand, in zone E, the values of the pressure coefficient are lower than zone 1. The combined pressure coefficient results in similar pressure coefficients, but these graphs imply that the width can have some influence on the pressure coefficient. That the width of the building has influence on the pressure coefficient is also shown in the comparison of the wind tunnel tests in Appendix E.3. These contour plots clearly indicate that the distribution of pressure coefficients shifts over the surface subjected to wind load for a different width.

It could be the this influence is found to be neglectable compared to the influence of the height/depth ratio and that this is the reason that is not used as a parameter to determine the pressure coefficient in the Eurocode. However, it should be noted that it definitely has some influence on the results.

Variant 3

Variant 3 allows to determine the influence of rounded corners. This effect can be taken into account when using the force coefficient, but not with the pressure coefficient. In Appendix E.4, the graphs of buildings with varying radii are plotted on top of each other. The figure shows that in zone D, the maximum values are exactly the same: 0.94 [-]. The larger the radius of the rounded corner, the faster the pressure coefficient decreases to the sides of the surface. This is in contrast to zone E, where there are differences up to 0.44 [-] over 50% of the surface. This effect of the rounded corners is endorsed by literature, which adds that the rounded corners produce a reduction of the critical angle of incidence. This makes these buildings efficient in resisting the wind load (Kasana et al., 2022).

So, according to the CFD simulation rounded corners have a beneficial effect on the total wind load on a building, because the pressure is lower on these rounded corners. In the Eurocode, this effect can already be taken into account when using the force coefficient instead of shape factors, such as the pressure coefficient. Since the amount of buildings with this specific appearance is rather limited, it is advised to use the force coefficient in those cases and to not implement it in the pressure coefficient guidelines.

Wind areas and terrain

In RWIND, the variant 1 is run for three locations and for all different terrains. The three locations are the Kaasmarkt (cheese market) in Alkmaar, the faculty of civil engineering of TU Delft and the Efteling in Kaatsheuvel. These three locations are positioned in a different wind area, which are compared in Figure 3.26 and Figure 3.27. From these graphs can be concluded that the wind area has limited influence on the pressure coefficient. There are differences, but these are minimal and especially the shapes of the graphs over the height of the building are the same. Since it has already been discussed that the output of the CFD model should mainly be evaluated qualitatively, the shape and the rough value of the coefficients is the best way to compare them.

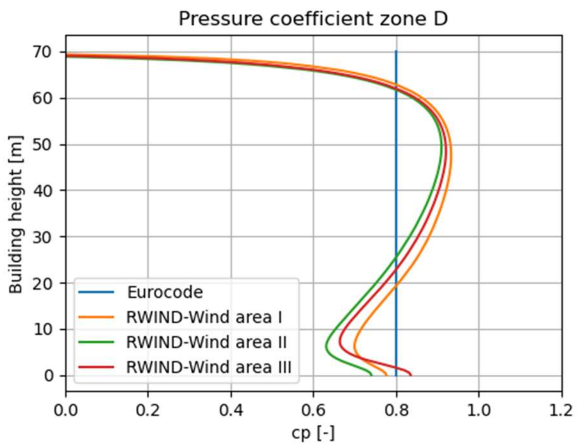


Figure 3.26 Pressure coefficient in zone D for different wind areas

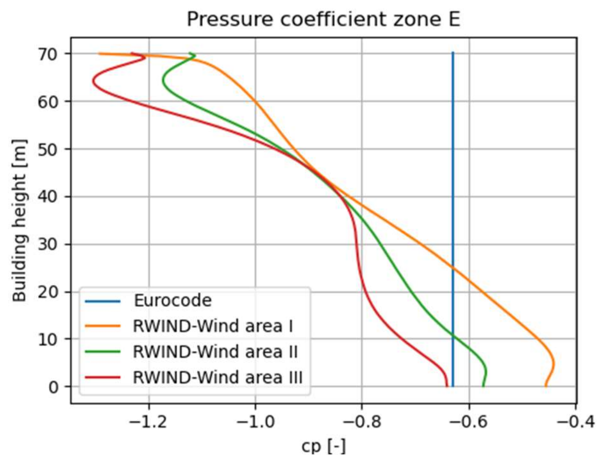


Figure 3.27 Pressure coefficient in zone E for different wind areas

The pressure coefficient for the three terrains – coast, rural and urban – have a similar shape in zone D, as can be seen in Figure 3.28. Previously in this chapter, it has been pointed out that the CFD simulation returns pressure coefficients that are too high. Therefore, the graphs should only be evaluated qualitatively. This is again proven by adding the graph of the pressure coefficients of the Eurocode to the overviews. It is clear that there are differences between the graphs of the different terrains. However, it can not be said with certainty if this is due to differences in convergence of the model or if there actually are differences due to the terrain. The pressure coefficients in zone E, Figure 3.29, show even more differences. The graphs of the coast and the urban terrain seem to be a relatively good representation of what is used in the Eurocode. However, the graph for the urban terrain exceeds the others significantly. This model has been ran several times in RWIND, but it gave the same results. Causes of this could be the convergence of the model or that the urban roughness length causes something in the simulation that results in different outcomes.

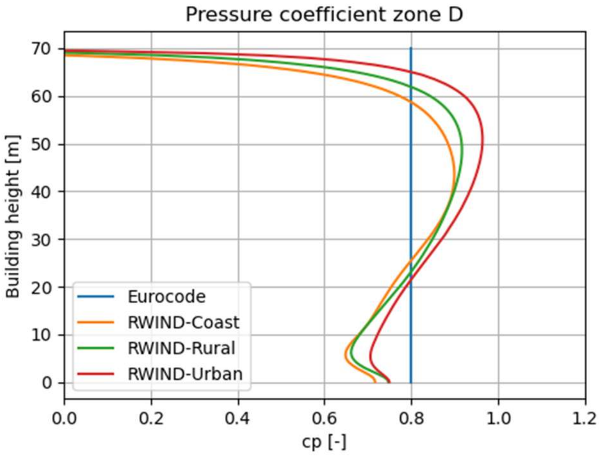


Figure 3.28 Pressure coefficient in zone D for different terrains

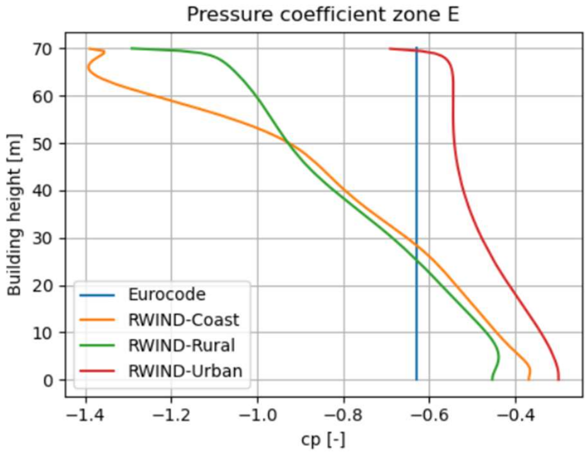


Figure 3.29 Pressure coefficient in zone E for different terrains

Overall, it can be concluded that the wind area and terrain have limited influence on the pressure coefficient. It cannot be excluded completely, but since the values given by RWIND are not completely reliable, nothing can be said with certainty.

3.2.2 Change of wind speed over the years

The comparison of the design codes in chapter 2 has shown that, overall, the wind pressures that are prescribed by the design codes have increased. However, it is unclear whether this is a result of an increase in the safety margin that is on the wind speed or that the wind has become stronger over the years. Therefore, apart from analysing specific reference periods for the design codes, the trend of the overall yearly maximum wind speed is evaluated as well. Five of the eight weather stations are suitable for reviewing their entire timeline. Due to moving the weather stations of De Bilt, Eelde and Maastricht, those datasets show a lot of fluctuations and discontinuities in the timeline. They have shown to be useful to review parts of the timeline, but not the entire length of it. However, De Kooy, Vlissingen, Schiphol, Hoek van Holland and Deelen do have suitable datasets for evaluating the measurements since they started.

The wind speeds that each of the design codes take into account, have increased. For the most inland region it increased from 20.5 m/s in the TGB 1972 to 24.5 m/s in the Eurocode. The wind area most closely to the coast increased from 26.0 m/s to 29.5 m/s[1]. This increase can have two explanations; The first one is that the measured wind speed at the weather station has increased and that the design codes simply follow this trend. The second explanation that with the development of the design codes, more safety is implemented.

By evaluating section 3.2.1.1, it can be concluded that the difference between the measured wind speed and the applied wind speed in the design codes has increased since the TGB 1972. This shows that the safety over the wind speed has increased, but it does not yet prove that the maximum wind speeds did not increase as well. To answer that question, a complete timeline of the maximum wind speed is given in Figure 3.30. The trend line, in the form of a second order polynomial, is plotted as well in these graphs. Overall, the trendline shows a decrease in yearly maximum wind speed. For example, at weather station Schiphol the measurements

[1] The TGB 1955 is not taken into account for this comparison, since the wind speed that this code uses as base is not actually measured at the locations.

started in 1951 and since that time, the maximum yearly wind speed has decreased from 22.5 m/s to 18.5 m/s. This is an decrease of 18%, which is quite significant. There is however one weather station, Hoek van Holland, where the maximum wind speed rises after a period of lower maximum wind speeds. Since this specific weather station is located closely to the sea, the development of the wind speed on the sea could have influenced the maximum wind speeds at this weather station. An explanation for the decrease of wind speed on land could be the increase of the building density.

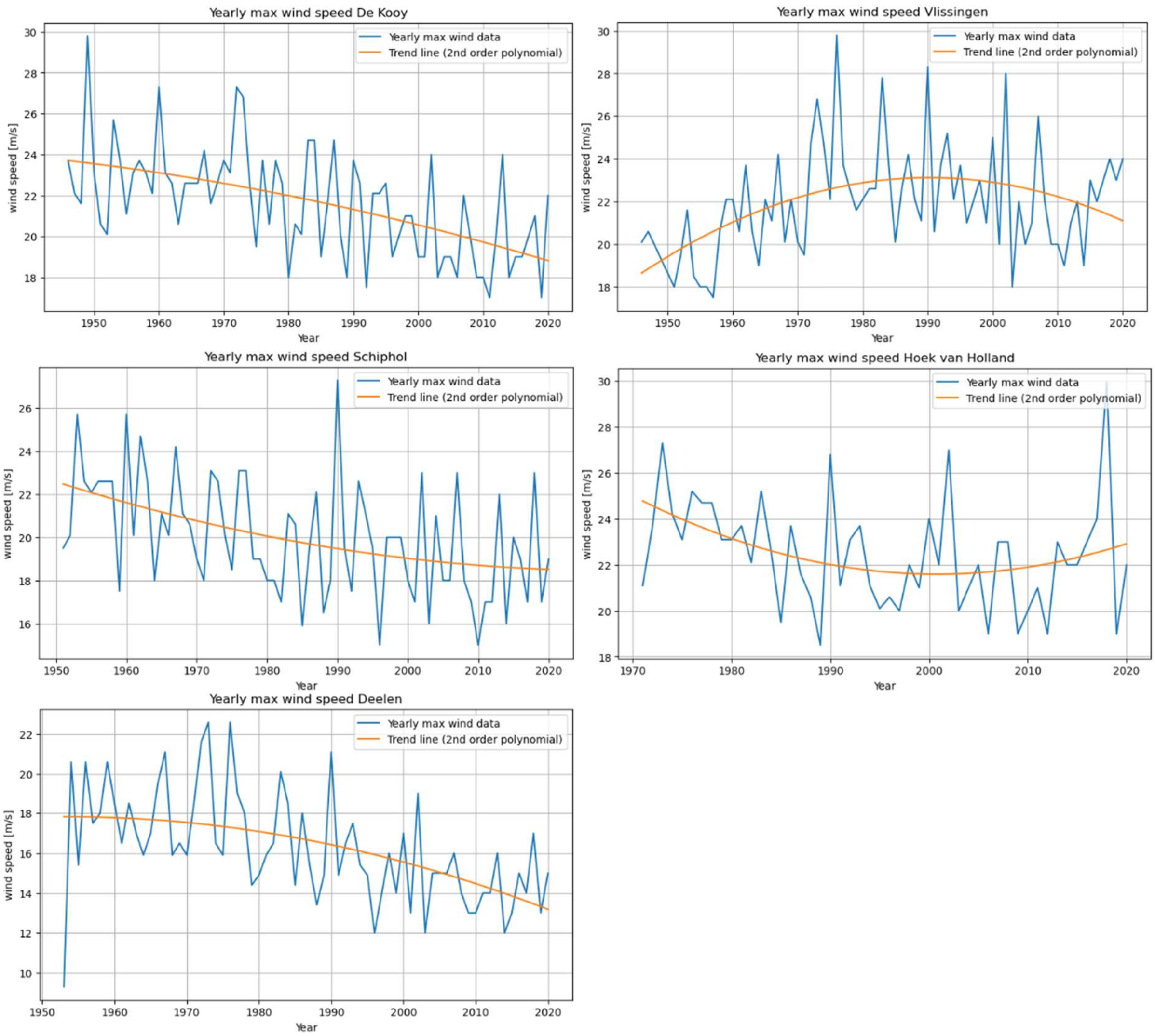


Figure 3.30 Yearly maximum wind speed at several weather stations

The KNMI has published a report on climate scenarios for the Netherlands in 2023. The report presents four scenarios for which several aspects, such as sea level, rain, temperature and wind, are predicted for the years 2050 an 2100. In this report, it is stated that the change in wind speed and its direction is limited from now (2023) to 2050 and 2100. It could be possible that during storms the wind gusts gain strength. However, these predictions are not certain. The KNMI confirms that the wind speeds above land decrease. Possibly, this is caused by the increasing building density on the land. On the other hand, the weather stations are located in rural areas and show the same trend. The KNMI states that the wind speeds above the North Sea do not decrease. This is important to note, because the heaviest storms originate above the sea. Storms from the north-west are the most dangerous for the

Netherlands, since these are the heaviest storms which can also cause a storm surge. The KNMI states that the chance on a storm from the north-west decreases slightly (KNMI, 2023b).

Furthermore, it should be noted that apart from the development of the wind speed itself, the measurement equipment has developed as well. For the weather station where the equipment has been renewed, the KNMI homogenized the measured data at that period to account for discontinuities in the measurements due to the abrupt changes (KNMI, n.d.). However, if the improvement in equipment is significant enough, it could still have changed the trendline. Nevertheless, with the available data, it is impossible to give an exact number to this influence for the Netherlands in general.

By combining the observations of the measured data and the evaluation of the KNMI itself, it can be concluded that overall the wind speeds in the Netherlands decrease.

3.3 Conclusion

This chapter discusses the building or location specific wind load, which is divided into two parts; the wind pressure q_p and the pressure coefficient c_p. This specific wind pressures and pressure coefficients are compared to the parameters that the design codes prescribe to give insight in how representative these are.

Wind pressure

For the evaluation of the wind pressure, KNMI measurements are used. Most of the wind pressures that are prescribed by the design code are shown to be based on measured wind speed data in this chapter. Some of the evaluated design codes, such as the TGB 1990 and the Eurocode, prescribe what kind of dataset and distribution have been used. The older codes do not, and for them, the best fit is determined by comparing the outcomes of multiple combinations to the wind speeds or pressure that are given in the design code.

For TGB 1955, this was impossible, because the value that was used as a base by the design code committee of that time is 29.0 m/s at De Bilt, which has never been measured there and is not obtained by evaluating the yearly maxima data. Next, to add extra safety, they rounded the obtained wind pressure up several times. This method did not align with the chosen method in this research and resulted in massive differences in outcome. Therefore, a proper comparison could not be performed. On the other hand, it can be said that a significant amount of extra safety is incorporated into the wind pressure by rounding it up this much.

For the other design codes comparisons, the outcomes are better explainable. Most weather stations provide wind speeds that are just below the prescribed wind speed in the design codes for that specific wind area. However, two weather stations stands out: Hoek van Holland and Vlissingen. These weather station are located in the province of Zuid-Holland and Zeeland, which fall under wind area II. However, the processed measurements result in wind speeds that are higher than the prescribed wind speed for wind area II and it even results in higher wind speeds than at weather station De Kooy, which is in wind area I. Based on these results, it could be argued that part of the coast line of the Netherlands should be wind area I, since the wind speeds obtained from the measurements along the coast are significantly higher than for wind area II. This would result in a wind area map that would be a combination between the map from the TGB 1990/Eurocode and TGB 1972. Figure 3.31 presents the proposed wind map.

Further research is needed to verify whether the entire coast line should become wind area I, as is proposed in Figure 3.31. This is out of the scope of this thesis, but all weather stations along the coast of the Netherlands should be analysed in order to provide a better substantiated proposal. Another possible option is to redefine the selection terrain categories that the Netherlands use and use a smaller roughness length for the coast. Other countries, such as Belgium for example allow the use of all five prescribed terrain categories and the Netherlands reduced this to three (section 2.2.3). However, this would not solve the difference in wind speeds that are measured.

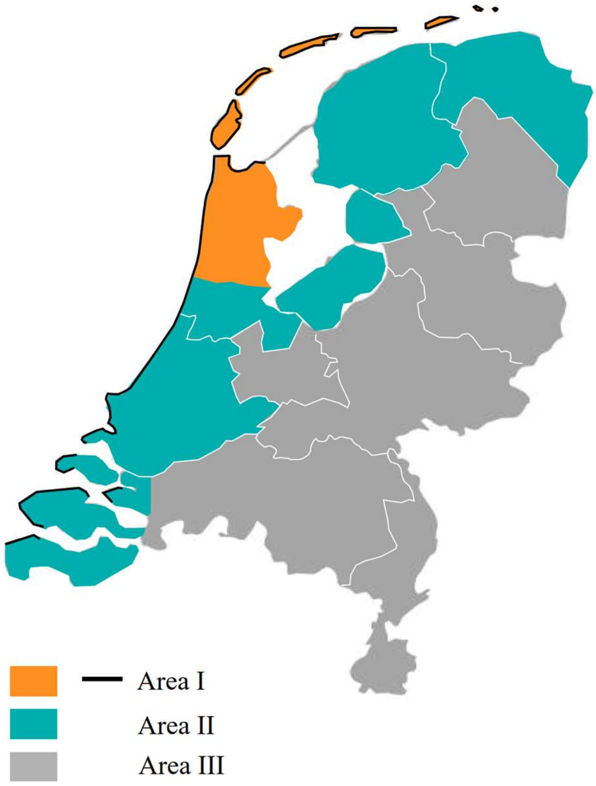


Figure 3.31 Proposed new wind area map of the Netherlands

Pressure coefficients

This research on the pressure coefficients has focused on zone D and E. Zone D is the windward side and zone E is the leeward side of the building. In the Eurocode a fixed value of +0.8 [-] is prescribed on the windward side (zone D) of all buildings, which means that is independent of the dimensions of the building. On the leeward side (zone E), the pressure coefficient is given in a range from -0.5 [-] to -0.7 [-] and this is determined by the height/depth ratio of the building. By evaluating CFD models in RWIND and wind tunnel experiments it is shown that pressure coefficients are not constantly distributed over the surface of a building. The outcomes of the CFD model result in higher values than the Eurocode, which is the reason that it is concluded that the CFD outcomes should be used with caution and preferably not quantitatively. Nonetheless, the shape of the CFD pressure coefficients over the height of the building are very similar to wind tunnel experiments, which makes the qualitative outcome useful.

The maximum wind pressure at the windward side occurs at about ⅔ of the height of the building. In the Eurocode only zone E is depending on the height/depth ratio of the building.

However, this research would argue that both zone D and zone E are depending on the height/depth ratio as well as the ratio to the width of the building. Both the CFD model and the wind tunnel experiments show this effect. It could be that the reasoning behind the fixed value for zone D in the Eurocode could be that this aims for some simplicity in the calculation, but no background information could be found that confirms this idea. In the older design codes, both zones are assumed to be constant.

It could be argued that a more detailed model should be used for the pressure coefficient, which account for all the dimensions of the building and would therefore make it more realistic. However, in practice, it is very inefficient to work with a detailed model for a coefficient. It is understandable that these coefficients are simplified to constant values over the entire height of a building. This saves a lot of time. The question is how much one would gain by applying a more complex calculation. For low rise buildings, this benefit is rather limited, but when increasing the building height, and therefore the forces caused by wind loads increase, it could be interesting to apply a coefficient that is depending on the height. This could be a similar model as for the wind pressure over the height of a building.

On the other hand, by assuming a constant over the full height of the building, which is the maximum value that appears on the entire surface, extra safety is incorporated into the design. This is very important as well. The proposal of a more detailed pressure coefficient is not proposed to seek the limits of safety, but it can give insight into a sharper calculation that could be possible. Especially when adding extra levels, the engineer will seek for overcapacity in the existing structure and this could be interesting.

Overall, the research on the pressure coefficient has interesting outcomes, but no absolute conclusion can be drawn. It should be noted that the height/width ratio is taken into account in the force coefficient. This coefficient is often applied for high and slender buildings, but not standard for low and mid rise buildings.

Change of wind speed over time

Finally, the change of wind speed over time is evaluated to enhance understanding of the increase of the wind pressure over the years. The comparison of the design codes in chapter 2 showed this increase. This increase may result from either an upward adjustment in the safety margins for wind speed or the wind has become stronger over the years. Five weather stations were selected and reviewed for their entire operational life. The yearly maximum winds are plotted in combination with a trend line. This second order polynomial trendline provides insight into the development of the maximum wind speed over the years. It showed that in general, the yearly maximum wind speed decreases slightly. This is confirmed by a climate report of the KNMI. However, the degree of this decline highly depends on the location of the weather station, which has as result that a uniform reduction rate cannot be determined for the Netherlands. A potential contributing factor for this trend could be the increasing building density in the Netherlands

This improved understanding of the building and location specific wind load can help when designing extra levels on top of an existing building as it increases sense and knowledge of the additional wind loads a building is subjected to when adding levels.

4 BEARING STRUCTURE

Adding layers to an existing building influences the wind load in a twofold manner: extra surface subjected to wind load and a difference in design codes for wind load between now and the past. When assessing an existing building for the possibility of adding levels, the status of the building itself needs to be assessed as well. There are roughly two things that need to be assessed; The first one is the load bearing system, which can be divided into the stability system (horizontal load bearing system), the vertical load bearing system and the foundation. The second one is the status of the building material. The load bearing systems will be discussed in section 4.1, 4.2 and 4.3. The development of the building materials over the years strictly falls outside of the scope of this thesis. However, since it has the potential to be of great influence on the total assessment of the existing building, it will be discussed shortly.

4.1 Influence of change in wind load design on the stability system

Before discussing the influence of change in wind load design on the stability system, first, in Figure 4.1 an overview is given of the main stability systems. In the figure, the stability systems are also related to a certain building height. The scope of this thesis limits to a building height of 70m, which means that the low-rise and mid-rise buildings are considered. Stability systems that are often applied in such buildings are shear walls, portal frames, concrete cores and braced frames (Oval, 2024). Besides the building height, the time of building is important as well. Only two buildings over 100 m have been built before the publication of the TGB 1972 and besides that, it is uncommon to add extra levels to a building of over 100 (Council on Tall Buildings and Urban Habitat, 2024). This is part of the reasoning behind limiting the scope to 70m building height. In this section, the influence of the extra levels as well as the change in wind load on the (existing) stability system of the building will be discussed.

Furthermore, depending on the dimensions of the building, it is possible that dilatations have been applied. This separates stability systems and therefore all parts should be checked separately as well.

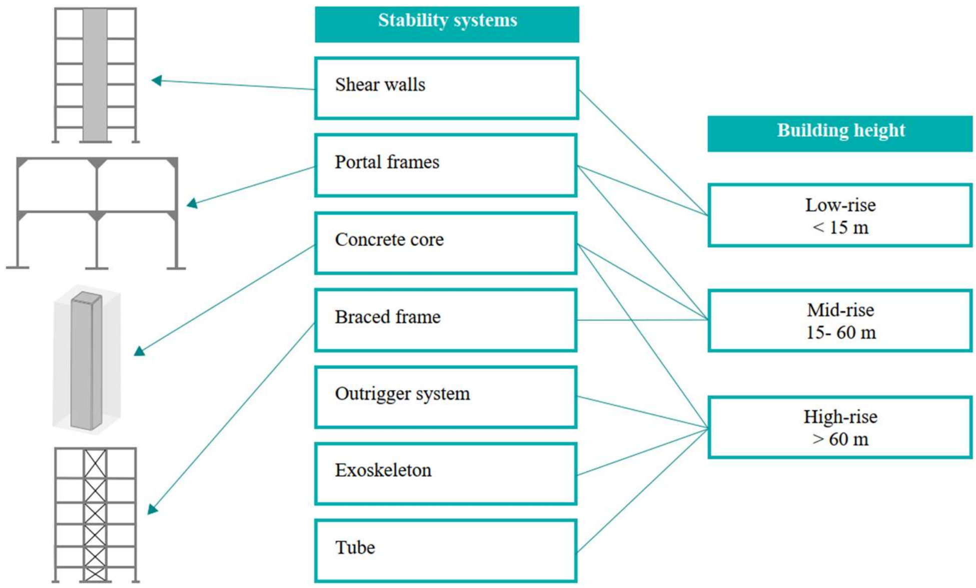


Figure 4.1 Stability systems related to building height

Influence of extra levels on existing stability system
The most evident consequence of adding levels on top of an existing building, is the increase in height which results in an increase of surface subjected to wind load. Therefore, the total wind load on the building increases. For shear walls, portal frames and concrete cores, this increased total wind load generates a higher bending moment as well. In the case of braced frames, this results in an increase of the normal forces in the braces. Checks need to be performed in order to verify that the existing structure can withstand the increased loads, which means that the overcapacity needs to be determined. For steel structures this also includes the connections, since these can become governing when the forces is these elements increase. If necessary, possibilities regarding strengthening the existing stability system should be explored.

Besides the increase of surface subjected to wind load, the increase of height has another consequence as well; a higher maximum wind pressure. The maximum wind pressure that is applied on the building, is depending on the maximum height of the building, so when the latter increases, the maximum wind pressure increases as well.

Furthermore, the extra levels itself need to be stable as well. Often steel frames are applied to provide lateral stability. Partly depending on the wishes of the architect, this can be either a braced frame or a portal frame. It is also possible to execute the stability system in timber. This type of structure has gained popularity due to an increased demand for more sustainable constructions. Timber is known for being a renewable material, which production releases less CO_2 into the air than the traditional concrete and steel production. Although, it is important to mention that it depends on the sustainable forestry practices and end-of-life scenarios of the timber as well whether one can really say that timber is sustainable (Abed et al., 2022).

All of the four considered stability systems are suitable for connecting a new stability system on top. The existing structure will possibly need alterations to provide a good base to which the new structure can be connected, but steel-steel and concrete-steel connections are very common and will therefore not be the largest challenge. However, in case of a portal frame, where the connections need to fixed and have to transfer bending moments, extra care is required for the execution.

Influence of change in wind load due to development of the codes
Chapter 2 showed that for most of the cases, the wind pressure that has to be applied according to the design codes has increased over the years. Especially for buildings with a height of over 40 m, the wind pressures prescribed by the design codes increases rapidly. So apart from the increased total wind load due to increase of height, the total wind load also increases due to increased prescribed wind pressure over time. This only has impact on the already existing part of the building, since the extra level will have to be calculated using the newest design code anyway. Not for every case the existing building has to be recalculated using the newest design code, so this does not always have influence on the design.

Besides the increase in wind pressure, the partial safety factors cause an increase in the wind load on the building as well. TGB 1955 and TGB 1972 did not have partial safety factors that needed to be applied on the loads. At that time, safety factors only needed to be applied on the material side. Table 4.1 shows the material factors (γ_m) for all design codes. With the publication of TGB 1990, the value of the material factors for steel and concrete have dropped significantly. The partial safety factors than have been introduced by TGB 1990 are presented in Table 4.2. The same values are used in the Eurocode. By combining the material factor and partial safety factor, in some cases, the outcome is similar to the material factor which was specified in the TGB 1955 or TGB 1972. For example, a concrete building in consequence class 3 (CC3) for TGB 1990:

$$\gamma_{total} = \gamma_m * \gamma_{f,q} = 1.2 * 1.5 = 1.8$$

This is the same as the material factor that was prescribed for concrete by the TGB 1955. This also holds for same other cases. In Appendix F.1 a complete overview is given of the combinations of material factors and partial safety factors, including the comparison with the material factors from TGB 1955 and TGB 1972. The reasoning behind this comparison is that is important to note that, although the partial safety factors increase the wind load on the building, in the end, it does not necessarily result in increased safety compared with the older design codes, when purely looking at safety factors. This partial safety factors are not within the scope of this research. However, for the total reflection of the topic, it is essential to be aware of these developments.

Table 4.1 Material factors

Material factor γ_m	Material		
Design code	**Steel**	**Concrete**	**Timber**
TGB 1955[1]	1.5	1.8	-
TGB 1972[2]	1.5	1.7	-
TGB 1990[3]	1.0	1.2	1.2
Eurocode[4]	1.0	1.5	1.2-1.3

[1] (BOI, 2023)
[2] (Verberkt, 2020)
[3] (Nederlands Normalisatie Instituut, 1991a), (Nederlands Normalisatie Instituut, 1991b), (Nederlands Normalisatie Instituut, 1991c)
[4] (Nederlands Normalisatie Instituut, 2011d), (Nederlands Normalisatie Instituut, 2011c), , (Nederlands Normalisatie Instituut, 2011e)

Table 4.2 Partial safety factor of TGB 1990 and Eurocode (Nederlands Normalisatie Instituut, 1990) & (Koninklijk Nederlands Normalisatie Instituut, 2019)

	CC1	CC2	CC3
$\gamma_{f,g}$	1.2	1.2	1.2
$\gamma_{f,q}$	1.2	1.3	1.5

4.2 Influence of change in wind load design on the vertical load bearing system

The vertical load bearing system is often part of the stability system as well. Therefore, the structure needs to be able to withstand horizontal forces and bending moments caused by the wind load, as well as vertical loads, which are permanent and live loads. For example concrete cores are often used as stability system, but they provide a vertical load bearing function for loads of each level simultaneously. In case both horizontal and vertical (axial) forces work on the same element, the 2nd order bending moment should be taken into account as well.

Influence of extra levels on existing vertical load bearing system
By adding levels to the existing building, vertical load is added. Depending on the determined overcapacity of the vertical load bearing structure, the maximum number of possible extra levels can be determined. It is advisable to use lightweight structures nonetheless, such as steel or timber frame constructions, to minimize the extra permanent loads. Although light weight structures are preferable, the requirements for sound and insulation should be met. By adding levels, permanent load as well as live loads are added to the total vertical loads.

Apart from the additional levels on top, the vertical loads often change for the levels of the existing building as well. This is due to either change of function, which requires extra live loads to be allowed or due to development of the design codes, which prescribes a higher live load for the same function. In order to fulfil these new requirements, it is possible that the floors or floor beams needs to be strengthened, which increases the permanent loads on the existing structure as well. On the other hand, the Eurocode allows to use psi factors on live loads for some levels, which reduces the loads again. There are cases where the existing structure does not need to be checked with the newest design codes. The guidelines for designing with existing structures are given in design code NEN 8700 series, which is shortly discussed in section 4.4.

Another opportunity to be seized is to reduce the permanent loads on the existing building in order to create capacity in the foundation for more loads for either the new level or for strengthening the existing structure. There are many examples from practice, where the existing masonry façade is replaced by a lightweight façade. Besides the loads that get reduced, this can be beneficial for the architect as well, since a façade can give a completely new look to the building (Wienerberger B.V., 2017). Another method to reduce the permanent loads is to remove the screed floor and replace it by a lightweight flooring system.

Influence of change in wind load due to development of the codes
The change of the wind load does not need to influence the vertical load bearing system directly. However, it does influence the bending moments in systems that have a double function, such as concrete cores. The concrete core does need to be able to withstand this.

4.3 Influence of change in wind load design on the foundation
The foundation is responsible for transferring the forces from both the stability system and the vertical load bearing system. Depending on its configuration, the stability system needs to transfer a bending moment (concrete core or shear walls) or a set of tension and compression forces (portal frame or braced frame), which is essentially the same. Figure 4.2 and Figure 4.3 provide an overview of the resulting forces at the foundation per stability system.

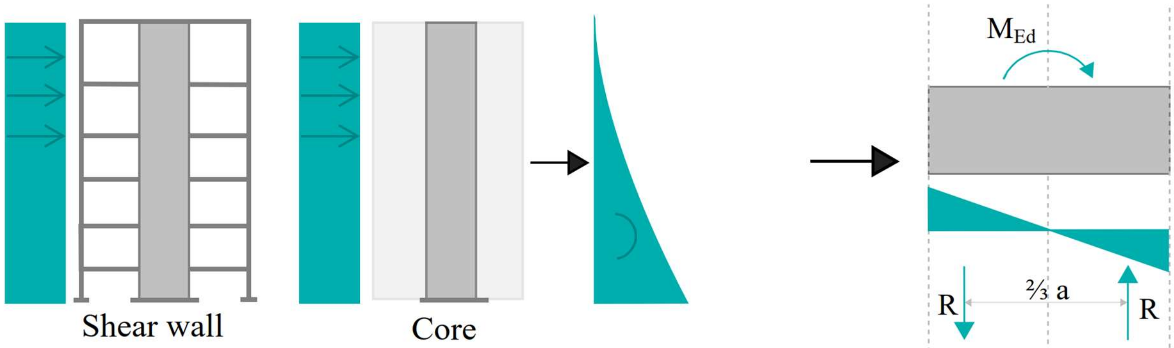


Figure 4.2 Principle of resulting force for shear wall or core

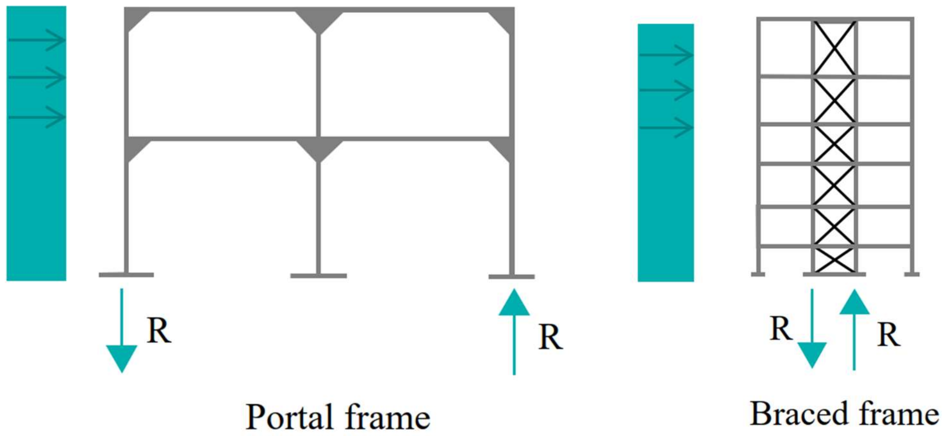


Figure 4.3 Principles of resulting forces for portal or braced frame

All stability systems transfer tensile and compression forces to the foundation. In an ideal situation the vertical loads are higher than these tensile forces, which causes the total resulting forces to be compressive. This is preferable, since most of the foundation piles work best in compression. In case of tension forces acting on the piles, concrete can be removed from the pile to investigate the rebar in the piles. For the foundations, the overcapacity needs to be determined as well. This is highly decisive for the design, since this is often governing for the amount and type of extra levels that can be placed on top of the existing building. Depending on the building and location, extra foundation piles can be placed in other to increase the bearing capacity. In practice, this solution is often applied. However, this is a complex project and has a lot of effect of the costs of the transformation.

There are several checks which need to be performed in order to determine the (over)capacity of the foundation/foundation piles. One of them is research into the available documents from the archive. Almost every municipality has a city archive where documents with calculations and drawings of the buildings are stored. This is one of the risks as well; it is possible that the piling layout plan is missing or incomplete. In this case, on-site foundation research needs to be performed in order to determine the type of piles. However, the pile depth may remain unknown. These cases need to be treated with caution. Adding foundation piles that will be the bearing structure for the newly added levels may be a solution for those cases. This way, the existing foundation will not be subjected to increase loads and it has already proven that it is sufficient for the original situation. Furthermore, a new set of cone penetration tests could be performed in order to determine the pile capacity at this specific location. Another check that gives insight into the state of the foundation is research on the settlements of the building. This can be measured by measuring the inclination of the floor levels.

4.4 Supplementary influences on the bearing structure

So far, the effects on increasing the height and the development of the designs codes on the existing structure has been discussed. However, other parameters influence the performance of the existing structure as well. The most important one is the material strength. Furthermore, for many transformations of buildings, the NEN 8700 series is applied. This is a series of design codes, specifically for renovations and assessment of existing buildings. Both of these topics are outside the scope of this thesis. However, in the next sections, a short elucidation is given in order to give a proper overview of the considerations for the case where extra levels are added to an existing building, because it is important to be aware of these elements.

Material strength
In design calculations, engineers use the 28-day strength of concrete. This is the guaranteed strength of concrete, but the concrete is not necessarily done curing after this period. Lots of research is performed on the assessment of concrete compressive strength years after it was cast. Theoretically, if concrete was maintained in a completely moist environment, it would increase in strength endlessly (Camp, 2022). In practice, even though the concrete is not continuously in a moist environment, the tests show that there is increase in compressive strength after the first 28 days. Apart from the environmental conditions, such as the moist environment, the long term compressive strength development of concrete is depending on the water-cement ratio, the curing conditions and the temperature (Mishra, 2020). Overall, the literature research says that the concrete will continue gaining strength after the 28-day mark. Depending on the period in time the concrete was cast, this can be up to 70%. Nowadays, the concrete mixtures rapidly increase strength during the first 28 days, but after that, this increase is significantly lower. Older concrete mixtures show a more linear increase in strength compared to these newer mixtures (Elfgren et al., 2006). Therefore, it is important in which time period the building is built. In many cases extra safety is hidden in this increased strength. Tests, destructive or non-destructive, need to clarify if this is the case. To obtain the compressive strength of concrete, it is common to perform a destructive test, where a core specimen is drilled and subjected to a compression test. Other test will define the remainder properties of the concrete specimen.

Another material that is applied in almost every building, is steel. In contrast to concrete, steel is not known for its ability to gain more strength during its lifetime. During production it reaches its aimed strength and it does not increase any further. The prescribed steel type, for example S235, means that the yield strength should have a minimum strength of 235 MPa, but in practice it could be more. However, the strength of steel that is in use in an existing building can only be determined by destructive tests (DT) and therefore the options to assign greater strength to a steel part in an existing building are rather limited.

Besides the possibilities regarding increased strength over time, it is of great importance as well to mention that the quality of materials in the existing building needs to be check thoroughly. First of all, through a visual inspection and if necessary, with other non-destructive tests (NDT) or DT's. This is especially the case for elements that will be exposed to increased loads due to the adding of extra levels. Steel elements, for example, need to be checked for corrosion, fractures and deformation. Reduction of cross section area due to corrosion is a common phenomena for steel structures that are located outside and can be a reason that the element does not suffice for reuse. For concrete, cracks and spalling of concrete are often encountered in older buildings and there is also the risk on corrosion of the rebar (Hendriks, 2023). These are some examples, but many more failures could occur in different building materials used in existing buildings. Another reason to check the elements on site, is that in some cases, the drawings with specifications on materials and dimensions is missing from the archive. This could also be a reason to perform NDT's or DT's.

NEN 8700
Simultaneously with the Eurocode, the NEN 8700 series was published in 2012. This is a design code made specifically for assessing existing buildings and performing renovations. It specifies the minimal safety level that an existing building must meet. For a renovation, such as adding extra levels to an existing building, the NEN 8700 could be applied as well. All the

new and renovated parts of the building must be designed according to the Eurocode. However, for the 'untouched' parts it is allowed to use the NEN 8700, which uses reduced partial safety factors. These reduced partial safety factors are up to 10% lower than the partial safety factors which are applicable to new constructions. This is partly due to the fact that a lower reference period may be applied; where new constructions are built for a lifetime expectancy of 50 years, is the lifetime expectancy for renovated buildings often around 30 years. Officially, the lifetime expectancy that should be applied is the maximum of:

- 15 years
- The leftover lifetime expectancy of the existing building, which is the original lifetime expectancy minus the number of years that the building is standing.

But in spite of that, the design code also states that for many cases 30 years should be the lower bound. However, a lot of project developers want the renovated buildings to completely comply with the Eurocode. This is often more expensive, but makes the building more future proof as well, since is allows for more loads on the floors for example, so it is possible to assign other functions to the building in the future (Lagendijk, 2014).

In section 4.3, the influence on the foundation is discussed. In the NEN 8700 series, there is a special geotechnical design code that discusses the allowed characteristic increase of loads on the foundation piles compared to the weight is has proven to withstand; NEN 8707. This design code was originally published in 2018. The maximum allowed increase according to NEN 8707 is 15% and this may not be exceeded when performing transformations of existing buildings (Koninklijk Nederlands Normalisatie Instituut, 2023).

4.5 Conclusion

This chapter discusses the influence of the combination of adding levels to an existing building and the increased wind loads on the existing building. Both the systems that provide the stability and the vertical load bearing have been reviewed.

Stability system within the scope of this thesis is shear walls, portal frames, concrete cores and braced frames. By adding levels on top of the existing building, the height of the building increases. This extra surface catches more wind, so the total horizontal load and the bending moment at the foundation increases. Besides that, the maximum wind pressure increases, because that is based maximum height of the building. Therefore, the stability system in the existing building needs to be able to take higher loads. Furthermore, the extra levels need to be stable as well. They need a new stability system, which ideally is connected to the existing stability system.

The development of the wind load in the design codes have influence as well. For almost every case, the wind pressure has increased, which means that the stability system needs to be able to take higher loads. Not necessarily all buildings will need to be rechecked with the newest design codes. However, for this thesis, the guidelines regarding renovations or transformations (NEN 8700) are outside of the scope. Furthermore, TGB 1955 and TGB 1972 only used characteristic wind loads, whereas the newer design codes use partial safety factors. This results in a larger gap between the loads that were used in the calculations with TGB 1955 and TGB 1972. For these two design codes, a higher material factor is applied, which provides extra safety on that side of the equation.

The vertical load bearing system is often the same system that provides the stability of the building. Adding levels increases the total vertical load as well with permanent and live loads. Buildings that are eligible for extra levels, are often subjected to a total transformation and they get a new function. With this new function, the loads on the floors most likely increase. The change of the wind load does not need to influence the vertical load bearing system directly. However, it does influence the bending moments in systems that have a double function, such as concrete cores. The concrete core does need to be able to withstand this.

Besides that, the bending moments could cause tensile forces in the foundation piles. Preferably, this effect is prevented. Furthermore, the foundation should be checked to verify whether the foundation can take the extra loads on the building. To minimize the extra vertical loads, lightweight structures should be applied for the new levels.

PART II
CASE STUDY

5 CASE STUDY | SCYE010

The lessons learned from the research in the previous chapters can now be applied to a case study: SCYE010. The goal is to create an optimized design for the top level. To do so, the case study will start with a general explanation of the building in section 5.1. This is followed by a description of the load bearing system of the existing building in section 5.2. In section 5.3, the wind load design is set up. This includes the design of the optimized top level, which applies all the research findings from this thesis. This chapter is then concluded in section 5.4.

5.1 General explanation of building

SCYE010 is a former office, which is transformed into a residential building with over 200 apartments. Originally, the office is from 1978, which means that the TGB 1972 was applicable. It was built for DCMR, an environmental service company from the Dutch government for the Rijnmond region. After the DCMR left, the building was empty for years, but in 2021 the new apartment building was delivered (Van Wijnen, 2023). Figure 5.1 and Figure 5.2 show the old and the new situation of the building. In the first stage of the design, the plan was to build additional levels on top to create extra apartments. However, this design was not executed, due to fact that the new layers would block the view on a monumental wind mill across the street.

Figure 5.1 SCYE010 old situation (Van Wijnen, 2023)

Figure 5.2 SCYE new situation (Van Nieuwkoop, 2021)

The building is divided into four parts, but the main focus will be on part B and C. This is the highest part of the building and the part which was considered for the transformation by the developer. Figure 5.3 highlights the different parts of the building.

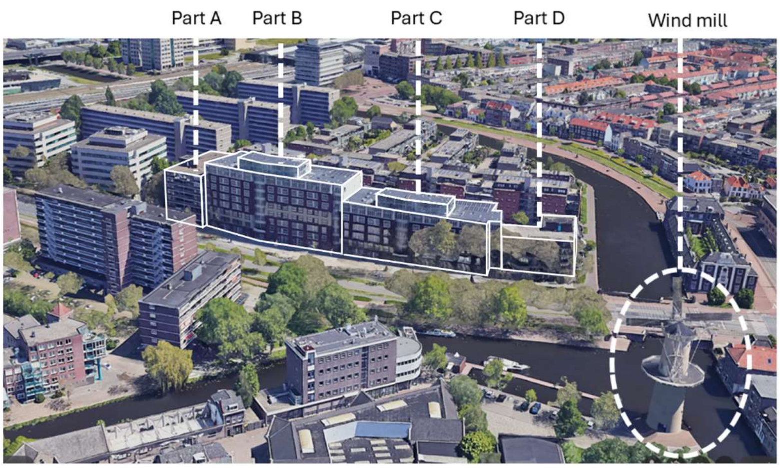


Figure 5.3 The four parts of SCYE010 in 3D view after transformation (Google Maps, 2024)

The original layout of the building (part B and C) is shown in Figure 5.4. In this original configuration, part B had 8 levels and part C has 6 levels. Part B and C area divided by a dilatiation. Both parts have a room for installations on top. In the first design of the architect, those two installations rooms were removed. Part B has got 1 larger level on top instead and part C got 2 levels, so the total height of part C matched the original height of part B (without the installation room). This design will be discussed later as well.

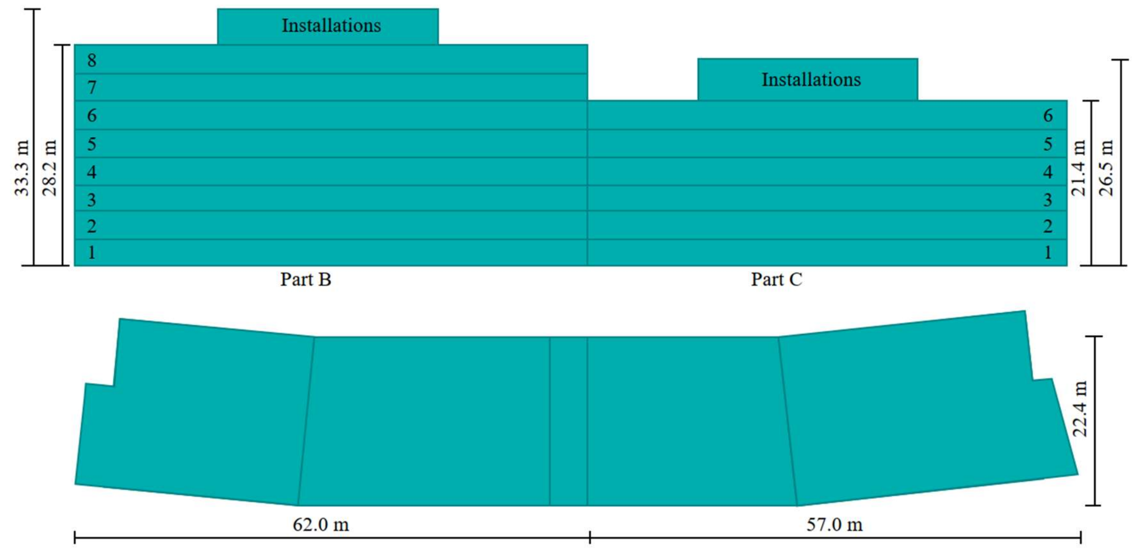


Figure 5.4 Original building layout (upper: front view, lower: top view)

Location

The building is located in Schiedam, in the province of Zuid-Holland. Figure 5.5 shows that SCYE010 is in the inland area of the TGB 1972 and wind area II for the Eurocode. A quick check of the ratio between the distance to the coast and the height of the building shows that SCYE010 is indeed located in the inland wind area and not in the coast area of TGB 1972:

$$\frac{a}{h} = \frac{20000}{28} = 755 > 50 \rightarrow inland$$

Furthermore, the building is located in an urban terrain.

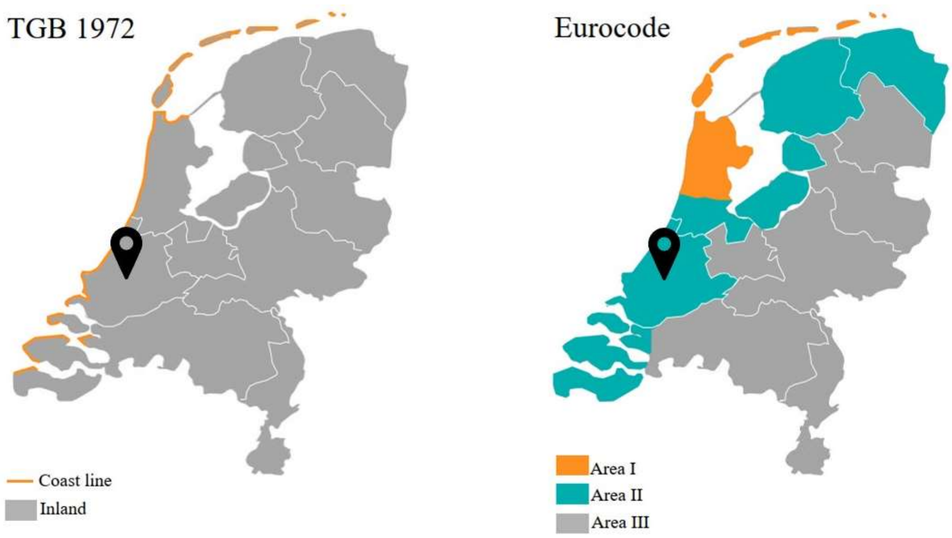


Figure 5.5 Location SCYE010 on map for wind load

5.2 *Load bearing system*

5.2.1 Lateral load bearing system | Stability system

The stability system of the building consists of multiple parts. In the transverse direction, the stability is given by the façade walls on both ends of the building and the walls in between the two parts, as is presented in Figure 5.6. The shear wall in the middle of the building (axis 19) consists of two concrete walls with a thickness of 200 mm. One wall provides stability for part B, the other one for part C. The façade walls on axis 10 and 19 provide stability as well and have a thickness of 250mm

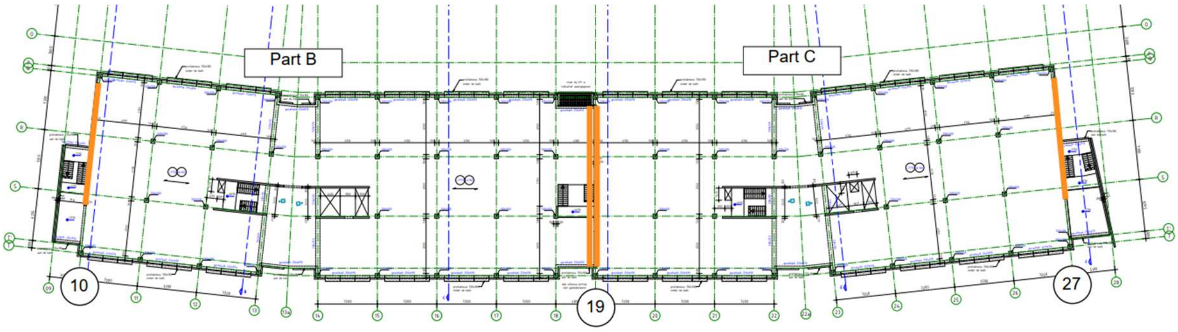


Figure 5.6 Shear walls in transverse direction (Pieters Bouwtechniek B.V., 2018a)

Stability in longitudinal directions is obtained by walls of the cores. The layout of these walls is presented in Figure 5.7 and all of these shear walls are executed in concrete with a thickness of 200 mm.

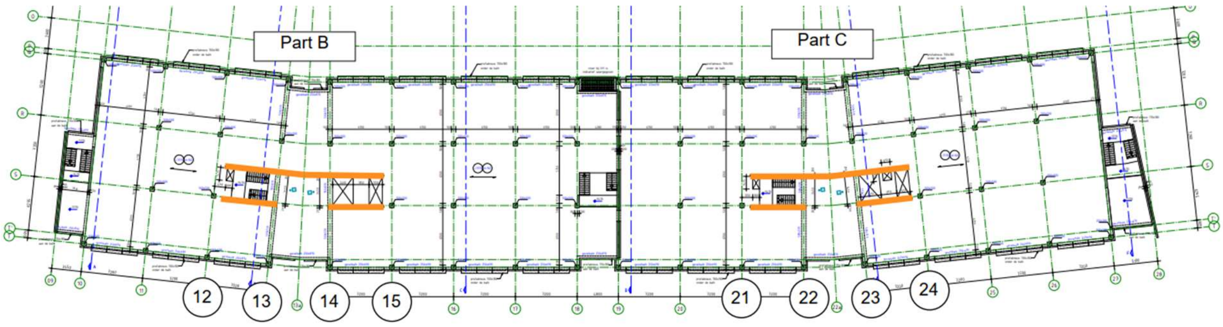


Figure 5.7 Shear walls in longitudinal direction (Pieters Bouwtechniek B.V., 2018a)

The parts on top, where originally the installations were located, have been replaced. For these new constructions a separate stability system is designed. Due to the relatively small size and weight of the new part, was a combination of a steel structure with wind braces and diaphragm action in timber walls possible. Figure 5.8 shows this specific part of the building from the Revit model.

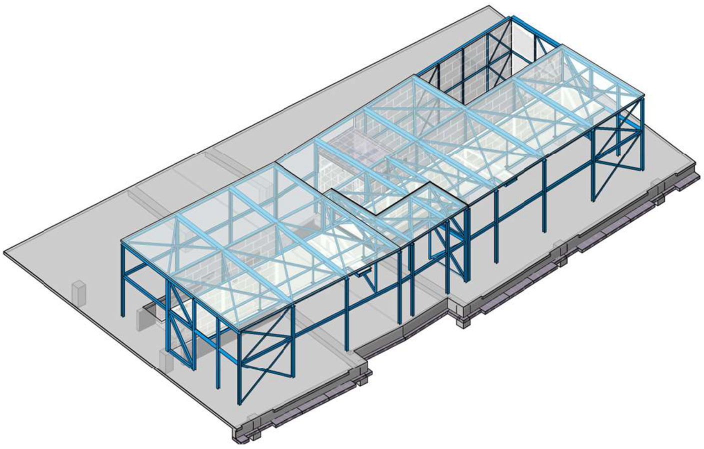


Figure 5.8 Stability system in top level for the final design (Pieters Bouwtechniek B.V., 2019)

The type of applied concrete is K300. The ‘K’ stands for ‘Kubussterkte’, which is Dutch for cubic strength. The value of 300 is the concrete strength after 28 days and its unit is kgf/cm^2 (Betoniek, 1970). This is an older unit, which is not part of the International System of Units (SI), but it can be transformed into N/mm^2;

$$\begin{cases} 1.0\, kgf/cm^2 = 98.0665\, kPa \\ 1000.0\, kPa = 1.0\, N/mm^2 \end{cases} \rightarrow 300\, kgf/cm^2 = 29.4\, N/mm^2$$

The Young’s modulus of K300 concrete is 30,000 N/mm^2 (Fisseha, 1998). This can be assumed to be the equivalent of C20/25, which is used nowadays.

5.2.2 Vertical load bearing system

The vertical load bearing system consists of prestressed concrete floors, which transfer the loads to columns, shear walls and the cores. In line with the columns, the floor is prestressed in the other direction as well. The grid is 7.2x7.2 m. Due to the prestressing, it was impossible to make large recesses in the floor. The maximum diameter is 180 mm, which is suitable for installations, but not for stairs. The dimensions of the columns are 500x500mm.

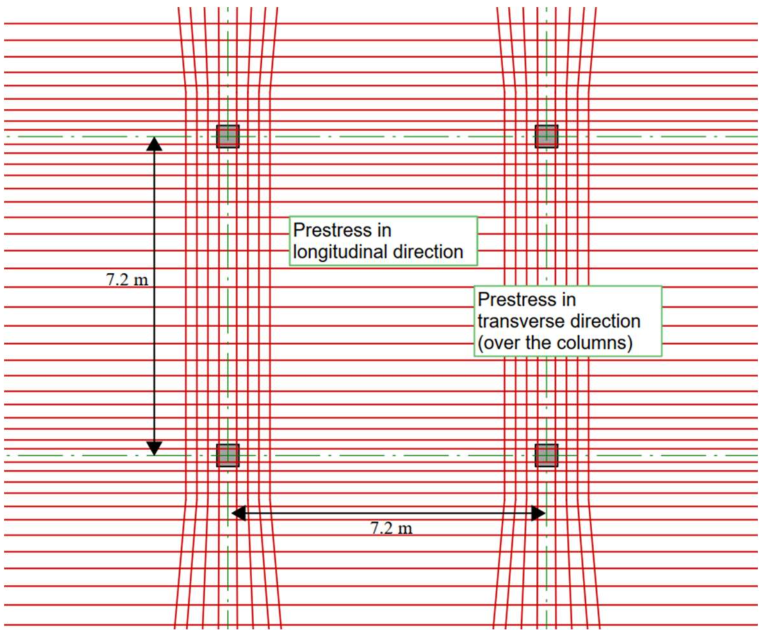


Figure 5.9 Prestress in the floors

Foundations
The foundation consists of in situ foundation beams with vibro piles (Ø480 mm). The pile depth is approximately -18.5m to -22.0m NAP. They have a characteristic capacity in the order of 1000 to 1200 kN. There is some overcapacity in the piles, this is due to the fact that according to the TGB 1972, extreme live loads are to be assumed on every level, while the Eurocode prescribes that this is only necessary for the first two levels and for the others, the momentane load, using psi factors, may be applied. The loads from the structure are transferred to the piles through foundation beams and blocks. However, the complete pile layout plan is not available from the archive. The layout plan could be composed from sketches in the calculations. To determine the bearing capacity of the piles extra standard penetration tests were performed.

Documents from the archive show that the concrete that is used for the foundation piles is K300. For the Young's modulus, the value of 30,000 N/mm^2 may be used, instead of the Young's modulus for cracked concrete (E_{cr} = 1/3 * 30,000 N/mm^2 = 10,000 N/mm^2), because all piles are in compression.

In the final design, balconies are added to the back of the building. A separate load bearing structure is designed using steel columns, which are supported by new foundation piles. Furthermore, to reduce the weight on the foundations, the original masonry façade is replaced by lightweight façade elements. This creates extra capacity in the foundation piles, but was also necessary, because the masonry was in bad shape. Additionally, this is beneficial for the architect, who has the opportunity to give a completely new look to the building.

5.3 Wind load design
The wind loads will be reviewed for five designs. The first three designs are the original design of the building from 1978, the intermediate design of the architect and the final design that has been executed in 2021. The other two are optimized designs, based on findings in this research. All of these designs will be discussed shortly, before diving into the calculation methods.

The first one is the original design. The calculations of the wind load from 1976 can be found in the archive of the municipality of Schiedam. Figure 5.10 presents the sketch of the original design.

The second one is one of the intermediate designs of the architect. As can be seen in Figure 5.11 in this design, extra levels have been added to part C and the extra levels cover a larger area than in the original design. This intermediate design was originally submitted to the municipality in 2017, but this was rejected due to the fact that this adaptation would block the view on the wind mill.

Therefore, the outlook of the final design ended up to be almost exactly similar to the original design. However, the levels that were used for installations in the original design have been removed and a new, lightweight structure has been added with its own stability system, which is connected to the existing stability system. The position of this top level is exactly the same as in the original design. In Figure 5.12 is indicated that the back of the building is also provided with balconies. These are added to the building to accommodate every apartment with outside space. Technically, this has influence on the wind load design, because the depth of the building increases. It could be argued that this is not a solid part, which catches a lot of wind, but for a complete and detailed wind load design, it should be taken into account. On the front of the building, loggias are made to provide outside space for those apartments as well. As these do not influence the original global shape of the building, this is not relevant for the wind load design. Strictly, the global wind load calculations do not need to be performed for the final design, since the building did not change; there is no added height, width of depth. Only, the new top level needs to be calculated according to the Eurocode, since it is replaced. However, in order to compare the designs well, the wind load calculations for all designs have been performed.

Two optimized designs are developed. The first uses the original plan of the top level, but the location of the top level on the roof of the building is optimized. Therefore, this design is in line with the wishes to not block the view on the wind mill any further. The second optimized design is not limited to the original top level plan, but its volume is optimized. This resulted in a larger coverage of the roof by the top level, but the level height has been reduced. The optimized designs are a combination of the previously presented designs. The software program RWIND is used to help determine the optimal solution. Figure 5.13 and Figure 5.14 present the sketches of these designs. The thoughts behind these designs will be explained in the next section (5.3.1).

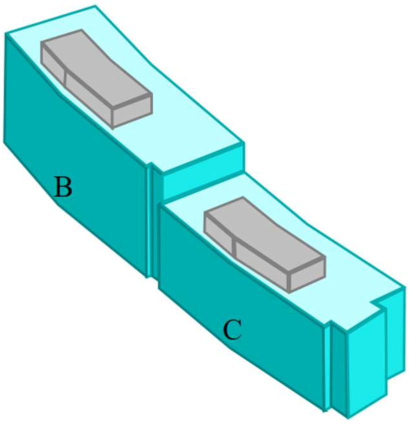


Figure 5.10 Original design

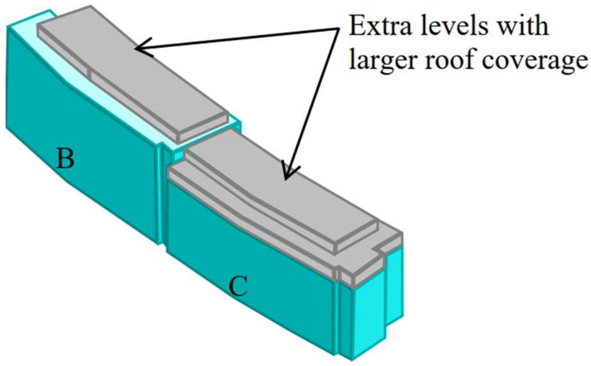


Figure 5.11 Intermediate design

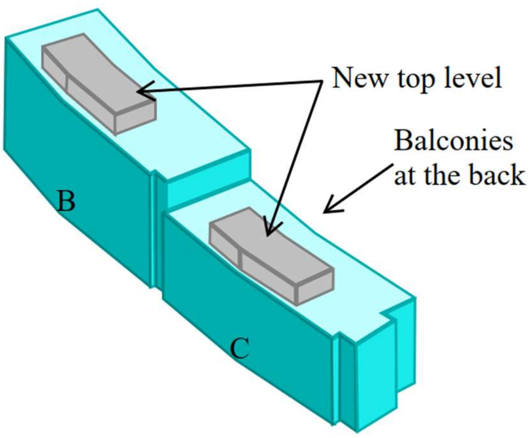


Figure 5.12 Final design

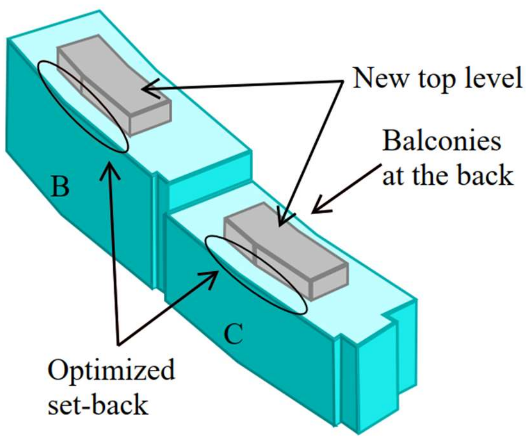


Figure 5.13 Optimized design - location

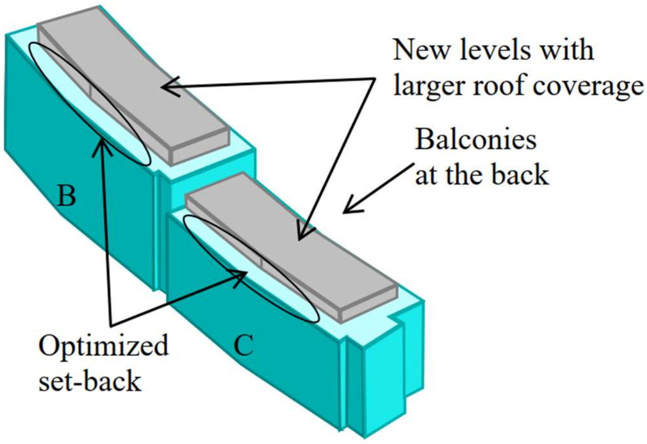


Figure 5.14 Optimized design - volume

5.3.1 Optimized designs

The previous section presents two optimized designs: The optimized location and the optimized volume for the top level. This sections highlights some of the relevant aspects of the design process, which leaded to the two optimized designs. First, the aims of the designs are elucidated. Next, the reasoning behind the set-back of the top levels is explained. The dimensions of the set-back and the set-back itself influence the wind load that the surface of the top level is subjected to. This effect is explained in the following section. Finally, the influence on the load bearing structure is discussed. This is an introduction to the effects, as they will be examined further in section 5.3.4.

Aim of the optimized designs

For the optimization of the design with the optimized location, there is one main requirement; the dimensions of neither of the parts of the buildings may be increased. The optimized design is based on a combination of the original and new designs. In this optimization process, CFD simulations in the software program RWIND are used to provide better insight into this optimization. The wind load in the transverse direction is governing, because the surface at this side of the building catches the most wind. Therefore, the width of the top-level will be limited to the width of the original and final design. This is also in agreement with the requirements of the municipality that this building may not block the view of the wind mill further.

The optimized volume design does neglect the requirements for the dimensions and aims for a maximisation of the area of the top level in combination with an optimized set back to reduce the wind pressure on this top level.

The possibility of extending the height even further by adding another level has been explored as well. By optimizing the position and volume of the levels, the extra forces on the foundation due to wind load could be limited. However, adding extra levels results in an increase of the permanent and live loads as well. This exceeded the allowed limits for increase of design forces on the foundation piles, making this variant not feasible. The vertical extension that is checked is for both variants 2 levels of 3.4 m, which is the standard level height for SCYE010. However, one of the requirements of the optimized location design is that the dimensions of these design are exactly the same as the original. For the optimized volume design the combination of the increased wind load in combination with the extra vertical loads from the top level, results in exceeding the original forces on the foundation by over 10%, which was found unacceptable.

Set-back of top level
As is indicated in Figure 5.13, the set-back of the top-level has been increased. In the original and final design this was about 1.0 m. The CFD simulation shows that the extra level is subjected to a high wind load, just as is assumed in the design codes. The contour plot of RWIND in Figure 5.15 visualizes this. The red colour, which is present on the majority of the surface perpendicular to wind direction 1, indicates that the wind pressures are the highest that are present on this building.

Running the intermediate design in RWIND as well, presented something interesting; the increased set-back of the top levels, which was about 3.5m, resulted in suction instead of pressure on the top level. In Figure 5.16, the blue and green colours on the plot show that it is about -0.22 to -0.88 kPa (=kN/m^2). This is caused by wind that gets pushed over the building and gets trapped in the corner between building and the top level, which causes turbulence.

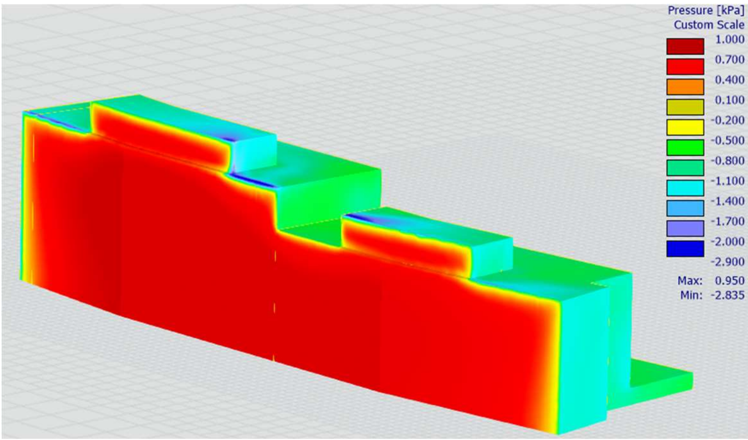


Figure 5.15 RWIND simulation original design

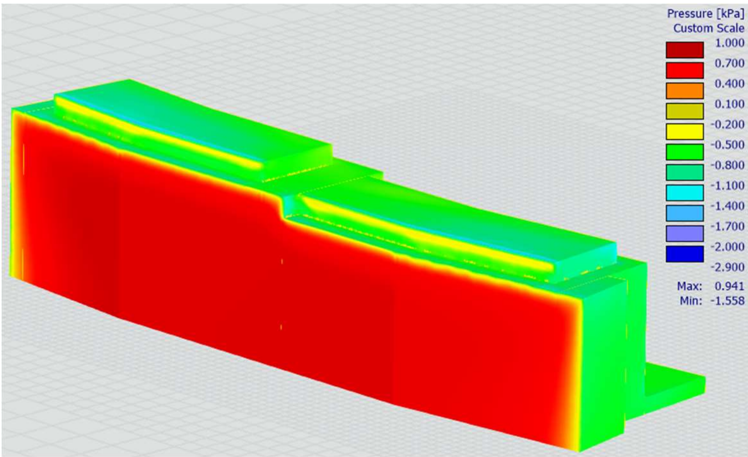


Figure 5.16 RWIND simulation intermediate design

A side view of intermediate design with a vector plot makes this perfectly visible. Figure 5.18 presents this side view. For the comparison, the vector plot of the original design is added as well (Figure 5.17). The figure clearly shows that for this case, the wind gets pushed over the building, including the top level. Due to this comparison, the set-back of the top-level became one of the design parameters of the optimized design.

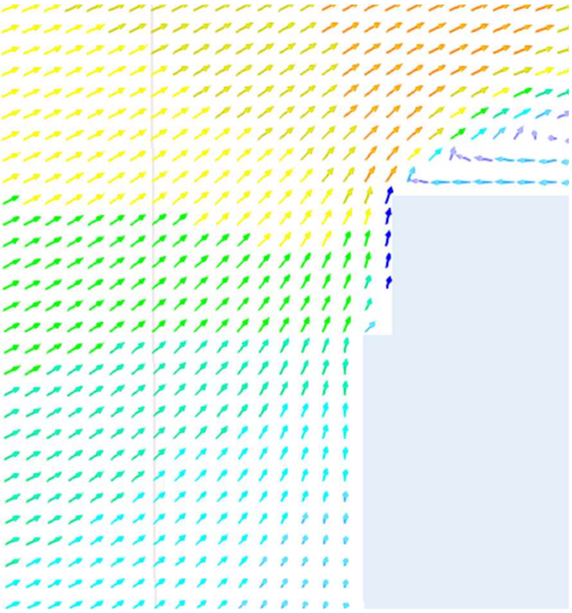


Figure 5.17 RWIND vector plot of the side of orginal design

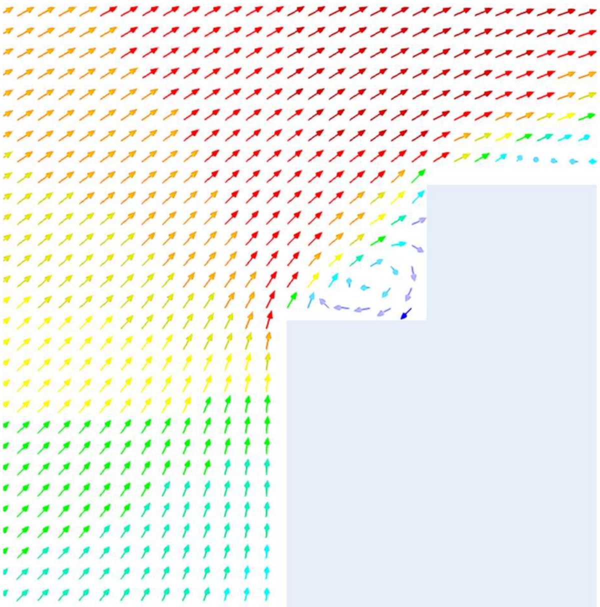


Figure 5.18 RWIND vector plot of the side of intermediate design

In RWIND it is an option to obtain the exact data along a line. The data is stored as a text file and is visualised using a Python script. Figure 5.19 presents this data for the original design and the intermediate design. The interesting thing is that is can also be compared directly with the wind pressures that have been used in the calculation for the original design (archive, green line) and at the time of the transformation (Eurocode, red line). The graph shows that by pushing the top-level 1.5 meters back, the maximum wind pressure on the top level reduces from 0.60 kN/m^2 to -0.39 kN/m^2.

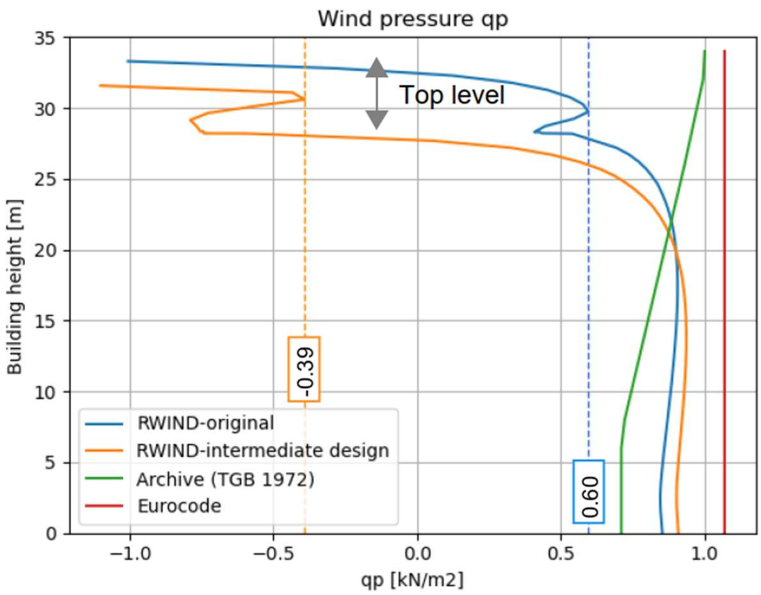


Figure 5.19 Wind pressures original design versus intermediate design

Through an iterative process, where the set-back of the top-level was moved back and forth, the optimal distance was found. The optimal solution is considered to be the design where the pressure on the top-level was minimized, while making sure there was pressure instead of suction. Figure 5.20 shows the results of all of the tested variants for the optimized location variant in comparison with the original and intermediate design. Based on the optimal pressure on the top level, variant 5 is chosen. This variant has a set-back of 2.5m, which proved to be the best match for reducing wind pressure on this top-level. Figure 5.20 only presents the results for part B of the building. For part C, the wind pressures will be lower due to the difference in height. The same amount of set-back is optimal for part C of the building. The graph that shows this, is stored in Appendix G.1.

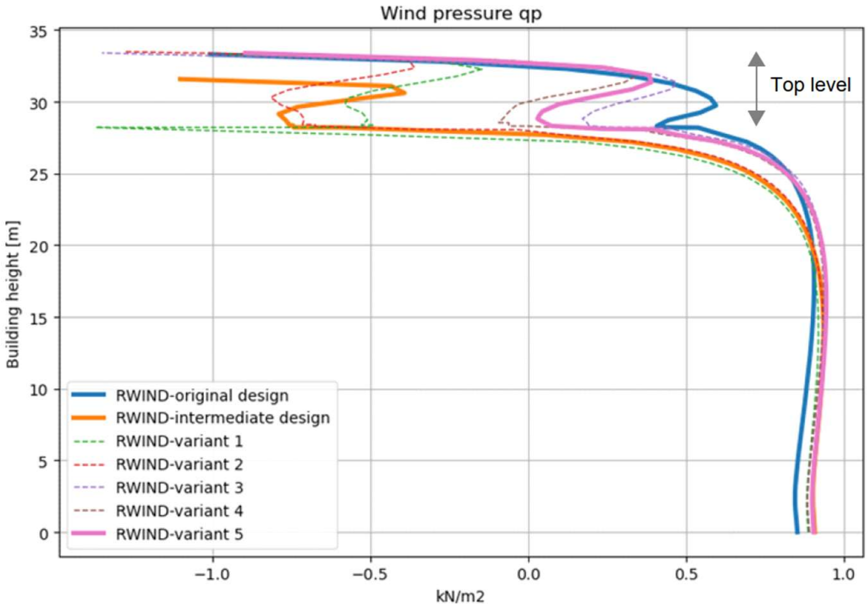


Figure 5.20 Wind pressure over height for all variants (Part B) – optimized location variant

For the optimized volume, the same strategy is applied. This results in an optimal set-back of 2.0 m on top of part B and 1.5 m on top of part C. In Appendix G.2, the graph of these tested variants is presented.

The optimized location design is presented in Figure 5.21 and its dimension in Table 5.1. The optimized volume design is shown in Figure 5.22. The dimensions of this design are in Table 5.2.

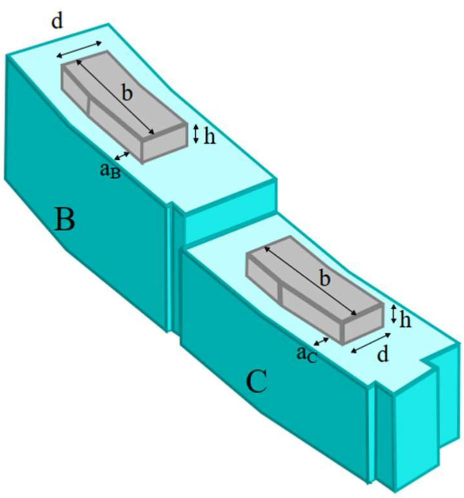


Figure 5.21 Optimized location design

Table 5.1 Dimensions of the optimized location design

Dimension	Value
h	5.1 m
$h_{B,total}$	33.3 m
$h_{C,total}$	26.5 m
b	30.0 m
d	10.0 m
$a_b = a_c$	2.5 m

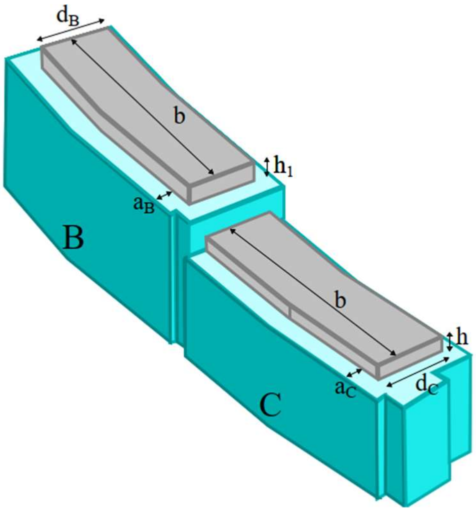


Figure 5.22 Optimized volume design

Table 5.2 Dimensions of the optimized volume design

Dimension	Value
h	3.4 m
$h_{B,total}$	31.6 m
$h_{C,total}$	24.8 m
b	55.5 m
d_B	20.4 m
d_C	19.4 m
a_b	1.0 m
a_c	1.5 m

Wind load

In the wind load calculation, the optimized designs will be compared to the other designs. By following the standard guidelines from the Eurocode, the maximum wind pressure needs to be assumed constant over the entire height of the building. However, by reviewing the output of RWIND, it could be argued that a reduced wind pressure may be assumed on the top level. Figure 5.23 presents the graphs of the optimized location model in comparison with the wind pressures according to TGB 1972 and Eurocode over the height of the building. To start, the graph of the optimized model indicates that the maximum wind pressure is lower than prescribed by the design codes. Secondly, the maximum wind pressure on the top level is significantly lower than the maximum wind pressure on the lower part of the building. For the detailed calculation of the optimized design, the wind pressure according to the Eurocode will be used for the lower part of the building, to provide sufficient safety and to account for small errors in RWIND. The difference between these wind pressures is $1.07 - 0.91 = 0.16\ kN/m^2$, which is means a multiplication factor of 1.2 is applied. To be on the safe side as well for the top level, the wind pressure obtained from RWIND is multiplied with 1.2 as well, which results in a wind pressure of $0.39 * 1.2 = 0.46\ kN/m^2$. For part C, a wind pressure of $0.47\ kN/m^2$ is obtained. This will be applied in section 5.3.3, where the wind load calculations are performed.

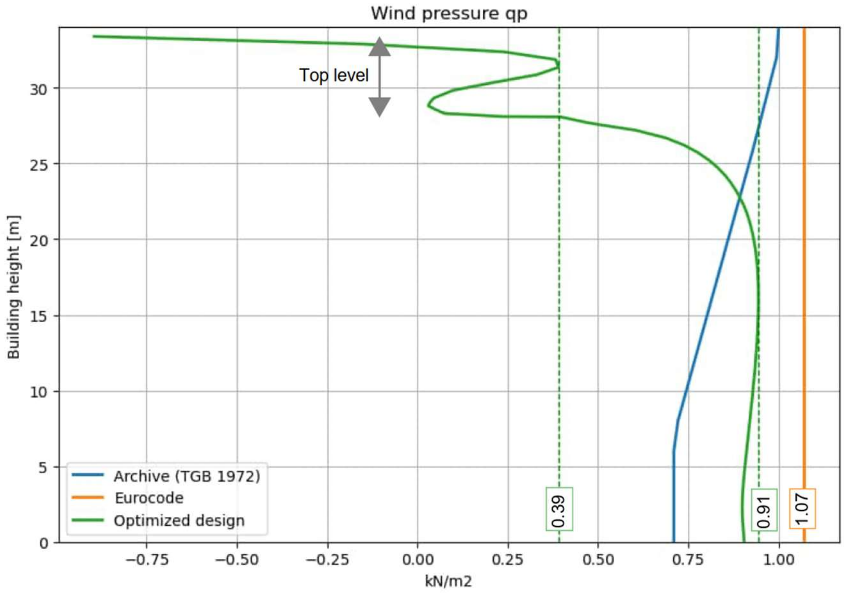


Figure 5.23 Optimized model in comparison with the archive and Eurocode calculation

For the optimized volume design, a similar graph is obtained, which can be used to determine the wind pressure on the top level. As can be seen in Appendix G.2, the wind pressure that RWIND gives as output is 0.38 kN/m^2 on the top level of part B. For this variant the difference between the wind pressure on the lower part of the building following from RWIND and the wind pressure according to the Eurocode is again a factor 1.19. Applying this factor on the wind pressure at the top level, a value of $0.38 * 1.2 = 0.46\ kN/m^2$ is obtained. For part C, a wind pressure of $0.36\ kN/m^2$ is obtained.

Influence on load bearing structures

For the transformation in 2019, a new stability system was designed for the top level. This will be used as a base for the optimized location design, but the optimized design requires small alternations. The influence of this optimized design in combination with the suggestions for changes in the wind load calculations will be discussed in section 5.3.3. Furthermore, the foundations need to be checked as well. This check will be performed in section 5.3.4.3. And finally, since the position of the top level has changed, a new system for the vertical load transfers needs to be designed. This will be elucidated in section 5.3.4.4.

The optimized volume design requires more drastic and complex solutions for the stability system and the vertical load bearing system. A completely new concept for the load bearing system of the top level is designed in order to make this possible. In advance, it should be said that this is a principle design for which the basic checks have been performed, but the complete calculations are not performed. The design will be further elucidated in the same sections as for the optimized location design.

5.3.2 Wind load models

The original building has been designed according to TGB 1972. This results in the wind load model over the height of the building as is presented in Figure 5.24. The engineer at that time has chosen to define Q3, which represents the wind load on the top-level where the installations used to be located (Groenenbeek & Poot B.V., 1975). Starting at z = 7 m, the wind pressure linearly increases to the maximum value. This is different that the approach that the Eurocode applies, where the wind pressure is constant over the height of the building.

For the intermediate design, a new wind load calculation needs to be performed. This was done according to the global Eurocode model, presented in Figure 5.25. For the final model, it is not mandatory to perform a new wind load calculation, because the building will not be changed in a way that changes the surfaces that are subjected to wind load. The global Eurocode model is perfect for making quick calculations and due to the assumption that the wind pressure is maximum over the entire height of the building, it is definitely on the safe side.

However, a more detailed model would be closer to reality and it would show the forces on the foundation more accurately. Figure 5.26 shows that at the top level, a reduced value for the wind load is used. The reduced wind pressure q_2 can have two causes; the first one is because the line load is calculated for the width of the top layer, instead of the entire building width. This decreases the load significantly, because the width of the top level is about half of the total building width. This holds for the optimized location design. The other check that can be performed is with a reduced value for the pressure on the top level. This is only applicable for the optimized designs and is considered to be an experiment, since it the Eurocode does not have rules for this. Although this method is not applicable in practice, it will be interesting to see the influence of this.

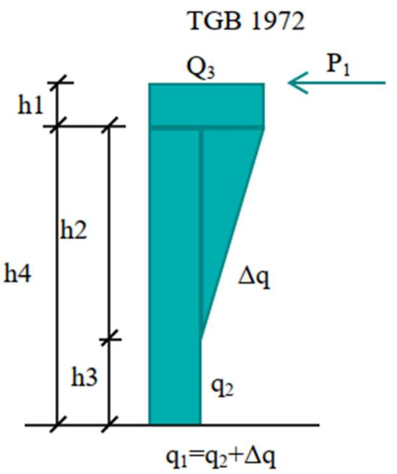


Figure 5.24 Wind load model according to TGB 1972

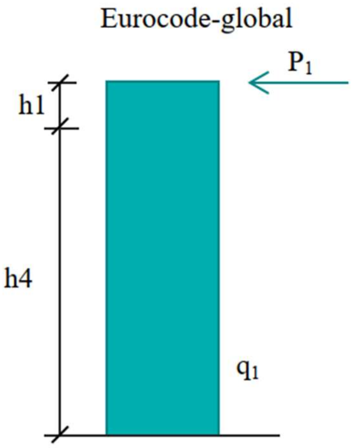


Figure 5.25 Wind load model according to Eurocode - global model

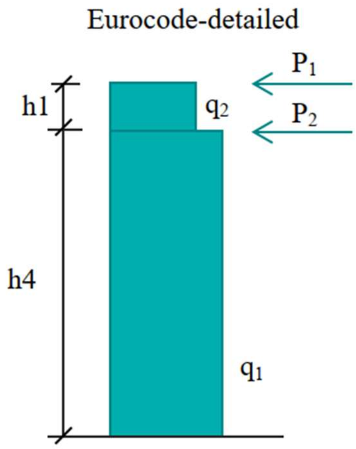


Figure 5.26 Wind load model according to Eurocode - detailed model

Furthermore, two wind directions are considered, which are shown in Figure 5.27. Wind direction 1 is the governing direction, since the surface at this side of the building catches the most wind.

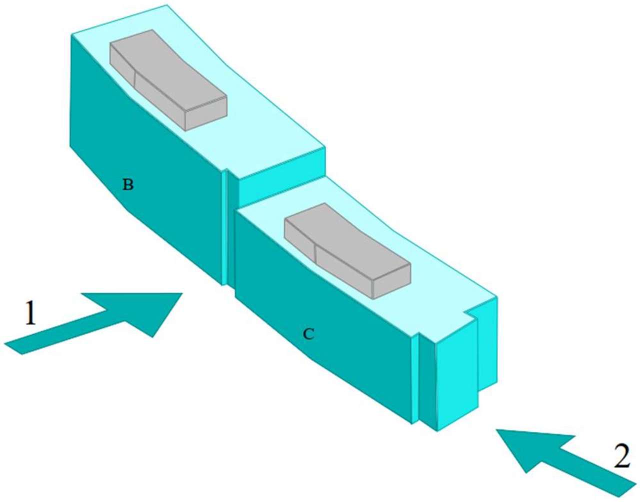


Figure 5.27 Wind directions

5.3.3 Wind loads calculations

In this section, the wind load calculations for the different designs of SCYE010 will be discussed and compared. This will give insight into the changed load that apply on the building since it was built in 1978. One of the differences is the wind load model that is applied. The three considered models are presented in the previous section (5.3.2). Furthermore, there is a change in applied wind load. This change in wind load has several other causes, which are considered points of interest, and on each of them will be elaborated in the following section. This approach combines all of the insights that the research of this thesis has provided. The results of this calculation can be used to design the new top level further and to asses whether the existing building can withstand the extra forces to which it is subjected.

This sections also highlights the steps of the wind load calculation and it provides the type of checks that have been performed in order to calculate wind load and to asses some elements of the existing building. The complete overview of the calculations is collected in Appendix H.3.

Points of interest

Reduction factors in Eurocode

First of all, the Eurocode implements some guidelines which reduce the wind load. One of them is factor 0.85 for the lack of correlation between maximum pressure and suction at the same time on a building (Koninklijk Nederlands Normalisatie Instituut, 2019).

Coefficients

The coefficients that are applied on this building are the pressure coefficient c_p, the friction coefficient c_{fr} and the structural factor $c_s c_d$. The application of these coefficients has changed a bit over the years. To start with the pressure coefficient; the share of the leeward side increases by 0.01, but the windward side remains the same. However, due to the implementation of the lack of correlation factor, the total pressure coefficient is lower than the one from the TGB 1972:

TGB 1972	$c_p = 0.8 + 0.4 = 1.2$
Eurocode	$c_p = 0.8 + 0.5 = 1.3\ (* 0.85 = 1.105)$

The second coefficient the friction coefficient c_{fr}. The Eurocode states that the friction force on the parallel sides of the building may be neglected when the total area of those parallel sides is smaller than four times the total area of the sides that are perpendicular to the wind direction. In Appendix H.1, for both parts and wind directions, a check has been performed to see whether the friction coefficient may be neglected. The checks shows that for both part the friction coefficient is allowed to be neglected in wind direction 1. In wind direction 2 this is not allowed. However, it could be argued that the wind from wind direction 2 has minimal effect anyway, since two buildings cover the sides of part B and C, which are part A and D. These have been highlighted in orange in Figure 5.28. These have an independent stability system, which does not transfer the forces to part B and C. Nevertheless, the wind from wind direction 2 should be taken into account for the full effect, since the situation could change in the future. According to the TGB 1972 the friction coefficient should be applied in every case.

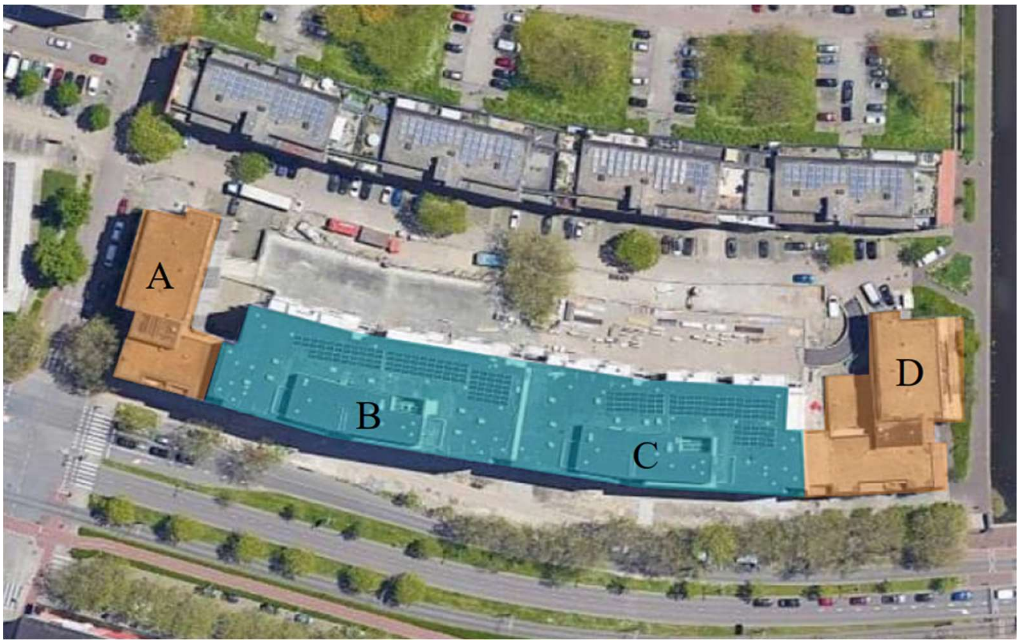


Figure 5.28 Buildings surrounding SCYE010 (Google Maps, 2024)

The structural factor $c_s c_d$ is equal to 1.0, due to the fact that this building has shear walls as stability system and its building height is lower than four times the depth of the building:

$$4 * d = 4 * 24.9 = 99.6 > h = 33.3\ m\ (part\ B) \rightarrow c_s c_d = 1.0$$

Wind pressure

During the literature review and the research part of this thesis, it has become clear that the wind pressure changed over the years. For SCYE010 in particular, this is caused by five effects.

The first one is the change of wind load that is prescribed by the design codes. According to the TGB 1972, for a building with a height of 33.3 m (part B of SCYE010), the wind pressure should be 1.01 kN/m^2 and the Eurocode prescribes 1.07 kN/m^2. However, it is important to check what has been used in the original design. For this case, in the archive calculations a wind pressure of 1.00 kN/m^2 is applied.

The second effect is the reduced width of the top level. The top levels have a width of 30.0 m, which is approximately half the width of one part of the building. For the global calculations, the maximum width is applied over the entire height of the building. However, applying a reduced width on the top level can have significant influence on the total loads that the building is subjected to. Especially, since the forces acting on the top level have considerable influence on the total bending moment on the foundation. This is due to fact that its moment arm is the largest at this point.

The third effect is the effect of the set back of the top level on the wind pressure. It has been concluded that an optimal positioned top level is subjected to a reduced wind pressure compared to the lower part of the building. There are not yet guidelines to decide on the set back that has to be applied in order to obtain this reduced wind pressure and therefore, an iterative process has been performed to determine this set back. The set backs for the optimized designs mainly are different due to the difference in height of the top level, the optimized location design has a top level height of 5.1 m and the optimized volume design has a level height of 3.4 m. The software program RWIND has been used to determine these reduced value of wind pressure on the top level and these have been multiplied by a factor of ~1.2 in order to account for the difference between the RWIND output and the values prescribed by the Eurocode. This is explained in section 5.3.1. The reduction of wind pressure on the top level in case of a set back is not discussed in the Eurocode, but might be allowed to be applied after wind tunnel testing or further research that validates these findings.

So far, the effects that are discussed are overall in line with the Eurocode. The next two effects are findings from the research, but should not be applied in real wind load calculations. The reason they are discussed and applied in this case study is to present the difference between the wind load that is used for designing buildings and the wind load that is location or building specific. Therefore, the fourth effect that is considered for this case study is the reduced location specific wind speed. Chapter 3 showed that the measured wind speeds at most weather stations resulted in a lower wind speed at this specific wind area than prescribed by the Eurocode. Figure 5.29 shows SCYE010 on the map of the Netherlands. It is located in the province of Zuid-Holland. To obtain the location specific wind for SCYE010, the closest weather station should be used, which would be representative of the location of SCYE010. From the earlier evaluated weather stations, the closest weather station to the city of Schiedam is Hoek van Holland. The research showed that Hoek van Holland measures wind speeds that are higher than the wind speeds according to the Eurocode for wind area II. Plus, Hoek van Holland is located right at the coast, which is not comparable to the location of SCYE010. The other previously considered weather stations are all relatively far away from SCYE010. Therefore, another weather station has been used to determine wind speeds at this location: Rotterdam Geulhaven. As can be seen in Figure 5.29, where Rotterdam Geulhaven and SCYE010 are indicated with a yellow dot, the weather station is relatively close to Schiedam, which makes it a better fit than Schiphol or De Bilt; two weather stations than are also positioned a bit more inland. There is another weather station in Rotterdam, which is most likely located at Rotterdam The Hague Airport. However, this is not specified on the website of the KNMI, nor anywhere else, which is why Rotterdam Geulhaven is used. The same steps as in section 3.2.1.1 have been executed in order to obtain the wind speed at Rotterdam Geulhaven, using the yearly maxima of a data set containing the average wind speed of the last 10 minutes of an hour for the period 1986 to 2011 and the Gumbel distribution. This results in a wind speed of $26.46 \approx 26.5$ m/s. This is slightly lower than what is prescribed by the Eurocode, which is 27.0 m/s. Although, the difference with the Eurocode increases when calculating the wind pressure, due to the squared wind speed in the formula for wind pressure. The full overview of this part of the calculation is given in Appendix H.2.

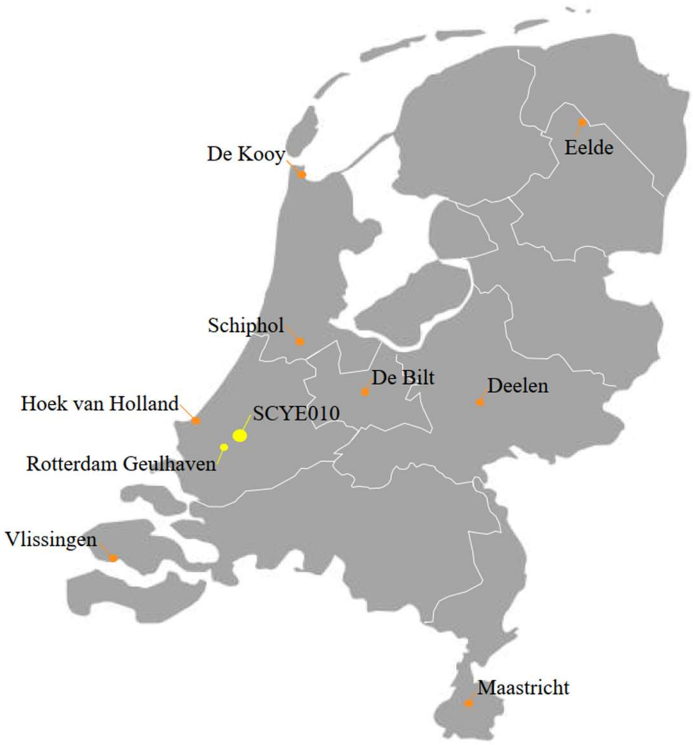


Figure 5.29 Overview of weather stations, including SCYE010

The final effect that is discussed in this section is the effect of the change in wind speed over the years. Section 3.2.2 has discussed the phenomenon that the wind speed slightly decreases over the years. For the case study an exact number of this decreased wind speeds is required. Again, the weather station Rotterdam Geulhaven is analysed for this location. In Appendix H.2, the analysis is performed and it results in a reduction of 0.25 m/s for the wind speed. It is important to bring again to the attention that these values should not be used in general, but are purely obtained to give insight into all of the elements that influence the wind pressure that is applied to a building. Furthermore, the trend of maximum wind speed does decrease constantly, but seems to stagnate a bit the last few years.

Dimensions

Another point of interest are the dimensions of the building that have been used in the calculation on wind loads. Especially the dimensions of the top level. Table 5.3 presents the dimensions of the top level according to the archive and according to drawings made by Pieters Bouwtechniek of the actual building. The difference between the values used for the width and height are significant. This is positive for the overcapacity of the piles, since they are designed for larger forces than they should be.

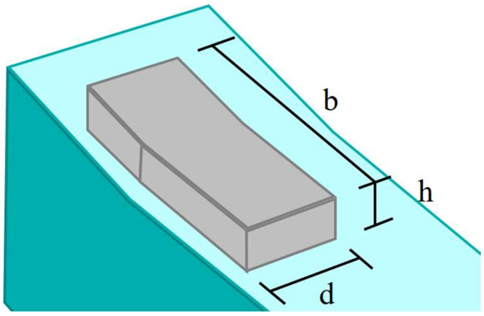


Figure 5.30 Dimensions of top level

Table 5.3 Dimensions of top level

	Dimensions archive [m]	Actual dimensions [m]
b	17.5	10.0
h	4.0	5.1
d	30.0	30.0

Foundation and deflections

The reaction forces in the piles are calculated assuming a fixed connection between the shear wall and the foundation. This will results in the largest bending moment and therefore, the largest pile reactions. A forget-me-not equation can be used to calculate this moment:

$$M_1 = \frac{1}{2} q_d \ell^2 \tag{6. 1}$$

This method is the left model in Figure 5.31. The right model assumes a spring connection and an infinitely stiff shear wall. The combination of the two is used to determine deflections at the top of the building, u_1 and u_2. The sum of these may not exceed the maximum allowed deflection of h/500.

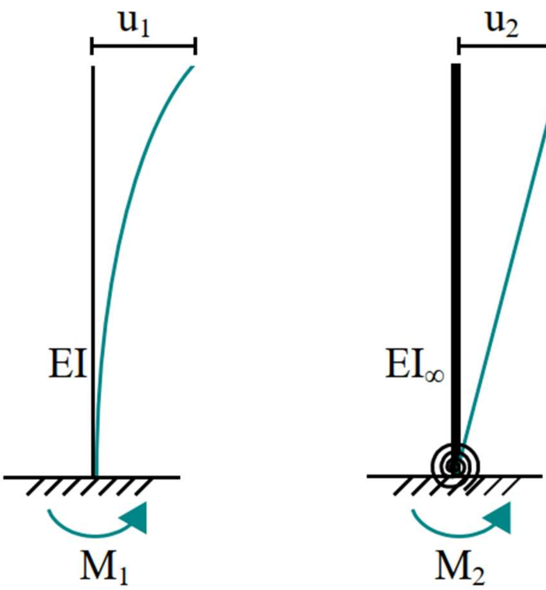


Figure 5.31 Two methods to calculate the bending moment and deflection

For the second method, the rotational spring stiffness of the pile group is needed. The rotational spring stiffness of the piles is obtained from the archival calculations. The method to determine this value can not be reconstructed completely. However, the values are on the low side, which means it is at least a conservative calculation.

To calculate the deflection, the first method makes use of a forget-me-not equation and the second method uses an equation that originates from virtual work. This is explained in Appendix H.4:

$$u_1 = \frac{\frac{1}{8}q\ell^4}{EI} \tag{6. 2}$$

$$u_2 = \frac{q\ell^3}{2C_r} \tag{6. 3}$$

Steps of the wind load calculation

To check the feasibility of the optimized designs for SCYE010, not only the wind loads on the building are determined; several other checks have been performed as well to obtain a complete overview of the possibilities of the transformation of the building.

1. Wind load calculation
   In the wind load calculation, the wind loads on the building are calculated as well as the total horizontal loads, the bending moment at the foundation and the resulting loads in the foundation are calculated.
2. Horizontal deflection of the building
   The horizontal deflection of the building is determined using two systems; one with a stiff connection between the stability system and the foundation and one with a spring connection. Together this results in the total horizontal deflection and must be lower than h/500.
3. Building weight calculation
   In the building weight calculation the total vertical forces on the foundation are calculated. This combines the permanent and live loads on the floors, as well as the weight of the structure itself and the wind loads that are calculated in step 1 of this process.

By following these steps through an iterative process, the results can be used to design a stability and vertical load bearing system for the new top level of SCYE010. Furthermore, it can show whether some elements need strengthening or other alternations. All of these steps are elaborated for the optimized designs in Appendix H.

5.3.4 Results

This section will mainly discuss the results of the wind load calculations. However, other calculations have been performed as well, such as the horizontal deflection and the building weight calculations. These will be shortly discussed, since they provide the information to verify whether the designs are feasible. Finally, the section will conclude with a discussion on the influence of the new top level on the stability system, vertical load bearing system and the foundation and a proposed design for it.

5.3.4.1 Wind load calculations

Table 5.4 and Table 5.5 show the results of the wind calculations for part B and C for the original design and the two optimized designs. In these tables three of the discussed effects are implemented:

1. Wind pressure according to Eurocode instead of TGB 1972
2. Reduced width of top level, instead of total building width
3. Reduced wind pressure on top level due to set back

The other two effects that were discussed in the previous section will be elaborated on later in this section.

For wind direction 1, the governing wind direction, the optimized volume design has lower resulting forces on the foundation than the optimized location design at both part B and C. This is mainly due to the lower maximum wind pressure on the building of the optimized volume design. This lower maximum wind pressure results from the height which is 1.7 m lower than the other design. However, for wind direction 2, it is clear that the reduced width of the top level, and thus lower wind load, of the optimized location design has more influence on the bending moment than the lower overall wind load on the building in the optimized volume design. Overall, it can be said that the detailed wind load model in combination with the optimized volume design results in similar forces on the foundation piles as the original design and would therefore be the preferred design. The complete tables are presented in Appendix H.3 as well.

Table 5.4 Wind loads on part B

Design	Original design	Optimized location	Optimized volume	
Wind load model	TGB 1972	Eurocode - detailed model	Eurocode - detailed model	Units
Wind pressure				
q_{p1}	1.00	1.07	1.05	kN/m^2
q_{p2}	0.71	0.46	0.46	kN/m^2
Wind direction 1				
$M_{wk,façade}$	16581	18281	16876	kNm
$M_{wd,façade}$	24872	27421	25313	kNm
R_k	288	317	293	kN
R_d	432	476	439	kN
Wind direction 2				
$M_{wk,façade}$	3849	4059	4128	kNm
$M_{wd,façade}$	5773	6089	6191	kNm
R_k	200	211	215	kN
R_d	301	317	322	kN

Table 5.5 Wind loads on part C

Design	Original design	Optimized location	Optimized volume	
Wind load model	TGB 1972	Eurocode - detailed model	Eurocode - detailed model	Units
Wind pressure				
q_{p1}	0.93	0.99	0.97	kN/m^2
q_{p2}	0.71	0.46	0.36	kN/m^2
Wind direction 1				
$M_{wk,façade}$	8270	9661	8405	kNm
$M_{wd,façade}$	12405	14492	12608	kNm
R_k	144	168	146	kN
R_d	215	252	219	kN

Wind direction 2				
$M_{wk,façade}$	2095	2217	2259	kNm
$M_{wd,façade}$	3143	3325	3388	kNm
R_k	109	115	118	kN
R_d	164	173	176	kN

To combine all of the findings from the research in this report, Table 5.6 provides the overview of all of the discussed effects, which are the three that were also visible in Table 5.4 and Table 5.5, plus the effect of the location specific wind speed and the effect of the change of wind speed over time. When evaluating these effects apart from each other, they have the similar effects; from the KNMI measurements follows a wind speed of 26.5 m/s and the change in wind speed over time is -0.25 m/s, which results in 27.0-0.25=26.75 m/s. Where 27.0 m/s is the base wind speed for wind area II that is used by the Eurocode. When combining these effects, the wind speed is further reduced to 26.25 m/s, which results in reduction of the wind pressure on the main part of the building of 4-6%, depending on the design that is considered. All of the effects together can result in reductions of the reaction forces in the foundation piles of up to 23%.

This table is specifically for the optimized volume design and for part B of the building. In Appendix H.3, the tables for building part C and for the optimized location design are stored. The effects result in similar reductions for each of the building parts and designs, therefore only one of them in shown in this section. Evaluating the influence these effects have on the resulting forces in the foundation piles, shows that the differences between the approximation of the location and building specific wind load is actually a lot lower than the wind load that is applied when using the global Eurocode calculation model (first column of Table 5.6). This is not without reason, because the Eurocode needs to provide wind pressure values that are safe and future proof. However, the comparison in Table 5.6 provide insight into the differences between the effects, which is useful background information for engineers. The calculations for the horizontal deflection and building weight are therefore based on the values presented in Table 5.4 and Table 5.5, which are more in line with the guidelines from the Eurocode.

Table 5.6 All of the effects applied on the optimized volume design for part B

Effect	Eurocode global model	Decreased width at top level	Reduced wind pressure at top level	Reduced wind speed according to data	Reduced due to changed wind speed over time	All effects	Units
q_{p1}	1.05	1.05	1.05	1.01	1.03	0.99	kN/m^2
q_{p2}	1.05	1.05	0.46	1.01	1.03	0.46	kN/m^2
Wind direction 1							
$M_{wk,façade}$	19535	19393	16842	18791	19163	15959	kNm
$M_{wd,façade}$	29302	29089	25262	28186	28744	23939	kNm
R_k	339	337	292	326	333	277	kN
R_d	509	505	439	489	499	416	kN

Wind direction 2							
$M_{wk,façade}$	4435	4361	4147	4266	4350	3927	kNm
$M_{wd,façade}$	6652	6542	6220	6399	6525	5891	kNm
R_k	231	227	216	222	227	205	kN
R_d	346	341	324	333	340	307	kN

5.3.4.2 Horizontal deflection

The horizontal deflection is based on the wind load values from Table 5.4 and Table 5.5 and uses two systems to determine the total horizontal deflection of the building parts. The first one assumes a fixed connection between the building (u_1) and the foundation and the second one assumes a spring connection (u_2). This is visualised in Figure 5.31. Summed, they may not exceed the maximum allowed deflection of h/500, where h is the building height. For both designs and for both building parts, all of the shear walls are checked and all of them match this requirement. The full calculation can be found in Appendix H.4, but Table 5.7 summarizes the results of the horizontal deflection of the optimized volume design for building part B and Table 5.8 for part C. For building part B, the highest Unity Check (UC) is obtained at axis 10: 0.92. This is most likely due to the low rotational spring stiffness that is used in the archival calculations. The shear wall and the foundation on axis 19 has similar specifications to axis 10. However, the rotational spring stiffness is 58% higher. It is unclear why this is the case and this could not be deducted from the archival calculations. Even though the UC is on the high side, it will be accepted, since it can be assumed that the rotational stiffness of the foundation of the shear wall on axis 10 is not that much weaker than axis 19, given the similarities. However, further inspection to find out why this axis is this much weaker than axis 19 might be insightful.

Table 5.7 Horizontal deflection of the optimized volume design, building part B

Wind direction	1		2		Unit
Axis	10	19	12-13	12-15	
u_1	1.7	0.9	5.1	0.9	mm
u_2	56.5	36.4	46.8	17.6	mm
u_{total}	58.2	37.3	51.9	18.5	mm
u_{max}	63.2	63.2	63.2	63.2	mm
UC	0.92	0.59	0.82	0.29	-

Table 5.8 Horizontal deflection of the optimized volume design, building part C

Wind direction	1		2		Unit
Axis	19	27	21-22	21-24	
u_1	0.3	0.5	1.6	0.3	mm
u_2	24.7	29.5	36.2	9.5	mm
u_{total}	25.0	30.0	37.8	9.8	mm
u_{max}	49.6	49.6	49.6	49.6	mm
UC	0.50	0.60	0.76	0.20	-

Table 5.9 and Table 5.10 present the horizontal deflections and the UCs of the optimized location design. These show to be very similar to the ones from the optimized volume design, which is in line with the expectation, since only the wind loads differ slightly, but the other specifications of each of the shear walls stay the same. For the optimized location design the maximum UC is 0.92 as well, so all of shear walls fulfil the requirement of not exceeding the maximum deflection of h/500. Therefore, based on the horizontal deflections, not one specific design is preferable above the other.

Table 5.9 Horizontal deflection of the optimized location design, building part B

Wind direction	1		2		
Axis	10	19	12-13	12-15	Unit
u_1	1.7	0.9	5.1	0.9	mm
u_2	56.5	36.4	46.8	17.6	mm
u_{total}	58.2	37.3	51.9	18.5	mm
u_{max}	63.2	63.2	63.2	63.2	mm
UC	0.92	0.59	0.82	0.29	-

Table 5.10 Horizontal deflection of the optimized location design, building part C

Wind direction	1		2		
Axis	19	27	21-22	21-24	Unit
u_1	0.3	0.5	1.6	0.3	mm
u_2	24.7	29.5	36.2	9.5	mm
u_{total}	25.0	30.0	37.8	9.8	mm
u_{max}	49.6	49.6	49.6	49.6	mm
UC	0.50	0.60	0.76	0.20	-

5.3.4.3 Building weight calculations

The building weight calculation of the optimized designs are compared to the original design. This is necessary to verify whether the new designs are feasible. The building weight calculation takes into account the permanent and live loads from each floor, as well as the load bearing structure itself and the wind loads. All of these loads together determine the load per foundation pile. According to the NEN 8700, the characteristic force on the foundation pile in the new situation may not exceed the force of the original situation by more than 15% (Koninklijk Nederlands Normalisatie instituut, 2023). However, during the design process of the original transformation around 2017, this norm was not yet published and in the design team decided that the maximum allowed increase was 10%.

The optimized designs meet the requirements of maximum 10% increase in vertical loads on the foundation piles. The opportunity to extend the building parts by an extra level has been explored as well. However, due to the extra vertical weight and additional wind loads, the forces on the foundation piles exceeded the allowed limits.

For the new situation, the situation of the optimized designs, the psi factor of 0.40 is applied for the live load on the second floor and upwards. This reduces the total live loads massively. Nevertheless, the largest part of the forces on the foundation piles comes from the permanent loads and these loads increase from 5.10 kN/m^2 to 5.45 kN/m^2 due to the addition of floor elements, including insulation. The foundation piles underneath the shear walls and the ones beneath the columns are checked and some of the reaction forces in the foundation piles even decrease for the optimized designs. The complete building weight calculations are stored in Appendix H.5.

5.3.4.4 The influence on the new and existing structure

The stability system

The new top levels of the two optimized designs require a new stability system which can be connected to the existing stability system. For the optimized location design a stability system can be applied that is similar to the stability system that is applied to the final design and the transformation of the building in 2021. This is shown in Figure 5.8 earlier this chapter. Even though the top level has an increased set back, the principle with wind braces and a steel structure can still be applicable.

The optimized volume design requires more drastic measures. The designed stability system consists of wind braces and diaphragm action in the roof that transfer the horizontal wind loads to the existing shear walls and columns for wind direction 1 and in wind direction 2, the existing shear walls will be extended to the height of the new top level. A preview of the principle design is given in Figure 5.32. This design also includes the vertical load bearing system.

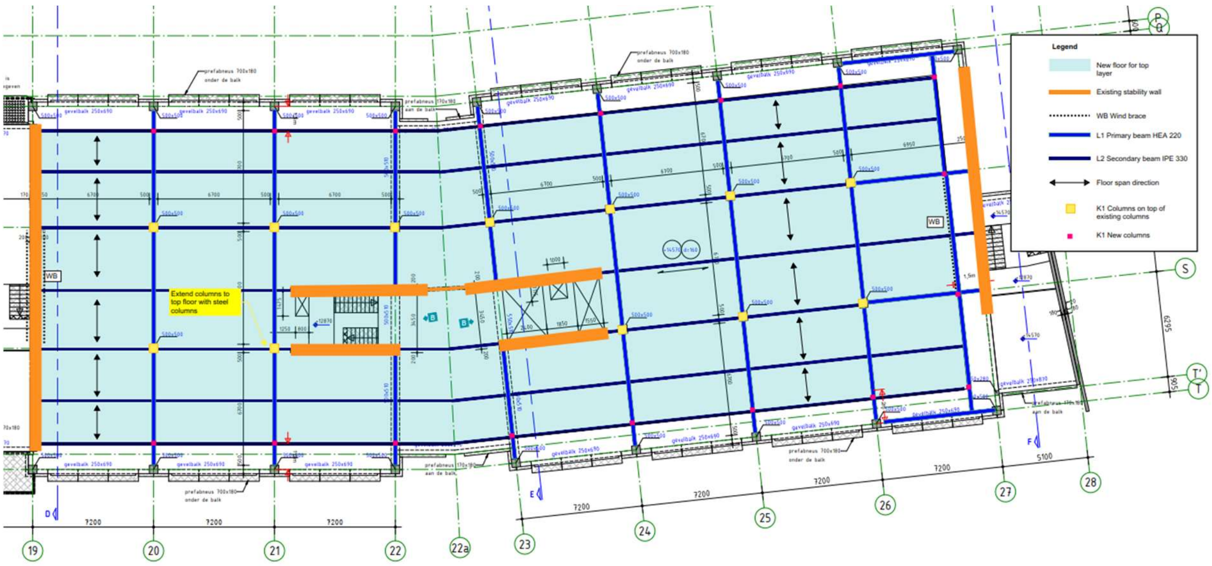


Figure 5.32 Constructive design for the optimized volume design - part C

For both the optimized designs, the shear walls are checked on deflection and the minimal increase in bending moment to ensure the stresses in the concrete would stay acceptable. Furthermore, the principle designs of the top levels, the stability systems combined with the vertical load bearing system are shown in Appendix H.6 and H.7.

The vertical load bearing system

Several elements of the vertical load bearing system of the existing building are in need of extra attention due to the new loads from the top level.

First of all, the floor of the top level, which is mostly roof in the original design. Especially for the optimized volume design is plays a large role, because this means that the function of this surface changes, which results in different requirements for the allowed live loads. Furthermore, the permanent loads on this surface will increase as well. The prestressed roof is not calculated for these extra loads. Therefore, a separate system of load bearing beams is designed in order to transfer the loads from the new top level directly to the columns and walls without subjected the existing roof to extra loads. A principle design of this option is shown in Figure 5.33. A system of primary and secondary steel beams is designed in order to span the 7.2 m in between the columns, in order to provide a direct transfer of forces to the columns. This is not possible with a 'simple' and lightweight timber system. A steel plate, positioned on top of the columns, prevents the primary beams from loading the prestressed concrete floor when it deflects. Another option would be to give the primary beam 'legs' that lift the beam completely. The advantage of this solution is that this makes it easy to implement installations under the floor system. The possible disadvantage is that it increases the height of the top level. The full overview of the vertical load bearing system of the optimized volume and location design is shown in Appendix H.6 and H.7.

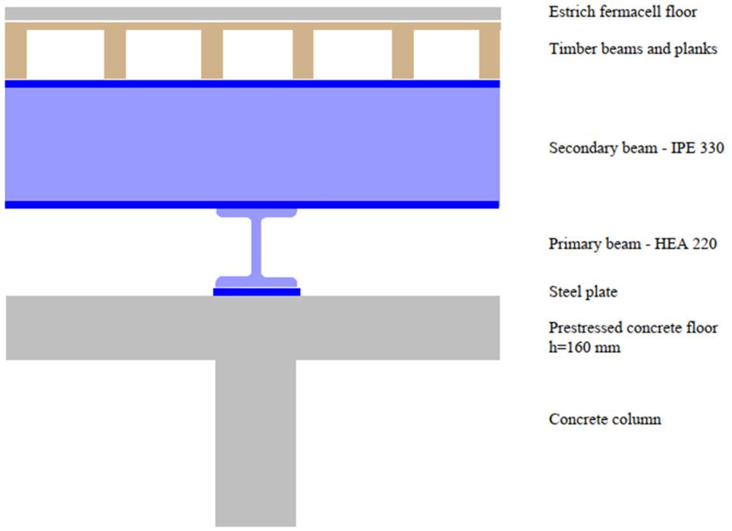


Figure 5.33 Principle design of floor of top level

Other elements that require extra attention are the concrete columns and walls that transfer the loads to the foundation. The increased stresses in these elements are checked for the two designs and both of them suffice. These checks are added to Appendix H.5.

The foundation

The tables in section 5.3.4.1 show that the forces on the foundations due to the wind load, increase for both of the optimized design. However, the building weight calculations which are discussed in section 5.3.4.3 show that for some cases, the total reaction forces in the foundation piles even decreases. From these checks can therefore be concluded that the existing foundation is able to withstand the forces resulting from the new top level and not extra measures need to be implemented.

During the transformation of the building in 2021, a separate foundation has been added for the balconies. Therefore, the forces resulting from this do not need to be added to the existing foundation.

The checks on the horizontal deflection, which uses the rotational capacity of the foundation, meet the requirements as well. So in conclusion can be said that the foundation is able to withstand the new horizontal and vertical forces.

5.4 Conclusion

For this case study, where the SCYE010 building in Schiedam was analysed, two types of top level designs have been optimized. For both designs, the opportunity for an extra level on top was explored, but the capacity in the existing building and its foundation was found insufficient. The first one is the optimized location design. This design makes use of the original layout of the top level, but with the help of a CFD model, an optimized set back of 2.5 m was given to this top level. The CFD model showed that this set back reduces the wind pressure on the top level, which results in minimized forces due to wind load on the stability and vertical load bearing systems. The second design optimized the volume of the top level. Again, a CFD model was used to determine the optimal depth, width and height of the level. This resulted in a lower height of the level, but one that covers a larger part of the roof. This reduced height of the top level allows for a smaller set back, of 1.0 m to 1.5 m, depending on the building part. Additionally, it reduces the wind pressure that has to be used over the entire height of the building, since that is based on the maximum building height.

To give a proper insight into the difference between the wind load that is implemented according to the Eurocode and the wind load that is location or building specific ánd incorporates the changes of wind speed over time, wind speed data from weather station Rotterdam Geulhaven is evaluated. This weather station is close to the city of Schiedam and is therefore assumed to be a better representation of the location of SCYE010 than the previously evaluated weather stations. This evaluation shows that an additional reduction of the wind speed, and thus wind pressure, can be implemented. It is important to highlight that this reduction may not be applied for design calculations, but is purely added to provide insight into the influence that the effect has.

To prove that the existing stability system and vertical load bearing system can withstand the forces resulting from the optimized designs, the original calculation is compared to the new calculation. The original calculation is according to the TGB 1972 and the new one according to the Eurocode. In chapter 2 is shown that the wind pressure that is prescribed by the design codes increases every time a new design code is published. However, some other guidelines in the Eurocode allows for more beneficial calculation methods. For example, the pressure coefficient c_p is 8% lower than the pressure coefficient from the TGB 1972. According to the Eurocode there are also situations where the friction coefficient may be neglected on the side of the buildings. In this case study this holds for the surfaces in wind direction 1.

Several checks have been performed in order to verify the feasibility of the optimized designs, such as maximum horizontal deflection of the building and the total building weight on the foundation. Both optimized designs prove to be feasible. However, the optimized volume design shows slightly lower resulting forces on the foundation piles and it creates a larger surface available for apartments. So, in conclusion can be said that the optimized volume design is the best fit for the top level of SCYE010.

It should be noted that the focus of this case study is on the front of the building. At the back of the building, balconies were added during the transformation in 2021. This extra depth of the building has been taken into account, but the effects of the balconies on the wind on the top level from the back side has not been investigated, apart from running a single CFD model. This showed that the balconies cause turbulence in the wind, but it was not further investigated. Additional research is necessary to give further insight into this effect.

PART III
CONCLUDING SUMMARY

6 DISCUSSION

This chapter provides an extensive discussion of this research project by critically evaluating the validity and significance. It starts with assessing the validity of the research and this is followed by the assessment of the results. Furthermore, the limitations of this research are assessed. Finally, the discussion explores the further implications of the research.

6.1 Validity of research

Validation of research is important in order to be able to draw proper conclusions. In this research, the validation of the results and the comparison of the results with the literature research is, for the most part already incorporated in the chapters. Therefore, all chapters will be discussed shortly and elaboration will be given if necessary.

Chapter 2 Comparison of the design codes for wind load

The literature research mainly entails the comparison of design codes for wind load. The primary focus of the literature research itself is to provide an overview of the relevant parameters for wind load and their background. It is not only used to base the research of the building and location specific wind load on, but it is part of the results, as it provides tools to asses the situation of adding levels to an existing building.

Chapter 3 Location or building specific wind load

The chapter of the research on the building and location specific wind load incorporates the comparison with the wind load from the design codes. This comparison is made to create understanding in the difference between an approximation of the actual wind load on a building and the wind load according to the design codes, but also acts as validation of the results. This can be interpreted as an additional safety that is implemented in the calculation, because the values that are used in the design codes should always be higher than the values that are obtained from the measurements with the same return period. In this chapter, the wind load is broken down into two parts; wind pressure and the pressure coefficient.

The location specific wind load is obtained by analysing measurement data of the KNMI. By choosing specific weather stations, the first selection is made, because some are known to be less reliable due to a relocation, new measurement equipment or changing of the surroundings. The KNMI has homogenised most of those datasets. This means that the irregularities are taken out of the dataset and they are suitable for comparison with modern data. However, analysing the data still showed that some weather stations where not suitable and therefore, these were not used for the research. This was done by first evaluating the complete dataset, before analysing the timespan of the reference periods.

The pressure coefficients have been analysed using CFD models. The program that is used is RWIND, which is essentially a digital wind tunnel. The Eurocode allows the use of such a numerical model, but only a verified and validated numerical CFD model is permitted. For this research, the CFD results are compared with existing wind tunnel tests from Tokyo Polytechnic University. They tested 22 models with different height/width/depth ratios, making them a perfect fit to compare with the variants that were considered for this research.

Chapter 4 Bearing structure

The next chapter discusses the bearing structure of both the existing building and the added levels. This is a combination of literature research and the application of findings from the previous chapters. This chapter contains advices on solutions that can be applied in practice. It should be noted that these advices are possible solutions and its application should be checked for specific buildings.

Chapter 5 Case study | SCYE010
The final part of this study contains the case study. All of the research findings are applied in chapters. At first, this case study was performed in order to present the effects of the research findings on an actual building and to showcase the differences. However, this process led to another promising research finding; the set back of the top level. This was not considered earlier, since the main focus was on the height/width/depth ratios of the building and what the change is when you extend the building vertically. The CFD analysis showed that a geometrical deviation of the top level could be very beneficial for the wind load calculation. The results are promising, but have not been validated using a wind tunnel experiment, since this was outside the scope of this study.

6.2 Assessment of the results

The assessment of the results includes a discussion whether the results of this research align with the expectations and why that is.

In general, the research findings are in line with the expectations; For the comparison of the design codes wind load was known on beforehand that the wind load calculations have been extended and did become more complex over the years, for example. Another thing is that the wind pressure that the design codes prescribe did increase over the years. Those are not things that had to be proven. The goal of this research is to give insight into those developments and to get a real feeling of what has actually changed over the years and how those developments have impact on transformations of existing buildings, especially when adding levels.

That was also the goal of the research into the location and building specific wind load. For this location specific wind load, some of the findings met the expectations. However, there were three main research findings that were not completely in line with the expectations. These will be discussed shortly.

1. The first one is from the analysis of the location specific wind pressure. The comparison for wind pressure showed that for most locations, the wind speeds in the design codes are indeed based on the measured wind speeds. After selecting a set with yearly maxima and using an extreme value distribution, a value was obtained that was slightly below the wind speed according to the design code that was evaluated. However, the results of two weather stations, Hoek van Holland and Vlissingen, showed that it could be argued that this weather station should be located in wind area I, instead of wind area II. The wind speeds measured at this weather station show a significant difference with the wind speed from the Eurocode for wind area II. This resulted in the proposition to list the coastline of the Netherlands under wind area I.
2. The second research finding is for the pressure coefficient. To start, it was expected that the pressure coefficient would not be constant over height, as is assumed in the design codes. All of the evaluated design codes use a constant value for the pressure coefficient on the windward and the leeward side. Only the Eurocode lets the pressure coefficient at the leeward side depend on the height/depth ratio and the value that results from this is applied as a constant over the total surface. The observation that the pressure coefficient is not constant over height is proven by comparing the results

with wind tunnel experiments. It is argued that it might be interesting to incorporate a deviating pressure coefficient over height into wind load calculations. However, this will not be useful in every type of building and it could complicate calculations for simple, low-rise buildings too much. Another noteworthy observation was the influence of the width of the building on the pressure coefficient. Both the CFD model and the wind tunnel experiment showed that this influence the distribution of the pressure coefficient over the surface subjected to wind load. In the determination of the pressure coefficient is not accounted for the width of the building, although this seems to have significant influence. It should be noted that in the Netherlands, often the c_s factor from the structural factor c_sc_d in incorporated in the pressure coefficient when a wind tunnel experiment is performed. This c_s factor accounts for the dimensions of the building. However, this factor is constant over the height as well and therefore, it does not influence the distribution of the pressure coefficient over the surface either.

3. The third interesting finding in this chapter was on the change of wind speed over time. Since the wind pressures according to the design codes generally increase, it was interesting to observe that the maximum wind speed shows a slight decrease over the years. This is confirmed by KNMI reports, but there is no clear reason for this, since most of the other climate related phenomena seem to become more extreme. Furthermore, the evaluated period is “only” 70 years and the global developments are not evaluated.

The research findings related to the bearing structure were as expected. Using the research findings from the previous chapters, effects on the bearing structure and the points of attention could be determined.

Finally, the case study resulted in an additional research finding that is a promising solution to reduce the wind pressure on top level that are added to existing buildings. The initial goal of the case study was to apply all of the research findings and to explore what kind of effect this has on the wind load calculations as well as on the existing building. Besides the conclusion that the findings have a beneficial influence, it also resulted into the finding that a set back of the top level can reduce the wind pressure on this level even more. This effect was not analysed in the research part of this thesis and was therefore discovered by accident. However, it shows to be very interesting, especially for designing new top levels. A set back was specifically analysed for this case study. On the other hand, geometrical deviations of the top level compared to the lower part of the building in general will have influence on the wind pressure. Other shapes, such as rounded corners have proven to influence the wind pressure distribution for example.

6.3 Research limitations

This study is unable to encompass all of the effects that influence the examined situation. However, awareness of these limitations gives the possibility of performing further research on these aspects. This section will discuss some of the main limitations of this research.

To start, only design codes since the TGB 1955 have been incorporated into this comparative research. The reasoning behind this choice is that stability calculations were not required before the publication of the TGB 1955. This makes the comparison complicated. However, transformations on buildings from before the use of the TGB 1955 do occur in practice.

Furthermore, building and location specific wind loads are very complicated to determine. First of all, it always is an approximation due to uncertainties in measurement equipment and with applying models to determine values. Secondly, the surroundings of a building constantly change; for example due to other building developments. Both modern city centres, such as Rotterdam, and smaller municipalities on the countryside are subjected to significant changes in the coming years. This makes that the wind pressure on a building should never be perfect for that specific building or location at that time, because one can never know how the surrounding will change in the future and the building must be safe in the future as well.

Another thing that was concluded in the evaluation of the building specific pressure coefficient, was that the resulting values from the CFD simulation exceeded the values from the Eurocode. It is complicated to pinpoint the exact reasons, however, there are some possible explanations:

The first one is the lack of a good verification model. For the first attempt, the settings from the example model that is provided by the project developer were used. This example was made in an older version of the program, and therefore, not the exact same steps could be followed. This resulted in values that were too far off from the Eurocode values. Therefore, some input parameters were changed. One of the applied changes was the used turbulence model. This gave better results, but were still a bit too high, which has been the reason too mainly evaluate the results qualitatively. Another setting that was tested, is the mesh that is applied in combination with the target residual value: This is input for the CFD simulation and this determines the level of detail for the CFD calculation. When the mesh density increased and target residual value decreased, the program quickly gave warnings and errors that the laptop that was used, is not suitable for these kind of calculations. Therefore, a standard mesh and target residual value were assumed.

The second possible explanation is that the CFD simulations that approach the results of wind tunnel experiments are extremely time consuming and it requires too much calculation capacity. For a CFD simulation that would give the same quality of the outcome of a wind tunnel experiment, extra servers must be hired, for example in a cloud, in order to facilitate such calculations. The fact that this is necessary, highlights the difference in quality that is currently obtained through CFD simulations. However, the advantage of the relatively quick CFD simulations is that it is possible to analyse many different variants in different locations. That was one of the requirements for this research and therefore, it is a good fit. However, with this quick calculation time, one need to give in on quality.

Finally, the focus of this research is on global wind loads on the total building. Therefore, local elements, such as canopies, balconies and rounded corners, are not taken into account. However, these local elements can have effect in the global wind analysis on the total wind moment. This level of detail is not implemented into this research, but these effects can follow from wind tunnel experiments for example.

6.4 Research implications

The main implication of this research is that this thesis helps to obtain better understanding into the development of the design codes regarding wind load and designing with wind load. Especially, for existing buildings where levels are being added to existing buildings. This improved insight into this topic will help with designing for these type of transformations in practice, but it also leads to new research topics, which will be discussed in the recommendations in chapter 0.

Furthermore, the results of the set back of the top level are very interesting. The beneficial reduction of pressure on this top level makes it attractive to apply in practice. In this case, the set back influences the wind stream positively. However, the set back of the top level might not be the only geometrical deviations that influences the wind pressure beneficially. Further research is necessary to determine what the ideal geometry of a top level is, because it is very interesting to explore these possibilities for engineers when designing a new top level.

Additionally, one of the implications of this research could be that the use of CFD models for wind load design will be researched more. For now, CFD models are not yet applied in practice for wind load design on buildings, but this research contributes to the understanding and possible benefits that could result from the use from these type of these models. Starting point is that safety must be guaranteed.

7 CONCLUSIONS AND RECOMMENDATIONS

This chapter provides the conclusions of the research and the recommendations for further research. The conclusion in section 7.1 answers the main research question, using the sub-research questions. Section 0 will conclude the thesis with the recommendations for further research.

7.1 Conclusion

The findings of the research will answer the main research question that is defined in section 1.4. This conclusion is given in section 7.1.1. Additionally, general recommendations are given of enhanced assessment of wind load on existing buildings when applying additional layers in section 7.1.2.

7.1.1 Answering the research question

The main research question of this thesis is:

"What is the influence of changes in the applied wind load since the TGB 1955 building design code on existing buildings when applying additional levels and how representative is the wind load that is prescribed by these design codes in the view of optimal designing for adding building levels?"

The answer to this question consists of many facets. The main research question is therefore broken down into three sub-research questions (as defined in section 1.4) and together they can be used to form an answer for the main research question.

Sub-research question 1

"What is the difference between the wind loads defined in the TGB 1955, TGB 1972, TGB 1990 and Eurocode?"

This sub-research question is mainly answered in chapter 2. The conclusion of this chapter answers this question best. However, that is a rather extensive conclusion, so therefore, a summary that captures the main and general findings will be given here.

To answer this sub-research question, first, the parameters that influence the wind load are defined; The main distinction can be made between wind pressure and coefficients. For the wind pressure conclusions are drawn about the location, the magnitude of the wind pressure and the complexity of the formulas of wind pressure.

The location of the building determines the wind area under which it falls. This is visualised in wind area maps that each design code provides. These maps have developed significantly and by combining the maps of all considered design codes, an area in the east of the province of Noord-Holland has been pointed out. Here, the difference between the divisions in high and lower wind pressures over the years is the largest. Therefore, it can be concluded that this is an extra point of attention when adding levels to existing buildings in this region. Furthermore, the magnitude of the wind pressure that is prescribed by the design codes in comparable wind areas did increase over the years. In the coastal areas, this difference is the largest and in the more inland areas the difference is less. However, the difference can still be significant. Over the years, the formulas that describe the wind pressure have become more complex, making the wind pressure on a building more realistic and a better fit to the situation of the building.

The second part for which the design codes are reviewed are the coefficients. Generally concluding, it can be said that the coefficients increased in amount and the formulas to determine these coefficients did become more complex as well. This way, especially since the introduction of the TGB 1990, the building specifics are better represented in the coefficients. Two coefficients should be highlight when comparing the design codes; These are the external pressure coefficient and the dynamic factor.

To start with the external pressure coefficient; This coefficient is important to focus on, since this is included in every considered design code and this coefficient is applicable to every wind load calculation. The net pressure coefficient is the sum of the coefficient at zone D and zone E. The value of this net pressure coefficient varies per design code; From 1.3 for TGB 1955 to 1.2 for the TGB 1972/TGB 1990 to 1.5 for the Eurocode when $h/d = 5$. When $h/d \leq 1$, the value for zone E is -0.5, resulting in a total pressure coefficient of 1.3. The Eurocode introduces another factor, which reduces the combined pressure coefficient: the lack of correlation factor of 0.85, which takes into account the effect that the extreme pressure at the windward side and the extreme suction at the leeward side do not occur simultaneously. By multiplying this factor with the pressure coefficient, the value becomes lower than the 1.2 from the TGB 1972 and TGB 1990. This results in a situation where the height/depth ratio of a building can become critical when adding levels, because by including this factor, a turning point occurs at $h/d \leq 3.2$ for which the combined pressure coefficient according to the Eurocode becomes lower than the values from the TGB 1972 and the TGB 1990.

Due to a combination of the implementation of more parameters and the more detailed wind load models that the design codes provide, the calculations of the newer design codes have become more detailed and the wind load is a better fit to the situation of the building. This is beneficial in case of adding levels to an existing building.

Sub-research question 2
"How does a building or location specific wind load relate to wind load defined by the design codes?"

To answer this sub-research question, methods had to be defined, which can be used to determine the building or location specific wind load. This method, the calculation and the results are discussed in chapter 3. Wind load on a building cannot be obtained directly. Therefore, a simplified formula of wind load is used to break the wind load down to two parameters: $q_w = q_p * c_p$, so wind pressure and the (external) pressure coefficient. For both parts, the interesting findings will be highlighted.

To start with the wind pressure; The obtained results show that at most weather stations the wind speeds are just below the prescribed wind speed in the design codes for that specific wind area. These are the results that are expected and this gap can be defined as extra safety. Next, the wind speeds were calculated to wind pressures. Due to the squared wind speed in the formula for wind pressure, the difference between the location specific value and the value presented in the design codes did increase.

Two things stood out in this comparison; The first one is the comparison between the results of the KNMI measurement dataset and the TGB 1955 wind speed. This difference is up to 30% for the wind speeds. The reason for these extremely high differences is because the wind speed that was used as a base by the design code committee of that time is 29.0 m/s at De Bilt, which has never been measured there and is not obtained by evaluating the yearly maxima data. Next, to add extra safety, they rounded the obtained wind pressure up several times. This method did not align with the chosen method in this research and resulted in massive differences in outcome. Therefore, a proper comparison could not be made. However, it can be said that a significant amount of extra safety is incorporated into the wind pressure.

The second thing that stood out were the high wind speeds that are obtained for weather stations Hoek van Holland and Vlissingen. These are significantly higher than the wind speeds that are prescribed by the design codes in their wind area. This holds for all the design codes for which these weather stations have been analysed. According to the TGB 1990 and the Eurocode, these weather stations are located in wind area II. However, the obtained wind speeds are more in line with the prescribed wind speeds at wind area I. The wind speeds are even higher than for the analysed weather station in wind area I: De Kooy. Therefore, an alteration to the current wind area map is proposed, where the coast line would fall under wind area I instead of wind area II.

The external pressure coefficient is the second parameter of the wind load that is analysed for its representativeness. For this analysis, only zone D and E are considered. The comparison between the design codes (sub-research question 1) showed that all the design codes prescribe constant values for the pressure coefficient in zone D and E. The Eurocode allows for zone E to depend on the height/depth ratio of the building, but assumes a constant value over entire surface subjected to wind load nonetheless. The outcomes of the CFD simulations resulted in higher values than the Eurocode, which is the reason that it is concluded that the CFD outcomes should be used with caution and not quantitatively. However, for a qualitative comparison they do suffice. Nonetheless, the shape of the CFD pressure coefficients over the height of the building are very similar to wind tunnel experiments, which makes the qualitative outcome useful. These models show that the pressure coefficient is not constant over the surface. When analysing the distribution of the pressure coefficient over the height, it is clear that it follows the same shape as the wind pressure and its maximum occurs at about ⅔ of the height of the building. This similarity with the wind pressure is as expected after analysing the standard formulas for the pressure coefficient. Accounting for this distribution over the height could be relevant, especially for mid and high rise buildings, as for the low-rise buildings, this would only complicate the calculation, without gaining much.

Another interesting finding is the influence of the width of the building on the distribution of the pressure coefficient over the surface that is subjected to wind load. Both the outcome of the CFD models and the wind tunnel experiments show that the width has influence on this distribution. This is interesting, because the width of the building is not one of the input parameters to determine the pressure coefficient in any of the design codes. The wind tunnel experiments show that the zone where the maximum pressure occurs shifts as well, depending on the width. From the analysis of the considered variants can be concluded that for a more slender building, the pressure coefficients are higher than for a wide building.

Furthermore, the change of wind speed over time is evaluated. This gives additional understanding of the reasoning behind the overall increase of the wind pressures according to the design codes over time; either the safety margin on the wind speed must be increased or the wind has become stronger. The analysis can be concluded that the maximum wind speed shows a slight decrease over a period of the last 50-70 years. This is confirmed by the climate report of the KNMI. The amount of decrease is strongly depending on the location of the weather station, which makes it impossible to give an absolute value for the decrease of the wind speed in the Netherlands in general.

Sub-research question 3
"Adding layers to an existing building influences the wind load in a twofold manner: extra surface subjected to wind load and a difference in design codes for wind load between the now and past. What influence does this changed wind load have on stability systems and vertical load bearing structures of the existing structure when extending an existing building?"

This question already makes the division between the two influences that the adding of layers to an existing building has on the wind load; extra surface subjected to wind load and a difference in prescribed wind load between now and when the building was designed. This question combines findings from sub-research question 1 with the practical matter of adding levels to an existing building. The answer to this question follows from the research in chapter 4.

The two described influences result in an increase of the wind load on the stability system. Additionally, adding levels means that the total height of the building increases. The applied wind pressure is based on this total height, and will therefore further increase. Furthermore, the extra levels need to be stabilized as well. They need a new stability system, which is connected to the existing stability system.

One of the two mentioned influences is the development of the wind load according to the design codes. For almost every case, the wind pressure has increased, which means that the stability system of a building structure needs to be able to withstand higher loads. Apart from that, TGB 1955 and TGB 1972 only used characteristic wind loads, whereas the newer design codes use partial safety factors. For TGB 1955 and TGB 1972, a higher material factor was applied, which provides extra safety on that side of the equation and the capacity to take the wind loads. Therefore, the comparison of loads between the original design and the new design should be between characteristic loads.

The vertical load bearing system is often the same system that provides the stability of the building. Adding levels increases the total vertical load as well with additional permanent and live loads. Buildings that are eligible for extra levels, are often subjected to a total transformation and they get a new function. With this new function, the loads on the floors most likely change, depending on the original and the new function. The change of the wind load does not need to influence the vertical load bearing system directly. However, it does influence the bending moments in systems that have a double function, such as concrete cores. The vertical bearing system, including the foundation does need to be able to withstand this.

For the research on bearing structure can be concluded that the main focus when adding levels should be on the optimal use of the substructure

7.1.2 General recommendations for enhanced assessment of wind load on existing buildings when applying additional layers

This section will provide general recommendations for enhanced assessment of wind load on existing buildings when applying additional layers. These are additional findings from this research that do not directly answer to the main research question, but are noteworthy.

The first recommendation is that a more detailed wind load calculation can have a significant impact on the total wind load compared to a more global one. This can be beneficial, as it better describes the building as it is. This could result in reduced total horizontal wind loads and a lower bending moment. The case study is an example where this detailed calculation showed to be extra beneficial, due to the reduced dimensions of the top level, that resulted in even lower wind loads.

Another recommendation is the application of a geometrical deviation of the top level, preferably in the governing wind direction. The case study showed that an optimized set back has a reducing effect on the wind pressure on the top level, but other configurations could be beneficial as well. There are no guidelines in the Eurocode regarding this reduction. However, a numerical model may be used according to the Eurocode, when there is a validation model available.

Additionally, it is important to be aware that some building types lend themselves better for the addition of levels. A good example of such a building types are building which have had an industrial function in the past. These are often made with a robust, concrete construction and high live loads of 5 kN/m^2 or more are assumed. These have quite some overcapacity and it is relatively easy to strengthen them. Another preferable characteristic of a building is that it already has multiple levels. When a building has multiple levels, there is often more overcapacity in the foundation. This is of importance, because not only the horizontal forces increase by adding level(s), but the vertical forces as well. Strengthening a foundation is a complex process, which is time consuming and has high investments costs, and is therefore it is preferable to make use of the overcapacity in the existing foundation. Finally, some types of stability systems lend themselves better for strengthening. A good example of such a system is a steel braced frame. It is relatively simple to add extra braces in comparison to strengthening a concrete core.

7.2 Recommendations for future research

In the preceding chapters, the discussion and conclusions are presented. Recommendations for further research are derived from these analyses and these include:

1. **Wind area map**: Based on the evaluated wind speed data from the KNMI, a new wind area map is proposed, because weather stations along the coast in wind area II show higher wind speeds than the Eurocode uses as base wind speed in this wind area. They do align better with wind area I. The new wind area map has the wind area map of the TGB 1990/Eurocode as a base, but the coast is now assigned to wind area I as well. An idea is given of what it could look like in Figure 3.31. However, to make a proper proposal that can be well substantiated, more research is required. This will include the evaluation of more weather stations along the coast line of the Netherlands.

Possibly, this research will also require to investigate the wind speed above the North Sea. The KNMI has offshore observation and measurement stations that provide such information. However, these stations are under pressure due to closing and disassembling of several oil rigs in the North Sea (KNMI, 2023a).

2. **Pressure coefficient**: In the design codes, from TGB 1955 until now, the pressure coefficient is given as a constant. Both CFD and wind tunnel experiments prove that the pressure coefficient is not a constant over the surface subjected to wind load. This constant value that is applied is the maximum value that occurs on the surface and is applied over the entire surface. For mid and high rise buildings it is interesting to find out whether it would be possible to create a simple model that approaches the real distribution of the pressure coefficient on the surface better. This can be either a deviation over the height of the building, or, the width can be incorporated too. Because the research showed that the width of the building can have significant influence on the distribution of the pressure coefficient as well. On the other hand, the additional calculation time and its complexity should be weight against the benefit that it would bring. Otherwise it will not be applied in practice.

3. **Influence of geometry of top level on wind load**: For the case study of SCYE010, a set back for the top level showed to result in a significant reduction in wind pressure on the top level. For the case study, the optimal set back is found through an iterative process. This relatively takes a lot of time, because the model needs to be altered and the running of the software can take up to several hours. For future research, it would be interesting to see whether it is possible to make this process more efficient by providing guidelines for the set back, which will most likely be based on the dimensions of the building, but also on the dimensions and shape of the set back itself. To start, it should be investigated if a set back is as beneficial for buildings with other dimensions as it is for the building in this case study. But not only a set back is an option. It might be interesting as well to search for the optimal geometries of the top level in order to reduce (wind) loads on the existing building. In any case, the outcomes of a numerical model, such as the used CFD models, need to be validated.

4. **Guideline for adding levels:** The results of this research could be used to develop guidelines for adding levels. For example in the form of rules of thump to show whether a building would be fit to add levels or how many levels could be added. For example, requirements for a minimum number of levels of the existing building or a certain ratio in building dimensions. Also, the location or building year could be a criterium. For example, buildings in a city are often better suitable than outside a city, due to surroundings. It should be noted that many factors determine whether levels can be added to an existing building, which cannot all be implemented in a few simple rules of thumb. However, providing these guidelines could help engineers in practice massively in the preliminary design phase, when the possibilities of adding levels is investigated. To start, this would require more research on the influence of the dimensions of a building, mainly the width and amount of levels, to determine what is the ideal situation for adding levels.

5. **Improved CFD simulation:** In the research on the pressure coefficients with a CFD model, it was noted that the CFD model resulted in values that are larger than the values from the Eurocode. Therefore, the decision was made to use the outcome mainly quantitatively. It was argued that a possible reason for these high outcome could be the lack of a detailed enough calculation. When the settings for the mesh density were increased or the residual target value was lowered, the laptop that was used instantly gave warnings that such calculations cannot be performed by this laptop. In order to obtain the same quality outcome as with a wind tunnel experiment, one would need to hire cloud servers or possibly an extremely powerful computer. For further research it would be interesting to perform more detailed CFD simulations to obtain better results for the pressure coefficient. Another development that might be interesting for this purpose is the use of Artificial Intelligence (AI). Some CFD software developers already explore the opportunities of implementing AI. This is interesting, because with applying machine learning and using a massive database of wind tunnel data, the CFD simulation can become easy, quick and therefore cheaper (Zampieri, 2022). This is rather futuristic, but nonetheless an noteworthy development in the field that could be investigated further.

6. **Decrease of wind speed**: In this thesis, a short evaluation of the development of the wind speed over the years has been performed. This resulted in the conclusion that the maximum wind speed has decreased over the last 50-70 years. The most used reason for this decrease is the urbanisation of the Netherlands. It would be very interesting to further research the effect of the urbanisation on the wind speed and the comparison wind loads in high density and ‘normal’ density urban locations.

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APPENDIX A DESIGN CODES

A.1 TGB 1955 | Explanation of moderate and high wind loads

In TGB 1955 the code makes a distinction between moderate and high loads. Which type is allowed to be used is depending on the material that is applied. Figure A. 1 presents the article which discusses this. Since the design code is in Dutch, a translation is provided in the section below the figure.

Art. 2 In afwijking van de tot dusverre geldende voorschriften wordt thans onderscheid gemaakt tussen matige en hoge windbelasting. De ervaring heeft namelijk geleerd, dat de windbelasting, berekend met stuwdrukken van 100, 85 en 70 kg/m² **(1, 0,85 en 0,7 kN/m²)**, tot te ongunstige resultaten en dus tot materiaalverspilling leidt. Deze hoge stuwdrukken komen zelden voor; zij werken slechts plaatselijk en op betrekkelijk kleine oppervlakken, zodat de gemiddelde druk over een groot oppervlak steeds kleiner is.

Veiligheidshalve blijft de berekening met bovengenoemde hoge stuwdrukken echter voorgeschreven, doch het wordt dan aanvaardbaar geacht hogere spanningen toe te laten. Als normale belasting wordt naast de hoge windbelasting een matige windbelasting aangenomen, die ongeveer 60% van de bovengenoemde hoge windbelasting bedraagt. Deze matige windbelasting wordt beschouwd als een veel voorkomende, min of meer permanente belasting, die op één lijn kan worden gesteld met het eigen gewicht en de nuttige belasting. Bij matige windbelasting worden derhalve geen hogere spanningen toegelaten. Het hangt af van de constructie en de verhouding van de verschillende belastingen, of de berekening met de hoge, dan wel met de matige windbelasting bepalend is.

Voorlopig worden alleen bij staalconstructies beide berekeningen uitgevoerd. Ten aanzien van hout- en steenconstructies is de commissie van oordeel, dat de daarvoor vastgestelde toelaatbare spanningen, in verhouding tot de eigenschappen van het materiaal, te globaal zijn, om een verfijning, als met bovenstaande rekenmethode wordt verkregen, reden van bestaan te geven. In die gevallen rekent men dus uitsluitend met hoge windbelasting, waarbij echter geen hogere spanningen worden toegelaten.

Figure A. 1 Art. 2 from TGB 1955 (Koninklijk Instituut van Ingenieurs, 1955)

Translation Art 2

As deviation on the previously governing regulations, now the code makes a distinction between moderate and high wind loads. Experience has learned that the wind load, which is calculated with wind pressure of 100, 85 and 70 kg/m^2 (1.00, 0.85 and 0.70 kN/m^2) gives too unfavourable results, and thus, unnecessary material waste. These high wind pressures rarely occurs; it works only locally and on small areas, which results in a lower average pressure over a large surface.

For safety reasons, the calculation with the high wind pressures, which have been discussed above, is mandatory. However, it is presumed acceptable that for those cases higher stresses in the material are allowed. A moderate wind load is used as normal load, next to the high wind load. This moderate load has a value of about 60% of the high wind load. This moderate wind load is assumed to be a common, more or less permanent load, such as self weight and live loads. In the case of moderate wind loads, it is not allowed to assumed high stresses in the material. Depending on the construction and the ratio between the loads, the calculation with high or moderate wind loads is governing.

As for now, only for steel constructions one may perform both calculations. The commission thinks that for timber and stone constructions the allowed stresses in combination with the properties of the materials are too global to allow for a refinement in the calculation as is discussed above. So, in these cases, only high wind load is applied and no higher stresses in the material may be assumed.

A.2 TGB 1972 | Wind pressure table

Tabel 6 Stuwdruk *q*

hoogte *h* boven maai-veld in m	1 aan Noordzeekust in N/m² (kgf/m²)	2 land in N/m² (kgf/m²)
≦ 7	970 (97)	710 (71)
8	990 (99)	730 (73)
9	1 010 (101)	750 (75)
10	1 020 (102)	770 (77)
15	1 070 (107)	830 (83)
20	1 120 (112)	880 (88)
25	1 150 (115)	930 (93)
30	1 190 (119)	970 (97)
35	1 220 (122)	1 010 (101)
40	1 250 (125)	1 040 (104)
45	1 270 (127)	1 070 (107)
50	1 300 (130)	1 100 (110)
55	1 320 (132)	1 120 (112)
60	1 330 (133)	1 140 (114)
65	1 350 (135)	1 160 (116)
70	1 360 (136)	1 180 (118)
75	1 380 (138)	1 200 (120)
80	1 390 (139)	1 220 (122)
85	1 400 (140)	1 230 (123)
90	1 410 (141)	1 250 (125)
95	1 420 (142)	1 260 (126)
100	1 430 (143)	1 280 (128)
110	1 450 (145)	1 300 (130)
120	1 460 (146)	1 320 (132)
130	1 480 (148)	1 340 (134)
140	1 490 (149)	1 360 (136)
150	1 500 (150)	1 380 (138)
160	1 510 (151)	1 400 (140)
170	1 520 (152)	1 410 (141)
180	1 530 (153)	1 430 (143)
190	1 540 (154)	1 440 (144)
200	1 550 (155)	1 450 (145)
250	1 570 (157)	1 510 (151)
300	1 600 (160)	1 560 (156)

Figure A. 2 Table from TGB 1972 with wind pressures (Nederlands Normalisatie Instituut, 1972)

A.3 Terrain categories

Terrain categories according to Eurocode

Tabel 4.1 — Terreincategorieën en terreinparameters

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

(2) De terreinruwheid voor een gegeven windrichting is afhankelijk van de grondruwheid en de afstand met gelijkmatige terreinruwheid in een hoeksector rond de windrichting. Kleine gebieden (minder dan 10 % van het beschouwde gebied) met afwijkende ruwheid mogen zijn verwaarloosd. Zie figuur 4.1.

Figure A. 3 Terrain categories from Eurocode (Nederlands Normalisatie Instituut, 2011b)

Terrain categories according to Eurocode - Dutch National Annex

Tabel NB.3 – 4.1 — Terreincategorieën en terreinparameters

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

Figure A. 4 Terrain categories from Eurocode - Dutch National Annex (Stichting Koninklijk Normalisatie Instituut, 2023)

Terrain categories according to Eurocode - Belgian National Annex

Categorie 0	Zee, directe blootstelling aan zeewinden
Categorie I	Vlakke horizontale gebieden zonder obstakels
Categorie II	Landelijke gebieden met geïsoleerde obstakels
Categorie III	Dorpen, voorsteden, industrie, wouden
Categorie IV	Steden

Figure A. 5 Terrain categories from Eurocode - Belgian National Annex (Bureau de Normalisation, 2010)

Terrain categories according to Eurocode - German National Annex

Table NA.B.1 — Terrain categories

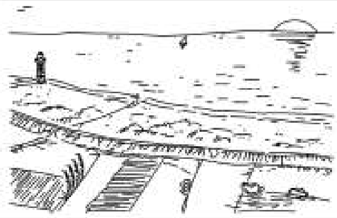
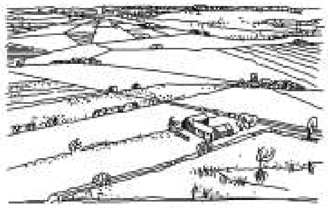
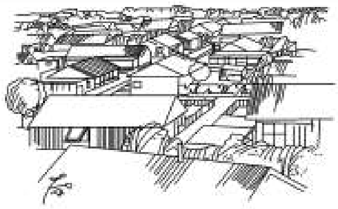
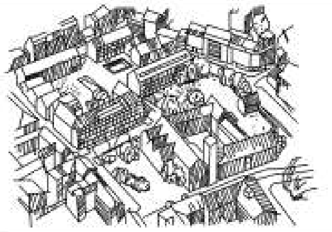
Terrain category I Open sea; lakes with a free surface of at least 5 km in wind direction; level, flat country without obstacles Roughness length $z_0 = 0{,}01$ m Profile exponent $\alpha = 0{,}12$	
Terrain category II Terrain with hedges, isolated farmsteads, houses or trees, e.g. agricultural areas Roughness length $z_0 = 0{,}05$ m Profile exponent $\alpha = 0{,}16$	
Terrain category III Suburbs, industrial or commercial areas; forests/woods Roughness length $z_0 = 0{,}30$ m Profile exponent $\alpha = 0{,}22$	
Terrain category IV Urban areas in which at least 15 % of the surface is covered with buildings of a mean height of more than 15 m Roughness length $z_0 = 1{,}05$ m Profile exponent $\alpha = 0{,}30$	

Figure A. 6 Terrain categories from Eurocode - German National Annex (Deutsches Institut für Normung, 2009)

Terrain categories according to Eurocode - British National Annex

NA.2.11 Procedure for determining the roughness factor $c_r(z)$ [BS EN 1991-1-4:2005, 4.3.2 (1)]

BS EN 1991-1-4:2005 Expressions (4.4) and (4.5) do not apply.

The classification of roughness categories has been simplified to give the following three terrain categories:

- Terrain category 0 is referred to as Sea;
- Terrain categories I and II have been considered together to give a single terrain category referred to as Country terrain;
- Terrain categories III and IV have been considered together to give a single terrain category referred to as Town terrain.

Figure A. 7 Terrain categories from Eurocode - British National Annex (BSI, 2008)

A.4 Formulas TGB 1972

To calculate the wind pressure, the formulas from section 2.2.1 are used. The tables with wind pressure that are provided in most of the design codes are used to verify the calculated results. Most of the design codes have clearly specified values and formulas that can be implemented to obtain the wind pressure, except for some parameters in the TGB 1972. The formula for wind pressure from the TGB 1972 includes two parameters which have to be deduced from a graph; roughness factor r and gust influence factor T_b. Obtaining values from a graph introduces extra error due to limited accuracy when reading from a graph. Formulas which reflect these graphs are formulated for the wind comparison tool in order to automatically calculate the wind pressure according to TGB 1972. However, slight alternations of the graphs are necessary, because the obtained values do not line up with the wind pressure values that are presented in the design code itself.

For example, there is a building with a height of 10 m and the location of the building is 'inland'. Therefore, $v_{u1} \ = 20.5\ m/s$ and $\alpha = 0.19$. Factor g has a fixed value of 4 and from Figure 2.13 can be seen that the r-value is 0.36. Figure A. 8 presents the graph of T_b. The vertical red line is at 10 m, which is the building height and the horizontal red line crosses the graph of Tb at this specific height. This shows that Tb should have a value of 1.88. When applying these values into the formulas of TGB 1972, the following wind pressure is obtained:

$$v_u = v_{u10}\left(\frac{h}{10}\right)^{\alpha} = 20.5 * \left(\frac{10}{10}\right)^{0.19} = 20.5\ m/s$$

$$q_p = \frac{v_u^2}{1.6}\left(1 + gr\sqrt{T_b}\right) = \frac{20.5^2}{1.6}\left(1 + 4 * 0.36\sqrt{1.88}\right) = 0.78\ kN/m^2$$

However, this obtained value does not match with the value provided by TGB 1972, which is 0.76 kN/m^2. Since the roughness factor has a fixed value of 0.36 until at this height, the error can only be on the side of T_b, which makes it possible to calculate the applied value of T_b in order to obtain a wind pressure of 0.76 kN/m^2.

$$q_p = \frac{v_u^2}{1.6}\left(1 + gr\sqrt{T_b}\right) \rightarrow T_b = \left(\left(\frac{q_p * 1.6}{v_u^2} - 1\right) * gr\right)^2$$

$$= \left(\left(\frac{0.76 * 1.6}{20.5^2} - 1\right) * 4 * 0.36\right)^2 = 1.76$$

This back-calculated value of T_b=1.76 is shown by the red-dotted line in Figure A. 8 and it shows that this is relatively far off from the value at a building height of 10 m.

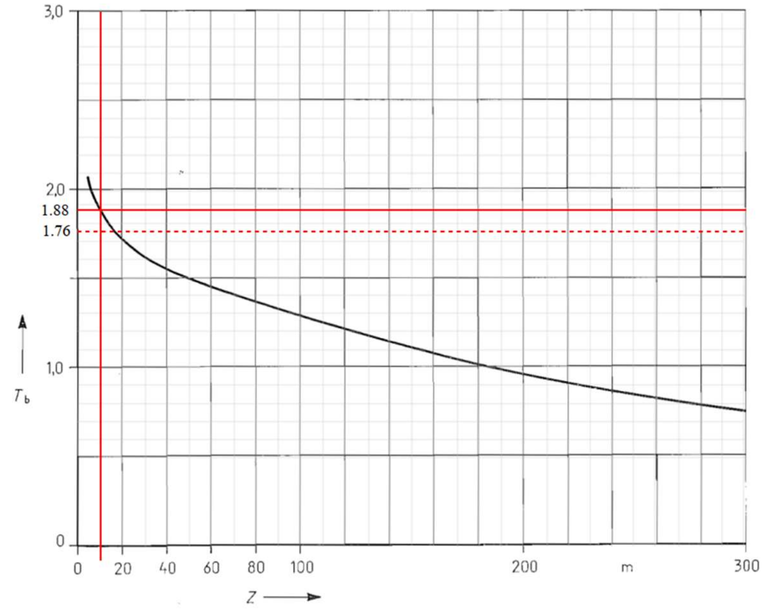


Figure A. 8 Gust influence factor Tb (Nederlands Normalisatie Instituut, 1972)

The formulas for the roughness factor and gust influence factor are iteratively determined to obtain graphs that result in the same graphs as provided in the design code and to match the values of the wind pressures that are prescribed. The formulas that are applied in the wind comparison tool are provided below.

$$for\ z \leq 10\ m \rightarrow r_{inland}(z) = 0.36$$
$$for\ z > 10\ m \rightarrow r_{inland}(z) = 0.35\left(\frac{10}{z}\right)^{0.195} \qquad (2.\ 37)$$

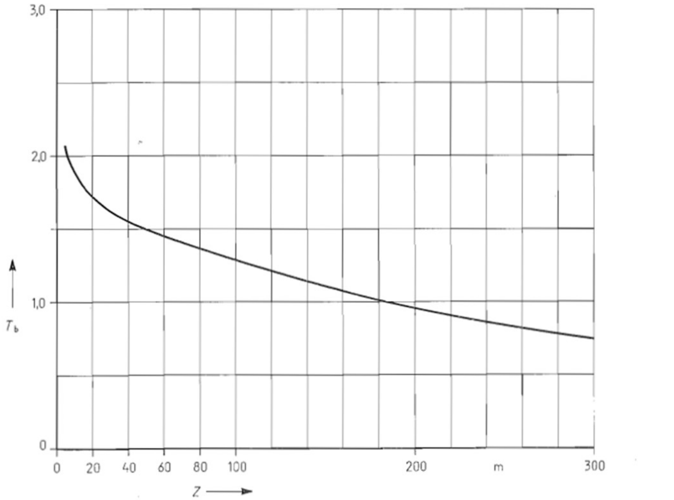
$$for\ z \leq 10\ m \rightarrow r_{coastline}(z) = 0.28$$
$$for\ z > 10\ m \rightarrow r_{coastline}(z) = 0.256\left(\frac{10}{z}\right)^{0.139} \qquad (2.\ 38)$$

$$for\ z \leq 60\ m \rightarrow T_b(z) = 1.86\left(\frac{10}{z}\right)^{0.137} \qquad (2.\ 39)$$

$$for\ z > 60\ m \rightarrow T_b(z) = \frac{1.9}{1 + 4.8 * 10^{-3} * z} \qquad (2.\ 40)$$

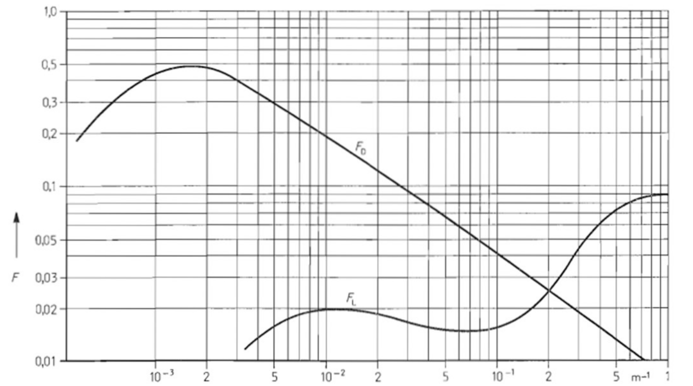
APPENDIX B DYNAMIC FORMULAS & PARAMETERS

	TGB 1972		TGB 1990		Eurocode	
Parallel to wind direction	$\varphi_1 = \frac{1 + 4r\sqrt{T_b + T_r}}{1 + 4r\sqrt{T_b}}$		$\phi_1 = \frac{1 + 7\,I(h)\,\sqrt{B + E}}{1 + 7\,I(h)\,\sqrt{B}}$		$c_d = \frac{1 + 2k_p * I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7I_v(z_s)\sqrt{B^2}}$	
	r	See Figure 2.13	E	$= \frac{0{,}0394 f_e^{-\frac{2}{3}}}{D(1 + 0{,}10 f_e h)(1 + 0{,}16 f_e b)}$	R^2	$= \frac{\pi^2}{2\delta} * S_{L(z_s, n_{1,x})} * R_h(\eta_h) * R_b(\eta_b)$
	T_b	See Figure B. 1				
	T_r	$= \frac{F_D S}{D}$	B	$= \frac{1}{0{,}94 + 0{,}021 h^{\frac{2}{3}} + 0{,}029 b^{\frac{2}{3}}}$	B^2	$= \frac{1}{1 + \frac{3}{2}\sqrt{\left(\frac{b}{L(z_s)}\right)^2 + \left(\frac{h}{L(z_s)}\right)^2 + \left(\frac{b}{L(z_s)}\frac{h}{L(z_s)}\right)^2}}$
	D	$= 0.05\ (timber)$				
		$= 0.03\ (masonry)$				
		$= 0.02\ (concrete)$	D	$= 0.03\ (timber)$		
		$= 0.01\ (steel)$		$= 0.02\ (concrete)$	$L(z_s)$	$= 300\left(\frac{z_s}{200}\right)^{\alpha}$
	F_D	See Figure B. 2		$= 0.01\ (steel)$		
	S	See Figure B. 3	$I(h)$	$= \frac{1}{\ln\left(\frac{h}{0{,}2}\right)}$	α	$= 0.67 + 0.05\ln(z_0)$
	f_e	$= \sqrt{\frac{0.25}{\delta}}$			h	$= Height\ building$
					b	$= Width\ building$
					k_p	See Figure B. 4
					f_e	$= 46/h$
Perpendicular to wind direction	$\varphi_2 = \frac{4\sqrt{T_r'}}{1 + 4r\sqrt{T_b}}$		$\varphi_2 = \frac{E_1}{\sqrt{B + E_1}}$			
	T_r'	$= \frac{F_L S}{D}$	E_1	$= \frac{0{,}0344 f_e^{-\frac{2}{3}}}{D * (1 + 0{,}12 f_e h) * (1 + 0{,}2 f_e b)}$		
			f_e	$= \sqrt{\frac{a}{\delta}} = \sqrt{\frac{0.384}{\delta}}$		



De vlaaginvloed (T_b) als functie van de hoogte (Z) van het door wind getroffen oppervlak van de constructie.
Fig. 4

Figure B. 1 Gust influence Tb (Nederlands Normalisatie Instituut, 1972)



Coëfficiënten F_L en F_D

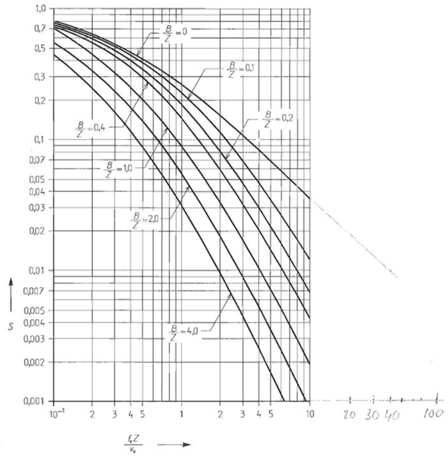
f_e = eigen frequentie van de constructie in perioden per seconde

$v_u = 1{,}25 \sqrt{\frac{q}{1 + 4r\sqrt{T_b}}}$ m/s

q = stuwdruk in N/m² volgens tabel 6 ter plaatse van het hoogste punt van de constructie.

Fig. 5

Figure B. 2 F_L and F_D (Nederlands Normalisatie Instituut, 1972)

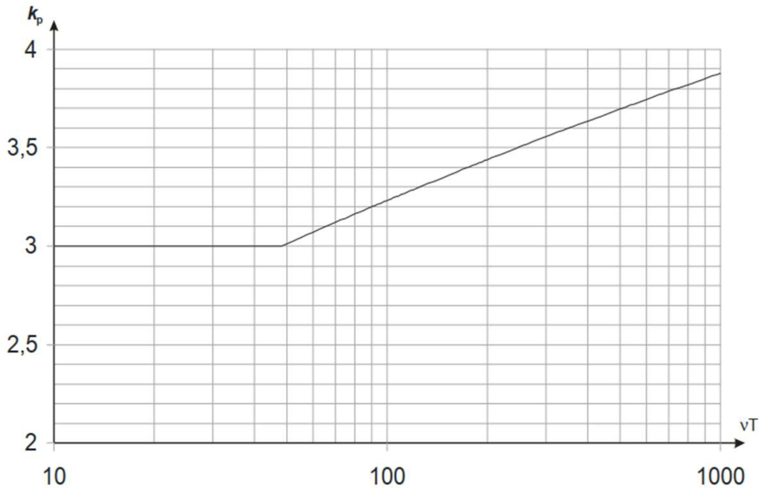


Coëfficient S

Z = hoogte van het door de wind getroffen oppervlak van de constructie in m.
B = gemiddelde dwarsafmeting van de constructie ⊥ de windrichting, in m.
f_e en v_u = zie fig. 5.

Fig. 6

Figure B. 3 S (Nederlands Normalisatie Instituut, 1972)



Figuur B.2 — Piekfactor

Figure B. 4 Peak factor kp (Nederlands Normalisatie Instituut, 2011b)

APPENDIX C OVERVIEW WIND PRESSURES FROM DESIGN CODES

C.1 Overview wind pressures from design codes

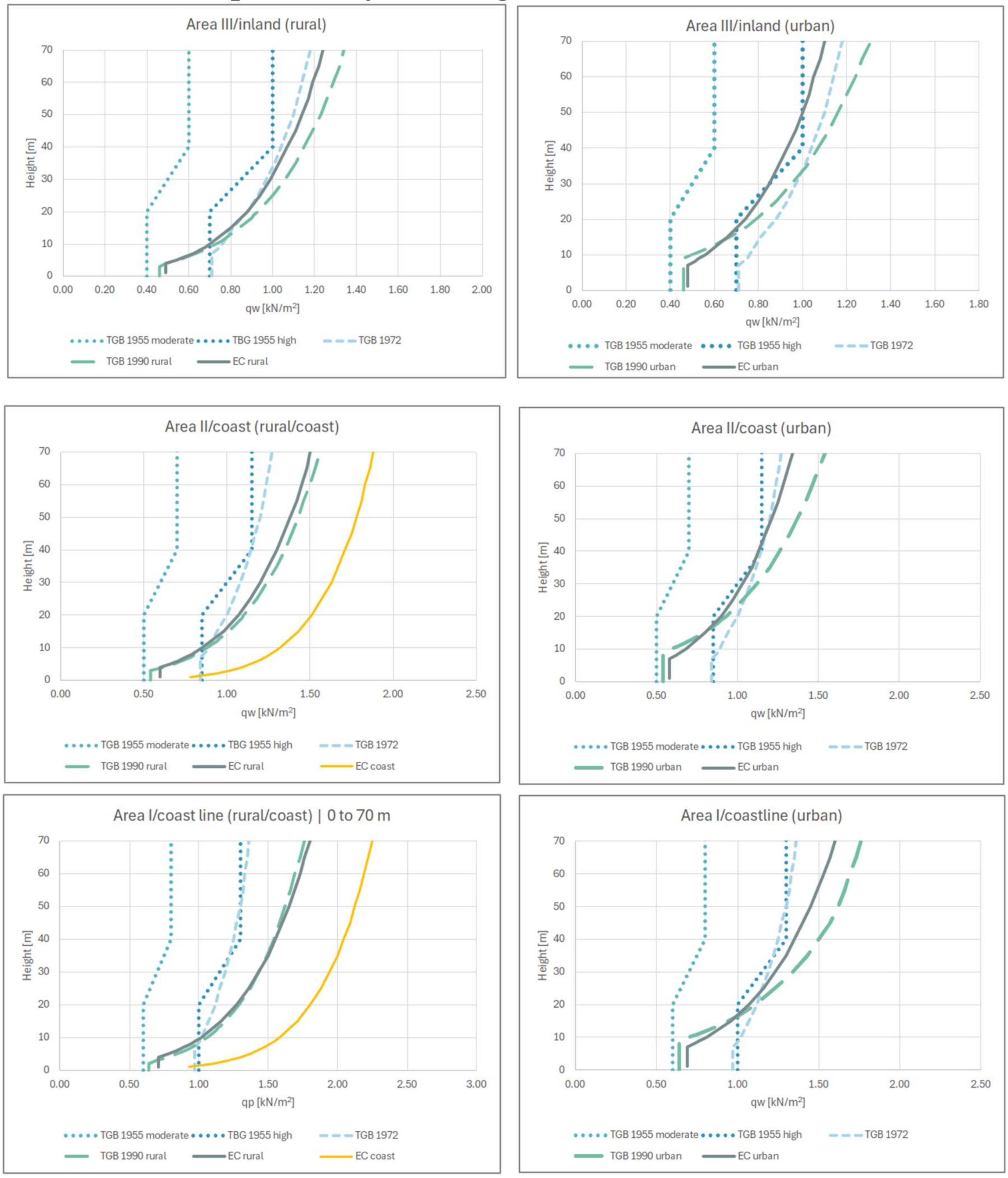


Figure C. 1 Overview of wind pressures from all design codes in all wind areas

C.2 Highlighted area from Figure 2.23

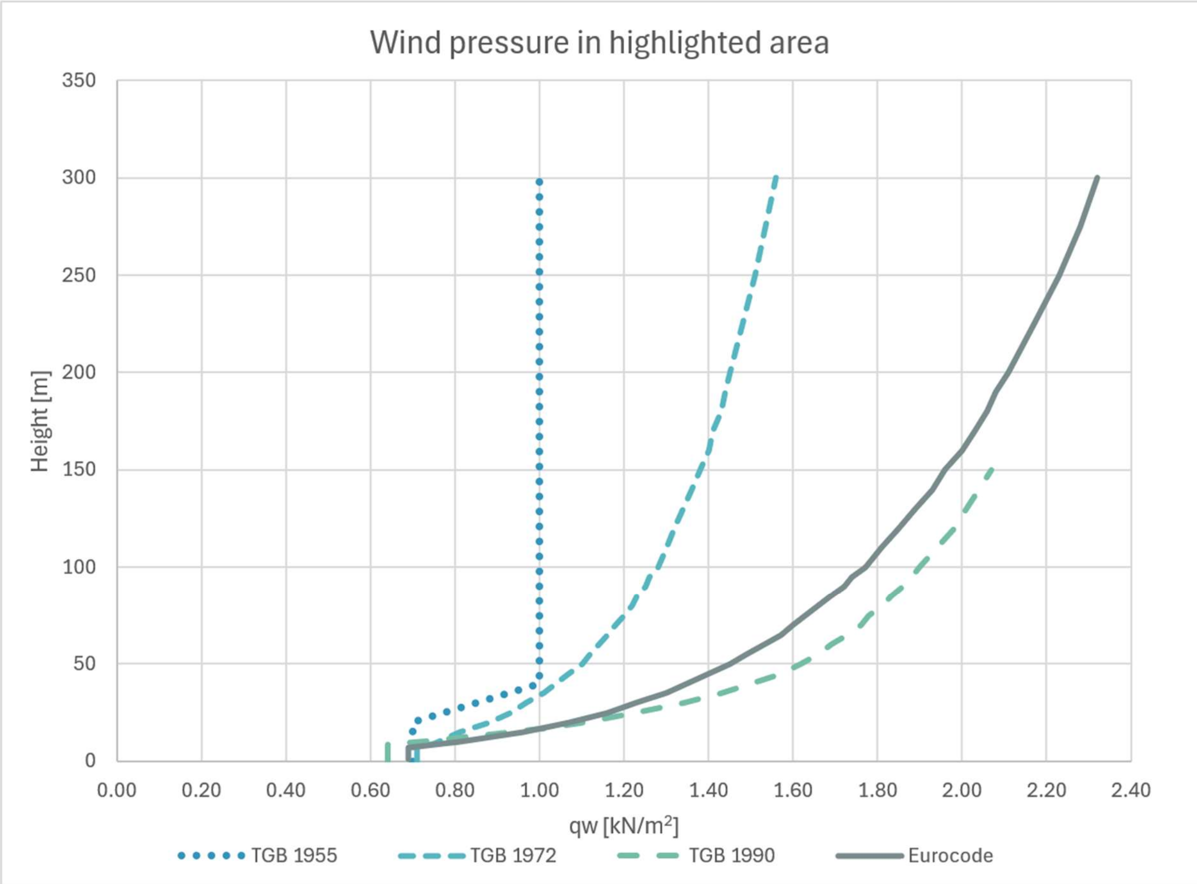
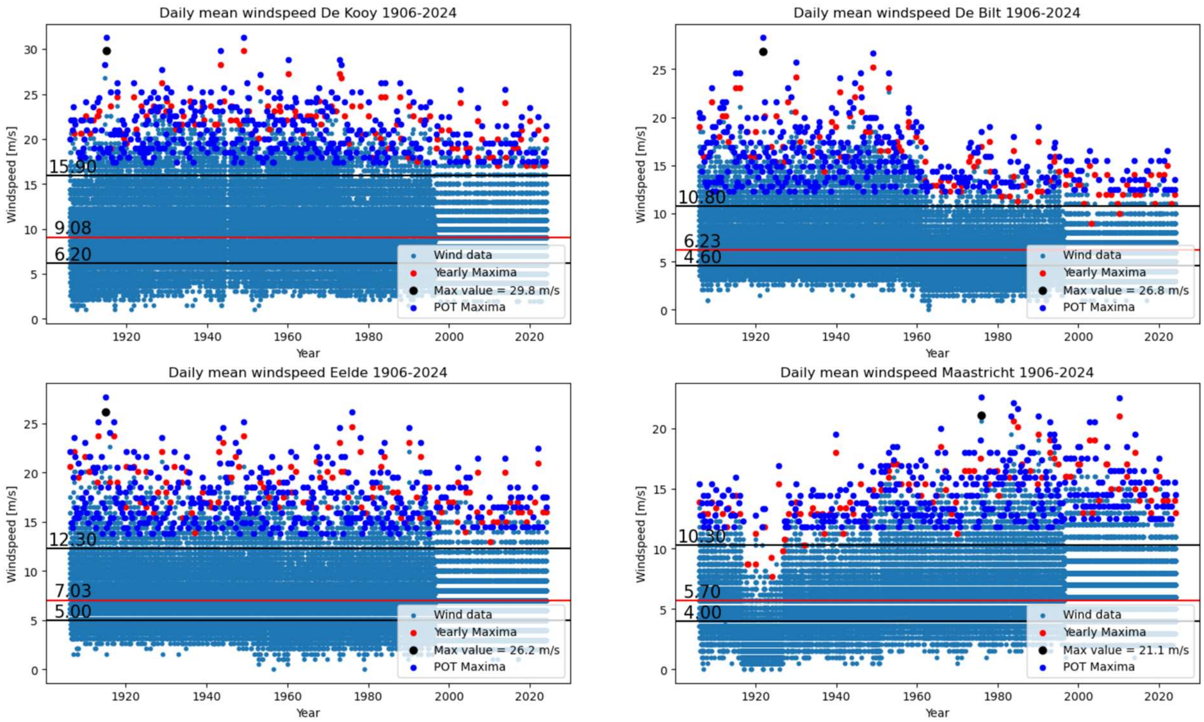


Figure C. 2 Wind pressure in the highlighted area from Figure 2.23

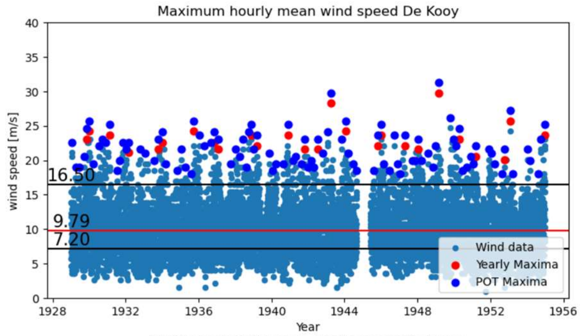
APPENDIX D KNMI DATA

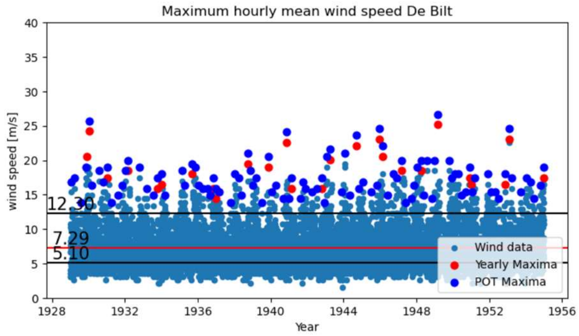
D.1 Wind speed data oldest weather stations

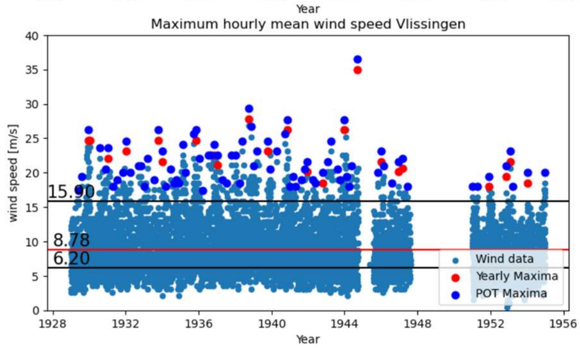


D.2 TGB 1955

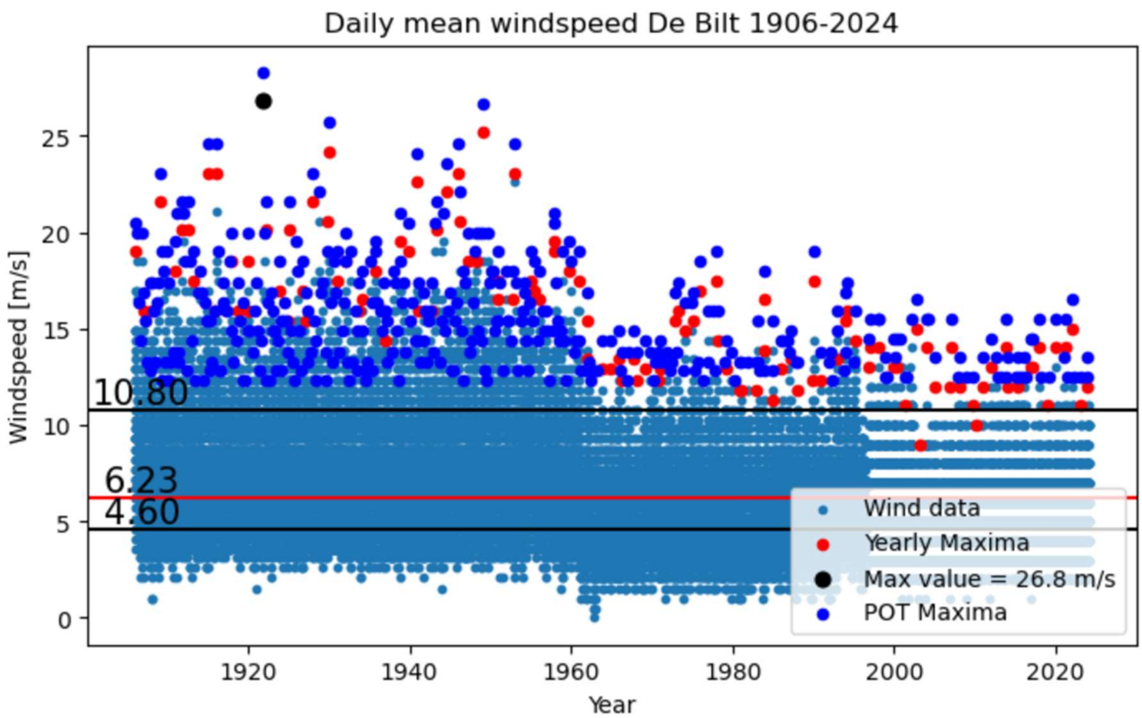
Complete wind data set



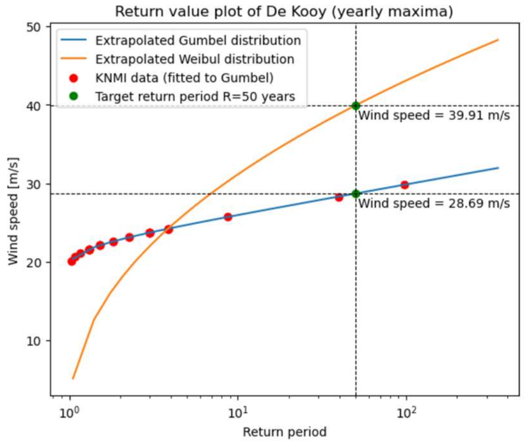


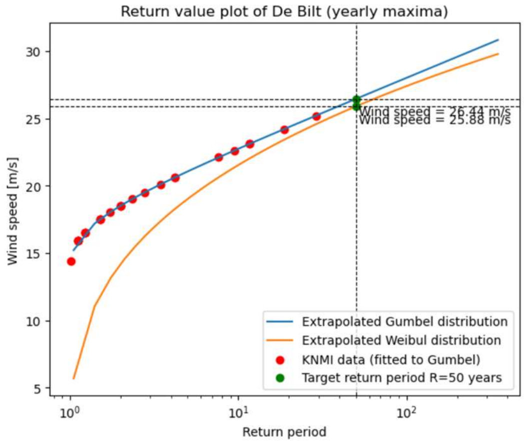


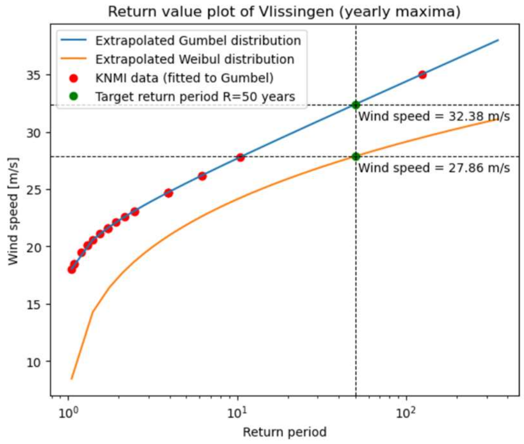
Complete data set at De Bilt with maximum value in lifetime



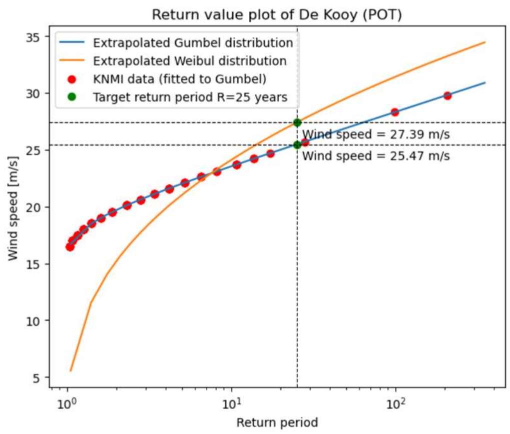
Return value plots of the yearly maxima

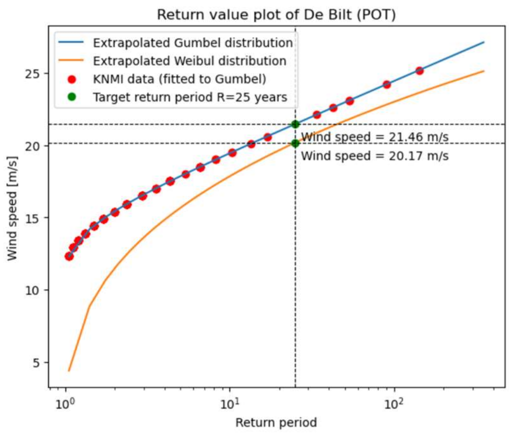


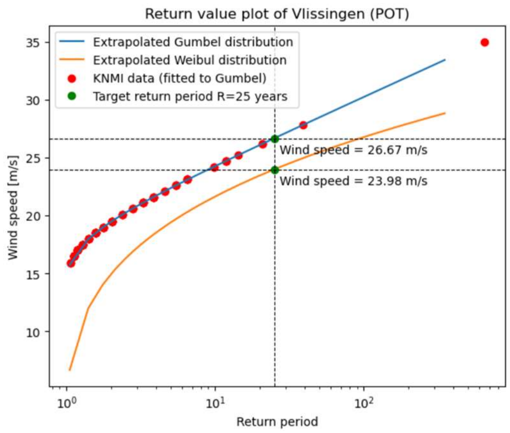




Return value plots of the POT values

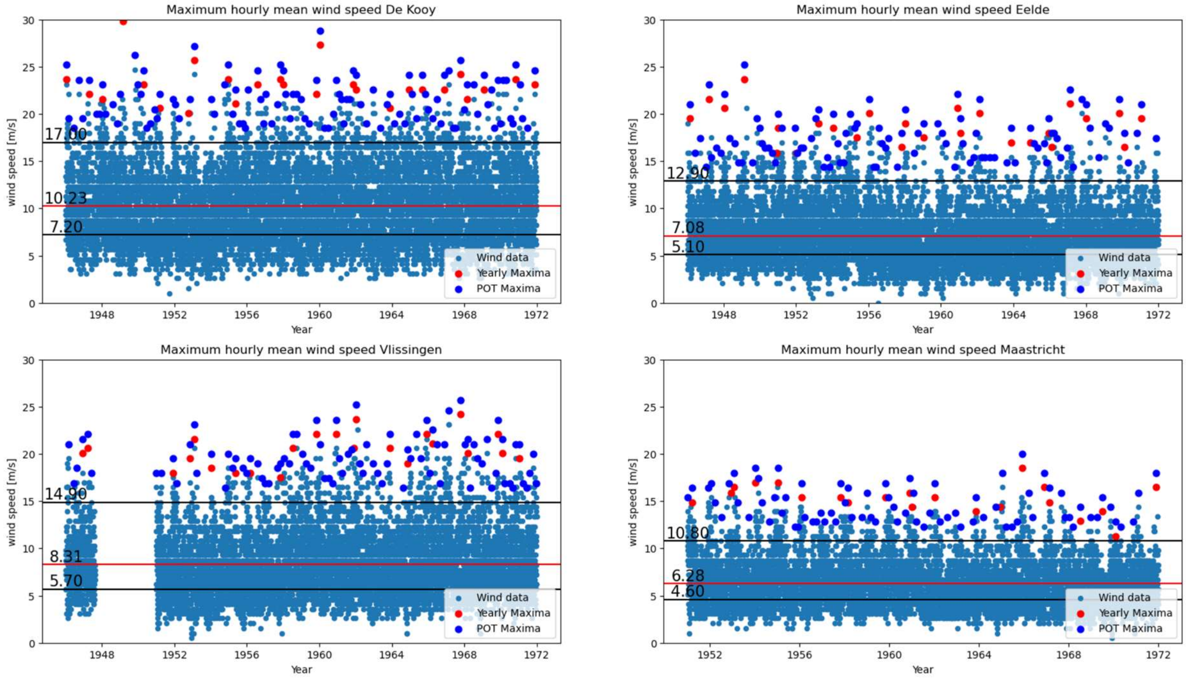




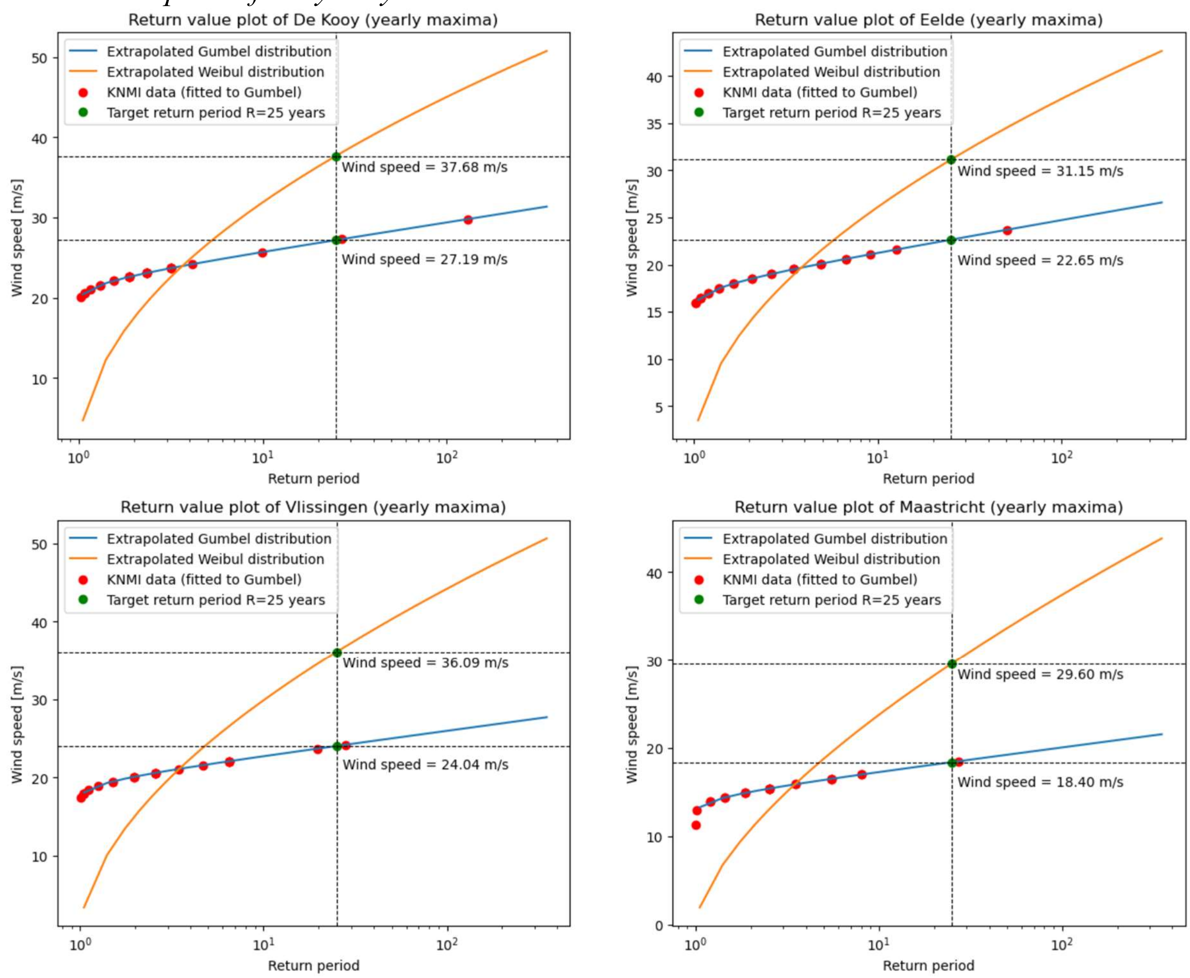


D.3 TGB 1972

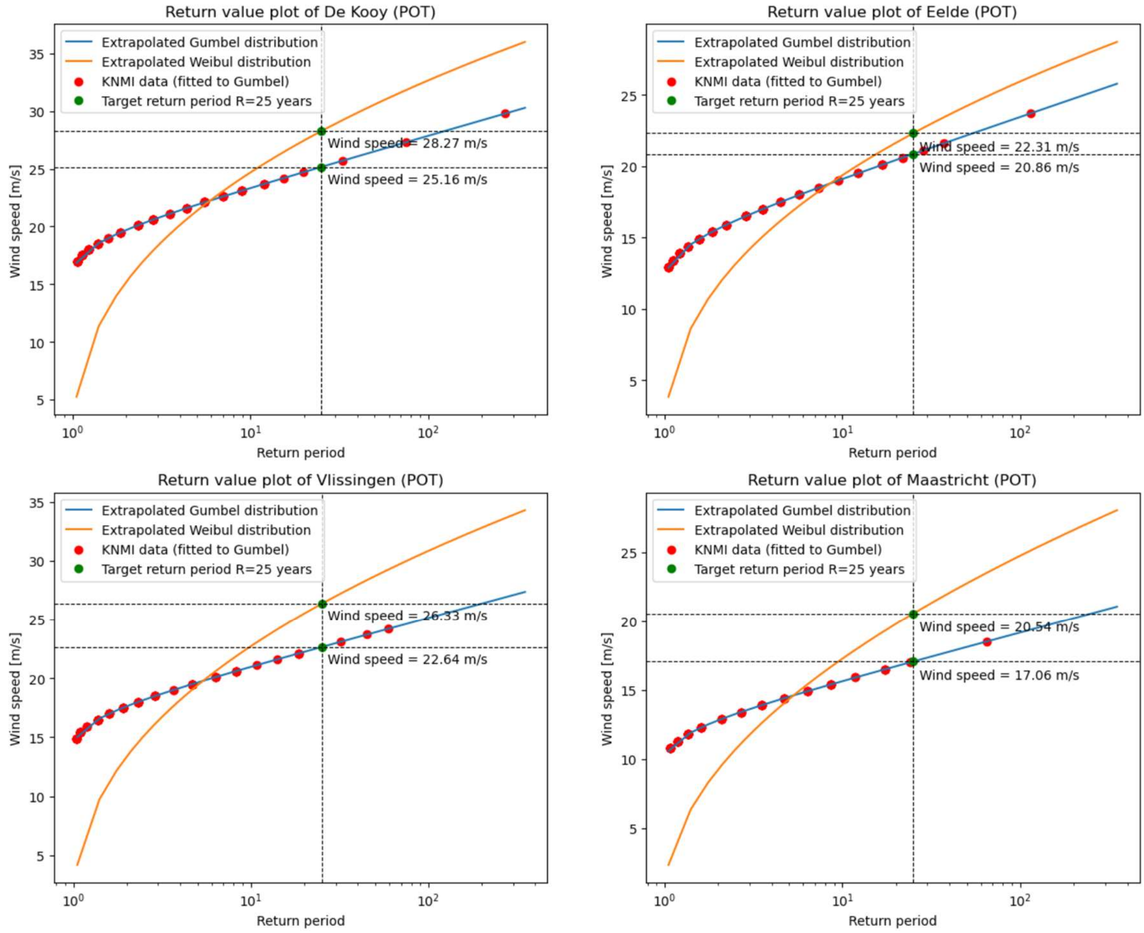
Complete wind data set



Return value plots of the yearly maxima



Return value plots of the POT values



D.4 TGB 1990

Table with r-factors to calculate back to a return period of 12.5 years. These work for both rural and urban terrain categories.

TNO-Bouw B-90-483 Jan. 1992 Pagina 122

De factor r waarmee de locale stuwdruk volgens tabel 5.1 de globale stuwdruk volgens tabel 6.1 moet worden vermenigvuldigd voor andere herhalingstijden dan 12,5 jaar is vermeld in de volgende tabel.

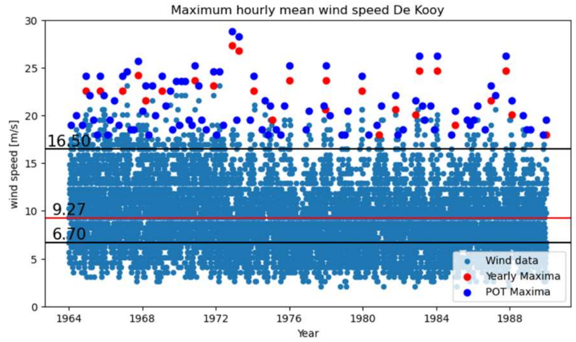
tabel B-2: Factoren voor herhalingstijden anders dan 12,5 jaar.

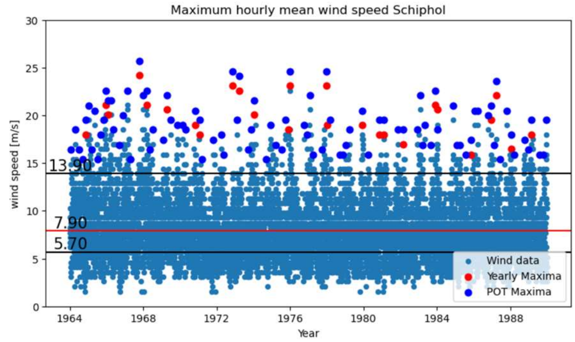
herhalingstijd	factor r gebied I	factor r gebied II	factor r gebied III
1 jaar	0,69	0,67	0,64
5 jaar	0,88	0,87	0,86
10 jaar	0,97	0,97	0,97
12.5 jaar	1	1	1
15 jaar	1,02	1,03	1,03
20 jaar	1,06	1,07	1,08
25 jaar	1,10	1,10	1,11
30 jaar	1,12	1,13	1,15
35 jaar	1,14	1,16	1,17
40 jaar	1,16	1,18	1,20
45 jaar	1,18	1,20	1,22
50 jaar	1,20	1,21	1,23

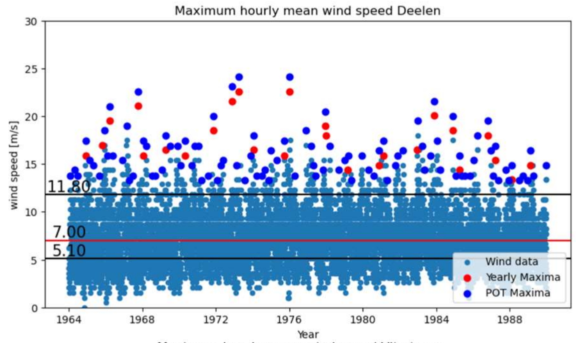
Opgemerkt wordt dat de factoren uit tabel B-2 gelden voor zowel onbebouwd als voor bebouwd/bebost gebied.

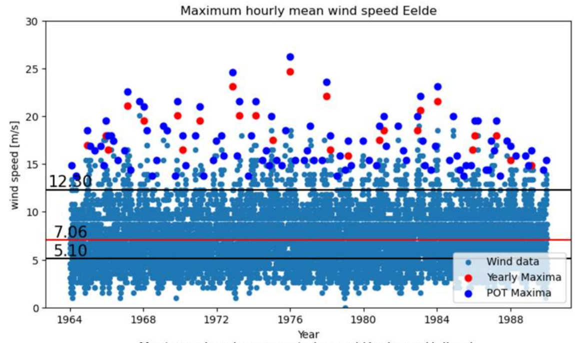
Figure D. 1 Table with r-factors (Van Staalduinen, 1992)

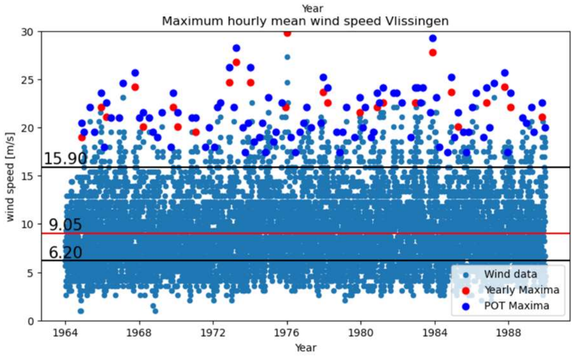
Complete wind data set

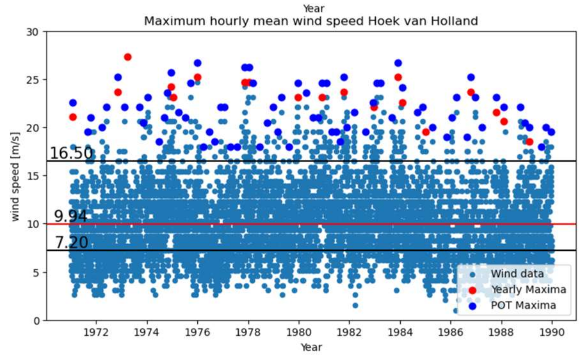


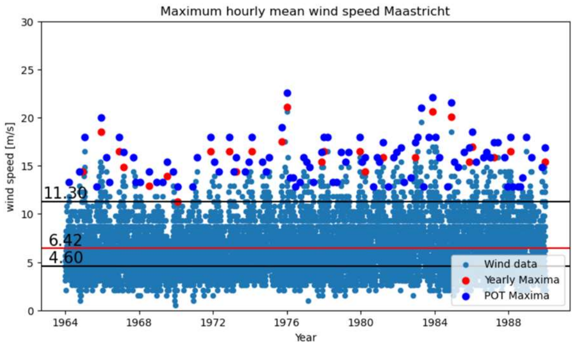




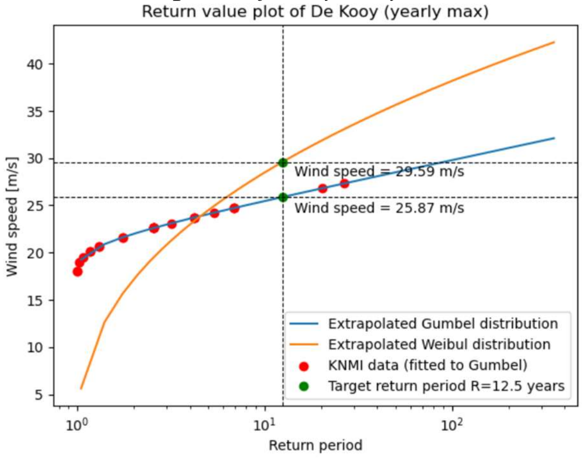


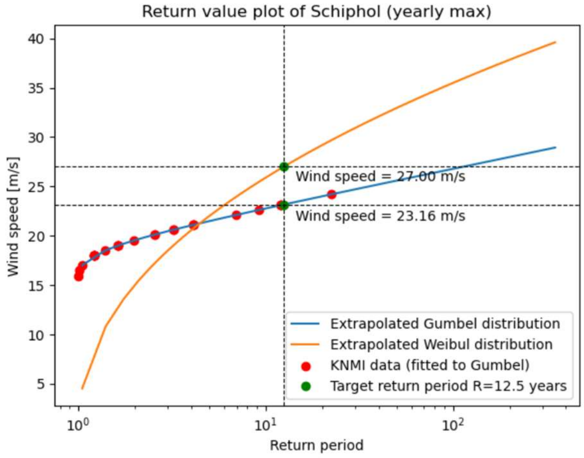


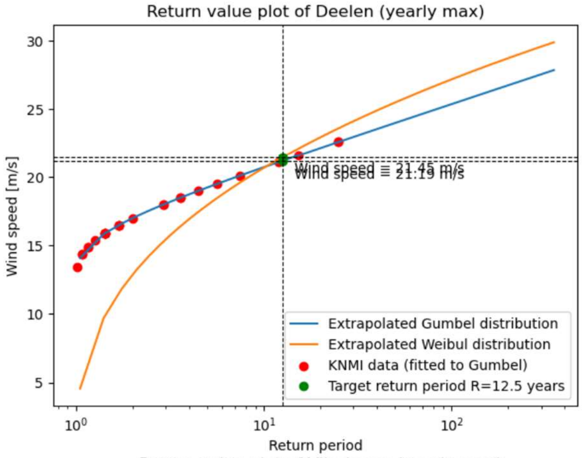


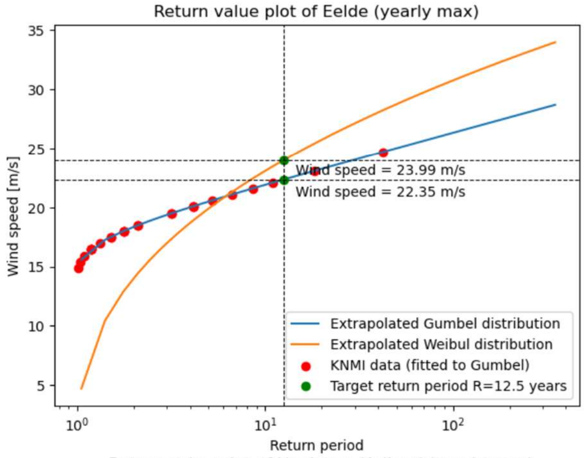


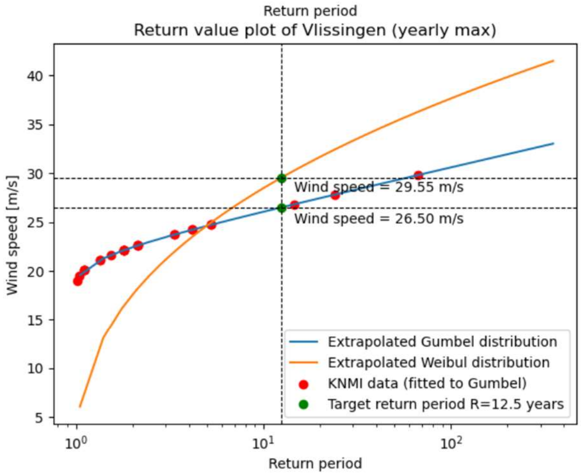
Return value plots of the yearly maxima

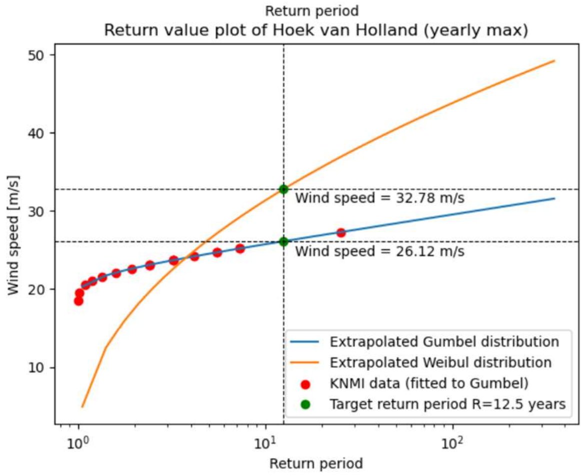


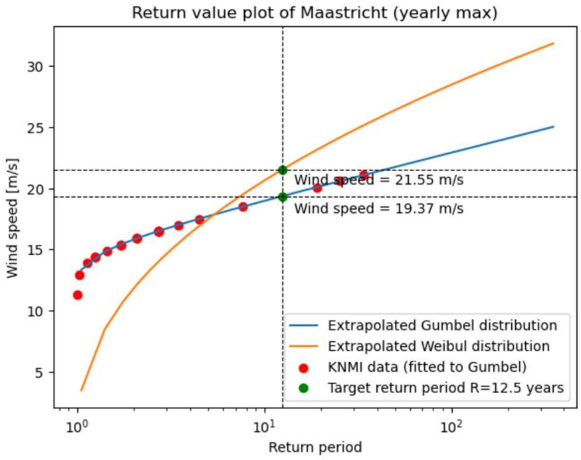












Friction speed calculation

	Wind area	$u_p(10)$	$u_{*p}(10)$	$u(60)$	$u_r(10)$	u_{*r}
TGB 1990	I	27.5	1.89	36.0	25.9	**2.25**
TGB 1990	II	25	1.72	32.7	22.4	**2.29**
TGB 1990	III	22.5	1.55	29.4	19.5	**2.22**
De Kooy	I	25.9	1.79	33.9	24.4	**2.12**
Schiphol	II	24.1	1.66	31.5	21.6	**2.21**
Deelen	III	21.5	1.48	28.1	18.6	**2.12**
Eelde	III	22.8	1.57	29.8	19.7	**2.01**
Vlissingen	II	27.0	1.86	35.3	24.2	**2.48**
Hoek van H.	II	26.5	1.82	34.6	23.7	**2.43**
Maastricht	III	19.7	1.35	25.7	17.0	**1.94**

$$u_p(10) = measured\ wind\ speed$$

$$u(60) = \frac{ln\left(\frac{60-d}{z_o}\right)}{ln\left(\frac{10-d}{z_o}\right)} * u_p(10) = 1.308 * u_p(10)$$

$$z_o = 0.03m$$
$$d = 0m$$

$$u(z) = \frac{ln\left(\frac{z-d}{z_{or}}\right)}{ln\left(\frac{60}{z_{or}}\right)} * u(60)$$

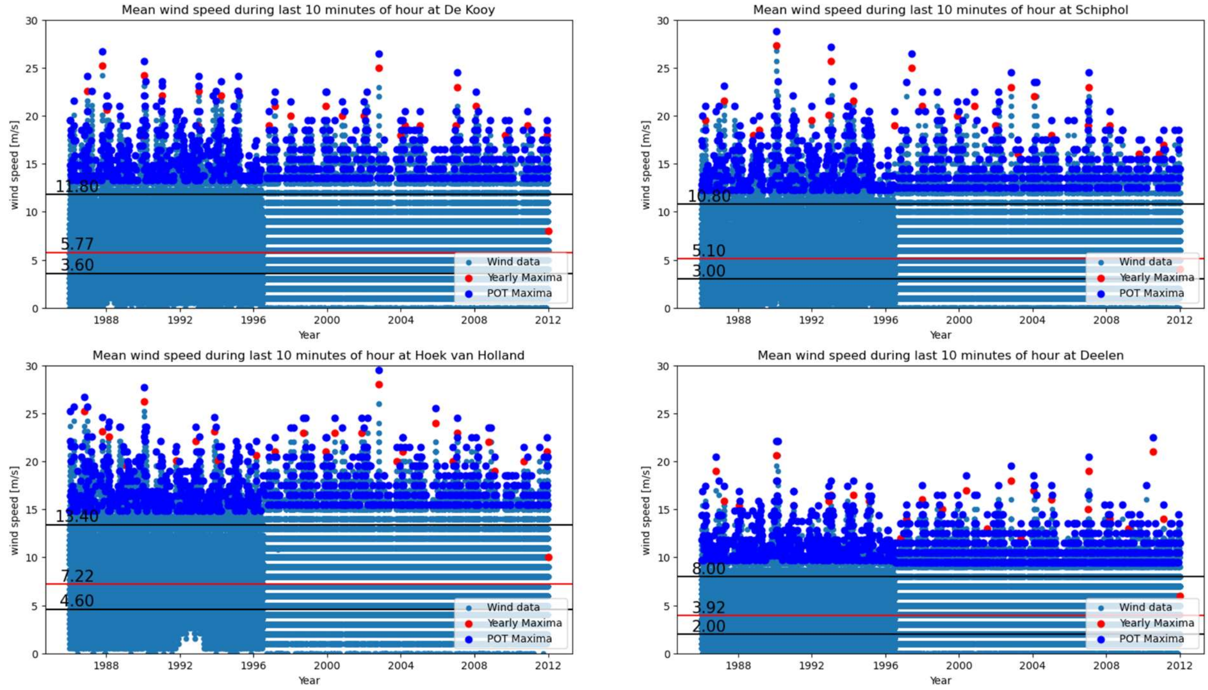
Wind area	z_{or}	d
I	0.1	0
II	0.2	0
III	0.3	0

$$U_z = \frac{u_*}{k} * \ln\left(\frac{z}{z_o}\right) \rightarrow u_{*r} = U_z * \frac{k}{\ln\left(\frac{z}{z_o}\right)},$$

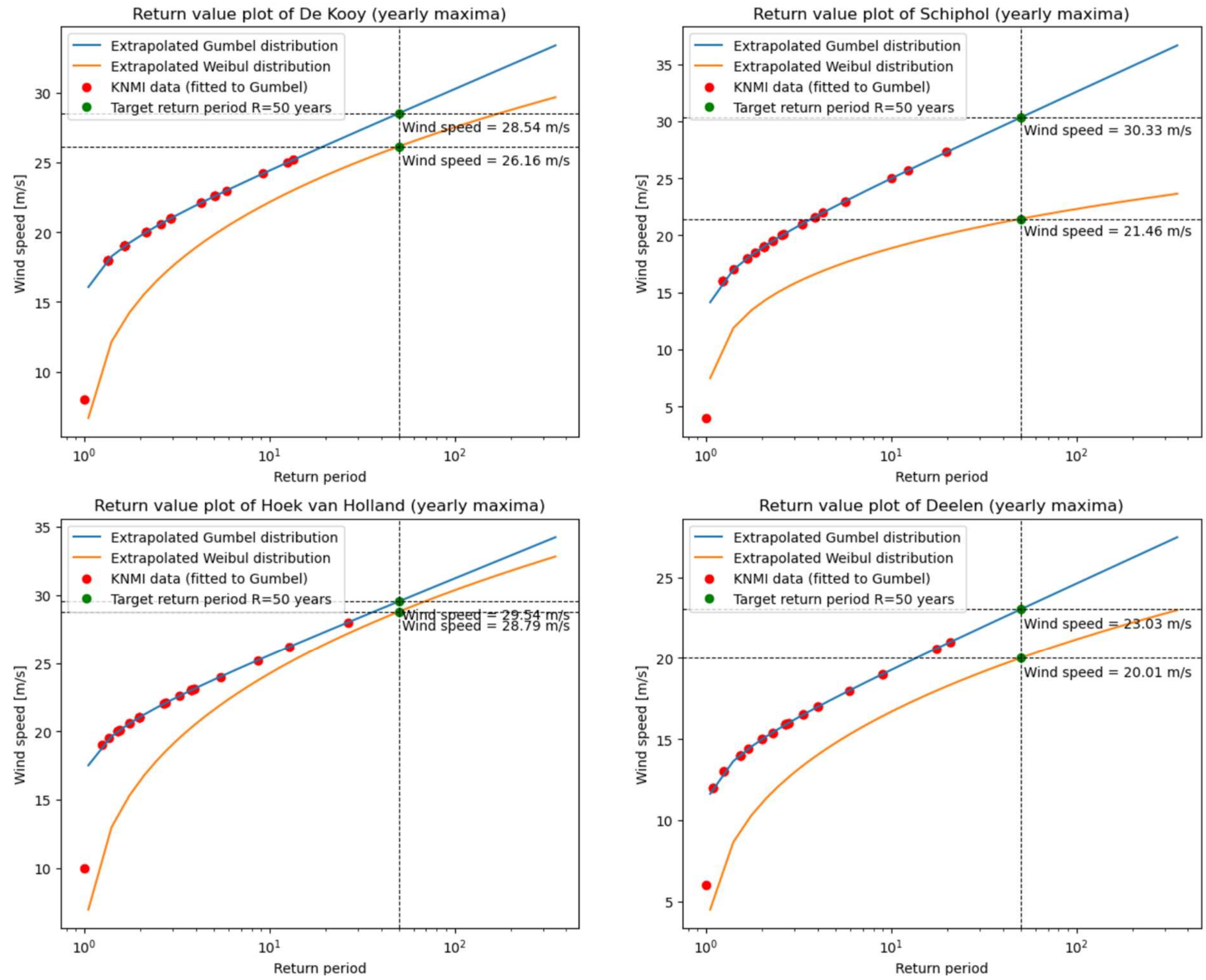
$$k = 0.4$$

D.5 Eurocode

Complete wind data set– 10-min data



Return value plots of the yearly maxima



APPENDIX E PRESSURE COEFFICIENTS

E.1 Variant 1

Variant 1 | Building height 30 m

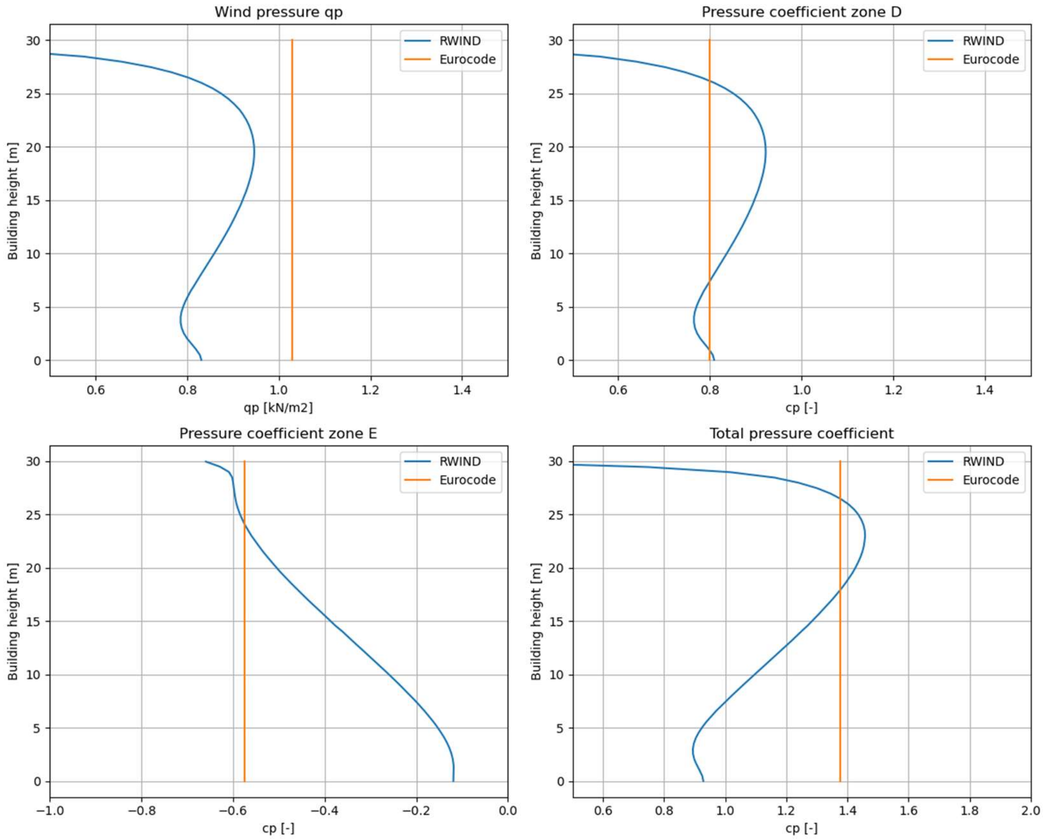


Figure E. 1 Overview RWIND output and Eurocode values | variant 1 - 30 m

Variant 1 | Building height 50 m

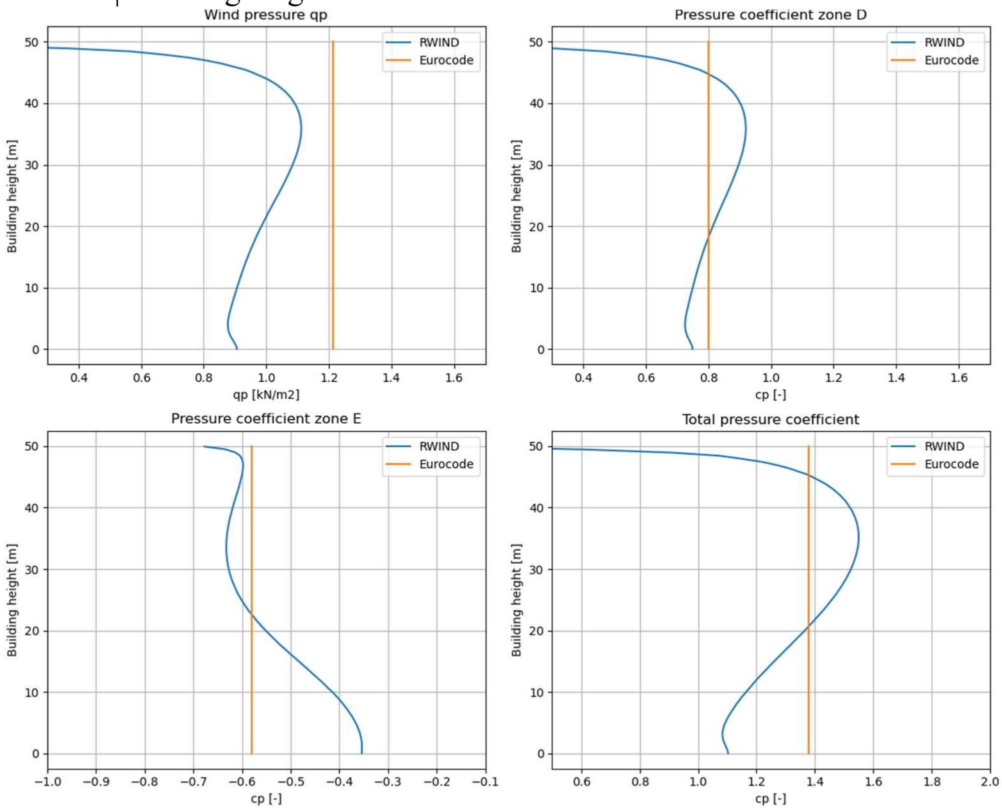


Figure E. 2 Overview RWIND output and Eurocode values | variant 1 - 50 m

Variant 1 | all heights

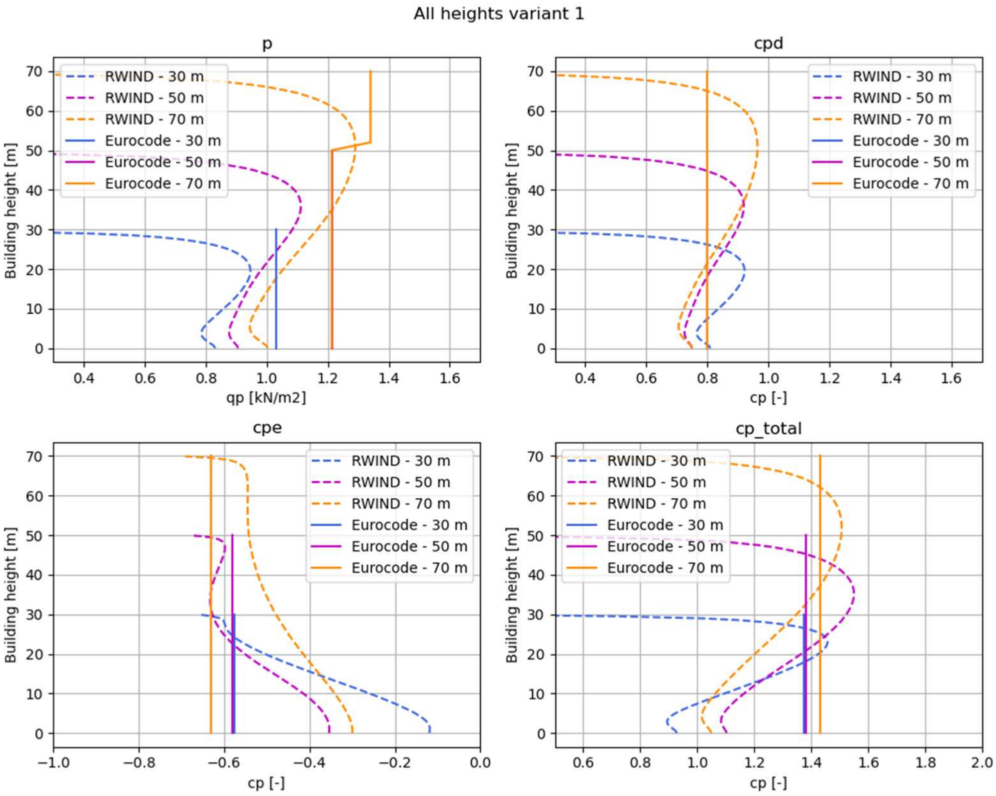


Figure E. 3 Overview RWIND output and Eurocode values | variant 1 - all heights

E.2 Variant 2

Variant 2 | all heights

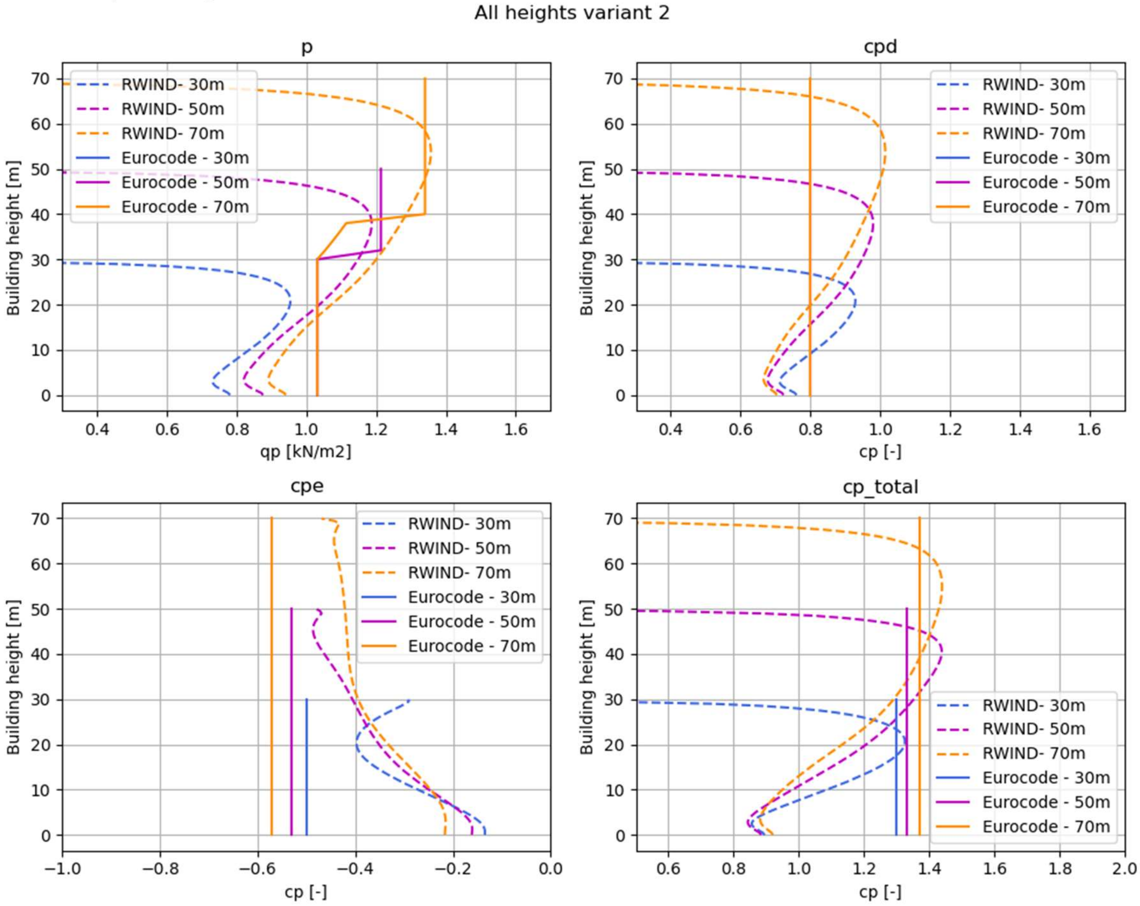


Figure E. 4 Overview RWIND output and Eurocode values | variant 2 - all heights

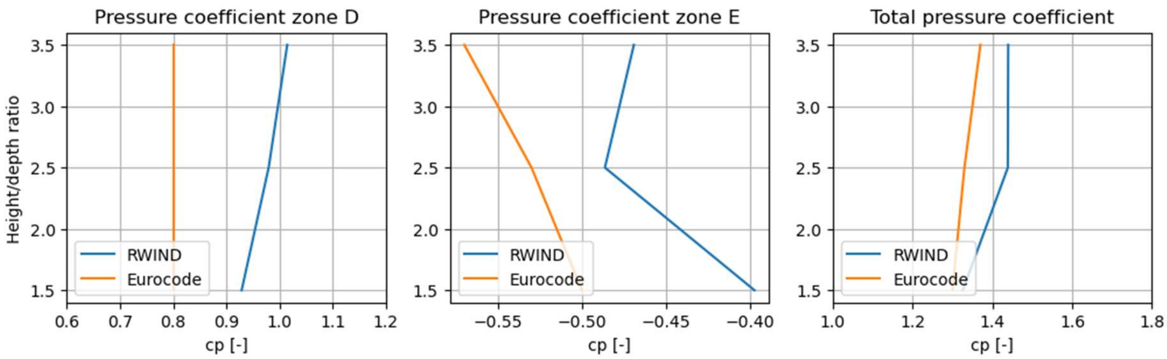


Figure E. 5 The h/d ratio versus the pressure coefficient

E.3 Comparison wind tunnel tests

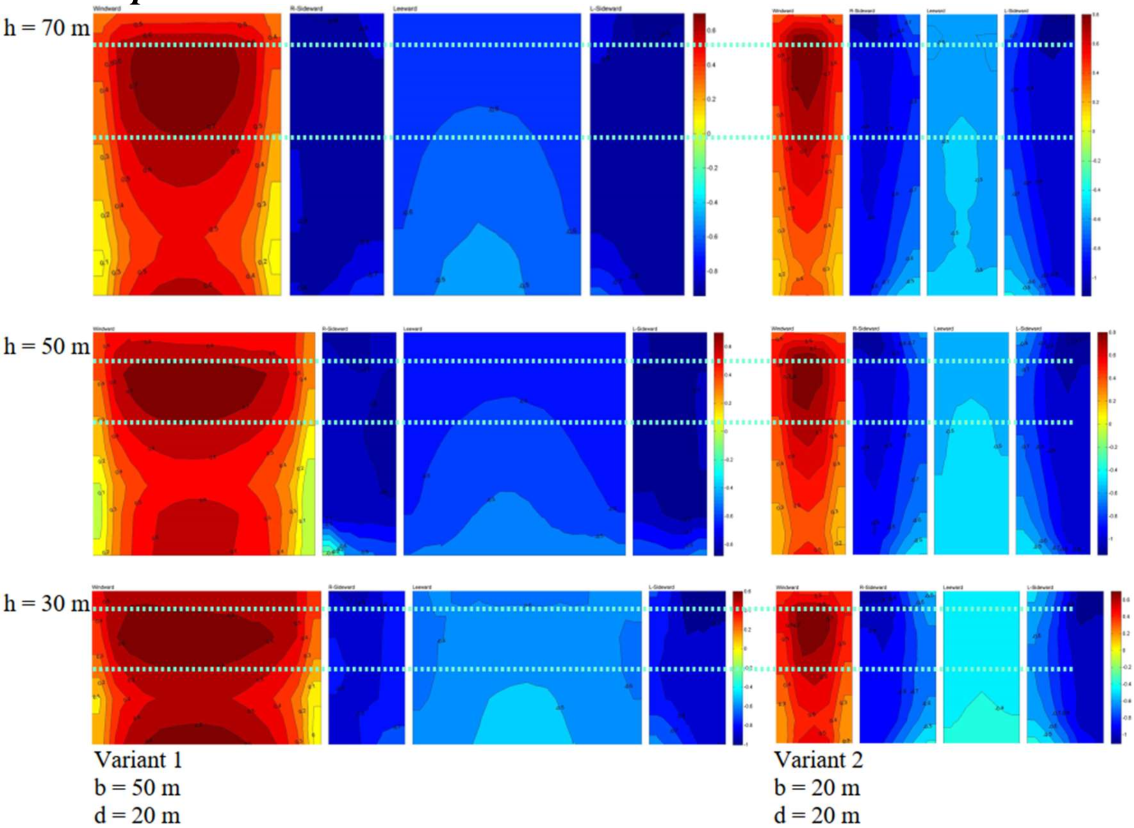


Figure E. 6 Different wind tunnel tests representing the variants 1 and 2

E.4 Variant 3

Variant 3 | all radii

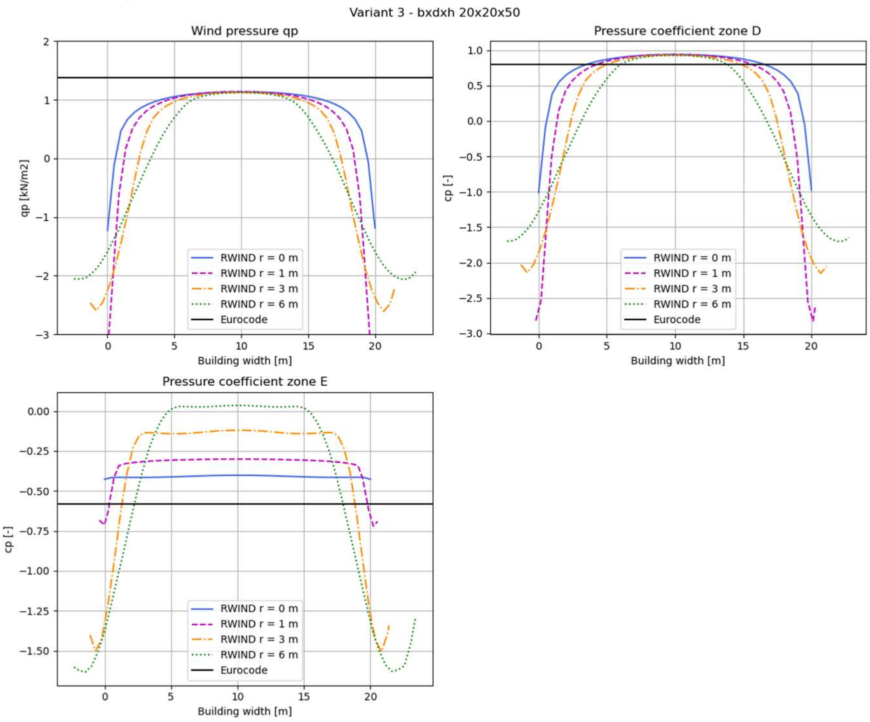


Figure E. 7 Overview RWIND output and Eurocode values | variant 1 - all radii

APPENDIX F PARTIAL SAFETY FACTORS

F.1 Material factors versus partial safety factors

Material factors γ_m

Design code	Steel	Concrete	Timber
TGB 1955	1.5	1.8	
TGB 1972	1.5	1.7	
TGB 1990	1.0	1.2	1.2
Eurocode	1.0	1.5	1.2

Formula

$\gamma_{total} = \gamma_m * \gamma_{f,q}$

Partial safety factors TGB 1990 & Eurocode

	CC1	CC2	CC3
$\gamma_{f,g}$	1.2	1.2	1.2
$\gamma_{f,q}$	1.2	1.3	1.5

Combinations

Steel | CC1	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955	1.5		1.5
TGB 1972	1.5		1.5
TGB 1990	1.0	1.2	1.2
Eurocode	1.0	1.2	1.2

Steel | CC2	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955	1.5		1.5
TGB 1972	1.5		1.5
TGB 1990	1.0	1.3	1.3
Eurocode	1.0	1.3	1.3

Steel | CC3	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955	1.5		1.5
TGB 1972	1.5		1.5
TGB 1990	1.0	1.5	1.5
Eurocode	1.0	1.5	1.5

Concrete | CC1	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955	1.8		1.8
TGB 1972	1.7		1.7
TGB 1990	1.2	1.2	1.4
Eurocode	1.5	1.2	1.8

Concrete | CC2	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955	1.8		1.8
TGB 1972	1.7		1.7
TGB 1990	1.2	1.3	1.6
Eurocode	1.5	1.3	2.0

Concrete | CC3	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955	1.8		1.8
TGB 1972	1.7		1.7
TGB 1990	1.2	1.5	1.8
Eurocode	1.5	1.5	2.3

Timber | CC1	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955			
TGB 1972			
TGB 1990	1.2	1.2	1.4
Eurocode	1.2	1.2	1.4

Timber | CC2	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955			
TGB 1972			
TGB 1990	1.2	1.3	1.6
Eurocode	1.2	1.3	1.6

Timber | CC3	γ_m	$\gamma_{f,q}$	γ_{total}
TGB 1955			
TGB 1972			
TGB 1990	1.2	1.5	1.8
Eurocode	1.2	1.5	1.8

F.2 Partial safety factors: Eurocode versus NEN 8700

Table E. 1 Partial safety factors Eurocode/NEN-EN 1990 (Nederlands Normalisatie Instituut, 2019)

NEN-EN 1990	**Permanent load**	**Governing load (not wind)**	**Governing wind load**
Combination 6.10a			
CC1	1.20	1.35	1.35
CC2	1.35	1.50	1.50
CC3	1.50	1.65	1.65
Combination 6.10b			
CC1	1.10	1.35	1.35
CC2	1.20	1.50	1.50
CC3	1.30	1.65	1.65

Table E. 2 Partial safety factors NEN 8700 (Nederlands Normalisatie Instituut, 2011a)

NEN 8700	**Permanent load**	**Governing load (not wind)**	**Governing wind load**
Combination 6.10a			
CC1	1.15	1.10	1.20
CC2	1.30 (1.20)	1.30	1.40
CC3	1.40 (1.30)	1.50	1.60 (1.50)
Combination 6.10b			
CC1	1.05	1.10	1.20
CC2	1.15	1.30	1.40
CC3	1.25 (1.20)	1.50	1.60 (1.50)

APPENDIX G CASE STUDY SCYE010 | OPTIMIZED DESIGNS

G.1 Optimized location design

Figure G. 1 shows the graphs of the wind pressure over the height of the building for all considered variants as well as for the original and intermediate design. The graph of the intermediate design looks differently compared to the other graphs. This is due to the increase in height for this design. Two levels of 3.4m are added to the original height (without the installation room), making part C as high as the main building of part B.

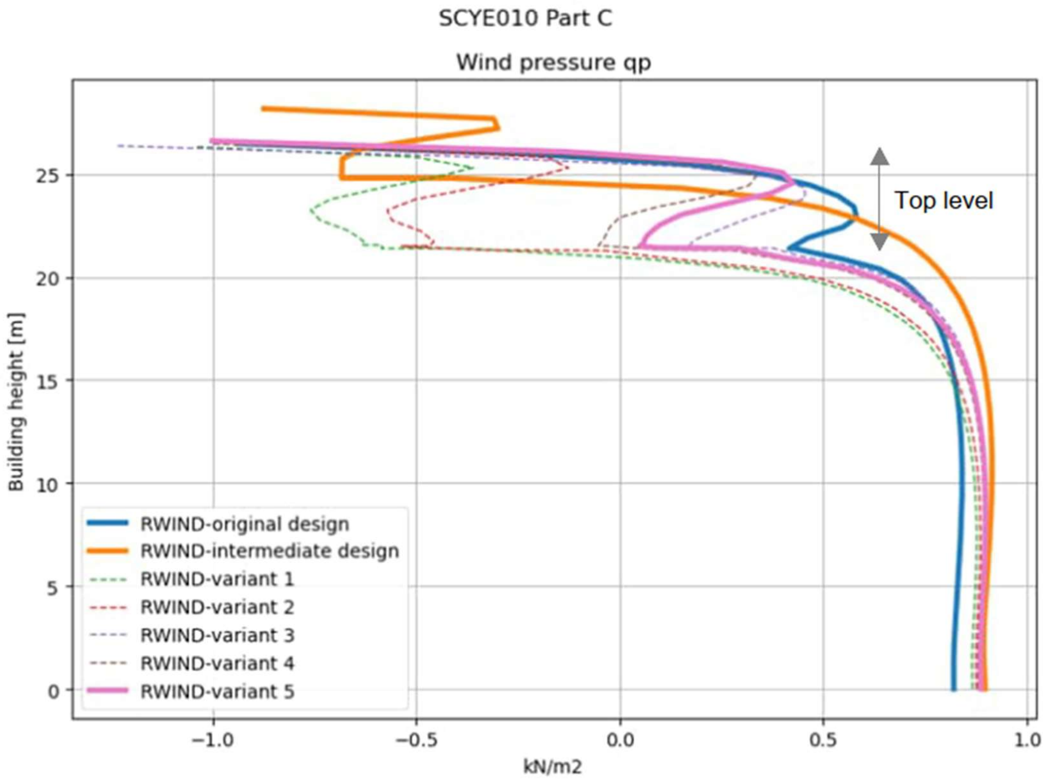


Figure G. 1 Wind pressure over the height of the building for all variants

In Figure G. 2, the wind pressure over the height of the building is shown for the optimized design and for the wind pressures according to the TGB 1972 and Eurocode. For the detailed calculation of the optimized design, the wind pressure according to the Eurocode will be used for the lower part of the building, to provide sufficient safety and to account for small errors in RWIND. The difference between these wind pressures is $0.99 - 0.90 = 0.09\ kN/m^2$, which is means a multiplication factor of 1.1 is applied. To be on the safe side as well for the top level, the wind pressure given by RWIND is multiplied with 1.1 as well, which results in a wind pressure of $0.43 * 1.1 = 0.47\ kN/m^2$. This will be applied in section 5.3.3, where the wind load calculations are performed.

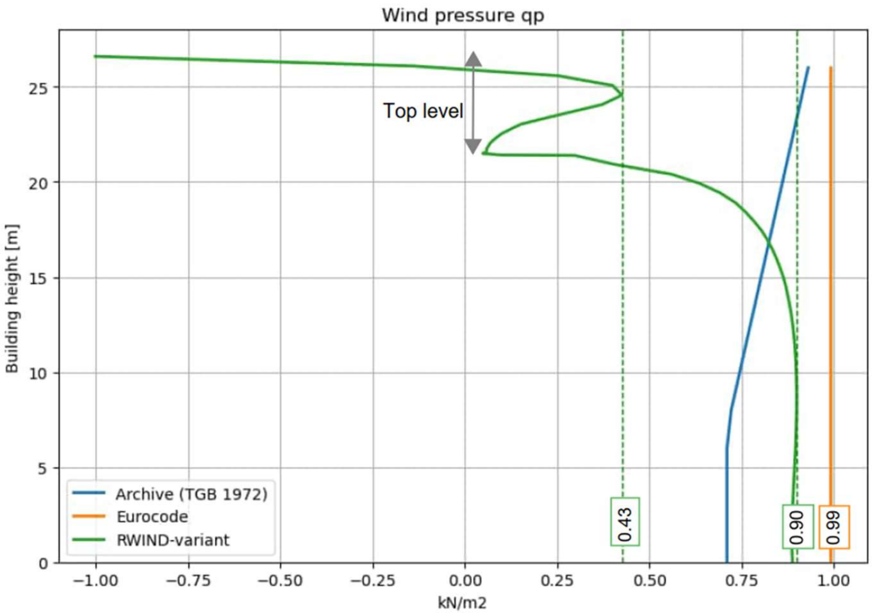


Figure G. 2 Optimized location design RWIND output compared to TGB 1972 and Eurocode

G.2 Optimized Volume design

For the optimized volume design, the same method is applied in order to obtain the wind pressure on the top level. Figure G. 3 and Figure G. 4 present the results of the optimized volume designs from RWIND in comparison to the TGB 1972 and the Eurocode.

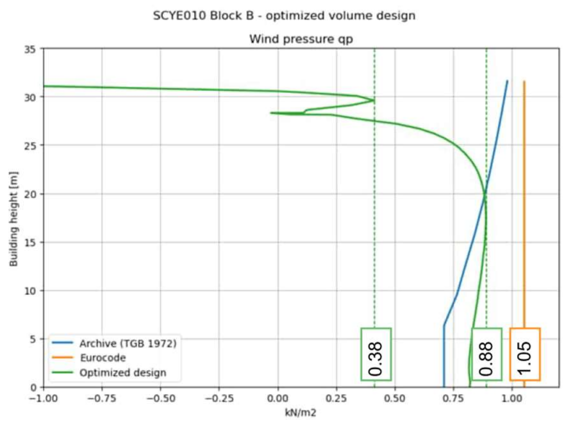


Figure G. 3 Optimized volume design RWIND, TGB 1972 and Eurocode results – part B

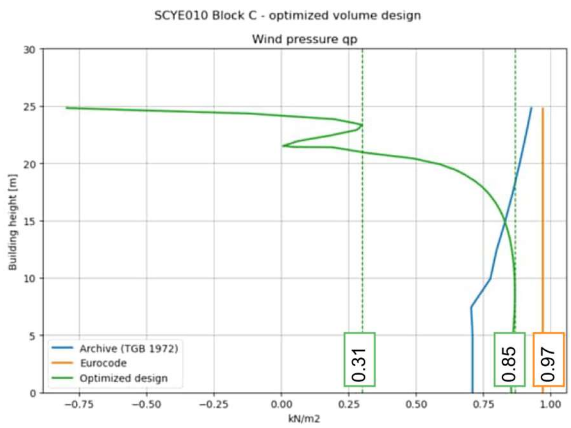


Figure G. 4 Optimized volume design RWIND, TGB 1972 and Eurocode results – part C

Using these results, the multiplication factors for the top levels can be determined, which will then be used to calculate the wind pressure on the top level.

For part B, the multiplication factor becomes $1.05/0.88 = 1.19$. Then, the wind pressure on the top level becomes $0.38 * 1.19 = 0.453 \approx 0.46\ kN/m^2$. For part C, the multiplication factor is $0.97/0.85 = 1.15$ and with this factor the wind pressure on the top level becomes $0.31 * 1.15 = 0.357 \approx 0.36\ kN/m^2$. These values are applied in the wind load calculations.

APPENDIX H CASE STUDY SCYE010 | WIND LOAD CALCULATIONS

This appendix discusses the wind load calculations on SCYE010, the case study. The following topics are elaborated in the appendix:

H.1 Check to verify neglection of wind friction coefficient

The archival calculations review part B and C separately, so the checks to see whether the perpendicular surface areas are at least 4 times larger than the parallel surfaces is performed per block as well. Furthermore, the checks will be performed for the two wind directions; 1 and 2 in Figure H. 1.

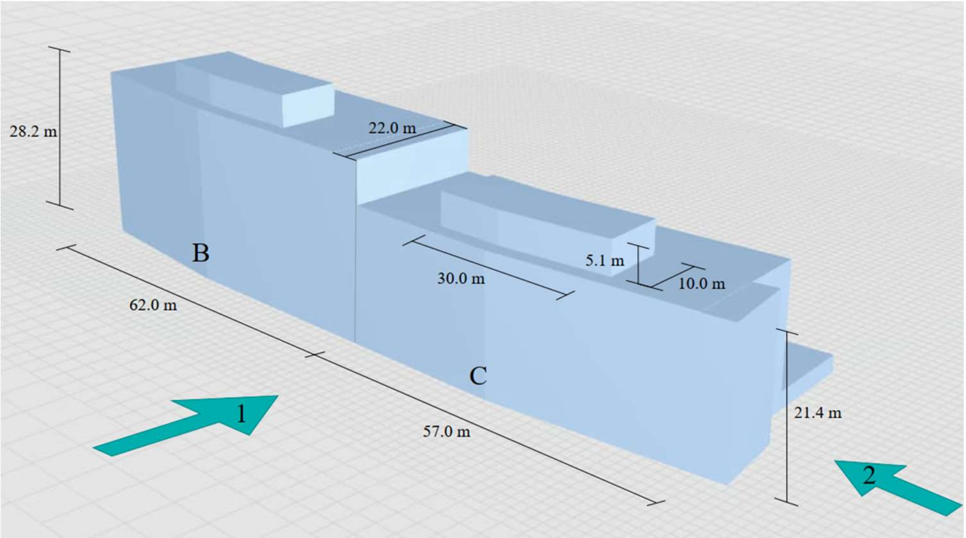


Figure H. 1 Dimensions and wind directions on SCYE010

Part B

Wind direction 1

Perpendicular surfaces

$$A_{perp,orignal\ building} = 2 * (62 * 28.2) = 2 * 1749 = 3498\ m^2$$

$$A_{perp,new\ levels} = 2 * (30 * 5.1) = 2 * 153 = 306\ m^2$$

$$A_{perp} = 3498 + 306 = 3804\ m^2$$

Parallel surfaces

$$A_{par,orignal\ building} = A_{par,sides} + A_{par,top}$$

$$= 2 * (22 * 28.2) + 22 * 62 = 2 * 620 + 1365 = 1605\ m^2$$

$$A_{par,new\ levels} = A_{par,sides} + A_{par,top}$$

$$= 2 * (10 * 5.1) + 10 * 30 = 2 * 51 + 300 = 402\ m^2$$

$$A_{par} = 1605 + 402 = 2007\ m^2$$

Check

$$A_{par} \leq 4 * A_{perp} \rightarrow 2007 \leq 4 * 3804 = 15216\ m^2$$

The statement is true, so the friction coefficient may be neglected on all sides for wind direction 1 on part B.

Wind direction 2
Perpendicular surfaces

$$A_{perp,orignal\ building} = 2 * (22 * 28.2) = 2 * 620 = 1240\ m^2$$

$$A_{perp,new\ levels} = 2 * (10 * 5.1) = 2 * 51 = 102\ m^2$$

$$A_{perp} = 1240 + 102 = 1342\ m^2$$

Parallel surfaces

$$A_{par,orignal\ building} = A_{par,sides} + A_{par,top}$$

$$= 2 * (62 * 28.2) + 62 * 22 = 2 * 1749 + 1365 = 4863\ m^2$$

$$A_{par,new\ levels} = A_{par,sides} + A_{par,top}$$

$$= 2 * (30 * 5.1) + 30 * 10 = 2 * 153 + 300 = 606\ m^2$$

$$A_{par} = 4863 + 606 = 5469\ m^2$$

Check

$$A_{par} \leq 4 * A_{perp} \rightarrow 5469 \nleq 4 * 1342 = 5368\ m^2$$

The statement is not true, so the friction coefficient may not be neglected on the sides for wind direction 2 on part B.

Part C
Wind direction 1
Perpendicular surfaces

$$A_{perp,orignal\ building} = 2 * (57 * 21.4) = 2 * 1220 = 1440\ m^2$$

$$A_{perp,new\ levels} = 2 * (30 * 5.1) = 2 * 153 = 306\ m^2$$

$$A_{perp} = 1440 + 306 = 1746\ m^2$$

Parallel surfaces

$$A_{par,orignal\ building} = A_{par,sides} + A_{par,top}$$

$$= 2 * (22 * 21.4) + 22 * 57 = 2 * 471 + 1254 = 2196\ m^2$$

$$A_{par,new\ levels} = A_{par,sides} + A_{par,top}$$

$$= 2 * (10 * 5.1) + 10 * 30 = 2 * 51 + 300 = 402\ m^2$$

$$A_{par} = 2196 + 402 = 2598\ m^2$$

Check

$$A_{par} \leq 4 * A_{perp} \rightarrow 2598 \leq 4 * 1746 = 6984\ m^2$$

The statement is true, so the friction coefficient may be neglected on all sides for wind direction 1 on part C.

Wind direction 2
Perpendicular surfaces

$$A_{perp,orignal\ building} = 2 * (22 * 21.4) = 2 * 471 = 942\ m^2$$

$$A_{perp,new\ levels} = 2 * (10 * 5.1) = 2 * 51 = 102\ m^2$$

$$A_{perp} = 942 + 102 = 1044\ m^2$$

Parallel surfaces

$$A_{par,orignal\ building} = A_{par,sides} + A_{par,top}$$

$$= 2 * (57 * 21.4) + 57 * 22 = 2 * 1220 + 1254 = 3694\ m^2$$

$$A_{par,new\ levels} = A_{par,sides} + A_{par,top}$$

$$= 2 * (30 * 5.1) + 30 * 10 = 2 * 153 + 300 = 606\ m^2$$

$$A_{par} = 3694 + 606 = 4300\ m^2$$

Check

$$A_{par} \leq 4 * A_{perp} \rightarrow 4300 \nleq 4 * 1044 = 4176\ m^2$$

The statement is not true, so the friction coefficient may not be neglected on the sides for wind direction 2 on part C.

H.2 Evaluation of weather station Rotterdam Geulhaven

H 2.1 Wind speed | location specific

Weather station Rotterdam Geulhaven is evaluated for the case study of SCYE010. From the KNMI website, the datasets with the average wind speed over the last 10 minutes of an hour. This data is presented in Figure H. 2. The reference period is the same as for the Eurocode and is the period of 1986 tot 2011. This dataset misses some years of the reference period of the Eurocode; the 10-min data for weather station Rotterdam Geulhaven starts at 1991, making it less reliable.

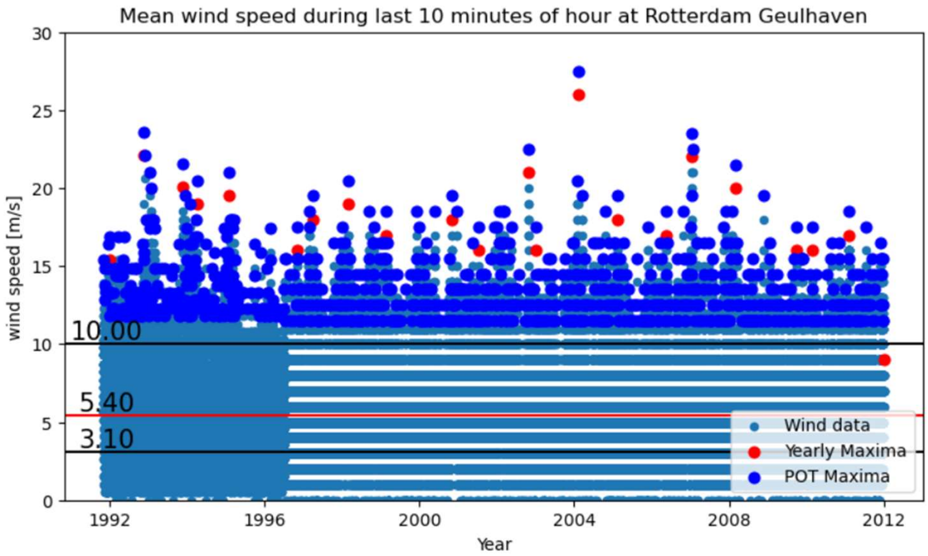


Figure H. 2 Wind speeds at weather station Rotterdam Geulhaven

Next, using the yearly maxima and the Gumbel distribution, the return value is obtained for a return period of 50 years. This is plotted in Figure H. 3 and results in a wind speed of ~26.5 m/s. This is the location specific wind speed.

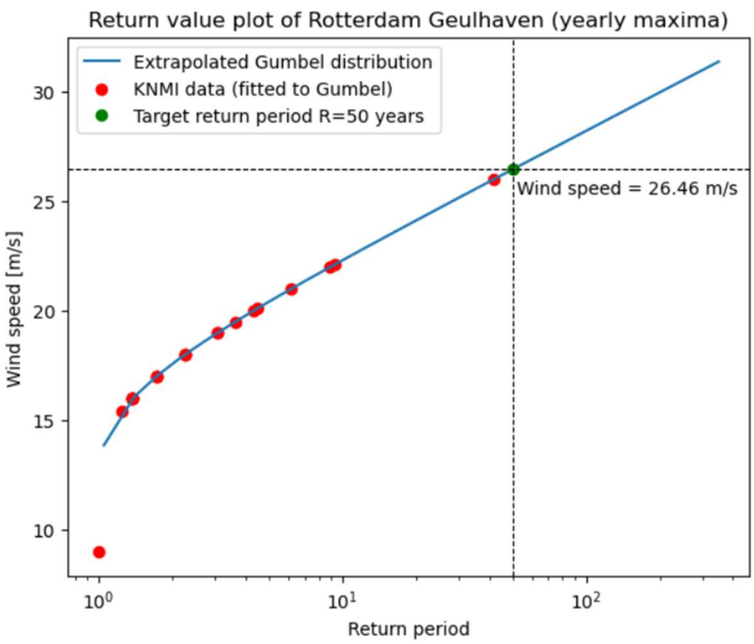


Figure H. 3 Return value plot of weather station Rotterdam Geulhaven

For the optimized designs, the location specific wind pressure is needed for the wind load calculations. Therefore, the heights of the different building parts for both designs is needed. This overview is given in Table H. 1.

Table H. 1 Heights per building part of the optimized designs

	Building part	z [m]
Optimized location design	B	33.3
	C	26.5
Optimized volume design	B	31.6
	C	24.8

Using the formula for wind pressure from the Eurocode, the wind pressure is obtained. This is given in Table H. 2.

Table H. 2 Location specific wind pressure

Optimized design	Building part	z [m]	Wind speed [m/s]	v_b [m/s]	z_0 [m/s]	$I_v(z)$	k_r	c_r	v_m [m/s]	q_p [kN/m^2]	$q_{p,\ Eurocode}$ [kN/m^2]	Difference	
Location	B	33.3	26.5	26.5	0.5	0.238	0.223	0.937	24.84	**1.03**	1.07	-0.04	-4%
Location	C	26.5	26.5	26.5	0.5	0.252	0.223	0.886	23.49	**0.95**	0.99	-0.04	-4%
Volume	B	31.6	26.5	26.5	0.5	0.241	0.223	0.926	24.53	**1.01**	1.10	-0.09	-8%
Volume	C	24.8	26.5	26.5	0.5	0.256	0.223	0.871	23.09	**0.93**	1.03	-0.10	-10%

H 2.2 Wind pressure | change of wind speed over time

Another effect that is taken into account for the comparison of the optimized designs is the change of wind speed over time. This is again evaluation for specifically the weather station Rotterdam Geulhaven. In Figure H. 4 and Figure H. 5, the yearly maxima of the reference period for the Eurocode and the following years up until now (2024) are plotted and using this data a trendline is obtained. The first figure uses a 1st order polynomial, which is a linear trend line, and the second uses a second order polynomial. Next, the reference period is compared to the wind speed at this year. This results in a decrease of 0.25 m/s in comparison to the Eurocode that was published in 2011.

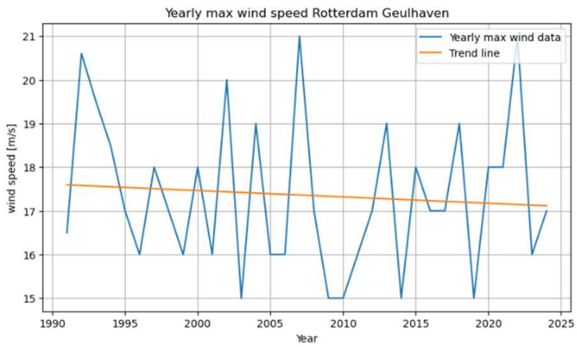


Figure H. 4 Trend of yearly maxima (1st order polynomial)

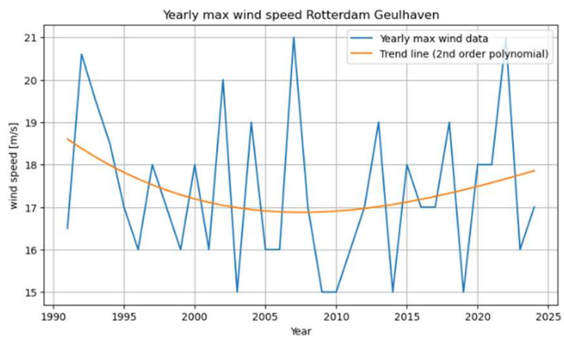


Figure H. 5 Trend of yearly maxima (2nd order polynomial)

SCYE010 is located in wind area II, which means the base wind speed that is prescribed is 27.0 m/s. Using the change in wind speed over time, this becomes $27.0 - 0.25 = 26.75\ m/s$ m/s. Table H.3 presents the results.

Table H. 3 Location specific wind pressure

Optimized design	Building part	z [m]	Wind speed [m/s]	v_b [m/s]	z_0 [m/s]	$I_v(z)$	k_r	c_r	v_m [m/s]	q_p [kN/m^2]	$q_{p,\text{ Eurocode}}$ [kN/m^2]	Difference	
Location	B	33.3	26.75	26.75	0.5	0.238	0.223	0.937	25.07	**1.05**	1.07	-0.02	-2%
Location	B	31.6	26.75	26.75	0.5	0.252	0.223	0.886	23.71	**0.97**	0.99	-0.02	-2%
Volume	C	26.5	26.75	26.75	0.5	0.241	0.223	0.926	24.76	**1.03**	1.10	-0.07	-6%
Volume	C	24.8	26.75	26.75	0.5	0.256	0.223	0.871	23.31	**0.95**	1.03	-0.08	-8%

H 2.3 Wind pressure | Combined effects

For the evaluation of all effects combined, the wind pressure on the levels below the top level is only changing due to the location specific wind speed and the change of wind speed over time. Combined these result in a wind speed of: $26.5 - 0.25 = 26.25\ m/s$. This wind speed is implemented in the calculations per optimized design and for each building part and the results and sub results are presented in Table H. 4.

Table H. 4 Wind pressure due to combined effects

Optimized design	Building part	z [m]	Wind speed [m/s]	v_b [m/s]	z_0 [m/s]	$I_v(z)$	k_r	c_r	v_m [m/s]	q_p [kN/m^2]	$q_{p,\text{ Eurocode}}$ [kN/m^2]	Difference	
Location	B	33.3	26.25	26.25	0.5	0.238	0.223	0.937	24.60	1.01	1.07	-0.06	-6%
Location	B	31.6	26.25	26.25	0.5	0.252	0.223	0.886	23.27	0.93	0.99	-0.06	-6%
Volume	C	26.5	26.25	26.25	0.5	0.241	0.223	0.926	24.30	0.99	1.10	-0.11	-10%
Volume	C	24.8	26.25	26.25	0.5	0.256	0.223	0.871	22.88	0.91	1.03	-0.12	-11%

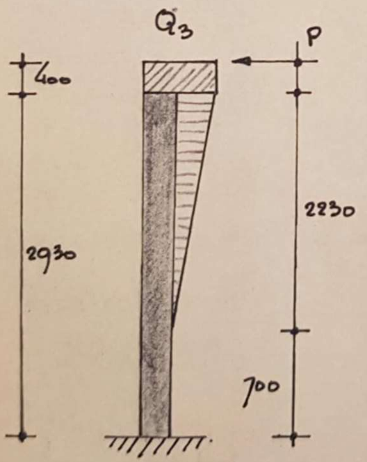
H.3 Wind load calculations

In this section, five overviews are presented:

H.3.1 Archival wind load calculations

Part B

Windbelastingen:



b = 22,50 m. lca = 62,00 m.

P : strijkwind = 30 x 17,50x 0,04 x 100 = 2100 kg

Q3 : 1,2 x 100 x 4 x 17,5 + 0,08 x 30 x 4 x 100 = 9350 kg

q_1 = 1,2 x 71 x 22,5 + 0,08 x 71 x 62 = 2270 kg/m^1

q_2 = 26 x 0,08 x 62 x 22,5 x 1,2 = 830 kg/m^1

Q = 2100 + 9350 + 66500 + 9250 = 87.200 kg

Mw = 2100 x 33,3 + 9350 x 31,3 + 66500 x 14,65 + 21,85 x 9250 = 1.540.000 kgm

wand voor = 350.000
wand achter = 420.000

Mw op 450^+ = 1.170.000 kgm (76 %)

Windbelasting deel B :

zie ook pagina 18 9 schema)

l = 62 m. P strijkwind = 2100 kg.

Q 3 = 1,2 x 100 x 4 x 29 + 0,08 x 100 x 4 x 17,5 = 14500 kg

q_1 = 1,2 x 71 x 62 + 0,08 x 71 x 22,40 = 5400 kg/m^1

q_2 = 1,2 x 26 x 62 + 26 x 0,08 x 22,40 = 1980 kg/m^1.

Q_w = 2100 + 14.500 + 158.200 + 22.000 = 196.800 kg

M wind = 2100 x 33,3 + 14.500 x 31,3 + 158.200 x 14,65 + 21,85 x 22.000 = 3.320.000 kgm

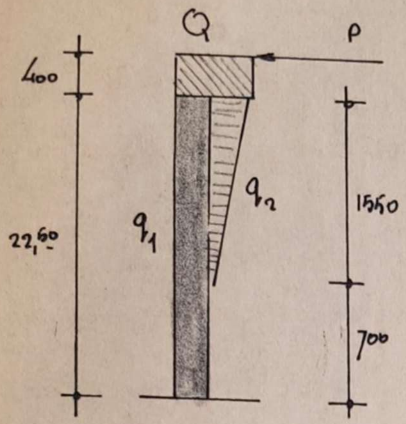
per zijde 1.660.000 kgm (1,66 x 10^6 kgm)

(Mw totaal = 2,57 x 10^6 kgm)

Part C

163

Windbelastingen:



b = 22,50 m. l = ca. 57,00 m.

P = strijkwind = 30 x 17,50 x 0,04 x 93 = 1950 kg

Q 3 =1,2 x 93 x 4 x 17,5 + 0,08 x 30 x 4 x 93 = 8700 kg

q_1 = 1,2 x 71 x 22,5 + 0,08 x 71 x 57= 2240 kg/m^1

q_2 = 19 x 0,08 x 57 + 19 x 22,5 x 1,2 = 600 kg/m^1

Q = 1950 + 8700 + 50400 + 4650 = 65700 kg

Mw = 1950 x 26,50 + 8700 x 24,50 + 50400 x 11,25 + 4650 x 17,33 =
910.000 kgm per wand voor ⟶ 205.000 kgm

Mw op 450^+ = 640 kgm per wand voor ⟶ 145.000 kgm
(70%)

Windbelasting:

(zie ook pagina : 163, schema)

b = 57 m.

P = strijkwind = 1950 kg (pagina : 163)

Q 3 = 1,2 x 93 x 4 x 29,50 + 0,08 x 93 x 4 x 17,5 = 13700 kg

q_1 = 1,2 x 71 x 57 + 0,08 x 71 x 22,40 = 4980 kg/m^1

q_2 = 19 x 57 x 1,2 + 19 x 0,08 x 22,40 = 1330 kg/m^1

Q = 1950 + 13700 + 112.050 + 10.300 = 138.000 kg
per wand ⟶ 69.000 kg

Mw = 1950 x 26,50 + 13700 x 24,50 + 112.050 x 11,25 + 10300 x 17,33 = 1.820.000 kgm

per wand = 910.000 kgm

H.3.2 Overview wind load part B

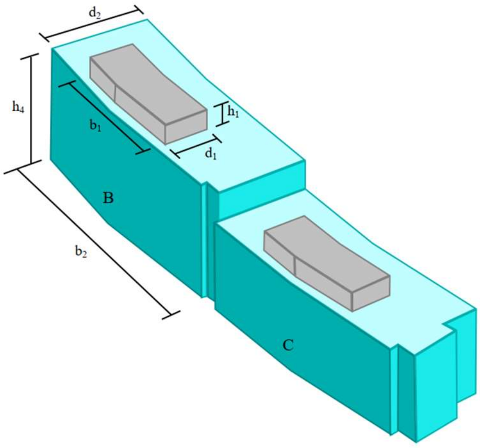


Figure H. 6 Original/optimized location design

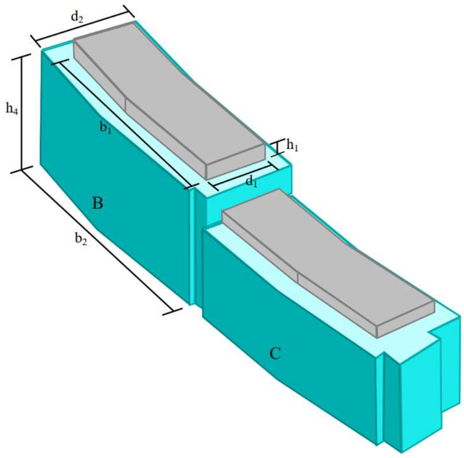


Figure H. 7 Optimized volume design

Table H. 5 Overview wind loads original design versus optimized designs (part B)

Design	Original design	Optimized location	Optimized volume	
Wind load model	TGB 1972	Eurocode - detailed model	Eurocode - detailed model	Units
Building dimensions				
h_1	4	5.1	3.4	m
h_2	22.3			m
h_3	7			m
h_4	29.3	28.2	28.2	m
h_{tot}	33.3	33.3	31.6	m
b_1	30	30	60	m
b_2	62	62	62	m
d_1	17.5	10	20.4	m
d_2	22.5	24.9	24.9	m
Wind pressure and coefficients				
q_{p1}	1.00	1.07	1.05	kN/m^2
q_{p2}	0.71	0.46	0.46	kN/m^2
Δq	0.26			kN/m^2
c_f	0.04	0.04	0.04	-
c_{pe}	1.20	1.33	1.34	-
lack of correlation		0.85	0.85	-
$c_s c_d$		1.00	1.00	-
Wind direction 1				
P_1	21.0	5.5	23.1	kN
P_2		53.2	12.2	kN
Q_2	145			kN
q_{w1}	54	75.0	74.1	kN/m
q_{w2}	19.8	32.2	31.4	kN/m
Q_{tot}	1964	2338	2233	kN
$Q_{tot,façade}$	982	1169	1117	kN
M_{wk}	33163	36562	33751	kNm
$M_{wk,façade}$	16581	18281	16876	kNm
$M_{wd,façade}$	24872	27421	25313	kNm
L_{wall}	21.6	21.6	21.6	m
a	14.4	14.4	14.4	m
$R_{k,tot}$	1151	1270	1172	kN
nr of piles	4	4	4	-
R_k	288	317	293	kN
R_d	432	476	439	kN

Design	Original design	Optimized location	Optimized volume	
Wind load model	TGB 1972	Eurocode - detailed model	Eurocode - detailed model	Units
Wind direction 2				
P_1	21.0	5.5	23.1	kN
P_2		53.2	12.2	kN
Q_2	93.6			kN
q_{w1}	22.7	35.4	35.0	kN/m
q_{w2}	8.3	6.3	30.0	kN/m
Q_{tot}	872	1090	1124	kN
$Q_{tot,façade}$	436.1	545	562	kN
M_{wk}	15395	16238	16511	kNm
$M_{wk,façade}$	3849	4059	4128	kNm
$M_{wd,façade}$	5773	6089	6191	kNm
L_{wall}	7.2	7.2	7.2	m
a	5	5	5	m
$R_{k,tot}$	802	846	860	kN
nr of piles	3	3	4	-
R_k	267	282	215	kN
R_d	401	423	322	kN

H.3.3 Overview wind loads part C

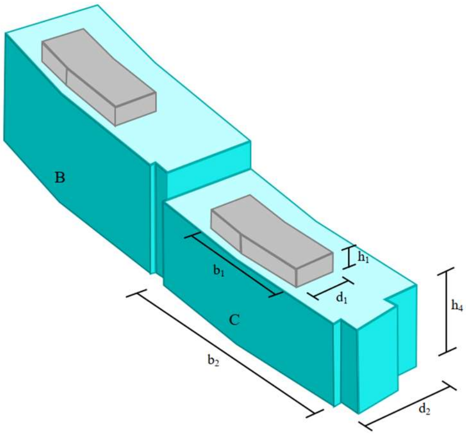


Figure H. 8 Original/optimized location design

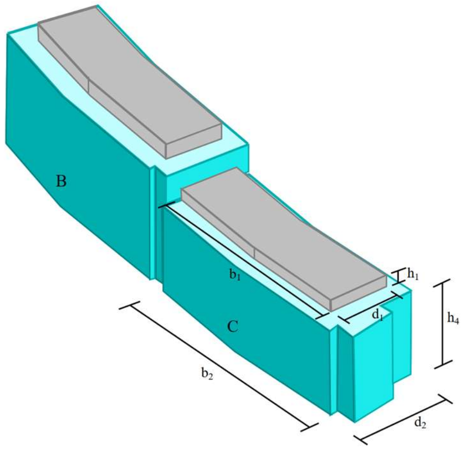


Figure H. 9 Optimized volume design

Table H. 6 Overview wind loads original design versus optimized designs (part C)

Design	Original design	Optimized location	Optimized volume	
Wind load model	TGB 1972	Eurocode - detailed model	Eurocode - detailed model	Units
Building dimensions				
h_1	4	5.1	3.4	m
h_2	15.5			m
h_3	7			m
h_4	22.5	21.4	21.4	m
h_{tot}	26.5	26.5	24.8	m
b_1	30	30	55.5	m
b_2	57	57	57	m
d_1	17.5	10	19.4	m
d_2	22.4	24.9	24.9	m
Wind pressure and coefficients				
q_{p1}	0.93	0.99	0.97	kN/m^2
q_{p2}	0.71	0.47	0.36	kN/m^2
Δq	0.02			kN/m^2
c_f	0.04	0.04	0.04	-
c_{pe}	1.20	1.33	1.34	-
lack of correlation		0.85	0.85	-
$c_s c_d$		1.00	1.00	-
Wind direction 1				
P_1	19.5	5.5	17.5	kN
P_2		44.3	7.9	kN
Q_2	135			kN
q_{w1}	50	63.8	63.0	kN/m
q_{w2}	1.3	29.6	22.8	kN/m
Q_{tot}	1280	1566	1450	kN
$Q_{tot,façade}$	640	783	725	kN
M_{wk}	16540	19323	16811	kNm
$M_{wk,façade}$	8270	9661	8405	kNm
$M_{wd,façade}$	12405	14492	12608	kNm
L_{wall}	21.6	21.6	21.6	m
a	14.4	14.4	14.4	m
$R_{k,tot}$	1151	671	584	kN
nr of piles	4	4	4	-
R_k	288	168	146	kN
R_d	432	252	219	kN

Design	Original design	Optimized location	Optimized volume	
Wind load model	TGB 1972	Eurocode - detailed model	Eurocode - detailed model	Units
Wind direction 2				
P_1	19.5	5.5	17.5	kN
P_2		44.3	7.9	kN
Q_2	87.0			kN
q_{w1}	22.3	32.4	31.9	kN/m
q_{w2}	0.6	6.3	28.5	kN/m
Q_{tot}	613	775	806	kN
$Q_{tot,façade}$	306.7	387	403	kN
M_{wk}	8381	8867	9035	kNm
$M_{wk,façade}$	2095	2217	2259	kNm
$M_{wd,façade}$	3143	3325	3388	kNm
L_{wall}	7.2	7.2	7.2	m
a	5	5	5	m
$R_{k,tot}$	437	462	471	kN
nr of piles	3	3	4	-
R_k	146	154	118	kN
R_d	218	231	176	kN

H.3.4 All effects applied | optimized location design

Part B

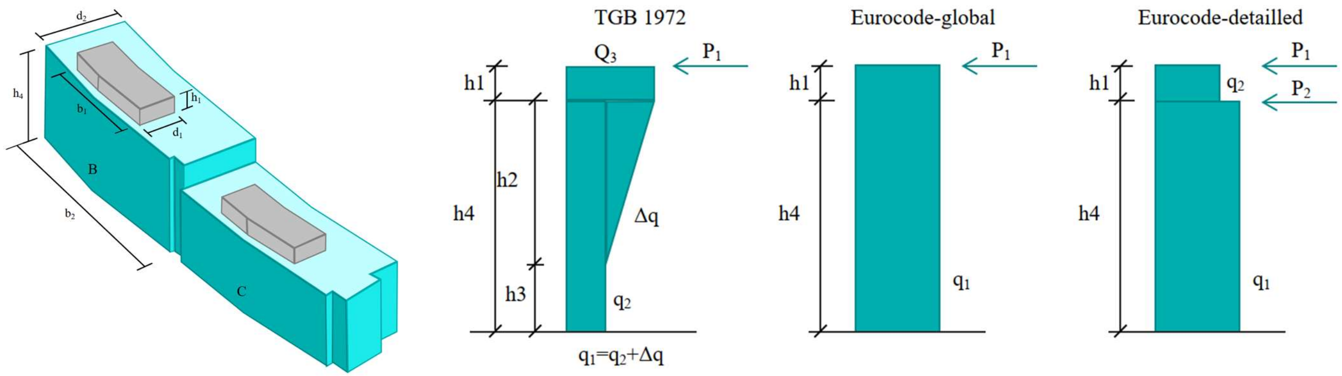


Table H. 7 Overview wind loads for all effects - optimized location design (part B)

Design	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	
Effect	-	Decreased width at top level	Reduced wind pressure at top level	Reduced wind speed according to data	Reduced wind speed due to climate	All effects	
Wind load model	Eurocode-global	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Units
Building dimensions							
h_1	5.1	5.1	5.1	5.1	5.1	5.1	m
h_4	28.2	28.2	28.2	28.2	28.2	28.2	m
h_{tot}	33.3	33.3	33.3	33.3	33.3	33.3	m
b_1	30	30	30	30	30	30	m
b_2	62	62	62	62	62	62	m
d_1	10	10	10	10	10	10	m
d_2	24.9	24.9	24.9	24.9	24.9	24.9	m
Wind pressure and coefficients							
q_{p1}	1.05	1.05	1.05	1.03	1.05	1.01	kN/m^2
q_{p2}	1.05	1.05	0.46	1.03	1.05	0.46	kN/m^2
c_f	0.04	0.04	0.04	0.04	0.04	0.04	-
c_{pe}	1.34	1.34	1.34	1.34	1.34	1.34	-
lack of correlation	0.85	0.85	0.85	0.85	0.85	0.85	-
c_sc_d	1.00	1.00	1.00	1.00	1.00	1.00	-

Wind direction 1							
P_1	12.6	12.6	5.5	12.4	12.6	5.5	kN
P_2	52.2	52.2	52.2	51.2	52.2	50.2	kN
q_{w1}	74.1	74.1	74.1	72.7	74.1	71.3	kN/m
q_{w2}	35.9	35.9	15.7	35.2	35.9	15.7	kN/m
Q_{tot}	2339	2339	2229	2294	2339	2147	kN
$Q_{tot,façade}$	1169	1169	1114	1147	1169	1074	kN
M_{wk}	37002	37002	33605	36298	37002	32426	kNm
$M_{wk,façade}$	18501	18501	16803	18149	18501	16213	kNm
$M_{wd,façade}$	27752	27752	25204	27223	27752	24319	kNm
L_{wall}	21.6	21.6	21.6	21.6	21.6	21.6	m
a	14.4	14.4	14.4	14.4	14.4	14.4	m
$R_{k,tot}$	1285	1285	1167	1260	1285	1126	kN
nr of piles	4	4	4	4	4	4	-
R_k	321	321	292	315	321	281	kN
R_d	482	482	438	473	482	422	kN
Wind direction 2							
P_1	12.6	12.6	5.5	12.4	12.6	5.5	kN
P_2	52.2	52.2	52.2	51.2	52.2	50.2	kN
Q_2							kN
q_{w1}	35.0	35.0	35.0	34.3	35.0	33.7	kN/m
q_{w2}	14.5	14.5	14.5	14.2	14.5	13.9	kN/m
Q_{tot}	1125	1125	1118	1104	1125	1076	kN
$Q_{tot,façade}$	563	563	559	552	563	538	kN
M_{wk}	16940	16940	16704	16617	16940	16075	kNm
$M_{wk,façade}$	4235	4235	4176	4154	4235	4019	kNm
$M_{wd,façade}$	6352	6352	6264	6231	6352	6028	kNm
L_{wall}	7.2	7.2	7.2	7.2	7.2	7.2	m
a	5	5	5	5	5	5	m
$R_{k,tot}$	882	882	870	865	882	837	kN
nr of piles	4	4	4	4	4	4	-
R_k	221	221	217	216	221	209	kN
R_d	331	331	326	325	331	314	kN

Part C

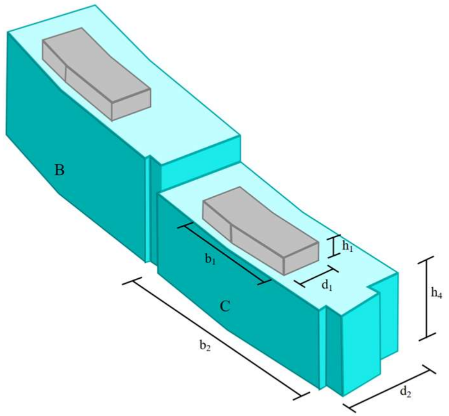


Table H. 8 Overview wind loads for all effects - optimized location design (part C)

Design	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	
Effect	-	Decreased width at top level	Reduced wind pressure at top level	Reduced wind speed according to data	Reduced wind speed due to climate	All effects	
Wind load model	Eurocode-global	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Units
Building dimensions							
h_1	5.1	5.1	5.1	5.1	5.1	5.1	m
h_4	21.4	21.4	21.4	21.4	21.4	21.4	m
h_{tot}	26.5	26.5	26.5	26.5	26.5	26.5	m
b_1	30	30	30	30	30	30	m
b_2	62	62	62	62	62	62	m
d_1	10	10	10	10	10	10	m
d_2	24.9	24.9	24.9	24.9	24.9	24.9	m
Wind pressure and coefficients							
q_{p1}	0.97	0.97	0.97	0.93	0.97	0.93	kN/m^2
q_{p2}	0.97	0.97	0.47	0.93	0.97	0.47	kN/m^2
c_f	0.04	0.04	0.04	0.04	0.04	0.04	-
c_{pe}	1.34	1.34	1.34	1.34	1.34	1.34	-
lack of correlation	0.85	0.85	0.85	0.85	0.85	0.85	-
c_sc_d	1.00	1.00	1.00	1.00	1.00	1.00	-

Wind direction 1							
P_1	11.6	11.6	5.5	11.2	11.6	5.5	kN
P_2	48.3	48.3	48.3	46.3	48.6	44.8	kN
q_{w1}	68.5	68.5	68.5	65.7	68.5	65.7	kN/m
q_{w2}	33.1	33.1	15.7	31.8	33.1	15.7	kN/m
Q_{tot}	1695	1695	1600	1625	1695	1537	kN
$Q_{tot,façade}$	847	847	800	812	847	769	kN
M_{wk}	21075	21075	18784	20206	21075	18095	kNm
$M_{wk,façade}$	10537	10537	9392	10103	10537	9047	kNm
$M_{wd,façade}$	15806	15806	14088	15154	15806	13571	kNm
L_{wall}	21.6	21.6	21.6	21.6	21.6	21.6	m
a	14.4	14.4	14.4	14.4	14.4	14.4	m
$R_{k,tot}$	732	732	652	702	732	628	kN
nr of piles	4	4	4	4	4	4	-
R_k	183	183	163	175	183	157	kN
R_d	274	274	245	263	274	236	kN
Wind direction 2							
P_1	11.6	11.6	5.5	11.2	11.6	5.5	kN
P_2	48.3	48.3	48.3	46.3	48.3	46.3	kN
q_{w1}	32.3	32.3	32.3	31.0	32.3	31.0	kN/m
q_{w2}	13.4	13.4	13.4	12.8	13.4	12.4	kN/m
Q_{tot}	820	820	814	786	820	780	kN
$Q_{tot,façade}$	410	410	407	393	410	390	kN
M_{wk}	9559	9559	9397	9165	9559	9015	kNm
$M_{wk,façade}$	2390	2390	2349	2291	2390	2254	kNm
$M_{wd,façade}$	3585	3585	3524	3437	3585	3381	kNm
L_{wall}	7.2	7.2	7.2	7.2	7.2	7.2	m
a	5	5	5	5	5	5	m
$R_{k,tot}$	498	498	489	477	498	470	kN
nr of piles	4	4	4	4	4	4	-
R_k	124	124	122	119	124	117	kN
R_d	187	187	184	179	187	176	kN

H.3.5 All effects applied | optimized volume design

Part B

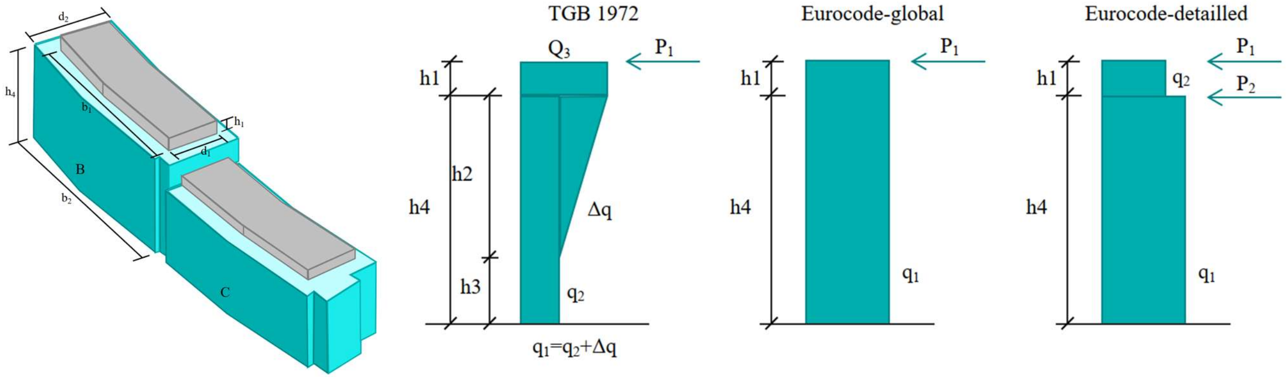


Table H. 9 Overview wind loads for all effects - optimized volume design (part B)

Design	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	
Effect	-	Decreased width at top level	Reduced wind pressure at top level	Reduced wind speed according to data	Reduced wind speed due to climate	All effects	
Wind load model	Eurocode-global	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Units
Building dimensions							
h_1	3.4	3.4	3.4	3.4	3.4	3.4	m
h_4	28.2	28.2	28.2	28.2	28.2	28.2	m
h_{tot}	31.6	31.6	31.6	31.6	31.6	31.6	m
b_1	60	60	62	62	62	60	m
b_2	62	62	62	62	62	62	m
d_1	24.9	20.4	24.9	24.9	24.9	20.4	m
d_2	24.9	24.9	24.9	24.9	24.9	24.9	m
Wind pressure and coefficients							
q_{p1}	1.05	1.05	1.05	1.01	1.03	0.99	kN/m^2
q_{p2}	1.05	1.05	0.46	1.01	1.03	0.46	kN/m^2
c_f	0.04	0.04	0.04	0.04	0.04	0.04	-
c_{pe}	1.34	1.34	1.34	1.34	1.34	1.34	-
lack of correlation	0.85	0.85	0.85	0.85	0.85	0.85	-
c_sc_d	1.00	1.00	1.00	1.00	1.00	1.00	-

Wind direction 1							
P_1	52.7	52.7	28.4	62.4	63.6	27.5	kN
P_2	12.2	12.2	0.0	0.0	0.0	2.0	kN
q_{w1}	74.1	74.1	74.1	71.3	72.7	69.9	kN/m
q_{w2}	71.8	71.8	32.5	71.3	72.7	31.4	kN/m
Q_{tot}	2400	2400	2230	2316	2362	2108	kN
$Q_{tot,façade}$	1200	1200	1115	1158	1181	1054	kN
M_{wk}	38785	38785	33683	37582	38326	31918	kNm
$M_{wk,façade}$	19393	19393	16842	18791	19163	15959	kNm
$M_{wd,façade}$	29089	29089	25262	28186	28744	23939	kNm
L_{wall}	21.6	21.6	21.6	21.6	21.6	21.6	m
a	14.4	14.4	14.4	14.4	14.4	14.4	m
$R_{k,tot}$	1347	1347	1170	1305	1331	1108	kN
nr of piles	4	4	4	4	4	4	-
R_k	337	337	292	326	333	277	kN
R_d	505	505	439	489	499	416	kN
Wind direction 2							
P_1	52.7	52.7	28.4	62.4	62.4	27.5	kN
P_2	12.2	12.2	0.0	0.0	0.0	1.9	kN
q_{w1}	35.0	35.0	35.0	33.7	34.3	33.0	kN/m
q_{w2}	30.0	30.0	35.0	33.7	34.3	32.8	kN/m
Q_{tot}	1154	1154	1134	1126	1148	1071	kN
$Q_{tot,façade}$	577	577	567	563	574	536	kN
M_{wk}	17446	17446	16588	17063	17401	15710	kNm
$M_{wk,façade}$	4361	4361	4147	4266	4350	3927	kNm
$M_{wd,façade}$	6542	6542	6220	6399	6525	5891	kNm
L_{wall}	7.2	7.2	7.2	7.2	7.2	7.2	m
a	5	5	5	5	5	5	m
$R_{k,tot}$	909	909	864	889	906	818	kN
nr of piles	4	4	4	4	4	4	-
R_k	227	227	216	222	227	205	kN
R_d	341	341	324	333	340	307	kN

Part C

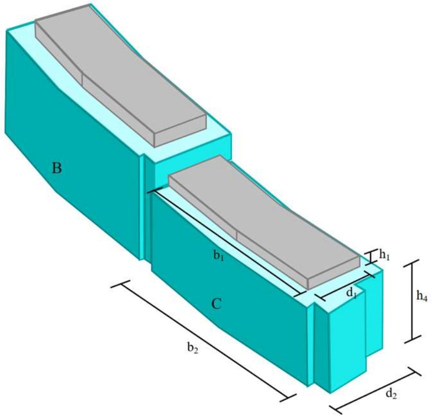


Table H. 10 Overview wind loads for all effects - optimized volume design (part C)

Design	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	Optimized volume	
Effect	-	Decreased width at top level	Reduced wind pressure at top level	Reduced wind speed according to data	Reduced wind speed due to climate	All effects	
Wind load model	Eurocode-global	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Eurocode - detailed model	Units
Building dimensions							
h_1	3.4	3.4	3.4	3.4	3.4	3.4	m
h_4	21.4	21.4	21.4	21.4	21.4	21.4	m
h_{tot}	24.8	24.8	24.8	24.8	24.8	24.8	m
b_1	57	55.5	55.5	55.5	55.5	55.5	m
b_2	57	57	57	57	57	57	m
d_1	24.9	19.4	21.9	21.9	21.9	19.4	m
d_2	24.9	24.9	24.9	24.9	24.9	24.9	m
Wind pressure and coefficients							
q_{p1}	0.97	0.97	0.97	0.93	0.95	0.91	kN/m^2
q_{p2}	0.97	0.97	0.36	0.93	0.95	0.36	kN/m^2
c_f	0.04	0.04	0.04	0.04	0.04	0.04	-
c_{pe}	1.34	1.34	1.34	1.34	1.34	1.34	-
lack of correlation	0.85	0.85	0.85	0.85	0.85	0.85	-
c_sc_d	1.00	1.00	1.00	1.00	1.00	1.00	-

Wind direction 1							
P_1	55.1	47.2	17.5	45.2	46.2	17.5	kN
P_2	0.0	7.9	7.9	7.6	7.7	7.4	kN
q_{w1}	63.0	63.0	63.0	60.4	61.7	59.1	kN/m
q_{w2}	63.0	61.3	22.8	58.8	60.1	22.8	kN/m
Q_{tot}	1617	1611	1450	1545	1578	1367	kN
$Q_{tot,façade}$	808	806	725	772	789	683	kN
M_{wk}	20732	20575	16811	19726	20151	15908	kNm
$M_{wk,façade}$	10366	10287	8405	9863	10075	7954	kNm
$M_{wd,façade}$	15549	15431	12608	14795	15113	11931	kNm
L_{wall}	21.6	21.6	21.6	21.6	21.6	21.6	m
a	14.4	14.4	14.4	14.4	14.4	14.4	m
$R_{k,tot}$	720	714	584	685	700	552	kN
nr of piles	4	4	4	4	4	4	-
R_k	180	179	146	171	175	138	kN
R_d	270	268	219	257	262	207	kN
Wind direction 2							
P_1	55.1	47.2	17.5	45.2	46.2	17.5	kN
P_2	0.0	7.9	7.9	7.6	7.7	7.4	kN
q_{w1}	31.9	31.9	31.9	30.6	31.3	30.0	kN/m
q_{w2}	31.9	28.5	28.5	27.3	27.9	26.7	kN/m
Q_{tot}	847	835	806	801	818	757	kN
$Q_{tot,façade}$	424	418	403	400	409	378	kN
M_{wk}	9932	9770	9035	9367	9569	8503	kNm
$M_{wk,façade}$	2483	2443	2259	2342	2392	2126	kNm
$M_{wd,façade}$	3724	3664	3388	3513	3588	3189	kNm
L_{wall}	7.2	7.2	7.2	7.2	7.2	7.2	m
a	5	5	5	5	5	5	m
$R_{k,tot}$	517	509	471	488	498	443	kN
nr of piles	4	4	4	4	4	4	-
R_k	129	127	118	122	125	111	kN
R_d	194	191	176	183	187	166	kN

H.4 Deflection

H 4.1 Overview of system

In Figure H. 10, the overview of the two systems is presented. Number 1 is based on a fixed connection between the shear wall and the foundation. Number 2 uses a spring connection which is based on the rotational spring stiffness of the foundation pile group.

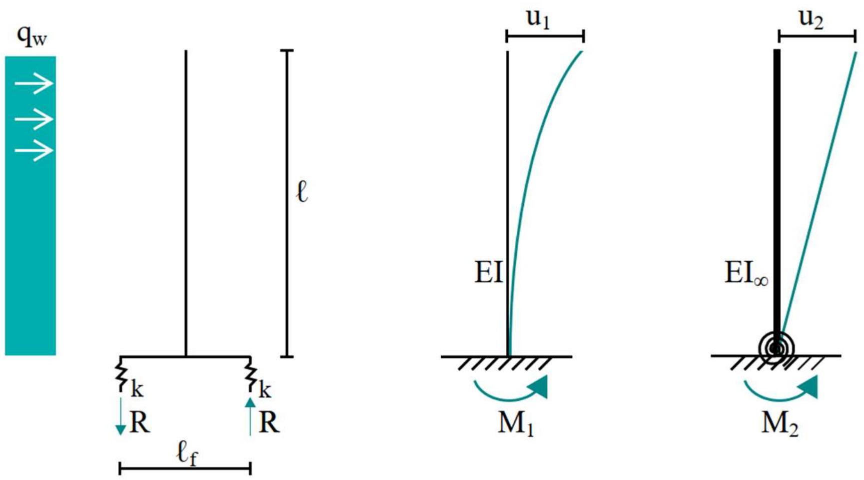


Figure H. 10 Overview of systems

For stability system 1, the calculation is straightforward and uses forget-me-nots:

$$M_1 = \frac{1}{2} q_d \ell^2$$

$$u_1 = \frac{\frac{1}{8} q \ell^4}{EI}$$

Stability system number 2 uses the rotational spring stiffness of the pile foundations C_r. This can be calculated using virtual work:

$$\varphi = \frac{\delta}{\frac{1}{2}\ell_f} = \frac{2\delta}{\ell_f}$$

$$Pile\ capacity: R = k * \delta$$

$$Bending\ moment: M = R * \ell_f$$

$$\rightarrow\ M = k * \delta * \ell_f$$

$$Bending\ moment\ for\ rotation\ spring:$$

$$M = C_r * \varphi$$

$$\rightarrow C_r = \frac{M}{\varphi}$$

$$\rightarrow C_r = \frac{k * \delta * \ell_f}{\frac{2\delta}{\ell_f}}$$

$$\rightarrow C_r = \frac{k * \ell_f^2}{2}$$

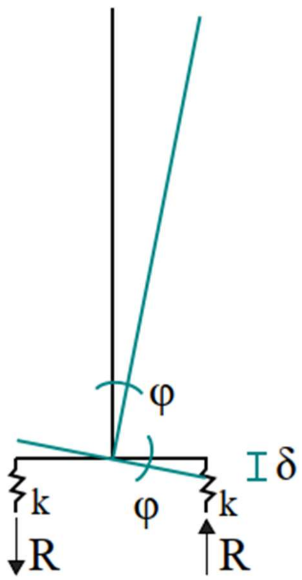


Figure H. 11 Virtual work system

However, the individual pile stiffness k could not be found in the archival calculation. What they did provide is the rotational spring stiffness C_r. Using the calculation with virtual work, the individual pile stiffness is obtained. This is compared to standard values of this stiffness. The results shows that this is relatively low. That is not necessarily a bad thing, because it is very likely that the actual pile stiffness is more, which makes this calculation conservative.

The obtained rotational spring stiffness can be filled into the formula of u_2 to calculate the deflection at the top of the building.

$$u_2 = \ell * \varphi = \ell * \frac{M_{cr}}{C_r} = \ell * \frac{\frac{1}{2} q \ell^2}{C_r} = \frac{q \ell^3}{2 C_r}$$

The results for the optimized location design are summarized in Table H. 11 for part B and Table H. 12 for part C. For the optimized volume design it is Table H. 13 and Table H. 14.The deflection at axis 10 for both designs is the only one which Unity Check (UC) is higher than 0.9. In theory, this sufficient. However, it is on the high side. This is most likely due to the low rotational spring stiffness that is provided by the archival calculations. The shear wall on axis 19 has similar specifications to axis 10. However, the rotational spring stiffness is 58% higher. It is unclear from the archive why that is. It is of course important to find out why this axis is assumed to be this much weaker. However, this amount of difference between the to axes is not realistic.

H 4.2 Deflection of the optimized location design

Table H. 11 Deflection part B

Part B		Axis 10	Axis 19	Axis 12-13	Axis 12-15	Unit
Building height	h	33.3	33.3	33.3	33.3	m
Height shear wall	l	28.2	28.2	28.2	28.2	m
Length arm	l_f	21.6	21.6	7.2	18.4	m
Pile stifness	$k_{v;pile}$	8670	13717	25175	9895	kN/m
Number of piles	n	4	4	4	8	[-]
Pile stiffness group of piles	$k_{v;pilegroup}$	34680	54868	100700	79160	kN/m
Rotational spring stiffness	C_r	8090150.4	12799607	2610144	13400204.8	kNm/rad
Wind load	q_k	42.7	43.5	10.5	20.3	kN/m
E-modulus	E	30000	30000	30000	30000	N/mm^2
Moment of inertia	I	5.985E+13	1.1586E+14	5.59872E+12	6.2295E+13	mm^4
Max allowed deflection	u_{max}	66.6	66.6	66.6	66.6	mm
Deflection system 1	u_1	1.9	1.0	5.0	0.9	mm
Deflection system 2	u_2	59.2	38.1	45.2	17.0	mm
Total deflection	u_{total}	61.1	39.1	50.2	17.8	mm
UC		0.92	0.59	0.75	0.27	-

Table H. 12 Deflection part C

Part C		Axis 19	Axis 27	Axis 21-22	Axis 21-24	Unit
Building height	h	26.5	26.5	26.5	26.5	m
Height shear wall	l	21.4	21.4	21.4	21.4	m
Length arm	l_f	21.6	21.6	7.2	18.4	m
Pile stifness	$k_{v;pile}$	7427	6323	13407	7532	kN/m
Number of piles	n	4	4	4	8	[-]
Pile stiffness group of piles	$k_{v;pilegroup}$	29708	25292	53628	60256	kN/m
Rotational spring stiffness	C_r	6930282.24	5900117.76	1390037.76	10200135.7	kNm/rad
Wind load	q_k	37.7	38.4	9.9	19.0	kN/m
E-modulus	E	30000	30000	30000	30000	N/mm^2
Moment of inertia	I	9.26858E+13	6.67969E+13	5.59872E+12	6.2295E+13	mm^4
Max allowed deflection	u_{max}	53.0	53.0	53.0	53.0	mm
Deflection system 1	u_1	0.4	0.5	1.5	0.3	mm
Deflection system 2	u_2	26.6	31.9	34.8	9.1	mm
Total deflection	u_{total}	27.0	32.4	36.3	9.4	mm
UC		0.51	0.61	0.69	0.18	-

H 4.3 Deflection of the optimized volume design

Table H. 13 Deflection part B

Part B		Axis 10	Axis 19	Axis 12-13	Axis 12-15	Unit
Building height	h	31.6	31.6	31.6	31.6	m
Height shear wall	l	28.2	28.2	28.2	28.2	m
Length arm	l_f	21.6	21.6	7.2	18.4	m
Pile stifness	$k_{v;pile}$	8670	13717	25175	9895	kN/m
Number of piles	n	4	4	4	8	[-]
Pile stiffness group of piles	$k_{v;pilegroup}$	34680	54868	100700	79160	kN/m
Rotational spring stiffness	C_r	8090150.4	12799607	2610144	13400204.8	kNm/rad
Wind load	q_k	40.8	41.6	10.0	21.0	kN/m
E-modulus	E	30000	30000	30000	30000	N/mm^2
Moment of inertia	I	5.985E+13	1.1586E+14	5.59872E+12	6.2295E+13	mm^4
Max allowed deflection	u_{max}	63.2	63.2	63.2	63.2	mm
Deflection system 1	u_1	1.8	0.9	4.7	0.9	mm
Deflection system 2	u_2	56.5	36.4	43.0	17.6	mm
Total deflection	u_{total}	58.3	37.3	47.7	18.5	mm
UC		0.92	0.59	0.75	0.29	-

Table H. 14 Deflection part C

Part C		Axis 19	Axis 27	Axis 21-22	Axis 21-24	Unit
Building height	h	24.8	24.8	24.8	24.8	m
Height shear wall	l	21.4	21.4	21.4	21.4	m
Length arm	l_f	21.6	21.6	7.2	18.4	m
Pile stifness	$k_{v;pile}$	7427	6323	13407	7532	kN/m
Number of piles	n	4	4	4	8	[-]
Pile stiffness group of piles	$k_{v;pilegroup}$	29708	25292	53628	60256	kN/m
Rotational spring stiffness	C_r	6930282.24	5900117.76	1390037.76	10200135.7	kNm/rad
Wind load	q_k	34.9	35.6	10.3	19.8	kN/m
E-modulus	E	30000	30000	30000	30000	N/mm^2
Moment of inertia	I	9.26858E+13	6.67969E+13	5.59872E+12	6.2295E+13	mm^4
Max allowed deflection	u_{max}	49.6	49.6	49.6	49.6	mm
Deflection system 1	u_1	0.3	0.5	1.6	0.3	mm
Deflection system 2	u_2	24.7	29.5	36.2	9.5	mm
Total deflection	u_{total}	25.0	30.0	37.8	9.8	mm
UC		0.50	0.60	0.76	0.20	-

H 4.4 Second order effect

The second order effect has been implemented into the calculation for deflection. First is was checked whether this has to be applied. This is done by performing this check:

$$F_{V,Ed} \leq 0.78 * \frac{n_s}{n_s + 1.6} * \frac{1}{1 + 0.7k} * \frac{EI}{L^2}$$

$$If\ this\ is\ true, 2^{nd} order\ effect\ needs\ to\ be\ applied$$

$$k = \frac{\theta}{M} * \frac{EI}{L} = \frac{EI}{CL}$$

$$n_s = number\ of\ levels$$

$$E_{cracked} = \frac{1}{3} * E = \frac{1}{3} * 30000 = 10000\ N/mm^2$$

$$L = height\ of\ stability\ wall$$

Part B

$n_s = 9$

$L = 28.2\ m$

Table H. 15 Check second order effect Part B

Axis	$C_{archive}$ [kNm/rad]	b_wall	L_wall	I_wall [m4]	k	F	$F_{v,Ed}$	2nd order effect
10	8090000	0.25	21.6	209.95	0.012	1734	6338	Yes
19	12800000	0.2	21.6	167.96	0.006	1393	6338	Yes
12-13	2610000	0.2	7.2	6.22	0.003	52	3361	Yes
12-15	13400000	0.2	18.4	103.83	0.004	862	5142	Yes

Part C

$n_s = 7$

$L = 22.2\ m$

Table H. 16 Check second order effect Part C

Axis	$C_{archive}$ [kNm/rad]	b_wall	L_wall	I_wall [m4]	k	F	$F_{v,Ed}$	2nd order effect
19	6930000	0.25	21.6	209.95	0.014	2678	5509	Yes
27	5900000	0.2	21.6	167.96	0.013	2144	5509	Yes
21-22	1390000	0.2	7.2	6.22	0.006	80	3131	Yes
21-24	10200000	0.2	18.4	103.83	0.006	1332	4553	Yes

Calculation 2nd order value

$$2^{nd} order\ effect = \frac{n}{n-1}$$

$$n = \frac{F_k}{F_{V,Ed}}$$

$$F_k = \frac{\pi^2 EI}{L_k^2}$$

Table H. 17 Calculation 2nd order value

Axis B	Axis C	L_k [m]	I_{wall} [m4]	Fk	$F_{v,Ed}$	n	n/(n-1)
10	10	3.4	0.028	240123	6338	37.9	1.03
19	19	3.4	0.014	122943	6338	19.4	1.05
12-13	12-13	3.4	0.005	40981	3361	12.2	1.09
12-15	12-15	3.4	0.012	104729	5142	20.4	1.05

H.5 Building weight calculations

H 5.1 Building weight calculations optimized location design

Standard middle column		7.2 x		7.2 =			51.84 m2			
		Original situation (conform archive)					New situation (dwellings)			
		Archive calculations (every floor extreme)					2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q		G	Q	G	Q
Roof	9	1.25	1.00	64.80	51.84		1.20	0.00	62.21	0.00
Entresol floor							1.20	0.90	62.21	46.66
Roof + installation/level	8	5.60	2.50	290.30	129.60		6.35	0.90	329.18	46.66
Floor level	7	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	6	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	5	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	4	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	3	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	2	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	1	5.10	2.50	264.38	129.60		5.45	2.25	282.53	116.64
Ground level	0	5.60	4.00	290.30	207.36		5.60	2.25	290.30	116.64
Columns				176					176	
Foundation				95					95	
Ftot,k existing				2767	1296				2868	560
Ftot,k additional									62	0
Ftot,k total					4063					3490
Ftot,d existing (NEN 8700)					5282					4456
Ftot,d additional (Eurocode)					0					84
Ftot,d total					5282					4540
Permanent load: Increase (+) / Decrease (-)				= (2822 + 114) / 2741				163 kN		5.90%
Live load: Increase (+) / Decrease (-)								-736 kN		-56.80%
Characteristic values: Increase (+) / Decrease (-)								-573 kN		-14.10%
Design values: Increase (+) / Decrease (-)								-742 kN		-14.04%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				0 kN
	wind	Design value				0 kN
	Building weight calc.		nr of piles	4	Per pile	1135 kN
						1135 kN
Old situation						
Per foundation pile	wind	Characteristic				0 kN
	wind	Design value				0 kN
	Building weight calc.		nr of piles	4	Per pile	1321 kN
						1321 kN

Standaard outer column		7.2 x		3.85 =		27.72 m2			
		Original situation (conform archive)				New situation (dwellings)			
		Archive calculations (every floor extreme)				2 levels extreme, rest Ψ_0			0.40
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	34.65	27.72	1.20	0.00	33.26	0.00
Entresol floor						1.20	0.90	33.26	24.95
Roof + installation/level	8	5.60	2.50	155.23	69.30	6.35	0.90	176.02	24.95
Floor level	7	5.10	2.50	141.37	69.30	5.45	0.90	151.07	24.95
Floor level	6	5.10	2.50	141.37	69.30	5.45	0.90	151.07	24.95
Floor level	5	5.10	2.50	141.37	69.30	5.45	0.90	151.07	24.95
Floor level	4	5.10	2.50	141.37	69.30	5.45	0.90	151.07	24.95
Floor level	3	5.10	2.50	141.37	69.30	5.45	0.90	151.07	24.95
Floor level	2	5.10	2.50	141.37	69.30	5.45	0.90	151.07	24.95
Floor level	1	5.10	2.50	141.37	69.30	5.45	2.25	151.07	62.37
Ground level	0	5.60	4.00	155.23	110.88	5.60	4.00	155.23	110.88
Columns				107.00				107.00	
Foundation				64.00				64.00	
Canopy				29.00				29.00	
Original façade				514.50				437.30	
New façade								55.00	
Ftot,k existing				2049	693			2081	348
Ftot,k additional								33	0
Ftot,k total					2742				2462
Ftot,d existing (NEN 8700)					3565				3158
Ftot,d additional (Eurocode)					0				45
Ftot,d total					3565				3203
Permanent load: Increase (+) / Decrease (-)				= (2056 + 61) / 2035				65 kN	3.18%
Live load: Increase (+) / Decrease (-)								-345 kN	-49.80%
Characteristic values: Increase (+) / Decrease (-)								-280 kN	-10.21%
Design values: Increase (+) / Decrease (-)								-362 kN	-10.16%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				0 kN
	wind	Design value				0 kN
	Building weight calc.		nr of piles	2	Per pile	1601 kN
						1601 kN
Old situation						
Per foundation pile	wind	Characteristic				0 kN
	wind	Design value				0 kN
	Building weight calc.		nr of piles	2	Per pile	1782 kN
						1782 kN

Stability wall axis 10		14.4 x		3.6 =		51.84 m2			
		Original situation (conform archive)				New situation (dwellings)			
		Archive calculations (every floor extreme)				2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	64.80	51.84	1.20	0.00	62.21	0.00
Entresol floor						1.20	0.90	62.21	46.66
Roof + installation/level	8	5.60	2.50	290.30	129.60	6.35	0.90	329.18	46.66
Floor level	7	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	6	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	5	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	4	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	3	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	2	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	1	5.10	2.50	264.38	129.60	5.45	2.25	282.53	116.64
Ground level	0	5.60	4.00	290.30	207.36	5.60	4.00	290.30	207.36
Concrete wall d=250 mm				2440.00				2440.00	
Foundation				115.00				115.00	
Ftot,k existing				5051	1296			5277	697
Ftot,k additional								0	0
Ftot,k total					6347				5974
Ftot,d existing (NEN 8700)					8251				7766
Ftot,d additional (Eurocode)					0				0
Ftot,d total					8251				7766

Permanent load: Increase (+) / Decrease (-)	= (2056 + 61) / 2035	226	kN	4.46%
Live load: Increase (+) / Decrease (-)		-599	kN	-46.20%
Characteristic values: Increase (+) / Decrease (-)		-373	kN	-5.88%
Design values: Increase (+) / Decrease (-)		-485	kN	-5.88%

Note: No moment allowances applied in archival calculations.

New situation							
Per foundation pile	wind	Characteristic				254	kN
	wind	Design value				381	kN
	Building weight calc.		nr of piles	10	Per pile	777	kN
						1158	kN
Old situation							
Per foundation pile	wind	Characteristic				230	kN
	wind	Design value				345	kN
	Building weight calc.		nr of piles	10	Per pile	825	kN
						1170	kN

Stability wall axis 19		14.4 x		3.6 =			51.84 m2			
		Original situation (conform archive)					New situation (dwellings)			
		Archive calculations (every floor extreme)					2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q		G	Q	G	Q
Roof	9	1.25	1.00	64.80	51.84		1.20	0.00	62.21	0.00
Entresol floor							1.20	0.90	62.21	46.66
Roof + installation/level	8	5.60	2.50	290.30	129.60		6.35	0.90	329.18	46.66
Floor level	7	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	6	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	5	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	4	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	3	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	2	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	1	5.10	2.50	264.38	129.60		5.45	2.25	282.53	116.64
Ground level	0	5.60	4.00	290.30	207.36		5.60	4.00	290.30	207.36
Concrete wall d= 200 mm				1950.00					1950.00	
Foundation				115.00					115.00	
Ftot,k existing				4561	1296				4787	697
Ftot,k additional									0	0
Ftot,k total					5857					5484
Ftot,d existing (NEN 8700)					7614					7129
Ftot,d additional (Eurocode)					0					0
Ftot,d total					7614					7129
Permanent load: Increase (+) / Decrease (-)					= (2056 + 61) / 2035				226 kN	4.94%
Live load: Increase (+) / Decrease (-)									-599 kN	-46.20%
Characteristic values: Increase (+) / Decrease (-)									-373 kN	-6.37%
Design values: Increase (+) / Decrease (-)									-485 kN	-6.37%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				317 kN
	wind	Design value				476 kN
	Building weight calc.		nr of piles	8	Per pile	891 kN
						1367 kN
Old situation						
Per foundation pile	wind	Characteristic				288 kN
	wind	Design value				432 kN
	Building weight calc.		nr of piles	8	Per pile	952 kN
						1384 kN

Stability wall axis 12-13		7.2 x		2 =		14.4 m2			
		Oude situatie (conform archiefberekeningen)				Nieuwe situatie (2 extra bouwlagen)			
		Archiefberekeningen alles extreem gerekend				2 lagen extreem, overig Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	18.00	14.40	1.20	0.00	17.28	0.00
Entresol floor						1.20	0.90	17.28	12.96
Roof + installation/level	8	5.60	2.50	80.64	36.00	6.35	0.90	91.44	12.96
Floor level	7	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	6	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	5	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	4	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	3	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	2	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	1	5.10	2.50	73.44	36.00	5.45	2.25	78.48	32.40
Ground level	0	5.60	4.00	80.64	57.60	5.60	4.00	80.64	57.60
Concrete wall 200 mm				975.00				975.00	
Foundation				115.00				115.00	
Ftot,k existing				1783	360			1846	194
Ftot,k additional								0	0
Ftot,k total					2143				2040
Ftot,d existing (NEN 8700)					2786				2652
Ftot,d additional (Eurocode)					0				0
Ftot,d total					2786				2652
Permanent load: Increase (+) / Decrease (-)				= (2056 + 61) / 2035				63 kN	3.51%
Live load: Increase (+) / Decrease (-)								-166 kN	-46.20%
Characteristic values: Increase (+) / Decrease (-)								-104 kN	-4.84%
Design values: Increase (+) / Decrease (-)								-135 kN	-4.84%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				282 kN
	wind	Design value				423 kN
	Building weight calc.		nr of piles	6	Per pile	442 kN
						865 kN
Old situation						
Per foundation pile	wind	Characteristic				267 kN
	wind	Design value				401 kN
	Building weight calc.		nr of piles	6	Per pile	464 kN
						865 kN

Stability wall axis 12-15		18.4 x		2 =		36.8 m2			
		Original situation (conform archive)				New situation (dwellings)			
		Archive calculations (every floor extreme)				2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	46.00	36.80	1.20	0.00	44.16	0.00
Entresol floor						1.20	0.90	44.16	33.12
Roof + installation/level	8	5.60	2.50	206.08	92.00	6.35	0.90	233.68	33.12
Floor level	7	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	6	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	5	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	4	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	3	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	2	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	1	5.10	2.50	187.68	92.00	5.45	2.25	200.56	82.80
Ground level	0	5.60	4.00	206.08	147.20	5.60	4.00	206.08	147.20
Concrete wall d=200 mm				2490.00				2490.00	
Foundation				115.00				115.00	
Ftot,k existing				4377	920			4537	495
Ftot,k additional								0	0
Ftot,k total					5297				5032
Ftot,d existing (NEN 8700)					6886				6542
Ftot,d additional (Eurocode)					0				0
Ftot,d total					6886				6542
Permanent load: Increase (+) / Decrease (-)				= (2056 + 61) / 2035				160 kN	3.66%
Live load: Increase (+) / Decrease (-)								-425 kN	-46.20%
Characteristic values: Increase (+) / Decrease (-)								-265 kN	-5.00%
Design values: Increase (+) / Decrease (-)								-344 kN	-5.00%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				211 kN
	wind	Design value				317 kN
	Building weight calc.		nr of piles	8	Per pile	818 kN
						1134 kN
Old situation						
Per foundation pile	wind	Characteristic				200 kN
	wind	Design value				300 kN
	Building weight calc.		nr of piles	8	Per pile	861 kN
						1161 kN

H 5.2 Building weight calculations optimized volume design

Standard middle column		7.2 x		7.2 =		51.84 m2			
		Original situation (conform archive)				New situation (dwellings)			
		Archive calculations (every floor extreme)				2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	64.80	51.84	1.20	0.00	62.21	0.00
Roof + installation/level	8	5.60	2.50	290.30	129.60	6.70	0.90	347.33	46.66
Floor level	7	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	6	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	5	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	4	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	3	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	2	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	1	5.10	2.50	264.38	129.60	5.45	2.25	282.53	116.64
Ground level	0	5.60	4.00	290.30	207.36	5.60	2.25	290.30	116.64
Columns				176				176	
Foundation				95				95	
Steel in top level								23.7	
Ftot,k existing				2767	1296			2972	560
Ftot,k additional								0	0
Ftot,k total					4063				3532
Ftot,d existing (NEN 8700)					5282				4592
Ftot,d additional (Eurocode)					0				0
Ftot,d total					5282				4592

Permanent load: Increase (+) / Decrease (-)	= (2822 + 114) / 2741	205 kN	7.41%
Live load: Increase (+) / Decrease (-)		-736 kN	-56.80%
Characteristic values: Increase (+) / Decrease (-)		-531 kN	-13.07%
Design values: Increase (+) / Decrease (-)		-690 kN	-13.07%

Note: No moment allowances applied in archival calculations.

New situation					
Per foundation pile	wind	Characteristic			0 kN
	wind	Design value			0 kN
	Building weight calc.	nr of piles	4	Per pile	1148 kN
					1148 kN
Old situation					
Per foundation pile	wind	Characteristic			0 kN
	wind	Design value			0 kN
	Building weight calc.	nr of piles	4	Per pile	1321 kN
					1321 kN

Standaard outer column		7.2 x		3.85 =			27.72 m2			
		Original situation (conform archive)					New situation (dwellings)			
		Archive calculations (every floor extreme)					2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q		G	Q	G	Q
Roof	9	1.25	1.00	34.65	27.72		1.20	0.00	33.26	0.00
Roof + installation/level	8	5.60	2.50	155.23	69.30		6.70	0.90	185.72	24.95
Floor level	7	5.10	2.50	141.37	69.30		5.45	0.90	151.07	24.95
Floor level	6	5.10	2.50	141.37	69.30		5.45	0.90	151.07	24.95
Floor level	5	5.10	2.50	141.37	69.30		5.45	0.90	151.07	24.95
Floor level	4	5.10	2.50	141.37	69.30		5.45	0.90	151.07	24.95
Floor level	3	5.10	2.50	141.37	69.30		5.45	0.90	151.07	24.95
Floor level	2	5.10	2.50	141.37	69.30		5.45	0.90	151.07	24.95
Floor level	1	5.10	2.50	141.37	69.30		5.45	2.25	151.07	62.37
Ground level	0	5.60	4.00	155.23	110.88		5.60	4.00	155.23	110.88
Columns				107.00					107.00	
Foundation				64.00					64.00	
Canopy				29.00					29.00	
Original façade				514.50					437.30	
New façade									55.00	
Steel in top level									13.80	
Ftot,k existing				2049	693				2138	348
Ftot,k additional									0	0
Ftot,k total					2742					2486
Ftot,d existing (NEN 8700)					3565					3231
Ftot,d additional (Eurocode)					0					0
Ftot,d total					3565					3231
Permanent load: Increase (+) / Decrease (-)				= (2056 + 61) / 2035					89 kN	4.32%
Live load: Increase (+) / Decrease (-)									-345 kN	-49.80%
Characteristic values: Increase (+) / Decrease (-)									-256 kN	-9.35%
Design values: Increase (+) / Decrease (-)									-333 kN	-9.35%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				0 kN
	wind	Design value				0 kN
	Building weight calc.		nr of piles	2	Per pile	1616 kN
						1616 kN
Old situation						
Per foundation pile	wind	Characteristic				0 kN
	wind	Design value				0 kN
	Building weight calc.		nr of piles	2	Per pile	1782 kN
						1782 kN

Stability wall axis 10		14.4 x		3.6 =			51.84 m2			
		Original situation (conform archive)					New situation (dwellings)			
		Archive calculations (every floor extreme)					2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q		G	Q	G	Q
Roof	9	1.25	1.00	64.80	51.84		1.20	0.00	62.21	0.00
Roof + installation/level	8	5.60	2.50	290.30	129.60		6.70	0.90	347.33	46.66
Floor level	7	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	6	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	5	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	4	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	3	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	2	5.10	2.50	264.38	129.60		5.45	0.90	282.53	46.66
Floor level	1	5.10	2.50	264.38	129.60		5.45	2.25	282.53	116.64
Ground level	0	5.60	4.00	290.30	207.36		5.60	4.00	290.30	207.36
Concrete wall d=250 mm				2440.00					2440.00	
Foundation				115.00					115.00	
Ftot,k existing				5051	1296				5233	651
Ftot,k additional									0	0
Ftot,k total					6347					5883
Ftot,d existing (NEN 8700)					8251					7648
Ftot,d additional (Eurocode)					0					0
Ftot,d total					8251					7648
Permanent load: Increase (+) / Decrease (-)					= (2056 + 61) / 2035				181 kN	3.59%
Live load: Increase (+) / Decrease (-)									-645 kN	-49.80%
Characteristic values: Increase (+) / Decrease (-)									-464 kN	-7.31%
Design values: Increase (+) / Decrease (-)									-603 kN	-7.31%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				234 kN
	wind	Design value				351 kN
	Building weight calc.		nr of piles	10	Per pile	765 kN
						1116 kN
Old situation						
Per foundation pile	wind	Characteristic				230 kN
	wind	Design value				345 kN
	Building weight calc.		nr of piles	10	Per pile	825 kN
						1170 kN

Stability wall axis 19		14.4 x		3.6 =		51.84 m2			
		Original situation (conform archive)				New situation (dwellings)			
		Archive calculations (every floor extreme)				2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	64.80	51.84	1.20	0.00	62.21	0.00
Roof + installation/level	8	5.60	2.50	290.30	129.60	6.70	0.90	347.33	46.66
Floor level	7	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	6	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	5	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	4	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	3	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	2	5.10	2.50	264.38	129.60	5.45	0.90	282.53	46.66
Floor level	1	5.10	2.50	264.38	129.60	5.45	2.25	282.53	116.64
Ground level	0	5.60	4.00	290.30	207.36	5.60	4.00	290.30	207.36
Concrete wall d= 200 mm				1950.00				1950.00	
Foundation				115.00				115.00	
Ftot,k existing				4561	1296			4743	651
Ftot,k additional								0	0
Ftot,k total					5857				5393
Ftot,d existing (NEN 8700)					7614				7011
Ftot,d additional (Eurocode)					0				0
Ftot,d total					7614				7011

Permanent load: Increase (+) / Decrease (-)	= (2056 + 61) / 2035	181 kN	3.98%
Live load: Increase (+) / Decrease (-)		-645 kN	-49.80%
Characteristic values: Increase (+) / Decrease (-)		-464 kN	-7.92%
Design values: Increase (+) / Decrease (-)		-603 kN	-7.92%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				293 kN
	wind	Design value				440 kN
	Building weight calc.		nr of piles	8	Per pile	876 kN
						1316 kN
Old situation						
Per foundation pile	wind	Characteristic				288 kN
	wind	Design value				432 kN
	Building weight calc.		nr of piles	8	Per pile	952 kN
						1384 kN

Stability wall axis 12-13		7.2 x		2 =		14.4 m2			
		Oude situatie (conform archiefberekeningen)				Nieuwe situatie (2 extra bouwlagen)			
		Archiefberekeningen alles extreem gerekend				2 lagen extreem, overig Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	18.00	14.40	1.20	0.00	17.28	0.00
Roof + installation/level	8	5.60	2.50	80.64	36.00	6.70	0.90	96.48	12.96
Floor level	7	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	6	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	5	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	4	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	3	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	2	5.10	2.50	73.44	36.00	5.45	0.90	78.48	12.96
Floor level	1	5.10	2.50	73.44	36.00	5.45	2.25	78.48	32.40
Ground level	0	5.60	4.00	80.64	57.60	5.60	4.00	80.64	57.60
Concrete wall 200 mm				975.00				975.00	
Foundation				115.00				115.00	
Ftot,k existing				1783	360			1834	181
Ftot,k additional								0	0
Ftot,k total					2143				2014
Ftot,d existing (NEN 8700)					2786				2619
Ftot,d additional (Eurocode)					0				0
Ftot,d total					2786				2619
Permanent load: Increase (+) / Decrease (-)				= (2056 + 61) / 2035			50 kN		2.83%
Live load: Increase (+) / Decrease (-)							-179 kN		-49.80%
Characteristic values: Increase (+) / Decrease (-)							-129 kN		-6.01%
Design values: Increase (+) / Decrease (-)							-168 kN		-6.01%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				286 kN
	wind	Design value				429 kN
	Building weight calc.		nr of piles	6	Per pile	436 kN
						865 kN
Old situation						
Per foundation pile	wind	Characteristic				267 kN
	wind	Design value				401 kN
	Building weight calc.		nr of piles	6	Per pile	464 kN
						865 kN

Stability wall axis 12-15		18.4 x		2 =			36.8 m2		
		Original situation (conform archive)				New situation (dwellings)			
		Archive calculations (every floor extreme)				2 levels extreme, rest Ψ_0		0.40	
		G	Q	G	Q	G	Q	G	Q
Roof	9	1.25	1.00	46.00	36.80	1.20	0.00	44.16	0.00
Roof + installation/level	8	5.60	2.50	206.08	92.00	6.70	0.90	246.56	33.12
Floor level	7	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	6	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	5	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	4	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	3	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	2	5.10	2.50	187.68	92.00	5.45	0.90	200.56	33.12
Floor level	1	5.10	2.50	187.68	92.00	5.45	2.25	200.56	82.80
Ground level	0	5.60	4.00	206.08	147.20	5.60	4.00	206.08	147.20
Concrete wall d=200 mm				2490.00				2490.00	
Foundation				115.00				115.00	
Ftot,k existing				4377	920			4506	462
Ftot,k additional								0	0
Ftot,k total					5297				4968
Ftot,d existing (NEN 8700)					6886				6458
Ftot,d additional (Eurocode)					0				0
Ftot,d total					6886				6458
Permanent load: Increase (+) / Decrease (-)					= (2056 + 61) / 2035		129 kN		2.94%
Live load: Increase (+) / Decrease (-)							-458 kN		-49.80%
Characteristic values: Increase (+) / Decrease (-)							-329 kN		-6.22%
Design values: Increase (+) / Decrease (-)							-428 kN		-6.22%

Note: No moment allowances applied in archival calculations.

New situation						
Per foundation pile	wind	Characteristic				214 kN
	wind	Design value				321 kN
	Building weight calc.		nr of piles	8	Per pile	807 kN
						1128 kN
Old situation						
Per foundation pile	wind	Characteristic				200 kN
	wind	Design value				300 kN
	Building weight calc.		nr of piles	8	Per pile	861 kN
						1161 kN

H 5.3 Check stresses in concrete columns and shear walls due to vertical forces

C20/25 $\sigma_{allowed}$= 17.7 *=25*0.85/1.2* [N/mm^2]

Optimized volume design	F_k [kN]*	F_d [kN]*	b [mm]	h [mm]	A [mm^2]	σ [N/mm^2]	$\sigma_{allowed}$ [N/mm^2]	UC
Inside column	3532	4592	500	700	350000	13.1	17.7	0.74
Perimeter colum	2486	3231	500	600	300000	10.8	17.7	0.61
Stab wall axis 10	5883	7648	250	21600	5400000	1.4	17.7	0.08
Stab wall axis 19	5393	7011	200	21600	4320000	1.6	17.7	0.09
Stab wall axis 12-13	2014	2619	200	7200	1440000	1.8	17.7	0.10
Stab wall axis 12-15	4970	6460	200	18400	3680000	1.8	17.7	0.10

Optimized location design	F_k [kN]*	F_d [kN]*	b [mm]	h [mm]	A [mm^2]	σ [N/mm^2]	$\sigma_{allowed}$ [N/mm^2]	UC
Inside column	3532	4592	500	700	350000	13.1	17.7	0.74
Perimeter colum	2486	3231	500	600	300000	10.8	17.7	0.61
Stab wall axis 10	5974	7766	250	21600	5400000	1.4	17.7	0.08
Stab wall axis 19	5484	7129	200	21600	4320000	1.7	17.7	0.09
Stab wall axis 12-13	2040	2652	200	7200	1440000	1.8	17.7	0.10
Stab wall axis 12-15	5032	6542	200	18400	3680000	1.8	17.7	0.10

Part B is governing, so for both designs only part B is checked
*Values from building weight calculations used

H 5.4 Check stresses in concrete columns and shear walls due to bending moments

C20/25 $\sigma_{allowed}$= 17.7 *=25*0.85/1.2* [N/mm^2]

$$\sigma_{max} = M/W$$

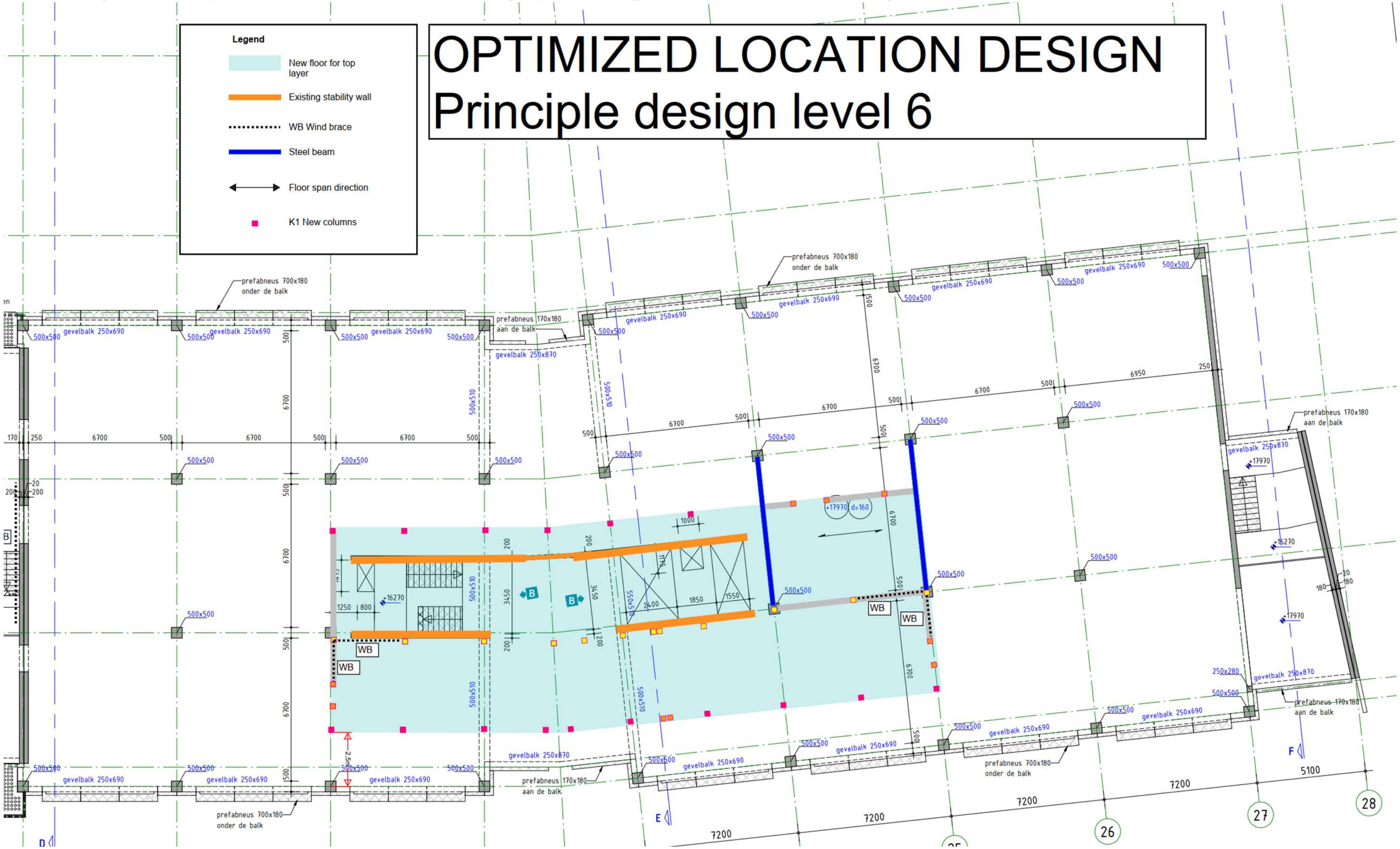
$$W = \frac{1}{6}bh^2$$

Optimized volume design	M_k [kNm]*	M_d [kNm]*	b [mm]	h [mm]	W [mm^3]	σ [N/mm^2]	$\sigma_{allowed}$ [N/mm^2]	UC
Stab wall axis 10	16876	25313	250	21600	1.94E+10	1.3	17.7	0.07
Stab wall axis 19	16876	25313	200	21600	1.56E+10	1.6	17.7	0.09
Stab wall axis 12-13	4128	6191	200	7200	1.73E+09	3.6	17.7	0.20
Stab wall axis 12-15	8256	12382	200	18400	1.13E+10	1.1	17.7	0.06

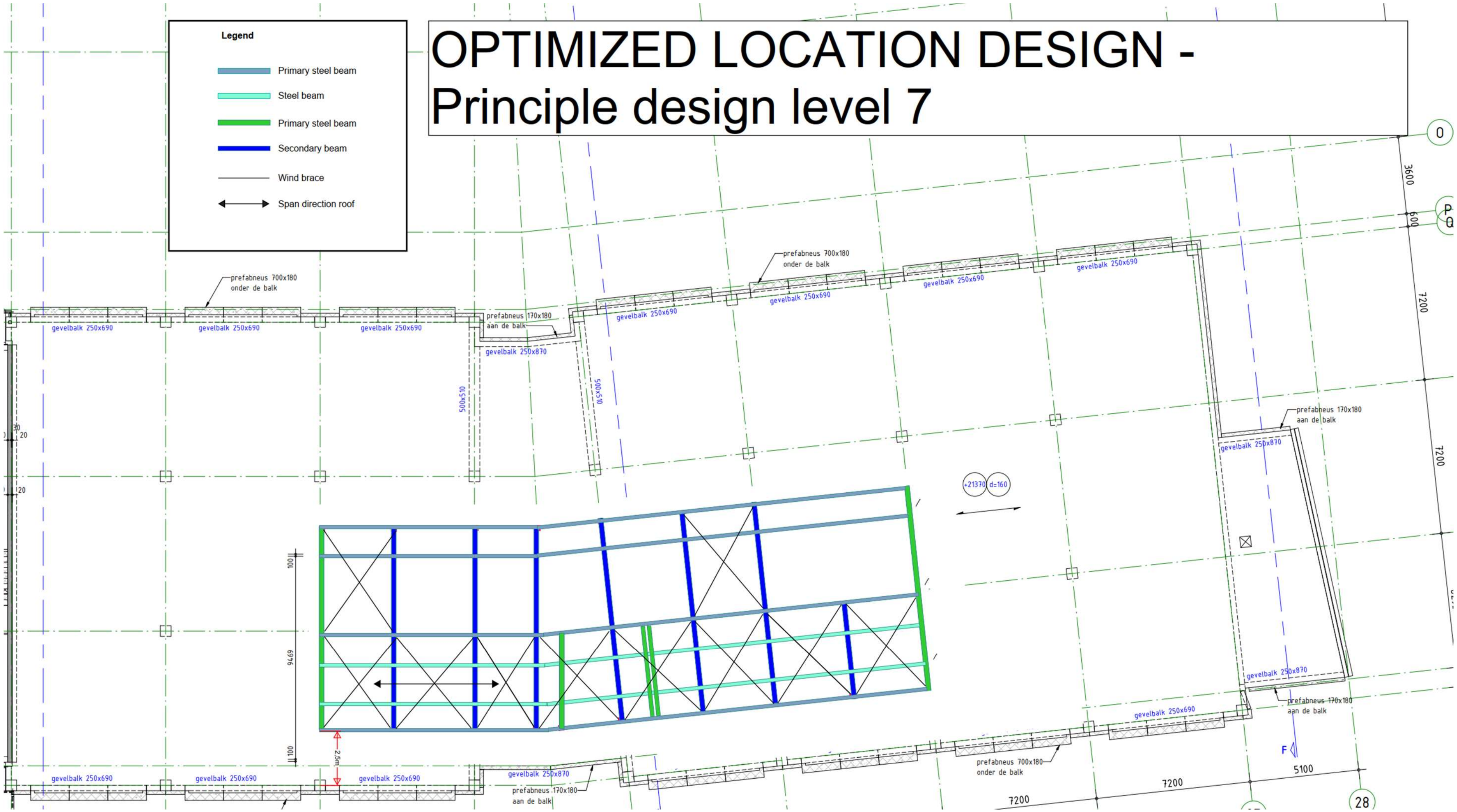
Optimized location design	M_k [kNm]*	M_d [kNm]*	b [mm]	h [mm]	W [mm^3]	σ [N/mm^2]	$\sigma_{allowed}$ [N/mm^2]	UC
Stab wall axis 10	18281	27421	250	21600	1.94E+10	1.4	17.7	0.08
Stab wall axis 19	18281	27421	200	21600	1.56E+10	1.8	17.7	0.10
Stab wall axis 12-13	4059	6089	200	7200	1.73E+09	3.5	17.7	0.20
Stab wall axis 12-15	8118	12178	200	18400	1.13E+10	1.1	17.7	0.06

Part B is governing, so for both designs only part B is checked
*Values from building weight calculations used

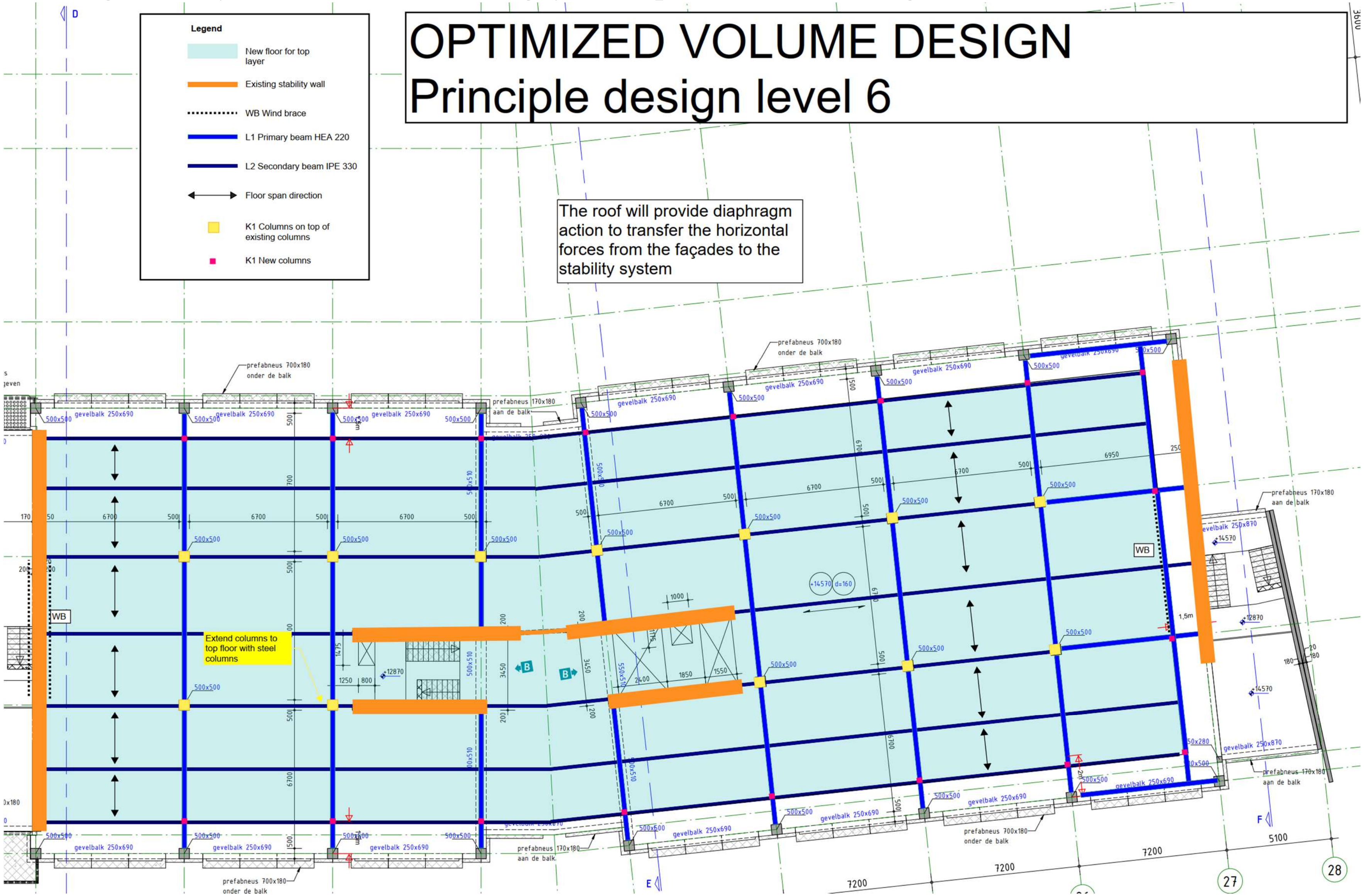
H.6 Design stability and vertical load bearing system | optimized location design



OPTIMIZED LOCATION DESIGN - Principle design level 7



H.7 Design stability and vertical load bearing system | optimized volume design



OPTIMIZED VOLUME DESIGN - Principle design level 7

The roof will provide diaphragm action to transfer the horizontal forces from the façades to the stability system