Tampere University

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VIBRATION OF A TALL BUILDING UNDER WIND INCIDENCE

European Wind Code EN 1991-1-4 Procedure Study

Faculty of Built Environment
Master's Thesis
Tampere, May 2019

Abstract

Vibration of a Tall Building under Wind Incidence
Faculty of Built Environment
Department of Civil Engineering
Master's Thesis
Master in Industrial Engineering
Professor *Sami Pajunen*
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In this thesis, a study is made about the correction and precision that EN 1991-1-4 has in the determination of vibrations in buildings induced by wind loading. A comparison is made between the existing procedure in the European Wind Code with respect to the rest of the most relevant wind codes and standards. In addition, the acceleration produced in tall buildings due to the incidence of wind EN 1991-1-4 determination procedure is studied. The CAARC Standard Tall Building is selected for the study because its dimensions and properties are known and standardized.

The study is carried out with the intention of making two main comparisons. The first is regarding to the treatment used by the European Wind Code about the two existing cases of wind incidence on buildings: the case with vertical correlation and the case with horizontal and vertical correlation. The latter is the most approximate representation of what happens in the reality of wind incidence on a building. Thereby, the second comparison developed is in order to obtain the acceleration through EN 1991-1-4 and by means of an FEM analysis.

It is concluded that there is a possibility of standardizing a wind code among the main countries mainly related to the along-wind vibration similar approaches. However, the results about the European Wind Code are satisfactory concerning the differentiation that it presents in relation to the two cases of horizontal correlation of the wind loads incidence discretized by the aerodynamic admittance function. Finally, there was an enormous parity noted in the acceleration results obtained by means of the FEM analysis with respect to those approved by the code, granting consistency, precision and verisimilitude to its established procedure. This study is expected to lay the groundwork to further research where the variables considered and examined in this analysis are altered.

Keywords: EN 1991-1-4, wind loads, structure dynamics, building acceleration, tall building, CAARC Building, wind code and standard.

The originality of this thesis has been checked by Turnitin Originality Check service.

Preface

This thesis has been developed from October 2018 to May 2019 during my educational stay in Tampere. The Professor Sami Pajunen of Tampere University offered me the possibility of conducting this project. The initial intention and consequent proposal was to work together with Doctoral Researcher Olli Lahti in the deepening in the existing procedures of EN 1991-1-4 vibrations study.

It is therefore essential for me to place on record my enormous gratitude to Olli for having been that patient and generous, always willing to solve all the doubts that have arisen. In the same way, I would like to thank Sami for all the help provided and the time he has dedicated to me, bringing light to all the concerns related to the elaboration of this thesis.

I am forever grateful to Alfonso Durán for trusting in me, giving me a position in the Department of Industrial Management at Carlos III University and having been enormously sympathetic to my decision to spend the second year of Master's Degree in Finland.

I would also like to thank my brothers Roberto and Ricardo for their support, I can only speak well of them and am thankful for all the things they have taught me. You can not ask anything more of their figure as a big brother.

To my parents. I dedicate this Master´s Thesis to them as a conclusion to my six year studies. I will be eternally grateful to them just for being the way they are.

Lastly, I wanted to express my utmost gratitude to my grandmother Elena, for being the most wonderful person I know.

Tampere, 10.5.2019

Juan José Ruiz Lucas

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List of Abbreviations, Symbols and Acronyms

α	sixty percent of gust speed coefficient
α'	terrain dependent factor for general power wind velocity profile law
α''	turbulence length related constant
α_0	dimensionless parameter that measures that degree of participation of overall flexural and overall shear lateral deformations in the building
α_R	Rayleigh first coefficient that affects mass
β	gust speed coefficient
β_R	Rayleigh second coefficient that affects stiffness
ΔP	pressure difference
Δt	time differentials
δ	total logarithmic decrement
δ_a	logarithmic decrement of aerodynamic damping for the fundamental mode
δ_d	logarithmic decrement of damping due to special devices
δ_s	logarithmic decrement of structural damping
η_b	aerodynamic admittance horizontal function factor
η_h	aerodynamic admittance vertical function factor
$\frac{1}{2}\rho V^2$	dynamic pressure
$\hat{\ddot{\theta}}(z)$	torsion acceleration
$\hat{\ddot{x}}_{max}$	along-wind acceleration

$\hat{\ddot{y}}(z)$	across-wind acceleration
$\hat{M}_R$	resonant component of the across-wind moment
$\hat{M}_T$	resonant component
μ_{ref}	reference mass per unit area defined by the following expression
ν	poisson ratio
ν	up-crossing rate
$\phi_1(z)$	first mode shape evaluated at height z
ϕ_{max}	mode shape value at the point with maximum amplitude
ψ_r	reduction factor for square sections with rounded corners that depends on Reynolds number
ψ_λ	end-effect factor for elements with free-end flow
ρ	air density
ρ_c	specific mass of the CAARC building
σ	standard deviation
σ_v	standard deviation of wind velocity
$\sigma_{a,x}$	standard deviation of the characteristic along-wind acceleration
$\mathbf{L}$	lower triangular matrix of the spectral matrix
$\tilde{q}_j$	force Spectra of the simulated wind loads with applied randomness in time domain
$\tilde{S}$	force Spectra of the simulated wind loads with applied randomness in frequency domain
ε	damping ratio
ζ	exponent of the mode shape
ζ'	damping ratio
ζ_{sj}	force spectra randomness value
${}_vc$	vertical correlation subscript
a_G	actor of galloping instability
a_{EN}	along-wind acceleration studied by EN 1991-1-4 of a CAARC building
A_{ref}	reference area
a_{sj}	randomness component

B^2 background factor

b_{sj} randomness component

C_0 orography factor

C_D Drag along-wind coefficient

C_d dynamic factor

C_e exposure factor

C_f force coefficient

C_L Drag across-wind coefficient

C_P pressure coefficient

$C_r(z_s)$ is the roughness factor for the reference height

C_sC_d structural factor

C_s size factor

C_z exponential decay

$C_{direction}$ direction factor

C_{dir} directional factor

C_{dyn} dynamic response factor

$C_{exposure}$ exposure factor

$C_{f,0}$ force coefficient of rectangular sections with sharp corners and without free-end flow

C_{fx} drag force coefficient

$C_{importance}$ importance factor

c_{jo} all velocity factor related constant, e.g., orography or terrain factor

C_{other} other factors

C_{prob} probability factor

C_{season} season factor

$C_{topography}$ topography factor

dA differential surface area

E_y Elastic Module for the flexure around y-axis

f_0 natural frequency

f_L	reduced frequency
F_w	wind induced force
G_q	wind velocity pressure gust factor
G_y	mode shape variation along the horizontal y-axis constant
G_z	mode shape variation along the horizontal z-axis constant
h_{ref}	reference height
I_0	mass moment of inertia
$I_v(z)$	turbulence intensity
I_y	inertia moment of a rectangle respect to the y-axis
I_z	turbulence intensity
K_l	turbulence factor
k_p	peak factor
K_r	terrain factor depending on the roughness length
$K_s(n)$	size reduction function
K_t	torsional moment
K_x	non dimensional coefficient along- wind acceleration factor
K_y	mode shape correction
K_y	standard deviation of the characteristic along-wind acceleration EN 1991 Annex C constant
K_z	standard deviation of the characteristic along-wind acceleration EN 1991 Annex C constant
m_0	mass per height
m_1	generalized mass
$m_{1,x}$	along wind fundamental equivalent mass
n_0	first mode natural frequency
N_e	number of elements
n_e	exponent with the value of the probability factor
n_n	frequency set among the higher frequencies of the modes that contribute significantly to the dynamic response
N_t	intervals of time

$n_{i,y}$ natural frequency of the considered flexural mode i of cross-wind vibration

p_z wind pressure

q_b basic velocity pressure

q_D along-wind wind stream net force

q_L across-wind wind stream net force

q_p peak dynamic pressure

q_z velocity pressure at height z

$q_{h_{ref}}$ velocity pressure at the reference height

R^2 resonance response factor

R_b aerodynamic admittance horizontal function

R_h aerodynamic admittance vertical function

r_{12} distance between two load points

$R_{v'}$ auto-correlation of fluctuating velocity

$S_L(z,n)$ spectral density function

$S_v(z,n)$ one-sided variance spectrum

$S_v(z,n)$ spectral density function

$S_{p'}$ Power spectral density function of wind pressure

$S_{p1'p2'}$ cross-spectrum of wind pressures

$S_{q,jk}$ force Spectra of the simulated wind loads

$S_{q1'q2'}$ cross-spectrum of wind forces

S_q spectral matrix of the force simulated loads

$S_{v'}$ Power spectral density function of $v'(z,t)$

Sc Scruton number

T_p total time

u^* friction velocity

v' fluctuating time dependent velocity component at reference height z

v_0 basic wind speed

v_b basic wind velocity

v_m mean velocity component at reference height z

$V_{b,0}$ fundamental value of the basic wind velocity

V_{CG} special onset wind velocity

V_{crit} critical wind velocity

V_{z_G} gradient wind speed

V_z wind speed at height z

x_i damping ratio to critical

x_{max} maximum displacement

z_0 roughness length

z_G gradient height

$z_{0,II}$ roughness length of terrain category II

z_{min} minimum reference height

z_s reference height

ABAQUS software suite for finite element analysis and computer-aided engineering

CAARC Commonwealth Advisory Aeronautical Council

AAF aerodynamic admittance function

ABS atmospheric boundary layer

AIJ Architecture Institute of Japan

AMD Active Mass Driver

AS/NZ Australia and New Zealand Standard

ASCE American Society Of Civil Engineers. USA Standard

ATMD Active Tuned Mass Damper

b building width

b' terrain dependent factor for general power wind velocity profile law

BBM base bending moment

C damping matrix

c' variable related to the exposure category

CFD Computational Fluid Dynamics

CFX10 turbulent model program

CNS China National Standard

Coh Coherence function

CWE) Computational Wind Engineering

d building along-wind length

d' variable related to the exposure category

E energy factor

EC exposure category

ECS European Committee for Standardization

ER Electro-Rheological

EUC European Code

Ex. Expression

FE Finite Element

FEM Finite Element Model

Fig. Figure

G gust effect factor

g peak factor

GFT Gust Factor Technique

GLF gust loading factor

GWL generalized wind load

h height

IAWE International Association of Wind Engineering

ISO International Organization for Standardization

IWC Indian Wind Code

K mode shape correction factor

K stiffness

K' shape parameter depending on the coefficient of variation of the extreme-value distribution of the probability factor

L(z) turbulence length scale

LES large eddy simulation

MDOF multi-degree of freedom

MR Magneto-Rheological

N number of nodes

n frequency

NAL NatHaz Aerodynamic Loads Data-bas

NBCC National Building Code Canada

NRC National Research Council

P pressure

p probability on annual exceeding

p'(z,t) fluctuating time dependent component of total wind pressure at height z

p(z,t) total time dependent wind pressure at height z

r time value for the interval

Re Reynolds number

RMS root mean square

S size reduction factor

s frequency value

sec seconds

SST shear stress transport

St Strouhal number

T averaging time

t time

TLCD Tuned Liquid Column Dampers

TMD Tuned Mass Damper

TSWD Tuned Sloshing Water Dampers

v(z,t) total time dependent wind speed at reference height z

z height coordinate

Chapter 1

Introduction

This chapter is the initial presentation of the project. It is a general clarification of what is the origin and interest of this piece of work. The current situation is detailed with the baseline requirements followed by the methodology and final objective. To conclude, a brief summary of the chapters is outlined as well as the interrelation that the different sections have in order to follow the thread that leads to the conclusions of the project.

1.1 Background and Motivation

This project arises from the intention of carrying out an in-depth study of the European Standard Wind Code over buildings EN 1991-1-4 on the determination of acceleration in large buildings. The chosen structure has been a standardized building called CAARC Standard Tall Building. Hence, for being able to make a proper and consistent comparison, there has also been a comparative implementation of an acceleration analysis of this selected CAARC Building in the Finite Element (FE) ABAQUS program. The work emerges in a situation of construction globalization and overproduction in which the proliferation of new constructions of increasing size seem incessant with the constant flow of people from the rural areas towards the great cities, and thus, buildings increasing size and number.

The EN 1991-1-4 Code will be analyzed entirely. It will start with a brief summary of the origin of the code, as well as its range of use. Therefore, it will also be evaluated possible alternative uses, and finally, the considerations of common adoption all along European countries, thanks to the specificities that it has in its Annex specifications. There will also be a consideration of the related codes that must be used in a parallel procedure to EN1991-1-4. Furthermore, it will be studied the wind load model, the peak wind pressure considerations, the National Annex (mainly used for the different countries wind or terrain particularities) and, above all, the examinations related to the dynamics of structures. This last element will be the most pertinent, as the study focuses on the determination of

the acceleration produced in large wind-induced buildings.

The intention is to determine the goodness of the code, the security it possesses, how conservative it is, the precision its analysis has and how reliable it is to reality. This mentioned reality will be modelled and simulated thanks to the FE ABAQUS program, focused on the study of the time dependent acceleration produced as well as its peaks in a large building. Therefore, the analysis will be carried out by the European Wind Code and the simulation FE program in a comparative approach.

It will also be necessary to carry out a comparative analysis between the main used and relevant wind codes in relation to building under wind incidence. With the help of this comparison, assessing their points in common as well as their differences, it will proceed to continue with the analysis of the EN 1991-1-4 Code. With the accomplishment of the Standards comparison, the way of considering influential elements in the study such as an average wind speed, wind field characteristics, turbulence intensity profiles, wind spectrum or turbulence length all mainly focused on the different ways of determining the acceleration in all this codes. Therefore, despite having clear similarities, also presents some significant differences, and the interest is to see the influence over the global analysis of the acceleration due to the incidence of wind in buildings structures. Hence, in the pretension of increasing the reliability of the benchmark, a relevant comparative study by Kwon and Kareem (2013) will be followed. Most of the codes have already been revised in the last years, but this does not eliminate the need for an in-depth analysis and make a comparison of the updated versions.

The motivation or main interest of the work is to respond to the quality of a code that has been applied in Europe since 2010. A study of the usefulness, and how reliable the study is by referring to the main book studies and most influential and respectable articles that affect and can give clarity and guidance to the resolution of the issue in question. Therefore, to corroborate to what extent the European Wind Code should be revised, or what are some of its main shortcomings. It is only with this in-depth analysis of a specific acceleration of tall building case that one can see the main pros and cons of this European Standard Wind Code over buildings wind dynamics carried out in comparison with other major international wind codes.

The whole study will be done with a division of two main cases in which the Code considers using the aerodynamic admittance function. A first simplified case in which there will only be vertical correlation and a case that is pretended to be more representative of the real situation in which there will be vertical and horizontal correlation. In this way it will also be possible to see how important it is to preserve the horizontal correlation with reality, and how the study is preserved in both cases.

The project is carried out cooperatively with the Doctoral Researcher of Structural Engineering Olli Lahti, who oversees the process of obtaining the values of the wind simulation dependent on time and discretized according to a series of nodes. The whole project is supervised by Civil Engineering Professor Sami Pajunen.

1.2 Objectives and Methodology

Following the antecedents or background previously exposed, it can be claimed that the main objective of this project is the determination of the correctness and precision of the European Wind Code EN 1991-1-4 in terms of the determination of the vibrations produced by the wind on a tall building. However, a series of secondary objectives can also be considered, such as a comparative analysis with other wind codes, a study of the limitations of the EN 1991-1-4 in a general way or a comparison between the two study cases of wind loads incising on a building, the simplified and the real, which are going to be analyzed in order to expose the goodness of the simplification.

For the Finite Elements FE study, ABAQUS program has been selected due to its calculation capacity, ease of use and plausibility in the results, since it represents reality in a reliable way.

In order to meet the objectives, a methodology has been followed with various stages mostly defined.

To begin with, a compilation study has been carried out throughout the main books and studies on the impact of wind on buildings. In the same way, it has investigated the state of art of the case in question, attending to the main publications and pertinent existing articles that were focused in relation to the study in question. All this research have served for the situation in the topic in question, and thus, starting point to the study carried out in this project. After this first step, it has been pertinent to carry out an in-depth study of the European Wind Code on buildings EN 1991-1-4. This study has not only been centered in the calculation of the acceleration produced by the incidence of the wind on buildings, but has also gone beyond everything related to the code as each section or annex has been treated.

Next, and in conjunction with this previous stage, the rest of the main Standards related to wind incidence on buildings have been investigated. Therefore, it has been possible to have a global image of how the study of acceleration is approached in the codes, continuing in the deepening of the subject in question.

Hence, the calculation of the acceleration produced by the wind according to the EN 1991-1-4 has been carried out, which is the first part of the study. This parts appear to be crucial as it must then be compared with the approximation made thanks to the numerical calculation with the FEM analysis. This study of acceleration has been carried out for the two main cases of the study such as the simplified one with only vertical correlation and the realistic one with vertical and horizontal correlation obtaining to different standard deviation distributions of the acceleration of the studied building.

After that, the study with the FE approximation has been carried out for the real case with an extrapolation aims focus as a future development for the only vertical

correlation case. Olli Lahti helped to determine the time- dependent wind values for each of the nodes of the two cases studied. In this sense, the author's work has been as a compilation attitude.

For the FEM development, along with the inherited values of wind, it was necessary to determine all the characteristics of a CAARC Standard Tall Building. This is the determination of its geometry, the relevant discretization in relation to the nodes in which the wind is applied uniformly as the horizontal correlation wind load decrease is considered by the use of the aerodynamic admittance function, determination of the properties such as the elastic properties, the density of the building and the total damping it possesses and thereby ensure the stiffness and natural frequency or mode shape of the type of building selected to be studied. According to this, it has been studied the values of all these variables that affect the building, as well as how they should be introduced into the FE ABAQUS program.

With all these factors and their introduction into the program already determined, it has been proceeded to make the study with the FE ABAQUS program. Due to the randomness of the wind simulated loads, there have been done 10 simulations for a statistical result approach. Hence, the acceleration standard deviation of the several simulations were obtained for the horizontal and vertical correlation case or the realistic representation case. With this study, it has been possible to proceed to the comparison with EN 1991-1-4 previously obtained acceleration standard deviation value. The comparison has been made with a critical attitude, observing which are the major deficiencies of the code and to what extent these deficiencies are acceptable. The main objective of this analysis of results has been in relation to the EN 1991- 1-4 Wind Code as it was intended to assess.

Finally, a series of main and resounding conclusions have been presented, followed by some of the possible future developments or research that appear to be interesting to continue with after this thesis.

1.3 Document Structure

In this section it will be carry out a structuring of the chapters of the project, to clarify the mode of attainment of the objectives in the report. You can see the common thread that has been followed to obtain the results.

The first chapter, is a brief introduction to the work that aims to give a clear and consistent idea of the project. The backgrounds to the project are commented along with the motivation and interest that it possesses and, finally, the methodology to reach the defined objectives is stated.

The second chapter introduces the theoretical framework and the state of art of the project in relation, firstly, with wind loads, followed by wind induced dynamics over buildings related to the differentiation between along and across-wind dynamics,

simulation tests under use, damping effects and human response considerations, then, there is a CAARC Building definition section and a compilation of main related studies section to conclude the chapter.

Furthermore, third chapter has an analog objective than chapter two, but in this case, the main objective is to establish all the theoretical framework related to the European Wind Standard EN 1991-1-4 and the comparison between the main used Standard Codes. The study of the EN 1991-1-4 will be done attending to wind loads, building dynamics under wind incidence and also the determination of the National Annex use. In the case of the comparison between Codes, it is done in relation not only of acceleration but also attending to the rest of variables that appear to be of interest.

The forth chapter comes as the main chapter of the project as it is where the real study development of acceleration takes place. In this sense it has been separated in three sections. First of the forth chapter section is an initial analysis of the Wind Loads Simulated values determination, CAARC Building considerations for the introduction to a FE analysis of an acceleration due to wind inducted loads and, finally, all the needed EN 1991-1-4 considerations that need to be taken into account before making the study. The second section of this chapter is the development of the study. Therefore, acceleration is firstly developed and obtained by the EN 1991-1-4 for the simplified situation and, then, the real case is carried out by the European Standard and FEM-study procedures. To conclude this chapter there is a result comparison between both approaches of acceleration and between the two simulation of wind incidence cases.

Finally, chapter five sets out the conclusions of all the chapters as well as the introduction of some developments or research that should take place in the future and can continue after this study of the acceleration of a CAARC Building under wind incidence in the European Wind Standard EN 1991-1-4 procedure.

Chapter 2

Wind Loads, Dynamics and CAARC Building Definition

This chapter introduces and set forth state of art wind loads, its dynamics over buildings and the definition of the CAARC Standard Tall Building, concluding with a compilation of the most resounding and related to the topic studies.

To begin with, we will go through wind loads considerations and the different use cases approaches. For this, we will take into consideration the two boundary layer wind loads z-direction distribution cases and, above all, the difference between the two approaches of wind incidence over a building that are studied in this thesis. This includes, the vertical correlated case and the vertical and horizontal case. Therefore, this first section is crucial in order to understand the thesis.

On the other hand and as a continuation to previous section, building dynamics will also be treated. This includes, a first separation of the along and cross-wind dynamics, some damping effects considerations, main building dynamics simulations due to wind incidence procedures under use and, to conclude, It's important to highlight some human response aspects due to wind loads, as the main project objective is the determination of acceleration in tall buildings.

To continue, some aspects of a CAARC Standard Tall Building will be considered. Therefore, all CAARC building sources and important standardized values will be treated, concluding with a comparative study reference of different simulated flows.

To conclude this chapter, some of the main wind loading over buildings studies are gathered and referenced, in order to make a state of art of the work that could serve as a starting point determining CAARC Standard Tall Building acceleration. Which is the main objective to be conceived in this project.

2.1 Wind Loads on Buildings

Wind Engineering can be described as the "rational treatment of interaction between wind in the atmospheric boundary layer and man and his works on the surface of earth" (Cermak, 1975) [1]. Wind gets its energy directly from the sun. Solar radiation is nonuniform, being stronger around the equator. The appearance of differences in temperature produce, in turn, also different zone pressures, that finally creates atmospheric wind flow circulations. There are also additional alterations to the atmospheric circulations due to geographical, seasonal and rotation effects of earth.

Winds acting near surface has additional effects that play an important role. Obstructions that occur close to the ground retard the movement of air reducing wind speed. Nevertheless, at some height above ground air flow is no longer affected by this kind of obstructions; the so-called gradient height, z_G, which depends on ground roughness. The not obstructed wind speed is called gradient wind speed, V_{z_G} and its constant above this z_G. The power law that follows the mean wind speed at a z height related to gradient wind speeds is given by the following expression. [2]

$$\frac{V_z}{V_{z_G}} = (\frac{z}{z_G})^{\alpha} \tag{2.1}$$

where α is 60% of the gust speed coefficient β.

In Figure 2.1 it can be seen the characteristic variation of wind speed dependent on time for a 100 s study. The wind speed consists of two principal components: mean wind velocity and a fluctuating/turbulent component. Convective movement or either ground roughness causes the latter. The mechanic turbulence predominating is assumed in the case of high wind speed with boundary layer flow. Turbulence is greater in rougher terrain compared to the smoother one. [2].

When considering wind flow upon buildings, the distribution of pressure appear on the envelope of the building. The challenge is to know these distributions. The air-flow speed V produces a pressure that follows Bernoulli's expression as follows.

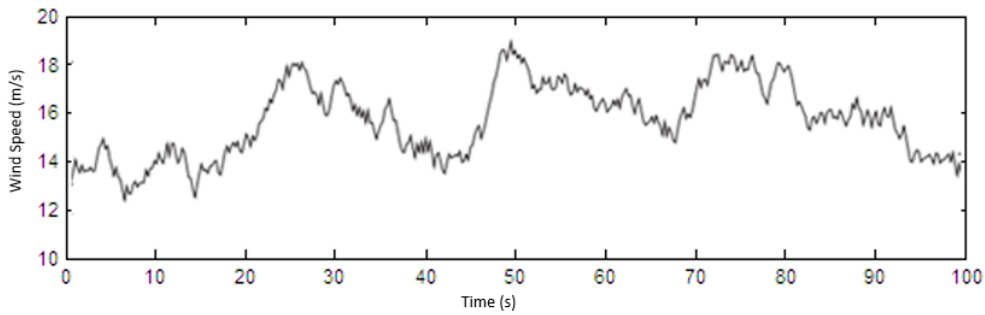


Figure 2.1: Time dependant wind velocity variation for a 100 s period study. [2]

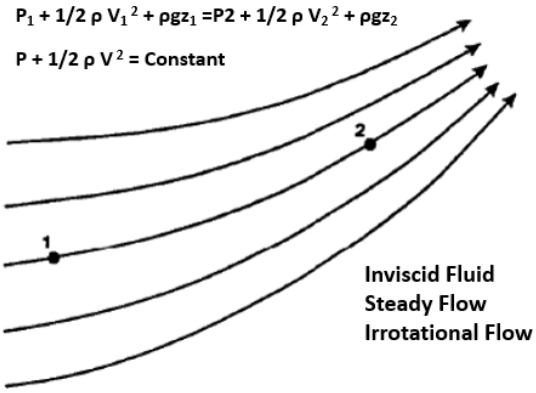


Figure 2.2: Stream lines of Bernoulli´s expression. [2]

$$P + \frac{1}{2} \cdot \rho \cdot V^2 = constant \tag{2.2}$$

where $\frac{1}{2}\ \rho\ V^2$ is the so-called dynamic pressure, P is pressure while ρ is the air density. This expression is only valid for ideal, non viscous, conditions.

Nevertheless, most of the buildings and structures are not streamlined but instead they are so-called "bluff bodies". In this case, wind flows generates "wakes" affecting this buildings. In Figure 2.4 it can be seen the typical wind flow incidence in a building. If air was ideal or non-viscous, Bernoulli´s equation could be applied anywhere. Nonetheless, the real viscous case shown in the wind flow incidence on the body (Fig. 2.4) , reflect that there are two upstream edges flows: an outer flow, with no viscosity effect, and an inner flow where there is. Both, outer and inner, wake regions are separated by an area of a higher number of vortexes. Hence, Bernoulli´s expression can only be applied in the outer movement region. [1]. In Fig. 2.3 it can be seen wind movement around the structure and the formation of vortexes due to wind pressures.

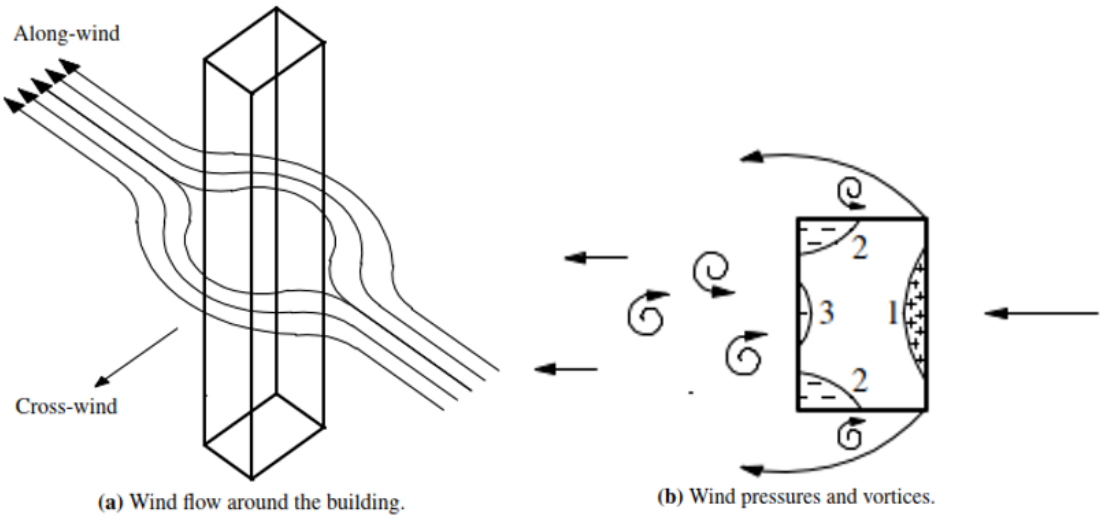


Figure 2.3: Bluff-body surrounded wind flow.

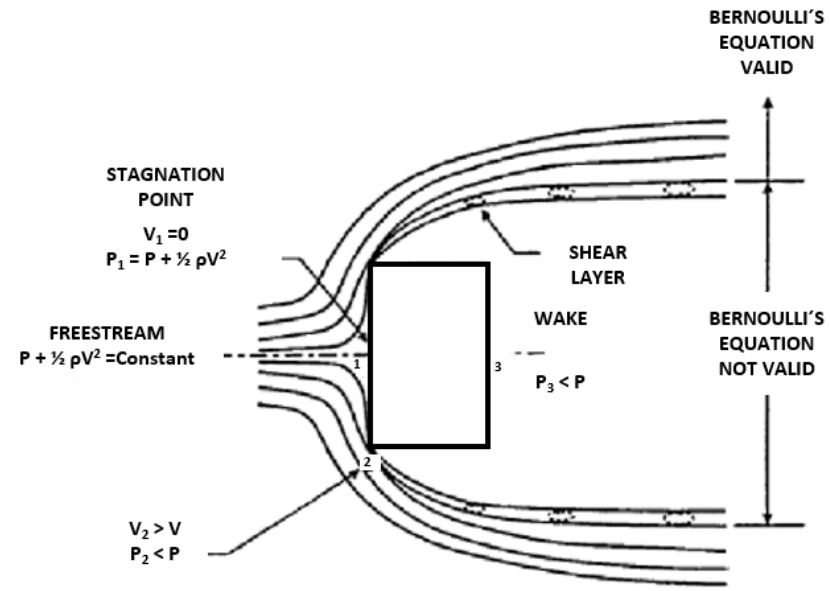


Figure 2.4: Bernoulli´s equation around a building. [2]

Pressures are normally expressed without dimension in order to be independent of velocities. This is done by introducing the so-called pressure coefficient C_P that is given by Expression 2.3.

$$C_P = \frac{\Delta P}{\frac{1}{2}\rho V_0^2} \tag{2.3}$$

where ΔP is the pressure difference induced by wind. In the case of a study of a slender building it is possible to make a 2D study with forces and pressure flows being uniform.

Uniform pressure or force incidence on a flat plate is shown in Figure 2.5. In this simplified situation, pressure wake is practically constant along the flat plate.

Nonetheless, the real situation of the boundary layer flow can be seen in Figure 2.6. In this other situation, results are drastically different with the formation of one, or even several, vortexes in the structure front, the creation of a separated down flow from the sides of the plate and also the lower suctions of the wake apparition. [3, 2]

Attending to Figure 2.7 it can be seen the practically uniform condition by observing C_P along the cube. On the other hand, in Figure 2.8 it is shown the second case where the mean pressure coefficient distribution obtained for real wind boundary layer flow has non-uniform distribution.

It is also necessary to define a drag coefficient. Wind stream net forces, are composed of along-wind and across-wind components: q_D and q_L, respectively.

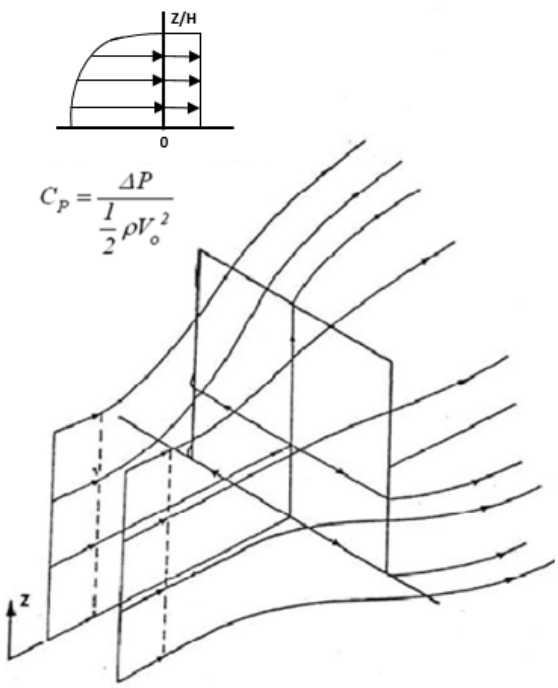


Figure 2.5: Uniform flow pattern of wind over a flat plate and center line pressure distribution. [2]

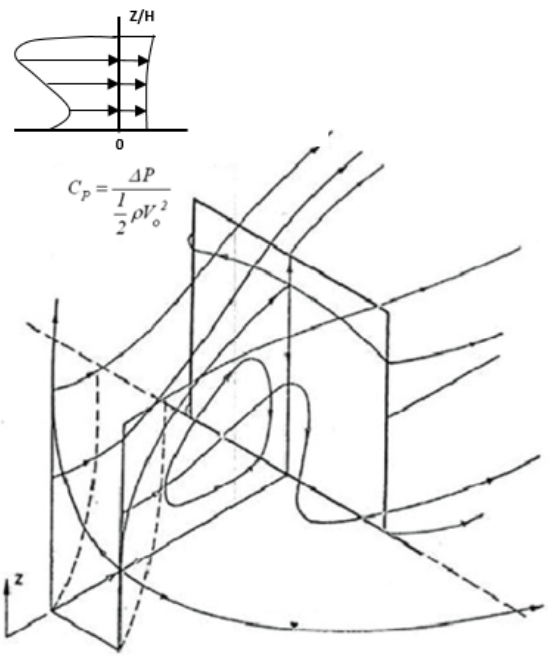


Figure 2.6: Nonuniform flow pattern of wind over a flat plate and center line pressure distribution. [2]

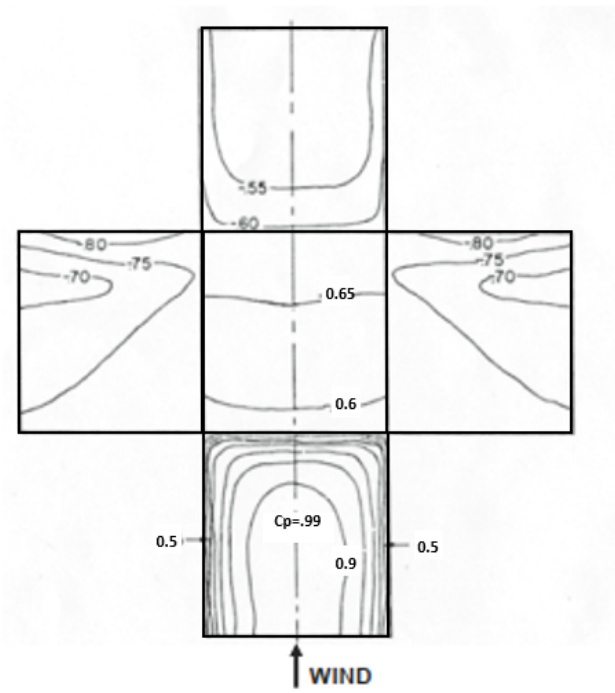


Figure 2.7: Uniform mean pressure coefficient distribution obtained for wind boundary layer flow on a cube. [2]

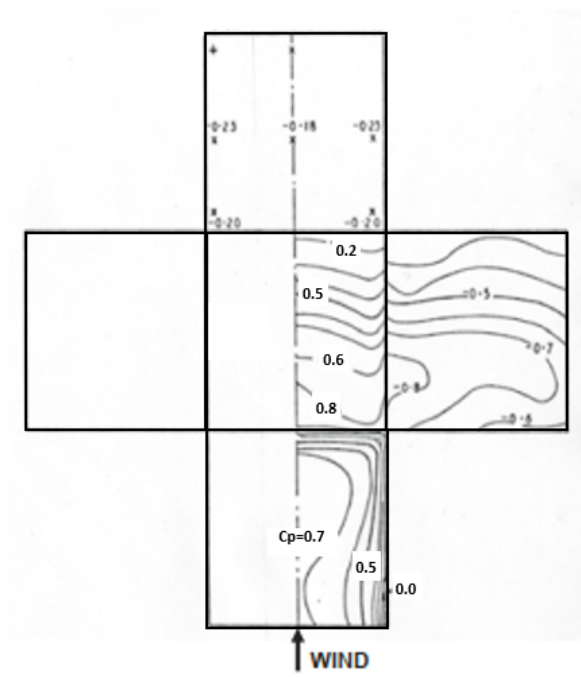


Figure 2.8: Mean pressure coefficient distribution obtained for real wind boundary layer flow on a cube. [2]

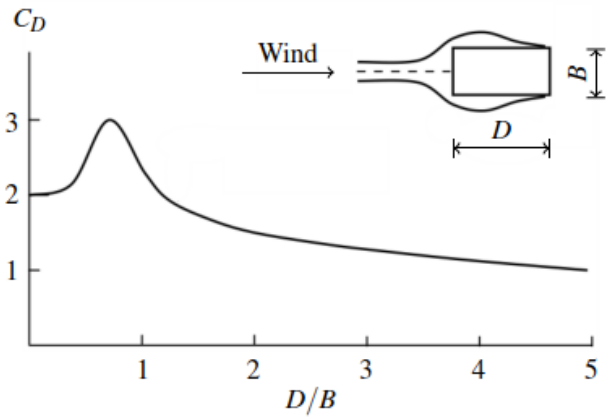


Figure 2.9: Along-wind force coefficient with respect to cross-section ratio in a smooth wind flow with $10^5 < \text{Re} < 10^6$. [5]

Drag along-wind coefficient is described by the following expression. [4].

$$C_D = \frac{q_D}{\frac{1}{2}\rho v_0^2 b} \tag{2.4}$$

Hence, C_D depends on the building width and length (as shown in Fig. 2.9) [5]. Drag across-wind coefficient is described by the following expression. [4].

$$C_L = \frac{q_L}{\frac{1}{2}\rho v_0^2 b} \tag{2.5}$$

Both drag coefficients, C_D and C_L, depend on the buildings geometry along with the wind attack angle.

2.2 Dynamics of Buildings with Wind Incidence

Wind is a great complexity phenomenon with many flow alternatives that occur due to the wind interference with buildings, as it has already been considered. Wind has a turbulent and gusty nature composed of numerous varied sized eddies and also has rotational properties that appear in a general air stream that moves over the earth's surface.

The main reason of the gusty nature of the air flows in the lower atmosphere levels comes from the interaction with surface topography. This turbulence gust effect decreases with height in contrast with average wind speed situation, that is directly proportional to height increase.

The appearance and strength of the turbulence dynamic loads component on a structure lies on the size of the eddy. Thus, bigger eddies make greater correlated

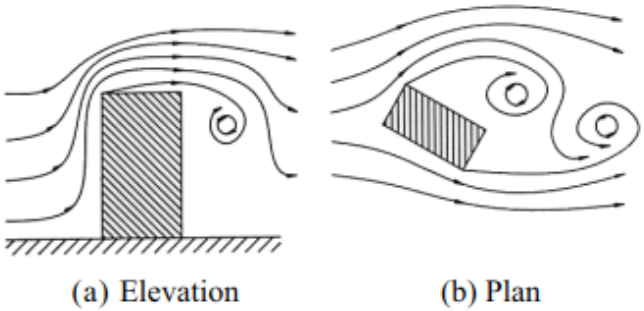


Figure 2.10: Two different case of eddies formation. [6]

pressures as they wrap the building while small eddies generates an uncorrelated situation of pressures on various parts of a structure. Two different eddy situation over a typical building can be seen in Figure 2.10. [6]

The dynamical respond to wind effects comes clear in some structures when the are tall or slender. It is well known the Tacoma Narrows Bridge collapse in 1940 due to the incidence of wind with just 19 m/s speed which failed after a flexure and torsional coupled mode oscillation. Along with this phenomenon there are several of others just like vortex shedding, buffeting, flutter or galloping that could occur due to wind loading. There are numerous diverse phenomenons arising buildings wind dynamic response with slender and tall structures being more sensitive to experience this phenomenons [4].

Wind load on a building varies depending on the wind mode but also the geometry of the wind-incidence building together with the geometries and nearness of the surrounding buildings. Due to the vortex shedding and gust nature of the wind, pressure are in a constant fluctuation that can result for sensitive structures in fatigue damage and additional structure dynamic. As it has been considered in the previous section, pressure is not uniformly distributed causing great complexity that could make peak wind incidence during a building lifetime vary from the conceive during the design stage [7].

It is also well considered and need to take special careful on sway accelerations due to vibrations in buildings that depends on the building natural frequencies. Structure dynamic depends on both mass and stiffness, and the acceleration obtained in response can be reduced by changing this structure properties by changing the damping. When the frequency of the more energetic gust matches the first structural modes, or lower frequency, a dynamic response tends to occur. This has a greater relevance in slender structures due to its low relative stiffness and damping. In this situation, even low gust can cause big pressures over the building. [8]

As it was claimed by Young Kim and Yu (2009), the prediction of acceleration induced by wind, that result to be of the principal ideas of designing in large structures study, depending on natural frequency estimation. Thus, excessive

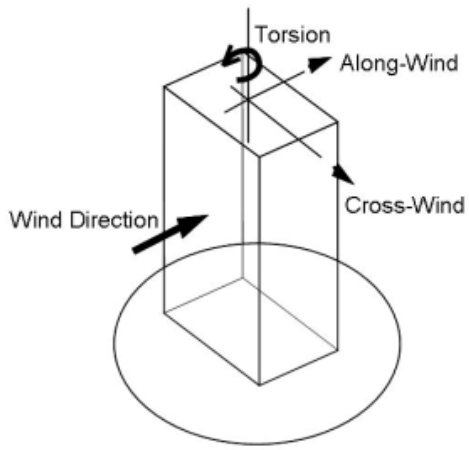


Figure 2.11: Wind Response Directions. [6]

structural cost may come from a excess of conservativeness in the prediction of the service performance. [9]

Wind codes procedure divides analysis in Static and Dynamic methodologies. The outer is quasi steady assumption based of the building being a fixed rigid body. It is not appropriate for tall buildings due to susceptibility of vibration in the wind due to slenderness and height (it is used up to 50 m). The latter or dynamic response is stated to be undertaken to define total loads on any building with a breadth ratio larger than five and with a first natural frequency inferior to 1 Hz (like the CAARC building going to be under study). The fluctuations frequency and magnitude depends on turbulence factors. This factors need to be simplified by the codes with several approximations, e.g., quasi-steady assumption with a single value equivalent static wind load in order to achieve the maximum peak force the building could face. [6]

The wind movement pattern generated around a structure is also a high complexity phenomenon. This is due to mean flow, the flow separation, the vortexes formation and wake develop due to wind approaching a building as it was already claimed to be. This could lead to constant pressure fluctuation on the building surface with large aerodynamic forces on the structure and intense loads acting on the building facades. All this phenomenons together lead to rectilinear and torsional modes as can be seen in Fig. 2.11 [6]. Depending on the aerodynamic load oscillations nature and the building properties, the oscillations amplitude will be defined. [7]

2.2.1 Along-Wind Dynamics

Along-wind response is composed of a mean part due to mean wind velocity and a fluctuating part due to wind velocity variations that is the responsible of building accelerations. the fluctuating component is composed of different and random sized eddies or gusts. The natural frequency of vibration of most structures is

greater than the fluctuating load caused by larger eddies. Because of this fact, there is no dynamic response. Hence, the so-called ”background” turbulence, or larger eddies, loading can be treated similarly as the mean wind. Nevertheless, the smaller gusts are more likely to appear, making more possible to induce the structure to vibrate near their first mode or natural frequency of vibration [9]. The dynamic effect of this phenomenon could be really concerning.

The gust-factor approach uses this separation in mean and fluctuating component as is treated in many Standards. As it was claimed by Davenport (1967), it is method used for the determination of fluctuating loads that allows to define turbulence at a specific site, makes a dynamic amplification but also consider the effects of size reduction. [10]

There is relative good accuracy in along-wind wind dynamic response on building prediction using the gust factor approach, due to the non-significant affectation of the wind by the surrounding and neighbour buildings.

2.2.2 Cross-Wind Dynamics

On the other hand, it is also necessary to pay attention to cross-wind dynamics. Tall chimneys, street lighting standards, cables and towers are frequently affected by the cross, or perpendicular, wind dynamics form of oscillation. This phenomenon raises importance the lower the damping is. There are three main mechanisms of cross-wind excitation in modern buildings [11, 6]: vortex shedding, the incidence turbulence mechanism and galloping, flutter and lock-in.

Vortex Shedding is the most common source of excitation that occur when flow separates from the surface of the building instead of following the body contour due to buildings being bluff-bodies. Approaching flow turbulence tends to minor shedding regularity, maintaining or even enhancing the strengths of the vortexes. Body vibration may also cause the vortex strength. As each of the vortexes are shed from a bluff-body, towards the shed vortex side a strong across-wind force is created. Hence, the vortexes alternate shedding induces a sinusoidal, or harmonic, across-wind load fluctuations on the building. [7] Depending on a Strouhal number, that will be developed in section 3.1, this vortex shedding can bring large oscillations and lead to building failure.

The incidence turbulence mechanism, or turbulence properties of wind, lead to changes in wind speed and direction that makes a variation in lift and drag forces. Due to this, it is also needed to be taken into account.

Advanced derivatives of across-wind movement such as galloping, flutter and lock-in are dependent on turbulence properties of the wake and aerodynamics, as it was studied by Holmes (2001) [7]. Galloping is a single degree of freedom way of aerodynamic uncertainty, which occurs for large buildings with certain across-sections. It is a across-wind shaking purely translational. Galloping variability will be started when the negative aerodynamic load is greater than the positive

damping force depending on the structural damping depending on a critical wind speed for galloping. For the case of flutter phenomenon, it is necessary to consider a two dimension bluff-body with the ability to move to move and with elastic restraint, just like bending and torsion deflections. A twisting can be made with the correct effective angle of attack with aero-elastic forces causing instabilities if they are not opposite to the structural damping. The aerodynamic instabilities in tall buildings are the so-called ´flutter´. [7]

For all these phenomenons, computational fluid dynamics had been used to study this phenomenons, e.g., Tamura (1999). [12]

2.2.3 Damping Effects

Damping act in a structure as an energy dissipating feature. Every system has a natural damping, from the simple bearing friction to a viscous damping. For the case of a tall building, damping has beneficial consequences in terms of absorbing energy, reducing motion and, thus, minoring structure dynamics. [8].

This beneficial effects reinforce structures stability in a cost effective way that control vibrations. However, it is sometimes a common use to reduce resonant vibrations as the cheapest option.

There are passive dampers such as: Tuned Sloshing Water Dampers (TSWD), Visco-Elastic Dampers, Tuned Mass Damper (TMD), Impact Type Dampers, Friction Dampers or Distributed Viscous Dampers x Tuned Liquid Column Dampers (TLCD), but also active/hybrid dampers like: Active Mass Driver (AMD) or Active Tuned Mass Damper (ATMD) and semi active dampers like: Hydraulic dampers, Variable Stiffness Dampers, Magneto-Rheological (MR) Dampers, Controllable Fluid Dampers, Electro-Rheological (ER) Dampers or Variable Friction Dampers. [13, 14]. Passive dampers tend to be the most used due the lower maintenance and capital cost. [15]

It is essential to make a precise estimation of natural frequencies in order to predict acceleration induced by wind and, thus, satisfy the serviceability requirements due the acceleration induced by wind loads not exceeding the established limits. Only scarce structure standards consider expressions for fundamental natural frequency determination in service vibration amplitudes, e.g., Eurocode [16] exposes a very simple empirical expression for the fundamental natural frequency of structures under wind load conditions. On the other hand, Architectural Institute of Japan [17] does consider "a regression curve of fundamental natural frequencies in service vibrations". It is given based on data measured in vibration field of more than 100 buildings, not being that much accurate for tall buildings. [9]

Kim Young and Yu (2009) did a study of finite element (FE) models for three different large structures in order to compare their fundamental natural frequencies for system identification technique values taken from the measures of acceleration on tunnel tests with remarkable agreement for all three buildings between the

measurements and natural frequency estimations from the calibrated FE-models. Also using this previous good results, acceleration prediction values also matched, with high accuracy, between FE model and Tunnel tests for the same typhoon. [9]

2.2.4 Dynamics Simulation of Wind Induced Buildings

When applying a design document it is highly necessary to consider the complexities of wind loading. There are several uncertainties and variations of the maximum wind loads a building could experience during its lifetime and it need to be ensured a correct building development, as it has already been said. Building failure due to wind incidence does not necessary have to do with the conservative nature of the selected Wind Loading on Structures Code due the fact that this codes does not consider building shapes or unusual building location in terms of the surrounding buildings characteristics.

Thus, it is a common procedure to apply simulations just like wind tunnels or computational/numerical fluid dynamics techniques providing data for making an accurate comparison with the Standards being used. [2]

2.2.4.1 Wind Tunnel Tests

Certain types of wind-induced structural responses cannot be estimated by analytically methods in several situations such as the case of high flexibility buildings that affects aerodynamic forces that act on it or uncommon aerodynamic building shapes. In order to ensure accuracy in the wind effects on buildings estimation, it is highly recommendable to use boundary-layer wind tunnel for aero-elastic model testing. [6]

It is generally utilized in the design stage of tall structures due the economical advantages in relation to the savings in the building costs that prevail over the tunnel testing simulation expenses. Australian Wind Code [11],consider wind tunnel to be a reliable alternative and it is permitted to use them in the design wind loads for any structure with a national committee for establishing a practise code for wind tunnels testings. Due to this code considerations, Australia has some of the leading advances in facilities for wind tunnel testing like Monash University wind tunnel. [11, 18]

Wind tunnel tests are used by engineers as a well known and powerful tool in order to determine the intensity and nature of wind forces incidence over structures and its surroundings. Is in the case of a high complexity situation of several structures when it becomes particularly useful to make this tunnel test data collecting. In this situations, it is not recommendable to just lie on codes for determining wind forces dynamics on the structure in order to achieve elevated accuracy.

Wind tunnel tests consists of an scale simulation (could be between 1:200 or even

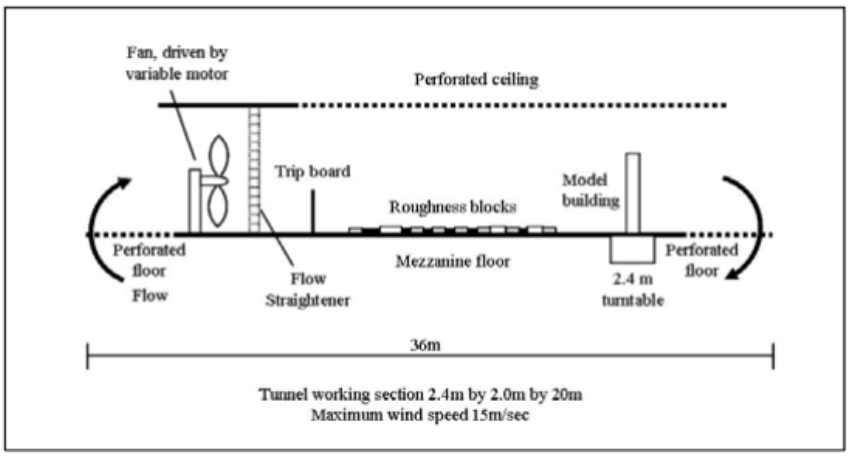


Figure 2.12: Open-circuit wind tunnel scheme. [6]

1:100) of the building and its surrounding structures and the introduction of a changeable wind blowing flow, varying angle, amplitude and frequency in order to represent and study all the possible situations that could occur. The way of changing the angle in generally accomplished by situating the whole simulated structures on a rotational structure so that just with a rotation of the platform, the new wind incidence could be studied.

A typical wind tunnel scheme can be seen in Figure 2.12 [6], concretely a open-circuit wind tunnel (there are also close-circuit wind tunnels that are used).

2.2.4.2 Computational Wind Engineering (CWE)

The fields where numerical simulation using Computational Fluid Dynamics (CFD) is becoming not only the most accurate in prediction but also the most powerful technique are in upsurge. They are becoming the first engineering choice for many cases even for applications where the is a fluid and the structure interaction. As happened with wind tunnels, CFD techniques are applicable in situations where the used wind loading codes are not directly or easily used. This usually happens in situations where tall buildings are being used with non-common structures.

The Architecture Institute of Japan (2008) set a number of guidelines in order to consider the relation of the computational aspects, i.e., boundary conditions, discretization of the grid, size of domains, etc., on the prediction precision as it was not methodically studied. [19]. These procedures deliver valued information on the applications for buildings surrounding flow. This operational group made several wind tunnel tests, field measure operations and computations influence utilizing different codes of Computational Fluid Dynamics CFD to seek the several types of computational variables for different flow fields.

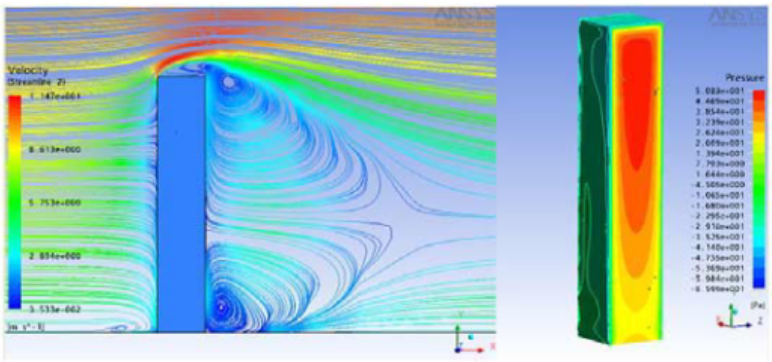


Figure 2.13: CWE building model in program CFX10. [6]

FE model analysis is normally also applied in several possible simulations such as the one employed to predict the natural frequency by Young Kim and Yu (2009) [9]. In the same way, Castro and Bertolli (2015) work used a numerical approach for the obtainment of the structural response by the application of the finite element method (FEM). [20]

Hence, as it was exhibit by Eindhoven University of Technology (2014), ”Computational Wind Engineering (CWE) has undergone a successful transition from just an emerging field into an increasingly established field in wind engineering research, practice and education”. [21]

They establish 3 key illustrations of the success of Computational Wind Engineering (CWE): ”the establishment of CWE as an individual research and application area in wind engineering with its own successful conference series under the umbrella of the International Association of Wind Engineering (IAWE); the increasing range of topics covered in CWE; and the history of overview and review papers in CWE”. With all of it, Eindhoven University of Technology (2014) exposes the promising future of the CWE due to being able to overcome many the problems it previously presented in comparison with alternative techniques. Nevertheless, it claims that: ”CWE has come a long way but there is still a long way to go, many problems to be tackled, many research questions to be addressed and many challenges to overcome, the strong progress established in the past 50 years provides a promising outlook for its future.” [21]

A 1:400 scale model of a 40x40x300 m^3 building is shown in Figure 2.13 with the shear stress transport (SST) as the turbulent model taken from program CFX10 by Mendis and Ngo (2007). [6]

2.2.5 Human Response to Building Dynamics - Comfort Study

When designing a structure for lateral wind there are several aspects that must be taken into consideration in order to accommodate structural dynamics to human.

Table 2.1: Human perception guideline over wind loading on building. [6]

LEVEL	ACCELERATION (m/s^2)	EFFECT
1	<0.05	Humans cannot perceive motion
2	0.05 - 0.1	a) Sensitive people can perceive slightly b) hanging objects may move
3	0.1 - 0.25	a) Majority of people will perceive motion b) level of motion may affect desk work c) long - term exposure may produce motion sickness
4	0.25 - 0.4	a) Desk work becomes difficult or almost impossible b) ambulation still possible
5	0.4 - 0.5	a) People strongly perceive motion b) difficult to walk naturally c) standing people may lose balance
6	0.5 - 0.6	Most people cannot tolerate motion and are unable to walk naturally
7	0.6 - 0.7	People cannot walk or tolerate motion
8	> 0.85	Objects begin to fall and people may be injured

Stability needs to be studied against overturning and uplift. Strength of the structure components should also be considered to avoid lifetime structural failure. Finally, serviceability needs to have special considerations with deflections needed to remain between proper limits by controlling drift that assure there are not cracking and damage of structural members. [6] Standards establish an ultimate limit state wind speed for ensuring this stability and strength limits so that wind storm peaks have a 5% probability of exceed in a 50 year period.

For the dynamics study, it is really important to pay special attention to human perception of motion induced by wind that is particularly sensitive to vibration even at a really low level. The human comfort is not generalized by any of the standards in the structure design. Human perception is claimed to be influenced by some psychological and physiological parameters that could intercept in the real vibration perception sensitive. This appear to affect in a low frequency range of up to 1 Hz for the tall buildings.

Studies reflect that aspects such as activity, orientation, body posture or even expectancy of the occupant could affect amplitude, accelerations or frequency perception subjected by the studied person. [22, 23]. Perception level can be seen in Table 2.1 that could act as a general guide of human criteria over building wind loading effects.

Establishing limits is needed for making a correct assessing of building characteristics. Wind forces are, therefore, tabulated using as a basis the Beaufort scale which is a function of frequency and vibration felt [22]. Upper limits where suggested by Irwin (1978) for specific frequencies and vibrations while peak accelerations where established by Melbourne and Cheung (1988). The study is, hence, carried out by the combinations of both studies. [24, 25]

In order to achieve the considered peak acceleration value, the r.m.s. value is assumed to be multiplied by a peak factor. This peak factor is commonly considered to be within 3 and 4 in the different conservative used approaches.

2.3 CAARC Standard Tall Building Model Specifications

Commonwealth Advisory Aeronautical Research Council Coordinators in the Field of Aerodynamics in 1969, made a definition for "standard tall building model for the comparison of simulated natural winds in wind tunnels" published by Wardlaw and Moss [26]. In the aim of making a different techniques contrast done in wind tunnels tests they established a simple model. The models dynamic response and measurements of pressure standardization lead to a much more reliable and comparable data for a much greater volume of tests [27].

In 1975, five centres made measures using the CAARC Standard Tall Building standard with the work being available for making a comparison by W.H. Melbourne and a followed results dialogue at a conference meeting in "the 5th International Conference on Wind Effects on Buildings and Structures done in London, on September 1975". [27, 28].

At that meeting it was agreed, after discussions, to achieve uniform presentation. The goal was to increase the comparison accuracy related to a tall isolated structure along with an individual techniques calibration of the results. [29, 30]

2.3.1 Basic Model Specifications

The building geometry is the one that follows and can be seen within the incidence angle in Fig. 2.14 with a numerical identification. It was specified a rectangular prismatic form. The CAARC aerodynamic coordinators determine it to be, with full scale dimensions, 30.48x45.72x183.88 m. Also, the structure was flat topped, not using parapets, and the exterior walls were also plane, not introducing any dimension disturbances.

With regard to the CAARC building dynamic characteristics establishment, the fundamental vibration mode is the only to be considered as it is assumed to be linear and in rotation about a ground level point.

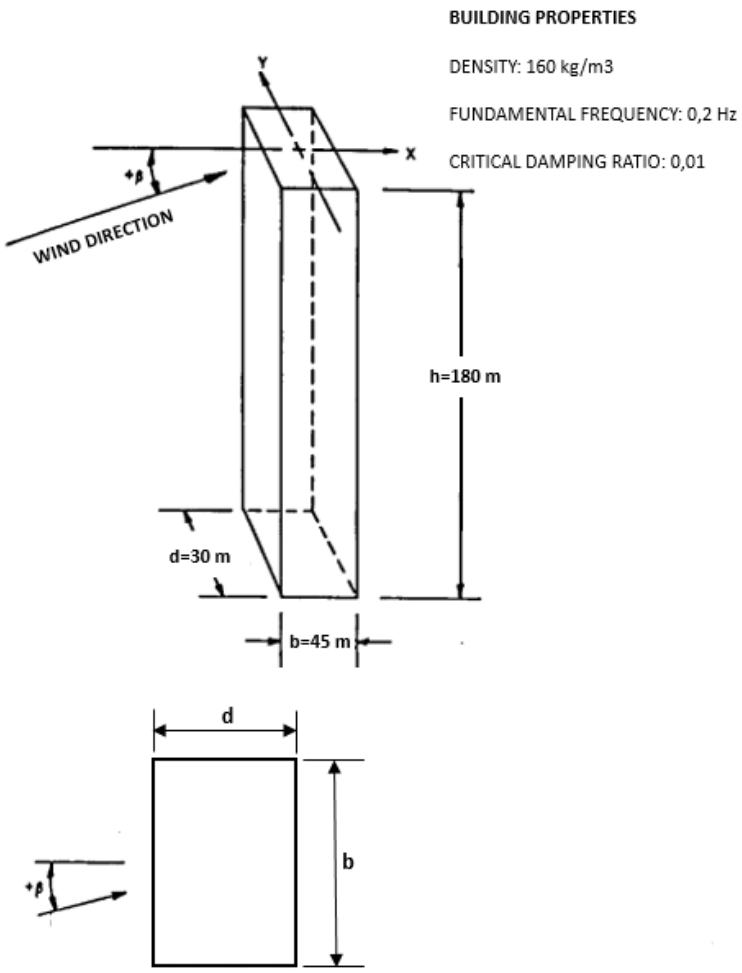


Figure 2.14: CAARC Building dimensions and properties. [27]

This natural frequency is considered to be 0,2 Hz for both principal axes that affect at the considered ground level. The weight distribution is taken as 160 kg/m^3. The structural logarithmic decrement is considered as 0.063 due a structural damping ratio of 1% of the critical, and thus facilitate the result comparison between simulations.[26, 27]

For the natural wind boundary layer it was recommended that the power law exponent for the boundary layer should be 0,28, and thus, being representative of wind incising over a urban development with an average height of the surrounding buildings of around 6 to 15 m. [26, 27]

2.3.2 Comparisons Study of a CAARC building in Various Simulated Wind Flows

Comparison between measurements on the CAARC Standard Tall Building was made by Monash University (1980). They made pressure measurements of the surface and also computed the response of six different wind flow establishments over a CAARC building.

They came to the conclusion that the accuracy was good enough with an elevated degree of agreement. They also saw small observable trends in relation to force measurement which could be assigned to divergences in the approaching longitudinal speed spectrum and to the blockage corrections requirement. Attending to the dynamic response measurements, there were no obvious trends with the majority of the compared data within a 15% difference. [27]

2.4 Compilation of Wind Loading Simulation Studies - State of Art

Following the procedures stated in section 2.2.4, there are several studies that have been done in the wind loading field, that have helped to continue advancing in the wind-incidence on buildings and structures research. Along the previous sections, some of them have been already mentioned.

It is a recurrent procedure, to apply both kind of studies previously commented, wind tunnel together with numerical or computational simulations, in order to look for specific results.

Lin, Letchford and Tamura (2005) studied the features of wind loads acting on large structures. They tested 9 cases, with diverse rectangular cross sections, studying them in a wind tunnel in order to achieve the features of wind loads on tall structures. They made the investigation in relation of ”mean and root mean square force coefficients, power spectral density, spanwise correlation and coherence along with the effect of three parameters: side ratio, elevation and aspect ratio on bluff-body movement”. They made their result comparison with results obtained from high frequency load balancing in two wind tunnels. Data extracted from the study was concisely stated in the ”Local wind forces acting on rectangular prisms. Proceedings of 14th National Symposium on Wind Engineering, 4–6 December 1996, Japan Association for Wind Engineering, Tokyo, pp. 263–268.” [31]

Tominagaa, Mochidab and Yoshie (2008) made an important research on practical applications of computational fluid dynamic (CFD) to pedestrian wind nature around structures. Assessment and prediction of pedestrian wind environment around buildings in the design stage have reached remarkable improvements due to computational fluid dynamic (CFD) softwares facilities and computation advances. Hence, the use of CFD techniques for this purpose requires guidelines in order to summarize important points such as the proposed by Tominagaa, Mochidab and Yoshie for the Architectural Institute of Japan (AIJ) in this paper. They evaluated seven study tests used to analyze the impact of diverse computational conditions for different flows making a comparison between computational fluid dynamic (CFD), wind tunnel tests, predictions and field measurements. The obtained results conducted together with the project utilized in the accuracy validation of computational fluid dynamic standards utilized in the practical uses of wind flow predictions. [19]

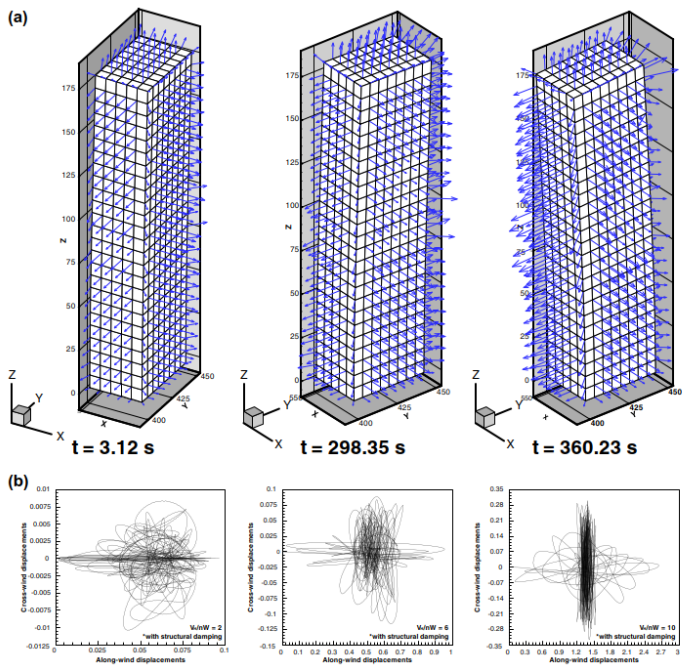


Figure 2.15: CAARC building aero-elastic analysis: (a) buildings deformed configurations; (b) Roof center point trajectories. [32]

Braun and Awruch (2009) studied a reproduction of the wind loads over a Commonwealth Advisory Aeronautical Council (CAARC) Building prototype. They demonstrated the applicability of computational fluid dynamics, CFD, technologies in wind study by the use of aerodynamic and aeroelastic analysis reproduced numerically. They made one of the prior's aeroelastic behaviour of a tall building simulation attempts by using computational fluid dynamics (CFD) engineering. The results were finally compared with wind tunnel measurements data where they took their conclusions. They came to a satisfactory arrangement with other predictions made experimentally and numerically, when there was smooth or low turbulence flow conditions for the measures of forces and aerodynamic coefficients over the structure envelope. Furthermore, they achieved a better agreement when the comparison to wind tunnel predictions on across-wind over the along-wind structural responses achieved was made with an aeroelastic analysis. In Fig. 2.15 can be seen results obtained in the CAARC building model with the aeroelastic analysis made by Braun and Awruch work. [32]

Also in 2009, Young Kim and Yu employed the FE study to make the standardization of analytic models to measure induced by wind acceleration responses of large structures under service level. As it was already mentioned, they made a precise estimation of natural frequencies. It is crucial in order to appropriately estimate acceleration due to wind-incidence to ensure enough service level conditions in the design stage of large structures. They studied three tall reinforced tall buildings using PC based FEM program in order to coincide with their fundamental natural frequencies to actual values that where extracted from the acceleration measures from the technique based on system identification. They

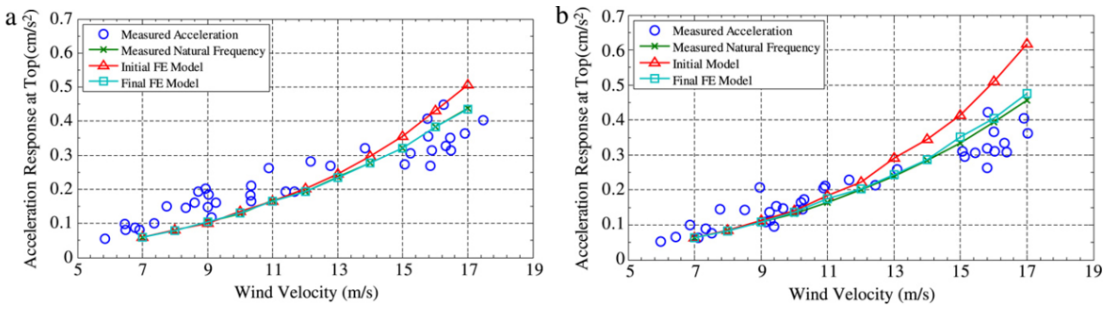


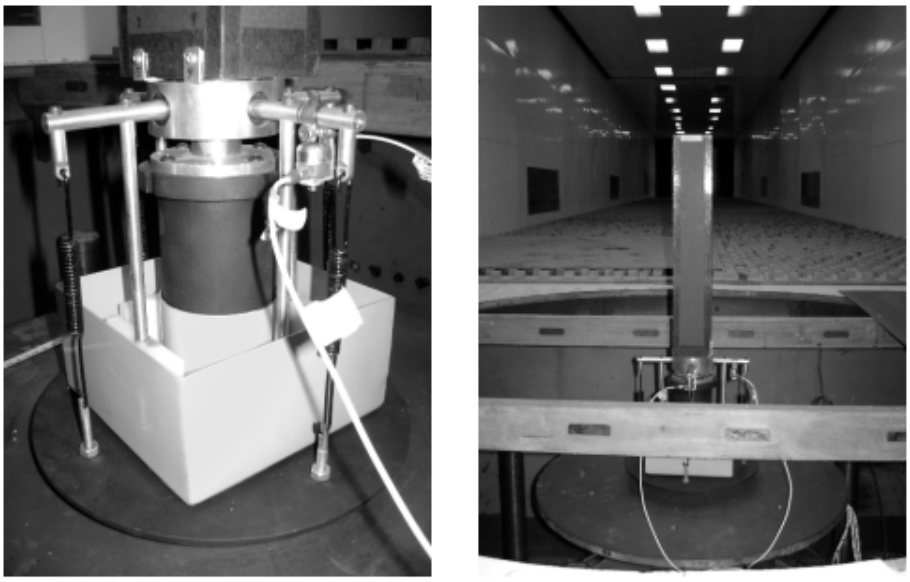
Figure 2.16: Prediction of RMS acceleration at the buildings top from Young Kim and Yu work (2009): (a) x-direction; (b) for y-direction. [9]

included a contemplation of the effect of end offsets in beams, modelled floor blocks instead of making the use of rigid diaphragm assumptions. They also included components that were not structural such as concrete envelope and walls bricks made of cement but also used higher elastic modulus of actual concrete specified value in order to make the correct modifications of FE models.

This calibrated finite element models and the measures presented extraordinary coincidence for all the studied structures in terms of natural frequency estimation. Using wind tunnel studies results together with the dynamic properties gotten from these calibrated finite element simulations, the acceleration response of a structure under a typhoon was anticipated with high accuracy and also in relation to previously measured accelerations with really high matching between the results showing the precision of the predictions. In Fig. 2.16 a graph is shown of the predicted RMS acceleration in x- and y- directions in comparison with the achieved values of Kim and Yu´s work. [9]

Yan and Li (2015) also applied a comparison between computational wind engineering (CWE) and wind tunnel tests in their studies. They realized and studied the generation of turbulent inflow conditions in order to match real wind flow characteristics over atmospheric boundary layer (ABS) due to its relevance in providing reliable predictions on building dynamics due to wind-incidence by using large eddy simulations (LES) that has turn into a major importance tool in CWE (computational wind engineering) [33]. In this sense, their main goal was to evaluate performances of different techniques that study turbulence flow appearance.

Concretely, they studied four methods for inflow turbulence generation, of which three where synthetic turbulence generation techniques, that where applied to simulate atmospheric boundary layer for large eddy simulation on tall building under wind loading. They made a comparison of these four cases with wind tunnel results and numerical simulations previously compiled in order to define advantages and disadvantages or limitations. In this sense, they achieve some remedial measures in order to improve the techniques studied performances. [34]



(a) Base-pivoted rigid model. (b) Dynamic balance in the wind tunnel.

Figure 2.17: Wind tunnel test in Castro and Bertolli (2015) work. [20]

Castro and Bertolli (2015) studied the determination of the along-wind dynamic response of the CAARC standard tall building. In their study, they claimed that even thought the usual procedure of evaluating structural behaviour under wind action is made by the Gust Factor Technique (GFT)[1], which provides equivalent static forces acting on the structure , it is not possible to develop a more detailed dynamic analysis. While the widely used frequency domain analysis can give a clear description of dynamic aspects involved, time domain analysis can give, under a non-linear structural behaviour, an accurate study of the phenomenon. They consider the fact that analytical methods that are used to determine structure dynamics under wind-incidence generally lead to unreliable results when applied to tall structures with uncommon geometries and surrounding buildings and, as it was already considered, employed in these situations wind tunnels tests. This way they implemented and discussed by a numerical and experimental procedure the determination of the dynamic longitudinal response of large buildings under the atmospheric boundary layer (ABS) wind action.

They simulated time series which defines the wind velocity field and are subsequently transformed into nodal forces by a quasi-static approach, with a numerical simplified procedure. Using a Finite Element Method (FEM), in time domain, the Longitudinal dynamic response of the structure was achieved. Also, the longitudinal fluctuating response was measured with wind tunnel tests performed with the aid of a base pivoted rigid model balance, Fig. 2.17, in order to make an accurate comparison of the two employed models in the well known CAARC Standard Tall Building. [20]

[1] "The gust-excited response is classically evaluated by applying an equivalent static load that is obtained as the product between the wind-induced mean static force and the gust loading factor" Pagnini and Piccardo (2017) [35]

Chapter 3

European Wind Code EN 1991-1-4 and Wind Standards Study

Along this chapter the application of the European Wind Code EN1991-1-4 will be discussed. General outlines in terms of wind load actions, dynamic response consideration and the introduction of the National Annex code will be presented, together with the established procedure for the study and determination of wind loads on buildings.

A comparative study of the main standards used worldwide apart from the one used in the European Union in relation to averaging time, wind velocity profile, along-wind, cross-wind, torsional loads and acceleration responses different considerations will take place. As it's the main interest comparison aspect. Wind tunnel test Standards considerations will also be treated to conclude the chapter.

3.1 EN1991-1-4. European Code for General Wind Load Actions

In this section, it will be discussed the way wind loads are considered in EN1991, Actions on Structures, and part (1-4), specific for Wind Loads. Thus, in the Euro-code series of European standards that are focused in construction, Euro-code 1 (EN1991) is for Actions on structures and describes the procedure to make the design of structures under loads. EN1991 has some reference values for different kind of loads for a large variety of materials meant to be utilized in construction.

EN1991 is divided in two parts (with first part also subdivided in seven divisions):

Part 1.1, Densities, self-weight and imposed forces in structures
Part 1.2, Actions on buildings with fire expose
Part 1.3, Snow loads
Part 1.4, Wind action

Part 1.5, Thermal action
Part 1.6, Execution actions
Part 1.7, Accidental Action
Part 2, Bridge and traffic loads

In relation to the project development, the EN 1991 part of the code that affects the study case is EN 1991-1-4.

EN1991-1-4 guides in the determination of wind dynamics pretended for civil engineering works and structural design of building for the loaded areas that are under consideration. The study is produced in the whole structure but also in parts of the buildings or structure attached elements such as safety/noise barriers, cladding units or components and their fixings.

This standard covers a varied building dimensions and also varied shapes. However, there are several cases where the code gives unsatisfactory answers, or even lack of them. Is in those cases where wind tunnel tests, or even sometimes full scale tests, may be the solution for achieving an answer.

EN 1991-1-4 is meant for direct client use, contractors designers and authorities of importance. Furthermore, this code is meant to be used in a parallel approach with EN 1990. Also with the rest of EN 1991 parts and EN1992 to EN1999 about structures design.

Hence, EN 1991-1-4 is the European Code that is going to be followed all along this study project to make the comparison with the Finite Element Model approach. The main reason of choosing this Code for the other ones discussed in the previous section is due to choosing the Code that applies in Finland for the location of the CAARC building.

3.1.1 EN 1991-1-4 Wind Load Study

The Commission of the European Community decided to establish a construction program in 1975 in the aim of make an harmonization and standardization of technical specification all along Europe. However, it was not until 2010 when all European national Standards where substituted by the arranged Eurocode. Nevertheless, the Eurocode Standard left a National Annex for a national choice depending on the country which gives the Code a greater specificity.

In Eurocode Standard, action effects and structure resistance are independently treated: the so-called performance based design. [2]

The European code series is consisted of ten document series: EN1990 to EN1999. EN1991 is the one chosen for the actions. EN 1990 are the basis of design. In them the general principles for classification: defines characteristics and values for the design utilized in structure calculations.

EN 1991 are the actions on structures. Where actions on structure values are specified. Is divided in ten volumes, for a different action specification as mentioned before.

Then, 6 European codes for the specification of the strength of structures calculations for a specific material: EN1992 for concrete structures design, EN1993 for steel structures design, EN1994 for steel and concrete structures design, EN 1995 for timber structures design, EN 1996 for masonry structures design and EN1999 for aluminum alloy building design.

Being the European code series finally completed with EN1997 for the design of geo-technical aspects and EN1998 for earthquake resistant structures design.

Every European code consists of 58 documents and its used in at least 28 countries.

EN 1991-1-4 specifies wind actions for the design of building structure design and works in civil engineering in every considered area: this includes areas such as the structure as a whole but also attached to the structure elements such as cladding components.

The European Standard Wind Code can be used in heights up to 200m Buildings and other works of civil engineering and no span greater than 200m bridges, satisfying the dynamic response criteria. So is the correct choice for the CAARC building characteristics.

EN 1991-1-4 does not apply much specification in characteristics such as wind turbulence vibrations, or torsional and bridge deck vibrations where is not enough with the fundamental mode. [2]

The wind induced force that acts either on the whole structure or a either just a structural element F_w can be determined by three procedures in EN 1991-1-4: A first and simplified procedure:

$$F_W = C_f \cdot C_s C_d \cdot q_p(z_e) \cdot A_r ef \tag{3.1}$$

The second with vector summation

$$F_W = C_s C_d \cdot \sum C_f \cdot q_p(z_e) \cdot A_{ref} \tag{3.2}$$

And a third that differentiates side of the structure pressures analog to the second but divided in external and internal pressures.

Where F_W is the wind induced force, ρ is the density of air, C_p is the pressure coefficient for the effect under consideration while C_d is called dynamic factor, A_{ref} is the reference area, C_s is a size factor taking into account the wind pressures on a building lack of correlation and q_p is the peak dynamic pressure. [16]

Wind loading value is the multiplied by a security factor that varies in case of load combination and needs to always be taken into account during the wind load design stage.

It is necessary to specify the wind that is taken into account in order to make the proper wind actions on building requirements. Hence, it is needed to determine the peak velocity pressure q_p as a primary parameter that accounts mean and turbulent component of wind loads. [2]. Following EN 1991-1-4 peak wind velocity is given by the next expression.

$$q_p(z_e) = [1 + 7 \cdot I_v(z_e)] \cdot \frac{1}{2} \cdot \rho \cdot V_m^2(z_e) = C_e(z_e) \cdot q_b \cdot C_0(z_e) \tag{3.3}$$

where

$$q_b = \frac{1}{2} \cdot \rho \cdot V_b^2 \tag{3.4}$$

and

$$C_e(z) = \frac{q_p(z)}{q_b} \tag{3.5}$$

where $I_v(z)$ is the turbulence intensity, C_0 is the orography factor, v_b is the basic wind velocity z is the height above ground; z_0 is the roughness length; ρ is the mass density of air, k_r is the terrain factor and q_b is the basic velocity pressure.

Basic wind velocity v_b is EN1991-1-4 defined using the fundamental basic wind velocity. [16] This fundamental basic wind velocity has a return of 50 years with 10 minute mean wind speed obtained statistically. The values are not given by EN 1991-1-4 but some countries gather typical values or either iso-lines in their National Annexes. [2]

Attending to terrain categories, roughness length and terrain factor, EN1991-1-4 provides 5 terrain categories (See Annex A [16]), with given values for the roughness length z_0. It is established to use $v(z)$=$v(z_{min})$ in case of $z < z_{min}$.

The exposure factor C_e have also a relatively high importance in the EN 1991-1-4 Code. It measures the dependence of wind effects depending on height and on the terrain roughness, wind velocity peaks and orography.

From EN 1991-1-4 it can be seen how C_e varies with height depending on the terrain category in Fig. 3.1. [16]

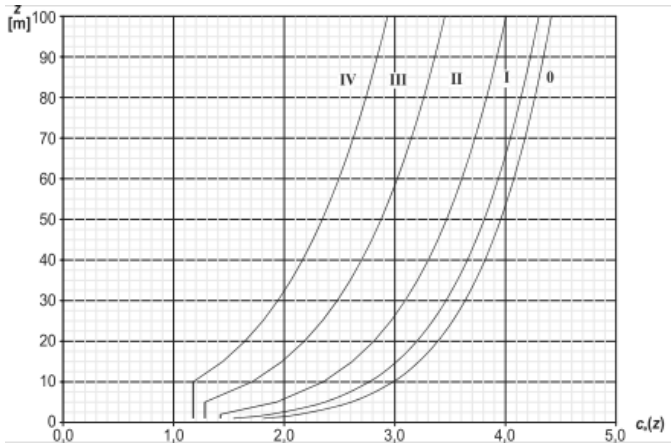


Figure 3.1: C_e variation with height depending on the terrain category. [16]

3.1.2 EN1991-1-4 Dynamic Response Considerations

Stathopoulos and Baniotopoulos (2007) talked about how prone slender structures are to suffering dynamic responses [2]. Due to this condition, it was specified by EN1991-1-4 a structural factor C_sC_d which takes into account the wind actions of peak wind pressures along with turbulence vibrations effect due to not occurring simultaneously.

This structural factor is divided in: C_s as a size factor taking into account the wind pressures on a building lack of correlation and C_d as the dynamic factor, for the resonance effect consideration. According to EN 1991-1-4 [16] is given by the following expression.

$$C_sC_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_s)} \tag{3.6}$$

where z_e is the reference height, B^2 is the background factor, allowing the pressure on the building surface lack of full correlation, I_v is the turbulence intensity, k_p is the peak factor (ratio of the maximum value of the response fluctuating part between the standard deviation) and R^2 is the resonance response factor, allowing for turbulence in resonance with the vibration mode.

The size factor C_s is also given by the following expression according to EN 1991-1-4.

$$C_s = \frac{1 + 7 \cdot I_v(z_e)\sqrt{B^2}}{1 + 7 \cdot I_v(z_e)} \tag{3.7}$$

The dynamic factor C_d, for its part, is given by the following expression according to EN 1991-1-4.

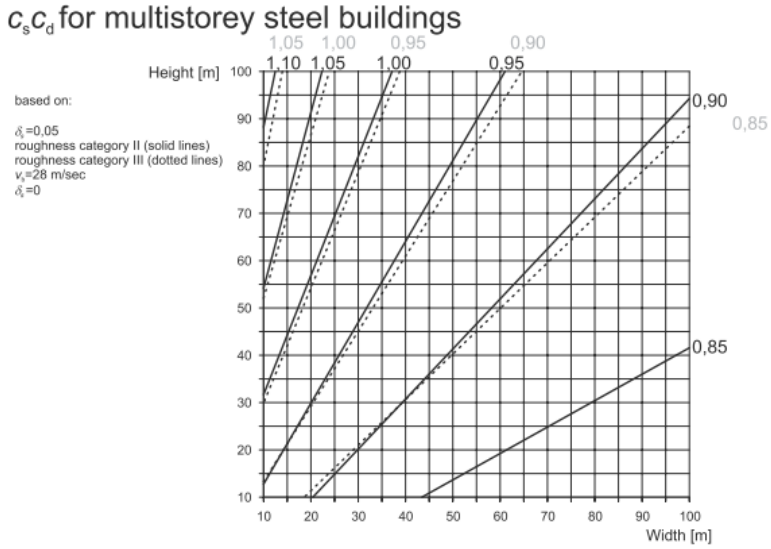


Figure 3.2: Estimations of C_sC_d for structural systems variations. [16]

$$C_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_e)\sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_e)\sqrt{B^2}} \tag{3.8}$$

According to the European Standard, the vibration in the along-wind direction fundamental mode has great relevant and with constant sign only when the under consideration structure is horizontal or vertical, e.g., a bridge or a building. Thus, the following mode contributions are negligible according to the standard.

EN1991-1-4 gives two calculation alternatives, Annex B and Annex C, for the background factor B^2 and the resonance response factor R^2 determination. This two procedures are explained in Chapter 4 Section 4.1.3. Safe estimations of C_sC_d for structural systems variation are given in Fig. 3.2 by EN1991-1-4. [16]

3.1.3 National Annex

EN 1991-1-4 established a number of clauses where there is a national choice allowance. Between those clauses terrain categories and wind climate are two of the main subjective fields, but also other selections such as force and pressure coefficients values selection, the dynamic response procedure choice or vortex excitation. In spite of this *a priori* freedom of selection, it is also established that every country must publish their National Annex whit the clauses value selection specifications.

Hence, it is not possible to use European Wind Standards without the use of this National Annex. Some decisions are not obligatory but may be sufficient to claim the only for information note statement while other choices are mandatory like the

wind climate definition. For all National Annexes it is necessary to provide a use guidance. [2]

3.2 Comparison Between Mainly Used Standards

There are a few international codes/standards, with some differentiation in their considerations on the study of wind loading: CNS 2012 (China) [36], AIJ 2004 (Japan) [37], AS/NZ 2011 (Australia/New Zealand) [11], ISO 2009 [38], NBCC 2010 (Canada) [39], Euro-code 2010 (Europe) [16], ASCE 2010 (USA) [40] and IWC 2012 (India) [41]. It is precise to make a comparison between them in order to understand their commonalities along with their differences and, thus, comprehend which codes are the most complete. The comparison key areas are the serviceability design considerations as well as the service requirements in along and across-wind directions.

There has been some researches on the earlier versions of codes comparison that studied that the definitions variation of characteristics of the wind field, such as those that include wind spectrum, mean wind velocity profile, length scale, turbulence intensity and the wind correlation in the structure, as the prior response quantities predictors contributors. [42, 43] Nevertheless, this study is going to be in a parallel approach to the one made by Kwon and Kareem (2013). [44]

Despite the fact that all the standards are focused on random vibration gust loading factor approaches [10], they define different parameters in order to asses the along-wind dynamic load and consequent effects on the structures together with diverse provisions for the across-wind and torsional forces. [44].

Kwon and Kareem (2013) made a comparative study in ”the key areas of comparison” including ”the provisions in the along-wind and across-wind directions for serviceability design as well as serviceability requirements”. They were compared for 3 example cases discussing the predictions differences and also suggesting to further upgrade the agreement between the codes in order to be able to determine what a global standard should include. [44]

3.2.1 Wind Loads and Characteristics in Standards/Codes

It is precise to make a global study of the most relevant features that the wind load assessment has depending on the standard. All standards recommend the use of wind tunnel test for large and irregular buildings. Nevertheless, standards are used, even in those situations, for the preliminary wind tunnel conditions design study with basic wind speed input, turbulence or terrain category determination.

The determination made by the different codes for basic wind velocity (v_0) varies

Table 3.1: Reference heights and average times. [44]

	ASCE	AS/NZ	AIJ	CNS	NBCC	EU	ISO[a]	IWC
Basic wind velocity (V_0)	3-s	3-s	10-min	10-min	1-h	10-min	3-s 10-min	3-s
Wind-induced response	1-h	10-min	10-min	10-min	1-h	10-min	10-min	1-h
Reference height (h_{ref})	0.6h[b]	h	h	h	h	0.6h	h	h

[a] ISO provides two procedures for determining loads: one for peak response and one for mean response.
[b] h = building height.

depending on each code considerations along with the wind induced response. Hence, IWC, AS/NZ, ASCE and ISO and standards define v_0 as a three second gust speed while, in other standards, it is defined as the wind speed mean, e.g., 10 min mean in AIJ, CNS, EN 1994 or even 1 hour mean in NBCC. Nevertheless, it is common for all codes to use a longer averaging time as can be seen in Table 3.1. [44] The actual currents advance in the idea of defining a shorter duration of the peak gust wind speed. It can also be seen in Table 3.1 the differences among the reference heights between the codes that affect the parameters in the gust effects factors and loads calculations. [45, 46]

The wind velocity profile is described, depending on the standard, by either a logarithmic (AS/NZ, EN 1991 and ISO) or by a power law (the rest of the codes including ISO for calculations of resonant response). The power law is generically given as:

$$v(z) = b'\left(\frac{z}{10}\right)^{\alpha'} v_0 \tag{3.9}$$

where α' and b' depend on the terrain (some examples of the power laws coefficients can be seen in Table 3.2 [44]), z is the relevant height and v_0 is the basic wind speed. On the other hand, the logarithm law is generically described as:

$$v(z) = \frac{u^*}{0.4} \ln\left(\frac{z}{z_0}\right) v_0 \tag{3.10}$$

where u^* is the friction velocity and z_0 is the roughness length. It can be seen in Table 3.3 the roughness length and friction velocity for profiles in AS/NZ, EN1991 and ISO.[44]

Table 3.2: Power law coefficient for CNS, AIJ, NBCC, ASCE, and IWC. [44]

Category	ASCE				AIJ		CNS		NBCC		IWC	
	3-s		1-h		10-min		10-min		1-h		3-s	
	α	b	α	b	α	b	α	b	α	b	α	B
EC1	–	–	–	–	0.35	0.394	–	–	–	–	0.0683[a]	1.030[a]
EC2	0.20	0.64	0.33	0.30	0.27	0.576	0.30	0.52	0.36	0.426	–	–
EC3	0.14	0.84	0.25	0.45	0.20	0.794	0.22	0.73	0.25	0.666	0.0850	0.980
EC4	0.11	1.00	0.15	0.65	0.15	1.000	0.15	1.00	0.14	1.000	0.0765	1.033
EC5	0.09	1.07	0.11	0.80	0.10	1.230	0.12	1.13	–	–	0.0690	1.072

[a] These coefficients are valid above 100 m height.

Table 3.3: Roughness length and friction velocity for profiles in AS/NZ, EN1991 and ISO. [44]

Category	AS/NZ[a]		EU		ISO[a]	
	u^*	z_0	u^*	z_0	u^*	z_0
EC1	0.086–0.102	2	–	–	0.1–0.109	3
EC2	–	–	0.094	1	–	–
EC3	0.075–0.083	0.2	0.086	0.3	0.081–0.089	0.3
EC4	0.064–0.070	0.02	0.076	0.05	0.069–0.074	0.03
EC5	0.055–0.061	0.002	0.068	0.01	0.059–0.064	0.003
EC6	–	–	0.062	0.003	–	–

[a] Friction velocity for AS/NZ and ISO standards are estimated from fit of tabulated values.

3.2.1.1 Along-Wind Loads Standards Comparison

A generic equation for wind pressures p_z on a structure with wind incidence can be defined as the expression given below.

$$p_z = q_z \cdot G \cdot C_p \tag{3.11}$$

where G is the gust effect factor, q_z is the height z velocity pressure and C_P is the pressure coefficient. Combining the affluent areas along with the acting wind pressures it is possible to determine the consequent loads. The velocity pressure q_z can be determined by the following expression.

$$q_z = \frac{1}{2}\rho V_0^2 \cdot C_{exposure} \cdot C_{topography} \cdot C_{direction} \cdot C_{importance} \cdot C_{other} \tag{3.12}$$

where ρ is the air density and all C´s are their respectively indicated factors.

The gust effect factor G for the standards is determined by the following expression.

$$G = \frac{GLF}{G_q} \tag{3.13}$$

where G_q is the gust factor of wind velocity pressure and GLF is the gust loading factor [47]. G is only utilized with G in ASCE7, while in AS/NZ, ISO and IWC it is the dynamic response factor (C_{dyn}) and in EN1991 is used as a structural factor (C_sC_d).

The peak factor, on the other hand, describes the peak of fluctuating response. The typical distribution is Gaussian with values that, depending on the standard, vary between 3 and 4. Its general expression is:

Table 3.4: Parameters comparison and peak factors. [44]

	g	T (s)	ν
ASCE	$\sqrt{2\ln(\nu T)} + \frac{0.577}{\sqrt{2\ln(\nu T)}}$	3600	f_0
AS/NZ	$\sqrt{2\ln(\nu T)}$	600	f_0
AIJ	$\sqrt{2\ln(\nu T)} + 1.2$	600	f_0C
CNS	2.5	600	f_0
NBCC	$\sqrt{2\ln(\nu T)} + \frac{0.577}{\sqrt{2\ln(\nu T)}}$	3600	f_0C
EU	$\sqrt{2\ln(\nu T)} + \frac{0.6}{\sqrt{2\ln(\nu T)}}$	600	f_0C
ISO	$\sqrt{2\ln(\nu T)} + \frac{0.577}{\sqrt{2\ln(\nu T)}}$	600	f_0
IWC	$\sqrt{2\ln(\nu T)}$	3600	f_0

f_0 = natural frequency of a building [Hz]; C = a factor which is a function of the background and resonant responses.

$$g = \sqrt{2 \cdot ln(\nu \cdot T)} + \frac{0,5772}{\sqrt{2 \cdot ln(\nu \cdot T)}} \tag{3.14}$$

where T is the average time and ν is the upcrossing rate that depends, and its similar, to the buildings natural frequency (Hz). A comparison between all the standards/codes can be seen in Table 3.4.

The gust factor G_q is meant to compensate the time average differences, such as the one that appears between basic wind velocity and wind velocity pressure or calculation of the induced response q_z. This gust factor G_q depends on the turbulence intensity I_z profile that is given in most of the codes by the following expression.

$$I_z = c'\left(\frac{10}{z}\right)^{d'} \tag{3.15}$$

where d' and c' are exposure category EC variables related and z is the meant height. On the other hand, EN1991 has adopted the following logarithmic law given in Ex. 3.16.

$$I_z = \frac{1}{\ln\left(\frac{z}{z_0}\right)} \tag{3.16}$$

where z_0 is the roughness length. On the other hand, in AS/NZ, IWC and ISO, the turbulence intensity profiles are tabular. All values depending on the code and the exposure category (EC) can be seen in Table 3.5. [44]

The background response factor (B) definition has a resounding variation depending on the chosen code, as it can be seen in Table 3.6, while in NBCC

Table 3.5: Turbulence intensity I_z profile parameters. [44]

EC	ASCE		AS/NZ[a]		AIJ		CNS		NBCC		EU	ISO	IWC[b]	
	c	d	c	d	c	d	c	d	c	d	z_0	z_0	c	d
1	–	–	0.45	0.30	0.53	0.40	–	–	–	–	–	3.000	0.47	0.30
2	0.45	0.167	–	–	0.36	0.32	0.39	0.30	0.51	0.36	1.000	–	–	–
3	0.30	0.167	0.32	0.30	0.26	0.25	0.23	0.22	0.27	0.25	0.300	0.300	0.45	0.40
4	0.20	0.167	0.26	0.30	0.20	0.20	0.14	0.15	0.16	0.14	0.050	0.030	0.40	0.45
5	0.15	0.167	0.19	0.30	0.16	0.15	0.12	0.12	–	–	0.010	0.003	0.35	0.48
6	–	–	–	–	–	–	–	–	–	–	0.003	–	–	–

Table 3.6: Background response factor depending on the Standard. [44]

	Background response factor (B)
ASCE	$\frac{1}{1+0.63\left(\frac{b+h}{L_{h_{ref}}}\right)^{0.63}}$
AS/NZ	$\frac{1}{1+\frac{[(0.26h^2)+0.46b^2]^{0.5}}{L_{h_{ref}}}}$
AIJ	$\frac{4(0.49-0.14\alpha)^2}{\left\{1+\frac{0.63\left(\sqrt{bh}/L_{h_{ref}}\right)^{0.56}}{(h/b)^k}\right\}^2}$ $\quad k=\begin{cases}0.07, & h/b \geqslant 1\\ 0.15, & h/b<1\end{cases}$
CNS	–
NBCC	$4/3\int^{914/h}\left[\frac{1}{1+\frac{xh}{457}}\right]\left[\frac{1}{1+\frac{xb}{122}}\right]\left[\frac{x}{(1+x^2)^{4/3}}\right]dx$
EU B	$\frac{1}{1+0.9\left(\frac{b+h}{L_{h_{ref}}}\right)^{0.63}}$
EU C	$\frac{1}{1+\frac{3}{2}\sqrt{\left(\frac{b}{L_{href}}\right)^2+\left(\frac{h}{L_{href}}\right)^2+\left(\frac{b}{L_{href}}\frac{h}{L_{href}}\right)^2}}$
ISO	$\left[\{1+0.2\alpha\}\Big/\left\{1+\frac{0.63(\sqrt{bh}/L_{h_{ref}})^{0.56}}{(\frac{h}{b})^k}\right\}\right]^2$ $\quad k=\begin{cases}0.07, & h/b \geqslant 1\\ 0.15, & h/b<1\end{cases}$
IWC	$\frac{1}{1+\frac{[36h^2+64b^2]^{0.5}}{2L_{h_{ref}}}}$

B is given in a graphic way. The resonant response factor R, on the other hand, is typically expressed as given below.

$$R = \frac{\pi S E}{4\zeta} \tag{3.17}$$

where S is the factor of size reduction, ζ is the damping ratio and E is an energy factor. [48]

The pressure coefficient C_p in Ex. 3.11 can be divided in pressures given internally and externally. All Codes give external pressure coefficients in both windward and leeward directions. Nevertheless, in contrast with the external pressure coefficients that are building height reasonable consistent, the internal pressure coefficients has bigger differences between the different codes.

3.2.1.2 Cross-Wind and Torsional Loads Standards Comparison

In terms of across-wind loading and torsional loading there are more differences between the codes. EN1991 and NBC use partial load approach to consider this cross and torsional loads, while ASCE uses a data-based design called NALD (NatHaz Aerodynamic Loads Data-base) and IWC gives a estimate determination process for the across-wind loads. Furthermore, torsion is considered in NBCC and EN1991 by and applied moment defined by the along-wind load, while NALD, AIJ, CNS ,AS/NZ and ISO give determining procedures for the across-wind and torsional loads based on bending moment.

Most codes have traditionally relied on simplified formats based on graphs and tables that could affect the level of accuracy depending on interpolation/extrapolation of the information.

3.2.1.3 Acceleration Response Standards Comparison

It is precise to ensure human comfort in tall buildings in terms of acceleration attending the serviceability requirements. Except for the CNS, every code provide expressions for defining along-wind accelerations, but it is not the case of across-wind and torsional accelerations that are sometimes neglected in terms of human comfort over serviceability requirements. The along-wind acceleration can be generally given as the following expression.

$$\hat{\ddot{x}}(z) = \frac{C_{fx} \cdot q_{h_{ref}} \cdot G_R \cdot b \cdot h \cdot K}{m_1} \cdot \phi_1(z) \tag{3.18}$$

where m_1 is the first mode generalized mass, C_{fx} is the drag force coefficient given by the absolute windward and leeward pressure coefficients sum , $q_{h_{ref}}$ is the velocity pressure at the reference height, K is the mode shape correction factor, h is the building height, b is the building width and $\phi_1(z)$ is the first mode shape evaluated at height z that is approximated with a power form. Definitions of $q_{h_{ref}}$, G_R, and K can be seen in Table 3.7 depending on the Standard. Moreover, for NBCC and ISO the along-wind acceleration is given by the following expression.

$$\hat{\ddot{x}}_{max} = \frac{G_R}{1 + \sqrt{G_B^2 + G_R^2}} \cdot (2\pi f_0)^2 x_{max} \tag{3.19}$$

where x_{max} is the maximum displacement and f_0 the natural frequency. In the case of NBCC and ISO, there is a lack of maximum displacement estimation. Nonetheless, there are different expressions to estimate the across-wind acceleration in ISO, NBCC together with AIJ, AS/NZ and NALD. As a general expression, it can be used the one is given below.

Table 3.7: Along-wind acceleration depending on the Standard. [44]

	$q_{h_{ref}}$	G_R	K
ASCE	$\frac{1}{2}\rho\bar{V}^2_{h_{ref}}$	$1.7 g_R I_{h_{ref}} \sqrt{R}$	$\frac{1.65^{\hat{\alpha}}}{\alpha+k+1}$
AS/NZ	$\frac{1}{2}\rho V^2_{h_{ref}}$	$\frac{2 g_R I_{h_{ref}} \sqrt{R}}{\left(1+2 g_v I_{h_{ref}}\right)}$	$\frac{\int_0^h V^2(z) z dz}{V_h^2 h^2}$
AIJ	$\frac{1}{2}\rho\bar{V}^2_{h_{ref}}$	$g_R I_{h_{ref}} \sqrt{R}$	$1 - 0.4 \ln k$
EU B	$\frac{1}{2}\rho\bar{V}^2_{h_{ref}}$	$2 g_R I_{h_{ref}} \sqrt{R}$	$\frac{(2k+1)\left[(k+1)\left[\ln(\frac{h}{z_0})+0.5\right]-1\right]}{(k+1)^2 \ln\left(\frac{h}{z_0}\right)}$
EU C	$\frac{1}{2}\rho\bar{V}^2_{h_{ref}}$	$2 g_R I_{h_{ref}} \sqrt{R}$	$\frac{k_y k_z}{\varphi_{max}}$; $k_y, k_z = \begin{cases} 1 & \text{uniform} \\ \frac{3}{2} & \text{linear} \\ \frac{5}{3} & \text{parabolic} \\ \frac{4}{\pi} & \textit{sinusoidal} \end{cases}$
IWC	$\frac{1}{2}\rho V^2_{h_{ref}}$	$\frac{2 g_R I_{h_{ref}} \sqrt{R}}{(1+2 g_v I_{h_{ref}})}$	$\frac{\int_0^h V^2(z) z dz}{V_h^2 h^2}$

$\alpha, \hat{\alpha}$ = exposure exponents of wind velocity profile; k, k_y, k_z = mode shape exponents; φ_{max} = maximum mode shape coefficient.

$$\hat{\ddot{y}}(z) = \frac{\hat{M}_R \cdot K_y}{m_0 \cdot h^2} \cdot \varphi_1(z) \tag{3.20}$$

where K_y is the mode shape correction, $\hat{M}_R$ is the resonant component of the cross-wind moment and m_0 is the mass per height. Used values and expressions can be seen in Table 3.8 [44]. For the torsion acceleration study it can generally be defined, attending the codes that consider it, as the following expression.

$$\hat{\ddot{\theta}}(z) = \frac{\hat{M}_T \cdot K_t}{I_0 \cdot h} \cdot \phi_1(z) \tag{3.21}$$

where K_t is the torsional moment of the mode shape, $\hat{M}_T$ is the resonant component and I_0 is the mass moment of inertia. Used values and expressions can be seen in Table 3.8 [44, 49]

3.2.2 Comparison Conclusions

Kwon and Kareem (2013) after making the comparison between all the codes in terms of similarities and differences, together with the places where those comparisons affect the design study conclude that "overall loads are reasonably consistent in the along-wind response but more scatter is observed in the across-wind response". [44]

In terms of wind velocity they claim that "the parameters associated with the wind

Table 3.8: Across-wind and torsional acceleration values depending on the Standard. [44]

	$\hat{M}_R$	K_y	$\hat{M}_T$	K_t
NALD	$g_R \bar{q}_h d h^2 \sqrt{\frac{\pi}{4\zeta} \sigma_{cm}^2 C_m(f_2)}$	$k+2$	$g_R \bar{q}_h d b h \sqrt{\frac{\pi}{4\zeta} \sigma_{cm}^2 C_m(f_3)}$	$k+1$
AS/NZ	$g_R \bar{q}_h b h^2 \sqrt{\frac{\pi}{4\zeta} \sigma_{cm}^2 C_m(f_2)}$	$0.72k+2.28$	–	–
AIJ	$g_R \bar{q}_h b h^2 \sqrt{\frac{\pi}{4\zeta} \sigma_{cm}^2 C_{m,R}(f_2)} C_{m,R}(f_1) = \lambda^2 F_L$	$2k+1$	$g_R \bar{q}_h b^2 h \sqrt{\frac{\pi}{4\zeta} \sigma_{cm}^2 C_{m,T}(f_3)} C_{m,T}(f_1) = \lambda^2 F_T$	$1.2k+0.6$
IWC	$g_R \bar{q}_h b h^2 \sqrt{\frac{\pi}{4\zeta} \sigma_{cm}^2 C_m(f_2)}$	$0.72k+2.28$	–	–

velocity characteristics contribute the most towards apparent differences in the resulting wind responses such as base shear/moment and peak/RMS acceleration [...] while discrepancies with other parameters such as peak factors, turbulence intensity, energy factors, etc. contribute to some differences in the overall loads, the standardization of wind loading codes/standards may be achievable by eliminating differences in the velocity profiles". [44] This is mainly correct in the case of the along-wind load effects over the gust loading approach, or effect, factor. In the case of the across-wind and torsional loads where there is an increase in wake effect relevance over buffeting effects in the along-wind direction, "a database-enabled design framework such as NALD used in ASCE is the most promising design procedure on tall buildings wind-incidence study", according to Kwon and Kareem, in order to increment accuracy. [44]

Therefore, following all this considerations of Kwon and Kareem study it can be stated to be possible the determination of a global standard, notwithstanding, it would not be an easy task to get every involved country in agreement.

3.2.3 Wind Standards Application of Wind Tunnel Experiments

Stathopoulos and Baniotopoulos (2007) claim "Experiments have been essential in the development of current design procedures for wind loads on structures" [2]. This is due to the fact that the design coefficients in codes and guidelines wind tunnel tests based. They made a study about European Wind Code and the experimental approach. They described some of the principles and some backgrounds over EN 1991-1-4. Additionally, they gave some of the main EN1991-1-4 properties. They also treated the principal aspects that are required for wind tunnel tests with boundary conditions, focused in wind forces for design stage buildings and the sometimes importance of full scale tests.

Wind tunnel tests are sometimes even utilized as alternative for standards in situations where the code does not present enough accuracy and it comes to be needed to obtain the wind load over the studied structure in a more precisely way. Stathopoulos and Baniotopoulos (2007) work is focused in the boundary

conditions and principles of wind tunnel experiments used as a on structures wind load finding tool. [2]

Factors within a wind tunnel test, e.g. the wind, building and its surroundings characteristics, and their particular behaviour, are on scale to the reality modelled. Hence, it is possible to determine wind pressures, velocities, moments, forces or accelerations. In order to properly make the wind loading study, data extracted from the tests is transferred into non-dimensional coefficients like pressure coefficients that can be defined in several alternatives, depending on the wind speeds reference, with the definition of different statistical properties. [6]

Nevertheless, despite the many virtues of wind tunnel tests, they also present some limitations. That is why experiments are sometimes carried out fully scaled. The procedure of wind tunnels is to scale the actual size of buildings. However, when it is precisely small elements that are to be simulated, full-scale simulation acquires a very relevant importance. Another case in which the full-scale technique is used is when the wind tunnel experiments themselves are intended to be tested. In this way, some wind Standards reflect cases in which the use of this simulation technique is allowed. [2]

Papers on fully scaled measures are given in ”The Journal of Wind Engineering and Industrial Aerodynamics” and ”The Journal of Wind and Structures” .

Chapter 4

Acceleration of a CAARC Building under Wind Incidence Study Development

This chapter is the main one within the project, since it carries out the study and comparative of the acceleration produced by the wind in a CAARC Standard Tall Building within the European Wind Code and of the Finite Elements FE numerical study.

This significant chapter has been divided into a first analysis were considerations on different aspects are developed: determination of nodal simulated wind loads, CAARC building for its introduction in the FE program and a necessary analysis carried out on EN 1991-1-4.

After that, the two analysis procedures were developed. Thereby, the determination of the acceleration suffered by a CAARC Building due to wind incidence has been obtained both by means of the European Wind Standard and by the Finite Elements FE numerical study. This study has been carried out based on a realistic scenario where there is vertical and horizontal correlation. There has also been a comparative between the realistic case with vertical and horizontal correlation and the simplified approach according to EN 1991-1-4.

Finally, this chapter concludes with a comparison on the results obtained along with a critical review on what was achieved for both the FEM-Simulation and European Standard approaches.

4.1 Wind Loads, CAARC Building and EN 1991-1-4 Analysis

In this first section of chapter 4, all relevant analysis needed for the development of CAARC Building acceleration study, will be introduced. Therefore, it will be separated in three main analysis aspects: Wind Loads Simulation value

determination according to EN 1991-1-4, all CAARC building considerations and the analysis needed for the FEM Simulation approach of determining acceleration and, finally, all the EN 1991-1-4 related aspects in order to achieve the acceleration following the Code specifications.

Hence, the first part of this section is a compilation work including the procedure to determine time dependent wind simulated loads discretizated along nodes for the vertical and horizontal correlated case. This research work was made by Olli Lahti.

The second subsection includes CAARC and Fluid specifications for the FEM approach, such as, how the discretization of the building is going to be done for the determination of acceleration study case in the FEM approach and which are the material properties, mode shape and building damping introduction values for the FEM procedure that will be selected.

To conclude this section, the EN 1991-1-4 considerations are to be stated, these include, the two main procedures that EN 1991-1-4 offers for determining acceleration, and the reason for choosing one between them, the aerodynamic admittance function consideration and its relation to the two study cases that are to be faced, the considerations of a CAARC building on EN 1991-1-4 analysis and, finally, the study of the considerations that EN 1991-1-4 states over cross-wind study.

4.1.1 Wind Loads Simulation - EN 1991-1-4 based

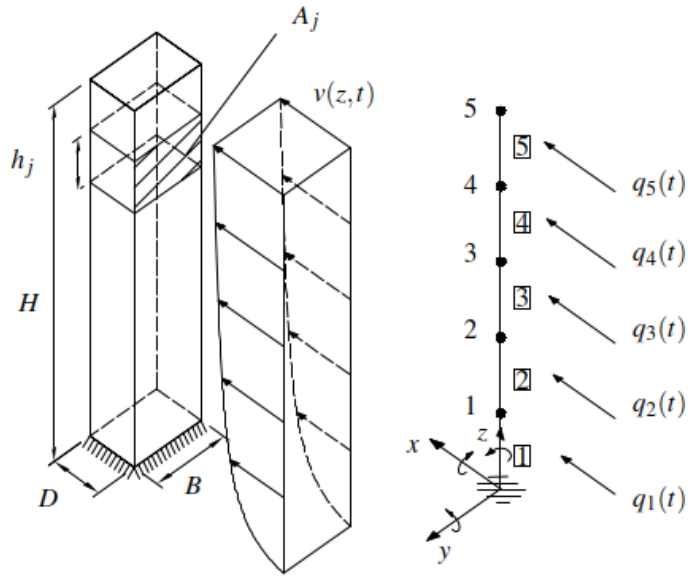
In order to be able to make the proper study of this thesis it is necessary to use a correct Simulation Wind Loads with the basis established by the EN1991-1-4. Wind velocity and wind pressure are going to be studied along this section. Full development of the study is carried out in Annex C, being stated in this section the main interest expressions.

In this case, the thesis´s author role was a recompilation work in order to be able to achieve wind load simulation values used in Section 4.2.2.2 FE-study. Furthermore, it is necessary to reflect this wind simulation values research was made by Olli Lahti.

Wind velocity must be defined for the case of study of this thesis. Thus, a building structure in a wind flow with a time dependent wind speed $v(z,t)$, assumed stationary Gaussian process, is going to be considered, composed of a mean and a fluctuating part as described. [4]

$$v(z,t) = v_m(z) + v'(z,t) \tag{4.1}$$

Hence, the study is going to be done attending to the wind force fluctuating component acting on a differential surface area dA, at height z, as the mean wind



(a) Building subjected to wind load. **(b)** Discretized model of the building.

Figure 4.1: Building simplification under study.

part remains constant. The fluctuating wind component has a mean of zero and standard deviation of σ_v. In Annex C it can be seen the full development.

Furthermore, in this project study there is going to be a vertical and horizontal correlation assumption, considering that along each of the nodes there is uniform wind incidence thanks to the use of the aerodynamic admittance function. The procedure will be the discretization shown in Fig. 4, as done by Castro and Bortoli (2015). On the other hand, there is a case that represents a simplified wind loading case on a building where there is only vertical correlation that gets out of the FEM simulation analysis bounds of this project. See Fig. 2.7 and 2.8 for the vertical and vertical and horizontal correlation justification, respectively [2].

Therefore, in the study aim of achieving the wind loads simulation values, a consideration of a building divided in N number of nodes and N_e number of elements is going to be done. Hence, following Annex C development, nodal wind load cross-spectral densities can be unified into a matrix of spectral density. Thus, the spectral matrix is defined by the following expression.

$$S_q(n) = \begin{bmatrix} S_{q,11}(n) & S_{q,12}(n) & \dots & S_{q,1N_q}(n) \\ S_{q,21}(n) & S_{q,22}(n) & \dots & S_{q,2N_q}(n) \\ \vdots & \vdots & \ddots & \vdots \\ S_{q,N_q1}(n) & S_{q,N_q2}(n) & \dots & S_{q,N_qN_q}(n) \end{bmatrix} \tag{4.2}$$

Introducing Cholesky factorization, spectral matrix can be factored as follows.

$$S_q(n) = \mathbf{L}\,\mathbf{L}^T \tag{4.3}$$

where

$$\mathbf{L}(n) = \begin{bmatrix} L_{11}(n) & 0 & 0 & \dots & 0 \\ L_{21}(n) & L_{22}(n) & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ L_{N_q1}(n) & L_{N_q2}(n) & L_{N_q3}(n) & \dots & L_{N_qN_q}(n) \end{bmatrix} \tag{4.4}$$

It is necessary to make a Fourier Function Transformation in order to raise the wind force simulation in the time domain, and thus, be able to make the proper study. Time period is divided in numerous steps as T_p into N_t intervals of time of Δt length: T_p= $N_p\Delta t$, with r as a time value for the interval $r \in \{\ 1,2, \cdot\ \cdot\ \cdot\ N_t\}$: t_r=$r \cdot \Delta t$. In the same way, it is precise to introduce s as a frequency value so that $s \in \{\ 1,2, \cdot\ \cdot\ \cdot\ N_t\}$, frequency n as: n_s=$\frac{s}{N_t \cdot \Delta t}$.

For the simulation of nodal fluctuating part wind forces it is precise to attend to Ex. 4.2 following Wittig and Sinha (1975) approach [50], also applying the needed randomness to the expression studied by Shinozuka and Jan (1972) [51].

$$\begin{Bmatrix} \tilde{S}_1(\frac{s}{N_t \cdot \Delta t}) \\ \tilde{S}_2(\frac{s}{N_t \cdot \Delta t}) \\ \vdots \\ \tilde{S}_{N_q}(\frac{s}{N_t \cdot \Delta t}) \end{Bmatrix} = \Big(\frac{N_t}{2 \cdot \Delta t}\Big)^{1/2} \mathbf{L}\Big(\frac{s}{N_t \cdot \Delta t}\Big) \begin{Bmatrix} \zeta_{s1} \\ \zeta_{s2} \\ \vdots \\ \zeta_{sN_q} \end{Bmatrix} \tag{4.5}$$

where the matrix ζ_{sj}=$a_{sj} + i\, b_{sj}$, with $E\,[a_{sj}] = E\,[b_{sj}] = 0$ and $E\,[a_{sj}^2] = E\,[b_{sj}^2] = 0.5$, introduces the random component.

Using discrete Fourier Transformation, it is possible to transform from frequency to time domain, and thus, get the expression for the nodal force fluctuating part with respect to time t_r on a generic node j from the N_e nodes (full development in Annex C).

$$\tilde{q}_j(r \cdot \Delta t) = \frac{1}{N_t \cdot \Delta t} \sum_{s=0}^{N_t-1} \tilde{S}_j(\frac{s}{N_t \cdot \Delta t})\, exp\Big(i\frac{2\pi rs}{N_t}\Big) \tag{4.6}$$

To conclude, aerodynamic admittance formulas for horizontal and vertical case need to be considered, and thus, obtain the simulated fluctuating part loads for each of the nodes and each of the 0,1465 s step that has been considered in the

600 s total simulation with its random component. Thus, the realistic case will be developed by introducing the aerodynamic admittance function proposed by EN 1991-1-4 to decrease loads according to the horizontal correlation.

4.1.2 CAARC Building and Fluid Specifications - FEM Analysis

It is precise to make some CAARC Building and Fluid specifications of the chosen considerations in relation to the FEM study. Hence, there are going to be considered the FEM related CAARC Building dimensions and discretizations, material properties, mode shape and structure damping along with the FEM simulated wind forces in the incidence to the studied CAARC building.

4.1.2.1 CAARC Building FEM dimensions considerations

CAARC Standard Tall Building model proposed in 1969 has the established dimensions, stated in section 2.3.1, of 30.48x45.72x183.88 m. Nevertheless, for this FE study a rounding of the values has been carried out with a CAARC Standard Tall building with the same dimensions as the one chosen by Braun and Awruch (2009) or Castro and Bertoli (2015) of 30 x 45 x 180 m. [32, 20]

In the aim of decrease output and imput information data, a simplification of a Multi Degree Of Freedom MDOF system is going to be used, as Hosseini and Larki (2011) considered, " [...], instead of working with $n \times n$ matrices, resulted from the n-Degree-Of-Freedom system, the engineer works with $n_r \times n_r$ matrices ($n_r \ll n$), belonging to a physically simplified system which has n_r degrees of freedom. [...] In fact, it is possible to introduce for a multi-story building an equivalent building with the same overall height, but having fewer number of stories with the same story mass while the stories heights are more so that their stiffness values are less and the proportion of stiffness and mass of the original building is kept in the simplified model". [52]

Therefore, for the case of a building with constant stiffness over height, as the one considered for the thesis study, only three parameters are going to define the building in contrast with a four parameter dependent building in which lateral stiffness decreases with height: "(1) a dimensionless parameter α_0 that measures that degree of participation of overall flexural and overall shear lateral deformations in the building" claimed by Miranda and Reyes (2002) to be neglected[1]; "(2) the ratio of lateral stiffness at roof level to that at the base of the building", neglected for our study case, "(3) the fundamental period of vibration; and (4) a modal damping ratio that can characterize the damping in the structure". [54, 53]

[1] "[...] An accurate estimation of the non-dimensional a0 was not necessary and that in many cases approximate values could be estimated based on the lateral resisting system of the building. For moment-resisting frame buildings they noted that this parameter typically varies between 5 and 20." Miranda and Reyes (2002) [53]

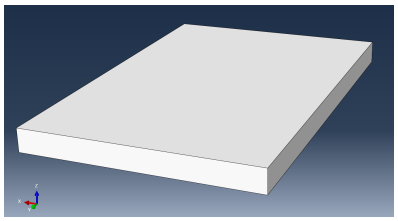


Figure 4.2: Node shape feature for the 60-story CAARC building model.

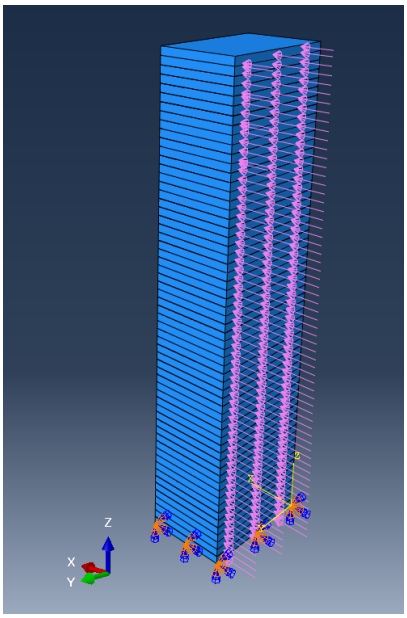


Figure 4.3: 60-story CAARC building model.

Hence, the positive multi-story building comparison work made by Taghavi and Miranda (2005), provides decision support in making the building modelling as a MDOF multi-story building of 60 nodes [54]. FEM model, thus, is going to be solved several times with several time histories of the fluctuating wind drag force in each of the nodes for the dynamic response analysis. This forces are going to be constant along the nodes due to the use of the aerodynamic admittance function proposed by EN 1991-1-4 to decrease loads according to the horizontal correlation. Therefore, following Castro and Bertoli (2015), the structure is going to be idealized as a vertical structure of 60 degrees of freedom, replacing the shearing mode model that used a mass, spring and damper system. The FE method will be used for the structure dynamic response computation with uniform time changing wind forces over this 60 nodes with each node is going to be 30 x 45 x 3 m, as shown in Fig. 4.2. The whole structure can be seen in Fig. 4.3.

4.1.2.2 CAARC Building FEM material properties

Attending to the material properties chosen for the FEM simulation of the CAARC Standard Tall Building, they are equally distributed and uniform along each of the nodes of the 60-story simulated building. Thus, it is precise to determine for each

of the nodes the elastic properties and the density values in order to ensure the desired stiffness of the simulated CAARC Building (structural damping also has to be chosen, however, it is developed in section 4.1.2.4). Due the fact that vibration analysis is computed with service limit loads, there is no need in adding plastic properties. [52]

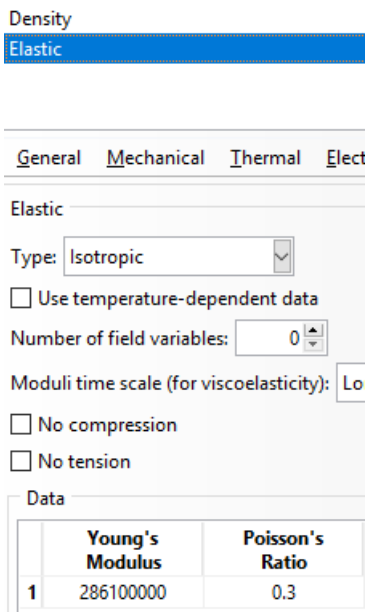
The Elastic Module for the flexure around y-axis E_y is defined by the following formula, as we have considered the FEM building to be wind-incised in the x axis as shown in Fig. 4.3. [8]

$$E_y = 3 \cdot \rho_c \left(\frac{4 \cdot \pi \cdot n_0}{d}\right)^2 \cdot \left(\frac{h}{1,875}\right)^4 = 2,861 \cdot 10^8 \, \frac{N}{m^2} \tag{4.7}$$

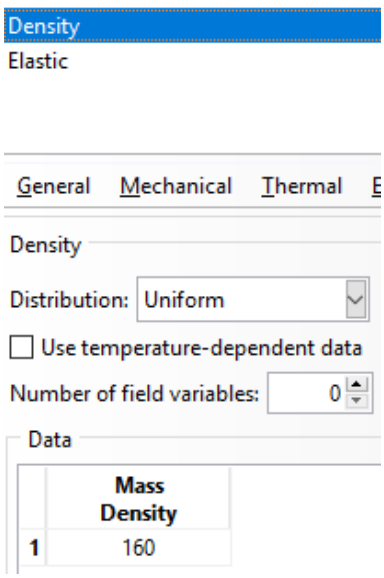
where h is building height, d is building along-wind length, n_0 is the first mode natural frequency that is considered 0.2 Hz for a CAARC Standard Building and ρ_c is the building mass density.

Continuing in the elastic properties definition, the poison ratio ν is chosen to be 0,3 as an average value for steel [8]. See figure 4.4(a) for elastic properties chosen values. On the other hand, it is precise to define the mass density. Specific mass is uniquely defined for every CAARC Standard Tall Building as 160 kg/m^3, so there is no decision needed to be taken. See figure 4.4(b).

In conclusion, with this considerations it is assumed that a correct discretization of the building has been made in terms of material properties. Hence, correct stiffness will be ensured with the well selected elastic properties and mass density once the natural frequency of 0,2 Hz is corroborated. [55, 56]



(a) Elastic values



(b) Mass density values

Figure 4.4: FEM material properties definition.

4.1.2.3 CAARC Building FEM mode shape study

CAARC Standard Tall Building, in relation to the dynamic properties considerations, was established to only consider the fundamental mode of vibration, being linear and rotating about a point at ground level. This natural frequency was assumed to be 0,2 Hz for both principal axes that affect at the considered ground level. It is necessary to ensure that our selected building with the selected properties and dimensions can guarantee the established fundamental mode of vibration previously stated. Therefore, it is going to be study that the first natural frequency is 0,2 Hz either numerically and with a FE study made by ABAQUS program.

Firstly, a building natural frequency is given by the following formula. [4, 8]

$$n_0 = \frac{\sqrt{\frac{E_y \cdot I_y}{\rho_c \cdot A}}}{2\pi} \left(\frac{1,875}{h}\right)^2 \tag{4.8}$$

with

$$I_y = \frac{1}{12} \cdot b \cdot d^3 \tag{4.9}$$

where n_0 is the first mode natural frequency, A is the cross section area (A=$b \cdot d$, E_y is the Elastic Module for the flexure around y-axis, I_y is the inertia moment of a rectangle respect to the y-axis, ρ_c is the specific mass of the CAARC building, h is building height and b is building width. Introducing all the previously stated values of each variable in Ex. 4.8, n_0 has the correct value of 0,19999 Hz.

On the other hand, it is equally important to make the FE modal analysis to assure that the building has the desired first frequency. Thus, introducing a *Frequency* study in the *Step* section of ABAQUS FE program, it is possible to get the first mode shapes. This way, the first eight mode shapes of the studied and simulated building where submitted in order to ensure the systems validity. The results can be seen in Table 4.1.

Table 4.1: First Mode Shapes results obtained in ABAQUS FE analysis.

Eigenvalue Output		
Mode No.	Frequency	
	RAD/s	Cycles/s
1	1.2389	0.19718
2	1.8085	0.28783
3	6.2015	0.98700
4	6.9663	1.10872
5	9.1715	1.45969
6	11.732	1.86721
7	17.081	2.71853
8	18.608	2.96156

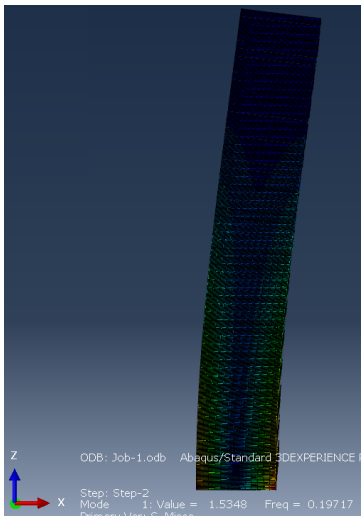


Figure 4.5: First Mode Shapes flexure plane xz obtained by ABAQUS FE program of the simulated CAARC Building.

The first mode frequency value n_0 is 0,19718 Hz, that is acceptable compared to the 0,2 Hz target (Fig. 4.1). Furthermore, It is also precise to study the plane where the first mode shape take place as it has to be the xz plane in this study case. In Fig. 4.5 it can be seen that the flexure plane is correct in relation to the x-directed simulated wind force incidence over the FEM building. This way, it is proved that mass and stiffness meets the marked values. This way only the damping is left in order to complete the acceptance of the FE simulated study building. [57, 58]

4.1.2.4 CAARC Building FEM damping study

The study concept relies on the comparison between the FEM analysis approach and the European Standard Code EN 1991-1-4 approach in solving the building wind-induced dynamic response. Thus, it is precise to assign the same damping to the CAARC simulated building in both FEM and EN 1991 cases.

Therefore, in section 4.2 where the study is developed, the damping decrement δ according to the European Code is obtained. In the mentioned section, it can be seen the value of the structural damping δ_s of $2\pi x_i = 0,063$ where x_i is the damping ratio to critical established as 0,01 for a CAARC Standard Tall Building. It is important to state that structural damping stays practically constant for every mode shape, this way it could be assume it is also 0,063 for the second natural frequency.

On the other hand, EN 1991-1-4 establishes a aerodynamic damping δ_a as the formula given by the following expression that has the value of 0,023 for the first natural frequency.

$$\delta_a = \frac{C_f \cdot \rho \cdot b \cdot V_m(z_s)}{2 \cdot n \cdot m_e} \tag{4.10}$$

where[2]

C_f : is the force coefficient for wind action in the wind direction. 2,19. (Ex. 3)

ρ : is the air density. 1,25 kg/m^3. (Table 4.4)

$V_m(z_s)$: is the mean wind velocity for reference height z=z_s. 16,13 m/s (Ex. 6)

b : is the width of the CAARC building structure. 45 m (Table 4.4)

n : is the natural frequency of the structure, that for the first mode is n_0=0,2 Hz.

m_e : is the along-wind fundamental equivalent mass per length of a CAARC building given by: $m_e = 160 \cdot b \cdot d$ =216000kg/m

Hence, aerodynamic damping δ_a, in contrast with structural damping δ_s, does vary with the different natural frequencies and is not possible to assume it is the same for every mode shape. For the first mode shape total structural damping is, thus, 0,086 (as $\delta_{a,0}$ is 0,023). [59]

On the other hand, ABAQUS damping property assignment to the simulated building, can be made by three ways: alpha and beta, composite and structural damping introduction. Following ABAQUS official website indications [60], alpha and beta is the correct way of introducing the desired damping as "composite modal damping allows you to define a damping factor for each material in the model as a fraction of critical damping" and "Structural damping assumes that the damping forces are proportional to the forces caused by stressing of the structure and are opposed to the velocity. Therefore, this form of damping can be used only when the displacement and velocity are exactly 90 grades out of phase" [60].

Therefore, it is necessary to establish alpha and beta values. They come from Rayleigh damping study. Rayleigh damping uses a damping matrix [C]. In the aim to simplify the modal analysis of damped MDOF systems, the so-called proportional or Rayleigh damping, is assumed whereby [C]= $\alpha_R M + \beta_R K$, with damping dependent on both mass M and stiffness K [8, 61]. In the aim of determining α_R and β_R for the study case, Rayleigh computation is to be followed, by the given expression. [8, 61]

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \end{Bmatrix} = \frac{1}{2} \begin{bmatrix} 1/2\pi n_0 & 2\pi n_0 \\ 1/2\pi n_n & 2\pi n_n \end{bmatrix} \begin{Bmatrix} \alpha_R \\ \beta_R \end{Bmatrix} \tag{4.11}$$

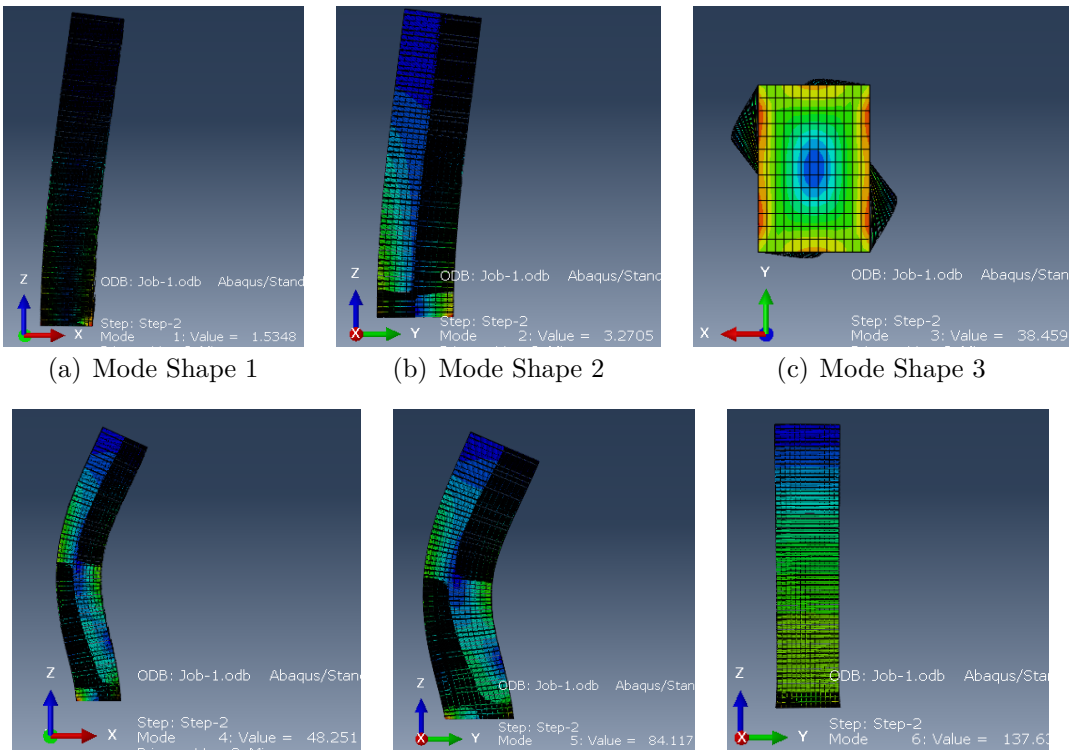
where ε_1 is the first damping ratio mode, ε_2 is the second damping ratio mode, n_0 is the first mode natural frequency that is considered 0.2 Hz and n_n is a frequency

[2]values source can be seen in EN 1991-1-4 study development made in Section 4.2 (Annex A)

”set among the higher frequencies of the modes that contribute significantly to the dynamic response” [8].

It is, thus, precise to determine which is the second natural frequency, and hence, obtain n_n. For its determination, it is necessary to go back to the already mentioned ABAQUS FE modal analysis stated in section 4.1.2.3 (*Frequency* study in the *Step* section of ABAQUS FE program). Attending to the results given by the simulation, it is possible to determine the second natural frequency of interest attending to the flexure plane. As the wind force is applied in x-axis the flexure plane of interest have to be xz. This way, as it can be seen in Fig. 4.6(a) and 4.6(d), the second natural frequency is Mode Shape No.4 of ABAQUS results and thus n_n is equal to 1.1055 Hz (Fig. 4.1).

Following *Dynamic of Structures*, Clough and Penzien (2003) [8], as it was previously stated, damping ratio ε can be assume to be the same for the first and second case attending to structural damping, but it is not continuous for the aerodynamic damping case. Therefore, ε_1 can be evaluated as 0,086/2π=0,013687 with the previously stated values for the first natural frequency, structural damping δ_s and corresponding aerodynamic damping $\delta_{a,0}$. On the other hand, returning to Ex. 4.10, aerodynamic damping of the second natural frequency $\delta_{a,n}$ can be obtained. This way, $\delta_{a,n}$ is given by Ex. 4.12.



(a) Mode Shape 1 (b) Mode Shape 2 (c) Mode Shape 3

(d) Mode Shape 4 (e) Mode Shape 5 (f) Mode Shape 6

Figure 4.6: First 6 mode shapes of the ABAQUS FE simulated CAARC building and their flexure plane indication by the xyz coordinate system.

$$\delta_{a,n} = \delta_{a,0} \cdot \frac{n_0}{n_n} \tag{4.12}$$

with the value of 0,004102. Once determining this second natural frequency aerodynamic damping, the second damping ratio for the Rayleigh study can be obtained using the unchanged structural damping δ_s with the following expression: ε_2=$(\delta_s + \delta_{a,n})/2\pi$=0,010679.

Therefore, in order to achieve the factors resulting from the simultaneous solution α_R and β_R it is needed to be solved Ex. 4.13, where all variables values has already been identified.

$$\begin{Bmatrix} \alpha_R \\ \beta_R \end{Bmatrix} = 2\,\frac{\omega_0 \cdot \omega_n}{\omega_n^2 - \omega_0^2} \begin{bmatrix} \omega_n & -\omega_0 \\ -1/\omega_n & 1/\omega_0 \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \end{Bmatrix} \tag{4.13}$$

with ω_0=$2\pi n_0$ and ω_n=$2\pi n_n$. Solving Ex. 4.13 the simultaneous solution α_R and β_R is obtained with tha values shown in Ex. 4.14.

$$\begin{Bmatrix} \alpha_R \\ \beta_R \end{Bmatrix} = \begin{Bmatrix} 0,030154 \\ 0,002449 \end{Bmatrix} \tag{4.14}$$

This values are then introduced in the defining property section of the ABAQUS FE simulated 60-story CAARC Building for each of the nodes.

4.1.2.5 Wind force in the FEM study

FEM model is going to be solved several times with several time histories of the fluctuating wind drag force in each of the nodes for the dynamic response analysis. This forces are going to be constant along the node as it is going to be a vertical and horizontal correlated study that is going to be carried out by introducing the aerodynamic admittance function proposed by EN 1991-1-4 to decrease loads according to the horizontal correlation.

Hence, the procedure is to define some *amplitudes* in ABAQUS FE program for every time differential and for every node. The simulated wind values matrix dependent on time and node are obtained by the procedure followed in section 4.1.1 and uniformly distributed along each node. This way, each of the defined *amplitudes* is assigned to a time dependent fluctuating *load* for each of the nodes related to a wind force incidence *step* that varies in every of the intervals until the total time simulation of 600 s, as the EN 1991-1-4 established with the averaging time T. [16]

4.1.3 EN 1991-1-4 Analysis

EN1991-1-4 guides in the wind actions determination pretended for civil engineering works and structural design of building for the loaded areas that are under consideration. The study is produced either in the whole structure and also in parts/elements tied to the structure such as cladding units, components and fixings or noise/safety barriers.

The main value is needed to be extracted from EN 1991-1-4 for the study project is the acceleration suffered in the building due to the wind load incidence. For achieving this acceleration, EN 1991-1-4 has to possible approaches that are shown in Annex B and C.

Another aspect that has to be studied are the across-wind acceleration study considerations in terms of vortex shedding and galloping.

To conclude the EN 1991-1-4 analysis, values of the aerodynamic admittance functions, R_b and R_h, and their variation depending on the assumption that we are making, are going to be studied. Thus, there are going to be two different studies depending on the values of R_b and R_h. Finally, a study of some considerations in respect to the application of the European Wind Code over a CAARC Standard Building are going to be exhibited.

4.1.3.1 Along-Wind Analysis - EN 1991-1-4 Annex B and C approaches

Annex B and C give alternatives in the procedure for calculating the structural factor C_sC_d that affects the acceleration value approach. In EN 1991-1-4, Section 6 Note 3, is claimed that "The procedure to be used to determine k_p, B and R may be given in the National Annex". Where k_p is the peak factor, B is the background factor and R is the resonance response factor, which are factors affecting the determination of the peak acceleration in both B and C Annexes. "A recommended procedure is given in Annex B. An alternative procedure is given in Annex C. As an indication to the users the differences in C_sC_d using Annex C compared to Annex B does not exceed approximately 5%". [16]

Also in section 6 "The National Annex may give a method for determining the along-wind displacement and the standard deviation of the along-wind acceleration. The recommended method is given in Annex B. An alternative method is given in Annex C". [16] Thus, due to the express recommendation of EN 1991-1-4 the Annex method selected for the study will be Annex B methodology, with the confidence that the error wont be significant and can be neglected.

4.1.3.1.1 Annex B Along-Wind Acceleration Determination

In the case of Annex B wind turbulence is studied with the turbulence scale: $L(z)$ that represents the average gust size for natural winds for heights z below 200m. It is determined using the following expression.

$$L(z) = \begin{cases} L_t\left(\frac{z}{z_t}\right)^{\alpha} & z \geq z_{min} \\ L(z_{min}) & z < z_{min} \end{cases} \tag{4.15}$$

Where z_t is the reference height equal to 200m, a reference length scale of L_t equals 300m, and α = 0,67 + 0,05 ln(z_o), where the roughness length z_o is in m. The minimum height z_{min} along with z_o are given in EN 1991-1-4 Table 4.1. [16]

According to EN 1991-1-4 Annex B, wind distribution over frequencies is expressed by the non-dimensional power spectral density function $S_L(z, n)$, which should be determined using the following expression.

$$S_L(z, n) = \frac{n \cdot S_v(z, n)}{\sigma_v^2} = \frac{6,8 \cdot f_L(z, n)}{(1 + 10,2 \cdot f_L(z, n))^{5/3}} \tag{4.16}$$

where:

$S_v(z, n)$ is the one-sided variance spectrum with σ_v being the standard deviation.

$f_L(z, n)$ is a non-dimensional frequency determined by the frequency n=$n_{1,x}$, the natural frequency of the structure in Hz, by the mean velocity $v_m(z)$ and the turbulence length scale $L(z)$ defined in (3.1).

$$f_L(z, n) = \frac{n \cdot L(z)}{V_m(Z)} \tag{4.17}$$

The structural factor C_sC_d, on the other hand, is studied in Annex B by the following expression.

$$C_sC_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_s)} \tag{4.18}$$

where:

k_p : is the peak factor, defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation, and follows the following expression.

$$k_p = \sqrt{2 \cdot ln(\nu \cdot T)} + \frac{0,6}{\sqrt{2 \cdot ln(\nu \cdot T)}} \qquad \text{or } k_p = 3 \text{ whichever is larger} \tag{4.19}$$

with ν being the up-crossing frequency and T being the averaging time for the mean wind velocity, T=600 seconds. It is possible to find $\nu \cdot T$ in Figure B.2 of EN 1991-1-4. [16] Also ν follows the following expression.

$$\nu = n_{1,x} \cdot \sqrt{\frac{R^2}{B^2 + R^2}} \qquad ; \nu \geq 0,08 \text{ Hz} \tag{4.20}$$

B^2 : is the background factor, allowing for the lack of full correlation of the pressure on the structure surface may be calculated using expression.

$$B^2 = \frac{1}{1 + 0,9 \cdot \left(\frac{b+h}{L(z_s)}\right)^{0,63}} \tag{4.21}$$

where b is the width and h the height of the structure. $L(z_s)$ is the turbulent length scale given before at reference height $z_s = 0,6 \cdot h$. According to the EC it is on the safe side to use $B^2 = 1$.

R^2 : is the resonance response factor allowing for turbulence in resonance with the considered vibration mode of the structure should be determined using expression.

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot R_h(\eta_h) \cdot R_b(\eta_b) \tag{4.22}$$

where

$S_L(z_s, n_{1,x})$ is the already defined non-dimensional power spectral density function.

δ is the total logarithmic decrement of damping that follows δ=$\delta_s + \delta_a + \delta_d$.

$R_h(\eta_h)$ and $R_b(\eta_b)$ are the aerodynamic admittance functions for a fundamental mode shape that will be discussed in the following section. They are determined by the following expressions.

$$R_h = \frac{1}{\eta_h} - \frac{1}{2 \cdot \eta_h^2}(1 - e^{-2 \cdot \eta_h}) \tag{4.23}$$

with

$$\eta_h = \frac{4,6 \cdot h}{L(z_s)} \cdot f_L(z_s, n_{1,x}) \tag{4.24}$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2}(1 - e^{-2 \cdot \eta_b}) \tag{4.25}$$

with

$$\eta_b = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x}) \tag{4.26}$$

$I_v(z_s)$: is the turbulence intensity at the height $z = z_s$. Defined by the following expression (assuming $z_s > z_{min}$):

$$I_v(z) = \frac{\sigma_v}{V_m(z_s)} = \frac{K_l}{C_0(z_s) \cdot ln(\frac{z_s}{z_0})} \tag{4.27}$$

where σ_v is the standard deviation of the one-sided variance spectrum. The mean velocity at height z_s is $v_m(z_s)$. z_o is the roughness length. K_l is the turbulence factor. The value of K_l may be given in the National Annex and its recommended value is 1,0. C_0 is the orography factor that according to National Annex may be assumed 1,0 as "the effects of orography may be neglected when the average slope of the upwind terrain is less than 3°". [16]

EN 1991-1-4 Annex B study of service displacement and accelerations for serviceability assessments of a vertical structure is determined as follows. [16]

Firstly, the maximum along-wind displacement is determined from the equivalent static wind force defined by the following expression.

$$F_w = C_s \cdot C_d \cdot C_f \cdot q_p(z_e) \cdot A_{ref} \tag{4.28}$$

On the other hand, the standard deviation $\sigma_{a,x}$ of the characteristic along-wind acceleration of the structural point at height z, according to Annex B EN 1991-1-4, should be obtained using the following expression.

$$\sigma_{ax}(z) = \frac{C_f \cdot \rho \cdot b \cdot I_v(z_s) \cdot V_m^2(z_s)}{m_{1,x}} \cdot R \cdot K_x \cdot \phi_{1,x}(z) \tag{4.29}$$

where:

C_f : is the force coefficient

ρ : is the air density

b : is the width of the structure

$I_v(z_s)$: is the turbulence intensity at the height $z = z_s$ (reference height) above ground

$V_m(z_s)$: is the mean wind velocity for $z = z_s$ (reference height)

R : is the square root of resonant response

K_x : is the non-dimensional coefficient

$\phi_{1,x}(z)$: is the fundamental along wind modal shape

$m_{1,x}$: is the along wind fundamental equivalent mass

EN 1991-1-4 defines a non dimensional coefficient, K_x, for determining the along-wind acceleration, defined by the following expression.

$$K_x = \frac{\int_0^h V_m^2(z) \cdot \phi_{1,x}(z)\, dz}{V_m^2(z_s) \cdot \int_0^h \phi_{1,x}^2(z)\, dz} \tag{4.30}$$

where:

$V_m(z_s)$: is the mean wind velocity

$\phi_{1,x}(z)$: is the fundamental along wind modal shape

Assuming $\phi_{1,x}(z)$=$(\frac{z}{h})^{\zeta}$ and $C_o(z)$=1, K_x can be defined as the following expression.

$$K_x = \frac{(2 \cdot \zeta + 1) \cdot \{(\zeta + 1) \cdot [ln(\frac{z_s}{z_0}) + 0,5] - 1\}}{(\zeta + 1)^2 \cdot ln(\frac{z_s}{z_0})} \tag{4.31}$$

where:

ζ : is the exponent of the mode shape

z_0 : is the roughness length

EN 1991-1-4 Annex B claim, in conclusion, that the characteristic peak accelerations are obtained by multiplying the standard deviation $\sigma_{a,x}$ by the peak factor k_p using the natural frequency as the upcrossing frequency $n_{1,x}$. [16]

4.1.3.1.2 Annex C Along-Wind Acceleration Determination

Wind turbulence is considered the same way as the Annex B procedure. Thus, the turbulence is given by identical considerations, so $L(z)$, $S_L(z,n)$ and $f_L(z,n)$ has same expressions.

Table 4.2: K and G as a mode shape function. [16]

Mode shape	Uniform	Linear	Parabolic	Sinusoidal
G:	1/2	3/8	5/18	$4/\pi^2$
K:	1	3/2	5/3	$4/\pi$

NOTE 1 For buildings with a uniform horizontal mode shape variation and a linear vertical mode shape variation $\Phi(y,z) = z/h$, $G_y = 1/2$, $G_z = 3/8$, $K_y = 1$ and $K_z = 3/2$.

NOTE 2 For chimneys with a uniform horizontal mode shape variation and a parabolic vertical mode shape variation $\Phi(y,z) = z^2/h^2$, $G_y = 1/2$, $G_z = 5/18$, $K_y = 1$ and $K_z = 5/3$.

NOTE 3 For bridges with a sinusoidal horizontal mode shape variation $\Phi(y,z) = \sin(\pi \cdot y/b)$, $G_y = 4/\pi^2$, $G_z = 1/2$, $K_y = 4/\pi$ and $K_z = 1$.

Furthermore, the structural factor C_sC_d and peak factor k_p are defined maintaining the same expression studied in Annex B. However, the expressions for the background factor B^2 and resonance factor R^2 are different. The background factor B^2 is defined by the following expression.

$$B^2 = \frac{1}{1 + \frac{3}{2} \cdot \sqrt{\left(\frac{b}{L(z_s)}\right)^2 + \left(\frac{h}{L(z_s)}\right)^2 + \left(\frac{b}{L(z_s)} \cdot \frac{h}{L(z_s)}\right)^2}} \tag{4.32}$$

where b is the width and h the height of the structure. $L(z_s)$ is the turbulent length scale given before at reference height $z_s = 0,6 \cdot h$. According to EN 1991-1-4 it is on the safe side to use $B^2 = 1$ [16]. The resonance factor R^2 is defined by the following expression.

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot K_s(n_{1,x}) \tag{4.33}$$

where $S_L(z_s, n_{1,x})$ is the already defined non-dimensional power spectral density function. δ is the total logarithmic decrement of damping that follows $\delta{=}\delta_s{+}\delta_a{+}\delta_d$. $K_s(n)$ is the size reduction function defined in the following expression.

$$K_s(n) = \frac{1}{1 + \sqrt{(G_y \cdot \phi_y)^2 + (G_z \cdot \phi_z)^2 + \left(\frac{2}{\pi} \cdot G_y \cdot \phi_y \cdot G_z \cdot \phi_z\right)^2}} \tag{4.34}$$

with $\phi_y = \frac{11,5 \cdot b \cdot n}{V_m(z_s)}$ $\phi_z = \frac{11,5 \cdot h \cdot n}{V_m(z_s)}$

The constants G_y and G_z depend on the mode shape variation along the horizontal y-axis and vertical z-axis, respectively. Constants G and K used for calculating acceleration in Annex C can be seen in Table 4.2

EN 1991-1-4 Annex C study of service displacement and accelerations for serviceability assessments of a vertical structure is determined with the same equivalent static wind force as in Annex B. [16] on the other hand, the expression for the standard deviation $\sigma_{a,x}$ of the characteristic along-wind acceleration of the structural point at height z, according to Annex C EN 1991-1-4, should be obtained using the following expression.

$$\sigma_{ax}(y,z) = C_f \cdot \rho \cdot I_v(z_s) \cdot V_m^2(z_s) \cdot R \cdot \frac{K_y \cdot K_z \cdot \phi_{1,x}(y,z)}{\mu_{ref} \cdot \phi_{max}} \tag{4.35}$$

where:

C_f : is the force coefficient

ρ : is the air density

$I_v(z_s)$: is the turbulence intensity at the height z=z_s above ground

$V_m(z_s)$: is the mean wind velocity for z=z_s (reference height)

R : is the square root of resonant response

$\phi(y,z)$: is the mode shape

ϕ_{max} : is the mode shape value at the point with maximum amplitude

μ_{ref} : is the reference mass per unit area defined by the following expression.

$$\mu_e = \frac{\int_0^h \int_0^b \mu(y,z) \cdot \phi_1^2(y,z)\, dydz}{\int_0^h \int_0^b \phi_1^2(y,z)\, dydz} \tag{4.36}$$

K_y, K_z : are constants given in Table 4.2

Finally, as in the EN 1991-1-4 Annex B, the characteristic peak accelerations in Annex C are obtained by multiplying the standard deviation $\sigma_{ax}(y,z)$ by the peak factor k_p using the natural frequency $n_{1,x}$. [16]

4.1.3.2 Cross-Wind Study - Vortex Shedding and Galloping

There is a need to make across-wind considerations according to EN 1991-1-4 Annex E. Hence, along side this Euro-code Annex E, there is a study of the Vortex Shedding and Galloping phenomenons that could affect in cross-section wind load incidence.

4.1.3.2.1 Vortex Shedding

Vortex shedding happens when vortexes are alternatively shedding in opposed structure sides. This rises to a shifting load perpendicularly acting to the direction of wind incidence. The structures vibration arises if the frequency of the shedding vortex coincide with the natural frequency of the building. This situation appears when wind speed equals the critical wind speed defined in EN1991-1-4 Section E.1.3.1 of Annex E. [16]

Critical wind speed is typically a recurrent wind speed that indicates that the number of load cycles and fatigue are to become significant.

The response induced by sheds vortexes is broad banded composed response that appears independently from the structure movement (that are normally of a greater importance for heavy steel structures and reinforced concrete structures), and response narrowed banded generating from motion wind force induction (light steel structures).

The vortex shedding effect should be analyzed, according to EN1991-1-4, when the ratio between the largest and smallest across-wind structure dimensions, taken in the perpendicular plane to the wind, is greater than six and the critical wind velocity V_{crit} is either equal or smaller than the characteristic ten minutes mean wind speed V_m given in EN 1991-1-4 Section 4.3.1 at the cross-section. [16]

V_{crit} is given in EN 1991-1-4 for the studied building as the following expression.

$$V_{crit} = \frac{b \cdot n_{i,y}}{S_t} \tag{4.37}$$

where

b : is the reference width of the cross-section at which resonant vortex shedding occurs

$n_{i,y}$: is the natural frequency of the considered flexural mode i of cross-wind vibration

S_t : is the Strouhal number given in EN 1991-1-4 E.1.3.2.

The predisposition of vibrations lies on the structural damping and the structural to fluid mass ratio given by the Scruton number S_c, in the following according to EN 1991-1-4 E.1.3.3 Expression .

$$S_c = \frac{2 \cdot \delta_s \cdot m_{i,e}}{\rho \cdot b^2} \tag{4.38}$$

where

δ_s : is the structural damping expressed by the logarithmic decrement.

ρ : is the air density under vortex shedding conditions

$m_{i,e}$: is the equivalent mass me per unit length for mode i

b : is the reference width of the cross-section at which resonant vortex shedding occurs

Considering all this aspects, there is no need to make a cross-wind study for our study case attending to vortex shedding.

4.1.3.2.2 Galloping

Galloping is a vibration self inducted of a flexible building in across-wind mode bend. The sections not circular that have more chances to galloping are cross-sections including 1-, L-, T- and U- sections.

Galloping oscillation begins with a specific characteristic wind speed V_{CG} and normally there is a amplitude rapid increase when there is a wind velocity fast develop. [16]

According to EN1991-1-4 Annex E section E.2, it should be ensured the following expression.

$$V_{CG} > 1,25 \cdot V_m \tag{4.39}$$

with

$$V_{CG} = \frac{2 \cdot Sc}{a_G} \cdot n_{1,y} \cdot b = 193,54 \ \ m/s \tag{4.40}$$

with a_G as the factor of galloping instability equal to 1,0 due to EN 1991-1-4 Section E.2.1, b as the structure width equal to 45 m, $n_{1,y}$ assumed as 0,2 Hz for a CAARC building (Table 4.4) and Sc as the Scruton number equal to 10,752 (Ex. 4.38) for this CAARC building.

and

V_m as the mean wind velocity at the height where there is expect of a galloping process: point of maximum oscillation amplitude. It has a maximum value of 17,86 m/s for z=h as can be seen using Expression 6.

Introducing all values in Expression 4.39 we can conclude that there is no Galloping issue affecting the study case according to EN1991-1-4.

4.1.3.3 Aerodynamic Admittance Function of Tall Buildings Analysis. EN 1991-1-4

The aerodynamic admittance function (AAF) has been used in order to make a relation between wind pressure on building surfaces and the oncoming wind velocity [62]. It is a common practise to assume quasi-steady situations in order to formulate wind effects in the along-wind direction following the "gust loading factor" (GLF) approach [10, 63]. A generalized wind load that employs the aerodynamic admittance function follows the synthesis of the wind velocity field with theories that permit the representation and study of wind pressure on buildings/structures under wind-incidence by using the generalized wind load (GWL) known as GWL-AAF in order to apply the gust loading factor (GLF) approach.

Zhou and Kareem (2002) showed a different definition of AAF dependant on the base bending moment (BBM), the so-called base bending moment aerodynamic admittance function (BBM-AAF). In contrast with the GWL-AAF, the BBM-AAF is independent of the mode shape. However, due to implied considerations of linear mode shape, it is equivalent in a numeric approach to the actual GWL-AAF for the main code functions and can be derived using wind tunnel experiments employing a great-frequency-based balancing [64]. The results showed that the "BBM-AAF results obtained from wind tunnel test [...] noteworthy discrepancies in the high frequency region" in comparison to GWL-AAF [65]. Zhou and Kareem (2002) partly attributed this scatter to "the choice of aerodynamic admittance function which did exhibit departure from those based on the strip and quasi-steady theories". [66]

Nevertheless, it is precise to follow GWL-AAF for code application. Attending the main Wind Standards considerations, GWL-AAF is exposed in relation of a function with many similarities involving the structures size in the vertical and horizontal directions. A comparison between GWL-AAF in mainly resound international standards can be seen in Table 4.3 [66, 43].

Table 4.3: Comparison between GWL-AAF in major International Standards. [66]

	Expressions of GWL-AAF
ASCE 7	$\chi(f) = R_H R_B (0.53 + 0.47 R_D)$
	$R_l = 1/\eta - 1/2\eta^2 (1 - e^{-2\eta})$, $\eta > 0$; $R_l = 1$, $\eta = 0$
	$R_H, \eta = 4.6 fH / V_{\bar{Z}}$; $R_B, \eta = 4.6 fB / V_{\bar{Z}}$; $R_D, \eta = 15.4 fD / V_{\bar{Z}}$
AS1170.2	$\chi(f) = \frac{1}{(1 + 3.5 fH / \bar{V}_H) \cdot (1 + 4 fB / \bar{V}_H)}$
NRCC	$\chi(f) = \frac{1}{(1 + 8 fH / (3\bar{V}_H)) \cdot (1 + 10 fB / \bar{V}_H)}$
RLB-AIJ	$\chi(f) = \frac{0.84}{(1 + 2.1 fH / \bar{V}_H) \cdot (1 + 2.1 fB / \bar{V}_H)}$
Eurocode	$\chi(f) = R_H R_B$
	R_H and R_B are the same as those in ASCE7.

Hence, following Eurocode EN1991-1-4 [16], it can be seen that the resonant response factor R^2 depends on the aerodynamic admittance functions, $R_h(\eta_h)$ and $R_b(\eta_b)$, just like the following expression.

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot R_h(\eta_h) \cdot R_b(\eta_b) \tag{4.41}$$

where the vertical or vertical and horizontal correlation consideration relies on the specification of $R_b(\eta_b)$ that is given by the following expression.

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2}(1 - e^{-2 \cdot \eta_b}) \tag{4.42}$$

with

$$\eta_b = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x}) \tag{4.43}$$

Thus, in order to make a vertical correlation approach in a simplified uniform study just like the one specified in section 2.1 (Fig. 2.7), EN1991-1-4 procedure is to assume η_b=0 and, hence, $R_b(\eta_b)$=1. On the other hand, in order to be able to realize the real case study, following the section 2.1 (Fig. 2.8) considerations, it is specified by EN 1991-1-4 to follow Ex. 4.42 with η_b>0 and, hence, $R_b(\eta_b)$<1 for the vertical and horizontal correlation approach study. [16]

4.1.3.4 Application of EN 1991-1-4 on CAARC building

In order to make the comparison between the Finite Element Model Analysis and the EN 1991-1-4 considerations of the application of wind loads on a building structure, the general building of the study is the CAARC Tall Standard Building model.

The standardization specifications for wind tunnel experiments on structure/building aerodynamics were firstly introduced by Wardlaw and Moss. It was specified the characteristic of a simple building: CAARC building. [32]

There has been several measuring tests over the established CAARC building model during 1970-1975 continuing Wardlaw and Moss studies. [26]

Hence, CAARC properties and dimensions are already established, and will be used in order to accomplished this project. Thus, the EN 1991-1-4 variables will depend on the CAARC dimensions and properties.

This way, we can see in Table 4.4 and Figure 4.7 the variables that are already defined and will be introduced in EN 1991-1-4 approach study.

Table 4.4: CAARC Standard Building and Wind physical values.

CAARC Standard Tall Building						Wind phisical properties	
Building geometry			*Dynamic properties*				
Height: h [m]	Width: b [m]	Along-wind lenght: d [m]	Natural frequency: n [Hz]	Specific mass: $m_{1,x}/A_{cs}$ [Kg/m3]	Damping ratio to critical: x_i	Specific mass : ρ [Kg/m3]	Dynamic viscosity : μ [Ns/m2]
180	45	30	0.2	160	0.01	1.25	$7{,}03 \times 10^{2}$

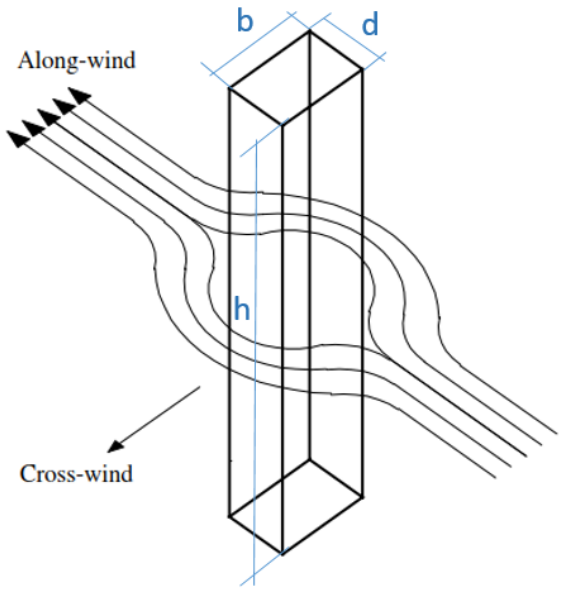


Figure 4.7: Characteristic variables for a building according to EN 1991-1-4.

Furthermore, there is a special mention needed for the determination of the natural frequency to use. The EN 1991-1-4 has established an expression for its determination shown below.

$$n_{1,x} = \frac{46}{h} \quad [Hz] \tag{4.44}$$

Despite this fact, the CAARC building has a known and established natural frequency equal to 0,2 Hz (Table 4.4). As it is known, and is below the 0,2555 Hz considered by the EN 1991-1-4, it is more conservative and recommendable to use the known 0,2 Hz CAARC's building natural frequency.

4.2 Acceleration of a CAARC Building Wind Induced - EN 1991-1-4 and FEM Approaches Development

The development of the EN 1991-1-4 and FEM approaches studies of acceleration determination in a CAARC Standard Tall Building with Wind Loads incidence is going to be executed in this section. The study is carried out in two differentiated comparative objectives: the specifications of the vertical correlated case undertaken by EN 1991-1-4 in order to observe the divergences with the closest to reality case and an actual situation of horizontal and vertical correlation studied comparatively by the European Standard EN 1991-1-4 and the FEM-approach.

Therefore, acceleration is firstly going to be developed and obtained by the EN 1991-1-4 for the simplified situation and, then, the real situation is going to be performed by the European Standard and FEM study. The comparison remarks of both approaches for the real case and both building wind incised considered cases done by the EN 1991-1-4 Standard will be stated to conclude the chapter.

4.2.1 Acceleration of a CAARC Building under Wind Loads Incidence - Vertical Correlation

Along this section the incidence of wind over a CAARC Standard Tall Building will be studied in a simplified case of just vertical correlation. Therefore, the study and determination of acceleration will be done following the established EN 1991-1-4 guideline considerations.

As it was already discuss in 4.1.3.1 the European Wind Code analysis is going to be made attending to EN 1991-1-4 Annex B, due the explicit recommendation of the code.

EN 1991-1-4 Annex B claim that "the characteristic along-wind peak accelerations are obtained by multiplying the standard deviation $\sigma_{a,x}$ by the peak factor k_p using the natural frequency as the upcrossing frequency $n_{1,x}$" [16]. The complete development of all formulas related to the code can be found in Annex A of this thesis.

$$a_{EN,vc}(z) = \sigma_{a,x,vc}(z) \cdot k_{p,vc} = 0,0996 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{4.45}$$

where

$a_{EN,vc}$: is the along-wind acceleration studied by EN 1991-1-4 of a CAARC building with vertical correlation (constant horizontal load).

$\sigma_{a,x,vc}$: is the standard deviation of the characteristic along-wind acceleration of the structural point at height z for the vertical correlation study.

$k_{p,vc}$: is the peak factor for the vertical correlation study.

This way the values of this variables must be determined in order to make the study. In Annex A, it can be seen the full development of all the expressions.

(1) The standard deviation for the vertical correlation approach $\sigma_{ax,d}(z)$, according EN1991-1-4 Annex B is defined by the following expression. [16]

$$\sigma_{ax,vc}(z) = \frac{C_f \cdot \rho \cdot b \cdot I_v(z_s) \cdot V_m^2(z_s)}{m_{1,x}} \cdot R_{vc} \cdot K_x \cdot \phi_{1,x}(z) = 0,0322 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{4.46}$$

where

C_f : is the force coefficient of structural elements of rectangular section with the wind blowing normally to a face. It is given in EN 1991-1-4 Section 7.6. As it can be seen in Annex A (Ex. 3), C_f=$2,19$

ρ : is the air density. 1,25 kg/m^3 (Table 4.4)

b : is the width of the structure. 45 m (Table 4.4)

$I_v(z_s)$: is the turbulence intensity at the height z=z_s above ground. It is given in EN 1991-1-4 Section 4.4. As it can be seen in Annex A (Ex. 5), $I_v(z_s)$=$0,214$

$V_m(z_s)$: is the mean wind velocity for the reference height z=z_s. It is given in EN 1991-1-4 Section 4.3.1. As it can be seen in Annex A (Ex. 6), $V_m(z_s)$=$16,128$

R_{vc} : is the square root of resonant response for the vertical correlation approach study. In is given in EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 11), R_{vc}^2=$0,388$

K_x : is the non-dimensional coefficient. It in EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 24), K_x=$1,634$

$\phi_{1,x}(z)$: is the fundamental along wind modal shape. It is given in EN 1991-1-4 Annex F. As it can be seen in Annex A (Ex. 26), $\phi_{1,x}(z)$=$\left(\frac{z}{180}\right)^{1,5}$

$m_{1,x}$: is the along-wind fundamental equivalent mass per length. As it can be seen in Annex A (Ex. 19), $m_{1,x}$=216000 kg/m (Table 4.4) [32]

(2) The peak factor $k_{p,vc}$ defined as "the ratio of the maximum value of the fluctuating part of the response to its standard deviation" for the case of vertical correlation approach study. According to EN 1991-1-4 Annex B is defined by the following expression. [16] The complete development of all formulas related to the peak factor determination can be found in Annex A.

$$k_{p,vc} = max\Big(\sqrt{2 \cdot ln(\nu_{vc} \cdot T)} + \frac{0,6}{\sqrt{2 \cdot ln(\nu_{vc} \cdot T)}};\ 3\Big) = 3,089 \tag{4.47}$$

where

T : is the averaging time for the mean wind velocity, T=600 seconds. (EN 1991-1-4 Annex B)

$\nu_v c$: is the up-crossing frequency for the case of vertical correlation approach study. It is given in EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 28), ν_{vc}=$0,106$

4.2.2 Acceleration of a CAARC Building under Wind Loads Incidence - Horizontal and Vertical Correlation

Along this section the incidence of wind over a CAARC Standard Tall Building will be studied in a closest to reality case of horizontal and vertical correlation. Therefore, the study and determination of acceleration will be done following the established EN 1991-1-4 guideline considerations and, then, with a FEM approach simulating wind incidence over a CAARC Building in ABAQUS FE-program.

4.2.2.1 EN 1991-1-4 Acceleration Determination - Horizontal and Vertical Correlation

This section has an analog resolution as in the Section 4.2.3.1 case, that can be fully seen in Annex A, taking into account the non-uniformity that appears in the wind load horizontal coordinate. As it was already explain in Section 2.1 there is a need to make this vertical and horizontal correlation approach study in order to understand and completely simulate the real behaviour of the building under the wind load incidence.

Hence, in order to be able to make this new study according to EN 1991-1-4, we are also going to define the same expression to get the acceleration. This expression is going to depend on the same factors that apart from R_b are going to be the same, as it was discussed in Section 4.1.3.2. The repercussion on the R_b change (from being equal to 1 to being smaller than 1), will affect the resonant factor R^2 that affects either $\sigma_{a,x}$ and the k_p of this real approach study.

The rest of the variables will remain with the same value as in the previous simplified of just vertical correlation study.

EN 1991-1-4 Annex B claim that "the characteristic along-wind peak accelerations are obtained by multiplying the standard deviation $\sigma_{a,x}$ by the peak factor k_p using the natural frequency as the upcrossing frequency $n_{1,x}$." [16]

$$a_{aw,EN}(z) = \sigma_{a,x}(z) \cdot k_p = 0,0542 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{4.48}$$

where

a_{EN} : is the along-wind acceleration studied by EN 1991-1-4 of a CAARC building in a vertical and horizontal correlation approach study (not constant horizontal load).

$\sigma_{a,x}$: is the standard deviation of the characteristic along-wind acceleration of the structural point at height z for the vertical and horizontal correlation approach study.

k_p : is the peak factor for the vertical and horizontal correlation approach study.

(1) The standard deviation $\sigma_{ax}(z)$, according EN 1991-1-4 Annex B is defined by the following expression. [16]

$$\sigma_{ax}(z) = \frac{C_f \cdot \rho \cdot b \cdot I_v(z_s) \cdot V_m^2(z_s)}{m_{1,x}} \cdot R \cdot K_x \cdot \phi_{1,x}(z) = 0,0181 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{4.49}$$

where

C_f : is the force coefficient of structural elements of rectangular section with the wind blowing normally to a face. It is given in EN 1991-1-4 Section 7.6. As it can be seen in Annex A (Ex. 3), C_f=2, 19

ρ : is the air density. 1,25 kg/m^3 (Table 4.4)

b : is the width of the structure. 45 m (Table 4.4)

$I_v(z_s)$: is the turbulence intensity at the height z=z_s above ground. It is given in EN 1991-1-4 Section 4.4. As it can be seen in Annex A (Ex. 5), $I_v(z_s)$=0, 214

$V_m(z_s)$: is the mean wind velocity for the reference height z=z_s. It is given in EN 1991-1-4 Section 4.3.1. As it can be seen in Annex A (Ex. 6), $V_m(z_s)$=16, 128

R : is the square root of resonant response for the vertical and horizontal correlation approach study. In EN 1991-1-4 Annex B is defined by the following expression.

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot R_h(\eta_h) \cdot R_b(\eta_b) = 0,122 \tag{4.50}$$

where

S_L : is the non-dimensional power spectral density function defined in EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 13), S_L=$0,073$

δ : is the total logarithmic decrement of damping. Is given in EN 1991-1-4 Annex F. As it can be seen in Annex A (Ex. 16), δ=$0,086$

$R_h(\eta_h)$: is the aerodynamic admittance functions for a fundamental mode shape in buildings height direction h. As it can be seen in Annex A (Ex. 20), $R_h(\eta_h)$=$0,093$

$R_b(\eta_b)$: is the aerodynamic admittance functions for a fundamental mode shape in buildings width direction b for the vertical and horizontal correlation approach study. Determined by the following expression. (EN 1991-1-4 Annex B). In contrast with the first approach, R_b differs from 1.

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2}(1 - e^{-2 \cdot \eta_b}) = 0,314 \tag{4.51}$$

with

$$\eta_b = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x}) = 2,566 \tag{4.52}$$

where

b : is the structure width. 45 m (Table 4.4)

$L(z_s)$: is the turbulence length scale that represents the average gust size for natural winds.It is given in EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 15) , $L(z_s)$=$198,53$ m

$f_L(z_s, n_{1,x})$: is a non-dimensional frequency with the frequency $n_{1,x}$ at a height z_s. It is given EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 14), $f_L(z_s, n_{1,x})$=$2,463$

K_x : is the non-dimensional coefficient. It in EN 1991-1-4 Annex B. As it can be seen in Annex A (Ex. 24), K_x=$1,634$

$\phi_{1,x}(z)$: is the fundamental along wind modal shape. It is given in EN 1991-1-4 Annex F. As it can be seen in Annex A (Ex. 26), $\phi_{1,x}(z)$=$\left(\frac{z}{180}\right)^{1,5}$

$m_{1,x}$: is the along-wind fundamental equivalent mass per length. As it can be seen in Annex A (Ex. 19), $m_{1,x}$=216000 kg/m (Table 4.4) [32]

(2) The peak factor k_p for the vertical and horizontal correlation approach is "defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation". According EN1991-1-4 Annex B is defined by the following expression. [16]

$$k_p = max\Big(\sqrt{2 \cdot ln(\nu \cdot T)} + \frac{0,6}{\sqrt{2 \cdot ln(\nu \cdot T)}};\ 3\Big) = 3 \tag{4.53}$$

where

T : is the averaging time for the mean wind velocity, T=600 seconds. (EN1991-1-4 Annex B)

ν : is the up-crossing frequency for the case of vertical and horizontal correlation approach study. ν is given in EN1991-1-4 Annex B by the following expression.

$$\nu = n_{1,x} \cdot \sqrt{\frac{R^2}{B^2 + R^2}} = 0,066 \qquad ; \nu \geq 0,08 \text{ Hz} \tag{4.54}$$

where

B^2 : is the background factor, allowing for the lack of full correlation of the pressure on the structure surface. B^2=1 (EN 1991-1-4 Annex B explanation of the chosen value is explained in Annex A (Ex. 29))

R^2 : is the resonance response factor for the case of vertical and horizontal correlation approach study allowing for turbulence in resonance with the considered vibration mode of the structure should be determined using expression. R^2=0,122 (Ex. 4.50)

$n_{1,x}$: is the natural frequency of the structure in Hz, that for a CAARC building is 0,2 Hz. (Section 4.1.3.4 and Table 4.4)

4.2.2.2 FEM Approach Acceleration Determination - Horizontal and Vertical Correlation

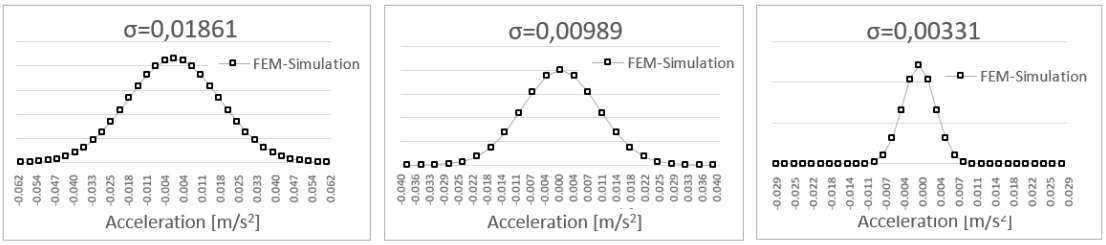
Since all the features of a CAARC Building for simulation in ABAQUS FE program had been defined, it is possible to proceed with the study. As it has been commented, the vibration determination study is carried out considering horizontal and vertical correlation between the wind loads in order to make the comparison with EN 1991-1-4 previously defined value. Hence, in each one of the 60 nodes in which the MDOF building has been discretised the wind load is uniform due to the use of the aerodynamic admittance function proposed by EN 1991-1-4 to decrease loads according to the horizontal correlation. All these aspects are broadly defined, along with material properties, mode shapes and structural damping in section 4.1.2 of this chapter.

Furthermore, the wind loads fluctuating component obtained by Olli Lahti with the procedure that can be found in section 4.1.1, widely developed in Annex C,

have been introduced for the FE-study. It is important to state that the wind loads are only the fluctuating part of the total wind force incising a building, as just the fluctuating part contributes to acceleration. Therefore, each of the time steps have been introduced with different forces for every time step and building node, as a matrix per node and time step. Each time step was 0,1465s being the total simulation time 600 s, in relation to the EN 1991-1-4 specified total time. This makes 4097 simulation steps.

These defined loads and building characteristics were introduced in ABAQUS, being able to make an accurate study of 4097 acceleration result values, for each simulation, in relation to the number of variations of wind loads. For determining the acceleration FE-obtained value, it had been necessary to make an statistical approach of the 4097 values, obtaining the standard deviation of the accelerations obtained by the FE program. The mean of the simulated results is zero, as expected due to the introduction of the fluctuating component of the wind loads which also has a mean of zero.

Nevertheless, as it has been mentioned, wind load simulation defined matrix has a random component. Therefore, in order to accomplish an accurate result of the induced vibration, there has been done 10 simulations of different 4097 step load values to obtain the global standard deviation of all of the obtained results and precisely be able to compare in section 4.3.1 with the EN 1991-1-4 acceleration standard deviation. Therefore, in the aim of making an accurate comparison with the z dependant standard deviation formula obtained in 4.2.2.1 from EN 1991-1-4, global standard deviation has been determined for three different heights (180, 120 and 60 m). Attending to this results, the global standard deviation of the 10 simulations of the complete FE-study is for the case of z=180m equal to 0,0186 m/s^2, for the case of z=120 m equal to 0,0099 m/s^2 and for the case of z=60 m equal to 0,0033 m/s^2, with a mean of zero in the three cases. In Annex D it can be seen all the acceleration values distributed along 600 s time for each of the simulations together with the obtained global standard deviation and the peak value for the simulations sets in the three considered heights. The three acceleration normal distributions depending on the height z can be seen in Fig. 4.8.



a) Distribution for z=180 m b) Distribution for z=120 m c) Distribution for z=60 m

Figure 4.8: Normal Distribution of acceleration depending on the height FEM results.

4.3 EN 1991-1-4 and FEM Results Comparison

In this section it will be expressed a comparative analysis of the results obtained in section 4.2. Thus, the main intention and goal of this project was the determination and analysis of the wind vibration's study conducted according to the European code EN1991-1-4, so this analysis of the results turns out to be essential.

In this way, the two previous comparisons ought to be analyzed. On the one hand, the differences between the results obtained by the European wind code for the simplified case of vertical correlation and the real case of vertical and horizontal correlation will be commented. On the other hand and to conclude, will also be explained the relationship between the study performed by EN1991-1-4 and the FE study for the determination of the standard deviation of the acceleration in the real case of horizontal and vertical correlation.

4.3.1 Comparison between vertical correlated and vertical and horizontal correlated EN 1991-1-4 values.

Throughout the project it has been commented in numerous occasions the two cases that were tried to compare in relation to the correlation of the wind loads along the building. Precisely, in the introductory chapter it was considered as one of the main objectives. Therefore, in section 2.1, the differentiation was described. Attending to Figure 2.7 it could be seen the practically uniform condition by observing C_P along the cube. On the other hand, in Figure 2.8 it was shown the second case where the mean pressure coefficient distribution obtained for real wind boundary layer flow has non-uniform distribution. [2]

In relation to EN 1991-1-4 establish procedure, the aerodynamic admittance function are introduced for the two cases differentiation as developed in section 4.1.3.3. Hence, following the introduced divergences it was specified that the resonant response factor R^2, and thus the acceleration standard deviation $\sigma_{ax}(z)$, depends on the aerodynamic admittance functions, $R_h(\eta_h)$ and $R_b(\eta_b)$, while the vertical or vertical and horizontal correlation differentiation is made by $R_b(\eta_b)$.

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2}(1 - e^{-2 \cdot \eta_b}) \tag{4.55}$$

with

$$\eta_b = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x}) \tag{4.56}$$

Thus, in order to make a vertical correlation approach in a simplified uniform study, EN1991-1-4 procedure was stated to assume $\eta_{b,vc}$=0 and, hence, $R_{b,vc}(\eta_{b,vc})$=1. On

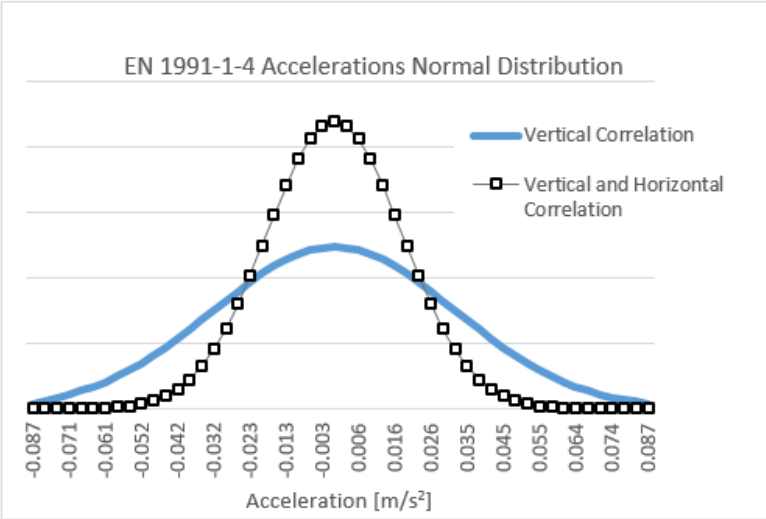


Figure 4.9: Normal Distribution of Acceleration in z=180m for the two EN 1991-1-4 studied cases.

the other hand, in order to be able to realize the real case study, it was specified by EN 1991-1-4 to follow Ex. 4.55 with η_b=2, 566 and, hence, $R_b(\eta_b)$=0, 314 for the real approach study. This aerodynamic admittance functions diversified values lead to different resonance response factor values, and thus, the obtainment of the two acceleration standard deviations:

$$\sigma_{ax,vc}(z)=0,0322 \cdot \left(\frac{z}{180}\right)^{1,5} \quad \text{and} \quad \sigma_{ax}(z)=0,0181 \cdot \left(\frac{z}{180}\right)^{1,5} \; m/s^2,$$

respectively for the vertical correlated case and the vertical and horizontal correlated case [16]. Considering a mean of zero, acceleration normal distribution for both cases in the building top (z=180 m) can be seen comparatively in Figure 4.9.

Thus, it can be concluded that EN 1991-1-4 adequately takes into account the horizontal correlation of wind loads with a big gap between the results. Returning to the Figures 2.7 and 2.8 and attending to the C_P values depending on the wind load assumption, as the specifications made by Stathopoulos and Baniotopoulos (2007), it can be claimed that the results meet expectations. [2]

It is undoubtedly remarkable the fact that to follow the simplified case is in the aim of ensuring to remain on the safety side on a great extent. Simplified case considers wind-incidence produced acceleration standard deviation increased in a 77% with respect to the real load distribution assumption. Furthermore, it is also worth mentioning that the peak factor K_P is higher in the simplified case than in the real one, with an even greater excess of conservativeness by the use of R_b=1.

4.3.2 Results Comparison between EN 1991-1-4 and FEM for the Horizontal and Vertical Correlation Case

The cornerstone of this project is the study of plausibility and reliable representation of reality carried out by EN 1991-1-4. Thus, the comparison to be made in this section is fundamental. The comparative study is made on the modelling of wind incidence in the real situation of horizontal and vertical correlation over a building according to the European Wind Code and the FEM-procedure. In order to do so, both cases used the aerodynamic admittance function with R_b=0,314, according to EN1991-1-4 Equation B.8, to decrease the value of wind loads and thus its consequent dynamics on the building.

On the one hand, along section 4.2.2.1 it was developed the acceleration standard deviation formula for the acceleration distribution in the case of the studied CAARC building under the mentioned boundary conditions. This z-dependant formula is given by the following expression.

$$\sigma_{ax}(z) = 0,0181 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{4.57}$$

Nevertheless, EN 1991-1-4 additionally determines a peak acceleration, that is also dependant with z. It is given by the use of a peak factor k_p that according to this studied case has the value of 3. Thereby, peak acceleration distribution function is given as the following expression for the studied building, as developed in section 4.2.2.1.

$$a_{aw,EN}(z) = 0,0542 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{4.58}$$

On the other hand, the acceleration has been determined according to the FEM procedure with the results given in Annex D. To do so, due to the random component existing in the wind load fluctuating component time step per node matrix, 10 simulations were carried out. With all the obtained values it has been possible to determine the standard deviation of the accelerations out of the 10 simulations set in order to determine the resulting acceleration normal distribution. This operation has been carried out three times for three equally distributed heights: 180, 120 and 60 m. In this way, the comparative study between EN 1991-1-4 and FEM has been carried out in 3 localized points seeking to respond to the degree of coincidence between the data as well as the representative of the vertical distribution function of accelerations that follows $(z/180)^{1,5}$.

In Table 4.5 it can be seen the comparative results between EN 1991-1-4 and FEM obtained acceleration standard deviation for the three under study heights. Therefore, in Figure 4.10 it can be seen the obtained Normal Distribution of acceleration standard deviation in a comparative view along its respective position in the CAARC building.

Table 4.5: Acceleration standard deviation and acceleration peak values obtained by EN 1991-1-4 and FEM-Simulation study for three different heights.

Procedure		std. deviation (m/s^2)			Peak value (m/s^2)					
	z (m)	180	120	60	180		120		60	
					3σ	Peaks mean	3σ	Peaks mean	3σ	Peaks mean
EN 1991-1-4		0.0181	0.0099	0.0035	0.0543		0.0296		0.0105	
FEM		0.0186	0.0099	0.0033	0.0558	0.0570	0.0297	0.0331	0.0099	0.0219

**mean is zero in all the cases*
**Peak mean is the average of the acceleration peaks obtained in absolute value from the 10 simulation sets.*

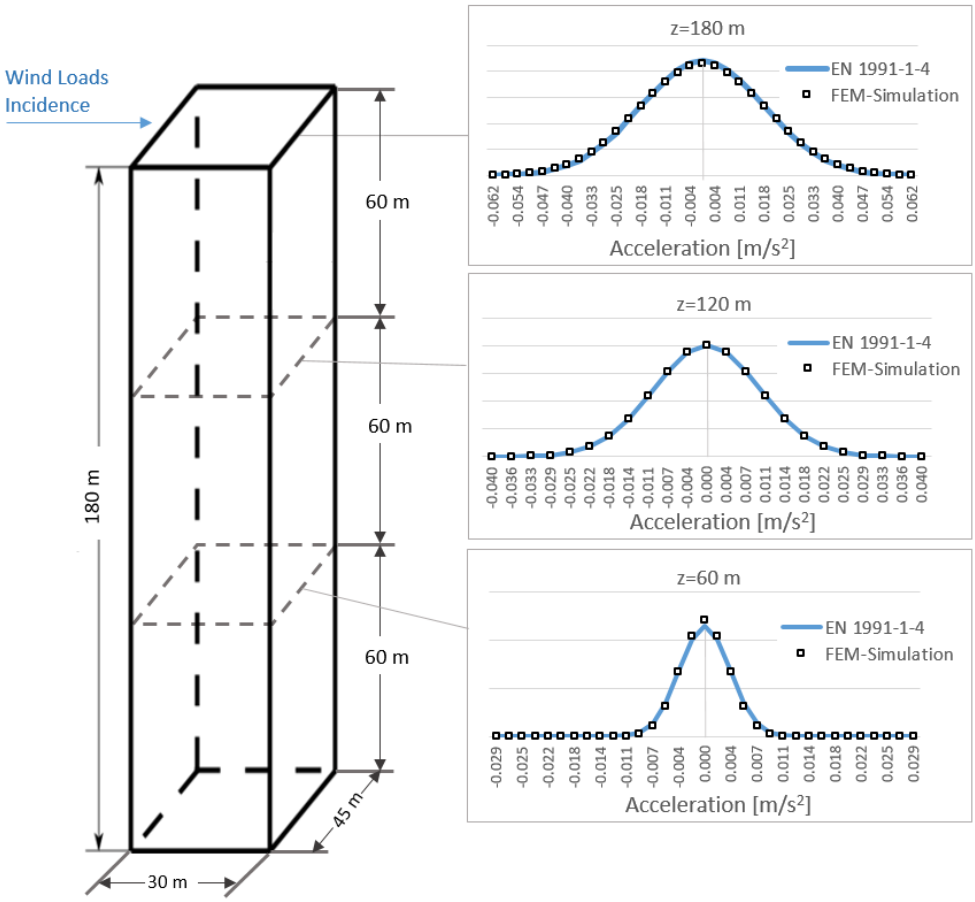


Figure 4.10: Comparison of Normal Distribution of acceleration depending on the height between EN 1991-1-4 and FEM results.

In relation to the obtained acceleration standard deviation, it is highly appreciable the great concordance that exists between the results. Additionally, this result match is maintained along the building in the 3 heights where the verification of the code is carried out. FEM standard deviation is 2,8% greater at 180 m height and 0,33% greater at 120 m height than EN1991-1-4, while it is 5,3% smaller for z=60 m. Hence, EN 1991-1-4 is 0,64% greater with respect to FEM, in average, in terms of the acceleration standard deviation out of the three studied heights. Therefore, it can be state how reliable the code is with respect to the simulation together with the correctness of the EN 1991-1-4 standard deviation of the acceleration followed function along the z-axis.

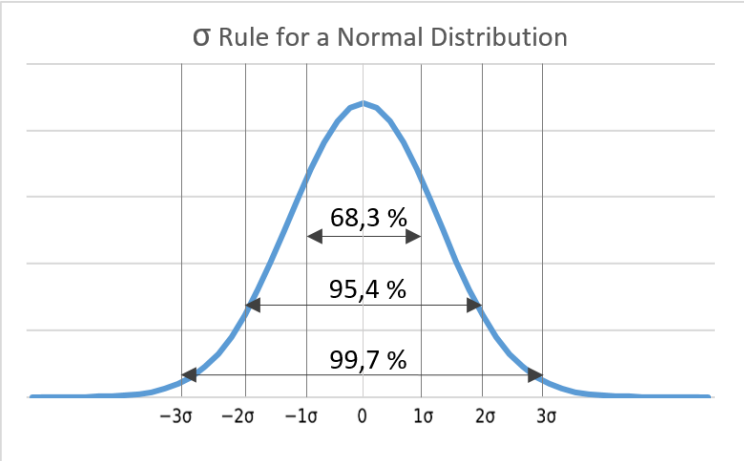


Figure 4.11: σ Rule for a Normal Distribution.

Table 4.6: Number of times 3 is exceeded for each of the simulation sets.

		Number of times 3σ is exceeded										
		Simulation										Average
		1	2	3	4	5	6	7	8	9	10	
z (m)	180	2	0	19	5	0	36	46	0	0	0	10.8
	120	0	0	26	2	4	41	0	0	10	1	8.4
	60	29	0	7	26	9	17	2	1	11	5	10.7

**Exceeding is consider for a value as either greater than +3σ or smaller -3σ.*
**It has been chosen for the comparison σ given by EN 1991-1-4.*

Furthermore, it is also of great importance to state some peak acceleration considerations. The related study is also carried out for the three FEM-studied heights. In Table 4.5 it can be seen an average of the absolute value of the peak values from the 10 simulations. In comparison with EN 1991-1-4 the result has high matching, with the greater difference presented in z=60 m, but not excessively divergence.

It is also relevant to make a 3σ study of the obtained normal distribution. Due the accuracy in the results between EN 1991-1-4 and FEM in terms of standard deviation, there is also a really approximated 3σ value for both cases, as also shown in Table 4.5. Therefore, as it can be seen in Figure 4.11, 99,73% of the generated values by a normal distribution of σ as its standard deviation, are within -3σ and +3σ. Evaluating all the obtained values from the FEM simulation sets for each height in relation to the 3σ value established by EN 1991-1-4 standard deviation of $0,0181(z/180)^{1,5}$, it can be stated that results meet expectations as out of 4097 simulated values it was expected around 11 exceeds, by either being greater than +3σ or smaller than -3σ. The number of exceeds among the values for each simulation and height together with an average per height, can be seen in Figure 4.6

Chapter 5

Conclusions and Further Researches

This last chapter is presented as the outcome in the form of conclusion of the ideas developed in the previous chapters. Hence, it will introduce the main conclusions that can be drawn from the realization of this project as well as future developments.

It will start by commenting the conclusions reached regarding the objectives set out in the first chapter, section 1.2. It is fundamental to establish the degree to which the objectives and goals previously set in this project have been satisfied.

Finally, the future researches and developments that are of interest after this project will be undertaken. In this way, the realization of this thesis serves as a starting point from which the aspects that are developed in this section but had to remain outside the boundaries of the project can be deepened.

5.1 Conclusions

The main objective of the project was to determine how well the European Wind Code EN 1991-1-4 currently applies in terms of the vibrations produced by the wind on a tall building. A series of secondary objectives were also considered, such as a comparative analysis with other wind codes, a study of the limitations of the EN 1991-1-4 in a general way and a comparison between the two study cases of wind loads incising on a building, both the simplified and the real, in order to expose the feasibility of the simplification.

Regarding the comparison made between the mainly used standards, the conclusion shares the direction stated by Kwon and Kareem (2013). After comparing all the codes in terms of similarities and differences, together with the places where these comparisons affect the design study, there are similarities in the along-wind response and the results tend to be more scattered attending to cross-wind dynamics. Hence, the main differences appear in terms of wind velocity

characteristics with a peak acceleration definition. [44]

However, a standardization could be made among the different norms by getting rid of the velocity profiles alternative procedures, thus eliminating the discrepancies concerning the turbulence intensity and the energy factors. Despite the differences, this goal is mainly achievable in the case of along-wind load effects over the gust loading approach factor. In the case of the across-wind and torsional loads, where there is an increase in wake effect relevance over buffeting effects in the along-wind direction, NALD database utilized in ASCE for increasing accuracy is a promising technique. Therefore, following all these considerations from the Kwon and Kareem study, it can be stated that the determination of a global standard is possible. Notwithstanding, it would not be an easy task to get every country involved in agreement.

The study was focused on the realization of two main comparisons aiming to study the accuracy and precision presented by the code.

On the one hand, the first main comparison pretended to be done was among the acceleration obtainment between the two study cases of wind load incidence over a building carried out by the European Wind Code. These two cases were the simplified case with only vertical correlation and the real case with vertical and horizontal correlation. As it has been developed, the way EN 1991-1-4 has to differentiate both cases is through the use of the aerodynamic admittance function in the horizontal axis $R_b(\eta_b)$. Thus, $R_b(\eta_b)$ is considered equal to one when there is no horizontal correlation, in the simplified case, and is less than one when horizontal correlation is considered, in the real case.

After developing the code procedures, it can be concluded that EN 1991-1-4 adequately considers the horizontal correlation of wind loads with a big gap between the results. Following the specifications made by Stathopoulos and Baniotopoulos (2007), it can be claimed that the results meet expectations [2]. It is undoubtedly remarkable the fact that to follow the simplified case is in the aim of ensuring the remain on the safety side on a great extent. Simplified case considers wind-incidence produced acceleration increased in a 77% with respect to the real load distribution assumption. Furthermore, it is also worth mentioning that the peak factor K_P is higher in the simplified case than in the real one, with an even greater excess of conservativeness by the use of R_b=1.

On the other hand, a comparison between EN 1991-1-4 and a FE-based approach in determining acceleration in the real case of horizontal and vertical correlation in order to evaluate the code precision has also been developed. In relation to the acceleration standard deviation, a remarkable concordance between the results was observed. Additionally, this result match is maintained along the building in three studied heights where the verification of the code had been carried out. FEM standard deviation is 2,8% greater for 180 m height and 0,33% greater in 120 m height than EN1991-1-4, while it is 5,3% smaller for z=60 m. Hence, EN 1991-1-4 is in average 0,64% greater with respect to FEM in terms of the acceleration standard deviation out of the three studied heights. Therefore, it can be stated

how reliable the code is regarding the simulation together with the correctness of the EN 1991-1-4 standard deviation of the acceleration followed function along the z-axis.

It was also of great importance to state some peak acceleration considerations for three FEM-studied heights. In comparison with EN 1991-1-4 the result has high matching without excessive divergence. Furthermore, a 3σ study of the EN 1991-1-4 normal distribution was performed. Evaluating all the obtained values from the FEM simulation sets for each height in relation to the 3σ value established by EN 1991-1-4, it can be stated that results meet the expectations. Out of 4097 simulated values, around 11 exceeding values were expected, either being greater than $+3\sigma$ or smaller than -3σ. The expected exceeding turned out to be above the average of values that are outside the limits marked according to the probabilistic area, giving rise to an overall positive conclusion about the accuracy of the results and the underlying safety in the European Wind Code EN 1991-1-4.

However, and in order to conclude, it is undoubtedly necessary to make variations in the introduced factors that have been selected for the study of vibrations of a building under wind incidence made by EN 1991-1-4. Hence, this project must be followed by the alteration of the parameters that have been chosen for this first study. It will be necessary to alter the dimensions of the building, its properties, its continuity in the direction of the z-axis or its center of gravity in order to carry out the study of EN 1991-1-4 with more unfavorable contour conditions.

5.2 Further Researches and Developments

Future research and developments that are of interest after the completion of this project are undertaken along this section. Therefore, the completion of this thesis, summarized by the conclusions development previously stated, serves as a starting point for further studies regarding the aspects that have remained outside the project boundaries.

The main intention was the study of the precision of the European Wind Code in the determination of vibrations produced in a building. Along the project, the ABAQUS FE program has been used for the comparison of results in the case of horizontal and vertical correlation consideration in relation to the EN 1991-1-4 aerodynamic admittance function. Thus, the development of the same comparative study with the use of the FE approach in the case of just vertical correlation is presented as a natural consequent study. In this stated aim, the procedure will be analogous to the previous case with wind loads that are not decreased by the aerodynamic admittance function and are also acting uniformly within each of the 60 nodes of the multi-degree of freedom CAARC Standard Tall Building under analysis.

Hence, the study of the European Wind Code must be carried out changing the considered values of the involved variables in this project. These variables should

be altered in an individualized way, that is, not to vary more than one variable at the same time, and thus, be able to make a sensitivity study of the variables in the European Wind Code and the FE procedure in a benchmarking way. Therefore, many elements have been considered and should be altered, such as the number of nodes of discretion, the dimensions and properties of the building that determine its subsequent natural frequency, the structural damping or the air properties, deviating from the standardized and fixed properties of a CAARC building. This procedure comes prior to the joint variation of two or more variables and the subsequent analog analysis of results.

Furthermore, as it has been mentioned, the study was carried out in a building of standard and known structure. However, it is of great interest to carry out the same study in a building whose characteristics are exceptional, as in the case of buildings in which there is no constant mass and shape in the z-axis direction, with different sections or with the movement of the center of gravity of the building away from its central axle. Therefore, the safety of EN 1991-1-4 in the determination of accelerations will be put on trial in an exigent manner with results of great interest concerning the late construction tendencies in relative apogee in big developed cities.

In conclusion and going back to the contour conditions alterations, it is also of great importance to analyze the influence of the environment surrounding the building. This factor is only reflected in the code with the table that can be seen in Annex B, and its influence must be tested. In this direction, it would be interesting to use the wind tunnel tests as another benchmarking technique. With these experiments, it would be possible to carry out a third variety of vibrations determination, which would give enormous robustness and consistency to the obtainment of the three-approach comparative results in order to evaluate EN 1991-1-4.

ANNEX A. EN 1991-1-4 Vertical Correlated Acceleration Determination

EN 1991-1-4 Annex B claim that "the characteristic along-wind peak accelerations are obtained by multiplying the standard deviation $\sigma_{a,x}$ by the peak factor k_p using the natural frequency as the upcrossing frequency $n_{1,x}$." [16]

$$a_{EN,vc}(z) = \sigma_{a,x,vc}(z) \cdot k_{p,vc} = 0,0996 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{1}$$

where

$a_{EN,vc}$: is the along-wind acceleration studied by EN 1991-1-4 of a CAARC building with vertical correlation (constant horizontal load).

$\sigma_{a,x,vc}$: is the standard deviation of the characteristic along-wind acceleration of the structural point at height z for the vertical correlation study.

$k_{p,vc}$: is the peak factor for the vertical correlation study.

This way the values of this variables must be determined in order to make the study.

(1) The standard deviation for the vertical correlation approach $\sigma_{ax,d}(z)$, according EN1991-1-4 Annex B is defined by the following expression. [16]

$$\sigma_{ax,vc}(z) = \frac{C_f \cdot \rho \cdot b \cdot I_v(z_s) \cdot V_m^2(z_s)}{m_{1,x}} \cdot R_{vc} \cdot K_x \cdot \phi_{1,x}(z) = 0,0322 \cdot \left(\frac{z}{180}\right)^{1,5} \frac{m}{s^2} \tag{2}$$

where

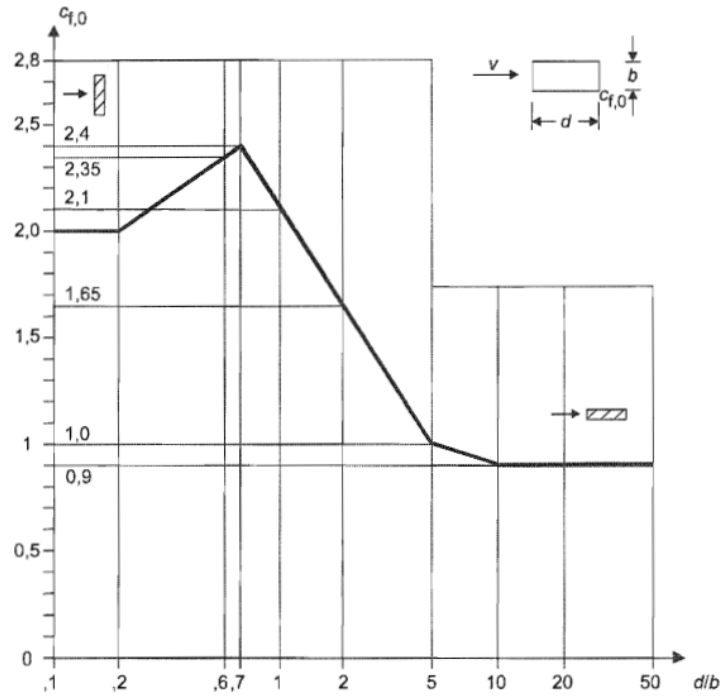


Figure 1: Force coefficient $C_{f,0}$ of rectangular sections with sharp corners and without free-end flow. [16]

C_f : is the force coefficient of structural elements of rectangular section with the wind blowing normally to a face.

It is given in EN 1991-1-4 Section 7.6 by the following expression.

$$C_f = C_{f,0} \cdot \psi_r \cdot \psi_\lambda = 2,19 \tag{3}$$

where

$C_{f,0}$: is the force coefficient of rectangular sections with sharp corners and without free-end flow as given by Figure 1. For a CAARC building with d/b=30/45, $C_{f,0}$ would be 2,38.

ψ_r : is the reduction factor for square sections with rounded corners that depends on Reynolds number. In this case it would be assumed as 1,0.

ψ_λ : is the end-effect factor for elements with free-end flow defined in EN 1991-1-4 Section 7.13 and is function of slenderness ratio λ determined in Table 1.

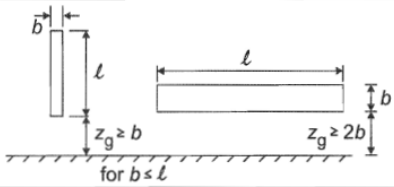
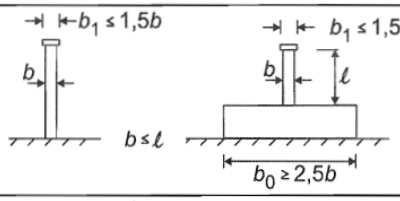
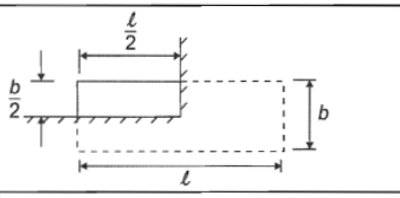
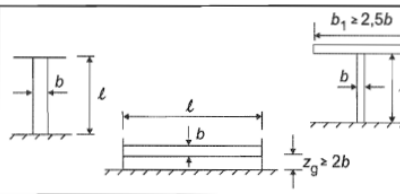
According to Table 1 (No.4), $\lambda = max(0,7 \cdot \frac{h}{b};\ 70)$=70.

As the solidity ratio φ, according to EN 1991-1-4 Section 7.13 is 1,0, attending Figure 2, the end-effect ratio ψ_λ is 0,92

ρ : is the air density. 1,25 kg/m^3 (Table 4.4)

b : is the width of the structure. 45 m (Table 4.4)

Table 1: Values of λ for structures sections given in EN 1991-1-4. [16]

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

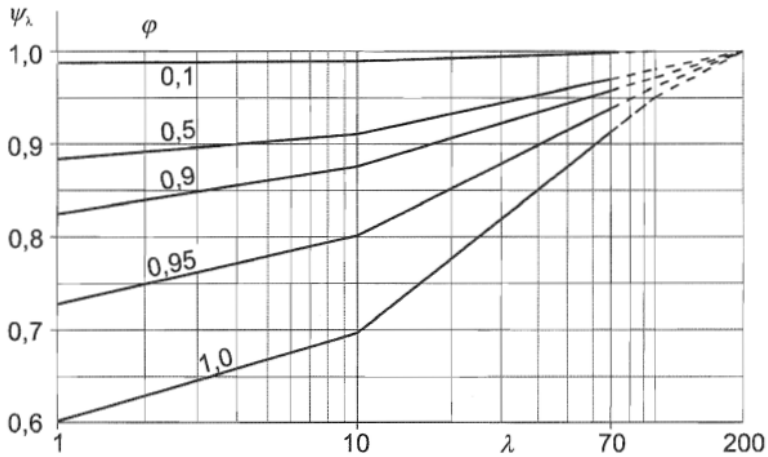


Figure 2: Indicative values of the end-effect factor ψ_λ as a function of solidity ratio φ versus slenderness λ. [16]

$I_v(z_s)$: is the turbulence intensity at the height $z = z_s$ above ground. (EN 1991-1-4 Section 4.4)

$$I_v(z) = \begin{cases} \frac{\sigma_v}{V_m(z)} = \frac{K_l}{C_0(z) \cdot ln(\frac{z}{z_0})} & z_{max} \geq z \geq z_{min} \\ \\ I_v(z_{min}) & z < z_{min} \end{cases} \tag{4}$$

where

z_{min} : is 10 m (Table 2)

z_s : is the reference height. z_s=$0,6 \cdot h$= 108 m (EN 1991-1-4 section 6.3.1)

Once this values are established, $I_v(z_s)$ can be described by the following expression.

$$I_v(z_s) = \frac{K_l}{C_0(z_s) \cdot ln(\frac{z_s}{z_0})} = 0,214 \tag{5}$$

where

K_l : is the turbulence factor. The value of K_l may be given in the National Annex. The recommended value for K_l is 1,0. (EN 1991-1-4 Section 4.4)

C_0 : is the orography factor as described (EN 1991-1-4 Section 4.3.3). C_0=1,0 as "the effects of orography may be neglected when the average slope of the upwind terrain is less than 3° " [16]

z_0 : is the roughness length. 1,0 (Table 2) [16]. The terrain category of the CAARC building is IV. (Annex B)

$V_m(z_s)$: is the mean wind velocity for the reference height z=z_s. Attending to EN 1991-1-4 Section 4.3.1 is given by the following expression.

$$V_m(z_s) = C_r(z_s) \cdot C_0(z_s) \cdot V_b = 16,128 \ m/s \tag{6}$$

where

$C_0(z_s)$: is the orography factor, taken as 1,0 according to EN 1991-1-4 Section 4.3.3.

$C_r(z_s)$: is the roughness factor for the reference height z=z_s greater than z_min (Table 2), given in EN 1991-1-4 Section 4.3.2 by the following expression.

$$C_r(z_s) = K_r \cdot ln\left(\frac{z_s}{z_0}\right) = 1,097 \tag{7}$$

where

Table 2: z_0 and z_{min} values depending on the terrain category. [16]

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

K_r : is the terrain factor depending on the roughness length Z_o that according to EN 1991-1-4 Section 4.3.1 is given by the following expression.

$$K_r = 0,19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0,07} = 0,234 \tag{8}$$

with $z_{0,II}$ as the roughness length of terrain category II. 0,05 (Table 2)

z_0 : is the roughness length. 1,0 (Table 2) [16]. The terrain category of the CAARC building is IV. (Annex B)

z_s : is the reference height. 108 m (EN 1991-1-4 Section 6.3.1)

V_b : is the basic wind velocity defined as a function of wind direction and time of year at 10 m above ground of terrain category II that according to EN 1991-1-4 Section 4.2 is given by the following expression.

$$V_b = C_{dir} \cdot C_{season} \cdot C_{prob} \cdot V_{b,0} = 14,7 \ m/s \tag{9}$$

where

C_{dir} : is the directional factor. In EN 1991-1-4 Section 4.2 the value of 1,0 is recommended.

C_{season} : is the season factor.In EN 1991-1-4 Section 4.2 the value of 1,0 is recommended.

C_{prob} : is the probability factor. Multiplies $V_{b,0}$ for determining V_b having the probability p and thus take into account an annual exceedence. (See *Note 1*)
According to EN 1991-1-4 Section 4.2 is given by the following expression.

$$C_{prob} = \left(\frac{1 - K' \cdot ln(-ln(1-p))}{1 - K' \cdot ln(-ln(0,98))}\right)^{n_e} = 0,7 \tag{10}$$

where

K' : is the shape parameter depending on the coefficient of variation of the extreme-value distribution. with the value of 0,2 (EN 1991-1-4 Section 4.2)

p : is the probability on annual exceeding. Assumed as 0,85. (See *Note 1*)

n_e : is an exponent with the value of 0,5 (EN 1991-1-4 Section 4.2)

$V_{b,0}$: is the fundamental value of the basic wind velocity. Unmodified basic value of wind speed for Finland is 21 m/s given for a 50 year return period.

Note 1. According to the ISO-10137 European acceleration limits are generally set as an approximate 1 year return. Due to this 50 year return basic wind velocity value there is a necessity to introduce C_{prob}.

R_{vc} : is the square root of resonant response for the vertical correlation approach study. In EN 1991-1-4 Annex B is defined by the following expression.

$$R_{vc}^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot R_h(\eta_h) \cdot R_{b,vc}(\eta_{b,vc}) = 0,388 \tag{11}$$

where

S_L : is the non-dimensional power spectral density function defined in EN 1991-1-4 Annex B by the following expression for z=z_s and n=$n_{1,x}$.

$$S_L(z_s, n_{1,x}) = \frac{n_{1,x} \cdot S_v(z_s, n_{1,x})}{\sigma_v^2} = \frac{6,8 \cdot f_L(z_s, n_{1,x})}{(1 + 10,2 \cdot f_L(z_s, n_{1,x}))^{5/3}} \tag{12}$$

$$S_L(z_s, n_{1,x}) = 0,073 \tag{13}$$

where

f_L : is a non-dimensional frequency with the frequency n=$n_{1,x}$ at the height z=z_s. Defined by the following expression defined in EN 1991-1-4 Annex B.

$$f_L(z_s, n_{1,x}) = \frac{n_{1,x} \cdot L(z_s)}{V_m(z_s)} = 2,463 \tag{14}$$

where

$n_{1,x}$: is the natural frequency of the structure in Hz, that for a CAARC building is 0,2 Hz. (Section 4.1.3.3 and Table 4.4)

$V_m(z_s)$: is the the mean velocity at the reference height. 16,13 m/s (Ex. 6)

$L(z_s)$: is the turbulence length scale that represents the average gust size for natural winds. It is defined in EN 1991-1-4 Annex B by the following expression (for $z_s > z_{min}$ like it was already defined. See Table 2).

$$L(z_s) = L_t\left(\frac{z_s}{z_t}\right)^{\alpha'} = 198,53\ m \qquad z_s \geq z_{min} \tag{15}$$

where

z_s : is the reference height: 108 m.

L_t : is the reference length scale: 300 m. (EN 1991-1-4 AnnexB)

z_t : is a reference height: 200 m. (EN 1991-1-4 Annex B)

α'' : is a constant defined: α''=$0,67$+$0,05 \cdot ln(z_o)$=0,67 (EN 1991-1-4 Annex B)

z_o : is the roughness length with the value of 1 m for terrain IV (Table 2 and Annex B).

S_v : is the one-sided variance spectrum. (EN 1991-1-4 Annex B)

σ_v : is the standard deviation. of the one-sided variance spectrum. (EN 1991-1-4 Annex B)

δ : is the total logarithmic decrement of damping. Is given in EN 1991-1-4 Annex F by the following expression.

$$\delta = \delta_s + \delta_a + \delta_d = 0,086 \tag{16}$$

where

δ_s : is the logarithmic decrement of structural damping. It is given by the following expression. [20]

$$\delta_s = 2 \cdot \pi \cdot x_i = 0,063 \tag{17}$$

whit x_i as the damping ratio to critical equal to 0,01 for the CAARC building studied as shown in Table 4.4. [32]

δ_a : is the logarithmic decrement of aerodynamic damping for the fundamental mode. As the mode shape $\phi(y, z)$ is constant for each height z, δ_a is given by the following expression. (EN 1991-1-4 Annex F)

$$\delta_a = \frac{C_f \cdot \rho \cdot b \cdot V_m(z_s)}{2 \cdot n_1 \cdot m_e} = 0,023 \tag{18}$$

where

C_f : is the force coefficient for wind action in the wind direction. For the CAARC building studied is equal to 2,19. (Ex. 3)

ρ : is the air density. 1,25 kg/m^3. (Table 4.4)

$V_m(z_s)$: is the mean wind velocity for reference height z=z_s. 16,13 m/s (Ex. 6)

b : is the width of the CAARC building structure. 45 m (Table 4.4)

n_1 : is the natural frequency of the structure in Hz, that for a CAARC building is 0,2 Hz. (Section 4.1.3.3 and Table 4.4)

m_e : is the along-wind fundamental equivalent mass per length. The specific mass of a CAARC building is known and has the value of 160 kg/m^3 (Table 4.4). Considering the CAARC building cross-section area as $b \cdot d$=45 $\cdot$ 30=1350 m^2.
Thus, the along-wind fundamental equivalent mass per length of a CAARC building is given by the following expression due to the specific mass of 160 kg/m^3 of a CAARC building. [32]

$$m_e = 160 \cdot b \cdot d = 216000 kg/m. \tag{19}$$

δ_d : is the logarithmic decrement of damping due to special devices (tuned mass dampers, sloshing tanks etc.) that is going to be neglected. (EN 1991-1-4 Annex F)

$R_h(\eta_h)$: is the aerodynamic admittance functions for a fundamental mode shape in buildings height direction h. Determined by the following expressions. (EN 1991-1-4 Annex B)

$$R_h = \frac{1}{\eta_h} - \frac{1}{2 \cdot \eta_h^2}(1 - e^{-2 \cdot \eta_h}) = 0,093 \tag{20}$$

where

η_h : is a non-dimensional variable defined in EN 1991-1-4 Annex B that is described by the following expression:

$$\eta_h = \frac{4,6 \cdot h}{L(z_s)} \cdot f_L(z_s, n_{1,x}) = 10,266 \tag{21}$$

where

h : is the building height: 180 m.

$L(z_s)$: is the turbulence length scale that represents the average gust size for natural winds.(EN 1991-1-4 Annex B). For the CAARC building studied it has the value of 198,53 m (Ex. 15)

$f_L(z_s, n_{1,x})$: is a non-dimensional frequency with the frequency $n_{1,x}$ at a height z_s. (EN 1991-1-4 Annex B). For the CAARC building studied it has the value of 2,463 (Ex. 14)

$R_{b,vc}(\eta_{b,vc})$: is the aerodynamic admittance functions for a fundamental mode shape in buildings width direction b for the vertical correlation approach study. Determined by the following expression. (EN 1991-1-4 Annex B)

$$R_{b,vc} = \frac{1}{\eta_{b,vc}} - \frac{1}{2 \cdot \eta_{b,vc}^2}(1 - e^{-2 \cdot \eta_{b,vc}}) \tag{22}$$

with

$$\eta_{b,vc} = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x}) \tag{23}$$

Nevertheless, as it was discussed, in this first study we are going to make an approach assuming a vertical correlation of the building. This situation implies an assumption of constant force in the horizontal (building width) direction which is an unreal simplification compared to the real nature of wind load incidence on a building.

Due to this assumption, as it was discussed in (Section 4.1.3.2) $R_{b,vc}$ will be considered to be 1,0 and $\eta_{b,vc}$ to be 0.

K_x : is the non-dimensional coefficient given in EN 1991-1-4 Annex B by the following expression.

$$K_x = \frac{\int_0^h V_m^2(z) \cdot \phi_{1,x}(z)\, dz}{V_m^2(z_s) \cdot \int_0^h \phi_{1,x}^2(z)\, dz} \tag{24}$$

According to EN 1991-1-4 Annex B, assuming $\phi_{1,x}(z) = \left(\frac{z}{h}\right)^{\zeta}$ and $C_o(z)$=1 (flat terrain, EN 1991-1-4 section 4.3.3) K_x can be approximated by the following expression.

$$K_x = \frac{(2 \cdot \zeta + 1) \cdot \{(\zeta + 1) \cdot [ln(\frac{z_s}{z_0}) + 0,5] - 1\}}{(\zeta + 1)^2 \cdot ln(\frac{z_s}{z_0})} = 1,634 \tag{25}$$

where

z_s : is the reference height: 108 m.

z_o : is the roughness length with the value of 1 m for terrain IV (Table 2 and Annex B).

ζ : is the exponent of the mode shape. According to EN 1991-1-4 Annex F it has the value of 1,5 for a CAARC building as a "slender cantilever buildings and buildings supported by central reinforced concrete cores". [16]

EN 1991-1- Annex F describes the following values for structures/buildings depending on its condition.

$\zeta = 0,6$: *for slender frame structures with non load-sharing walling or cladding.*

$\zeta = 1,0$: *for buildings with a central core plus peripheral columns or larger columns plus shear bracings.*
$\zeta = 1,5$: *for slender cantilever buildings and buildings supported by central reinforced concrete cores.*

$\phi_{1,x}(z)$: is the fundamental along wind modal shape. Is given in EN 1991-1-4 Annex F by the following expression.

$$\phi_{1,x}(z) = \left(\frac{z}{h}\right)^{\zeta} = \left(\frac{z}{180}\right)^{1,5} \tag{26}$$

where h is the CAARC building height: 180 m. ζ is the exponent of the mode shape that according to AN 1991-1-4 Annex F for the CAARC building being studied is 1,5.

$m_{1,x}$: is the along-wind fundamental equivalent mass per length. 216000 kg/m. (Table 4.4) (Ex. 19) [32]

(2) The peak factor $k_{p,vc}$ "defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation" for the case of vertical correlation approach study. According to EN 1991-1-4 Annex B is defined by the following expression. [16]

$$k_{p,vc} = max\left(\sqrt{2 \cdot ln(\nu_{vc} \cdot T)} + \frac{0,6}{\sqrt{2 \cdot ln(\nu_{vc} \cdot T)}};\ 3\right) = 3,089 \tag{27}$$

where

T : is the averaging time for the mean wind velocity, T=600 seconds. (EN 1991-1-4 Annex B)

$\nu_v c$: is the up-crossing frequency for the case of vertical correlation approach study. ν_d is given in EN 1991-1-4 Annex B by the following expression.

$$\nu_{vc} = n_{1,x} \cdot \sqrt{\frac{R_{vc}^2}{B^2 + R_{vc}^2}} = 0,106 \qquad ; \nu \geq 0,08 \text{ Hz} \tag{28}$$

where

B^2 : is the background factor, allowing for the lack of full correlation of the pressure on the structure surface. B^2=1 (EN 1991-1-4 Annex B)

$$B^2 = \frac{1}{1 + 0,9 \cdot \left(\frac{b+h}{L(z_s)}\right)^{0,63}} \tag{29}$$

where

b : is the width of the CAARC building. 45 m (Table 4.4)

h : is the height of the CAARC building. 180 m (Table 4.4)

$L(z_s)$: is the turbulence length scale that represents the average gust size for natural winds.(EN 1991-1-4 Annex B). $L(z_s)$=198,53 m (Ex. 15)

According to EN 1991-1-4 it is on the safe side to use $B^2 = 1$.

NOTE 2: by substituting the values, B^2 would be 0,56. Nevertheless, following EN 1991-1-4 and being on the safe side, B^2 will be assumed as 1.

R_{vc}^2 : is the resonance response factor for the vertical correlation approach study allowing for turbulence in resonance with the considered vibration mode of the structure. R_{vc}^2=0,388 (Ex. 11)

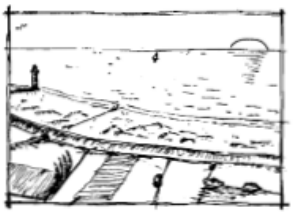
$n_{1,x}$: is the natural frequency of the structure in Hz, that for a CAARC building is 0,2 Hz. (Section 4.1.3.3 and Table 4.4)

ANNEX B. Terrain category - EN 1991-1-4

Terrain category definition made in EN 1991-1-4 Annex A.

Terrain category 0

Sea, coastal area exposed to the open sea

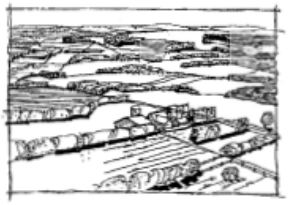


Terrain category I

Lakes or area with negligible vegetation and without obstacles

Terrain category II

Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights

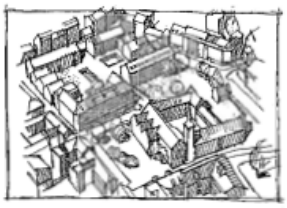


Terrain category III

Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)

Terrain category IV

Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m



ANNEX C. Wind simulated Loads Determination

In order to be able to make the proper study of this thesis it is necessary to use a correct Simulation Wind Loads with the basis established by the EN1991-1-4. Wind velocity and wind pressure are going to be studied along this section.

In this case, the thesis´s author role was a recompilation work in order to be able to achieve wind load simulation values used in Chapter 4 FE study. Furthermore, it is necessary to reflect this wind simulation values research was made by Olli Lahti.

Wind velocity must be defined for the case of study of this thesis. Thus, a building structure in a wind flow with a time dependent wind speed $v(z,t)$, assumed stationary Gaussian process, is going to be considered, composed of a mean and a fluctuating part as described. [4]

$$v(z,t) = v_m(z) + v'(z,t) \tag{30}$$

where mean velocity $v_m(z)$ follows the following logarithmic law.

$$v_m(z) = c_{jo}\ \ln\left(\frac{z}{z_0}\right) \tag{31}$$

where c_{jo} represent all a constant factors gathering such as orography and terrain factor. Hence, the study is going to be done attending to the wind force fluctuating component acting on a differential surface area dA, at height z, as the mean wind part remains constant. The case of study for the CAARC Standard Tall Building can be seen in Fig. 3. The fluctuating wind component has a mean of zero and standard deviation of σ_v. Power spectral density function of $v'(z,t)$ is obtained with Fourier Transformation. n is the frequency. [4, 67]

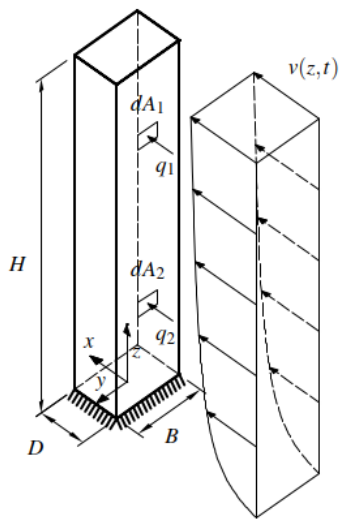


Figure 3: Study case for a CAARC building with wind force acting on a differential surface area.

$$S_{v'}(z,n) = \lim_{T\to\infty} \frac{1}{T} \int_{-T/2}^{T/2} R_{v'}(z,\tau)\, exp(-i2\pi nt) d\tau \tag{32}$$

where $R_{v'}$ is the auto-correlation of fluctuating velocity that is computed by the following expression.

$$R_{v'}(z,\tau) = \lim_{T\to\infty} \int_{-T/2}^{T/2} v'(z,t)\, v'(z,t+\tau)\, d\tau \tag{33}$$

Spectral density function $S_v(z,n)$ of v' is given by an empirical formula in the wind standards. According to EN1991-1-4 [16] it is given by the following expression.

$$S_L(z,n) = \frac{n \cdot S_v(z,n)}{\sigma_v^2} = \frac{6,8 \cdot f_L(z,n)}{(1+10,2 \cdot f_L(z,n))^{5/3}} \tag{34}$$

where f_L denotes the reduced frequency given by the following expression

$$f_L(z,n) = \frac{n \cdot L(z)}{v_m(z)} \tag{35}$$

Surface´s pressure induced by wind flow follows the equation of Bernoulli previously considered. In figure 2.4 it can be seen the typical wind flow incidence with wakes appearance in a building where if air was ideal or non-viscous, Bernoulli´s equation could be applied. Hence, it can only be applied in the outer

flow region [1]. In Fig. 2.3 it can be seen wind flow around the building and the formation of vortexes due to Wind pressures.

Pressures are normally expressed without dimension in order to be independent of velocity by the so-called pressure coefficient C_P, previously defined.

$$C_P = \frac{\Delta P}{\frac{1}{2}\rho V_0^2} \tag{36}$$

where ΔP is the pressure difference induced by wind. At point 1 in Fig. 2.3(b), the so-called stagnation point, ΔP equals to $1/2\rho v_0^2$, and thus, $C_p = 1$ being smaller than 1 in the rest of the areas. Time and height dependent wind velocity pressure at height z is obtained from the following expression assuming $v' << v_m$. [4]

$$p(z,t) = \frac{1}{2}\rho\Big(v_m^2(z) + 2\, v_m\, v'(z,t)\Big) \tag{37}$$

where it is appreciable the mean and turbulent pressure components. The latter component of wind pressure is, thus, described by the following expression. [4]

$$p'(z,t) = \rho\, v_m(z) v'(z,t) \tag{38}$$

Hence, wind pressure power spectral density function is described by the following expression.

$$S_{p'}(z,n) = \rho^2\, v_m^2(z) S_{v'}(z,n) \tag{39}$$

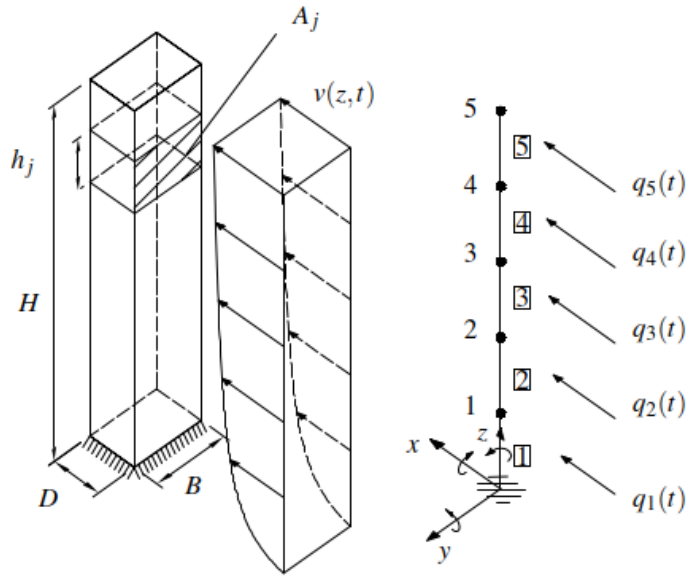
According to Simiu and Scanlan (1986), cross-spectrum can be obtained by the following expression. [68]

$$S_{p1'p2'} = \sqrt{S_{p1'} \cdot S_{p2'}} \cdot Coh(r_{12}, z_1, z_2, n) \tag{40}$$

where distance between two load points is r_{12}. Coherence function assuming vertical distance between the two load points is described by the following expression. [68, 4]

$$Coh(z_1, z_2, n) = exp\left(- C_z \cdot \frac{n \cdot |z_1 - z_2|}{\frac{1}{2} v_m(z_1) v_m(z_2)} \right) \tag{41}$$

where C_z is vertical direction exponential decay constant. Attending to Eq. 39, Eq. 43 and Eq. 40 it is possible to get the following expression.



(a) Building subjected to wind load. **(b)** Discretized model of the building.

Figure 4: Building under study simplification.

$$S_{q1'q2'}(n) = \rho^2 \, v_m(z_1) \, v_m(z_2) \sqrt{S_{v'}(z_1, n) \cdot S_{v'}(z_2, n)} \, Coh(r_{12}, z_1, z_2, n) \, dA_1 dA_2 \tag{42}$$

As cross-spectrum of wind forces is described by the following expression. [4, 67]

$$S_{q1'q2'}(n) = S_{p1'p2'} \, dA_1 dA_2 \tag{43}$$

In this project study there is going to be a vertical and horizontal correlation assumption, considering that along each of the nodes there is uniform wind incidence thanks to the use of the aerodynamic admittance function. The procedure will be the discretization shown in Fig. 4, as done by Castro and Bortoli (2015). On the other hand, there is a case that represents a simplified wind loading case on a building where there is only vertical correlation that gets out of the FEM simulation analysis bounds of this project. See Fig. 2.7 and 2.8 for the vertical and vertical and horizontal correlation justification, respectively [2].

Hence, force Spectra of the simulated wind loads is defined by the following expression.

$$S_{q,jk}(n) = \sqrt{S_{q,j}(n) \cdot S_{q,k}(n)} \cdot exp\left(- C_z \cdot \frac{n \cdot |z_j - z_k|}{\frac{1}{2} v_m(z_j) + v_m(z_k)} \right) \tag{44}$$

Nodal wind load cross-spectral densities can be unified into a matrix of spectral density. Thus, the spectral matrix is defined by the following expression.

$$S_q(n) = \begin{bmatrix} S_{q,11}(n) & S_{q,12}(n) & \dots & S_{q,1N_q}(n) \\ S_{q,21}(n) & S_{q,22}(n) & \dots & S_{q,2N_q}(n) \\ \vdots & \vdots & \ddots & \vdots \\ S_{q,N_q1}(n) & S_{q,N_q2}(n) & \dots & S_{q,N_qN_q}(n) \end{bmatrix} \tag{45}$$

Introducing Cholesky factorization, spectral matrix can be factored as follows.

$$S_q(n) = \mathbf{L}\,\mathbf{L}^T \tag{46}$$

where

$$\mathbf{L}(n) = \begin{bmatrix} L_{11}(n) & 0 & 0 & \dots & 0 \\ L_{21}(n) & L_{22}(n) & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ L_{N_q1}(n) & L_{N_q2}(n) & L_{N_q3}(n) & \dots & L_{N_qN_q}(n) \end{bmatrix} \tag{47}$$

It is necessary to make a Fourier Function Transformation in order to raise the wind force simulation in the time domain, and thus, be able to make the proper study. Time period is divided in numerous steps as T_p into N_t intervals of time of Δt length: $T_p = N_p \Delta t$, with r as a time value for the interval $r \in \{\ 1,2, \cdot\ \cdot\ \cdot\ N_t\}$:

$$t_r = r \cdot \Delta t \tag{48}$$

In the same way, having s as a frequency value so that $s \in \{\ 1,2, \cdot\ \cdot\ \cdot\ N_t\}$, frequency n can be defined by the following expression.

$$n_s = \frac{s}{N_t \cdot \Delta t} \tag{49}$$

For the simulation of nodal wind forces it is precise to attend to Ex. 45 following Wittig and Sinha (1975) approach [50], also applying the needed randomness to the expression studied by Shinozuka and Jan (1972) [51].

$$\begin{Bmatrix} \tilde{S}_1(\frac{s}{N_t \cdot \Delta t}) \\ \tilde{S}_2(\frac{s}{N_t \cdot \Delta t}) \\ \vdots \\ \tilde{S}_{N_q}(\frac{s}{N_t \cdot \Delta t}) \end{Bmatrix} = \Big(\frac{N_t}{2 \cdot \Delta t}\Big)^{1/2} \mathbf{L}\Big(\frac{s}{N_t \cdot \Delta t}\Big) \begin{Bmatrix} \zeta_{s1} \\ \zeta_{s2} \\ \vdots \\ \zeta_{sN_q} \end{Bmatrix} \tag{50}$$

where ζ_{sj}=a_{sj} + $i\, b_{sj}$, so that

$$E\,[a_{sj}] = E\,[b_{sj}] = 0 \tag{51}$$

and also

$$E\,[a_{sj}^2] = E\,[b_{sj}^2] = 0.5 \tag{52}$$

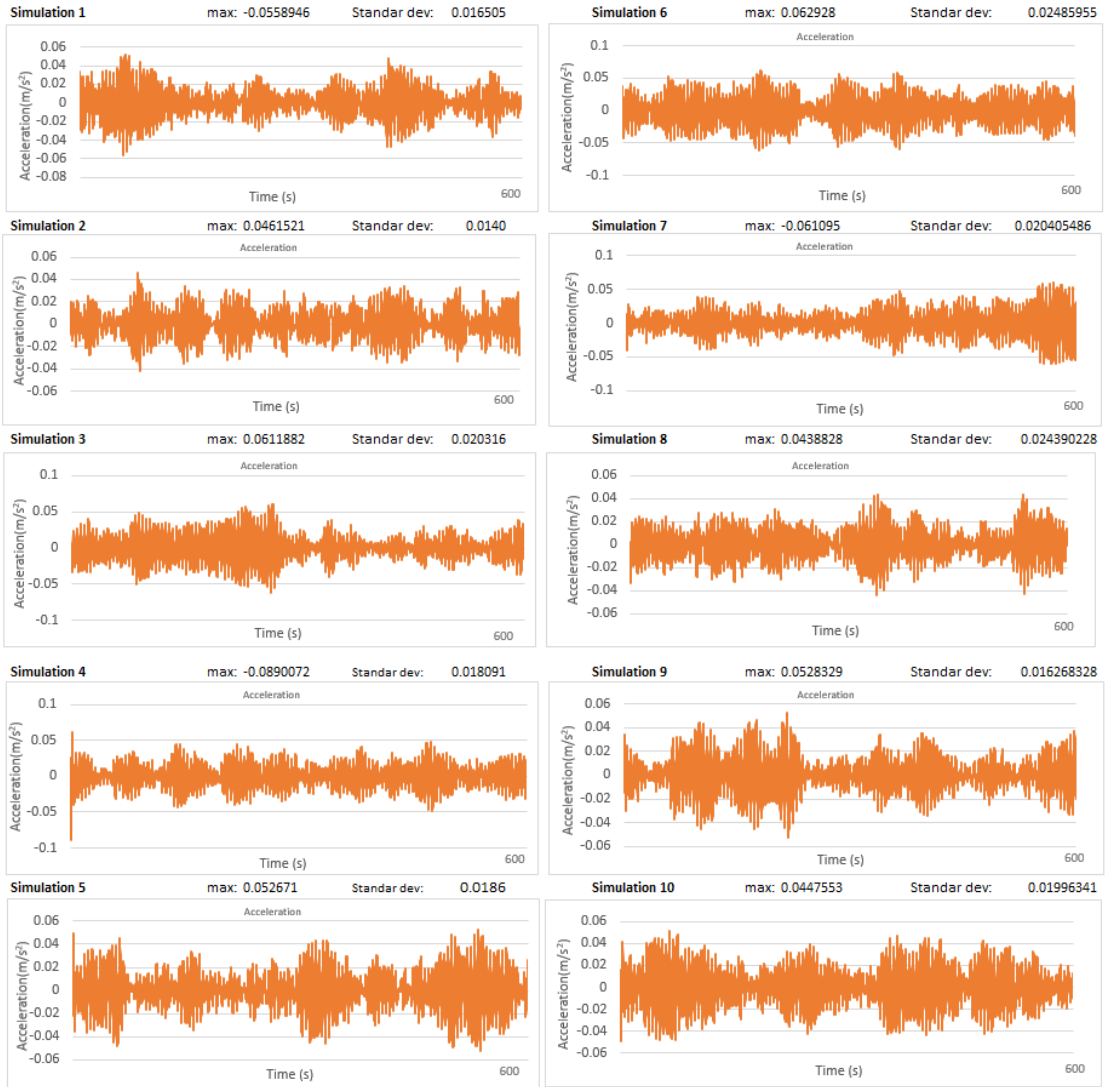
Using discrete Fourier Transformation, it is possible to transform from frequency to time domain, and thus, get the expression for the nodal force with respect to time t_r on a generic node j from the N_e nodes.

$$\tilde{q}_j(r \cdot \Delta t) = \frac{1}{N_t \cdot \Delta t} \sum_{s=0}^{N_t-1} \tilde{S}_j(\frac{s}{N_t \cdot \Delta t})\, exp\Big(i\frac{2\pi r s}{N_t}\Big) \tag{53}$$

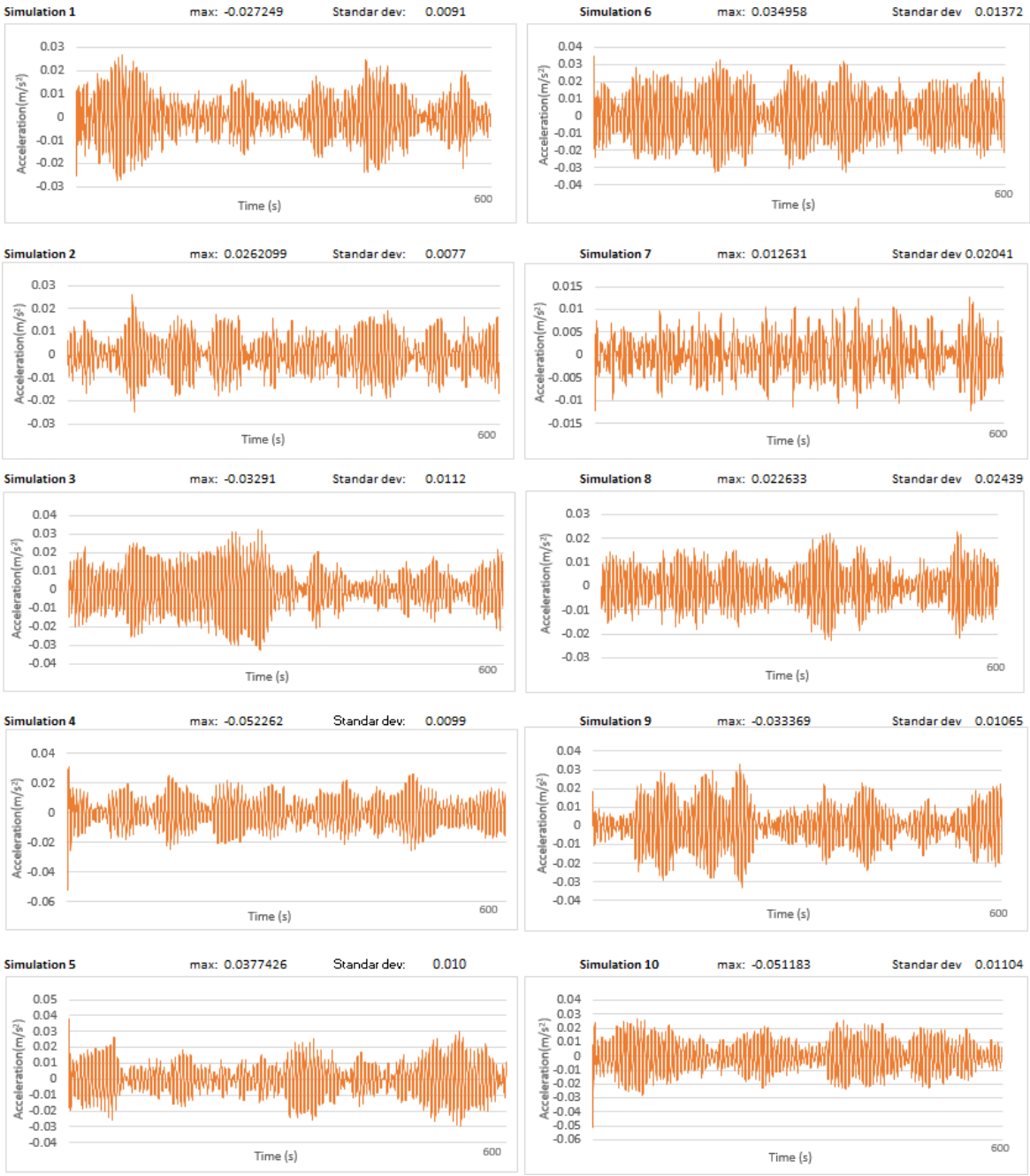
To conclude, aerodynamic admittance formulas for horizontal and vertical case need to be considered, and thus, obtain the simulated fluctuating part loads for each of the nodes and each of the 0,1465 s step that has been considered in the 600 s total simulation with its random component. Thus, the realistic case will be developed by introducing the aerodynamic admittance function proposed by EN 1991-1-4 to decrease loads according to the horizontal correlation.

ANNEX D. FEM Acceleration Results Distribution

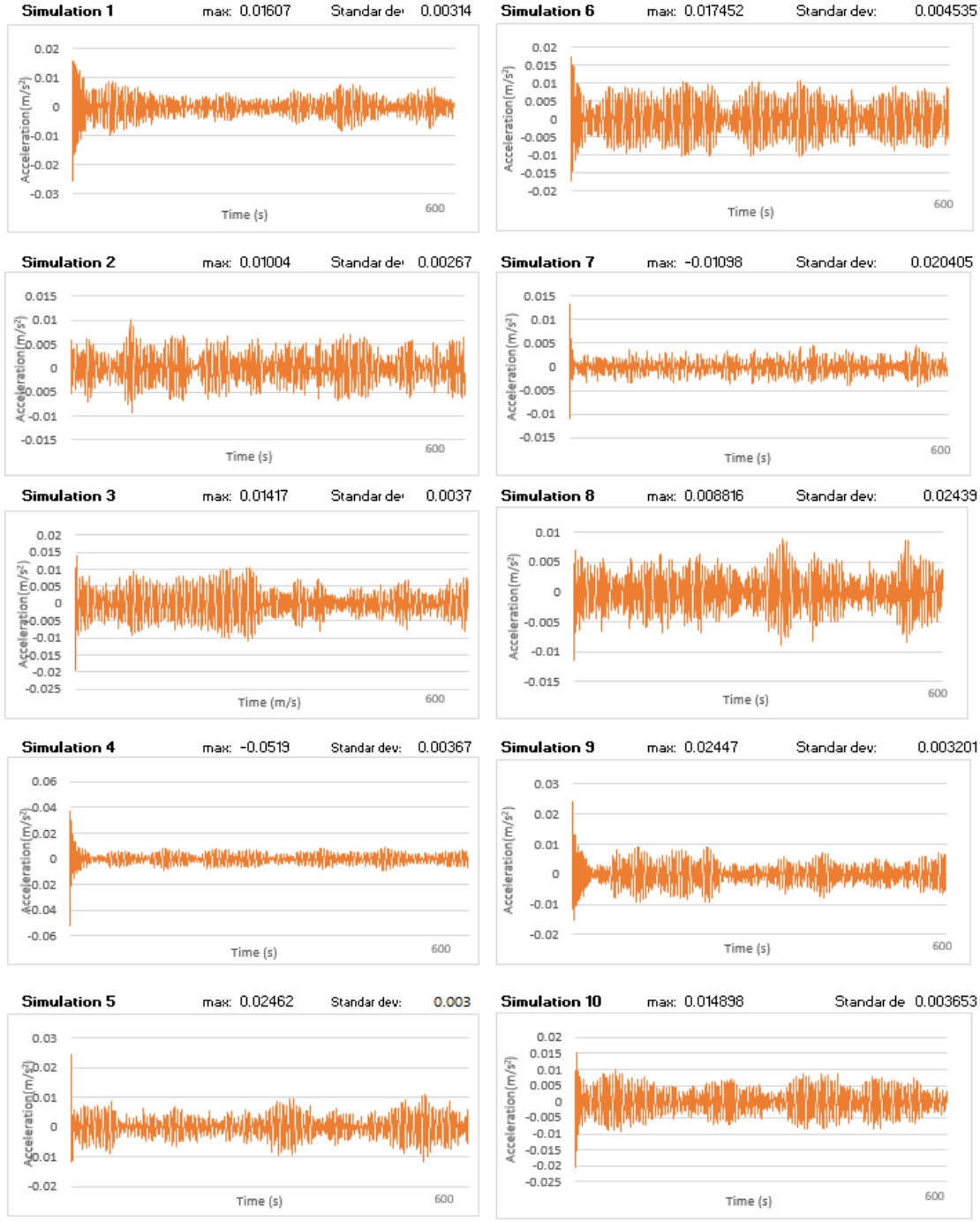
D.1. Acceleration results in the FEM-procedure for the studied 10 Simulations and 180 m height. Global standard deviation is 0,01861 m/s^2.



D.2. Acceleration results in the FEM-procedure for the studied 10 Simulations and 120 m height. Global standard deviation is 0,00988 m/s^2.



D.3. Acceleration results in the FEM-procedure for the studied 10 Simulations and 60 m height. Global standard deviation is 0,00331 m/s^2.



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