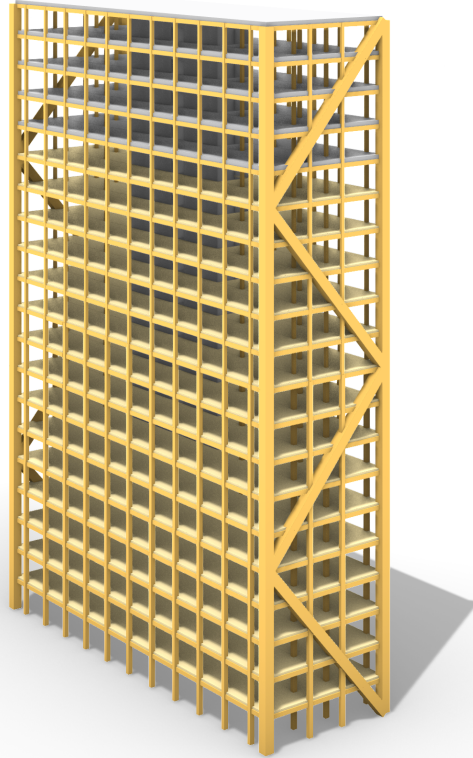




CHALMERS
UNIVERSITY OF TECHNOLOGY



A Hybrid Office Tower - ULS and SLS Dynamic Wind Study

A Study of Parameters Affecting Different Wind Response Modes
and Dynamic Performance of Tall Buildings

Master's thesis in Structural Engineering and Building Technology

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CHALMERS UNIVERSITY OF TECHNOLOGY
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FE-model generated using Grasshopper from Rhino 8 for a 20-storey hybrid building.
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ABSTRACT

This thesis investigates the dynamic response of tall timber–concrete hybrid office buildings subjected to wind-induced vibrations, with a focus on occupant comfort according to ISO 10137:2008. It evaluates how different structural configurations influence the along-wind, across-wind, torsional, and combined responses of a high-rise building. A parametric study was carried out, varying core thickness, bracing system, aspect ratio, and added mass.

Finite element models were developed and linked to a parametric workflow using Grasshopper and Python. The results show that the along-wind response governs all configurations, while the relative importance of across-wind and torsional responses varies with the structural strategy. However, considering only the along-wind response is not sufficient for investigated configurations, and all three response modes should be assessed together.

Serviceability in terms of occupant comfort governed both structural configuration and height. While the ultimate limit state was analysed, it was not governing for all studied configurations. Wind-induced accelerations were most effectively reduced by increasing stiffness through diagonal bracing or a thicker core. Changes in aspect ratio significantly reduced accelerations without additional material, indicating that early floor plan optimisation can be effective. However, with increasing height, timber–concrete hybrid structures become less material-efficient, requiring large concrete cores and added mass to meet serviceability criteria. The most efficient configurations were those up to 20 storeys.

Overall, the thesis highlights the importance of considering all three response modes early in the design of tall timber–concrete hybrid buildings, as structural configuration strongly influences wind-induced vibration performance.

Keywords: hybrid buildings, timber, concrete, high-rise buildings, wind-induced acceleration, along-wind, across-wind, torsion, comfort criteria, finite element analysis

Ett Hybridkontorshöghus – ULS och SLS Dynamisk Vindstudie

En Studie av Parametrar som Påverkar Olika Vindresponsmoder samt Dynamisk Prestanda hos Höga Byggnader

Examensarbete inom masterprogrammet Structural Engineering and Building Technology

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SAMMANFATTNING

Detta examensarbete undersöker den dynamiska responsen hos höga hybridkontorsbyggnader i trä–betong som utsätts för vindinducerade vibrationer, med fokus på brukarkomfort enligt ISO 10137:2008. Det utvärderar hur olika strukturella konfigurationer påverkar en höghusbyggnads längs-vinds-, tvärvinds- och torsionsrespons. En parametrisk studie genomfördes där tjockleken på hisschaktet, stagsystem, sidoförhållande och tillagd massa varierades.

Finita elementmodeller utvecklades och kopplades till ett parametriskt arbetsflöde med Grasshopper och Python. Resultaten visar att längs-vindsresponsen styr samtliga undersökta konfigurationer, medan den relativa betydelsen av tvärvinds-, torsions- och kombinerad respons varierar med den strukturella strategin. Att enbart beakta längs-vindsresponsen är dock inte tillräckligt för samtliga studerade konfigurationer, och alla tre responsmoder bör tas i beaktning.

Brukbarhetskrav i form av komfort för brukare styrde både den strukturella konfigurationen och byggnadens höjd. Även om brottgränstillståndet analyserades var det inte dimensionerande för samtliga studerade konfigurationer. Vindinducerade accelerationer reducerades mest effektivt genom ökad styvhet via diagonala stagningar eller ett tjockare hisschakt. Förändringar i sidoförhållande reducerade accelerationsnivåerna avsevärt utan extra material, vilket indikerar att tidig optimering av planlösningen kan vara ett mycket effektivt alternativ. Vid ökande höjd blir dock trä–betonghybrida strukturer mindre materialeffektiva och kräver stora betonghisschakt och tillagd massa för att uppfylla brukbarhetskraven. De mest effektiva konfigurationerna var de upp till 20 våningar.

Sammantaget understryker examensarbetet vikten av att beakta alla tre responsmoder tidigt i designen av höga trä–betonghybrida byggnader, eftersom den strukturella konfigurationen har stor inverkan på prestandan av vindinducerade vibrationer.

Nyckelord: hybridbyggnader, trä, betong, höga byggnader, vindinducerade accelerationer, medvindsrespons, tvärvindsrespons, torsion, komfortkriterier, finita elementanalys

Preface

In this study, dynamic wind analyses have been conducted on hybrid timber–concrete high-rise office buildings. The study was carried out from January 2026 to June 2026 at the Department of Structural Engineering, Chalmers University of Technology.

The project was conducted under the supervision of Thomas Hallgren at COWI. The work was carried out within the Buildings and Structures department at COWI in Gothenburg. We would like to express our sincere gratitude to Thomas Hallgren for his extensive expertise in wind dynamics and for his valuable guidance throughout the project. We would also like to thank COWI for providing us with the opportunity, as well as the tools and resources necessary to complete this research.

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Gothenburg, May 2026

Bingjie Hu

Krisjanis Kucins

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Notations

Roman upper-case letters

A	Cross-sectional area
A_c	Overall structural envelope area
A_m	Mean area enclosed by the centerline of a thin-walled cross-section
A_t	The equivalent translational acceleration of torsional acceleration.
A_θ	Angular acceleration of torsional vibration
B^2	Background response factor
C	Torsional constant of a beam cross-section
C_e	Exposure coefficient for snow load calculation
C'_L	Root-mean-square (RMS) overturning moment coefficient (AIJ)
C_t	Thermal coefficient for snow load calculation
C'_T	Root-mean-square (RMS) torsional moment coefficient (AIJ)
E	Modulus of elasticity
F	Non-dimensional wind energy spectrum function
F_w	External wind pressure on buildings
F_{ULS}	Resultant ultimate limit state load combination value
F_{SLS}	Resultant serviceability limit state load combination value
G	Shear modulus
G_D	Along-wind structural gust factor
G_k	Characteristic value of permanent action
H	Reference structural height
H_{xi}	Horizontal force acting on bracing member i in x-direction
H_{yi}	Horizontal force acting on bracing member i in y-direction
I	Area moment of inertia
$I(z)$	Mass moment of inertia distribution along height
I_M	Generalised polar mass moment of inertia for the first torsional mode
I_p	Polar mass moment of inertia
I_T	Generalized mass moment of inertia for torsional vibration (AIJ)
I_v	Turbulence intensity
I_z	Total polar mass moment of inertia obtained by summing all nodal contributions
$I_{z,i}$	Polar mass moment of inertia contribution from node i
K_{M1}, K_{M2}	Empirical dimensionless coefficients for torsional spectrum calculations
L	Beam span length
M_L	Generalised building mass corresponding to the first across-wind mode
$Q_{k,1}$	Characteristic value of the leading variable action
$Q_{k,j}$	Characteristic value of the accompanying variable action j
R^2	The resonant response factor
R_L^2	Resonant response factor for across-wind structural modes
R_M^2	Resonant response factor for torsional structural modes
S_L, S_M	Normalized across-wind energy spectrum and torsional wind spectrum

S_{M1}, S_{M2}	Intermediate spectrum component functions evaluating torsional torque
S_T	Global torsional stiffness
T	Reference time
T_a	Return period
W_L, W_T	Across-wind and torsional structural static load profiles per layer (AIJ)
W_e	Wind pressure acting on external surfaces
$\ddot{X}_{\max}$	Peak acceleration due to along-wind response

Roman lower-case letters

a_{comb}	Total combined acceleration obtained through vector summation
a_D	Along-wind structural acceleration
a_i	Distance from arbitrary position to center of bracing unit i in x -direction
a_L	Across-wind structural acceleration
a_{pL}	Peak value of across-wind acceleration at height z
a_{pM}	Peak value of torsional acceleration at height z
$a_{L\max}$	Maximum across-wind acceleration at the top of the building (AIJ)
$a_{T\max}$	Maximum torsional acceleration at the top of the building (AIJ)
$a_{T,x}, a_{T,y}$	Torsional acceleration components in global x - and y -directions
b	Width of the building
b_i	Distance from arbitrary position to center of bracing unit i in y -direction
b_o	Outer cross-sectional width of a hollow section
$c_0(z)$	Topography factor
c_d	Dynamic factor mapping structural scale and turbulence sensitivity
c_{dL}	Across-wind dynamic factor incorporating resonance and peak effects
c_{dM}	Torsional dynamic factor incorporating resonance and peak effects
c_{dir}	Directional wind climate correction factor
c_f	Aerodynamic force coefficient
c_L	Lateral force coefficient associated with across-wind overturning moment
c_M	Aerodynamic moment coefficient mapping base torque behaviour
$c_r(z)$	Roughness factor assessing terrain roughness variations over height
c_s	Size factor capturing non-simultaneous peak wind pressure effects
c_{season}	Seasonal wind velocity adjustment factor
d	Depth of the building
f_i	Natural frequency of a structural member for mode i
f_L	Equivalent across-wind static wind action per unit length
f_{LM}	Ratio threshold between structural across-wind and torsional frequencies
g_{aL}, g_{aT}	Response acceleration peak factors in across- and torsional directions (AIJ)
g_L, g_M	Statistical peak factors for across-wind and torsional actions
g_u	Peak factor for background wind velocity variations

h	Height of the building
h_i, h_o	Inner and outer cross-sectional heights of a hollow section
h_{ref}	Standard national baseline reference height
k_p	Statistical peak factor scaling standard deviations to peak responses
k_r	Terrain-dependent roughness coefficient factor
k_{L1}, k_{L2}	Directional factor parameters scaling across-wind spectrum components
m	Aspect-ratio flag indicator (m=1 or 2)
m_e	Equivalent mass per unit length assigned to the fundamental mode
m_i	Discrete mass of node i
m_M	Equivalent structural torsional static wind moment per unit length
m_L	Generalised mass of the building in the first across-wind mode
$n_{1,x}$	Fundamental along-wind natural frequency
n_L, n_M	Fundamental across-wind and torsional natural frequencies
n_{LM}	Governing minimum across-wind or torsional structural frequency
n_{s1}, n_{s2}	Across-wind shedding excitation and flow reattachment frequencies
q_b	Basic velocity pressure
q_H	Reference design velocity pressure at top-of-structure height H (AIJ)
q_m	Mean velocity pressure
$q_p(z)$	Peak velocity pressure containing mean and transient peak fluctuations
r	Radial distance vector from the centre of torsion to the target point
r_i	Distance from node i to the centre of gravity
s	Design value of snow load on roofs
s_k	Characteristic value of snow load on the ground
t	Element wall thickness parameter
$v_b, v_{b,0}$	Basic design wind velocity and fundamental geographical baseline wind velocity
$v_{b,2}, v_{50}$	Wind velocity limits referenced to 2-year and 50-year return periods
v_m	Mean wind velocity
v_m^*	Non-dimensional reduced mean velocity evaluated at roof height h
v_{T_a}	Scaled baseline velocity mapping to arbitrary target return periods
x_i, y_i	Plan coordinates of node i
x_{cog}, y_{cog}	Plan coordinates of the structural centre of gravity
γ_C, γ_C	Normalized non-dimensional scale metrics profiling coordinate frequency parameters
z	Vertical coordinate elevation parameter measuring from ground plane up
z_0	Local roughness length metric
z_{min}, z_{max}	Code-regulated operational elevation bounds for roughness equations
z_s	Effective benchmark structural monitoring height

Greek letters

α_{pM}	Torsional acceleration contribution
$\alpha_{pM,x}$	Torsional acceleration component acting along global x-axis
$\alpha_{pM,y}$	Torsional acceleration component acting along global y-axis

β_{L1}, β_{L2}	Damping-like structural configuration constants for across-wind spectrum functions
β_{M1}, β_{M2}	Wind profile exponents scaling torsional spectrum functions
γ_d	Partial factor for safety class evaluation
γ_{LM}	Directional structural action combination factors balancing L and M actions
δ_a, δ_s	Logarithmic aerodynamic damping and structural damping decrements
δ_L, δ_M	Total direction-specific damping values (across-wind and torsional)
θ	Geometric plan aspect angle of the building boundary lines
λ	Slenderness ratio
μ	Snow load shape coefficient
ν	Cross-up frequency tracking background-to-resonant transient boundaries
ξ	Structural modal exponent mapping elevation mode lines
ρ	Structural material mass density property
ρ_{air}	Air mass density
$\sigma_{\ddot{x}}$	Standard deviation of along-wind direction acceleration
σ_{aL}	Standard deviation of across-wind direction acceleration
σ_{aM}	Standard deviation of torsional direction acceleration
Φ_1, Φ_L, Φ_M	First fundamental structural mode shapes (along, across, and torsional)
Φ_b, Φ_h	Aerodynamic structural spatial correlation reduction functions
ϕ_L, ϕ_T	Vibration mode shape correction metrics (AIJ)
φ	Solidity ratio tracking solidity metrics
$\psi_{0,j}$	Combination factor tracking accompanying variable structural actions
ψ_r, ψ_λ	Aerodynamic shape mitigation factors for edge rounding and slenderness

List of Acronyms

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

AIJ	Architectural Institute of Japan
BFS	Boverkets författningssamling (Swedish National Board of Housing, Building and Planning)
CLT	Cross Laminated timber
EC	Eurocode
EN	European Norm (Eurocode Standard)
GL	Glue Laminated Timber
ISO	International Organization for Standardization
RMS	Root-Mean-Square
SLS	Serviceability Limit State
SRSS	Square Root of the Sum of Squares
SS	Svensk Standard (Swedish Standard)
ULS	Ultimate Limit State
UR	Utilisation Ratio

1 Introduction

This thesis focuses on the design of timber–hybrid buildings with emphasis on along-wind, across-wind, torsional, and combined wind-induced vibrations. This chapter provides an overview of the thesis by introducing the background and problem description, as well as presenting the aim, objectives, research questions, limitations, and ethical, ecological and social aspects of the study.

1.1 Background

The utilisation of timber in high-rise buildings has gained increasing interest in recent years. Due to stricter regulations regarding carbon footprint and the growing demand for sustainable and high-value buildings, developers are increasingly exploring the use of timber in taller structures (Tupenaite et al., 2023). The lightweight nature of timber can reduce the size of vertical load-bearing members and foundations, while high-rise timber buildings are often associated with architectural prestige and increased commercial attractiveness (Zurich North America, 2023).

However, despite these advantages, the relatively low stiffness and low mass of timber structures create challenges in meeting serviceability requirements related to wind-induced vibrations, even for buildings of moderate height (Johansson et al., 2016). Nevertheless, several examples have demonstrated that the dynamic behaviour of tall timber buildings can be improved by combining timber with materials such as concrete, providing additional mass and stiffness (Pastori et al., 2022).

Consequently, the question arises whether the height of timber–hybrid buildings is governed by ultimate limit state (ULS) or serviceability limit state (SLS) criteria. One challenge in the assessment of dynamic wind actions and accelerations is that the current Eurocode considers only along-wind response. Due to the susceptibility of timber–hybrid buildings to dynamic vibrations, investigation of across-wind, torsional, and combined response modes may also be crucial (Chen et al., 2026). Previous studies have shown that hybrid office towers, due to the open floor plans typically required, may be more susceptible to wind-induced torsional vibrations compared to residential buildings (Attia & Ingemarsson, 2025).

While timber–hybrid structures are generally considered more sensitive to wind-induced vibrations, the question remains which structural strategies are the most efficient for mitigating specific vibration modes while still maintaining efficient use of materials.

1.2 Problem Description

Due to their low mass and relatively low stiffness, timber-hybrid buildings are highly susceptible to dynamic wind actions, leading to design challenges for both Ultimate (ULS) and Serviceability Limit States (SLS), with a focus on occupant comfort criteria due to acceleration.

Previous research by (Dahlén & Niemi-Impola, 2023) evaluated how timber–concrete

hybrid systems meet the comfort criteria defined by ISO 10137, while (Alkhateb & Kallmér, 2024) extended this evaluation to cover both serviceability and ultimate limit states. Although these studies primarily focused on residential buildings, (Attia & Ingemarsson, 2025) investigated timber–hybrid systems for high-rise office buildings with open floor plans. However, all of the mentioned studies considered only along-wind response, raising questions regarding the significance of across-wind, torsional, and combined wind-response modes.

Existing mitigation strategies primarily rely on increasing structural stiffness or mass by modifying the primary stabilising system or adding heavy layers. However, for timber-hybrid buildings, adding mass cancels out the inherent benefits of lightweight construction and necessitates larger member cross-sections, raising questions regarding the efficient use of the material mix. Consequently, this study investigates whether wind-induced vibrations can instead be mitigated during the early design stage by optimising the floor plan aspect ratio, thereby improving dynamic performance without requiring additional structural materials.

1.3 Aim and Objectives

The aim of this master’s thesis is to investigate how timber and concrete can be combined to enable the construction of taller timber office buildings. The study focuses on timber–concrete hybrid structures, where timber is used as the primary structural material, while additional materials are utilised to improve structural performance and stability under dynamic wind loading.

To achieve this aim, the following objectives are defined:

- Evaluate the dynamic wind response of a timber–concrete hybrid high-rise office building according to current and upcoming Eurocode procedures.
- Investigate the influence of along-wind, across-wind, torsional, and combined accelerations on the structural behaviour and occupant comfort of tall timber buildings.
- Perform a parametric study to examine how variations in building height, floor plan aspect ratio, and stabilising system affect the wind-induced response of the structure.
- Compare the significance of across-wind and torsional response modes with the along-wind response described in the current Eurocode.
- Evaluate both serviceability limit state (SLS) and ultimate limit state (ULS) criteria for the investigated structural configurations.

1.4 Research Questions

The specifications of the issue being investigated are broken down into four questions below.

- Are torsional accelerations more governing than along-wind and across-wind accelerations for the height limitation of a high-rise timber–concrete hybrid of-

office tower?

- What effects do different building configurations have on the along-wind, across-wind, torsional, and combined accelerations?
- Which design limit state ultimately limits building height — SLS or ULS?
- Which building configuration optimises the efficient use of the material mix in terms of strength and mitigation of wind-induced accelerations?

1.5 Method

Literature Review An in-depth literature review of relevant standards, regulations, and design guidelines related to wind actions on high-rise buildings was conducted, specifically regarding along-wind, across-wind, and torsional wind-induced vibrations. Applicable standards and limitations related to vibration comfort criteria were also reviewed.

Analysis of Existing Structures Existing high-rise timber and timber–hybrid buildings were analysed to investigate how structural materials and stabilising systems are utilised to mitigate wind-induced vibrations.

Analytical Modelling An analytical calculation method was developed to model along-wind, across-wind, torsional, and combined wind-induced vibrations in accordance with the relevant formulas provided in international and European standards.

FE Modelling First, verification of the FEM-Design model was performed, followed by a mesh convergence study. Thereafter, the wind calculation model and the reference building model were developed. Accelerations for each wind response mode, as well as the combined response, were computed.

Parametric Study and Optimisation A parametric study was carried out by varying individual parameters for several building heights. The influence of each parameter on the dynamic behaviour of the structure was evaluated through serviceability and ultimate limit states. Based on the results, the most effective parameters for improving structural performance were identified and combined to develop an optimised building configuration.

1.6 Limitations

This study is carried out on a timber–concrete high-rise office building with a timber beam–column structural system and one concrete core. The study is limited to a specific core geometry, where only the core thickness is varied. The core placement is fixed at the centre of the building. The total floor plan area remains unchanged, although the effect of varying the floor plan aspect ratio is investigated.

Dynamic wind loads are evaluated based on the wind climate of Gothenburg and terrain category III in accordance with the current and upcoming Eurocodes. Diaphragm action, connection design, and vibration analysis of floors and beams are not included

in the study. Seismic loads are also excluded due to the geographical location of the reference building.

1.7 Ethical, Ecological, and Social Aspects

From an ecological perspective, concrete remains the most widely used construction material in the world, while its production is associated with significant greenhouse gas emissions (Gagg, 2014). Therefore, research into how renewable materials such as timber can be used efficiently may contribute to reducing the carbon footprint of buildings. In many projects, commonly used structural solutions are selected instead of the most material-efficient alternatives (Mei & Wang, 2021). For this reason, the thesis applies a parametric study combined with a genetic algorithm to investigate configurations that achieve both efficient structural utilisation and reduced concrete consumption. In addition, the study examines whether wind-induced vibrations can be mitigated during the early design stage through optimisation of the floor plan aspect ratio, without requiring additional structural materials.

From a social perspective, office buildings should be designed not only to maximise economic value for developers, but also to provide comfortable working conditions for occupants. Therefore, control of wind-induced vibrations, particularly accelerations, is essential for occupant comfort. Furthermore, previous projects and studies have indicated that timber buildings are often perceived as providing a healthier and more pleasant indoor environment.

From an ethical perspective, it is important to maintain transparency regarding the assumptions and limitations of the study. Although the current Eurocode primarily covers along-wind response, it is important to investigate additional wind response modes in order to better represent the true dynamic behaviour of the building and to account for them in the design process. At the same time, the results of this study should not be interpreted as direct design guidance without project-specific verification.

2 Theory

This theory chapter outlines the material properties of timber and concrete, structural systems for timber–concrete hybrids, and case studies of existing high-rise structures. It establishes the fundamental concepts and analytical methods for evaluating the dynamic response of tall buildings under wind loading, covering structural dynamics, boundary conditions, comfort criteria, and acceleration modes. Furthermore, along-wind, across-wind, and torsional wind actions are evaluated using current and upcoming Eurocodes, with their limitations compared against AIJ and ISO standards.

2.1 Timber

Timber is a dominant structural material in low-rise construction (National Association of Home Builders (NAHB), 2025). By storing absorbed carbon dioxide throughout its service life and replacing carbon-intensive materials like concrete and steel, timber significantly lowers a building's overall embodied carbon emissions (Swedish Wood, 2022).

2.1.1 Properties of Timber

The properties of structural timber depend on its microstructure, macrostructure, environmental conditions, and imperfections—distinguishing it from "wood," which refers to small, clear specimens (Swedish Wood, 2022). Wood itself is an anisotropic natural composite of cellulose, hemicellulose, and lignin whose mechanical properties vary heavily by grain orientation (Blaß & Sandhaas, 2017). Consequently, structural applications primarily distinguish between loading parallel and perpendicular to the grain while neglecting radial and tangential variations (Swedish Wood, 2022). While the material is significantly stronger and stiffer parallel to the grain, the actual performance of structural timber is ultimately limited by natural imperfections such as knots, spiral grain, and juvenile wood (Swedish Wood, 2022).

2.1.2 Timber Products

To overcome the dimensional limitations, anisotropic behavior, and natural imperfections of solid timber, wood is processed into Engineered Wood Products (EWPs) to enable larger cross-sections and taller structures (Blaß & Sandhaas, 2017). Combining multiple timber elements into a single member distributes defects like knots and pitch pockets, resulting in more homogeneous and predictable material properties. (Swedish Wood, 2022).

2.1.2.1 Glue Laminated Timber

Glued laminated timber (glulam) is an engineered wood product made of finger-jointed, kiln-dried timber laminations bonded together with parallel grain alignment to act as a single structural unit (Blaß & Sandhaas, 2017; Swedish Wood, 2022). Glulam can be manufactured into straight, built-up, or curved members, and can be arranged as a combined beam with stronger lamellae placed in the outer, highly stressed layers

(Figure 2.1).

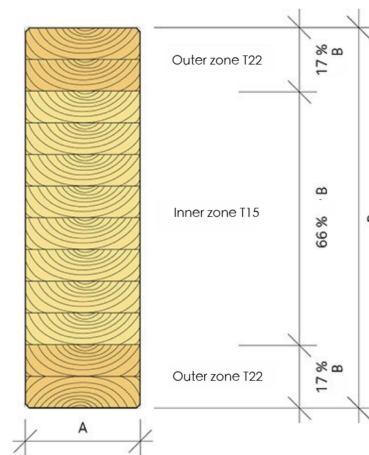


Figure 2.1: Glulam composition (Swedish Wood, n.d.-a).

2.1.2.2 Cross-Laminated Timber

Cross-laminated timber (CLT) consists of perpendicular layers of kiln-dried boards, typically 15–40 mm thick, bonded with structural adhesives (Blaß & Sandhaas, 2017). Arranged in configurations of 3, 5, or 7+ layers (Figure 2.2), CLT panels handle both out-of-plane and in-plane loading. Because the modulus of elasticity is significantly lower perpendicular to the grain, panel stiffness is primarily governed by layers oriented parallel to the principal stress direction (Swedish Wood, 2022).

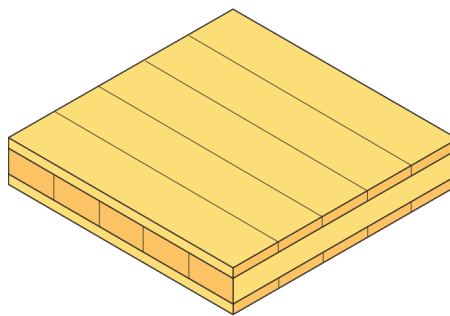


Figure 2.2: CLT panel with three layers (Swedish Wood, 2022).

2.1.3 Environmental and Sustainability Aspects

With roughly 70% of Sweden's land area forested, sustainable forest management and mandatory regeneration ensure a long-term timber supply (Swedish Wood, n.d.-b, n.d.-c). Actively growing forests maximize carbon dioxide absorption, providing significant climate benefits when locally sourced wood replaces carbon-intensive alternatives and minimizes transport emissions (Swedish Wood, n.d.-b, n.d.-c). Prefabrication enhances production efficiency, reduces material waste, limits construction site deliveries, and lowers structural self-weight—thereby minimizing foundation requirements and asso-

ciated emissions (Swedish Wood, n.d.-d). Decommissioned timber supports a circular economy through three cascading steps: direct structural reuse, recycling into lower-grade products, or burning for energy recovery to replace fossil fuels (Barth & McKinnon, 2023).

2.2 Concrete

The construction industry consumes more concrete than all other building materials combined (Gagg, 2014). Concrete is a composite of a binder, sand, gravel, and water. Admixtures can be added to improve its fresh or hardened properties, enabling its use in increasingly complex structures and harsh environments (Mehta & Monteiro, 2006).

2.2.1 Properties of Concrete

Concrete is a stiff material with a compressive strength significantly exceeding its tensile capacity, which is typically only 10% of its compressive strength (Al-Emrani et al., 2013). This disparity makes concrete highly susceptible to cracking from both applied loads and internal restraints (Engström, 2007). Despite this, its excellent durability against weathering and moisture makes it ideal for environments where timber or steel might decay or corrode (Mehta & Monteiro, 2006).

To manage tensile stresses, concrete is reinforced with steel bars, which provide necessary tensile capacity and control crack distribution, though they only become effective after cracking occurs (Al-Emrani et al., 2013; Engström, 2007). For increased efficiency and longer spans, prestressing can be employed (Mehta & Monteiro, 2006).

2.2.2 Environmental and Sustainability Aspects

While concrete gradually absorbs CO_2 through atmospheric carbonation over its service life, its primary climate impact stems from cement production, which accounts for 7–8% of global CO_2 emissions due to rapid urbanization (Barbhuiya et al., 2024; Mehta & Monteiro, 2006). The majority of these emissions originate from the calcination of limestone inside kilns heated to approximately 1450°C, generating 0.5–0.7 tonnes of CO_2 per tonne of clinker (Barbhuiya et al., 2024; Mehta & Monteiro, 2006). Additionally, the substantial thermal energy required to sustain these temperatures is traditionally supplied by fossil fuels, contributing further to greenhouse gas totals and associated air pollutants (Barbhuiya et al., 2024).

2.3 Structural System

To ensure global stability against lateral forces, structures utilize various stabilising systems. Diagonal bracings resist loads through axial force, while CLT or concrete shear walls resist in-plane shear and bending. Rigid frames rely on moment connections, and horizontal diaphragms distribute these horizontal forces to the vertical elements.

2.3.1 Shear Wall Structure

Shear walls utilize high in-plane stiffness to resist vertical, wind, and seismic loads. Individual planar walls act as vertical cantilevers, while assemblies around lift shafts or stairwells form three-dimensional cores with enhanced torsional stiffness, making them ideal for medium- to high-rise structures (Smith & Coull, 1991).

Though ideal for repetitive residential layouts, shear walls can restrict open-plan offices. In mixed frame-wall systems, these walls serve as the primary lateral resistance while the frame transfers gravity loads (Swedish Wood, 2022).

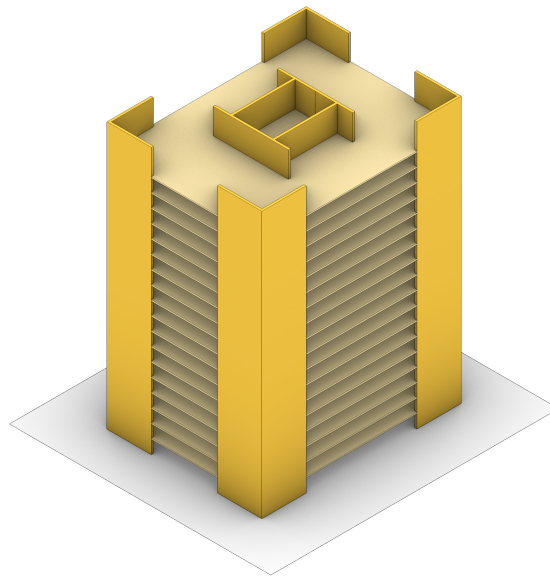


Figure 2.3: Example of shear wall structure with CLT-panels.

2.3.2 Braced System

Braced systems utilize diagonal members to form a vertical truss with floor girders, resisting lateral loads through alternating tension or compression depending on the load direction (Swedish Wood, n.d.-a). Steel is the most common material for these braces due to its high yield strength, offering high lateral stiffness with minimal material volume. While timber bracing requires bulkier cross-sections for buckling resistance and demands critical connection design to prevent splitting along the grain or crushing perpendicular to it (Smith & Coull, 1991).

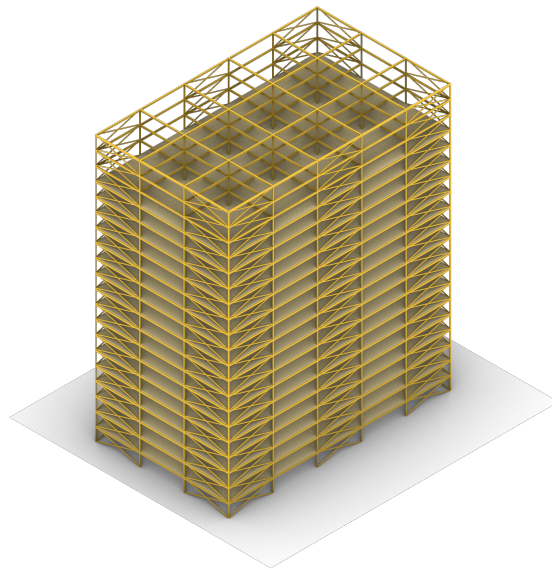


Figure 2.4: Example of braced system.

2.3.3 Core Structure

Acting as a cantilever from the ground, a structural core integrates staircases and lift shafts to serve as the primary shear wall system, resisting vertical loads, shear forces, and overturning moments (Smith & Coull, 1991).

Cores are commonly combined with a column-beam frame to form a dual structural system. While the core provides the vast majority of lateral resistance, stiffness compatibility ensures the surrounding frame participates dynamically—with columns carrying the bulk of the vertical loads. (Smith & Coull, 1991).

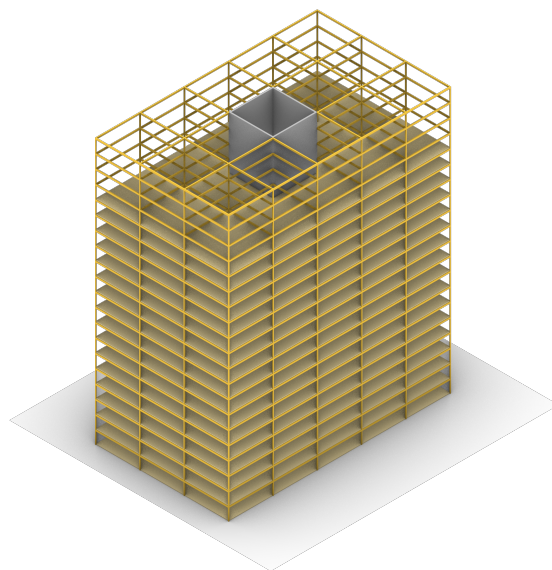


Figure 2.5: Example of core structure combined with column-beam system.

2.3.4 Timber-Concrete Hybrid Solutions

Timber–concrete hybrid solutions can enable more efficient use of materials by placing each material where it performs best and can help satisfy comfort criteria (Pastori et al., 2022). A wide variety of structural combinations is possible. Several typical configurations are listed below.

Concrete podium Concrete structures can be used as a podium or basement to provide a stiff and robust base for a timber superstructure. A common solution is to use concrete in the first storeys of the building. Although some projects utilise concrete at greater heights, the currently tallest hybrid timber building, Ascent MKE, employs a seven-storey concrete podium with mass-timber construction above (Fernandez et al., 2020).

Due to its durability in harsh environments and its ability to achieve larger spans, concrete podium levels are well suited to uses such as parking garages or commercial spaces (Mehta & Monteiro, 2006).

Concrete core/shear walls A concrete core and/or concrete shear walls increase the global stiffness of the building and commonly act as the primary stabilising system. Timber floor panels work as diaphragms, transferring horizontal loads to the core and shear walls. This system is commonly used in timber high-rise buildings and represents an efficient way to improve lateral stability while allowing the remaining structural system to be predominantly timber (Chen et al., 2026).

Due to the added equivalent mass and stiffness, concrete cores can also improve the building’s response to wind-induced vibrations by reducing acceleration levels. This approach has been applied in the HAUT building in the Netherlands, where the concrete core plays a key role in the dynamic behaviour of the structure and contributes to a favourable modal response, including control of torsional effects (Verhaegh et al., 2021).

Concrete floor panels The use of prefabricated concrete floor panels can enable longer spans without increasing the overall thickness of the floor structure. In hybrid solutions, concrete floors increase the mass of the building, which can be beneficial for occupant comfort by reducing wind-induced accelerations (Azanaw, 2025).

A common solution is the use of timber–concrete composite (TCC) floor panels, where the timber (e.g., CLT) primarily resists tensile stresses, while the concrete slab works in compression (Swedish Wood, 2022).

When additional mass is required to satisfy comfort criteria, an efficient strategy is to place concrete floors in the upper part of the building, where their influence on the dynamic response is greatest. This approach has been successfully applied in Mjøstårnet in Norway, where concrete slabs were used in the top seven storeys of the building (Abrahamsen, 2017).

Concrete megastructure In seismic regions or for very tall high-rise buildings, a concrete megastructure filled with timber structures can be an efficient hybrid solution. In such systems, the concrete megastructure provides global lateral resistance and carries the primary vertical loads, while the timber structure functions at a local level, typically spanning 3–4 storeys and resting on the concrete megastructure floors (Pan et al., 2021).

By adopting this approach, the overall building weight can be reduced compared with a fully concrete structure, and timber members can be designed more efficiently, as they carry loads primarily at a local scale (Pan et al., 2021).

This structural concept is being used in the construction of the 39-storey Atlassian Central building in Sydney, which is expected to be completed in 2027. Upon completion, it will be the tallest hybrid timber building to date (Page et al., 2025).

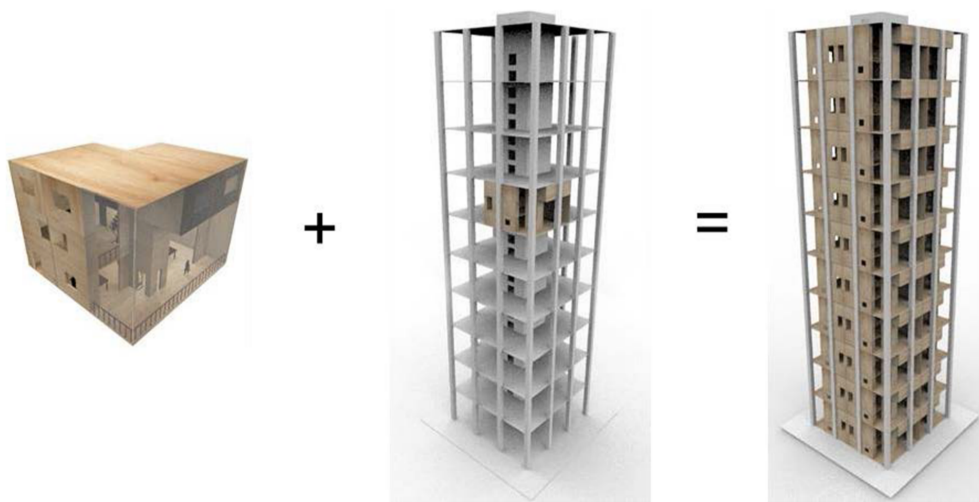
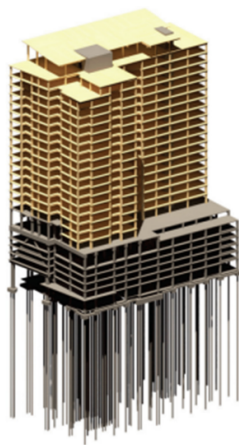
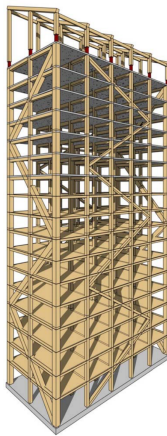
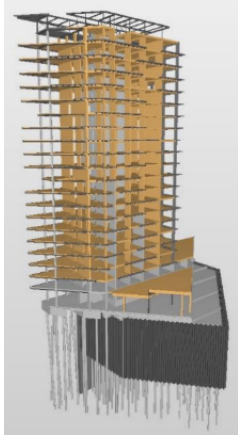


Figure 2.6: Example of concrete megastructure (Pan et al., 2021).

2.4 Case Studies

In recent years, several timber buildings have been constructed at heights of 50 metres and above. A shared challenge across these projects has been controlling wind-induced vibrations. However, as there are still no fully developed regulations for assessing wind-induced vibrations in timber–concrete hybrid buildings, different analytical approaches have been adopted. This section summarises the vibration criteria employed in designing some of the world’s tallest timber buildings and explains how lateral stability was achieved.

2.4.1 Comparison of Tall Timber Buildings

Schematic of the structure ¹	Ascent MKE	Mjøstårnet	Haut
			
Country	United States	Norway	Netherlands
Height [m]	86.6	85.4	73.0
Completed [year]	2022	2019	2021
Structural system	Concrete podium + 2 concrete cores + GL columns and beams + CLT floors	GL columns + beams + braces and CLT floors and concrete floors	Concrete podium + concrete core + CLT shear walls + GL beams + steel and concrete edge beams + TCC floors
Considered wind effects	Not mentioned	Along-wind acceleration	Along-wind and across-wind acceleration ²
Calculation method	Not mentioned	Eurocode 1-4 (2010)	NBCC Guidelines (2017, 2018)
Design natural frequency [Hz]	Not mentioned	$n_{1,x} = 0.33$ $n_{1,y} = 0.37$	$n_{1,x} = 0.43$ $n_{1,y} = 0.53$ $n_{1,t} = 0.57$
Damping design [%]	Not mentioned	Damping ratio of 1.9	Damping ratio of 1.5

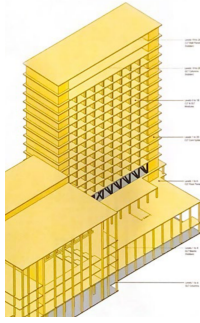
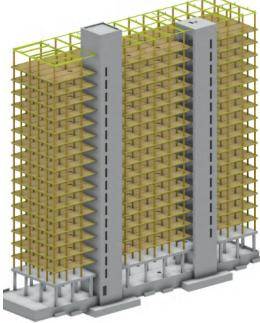
¹ Schematic representations adapted from (Fernandez et al., 2020), (Abrahamsen, 2017), and (Verhaegh et al., 2021).

² The analysis of across-wind acceleration was not specifically reported. Since the method of NBCC was used, it is assumed that an across-wind acceleration was analysed.

Ascent MKE is a 25-storey, 86.6-metre-high residential concrete–timber hybrid building. It is currently recognised as the world’s tallest timber–concrete hybrid building (Chen et al., 2026). Completed in 2022, it is located in Milwaukee, USA. The building consists of 25 floors. The first seven floors, which are used as a car park, are made of reinforced concrete using post-tensioned flat plate slabs. The top eighteen floors, which are used as residential spaces, are made of mass timber with GL columns, beams, and one-way spanning CLT slabs. The building’s lateral stability is achieved using two reinforced concrete cores spanning its full height (Fernandez et al., 2020).

Mjøstårnet is an 18-storey timber building located in Brumunddal, Norway. Its superstructure consists of glulam truss bracings along the façades, as well as glulam columns and beams. These bracings are the primary structural components for handling horizontal forces. The CLT cores and shear walls provide secondary vertical support but do not contribute to the building's stiffness against lateral forces. The concrete base slab of Mjøstårnet is supported by piles driven into the bedrock below. Additional concrete floor slabs are located on floors 12–18. These add weight to the building and mitigate vibration, which is important given the building's slenderness in its strong direction. However, the 18th floor slightly exceeds the residential comfort criteria regarding peak acceleration limits, and this has been accepted by the client (Abrahamsen, 2017). The remaining storeys employ lightweight timber cassette composite floors. Together with the concrete slabs, these create a rigid diaphragm that ensures efficient wind load transfer to the bracings (Chen et al., 2026).

Haut Amsterdam is a 21-storey, 73-metre-high concrete–timber hybrid residential building. Completed in 2021, it is located in Amsterdam, the Netherlands. The ground and first floors were constructed from concrete to provide robust support for the timber tower above. The top nineteen floors of the building are made of prefabricated timber–concrete composite (TCC) panels, supported by glulam beams, CLT walls, and glulam columns. Lateral stability is provided by a concrete core spanning the entire height of the building. Due to architectural considerations, the concrete core was placed eccentrically, introducing torsional effects from wind loads. To reduce these effects and ensure that the dominant mode is not torsional, the CLT walls were designed to form part of the stability system, and the concrete core was stiffened by extending its walls to a greater width outside the shaft. The CLT walls were designed to contribute to the structure's stability only in SLS, while the concrete core was designed to transfer and resist full wind loads in ULS (Verhaegh et al., 2021).

Schematic of the structure ¹	Sara Kulturhus	Brock Commons	Fyrtornet
			
Country	Sweden	Canada	Sweden
Height [m]	74.0	53.0	51.0
Completed [year]	2021	2017	2024
Structural system	GL columns and beams + steel columns and beams + CLT modules and cores	Concrete podium + 2 concrete cores + GL columns + CLT floors	GL columns and beams and braces + CLT core and floors
Considered wind effects	Not mentioned	Along-wind and across-wind acceleration ²	Along-wind, across-wind, torsional acceleration and their combination
Calculation method	Not mentioned	NBCC Guidelines (2017, 2018)	Eurocode 1-4 + NBCC + wind tunnel test
Design natural frequency [Hz]	Not mentioned	$n_{1,x} = 0.5$	$n_{1,x} = 0.72$ $n_{1,y} = 0.73$ $n_{1,t} = 0.97$
Damping design [%]	Not mentioned	Damping ratio of 1.5	Damping ratio of 1.9

¹ Schematic representations adapted from (Kılınc, 2024), (*Brock Commons tall wood building*, 2016), and (Giampellegrini et al., 2023).

² The analysis of across-wind acceleration was not specifically reported. Since the method of NBCC was used, it is assumed that an across-wind acceleration was analysed.

Sara Kulturhus is a mixed-use 20-storey building located in the city of Skellefteå in northern Sweden. The tower section of the building houses a hotel and rises above the conference and performance centre. This presented an important design challenge, as it meant creating large spans without vertical supports within the space. A lattice beam system comprising glulam beams and steel tension and compression rods supports the 13.5 m openings on the second floor. On the fifth floor, steel H-section beams provide a base for the hotel floors. From the sixth floor upwards, prefabricated CLT modules were tied together using steel brackets to form a structural lattice. These modules were placed within, and connected to, the two CLT cores at the short ends of the structure. Together with the additional weight of the top two concrete floor slabs, this provided the necessary stiffness for the tower. The concrete floors act as a damper, slowing the

building's acceleration from wind loads (Kılınç, 2024).

Brock Commons is an 18-storey, 53.0-metre-high concrete–timber hybrid student residence. Completed in 2017, it is located in Vancouver, Canada. The first storey and two elevator shafts are made of reinforced concrete, while all the other storeys are made of GL columns and two-way spanning CLT panels. In order to make full use of the CLT panels and avoid the need for beams, the column spacing is relatively close at 2.85 m × 4.0 m. The building's lateral stability is provided by two concrete cores. The structure's first mode has a frequency of 0.5 Hz and the design acceleration level at level 18 was set to 1.5% of gravity. To fulfil comfort requirements, the structural analysis accounts for an increase in damping from non-structural components, such as finishes and partitions (Fast et al., 2016).

Fyrtoronet is an 11-storey office building located in the Hyllie city district of Malmö, Sweden. Its superstructure comprises a glulam framework of columns and beams with CLT floor slabs. Horizontal and torsional stability is provided by a CLT core and glulam bracings (Larsson et al., 2025). Due to the sloped façade on one side of the building, the glulam bracings are shifted, with horizontal loads transferred between them and the core. Where the glass façade is vertical, the bracing system is continuous across the entire building. All of the floor slabs are made of CLT, except for the top floor slab, which is made of concrete to reduce wind-induced accelerations at the top (Giampellegrini et al., 2023). The entire structure stands on a single basement level of concrete, which is supported by a flat mat foundation on moraine. Wind-induced lifting forces are resisted by anchorage in the bedrock below the building. An ambient vibration test was performed at different stages of construction. In many cases, the translational and torsional vibrations were not independent, so they were considered as a combined effect in both the analytical and wind tunnel assessments. The parameter study indicated that the mass of the top floor and the stiffness of the core–panel connection have the most significant impact on the stiffness of the building (Chen et al., 2026).

2.4.2 Upcoming and Proposed Structures

Atlassian Central is a 39-storey, 183-metre-high commercial building made of concrete, steel, and timber. Construction is underway and it is planned to be completed in 2027. It is located in Sydney, Australia. The building consists of a concrete and steel mega-frame with a concrete floor every four floors. Once the mega-frame is complete, the space between the concrete floors will be filled with three levels of glulam-framed structures and one-way spanning CLT slabs. The timber is only used to carry loads on a local, three-storey scale. Lateral stability will be provided by a concrete core and a massive steel bracing system in the façades (Page et al., 2025).

River Beech Tower research project explores the possibility of constructing an 80-storey, 244-metre-tall timber residential tower in Chicago, USA. The building will consist of two slender towers connected by glulam bracing to create a wider footprint and increase stability. The structural system will consist of CLT shear walls, glulam bracing, and an LVL diagrid structure that efficiently carries both vertical and horizontal loads,

thereby reducing material consumption compared with traditional column–bracing systems (Sanner et al., 2017).

W350 is a research project envisioning a 70-storey timber-hybrid super-tall building in Marunouchi, Japan. It is planned to be 80 m × 80 m in plan and 350 m in height, with an outer frame including a steel–timber hybrid moment frame and a steel braced tube frame with dampers. This outer frame is designed as a stiff tube system for seismic control and to withstand all lateral loads. The steel braces with dampers in the outer frame further mitigate the effects of strong winds and earthquakes on the building’s stiffness. The internal frame, which is made of timber, only needs to resist vertical loads and therefore lacks walls and braces. The building is then clad with a timber interface made of wood with a smaller cross-section (Harada et al., 2020).

2.5 Wind Dynamics

Wind loading on tall buildings can be described as a loading chain consisting of four links. The first link represents the local wind climate, derived from historical meteorological measurements and models. This information is used to estimate design wind speeds for a specified return period.

The second link considers the local site conditions and their associated factors. These include terrain roughness, topography, and site exposure, all of which influence the wind characteristics approaching the building.

When the wind reaches the building, aerodynamic effects are generated due to the interaction between the wind profile and the structure. This third link addresses how the building’s geometry and aerodynamic properties influence both the magnitude and the dynamic characteristics of the wind load (Abu-Zidan et al., 2022).

Finally, the fourth link concerns the building’s dynamic response to wind excitation. Here, the mass, stiffness, damping, and natural frequencies of the structure, together with the frequency content and magnitude of the wind load, govern the resulting dynamic effects on the building. An illustration of the wind loading chain is shown in Figure 2.7.

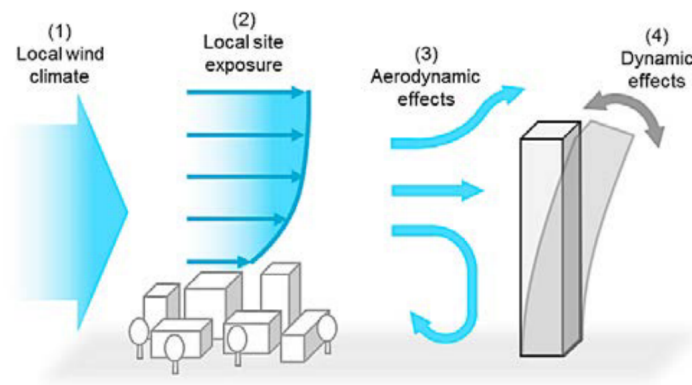


Figure 2.7: Schematic depiction of wind loading chain (Abu-Zidan et al., 2022).

2.5.1 Wind Characteristics

The wind velocity at a given height in the atmosphere can be expressed as the sum of a mean wind component (static) and a dynamic component (turbulence). Turbulence represents the fluctuating part of the flow superimposed on the mean wind and arises from eddies of various sizes generated by thermal effects, surface roughness, and other atmospheric processes. The mean wind is defined as the wind speed averaged over a specified time interval, typically 10 min, and represents the background flow. Gusty wind consists of short-term fluctuations in wind speed and direction that contribute significantly to the dynamic loading on structures (Abu-Zidan et al., 2022).

The wind profile near the ground forms the atmospheric boundary layer (ABL), where wind speed increases with height due to decreasing surface friction. See Figure 2.8. Above the ABL, the gradient wind occurs, where the influence of surface friction becomes negligible. Terrain roughness strongly affects the mean wind speed close to the ground, the turbulence intensity, and the thickness of the boundary layer. In Eurocode, five terrain categories are defined, as presented in Figure 2.9.

The wind profile at height z is described by the mean wind speed $v_m(z)$. A logarithmic velocity profile is used to define the roughness factor $c_r(z)$, which accounts for both terrain roughness and height above ground. This equation applies to mean wind speed profiles, not gust speed profiles. To determine the peak velocity pressure $q_p(z)$ at height z , both the mean wind speed and the short-term velocity fluctuations must be considered (Abu-Zidan et al., 2022).

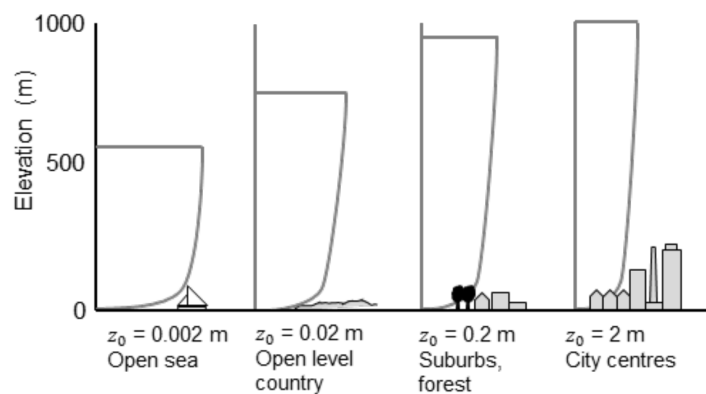


Figure 2.8: Mean wind profiles of different terrains (Abu-Zidan et al., 2022).

Table 4.1 — Terrain categories and terrain parameters

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

Figure 2.9: Terrain categories and terrain parameters, Eurocode.

2.5.2 Wind Effects on Buildings

Turbulent wind interacting with building surfaces can cause buffeting. Buffeting refers to irregular, random load variations on a structure and arises when turbulence creates uneven and continuously changing pressure distributions. These fluctuating pressures produce frequent, irregular vibrations and deformations in both the along-wind and across-wind directions (Mendis et al., 2007).

Vortex shedding occurs when wind flows past a non-streamlined body, creating alternating vortices in the wake behind the building. This phenomenon results in periodic transverse forces acting on the structure. When the vortex shedding frequency approaches the building's fundamental frequency, resonance may develop. To mitigate these responses, designers may implement aerodynamic modifications or damping systems (Abu-Zidan et al., 2022).

Slender structures are more susceptible to galloping and flutter, which are wind-induced aeroelastic instabilities. Galloping is a self-excited oscillation that occurs when aerodynamic forces increase with structural displacement. Flutter is a coupled bending–torsional instability that can develop at high wind speeds and may lead to sudden and severe failure (Mendis et al., 2007).

2.5.3 Wind-induced Responses

Wind flow around a building generates complex, unsteady patterns characterised by flow separation, vortex shedding, and wake formation. These effects create fluctuating wind pressures on the building, resulting in global aerodynamic loads as well as localised, varying forces on the façade. Under the combined effect of these fluctuating loads, the building responds through vibrations in the along-wind, across-wind, and torsional directions. These directions are illustrated in Figure 2.10. The along-wind response is mainly associated with turbulence in the approaching wind. The across-wind response is largely governed by vortex shedding and lateral lift forces. The torsional response arises from asymmetry in geometry and pressure distribution. The amplitude of

these oscillations depends on both the characteristics of the aerodynamic forces and the dynamic properties of the structure, which are evaluated in the third and fourth stages of the wind loading chain (Abu-Zidan et al., 2022).

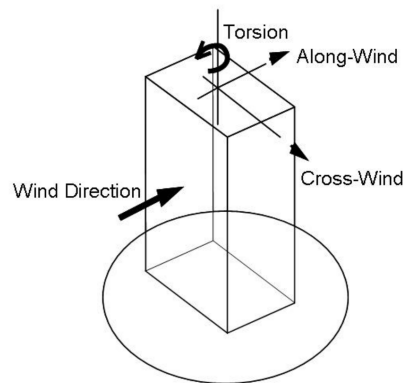


Figure 2.10: Wind response directions (Mendis et al., 2007).

2.5.3.1 Along-wind Response

The along-wind response of a building consists of a mean component and a fluctuating component. The mean component is caused by the average wind speed, while the fluctuating component is caused by turbulence in the approaching wind. Large-scale turbulence, which occurs less frequently and varies slowly, does not typically excite the structure dynamically. This is due to its low-frequency content relative to the natural frequencies of most buildings. Therefore, it can be treated as part of the mean wind. This contribution is represented as the background component, B , in the structural response spectrum.

Smaller-scale turbulence contains higher-frequency content that may approach the building's natural frequencies, leading to resonant dynamic responses that amplify the load effects. This contribution is represented by the resonant component, R , in the structural response spectrum. Design codes often use the gust-factor approach, in which the response is separated into mean and fluctuating parts.

The mean load is determined from mean wind speeds and aerodynamic coefficients, while the fluctuating load accounts for turbulence intensity, size effects, and dynamic amplification. This method provides an effective prediction of the along-wind response when the wind flow is not significantly influenced by nearby tall buildings or complex terrain (Abu-Zidan et al., 2022).

2.5.3.2 Across-wind Response

The governing phenomenon in the across-wind response of tall buildings is vortex shedding. Vortex shedding occurs when vortices form and are shed alternately from each side of a non-streamlined body. This generates fluctuating across-wind forces, as illustrated in Figure 2.11. These alternating forces can cause the building to sway perpendicular to the wind direction. If the vortex shedding frequency coincides with the building's natural frequency, resonance may occur, leading to large oscillations (Abu-Zidan et al., 2022).

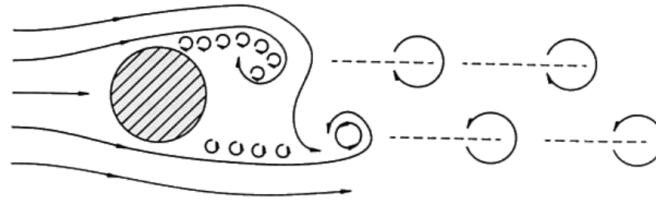


Figure 2.11: Vortex formation (Abu-Zidan et al., 2022).

When the building's motion becomes large enough to influence the vortex formation process itself, a phenomenon known as lock-in may occur. The structural swaying can then intensify as the shedding frequency synchronises with the building's natural frequency over a wider range of wind speeds. This sustained resonance can lead to severe serviceability problems or even structural issues (Abu-Zidan et al., 2022).

Another contributor to across-wind response is incident turbulence. This occurs when turbulence in the approaching wind directly generates across-wind forces on the structure. The magnitude of this effect depends strongly on the building shape and its susceptibility to generating lift due to changes in wind speed. However, this phenomenon is generally more critical in bridge design than in the across-wind response of tall buildings (Abu-Zidan et al., 2022).

2.5.3.3 Torsional Response

When subjected to wind forces, a torsional response may occur, referring to a twisting motion about the building's vertical axis. Unlike the two translational responses, torsion involves rotational motion, which can cause additional occupant discomfort due to a perceived rotating horizon. This response arises when lateral wind forces induce a net moment about the building's central vertical axis. Wind-induced torsion can lead to increased displacements and accelerations at the building edges and may also amplify the swaying of perimeter structural elements. A flow chart illustrating the causes of wind-induced torsional response is presented in Figure 2.12 (Singh et al., 2024).

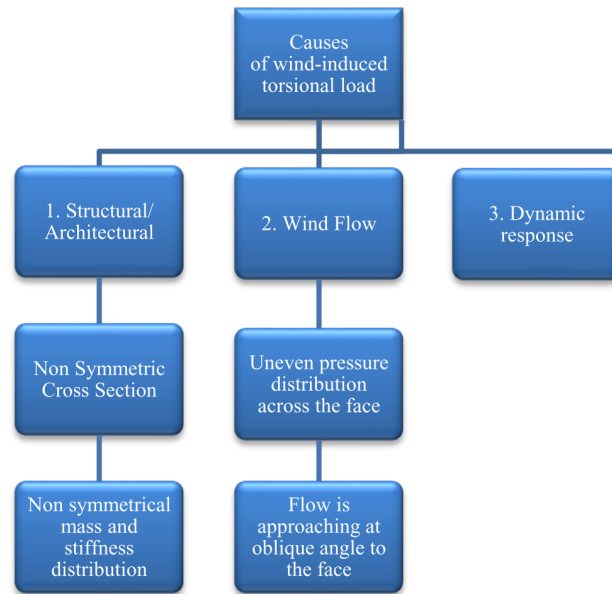


Figure 2.12: Causes of wind-induced torsion in tall buildings (Singh et al., 2024).

One factor that has a significant impact on torsional response is asymmetry in the building's geometry or structural properties. When the centre of mass and the centre of stiffness do not coincide, torsional motion is induced. This commonly occurs in buildings with irregular or non-symmetric plan shapes. However, even in geometrically symmetric buildings, torsion can develop because real wind flow is rarely uniform. Uneven pressure distributions around the building perimeter may arise from flow separation, turbulent gusts, and oblique wind directions, which can also induce twisting (Singh et al., 2024).

Torsional behaviour is further influenced by the dynamic interaction between the building's vibration characteristics and the wind. Buildings have natural vibration modes that include torsional modes. When fluctuating wind forces excite these modes, the torsional response can be significantly amplified. This shows that torsional motion is not only a static effect of wind pressure distribution but also a dynamic response governed by the building's natural frequencies and damping properties.

2.5.4 Human Perception and Comfort Considerations

Human perception of wind-induced motion is created through a combination of sensory systems. The sensory system includes the vestibular system in the inner ear, visual and auditory cues, and proprioceptive feedback provided by muscles and joints. Humans are most sensitive to frequencies ranging between 0.6 Hz and 1 Hz. Depending on the duration and intensity, wind-induced motions can range from being barely perceptible to causing various symptoms. The symptoms arising can include annoyance, nausea, motion sickness, distraction, and in severe cases vertigo (Johann et al., 2015).

To quantify comfort, different properties of motion can be assessed and measured. For ISO 10137, acceleration is the primary measure, as humans are most sensitive to it at low frequencies where the dynamic response of tall buildings occurs. The threshold for

perception varies across the spectrum, with the lowest tolerances around 0.1 Hz to 1 Hz. The waveform and duration of motion also influence comfort as well as the direction of vibration. Motion along different axes is perceived differently. Additionally, wind flow is often considered in terms of statistical return periods to assess likely exposure.

ISO 10137 provides a framework to assess wind-induced motion in terms of human comfort and serviceability. In the standard, peak horizontal acceleration limits A (in m/s^2) are defined as a function of the building's natural frequency f_0 (in Hz). Two separate thresholds are provided: one for residential buildings and one for office buildings. The diagram is displayed in Figure 2.13. The standard uses a one-year wind return period in the structural directions of the building, normally in along-wind and across-wind directions. Torsion should not exceed the basic evaluation curve in Figure 2.13. The acceleration in torsion stands for the equivalent translational acceleration defined as:

$$A_t = r \cdot A_\theta(t), \quad (2.1)$$

where:

r = distance from the centre of torsion to the objective point,

$A_\theta(t)$ = angular acceleration of the torsional vibration.

While ISO 10137 does not directly address health effects such as nausea, it establishes perceptibility thresholds that ensure general comfort. To provide a practical reference, a table from (Abu-Zidan et al., 2022) is used, as shown in Figure 2.14. This table presents typical human responses to various levels of building acceleration.

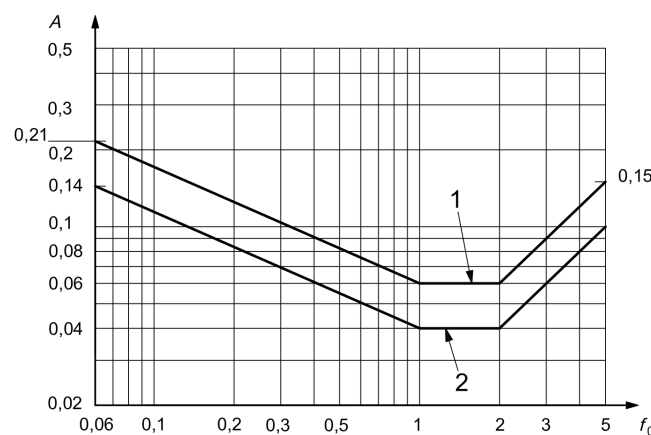


Figure 2.13: Evaluation curves for wind-induced vibrations in buildings in a horizontal (x, y) direction for a one-year return period (ISO 10137, 2008).

Level	Acceleration (m/s ²)	Effect
1	< 0.05	Humans cannot perceive motion
2	0.05 - 0.1	Sensitive people can perceive motion, and hanging objects may move slightly.
3	0.1 - 0.25	Majority of people will perceive motion. May affect desk work, and long-term exposure may produce motion sickness.
4	0.25 - 0.4	Desk work becomes difficult or almost impossible, but ambulation is still possible.
5	0.4 - 0.5	People strongly perceive motion and walking naturally is difficult. 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

Figure 2.14: Human perception levels to building motion (Abu-Zidan et al., 2022).

2.5.5 Wind Tunnel Tests

In complex scenarios, wind tunnel tests can simulate a more accurate context for the building. They can determine wind loads, pressure, and dynamic responses on tall buildings, especially when analytical methods and code equations are inadequate. Wind tunnel tests include scaled physical models placed in a boundary layer wind tunnel exposed to simulated atmospheric wind conditions from multiple directions. Surrounding buildings, wind directionality, and terrain effects can also be included in the model to reflect more realistic site conditions (Abu-Zidan et al., 2022).

The tests can be divided into two categories. The first is rigid model tests, which measure overall force and pressure time histories. The second is flexible model tests, which capture coupled interaction between wind forces and structural motion. Data on mean and fluctuating wind loads and dynamic responses can be extracted from these experiments, including acceleration and deflection measures. The results provide an essential basis for further design of strength, stability, and serviceability (Abu-Zidan et al., 2022).

2.6 Natural Frequencies of Structural Systems

In this section, the fundamentals of natural frequencies and their corresponding analytical equations are introduced. Two different theories are studied, and their appropriate

applications are described. The influence of geometry, boundary conditions, material properties, and cross-sectional shape on natural frequencies in practice is also discussed.

2.6.1 Bending Modes - Euler-Bernoulli Beam Theory

The Euler–Bernoulli beam theory is an approach for modelling the bending behaviour of slender structural members. It assumes that plane cross-sections remain plane and perpendicular to the neutral axis during bending. This implies that rotary inertia and transverse shear deformation are negligible. The Euler–Bernoulli theory is applicable to long, slender beams where shear effects are small compared with bending effects. The beam’s response to vibration is dominated by bending stiffness, and since shear deformation is ignored, the theory can overestimate stiffness and thus over predict natural frequencies for deep or thick beams (Rao, 2007).

The i -th bending mode frequency for a fixed–free boundary condition is given by (Blevins, 2016):

$$f_i = \frac{\lambda_i^2}{2\pi L^2} \sqrt{\frac{EI}{\rho A}} \quad (2.2)$$

where:

E = modulus of elasticity, [Pa],

I = moment of inertia, [m⁴],

ρ = material density, [kg/m³],

L = beam length, [m],

A = cross-sectional area, [m²],

$\lambda_i =$	i		λ_i
	1		1.875
	2		4.694
	3		7.855

2.6.2 Torsional Modes - St. Venant Theory

Saint-Venant torsion theory describes the torsional response of prismatic structural members subjected to an applied torque. The theory assumes that cross-sections behave as rigid planes and twist uniformly about the longitudinal axis. Warping effects due to the cross-sectional shape are not taken into account. Under these assumptions, the torsional rigidity of a section depends on its geometry and the material’s shear modulus, as torsion produces a shear stress distribution related to the rate of twist. This model is appropriate for solid or closed thin-walled sections subjected primarily to torque. It provides a basis for calculating torsional stiffness as well as torsional natural frequencies in vibration problems (Haukaas, n.d.).

The i -th bending mode frequency for a fixed–free boundary condition is given by (Blevins, 2016):

$$f_i = \frac{\lambda_i^2}{2\pi L^2} \sqrt{\frac{CG}{\rho I_p}} \quad (2.3)$$

where:

C = torsion constant of beam cross section, [m⁴],

G = shear modulus, [Pa],

ρ = material density, [kg/m³],

L = beam length, [m],

I_p = polar moment of inertia of cross section about torsion axis, [kg·m],

$$\lambda_i = \begin{array}{c|c} i & \lambda_i \\ \hline 1 & \frac{\pi}{2} \\ 2 & \frac{3\pi}{2} \\ 3 & \frac{5\pi}{2} \end{array}$$

The equations for C and I_p for a thin-walled rectangular cross-section are as follows:

$$C = \frac{1}{3} t^3 b + \frac{4A_m^2}{\oint_{\Gamma_m} \frac{1}{t} ds} \quad (2.4)$$

$$I_p = \frac{b_o h_o^3 - b_i h_i^3}{12} + \frac{b_o^3 h_o - b_i^3 h_i}{12} \quad (2.5)$$

where:

A_m = mean area of the cross section [m²],

t = wall thickness of the cross section [m],

b_o, h_o = outer width and height of the cross section [m],

b_i, h_i = inner width and height of the cross section [m],

2.7 Wind Actions

In this section, the methods for computing relevant wind actions governing the ULS and SLS design of a timber–concrete office tower are summarised, according to the current and upcoming Eurocodes. The limitations of the current code are also discussed.

2.7.1 Limitations of the Eurocode Wind Model

Wind actions on the building are analysed using the standards currently valid in Sweden: Eurocode SS-EN 1991-1-4:2005, with its national annex, as well as regulations according to BFS 2024:1. For wind actions not covered within these standards, the upcoming Eurocode SS-EN 1991-1-4:2026 is used.

For computing wind actions on the building, SS-EN 1991-1-4:2005 Section 1 limits the maximum height of the building to 200 m.

As the current regulations do not cover across-wind and torsional actions, the upcoming SS-EN 1991-1-4:2026 is used. This code specifies the following limitations for build-

ings analysed within the scope of Annex F: the width-to-depth ratio must fall within the following range:

$$3 \leq \frac{h}{\sqrt{b \cdot d}} < 6 \quad (2.6)$$

where:

b = width of the building;

d = depth of the building;

h = height of the building.

The code provides guidance on procedures to follow if the ratio exceeds these limits; however, no further guidance is given for floor plans with ratios below the specified range.

The procedure described in SS-EN 1991-1-4:2026, Annex F, may be applied when the building has a vertically uniform mass distribution, a linear vibration mode, and when the building dimensions, first fundamental frequency, and mean wind velocity satisfy the following conditions:

$$0.2 \leq \frac{d}{b} \leq 5 \quad (2.7)$$

$$\frac{v_m(h)}{n_{LM} \cdot \sqrt{b \cdot d}} \leq 10 \quad (2.8)$$

where:

$v_m(h)$ = is the mean wind velocity, evaluated at a height $z = h$;

n_{LM} = The first across-wind or the first torsional frequency.

2.7.2 Equivalent Wind Actions

To account for both the static and dynamic response of the building in the ULS and SLS verification, equivalent wind actions are determined according to the Eurocode, where the dynamic effects of wind loading are represented by equivalent static actions.

According to the second-generation Eurocode, equivalent wind actions are evaluated for along-wind, across-wind, and torsional response. Verification of the relevant load combinations is also required.

2.7.2.1 Along-Wind

Wind pressure consists of external and internal wind components. Only the external component influences the structure's global strength and stability, while the internal component is taken into account when analysing local structural members. The external wind pressure is determined based on shape factors, the building's dynamic response, and the velocity pressure.

According to SS-EN 1991-1-4:2005, the external wind pressure is calculated using the following expression:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z) \quad (2.9)$$

The structural factor $c_s c_d$ represents the combined influence of the structure's shape and its dynamic response to wind, modifying the wind pressure to account for turbulence and aerodynamic effects. The following expressions are according to SS-EN 1991, gen. 1/NA:2025 6.1(1) and 6.3.1.

$$c_s c_d = \frac{1 + 2k_p I_v(z) \sqrt{B^2 + R^2}}{1 + 2k_p I_v(z_s)} \quad (2.10)$$

$$k_p = \begin{cases} \sqrt{2 \ln(\nu T)} + \frac{0.6}{\sqrt{2 \ln(\nu T)}}, & \text{for dynamically sensitive structures} \\ 3.0, & \text{for static structures} \end{cases} \quad (2.11)$$

Where T is the return period for wind loads (600 seconds).

According to SS-EN 1991, gen. 1/NA:2025 6.3.1, the cross-up frequency ν defines the transition between background turbulence effects and resonant vibrations based on the structure's dynamics. It is calculated as follows:

$$\nu = n_{1,x} \frac{R}{\sqrt{B^2 + R^2}} \quad (2.12)$$

$$n_1 = \frac{46}{h} \quad (2.13)$$

The equations for background factor B^2 are covered by SS-EN 1991, gen. 1/NA:2025 6.3.1 and SS-EN 1991-1-4:2005 Annex F.

$$B^2 = \exp \left[-0.05 \left(\frac{h}{h_{\text{ref}}} \right) + \left(1 - \frac{b}{h} \right) \left(0.04 + 0.01 \left(\frac{h}{h_{\text{ref}}} \right) \right) \right] \quad (2.14)$$

where:

$$\begin{aligned} h &= \text{height of the building, [m]}, \\ h_{\text{ref}} &= 10 \text{ m, national reference height}, \\ b &= \text{width of the building} \end{aligned}$$

The resonance response factor R^2 allows for turbulence in resonance with the considered vibration mode of the structure. The expression is according to SS-EN 1991, gen. 1/NA:2025 6.3.1.

$$R^2 = \frac{2\pi \cdot F \cdot \Phi_b \cdot \Phi_h}{\delta_s + \delta_a} \quad (2.15)$$

The wind energy spectrum F describes the distribution of wind energy across different frequencies according to SS-EN 1991, gen. 1/NA:2025 6.3.1.

$$F = \frac{4 \cdot y_C}{(1 + 70.8 \cdot y_C^2)^{5/6}} \quad (2.16)$$

$$\gamma_C = \frac{150 \cdot n_{1,x}}{v_m(h)} \quad (2.17)$$

$$\Phi_h = \frac{1}{1 + 2 \cdot n_{1,x} \cdot \frac{h}{v_m(h)}} \quad (2.18)$$

$$\Phi_b = \frac{1}{1 + 3.2 \cdot n_{1,x} \cdot \frac{b}{v_m(h)}} \quad (2.19)$$

Structural damping δ_s is displayed in Table F.2 of SS-EN 1991-1-4:2005. The logarithmic decrement of aerodynamic damping δ_a for along-wind vibrations is estimated according to SS-EN 1991-1-4:2005 Annex F.

$$\delta_a = \frac{c_f \cdot \rho \cdot b \cdot v_m(z_s)}{2 \cdot n_1 \cdot m_e} \quad (2.20)$$

The equivalent mass per unit length of the fundamental mode is calculated according to SS-EN 1991-1-4:2005 Annex F.

$$m_e = \frac{\int_0^l m(s) \cdot \Phi_1^2(s) ds}{\int_0^l \Phi_1^2(s) ds} \quad (2.21)$$

$$\Phi_1(z) = \left(\frac{z}{h}\right)^\xi \quad (2.22)$$

The ξ depends on the structural system.

The force coefficient c_f is defined in SS-EN 1991-1-4:2005 section 7.

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda \quad (2.23)$$

The ψ_λ is defined by two factors λ and φ . The effective slenderness λ is given by SS-EN 1991-1-4:2005 Table 7.16 as presented below. The solidity ratio φ is given by the expression:

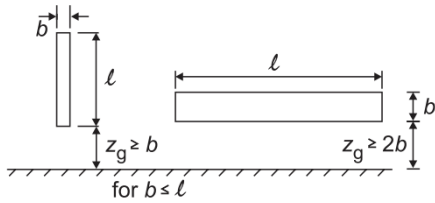
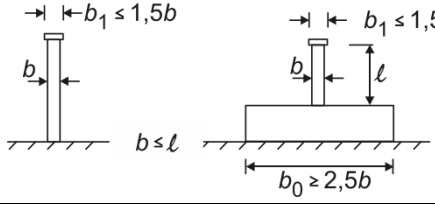
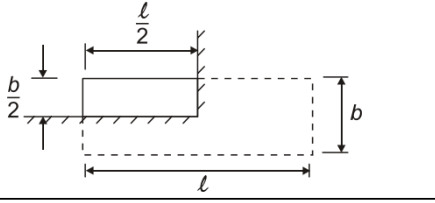
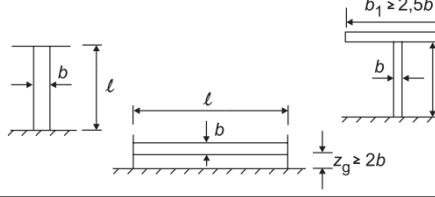
$$\varphi = \frac{A}{A_c} \quad (2.24)$$

where:

A = sum of the projected areas of the members,
 A_c = overall envelope area, $l \cdot b$

The ψ_λ can be retrieved from Figure 7.36 in SS-EN 1991-1-4:2005.

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

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

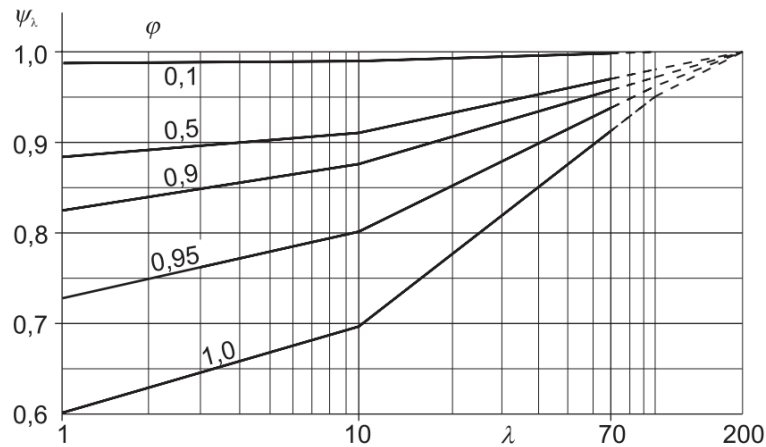


Figure 7.36 — Indicative values of the end-effect factor ψ_λ as a function of solidity ratio ϕ versus slenderness λ

The reduction factor ψ_r for a square cross-section with rounded corners can be estimated as illustrated in SS-EN 1991-1-4:2005 Figure 7.24.

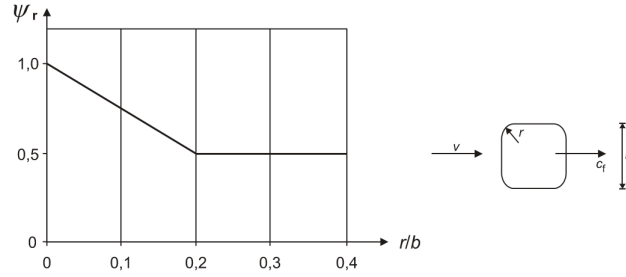


Figure 7.24 — Reduction factor ψ_r for a square cross-section with rounded corners

The force coefficient $c_{f,0}$ of rectangular sections with sharp corners and without free-end flow is given by SS-EN 1991-1-4:2005 Figure 7.23.

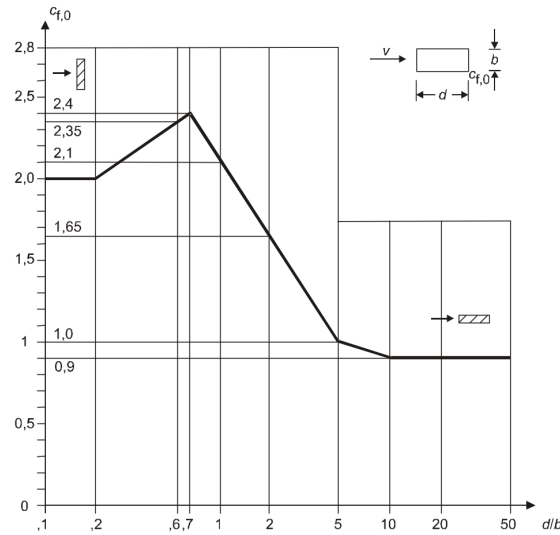


Figure 7.23 — Force coefficients $c_{f,0}$ of rectangular sections with sharp corners and without free end flow

The mean wind velocity $v_m(z)$ at height z is defined according to SS-EN 1991-1-4:2005 Section 4.

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b \quad (2.25)$$

$$z_s = 0.6 \cdot h \quad (2.26)$$

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} \quad (2.27)$$

where:

- c_{dir} = the directional factor,
- c_{season} = the season factor,
- $v_{b,0}$ = the fundamental value of basic wind velocity

The roughness factor $c_r(z)$ covers the influence of height and terrain on wind velocity.

$$c_r(z) = \begin{cases} k_r \cdot \ln\left(\frac{z}{z_0}\right), & z_{\min} \leq z \leq z_{\max} \\ c_r(z_{\min}), & z < z_{\min} \end{cases} \quad (2.28)$$

$$k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \quad (2.29)$$

The z_0 and $z_{\min}$ depend on the terrain type as seen in SS-EN 1991-1-4:2005, shown in Figure 2.9.

The orography factor $c_0(z)$ is assumed to be 1 for flat terrain. The detailed procedure to estimate $c_0(z)$ is provided in SS-EN 1991-1-4:2005 Annex A.

The turbulence intensity $I_v(z)$ includes the relative fluctuations in wind speed due to atmospheric turbulence at height z . The term depends on height and terrain roughness as expressed in BFS 2024:1.

$$I_v(z) = \frac{1}{c_0(z) \cdot \ln \frac{z}{z_0}} \quad (2.30)$$

The peak velocity pressure $q_p(z)$ at height z includes mean and short-term velocity fluctuations and is estimated according to EKS Article 7.

$$q_p(z) = [1 + 2 k_p I_v(z)] \left[k_r \ln\left(\frac{z}{z_0}\right) c_0(z) \right]^2 q_b \quad (2.31)$$

The basic velocity pressure q_b is calculated according to SS-EN 1991-1-4:2005 Section 4.

$$q_b = \frac{1}{2} \cdot \rho_{air} \cdot v_b^2 \quad (2.32)$$

2.7.2.2 Across-Wind

As the current Eurocode SS-EN 1991-1-4:2005 does not define a method for calculating across-wind actions, the upcoming code SS-EN 1991-1-4:2026 was used to estimate the equivalent across-wind static action per unit length acting on the building. All basic input data describing the building geometry and wind actions, including the mean velocity pressure q_m , were calculated according to the current Eurocode using Equation 2.32.

All following equations are taken from SS-EN 1991-1-4:2026, Annex F.

According to SS-EN 1991-1-4:2026, the equivalent across-wind static action per unit length $f_L(z)$ is calculated using the following expression:

$$f_L(z) = 3 \cdot q_m(h) \cdot c_L \cdot b \cdot \left(\frac{z}{h}\right) \cdot c_{dL} \quad (2.33)$$

where:

$q_m(h)$ = the mean velocity pressure at the top of the building, evaluated using mean velocity pressure and equation 2.32

The lateral force coefficient c_L , associated with the fluctuating overturning moment in the across-wind plane, is defined as:

$$c_L = -0.015 \cdot \left(\frac{d}{b}\right)^2 + 0.14 \cdot \frac{d}{b} + 0.03 \quad (2.34)$$

The across-wind dynamic factor c_{dL} , which accounts for across-wind resonant response and the across-wind peak factor, is determined as:

$$c_{dL} = g_L \cdot \sqrt{1 + R_L^2} \quad (2.35)$$

The resonant response factor R_L^2 in the across-wind direction, which accounts for the amplification effects of wind turbulence at the natural frequency of the across-wind vibration mode, is estimated according to:

$$R_L^2 = \frac{\pi^2 \cdot S_L}{2 \cdot \delta_L} \quad (2.36)$$

Parameter m , which considers the floor plan aspect ratio, is defined as:

$$m = \begin{cases} 1 & \text{for } d/b < 3 \\ 2 & \text{for } d/b \geq 3 \end{cases} \quad (2.37)$$

The normalized spectrum of the across-wind action S_L , which depends on the aspect ratio of the floor plan and the dimensionless coefficients k_{L1} , k_{L2} , β_{L1} , and β_{L2} , is evaluated using the following equations:

$$S_L = \sum_{j=1}^m \frac{4k_{Lj} \cdot (1 + 0.6 \cdot \beta_{Lj}) \cdot \beta_{Lj}}{\pi} \frac{\left(\frac{n_L}{n_{sj}}\right)^2}{\left[1 - \left(\frac{n_L}{n_{sj}}\right)^2\right]^2 + 4 \cdot \beta_{Lj}^2 \cdot \left(\frac{n_L}{n_{sj}}\right)^2} \quad (2.38)$$

$$k_{L1} = 0.85; \quad k_{L2} = 0.02 \quad (2.39)$$

$$\beta_{L1} = \frac{(d/b)^4 + 2.3 \cdot (d/b)^2}{[2.4 \cdot (d/b)^4 - 9.2 \cdot (d/b)^3 + 18 \cdot (d/b)^2 + 9.5 \cdot (d/b) - 0.15]} + \frac{0.12}{(d/b)} \quad (2.40)$$

$$\beta_{L2} = 0.28 \cdot \left(\frac{d}{b}\right)^{-0.34} \quad (2.41)$$

The main across-wind excitation frequency n_{s1} and the flow reattachment frequency n_{s2} are determined from:

$$n_{s1} = \frac{0.12}{\left\{1 + 0.38 \cdot \left(\frac{d}{b}\right)^2\right\}^{0.89}} \frac{v_m(h)}{b}; \quad n_{s2} = \frac{0.56}{\left(\frac{d}{b}\right)^{0.85}} \frac{v_m(h)}{b} \quad (2.42)$$

The across-wind peak factor g_L for the averaging time interval $t = 600$ s is calculated using the following equation:

$$g_L = \sqrt{2 \cdot \ln(n_L \cdot t)} + \frac{0.577}{\sqrt{2 \cdot \ln(n_L \cdot t)}} \geq 2.8 \quad (2.43)$$

2.7.2.3 Torsional

Similar to the across-wind actions, the current Eurocode SS-EN 1991-1-4:2005 does not define a method for calculating torsional actions. Therefore, the upcoming code SS-EN 1991-1-4:2026 was used to estimate the equivalent torsional static action per unit length acting on the building.

All following equations are taken from SS-EN 1991-1-4:2026, Annex F.

According to SS-EN 1991-1-4:2026, the equivalent torsional static action per unit length $m_M(z)$ is computed using the following expression:

$$m_M(z) = 1.8 \cdot q_m(h) \cdot c_M \cdot b \cdot d \cdot \left(\frac{z}{h}\right) \cdot c_{dM} \quad (2.44)$$

The moment coefficient associated with the fluctuating base torque c_M which takes into account aspect ratio of floor area is estimated according to:

$$c_M = \frac{b}{d} \cdot \left[0.0066 + 0.015 \cdot \left(\frac{d}{b}\right)^2 \right]^{0.78} \quad (2.45)$$

The torsional dynamic factor c_{dM} , which accounts for torsional resonant response and the torsional peak factor.

$$c_{dM} = g_M \cdot \sqrt{1 + R_M^2} \quad (2.46)$$

The torsional resonant response factor R_M^2 , which accounts for the amplification of the effects of wind turbulence at the natural frequency of the torsional vibration mode, is taken from:

$$R_M^2 = \frac{\pi^2 \cdot S_M}{2 \cdot \delta_M} \quad (2.47)$$

The reduced mean velocity v_m^* , evaluated at height $z = h$ and using the first fundamental torsional frequency n_M , is computed using the following expression:

$$v_m^* = \frac{v_m(h)}{n_M \cdot \sqrt{b \cdot d}} \quad (2.48)$$

Coefficient S_M depends on the reduced mean velocity, the coefficients S_{M1} and S_{M2} , and the dimensionless coefficients β_{M1} , β_{M2} , K_{M1} , and K_{M2} . It is estimated for three different reduced mean velocity ranges using the following expressions:

$$S_M = \begin{cases} S_{M1} \cdot \left(\frac{v_m^*}{4.5}\right)^{2 \cdot \beta_{M1}} & \text{if } v_m^* \leq 4.5 \\ S_{M1} \cdot \exp\left\{3.5 \cdot \ln\left(\frac{S_{M2}}{S_{M1}}\right) \cdot \ln\left(\frac{v_m^*}{4.5}\right)\right\} & \text{if } 4.5 < v_m^* < 6 \\ S_{M2} \cdot \left(\frac{v_m^*}{6}\right)^{2 \cdot \beta_{M2}} & \text{if } 6 \leq v_m^* \leq 10 \end{cases} \quad (2.49)$$

$$S_{M1} = \frac{0.14 \cdot K_{M1}^2 \cdot d \cdot (b^2 + d^2)^2}{\pi \cdot l^2 \cdot b^3} \cdot (4.5)^{2 \cdot \beta_{M1}} \quad (2.50)$$

$$S_{M2} = \frac{0.14 \cdot K_{M2}^2 \cdot d \cdot (b^2 + d^2)^2}{\pi \cdot l^2 \cdot b^3} \cdot (6)^{2 \cdot \beta_{M2}} \quad (2.51)$$

$$\beta_{M1} = \frac{(d/b) + 3.6}{(d/b)^2 - 5.1 \cdot (d/b) + 9.1} + \frac{0.14}{(d/b)} + 0.14 \quad (2.52)$$

$$\beta_{M2} = \frac{0.044 \cdot (d/b)^2 - 0.0064}{(d/b)^4 - 0.26 \cdot (d/b)^2 + 0.1} + 0.2 \quad (2.53)$$

$$K_{M1} = \frac{-1.1 \cdot (d/b) + 0.97}{(d/b)^2 + 0.85 \cdot (d/b) + 3.3} + 0.17 \quad (2.54)$$

$$K_{M2} = \frac{0.077 \cdot (d/b) - 0.16}{(d/b)^2 - 0.96 \cdot (d/b) + 0.42} + \frac{0.35}{(d/b)} + 0.095 \quad (2.55)$$

The torsional peak factor g_M for averaging time interval $t = 600$ s is taken from the following equation:

$$g_M = \sqrt{2 \cdot \ln(n_M \cdot t)} + \frac{0.577}{\sqrt{2 \cdot \ln(n_M \cdot t)}} \geq 2.8 \quad (2.56)$$

2.7.3 Wind-Induced Acceleration

Wind-induced accelerations are limited by SS-ISO 10137:2008, which specifies comfort criteria by defining maximum allowable accelerations for occupants as seen in Figure 2.13. These limits apply to peak accelerations and are based on wind loads corresponding to a one-year return period.

In SS-EN 1991-1-4:2005, the basic wind speed v_b is defined for a 50-year return period. To convert the peak acceleration to a one-year return period, the following expression is used. Since the relationship between the basic wind speed v_b and acceleration is non-linear for return periods below two years, a two-year return period is used in this equation. Using the basic wind speed for a two-year return period, $v_{b,2}$, the calculated peak acceleration corresponds to a one-year return period.

$$v_{T_a} = 0.75 v_{50} \sqrt{1 - 0.2 \ln \left[-\ln \left(1 - \frac{1}{T_a} \right) \right]} \quad (2.57)$$

2.7.3.1 Along-Wind

Along-wind acceleration is caused by wind turbulence and is computed in the direction of the wind flow from the wind actions acting on the building.

The maximum along-wind acceleration $\ddot{X}_{\max}(z)$ is defined in SS-EN 1991-1-4:2005/NA:2025 (NA.5.8) as follows.

$$\ddot{X}_{\max}(z) = k_p \sigma_{\ddot{x}}(z) \quad (2.58)$$

The standard deviation of along-wind acceleration $\sigma_{\ddot{x}}$ is estimated according to:

$$\sigma_{\ddot{x}}(z) = \frac{3 \cdot I_v(h) \cdot R \cdot q_m(h) \cdot b \cdot c_f \cdot \phi_{1,x}(z)}{m} \quad (2.59)$$

2.7.3.2 Across-Wind

Across-wind acceleration is caused by wind turbulence as well as vortex shedding in the wake of the building. The procedure presented in this section follows SS-EN 1991-1-4:2026, Annex F, and applies to equivalent static wind loads considering across-wind forced acceleration at a design wind speed lower than the non-dimensional critical wind speed for vortex-induced vibration or aeroelastic instability. If the design wind speed exceeds the critical wind speed, the procedure for across-wind dynamic and aeroelastic response of slender structures defined in Annex G must also be considered.

The peak value of the across-wind acceleration a_{pL} at height z of the building is estimated using the following expression:

$$a_{pL}(z) = g_L \cdot \sigma_{aL}(z) \quad (2.60)$$

The standard deviation of the across-wind acceleration σ_{aL} at height z can be computed as follows.

$$\sigma_{aL}(z) = \frac{0.5 \cdot \rho \cdot v_m^2(h) \cdot b \cdot h}{m_L} c_L \cdot R_L \cdot \Phi_L(h) \cdot \Phi_L(z) \quad (2.61)$$

Where the generalised mass of the building m_L in the first across-wind mode is estimated from.

$$m_L = \int_0^h m(z) \cdot \Phi_L^2(z) dz \quad (2.62)$$

2.7.3.3 Torsional

Torsional vibration is initiated by an asymmetric wind pressure distribution on the windward, side, and leeward faces of the building. It is caused by both wind turbulence and vortex shedding in the building wake. The procedure presented in this section follows SS-EN 1991-1-4:2026, Annex F.

The peak value of the torsional acceleration a_{pM} at height z of the building is computed using the following expression:

$$a_{pM}(z) = g_M \cdot \sigma_{aM}(z) \quad (2.63)$$

The standard deviation of the angular acceleration σ_{aM} at height z :

$$\sigma_{aM}(z) = \frac{0.3 \cdot \rho \cdot v_m^2(h) \cdot b \cdot d \cdot h}{I_M} C_M \cdot R_M \cdot \Phi_M(h) \cdot \Phi_M(z) \quad (2.64)$$

Where the generalised polar mass moment of inertia I_M of the building in the first torsional mode can be computed as follows.

$$I_M = \int_0^h I(z) \cdot \Phi_M^2(z) \cdot dz \quad (2.65)$$

2.7.4 Combination of Wind Actions

Wind actions and induced accelerations defined in the previous sections are evaluated separately; however, in reality, the along-wind, across-wind, and torsional responses interact with the building simultaneously. Nevertheless, the maximum values of the different responses do not occur at the same time. This section presents the procedure for combining wind actions.

As the current Eurocode does not define a procedure for wind action combinations, the upcoming code SS-EN 1991-1-4:2026 was used. It defines three combination rules, which are summarised in Table 2.1. The code notes that these combinations are primarily intended for wind actions and effects proportional to wind actions, while combinations of accelerations and acceleration-dependent effects may be slightly non-conservative.

The along-wind gust factor G_D can be computed as follows, using the peak factor for turbulent wind $g_u = 2.9$.

$$G_D = c_{sd} \cdot (1 + g_u \cdot I_u(z))^2 \quad (2.66)$$

Combination coefficients of across-wind and torsional actions are estimated from Table 2.2, where n_1 is the lower of the across-wind and torsional fundamental frequencies.

The ratio between the across-wind and torsional frequencies f_{LM} is estimated according to:

Table 2.1: Table F.3 — Combination rules for action effects for each wind direction

Combination	Along-wind	Across-wind	Torsional
1	F_w	$0.4F_L$	$0.4m_M$
2	$F_w \cdot \left(0.4 + \frac{0.6}{G_D}\right)$	F_L	$\gamma_{LM}m_M$
3	$F_w \cdot \left(0.4 + \frac{0.6}{G_D}\right)$	$\gamma_{LM}F_L$	m_M

Table 2.2: Table F.4 — Combination coefficients of L and M

d/b	$n_1b/v_m(h)$	γ_{LM}		
		$f_{LM} = 1.0$	$f_{LM} = 1.1$	$f_{LM} = 1.4$
≤ 0.5	≤ 0.1	0.95	0.61	0.55
	0.2	0.55	0.67	0.61
	0.3	0.55	0.73	0.67
	0.6	0.79	0.79	0.79
	≥ 1	0.84	0.84	0.84
1	≤ 0.1	0.84	0.55	0.55
	0.2	0.61	0.55	0.55
	0.3	0.55	0.55	0.55
	0.6	0.67	0.67	0.67
	≥ 1	0.73	0.73	0.73
≥ 2	≤ 0.05	0.79	0.55	0.55
	≥ 0.1	0.55	0.55	0.55

$$f_{LM} = \begin{cases} n_L/n_M & \text{for } n_L \geq n_M \\ n_M/n_L & \text{for } n_L < n_M \end{cases} \quad (2.67)$$

When combining the wind actions, it is important to account for the direction of each action, as the along-wind, across-wind, and torsional responses act in different directions.

2.7.5 Across-wind and Torsional Response in AIJ, and ISO Standards

As the upcoming Eurocode contains several limitations, the possibilities for calculating across-wind and torsional wind-induced actions and accelerations outside the scope of SS-EN 1991-1-4:2026 were investigated through comparison with the recommendations of the Architectural Institute of Japan (AIJ) and the International Standard ISO 4354, Wind actions on structures.

2.7.5.1 AIJ

Compared with SS-EN 1991-1-4:2026, the AIJ recommendations have a wider application scope for lower buildings, as across-wind and torsional vibrations are limited only by a maximum limit for the ratio, as shown in the following equation. In addition, there are no restrictions requiring uniform mass distribution or linear vibration modes.

$$\frac{H}{\sqrt{B D}} \leq 6 \quad (2.68)$$

Wind actions The procedures for calculating across-wind and torsional actions are similar to those in SS-EN 1991-1-4:2026; however, the AIJ equations also include correction coefficients accounting for vibration mode shapes. The relevant equations are presented below.

$$W_L = 3 q_H C'_L A \frac{Z}{H} g_L \sqrt{1 + \phi_L^2} R_L \quad (2.69)$$

where:

W_L = across-wind load at height Z , [N]

q_H = the design velocity pressure at the reference height, [N/m²],

C'_L = rms overturning moment coefficient

A = projected area at height Z , [m²]

Z = height, [m]

H = reference height (roof height), [m]

g_L = peak factor for response acceleration in the across-wind direction

ϕ_L = correction coefficient for vibration mode

R_L = resonance factor for across-wind vibration

$$W_T = 1.8 q_H C'_T A B \frac{Z}{H} g_T \sqrt{1 + \phi_T^2} R_T \quad (2.70)$$

where:

W_T = torsional wind load at height Z , [N]

C'_T = rms torsional moment coefficient

B = projected breadth of building, [m]

g_T = peak factor for response acceleration in the torsional direction

R_T = resonance factor for torsional vibration

Accelerations Across-wind and torsional accelerations are calculated using the following expressions. Although the equations differ from those in the upcoming Eurocode, they account for similar physical variables. To determine across-wind acceleration, the generalised mass of the building for across-wind vibration must first be evaluated, while torsional acceleration requires calculation of the generalised mass moment of inertia. Since the AIJ procedure includes mode shape correction factors for the generalised wind force, it can be applied to a wider range of building configurations.

$$a_{L\max} = \frac{q_H g_{aL} B H C'_L \lambda \sqrt{R_L}}{M_L} \quad (2.71)$$

where:

- $a_{L\max}$ = maximum across-wind acceleration at the top of the building, [m/s²],
- g_{aL} = peak factor for response accelerations in the across-wind direction
- λ = mode correction factor of general wind force,
- M_L = generalised mass of the building for across-wind vibration, [kg].

$$a_{T\max} = \frac{0.6 q_H g_{aT} B^2 H C'_T \lambda \sqrt{R_T}}{I_T} \quad (2.72)$$

where:

- $a_{T\max}$ = maximum torsional acceleration at the top of the building, [rad/s²]
- g_{aT} = peak factor for response accelerations in the torsional direction
- I_T = generalised inertia moment of the building for torsional vibration, [kgm²],

Applicability of AIJ for European Wind Conditions While the AIJ recommendations can be applied to a broader range of buildings, there are several limitations regarding their use outside Japan.

First, the acceleration calculations are based on the 10-minute mean wind speed with a one-year recurrence period measured 10 m above ground level, which is defined using maps provided in the AIJ recommendations. Consequently, the procedure is tailored specifically to Japan and its climate model, with no guidance on how to derive equivalent values from the fundamental wind velocity or other parameters used in the Eurocode wind model.

In addition, differences in terrain category definitions and other wind model assumptions may affect the consistency of results obtained using the Japanese recommendations. Therefore, although the overall procedure for calculating wind actions is similar to that of the Eurocode and has a wider application scope, no guidance is provided on how to convert input parameters from countries other than Japan into equivalent AIJ values.

2.7.5.2 ISO

Although the ISO standard has limitations similar to those of the upcoming Eurocode regarding building height and floor-plan aspect ratio, it does not require a uniform mass distribution for across-wind actions. For torsional actions, however, the mass distribution along the vertical axis is assumed to be approximately uniform. In addition, there are no requirements for vibration mode shapes to be linear. The standard also provides guidance on more advanced approaches, not included in the standard, for cases where more accurate estimations are required, including the use of archives of wind tunnel test data.

Wind actions While the equations for wind actions are similar to those in the upcoming Eurocode, they also include mode shape correction factors, allowing their application to non-linear vibration modes. The relevant equations for calculating equivalent static wind loads are presented below.

$$F_L = q_{\text{site, m}} (C_{\text{Fm}} C_{\text{dyn, m}})_L b \quad (2.73)$$

$$(C_{\text{Fm}} C_{\text{dyn, m}})_L = 3C'_L \left(\frac{z}{h}\right)^k g_L \sqrt{1 + R_L^2} \quad (2.74)$$

where:

F_L = crosswind equivalent static load per unit height at height z

$q_{\text{site, m}}$ = site mean velocity pressure

b = breadth of the structure

h = reference height (mean roof height)

z = height

C'_L = standard deviation of the crosswind base bending moment coefficient

g_L = peak factor of the crosswind base bending moment

ν = cycling rate of the vibration (first mode frequency of crosswind vibration)

T = averaging time for the site mean velocity pressure

R_L = resonance factor for the crosswind base bending moment

K = mode shape correction factor

$$M_T = q_{\text{site, m}} (C_{\text{Fm}} C_{\text{dyn, m}})_T b^2 \quad (2.75)$$

$$(C_{\text{Fm}} C_{\text{dyn, m}})_T = 3C'_T \left(\frac{z}{h}\right)^k g_T \sqrt{1 + R_T^2} \quad (2.76)$$

where:

M_T = torsional equivalent static moment per unit height at height z

$q_{\text{site, m}}$ = site mean velocity pressure

C'_T = standard deviation of the torsional moment coefficient at the base

g_T = peak factor of the torsional moment at the base

ν = cycling rate of the vibration (lowest natural frequency of torsional vibration)

R_T = resonance factor for the torsional base moment

K = mode shape correction factor

Accelerations In contrast to the upcoming Eurocode and AIJ recommendations, the ISO standard uses a different approach for calculating accelerations, based on peak deflection rather than generalised mass or mass moment of inertia. The equations for calculating across-wind and torsional accelerations according to ISO 4354 are listed below.

$$\ddot{x}_L = (2\pi f_L)^2 \frac{R_L}{\sqrt{1 + R_L^2}} x_L \quad (2.77)$$

$$\ddot{\theta}_T = (2\pi f_T)^2 \frac{R_T}{\sqrt{1 + R_T^2}} \theta_T \quad (2.78)$$

where:

$\ddot{x}_L$ = peak crosswind acceleration

$\ddot{\theta}_T$ = peak angular acceleration

x_L = peak crosswind deflection obtained from the crosswind equivalent static load

θ_T = peak angular deflection obtained from the torsional equivalent static wind load

Applicability of ISO 4354 for European Wind Conditions ISO 4354 can be applied to a wider range of buildings, since it allows non-linear vibration modes and is not restricted to uniform vertical mass distributions. However, similarly to the AIJ recommendations, one of the main challenges is aligning Eurocode-based input data with the ISO framework. Although the ISO standard is formulated in a universal manner and defines approaches based on either mean wind velocity averaged over 10 minutes or peak gust velocity averaged over 3 seconds, making it applicable worldwide, it lacks direct guidance on how these parameters should be related to Eurocode wind variables. In addition, several variables are defined in a general manner, and some equations are not sufficiently explained, making independent application of the standard difficult without complementary literature.

2.8 Global Deflection

The global horizontal deflections are determined in accordance with SS-EN 1990-1:2023+A1:2026 A.1.8.2.3. The horizontal displacements should be calculated using the parameters shown in Figure 2.15. Where:

u = the overall horizontal displacement over the building height h ,

u_i = relative horizontal displacement over a storey height h_i

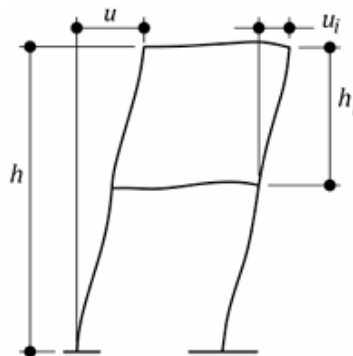


Figure 2.15: Definition of horizontal displacements.

According to Table A.1.11 in SS-EN 1990-1:2023+A1:2026, the suggested maximum horizontal displacement for a multi-storey building with the characteristic load combination is calculated as:

$$u \leq \frac{h}{500} \quad (2.79)$$

3 Methodology

This chapter presents the methodology adopted in this thesis. The methodological procedure, verification of the finite element software, building geometry, applied loads, member sizing, damping assumptions, and finite element modelling approach are described. In addition, the parametric study and the investigated structural modifications are presented. The methodology was developed to evaluate the dynamic response and structural performance of a tall hybrid timber building subjected to wind loading.

3.1 Analysis Procedure

The analysis procedure began with the verification of the FEM-Design model, followed by a convergence study to ensure reliable FE-analysis results. Thereafter, the wind calculation model and the reference model were developed. Based on the FE-analysis, the fundamental frequencies, equivalent masses, and generalised polar mass moments of inertia were extracted for each building configuration. Using these properties together with the wind pressure profiles, the accelerations for each wind mode, as well as the combined accelerations, were calculated manually. The resulting accelerations were then evaluated against the comfort criteria specified in ISO 10137, displayed in Figure 2.13.

The parametric study was subsequently carried out by varying individual parameters for several building heights. The influence of each parameter on the dynamic behaviour of the structure was assessed through SLS analysis and compared against the ISO comfort limits. Based on the results, the most effective parameters for improving the structural response were identified and combined to form an optimised model.

Finally, both SLS and ULS analyses were performed for the optimised model. Vertical loads and dynamically amplified wind loads were applied in FEM-Design for structural verification and member sizing. The dynamic amplification factors were determined using the fundamental frequencies and equivalent masses obtained from the previous analyses. The resulting structural stresses and deflections were evaluated against the Eurocode ULS and SLS requirements, as well as the ISO comfort limits, to ensure that the structural behaviour remained within acceptable limits.

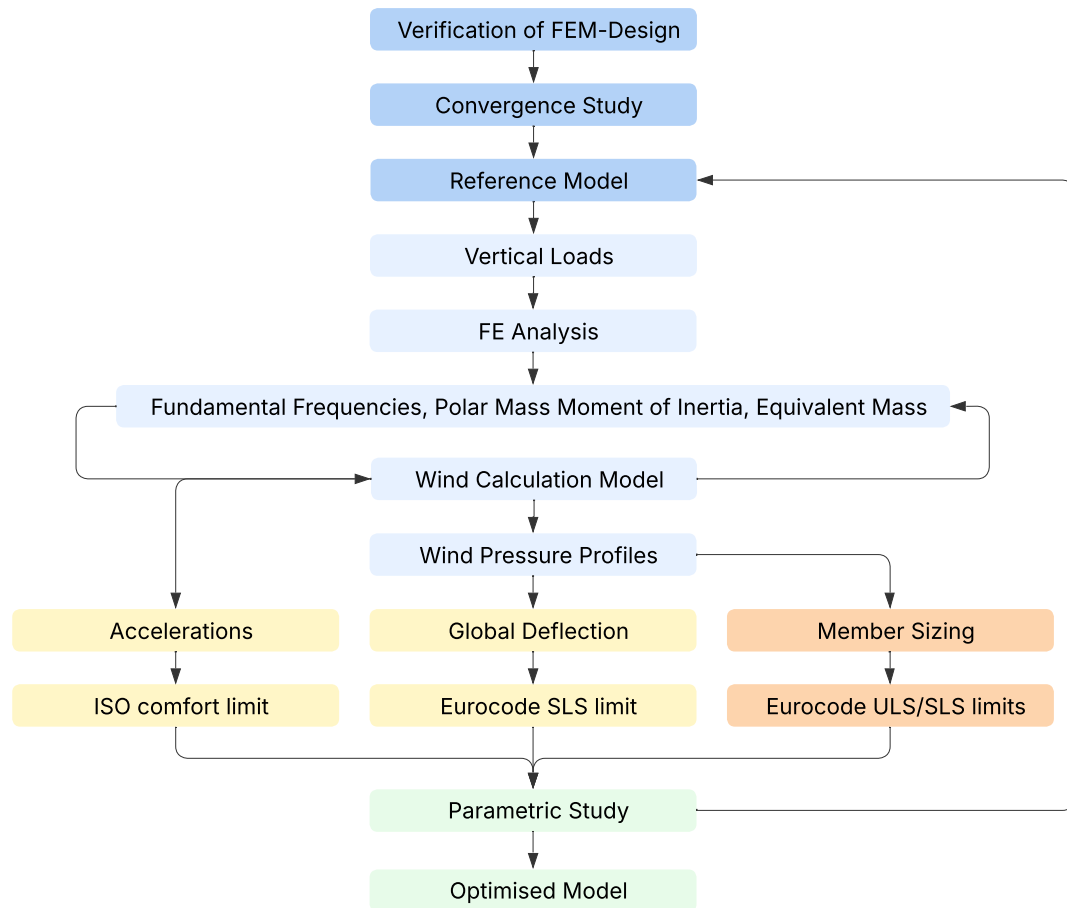


Figure 3.1: Flow chart summarising the methodology procedure.

3.2 Use of Artificial Intelligence

Artificial Intelligence (AI) was primarily used as a language-support tool during the preparation of this thesis. It was utilized throughout all chapters for spelling, grammar, and readability improvements. AI was also used to compress text volume in Chapter 2 and 5.

AI was also used to support Python scripting, including code review, suggestions for improving computational efficiency, and assistance with developing scripts for plotting results. Furthermore, it was used to support the presentation of results in Overleaf, including the creation of tables and the summarization of notation. In Section 4.4 and 4.5, AI was used to assist in summarizing trends observed in the results.

All AI-generated suggestions were critically reviewed and revised before inclusion in the thesis. The authors take full responsibility for the content of the thesis.

3.3 Verification of the Finite Element Software

This section presents the methodology used to verify the finite element model and its structural behaviour under both ultimate and serviceability limit states. The verification

is carried out through a series of benchmark comparisons between analytical calculations and FEM results for individual structural elements, global deflections, torsional response, and dynamic properties. Together, these checks establish the reliability and accuracy of the modelling approach used in the subsequent analyses.

3.3.1 Verification of Structural Elements in ULS and SLS

To verify the behaviour in both ULS and SLS, a single beam element and a single column element were modelled in FEM-Design. Considering the different cross-sectional categories, the largest cross-section, 760×810 mm, was assigned to the column element. For the beam element, the cross-section 280×495 mm was used. No discrepancies were observed between the analytical results and the FEM-Design results for a single structural element, as presented in Table 3.1. SS-EN 1995-1-1:2025 was used to conduct the verifications. Full verification of the beams and columns is presented in Appendix A.1.

Table 3.1: Comparison of utilisation for the column and the beam between analytical methods and FEM Design.

Structural Type	Check	Analytical	FEM Design
Beam GL30c	Bending	60 %	60 %
	Lateral torsional buckling	60 %	60 %
	Shear	81 %	81 %
	Deflection	10.37 mm	10.37 mm
Column GL30c	Axial compression and bending	63 %	63 %
	Flexural buckling	79 %	79 %

3.3.2 Verification of Lateral Deflections

To verify FEM-Design at the global level, a lateral deflection check was conducted for a simplified model that imitates a concrete core.

For the verification, two structural elements were used: a solid rectangular concrete column with the same dimensions as the elevator shaft ($19.2 \text{ m} \times 4.8 \text{ m}$) and a hollow rectangular cross-section with the same dimensions and a wall thickness of 0.25 m. Both elements were modelled with a height of 100 m, and loads were applied based on a parabolic wind profile computed for along-wind loading. The wind profile was discretised into twenty trapezoidal segments, each 5 m in height. In the FEM model, the solid section was represented by a beam element, whereas the hollow section was modelled using shell elements. In the hand calculations, shear correction coefficients according to Timoshenko beam theory were applied. (Cowper, 1966) The results are summarised in Table 3.2.

Table 3.2: Lateral deflection verification.

Element	FEM [mm]	Analytical [mm]	Rel. Diff. [%]
Solid rectangular column	97.65	95.42	2.29
Hollow rectangular column	301.28	292.98	2.76

3.3.3 Verification of Dynamic Properties

To verify the dynamic behaviour and properties of the FE model, three different structural elements were modelled. Their eigenfrequencies obtained from FEM-Design were checked against analytical formulas. Full verification of the beams and columns is presented in Appendix A.3. The structural elements increased in complexity, and the geometries are presented in Table 3.3. The first bending and torsional modes for each element are presented in Table 3.4.

Table 3.3: Simplified structural models used in eigenfrequency verification.

Structural Type	Size [mm]	Height [m]	Material	Element Type	Mesh Size [m]
Glulam Column	270×140	10.0	GL30c	Beam	0.2
Steel Tube Profile	450×250	10.0	S355	Shell	0.1
Concrete Core	19 200×4 800	100.0	C30/37	Shell	1

Table 3.4: Comparison of the first bending mode and the first torsional mode using analytical and numerical methods for the glulam column, steel tube, and concrete shaft.

Mode	FEM Design [Hz]	Analytical [Hz]	Relative Difference [%]
Glulam Column			
Bending	1.201	1.244	3.42
Steel Tube			
Bending	2.964	3.163	6.29
Torsion	62.220	63.817	2.50
Concrete Shaft			
Bending	0.433	0.453	4.51
Torsion	3.264	3.242	0.64

3.3.4 Convergence Study

To evaluate mesh sensitivity, a convergence study was carried out for a 10-storey model with geometry similar to the final reference model discussed in the following section. The convergence study analysed six parameters, categorised into global- and local-scale parameters:

Global-Scale Parameters (SLS Behaviour)

- Lowest natural frequency

- Maximum building deflection

Local-Scale Parameters (ULS Behaviour)

- Maximum bending moment in the GL beam
- Maximum bending moment in the CLT panel
- Maximum compressive force in the GL column
- Maximum compressive force in the concrete shaft

For the local-scale study, one representative element of each line element type was selected, and the maximum value within the element was used in the convergence analysis. For the elevator shaft, a specific point unaffected by local stress concentrations was selected. For the global-scale parameters, the lowest natural frequency and the maximum deflection at the midpoint of the elevator shaft were evaluated.

Meshes with the following sizes were investigated: 2.0 m, 1.5 m, 1.0 m, 0.8 m, 0.7 m, 0.6 m, 0.5 m, 0.4 m, 0.3 m, and 0.2 m. Furthermore, a combined mesh solution was evaluated, utilising a finer mesh for the elevator shaft (s) and a coarser mesh for the rest of the model (m), with the following sizes: 0.9 m (m) + 0.3 m (s), 0.6 m (m) + 0.3 m (s), and 0.4 m (m) + 0.3 m (s). The results of the convergence study are summarised in Figure 3.2.

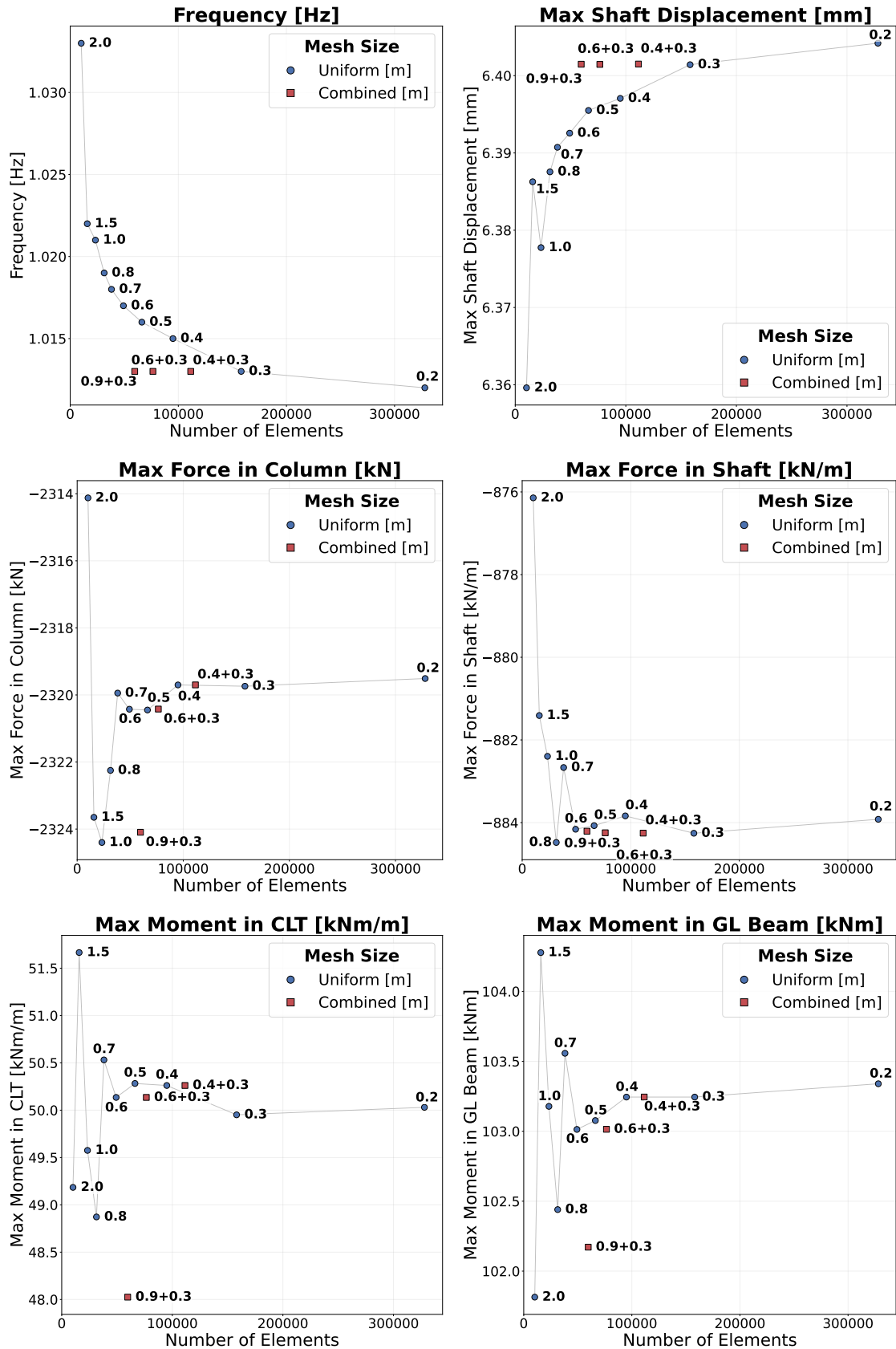


Figure 3.2: Convergence study results, variation in different parameters with increasing number of finite elements. For the combined values, the first number indicates the global mesh size of the model, while the second number indicates the refined mesh size used for the elevator shaft.

The results shown in Figure 3.2 indicate that, to achieve convergence for the global-scale parameters, a mesh size of 0.3 m or smaller was required for the elevator shaft. For the remaining structural elements, sufficiently accurate results were achieved with a mesh size of 0.6 m or smaller. To reduce computational cost while maintaining sufficient accuracy, a combined mesh strategy was adopted, using a mesh size of 0.3 m for the elevator shaft and 0.6 m for all other elements.

3.4 Building Geometry of Reference Building

The building analysed in this thesis is based, in both geometry and floor plan, on the multi-storey office building Habitat 7 in Gothenburg. The reference building is constructed with a concrete core combined with a glulam and CLT structural frame. This hybrid structural system forms the basis of the analysis presented in this thesis. The building has an overall plan dimension of 53.6 m $\times$ 17.2 m and a uniform storey height of 4 m throughout the entire building. The roof is assumed to be flat. The initial building configuration considered for analysis is presented in Figure 3.3.

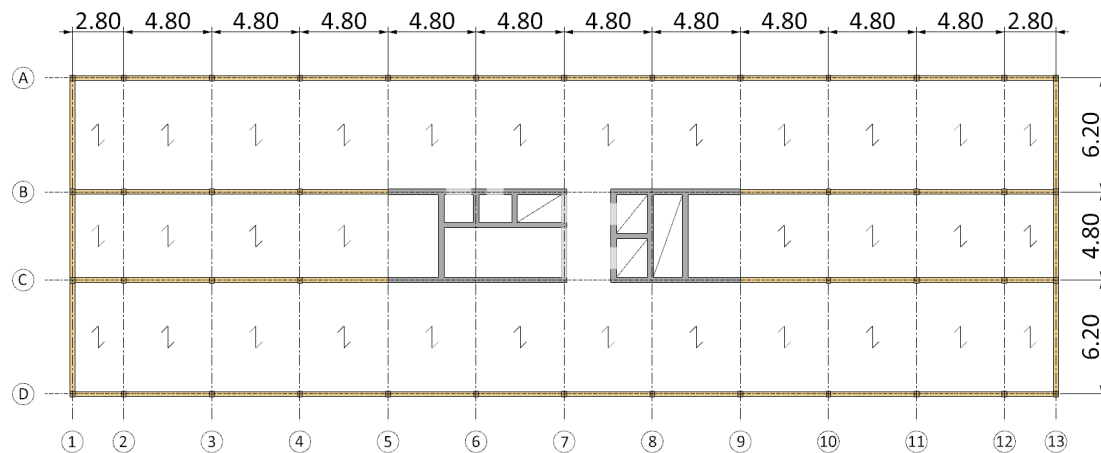


Figure 3.3: Schematic overview of floor plan.

3.5 Finite Element Model

The FE model of the reference building was modelled in the StruSoft software FEM-Design using the FEM-Design API toolbox for Grasshopper. The geometry was created parametrically to enable adjustments with regard to the parametric study later on. The cross-sections and material properties were assigned within the Grasshopper script, as well as the load application.

The columns and beams were defined as 1D members, while the floor slabs and the core walls were defined as 2D planar elements. All connections were assumed to be pinned. The connection between the core and foundation was assumed to be rigid, as well as the connections between the core walls (StruSoft, 2026).

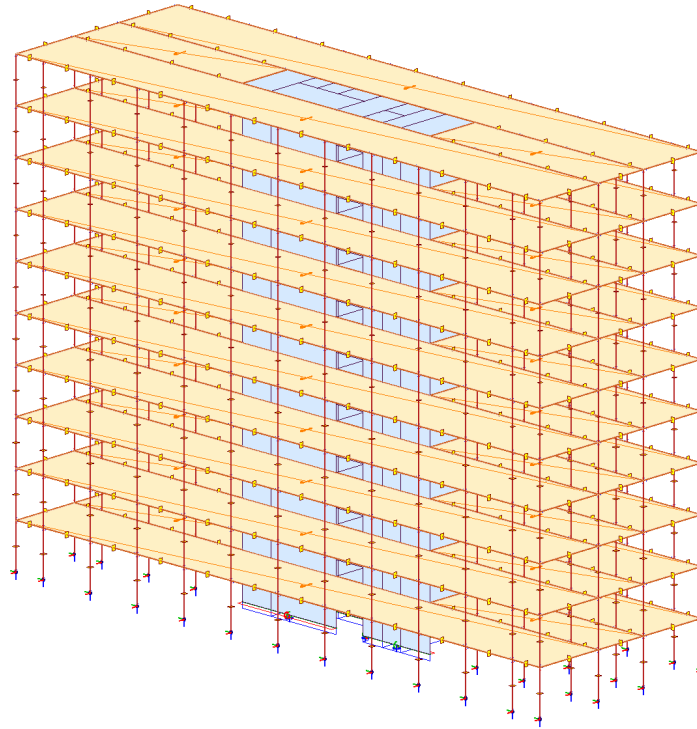


Figure 3.4: Reference building geometry in FEM-Design, 10-storey building.

3.6 Applied Loads

In this section, an overview of the applied loads is presented. The loads include both vertical and lateral loads, as well as the various load combinations.

3.6.1 Vertical Loads

The vertical loads considered are the self-weight of structural elements, the imposed live loads, and the snow load on the top floor. The imposed live loads and the snow load used are determined according to SS-EN 1991, gen. 1/NA:2025 and BFS 2024:6, for building category B. The self-weight dead loads are determined using SS-EN 1991, gen. 1/NA:2025, as well as product descriptions from EWP suppliers.

Self weight The self-weight of the building includes structural elements such as CLT slabs, glulam beams, and columns. The material densities are presented in Table 3.5. In addition to the self-weight of the structural elements, there are superimposed dead loads. The facade load is applied as a line load around the perimeter of the slabs on each storey. In a more realistic and detailed design, the slabs do not end at the column perimeter. The slabs are extended by 0.5 m in both the x- and y-direction around the entire perimeter. This additional self-weight is also applied as a line load around the building perimeter at each storey, and is thus not physically modelled in the FE model. The resulting dead loads are presented in Table 3.5.

Table 3.5: Self-weight of structural materials and superimposed dead load.

Component	Load
Concrete	2550 kg/m ³
CLT	420 kg/m ³
Glulam GL30c	430 kg/m ³
Facade	3 kN/m
Floor buildup excluding CLT	1.8 kN/m ²
Extended slab	1.5 kN/m

Imposed live load The imposed live load for the CLT slabs includes the office live load, category B, and the additional load due to partition walls. The imposed live load used in the core and stairs is determined as category C. The live loads used according to SS-EN 1991, gen. 1/NA:2025 are presented in Table 3.6.

Table 3.6: Characteristic live loads.

Considered area	Load
Office	2.5 kN/m ²
Partition walls	0.5 kN/m ²
Core/Stairs	3.0 kN/m ²

Snow load The characteristic snow load on a flat roof is determined according to BFS 2024:6. The equation and the relevant factors for this thesis are given below:

$$s = \mu_i C_e C_t s_k = 1.2 \text{ kN/m}^2 \quad (3.1)$$

where:

μ_i = snow shape coefficient,

s_k = characteristic value of snow on the ground,

C_e = exposure coefficient,

C_t = thermal coefficient

with the values:

$$\mu_i = 0.8 \text{ (roof slope of } 3^\circ),$$

$$s_k = 1.5 \text{ kN/m}^2,$$

$$C_e = 1.0, \quad C_t = 1.0$$

Vertical line load within core The concrete core consists of walls and floor slabs that are assumed to be rigidly connected, thereby contributing to the overall stiffness of the building. However, certain prefabricated floor slabs within the core geometry, such as staircase slabs, are connected using pinned connections and are therefore not intended to contribute to the stiffness of the core. Consequently, these slabs were not modelled as structural elements in the finite element model. Instead, their self-weight

was represented by equivalent line loads applied at the corresponding locations. The imposed loads acting on these slabs were similarly accounted for through additional line loads. See Figure 3.5 for an example of the line load application within the core geometry.

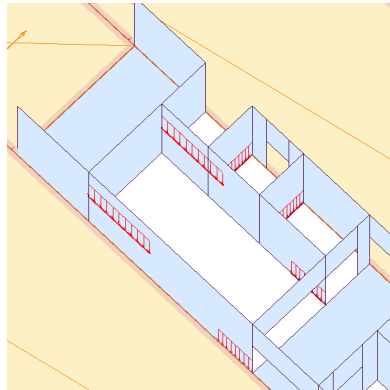


Figure 3.5: Core line loads equivalent to permanent and imposed surface load for one storey.

3.6.2 Lateral Loads

The lateral loads include three wind actions: along-wind, across-wind, and torsional actions. In the default interface of FEM-Design, applied loads can vary linearly along a specified height. In this project, the wind profiles were calculated according to the standards mentioned above and discretised into an arbitrary number of points, with linear variation between them, using Grasshopper and the FEM-Design API. Figure 3.6 displays an example of the resulting load application on a 10-storey building.

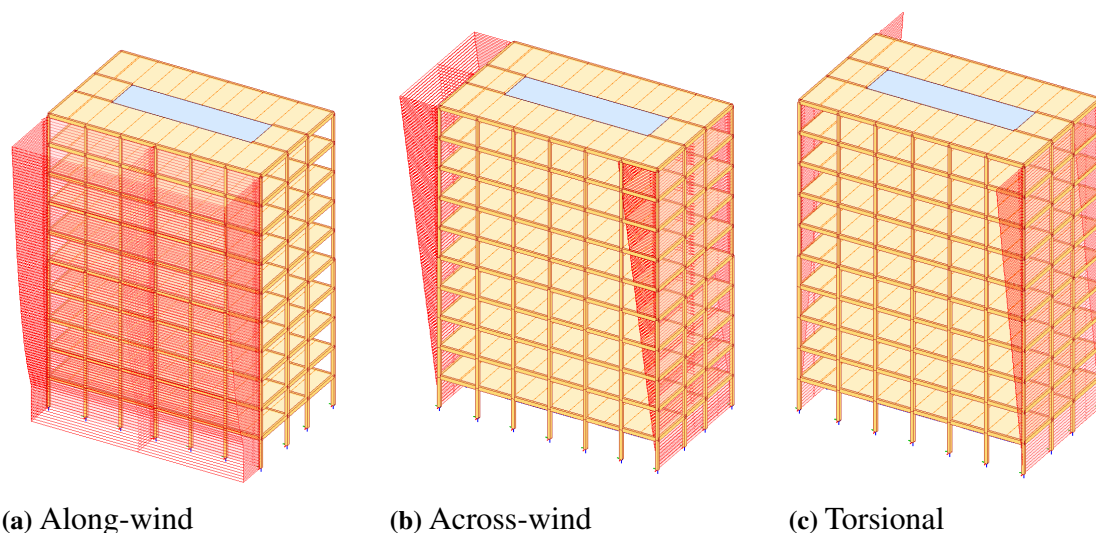


Figure 3.6: Overview of the three wind load applications in FEM-Design.

Thereafter, a non-structural cover that collects the wind loads and distributes them proportionally to the underlying structure was placed on the three sides. An example of

how the cover distributes the load to the structure is presented in Figure 3.7.

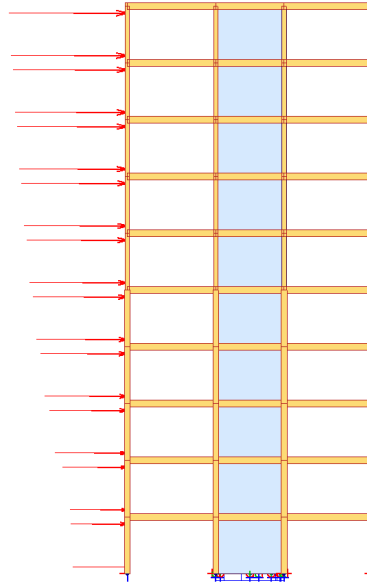


Figure 3.7: Example of converted cover load for along-wind action.

3.6.3 Design Factors and Load Combinations

The load combinations assessed were the ultimate limit state load combination and the characteristic serviceability limit state load combination. The ULS load combination is presented in Equation 3.2 in accordance with BFS 2024:6. The characteristic SLS load combination is presented in Equation 3.3 in accordance with SS-EN 1990:2002. The relevant Ψ -factors were determined in accordance with BFS 2024:6 Table 3:5.

$$\sum F_{ULS} = 1.2 \cdot \gamma_d \cdot \sum_i G_{k,i} + 1.5 \cdot \gamma_d \cdot Q_{k,1} + 1.5 \cdot \gamma_d \cdot \sum_{j>1} \psi_{0,j} Q_{k,j} \quad (3.2)$$

$$\sum F_{SLS} = \sum_i G_{k,i} + Q_{k,1} + \sum_{j>1} \psi_{0,j} Q_{k,j} \quad (3.3)$$

where:

$$\gamma_d = 1.00, \text{ safety class 3}$$

The primary evaluation in the ULS load combinations focused on the compressive stresses in the concrete core. The load combinations were divided into two scenarios: imposed load-leading combinations and wind load-leading combinations. Snow load was applied in both scenarios.

For the SLS load combinations, the main objectives were to evaluate the tensile stresses in the concrete core and the global deflection. The same two scenarios were considered: imposed load-leading combinations and wind load-leading combinations. However, the vertical variable loads were considered favourable when evaluating the wind load-leading SLS combinations. Therefore, snow load and imposed load were not applied in the wind load-leading SLS combinations.

3.7 Implementation of Wind Calculation Model

In this section, the implementation of the wind calculation model is described. The key computational procedures used in the wind load and dynamic analysis of the building model are presented. It covers how fundamental vibration modes are extracted and classified, how the building's polar mass moment of inertia and equivalent mass are derived from the FE model, how damping is defined, and how wind load profiles are determined. Together, these steps establish the input parameters and modelling assumptions required for the subsequent wind response analysis.

3.7.1 Fundamental Frequencies

The fundamental frequencies were extracted from the FE model and evaluated to determine whether the corresponding vibration modes were pure or hybrid using a Python script implemented in Grasshopper. The eigenfrequency results were extracted together with their modal mass participation factors in the global X-, Y-, and Z-directions. The modes were then classified according to their dominant translational response, where modes with significantly higher Y participation than X participation were classified as along-wind modes, while modes with higher X participation than Y participation were classified as across-wind modes. Modes with negligible participation in all translational directions were defined as torsional modes.

Thereafter, each mode was assessed with regard to modal purity. Translational modes with modal mass participation above approximately 60% were considered pure modes, whereas lower participation indicated hybrid behaviour. A torsional mode was considered pure when the translational participation in all directions was negligible. Otherwise, it was classified as a hybrid torsional mode. The participation threshold was determined through repeated observations of vibration mode shapes for different building models. See Table 3.7 for an overview of the classification of mode types.

Table 3.7: Classification criteria for vibration modes.

Classification	Criteria	Type
Along-wind mode	$Y_i > X_i$	Pure: $Y_i > 60\%$
		Hybrid: $Y_i < 60\%$
Across-wind mode	$X_i > Y_i$	Pure: $X_i > 60\%$
		Hybrid: $X_i < 60\%$
Torsional mode	$X_i = 0\%, Y_i = 0\%, Z_i = 0\%$	Pure: $X_i, Y_i, Z_i = 0\%$
		Hybrid: $X_i, Y_i, Z_i > 0\%$

3.7.2 Polar Mass Moment of Inertia

The polar mass moment of inertia of the building, $I(z)$, is required to calculate the generalised polar moment of inertia of the building, which is used in Equation 2.64 to compute the torsional acceleration (Young et al., 2012). In the wind calculation model, $I(z)$ was computed about the building's vertical z-axis through the building's

mass centre using discrete masses at each node in the FE model. A Python script was developed together with a component in the FEM-Design API, enabling access to a shared-connection .bsc file. From the .bsc file, the nodal coordinates and nodal masses could be extracted.

The polar mass moment of inertia contribution from each node was calculated using Equations 3.4 and 3.5.

$$I_{z,i} = m_i r_i^2 \quad (3.4)$$

$$r_i^2 = (x_i - x_{cog})^2 + (y_i - y_{cog})^2 \quad (3.5)$$

where:

$$\begin{aligned} x_i, y_i &= \text{coordinates of node } i \\ x_{cog}, y_{cog} &= \text{coordinates of the centre of gravity} \\ m_i &= \text{mass of node } i \end{aligned}$$

The total inertia is obtained by summing all nodal contributions, see Equation 3.6.

$$I_z = \sum_{i=1}^n I_{z,i} \quad (3.6)$$

3.7.3 Equivalent Mass

The equivalent mass in this thesis was computed using two different approaches: one for building configurations with uniform mass distribution and one for configurations with varying mass distribution. For the uniform mass distribution configurations, the equivalent mass per unit height was determined using mode-shape-weighted integration according to Equation 2.21. For the varying mass distribution configurations, the method presented in SS-EN 1991-1-4:2005 F.4 (2) was adopted, where the equivalent mass was approximated using the average mass over the upper third of the structure, which results in more conservative results.

The loads used to determine the distributed mass quantities were extracted from the FE model through exported reaction force data using the FEM-Design API. The permanent load was obtained by dividing the total vertical reaction force by the number of storeys. Similarly, the imposed load was extracted from the corresponding imposed load case in FEM-Design. These loads per storey were then combined and converted into mass per unit height.

For the varying mass distribution configurations, the load per storey was also included in the calculations. However, since the mass distribution varied along the building height due to the added concrete slabs, the total mass within the upper third of the structure was recalculated. The equivalent mass per unit height was thereafter determined by dividing the total mass within the upper third by the corresponding height.

3.7.4 Damping

The logarithmic decrement of damping, δ , for the fundamental bending mode may be estimated as the sum of the logarithmic decrement of structural damping, δ_s , and the logarithmic decrement of aerodynamic damping for the fundamental mode, δ_a . Equation 2.20 presents the calculation of δ_a , while δ_s is a tabulated value depending on the structural type. Figure 3.8 presents Table F.2 from SS-EN 1991-1-4:2005, which provides the approximated values of δ_s for various structural types. However, there are no values stated for a hybrid timber-concrete building. In this project, the concrete core provides the primary lateral load-resisting system and thus governs the dynamic response. Therefore, δ_s was chosen to be the same as that of a reinforced concrete building. Since no special devices such as tuned mass dampers were used in this project, the logarithmic decrement of damping due to special devices was not considered.

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

Structural type	structural damping, δ_s
reinforced concrete buildings	0,10
steel buildings	0,05
mixed structures concrete + steel	0,08
reinforced concrete towers and chimneys	0,03

Figure 3.8: Part of Table F.2, SS-EN 1991-1-4:2005.

3.7.5 Application of Wind Loads

The three wind pressure profiles were computed within the Python script and applied using the FEM-Design API. Due to the 0.5 m slab extension around the perimeter of the building, the dimensions used for wind load calculations were 54.6 m $\times$ 18.2 m. However, these dimensions were not directly modelled in FEM-Design. Since the wind load was applied to the modelled grid dimensions of 53.6 m $\times$ 17.2 m, a modification factor was introduced to increase the wind load proportionally to the reduced loaded area in the FEM-Design model.

In this project, the along-wind profile was calculated using SS-EN 1991-1-4:2005 and BFS 2024:6. The variation with height was derived using a logarithmic wind profile. The across-wind and torsional actions were calculated using SS-EN 1991-1-4:2026 as a guiding document, in combination with SS-EN 1991-1-4:2005 and BFS 2024:6. In SS-EN 1991-1-4:2026, both the across-wind and torsional actions are approximated as linearly varying with height.

3.8 Acceleration

The along-wind, across-wind, and torsional accelerations were computed according to Section 2.7.3. The combined acceleration was subsequently determined in accordance with Section 2.7.4 using the wind calculation model.

The torsional acceleration α_{pM} was assumed to act diagonally with respect to the building axes. Therefore, it was decomposed into its x - and y -components in order to be included in the calculation of the combined acceleration. The geometric angle of the building was defined as:

$$\theta = \tan^{-1} \left(\frac{d_{\text{building}}}{b_{\text{building}}} \right) \quad (3.7)$$

where d_{building} and b_{building} denote the depth and width of the building, respectively. The torsional acceleration was then decomposed into:

$$\alpha_{pM,x} = \alpha_{pM} \sin \theta \quad (3.8)$$

$$\alpha_{pM,y} = \alpha_{pM} \cos \theta \quad (3.9)$$

The total combined acceleration was subsequently obtained through vector summation of the translational and torsional acceleration components. The resultant acceleration was computed using the square-root-of-sum-of-squares (SRSS) method:

$$a_{\text{comb}} = \sqrt{(a_D + a_{T,y})^2 + (a_L + a_{T,x})^2}$$

where a_D represents the along-wind acceleration, a_L represents the across-wind acceleration, and $a_{T,x}$ and $a_{T,y}$ denote the torsional acceleration components in the global x - and y -directions, respectively.

To evaluate the combined acceleration against the ISO 10137 comfort criteria, the frequency corresponding to the dominant vibration mode was selected.

3.9 Global deflection

The global deflection verification was conducted in accordance with Section 2.8. The results were extracted manually from the computed FEM-Design models as the maximum displacement in the global Y -direction at the top of the building.

3.10 Member Sizing

A preliminary sizing for the structural elements was performed to meet both ULS and SLS criteria. The initial floor plan and grid are based on Habitat 7 as well as the material of the structural members. The calculated structural members are CLT slabs, glulam beams, and columns.

For the preliminary sizing of the beam, two different loading scenarios were adopted. One case for the inner beam with slabs on each side and another case for the facade beam with a slab on one side and facade on the other side. Snow loads are not included in the beam design. The assumption is that there are roof beams to handle this load case, which is not included in the preliminary sizing. The verifications for beams

include bending verification with lateral torsional buckling, shear verification, and instantaneous and final deflection.

The column sizing includes the same two loading cases, including snow load. To improve structural efficiency, the column cross-sections vary every five floors. For preliminary sizing, the design includes up to 30 floors. The verifications include axial compression considering flexural buckling and combined axial compression and bending considering a moment from unintended inclination. Full verification of the beams and columns is presented in Appendix A.1.

The CLT slab sizing was performed for both spans using the conservative assumption that the outer span ended at the centreline of the outer beam, although the CLT panel extended 0.5 m beyond the beam. In reality, this overhang would reduce both the mid-span bending moment and deflection. To simplify the design, the same panel size was used for both span lengths. The CLT slab verifications include bending, shear, instantaneous deflection, and final deflection checks. Vibration verification was excluded from this study. The governing limit state for the CLT slab design was SLS.

Table 3.8: Column and beam dimensions for different storeys.

Structural element	Storeys	Cross-section [mm]	Material
Column	1–5	570 × 765	GL30c
	6–10	570 × 675	GL30c
	11–15	495 × 585	GL30c
	16–20	495 × 540	GL30c
	21–25	360 × 405	GL30c
	26–30	280 × 315	GL30c
Beam	1–30	495 × 280	GL30c
Slab	1–30	CLT 260 L7s2	C24/C14

The concrete core geometry and relevant openings are entirely based on the concrete core of Habitat 7. The placement of its walls and openings will not be altered during the course of this project. An initial wall thickness of 300 mm was assumed. The maximum compressive and tensile stresses in the global behaviour of the concrete core were assessed using FEM-Design for different storey heights. If the concrete core reaches its tensile strength capacity and cracks, it would reduce the global stiffness the concrete core provides to the building configuration. This should therefore be considered during the evaluation of the different building configurations. The other structural elements, such as columns, beams, and CLT slabs, have dimensions as stated above. The results are presented in Table 3.9. The stiffness of a 300 mm concrete core is sufficient for the 10-storey building. However, the utilisation ratios increase significantly for both compression and tension in the 20-storey and 30-storey buildings.

The overall translational deflection was also assessed using FEM-Design for different storey heights. As the current code does not define lateral deflection limits, the deflection limits were determined for different storey heights in accordance with SS-EN

Table 3.9: Utilisation of the concrete core in compression and tension for different storey heights.

Storeys	ULS σ_c [MPa]	UR σ_c [%]	SLS σ_{ct} [MPa]	UR σ_{ct} [%]
10	4.17	21	0	0
20	14.73	74	3.39	167
30	36.19	181	14.82	730

1990-1:2023+A1:2026 Table A.1.11. The results are presented in Table 3.10. The utilisation ratio also increases significantly as the height increases.

Table 3.10: Maximum translational deflection for different storey heights.

Storeys	Deflection [mm]	Limit [mm]	UR [%]
10	8.37	80	10
20	152	160	95
30	953	240	397

Since alterations will be made regarding the stiffness of the building later in this project in the parametric study, the 300 mm thickness is used for the reference building as a starting point. The preliminary study of the concrete core and the global translational deflection indicates the need for an altered building configuration to reach the ULS and SLS limits.

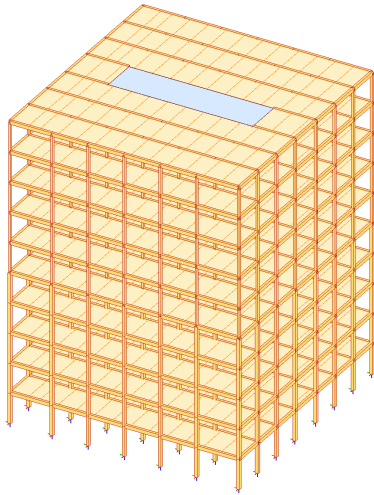
3.11 Parametric Study

A parametric study of the hybrid building's dynamic response and load-bearing capacity was performed. As written in Chapter 2, various mitigation measures could be added to improve the building's resistance to wind actions. With these methods in mind, several parameters were varied individually, and the effects of these modifications were evaluated with regard to SLS comfort criteria and ULS load-bearing capacity. Certain parameters in this study were evaluated with a focus on their impact on the first three natural frequencies and their corresponding modes. The purpose of other parameters was to evaluate the possibility of achieving a different governing vibration mode.

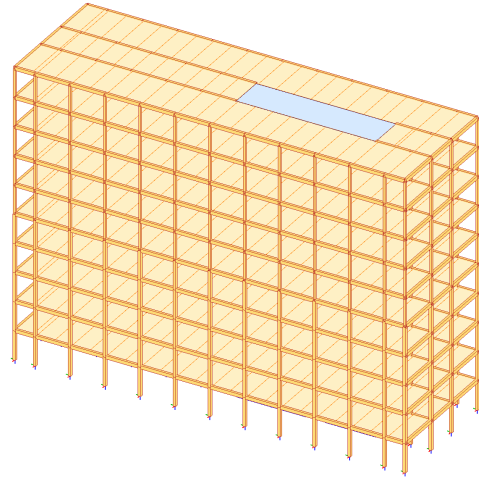
The various parameters in the different case studies conducted were:

- Different aspect ratios,
- Eccentricity of the concrete core by moving it away from the mass centre,
- Different concrete core thicknesses,
- Addition of top concrete slabs,
- Addition of X-bracings to the building's short sides,
- Addition of diagonal bracings to the building's short sides.

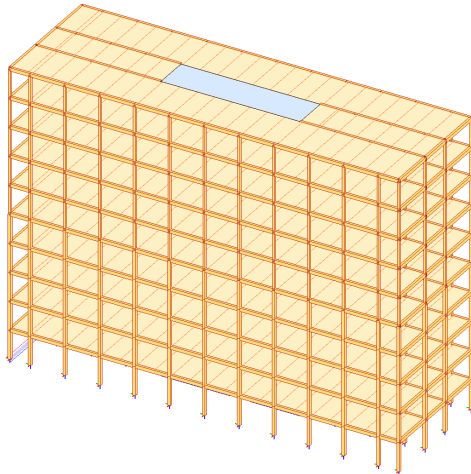
Representation of the different parameter variations are shown in Figure 3.9.



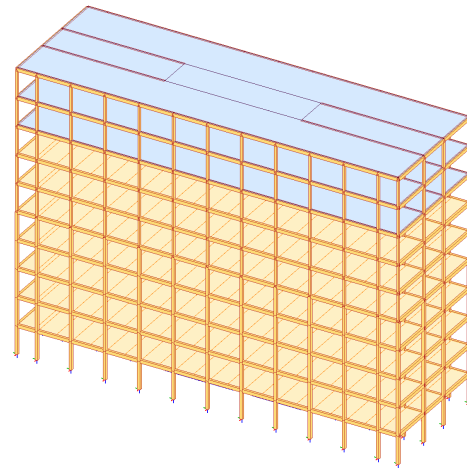
(a) Different aspect ratios



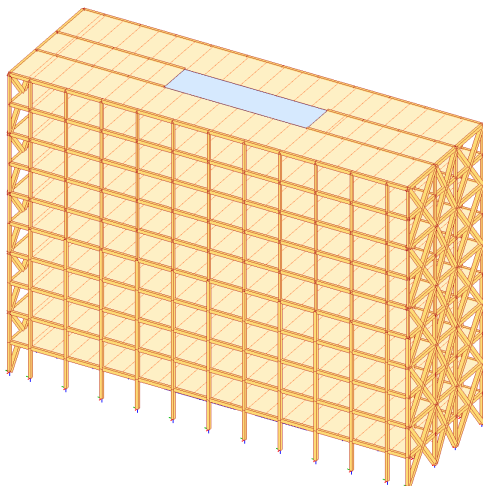
(b) Eccentricity of the concrete core



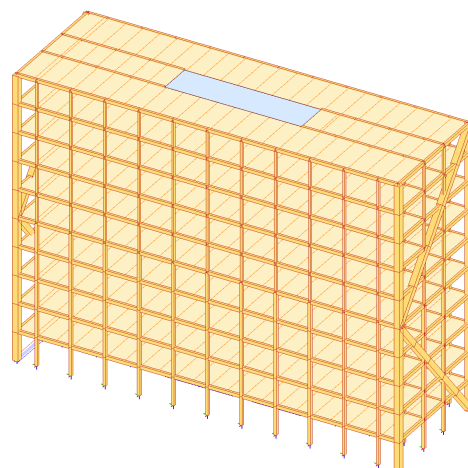
(c) Various core thicknesses



(d) Adding concrete slabs on the uppermost storeys



(e) X-bracings on the short sides



(f) Diagonal bracings on the short sides

Figure 3.9: Overview of the parameter variations and building configurations

3.12 Optimisation

After evaluating the influence of different parameters in the parametric study, a model optimisation was conducted. To improve optimisation efficiency and reduce the number of computed models, a multi-objective evolutionary optimisation was performed by extending the existing Grasshopper/Python/FEM-Design API model-generation script with the Wallacei plugin.

Wallacei X uses the NSGA-II algorithm as the primary evolutionary algorithm and applies the K-means method for clustering (Makki et al., 2019). The input genes (variables) used in the optimisation are summarised in Table 3.11. The following objectives were defined:

- Optimise the combined acceleration UR towards the target value of 98%
- Minimise the amount of concrete

Table 3.11: Genetic algorithm optimisation variables (genes), steps, and parameter ranges across different building heights.

Genes	Step	15-storey	20-storey	25-storey	30-storey
Number of concrete slabs [slab]	1	1–8	1–10	3–15	3–15
Thickness of slabs [mm]	50	200–450	300–500	400–600	450–700
Thickness of shaft walls [mm]	50	300–500	300–500	400–600	500–800
Dim. of diagonal bracing [mm]	200–300	1000–1200	1000–1200	1200–1500	1200–1500

The optimisation was performed for 15-, 20-, 25-, and 30-storey building configurations. Since each model was generated and analysed in FEM-Design with an average runtime of approximately 10 minutes, the population size was reduced to limit the total computational time. Based on the results of the initial parametric study, only practical ranges of gene values were considered, reducing the average total population size to approximately 400–500 combinations. Due to time limitations, both the generation size and the number of generations were kept at approximately eight.

The following algorithm parameters were used:

- Crossover probability: 0.9
- Mutation probability: 1/population size
- Crossover distribution index: 20
- Mutation distribution index: 20
- Random seed: 1

Since the 10-storey reference building had an acceleration UR of only 103.8%, a full optimisation procedure was considered unnecessary. Instead, several minimal manual

modifications were investigated individually. These included adding 300 × 300 mm diagonal bracing, increasing the shaft wall thickness by 50 mm, and adding a single 200 mm concrete slab at the top floor.

4 Results and Discussion

This chapter presents the results and discussion of the parametric studies carried out to evaluate how different structural configurations influence wind-induced accelerations, structural efficiency, and governing design criteria in tall timber–concrete hybrid buildings. Early findings from bracing layouts, slab mass distribution, and core eccentricity are first presented to justify the modelling choices, followed by an assessment of how key design parameters affect along-wind, across-wind, torsional, and combined responses. The chapter further identifies governing vibration modes, compares ultimate and serviceability limit states across building heights, and evaluates the efficiency of optimised structural configurations in terms of material use and dynamic performance.

4.1 Early Findings

This chapter presents observations from the initial analysis phase that were considered in the development of the final analysis model but were excluded from the final results. The effects of different bracing configurations, slab mass distributions, and concrete core eccentricity were investigated. More elaborate results about the early findings could be found in Appendix C.

4.1.1 Bracing Configurations

To increase the lateral stiffness in the wind direction, bracings were introduced on the building's short sides. In the parametric study, two types of bracing configurations were investigated: X-bracings and diagonal bracings. X-bracings with cross-sections of 500×500 mm and 250×250 mm were assessed for the 10-, 20-, and 30-storey buildings. The X-bracings spanned two storeys in height. Diagonal bracings with cross-sections of 500×500 mm, 800×800 mm, and 1200×1200 mm were arranged in a zig-zag pattern on the short sides of the building, connecting the outer corners every five storeys. The efficiency of material usage and the reduction in utilisation ratio were compared between the two bracing configurations.

Table 4.1: Comparison of utilisation ratio [%] of combined acceleration between X-bracing at 500 mm and Diagonal bracing at 800 mm.

Storeys	X-bracing 500 mm	Diag. Bracing 800 mm	Abs. Difference
10	89.2%	78.5%	10.7%
15	156.8%	143.9%	12.9%
20	210.4%	196.0%	14.4%
25	250.4%	238.8%	11.6%
30	278.3%	270.1%	8.2%
Mean Difference			11.6 %

The diagonal bracing demonstrated more efficient timber usage than the X-bracing

when the utilisation ratios of the two configurations were compared. The diagonal bracing with a cross-section of 800×800 mm used approximately the same volume of timber as the 500×500 mm X-bracing with a mean volume difference of 0.7 m^3 . However, a mean reduction of 11.6% in utilisation ratio was observed for the 800×800 mm diagonal bracing compared to the 500×500 mm X-bracing, as shown in Table 4.1. Larger cross-sections than 500×500 mm were not assessed for the X-bracings due to the unrealistic degree of coverage on the building's short sides. Considering the efficiency of the diagonal bracings, the diagonal bracings was adopted for the combined building configurations. Full comparison of both types is shown in Appendix C.

4.1.2 Slab Mass Distribution Variation

To mitigate the top acceleration of the building, additional mass was introduced in the form of concrete slabs. Two approaches were assessed. First, 300 mm thick concrete slabs were added to the top one, three, and five floors of the 10-, 15-, 20-, 25-, and 30-storey buildings. The same buildings were then analysed with the top three slabs increased to a thickness of 500 mm. This allowed for a comparison of the effectiveness of increasing mass either by adding more slabs or by increasing their thickness. Finally, the influence of mass location was evaluated by placing a single 900 mm thick concrete slab at the top and comparing it with three 300 mm slabs of equal total volume. The same comparison was carried out using a 1500 mm thick slab at the top and three 500 mm slabs.

Table 4.2: Comparison of utilisation ratio [%] of combined acceleration between 1 x 1500 mm and 5 x 300 mm slab configuration.

Storeys	1 x 1500 mm	5 x 300 mm	Abs. Difference
10	63.0%	63.5%	0.5%
15	100.6%	113.6%	13.0%
20	151.8%	162.5%	10.7%
25	189.2%	201.2%	12.0%
30	217.5%	228.7%	11.2%

The 1×1500 mm slab configuration exhibited the lowest utilisation ratio across all building heights. Concentrating the mass at the top of the building proved to be the most efficient for wind response, although its effect is primarily that of a concentrated dead weight. This creates a significant load for the columns supporting the slabs.

However, the 5×300 mm configuration is not far behind in performance. To balance structural integrity with wind response, the distributed solution is more beneficial. Even though the 5×300 mm configuration performs slightly worse than the 1×1500 mm slab in terms of the utilisation ratio, it successfully combines structural performance with the mitigation of wind-induced vibrations. For this reason, slab count variation is the primary parameter for further investigation in the subsequent combined building configurations. A full comparison of different slab mass distributions is shown in Appendix C.

4.1.3 Core Eccentricity

Initially, the effect of eccentric placement of the concrete core was investigated to explore whether increasing the distance between the centre of mass and the rotational axis of the building would change the governing wind response mode.

However, this resulted in more complex structural behaviour and introduced more complex hybrid vibration modes, making it difficult to clearly distinguish whether a specific mode should be classified as along-wind or torsional. Furthermore, the vibration modes were no longer linear. Since the procedure in SS-EN 1991-1-4:2026 for calculating across-wind and torsional effects is based on the assumption of linear vibration modes, the obtained results for buildings with eccentric cores were considered unreliable.

Therefore, the subsequent analyses were limited to buildings with a central placement of the concrete core.

4.2 Effects of Building Configuration on Accelerations

In this section, an investigation of parameters affecting stiffness, vertical mass distribution, and the building's floor plan ratio was conducted. These parameters were varied through changes in concrete core thickness, concrete slab distribution, timber diagonal bracing, and floor aspect ratio, in order to evaluate their influence on along-wind, across-wind, torsional, and combined accelerations. A more detailed investigation, including the impact of X-bracing and additional concrete slab variations, is presented in Appendix C.

4.2.1 Along-Wind Acceleration

First, the effects of building configuration on the along-wind response were investigated. In Figure 4.1, the results are compared with the corresponding limit curves. As the axes are logarithmic to improve interpretation of the results, the same data are also presented in Figure 4.5, where they are expressed as acceleration utilisation ratios (UR) with respect to frequency.

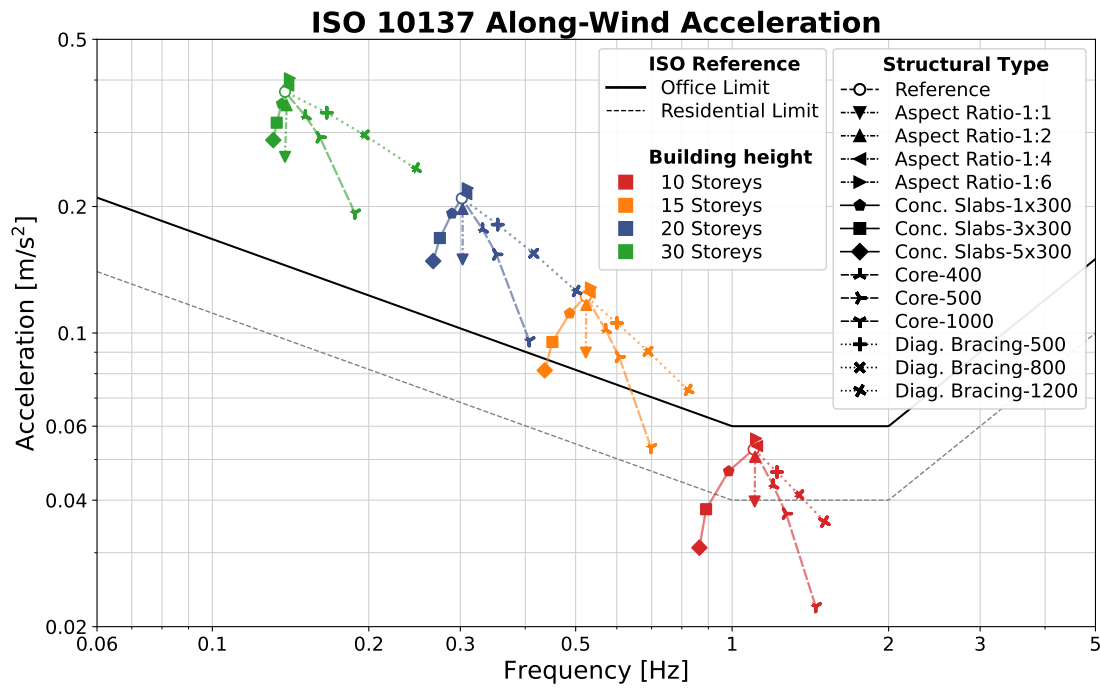


Figure 4.1: ISO 10137 comfort limits for along-wind acceleration of different structural types and configurations.

In the following figures, only acceleration utilisation ratios are presented. It should be noted that the ISO 10137 limit curve is not constant. The lowest limit values occur between 1 and 2 Hz, where human sensitivity to vibrations is greatest. Outside this range, higher accelerations are permitted. Therefore, a decrease in frequency may improve the utilisation ratio even if the acceleration remains unchanged. As this thesis focuses on occupant comfort in tall buildings, the following plots are presented in terms of utilisation ratio rather than acceleration.

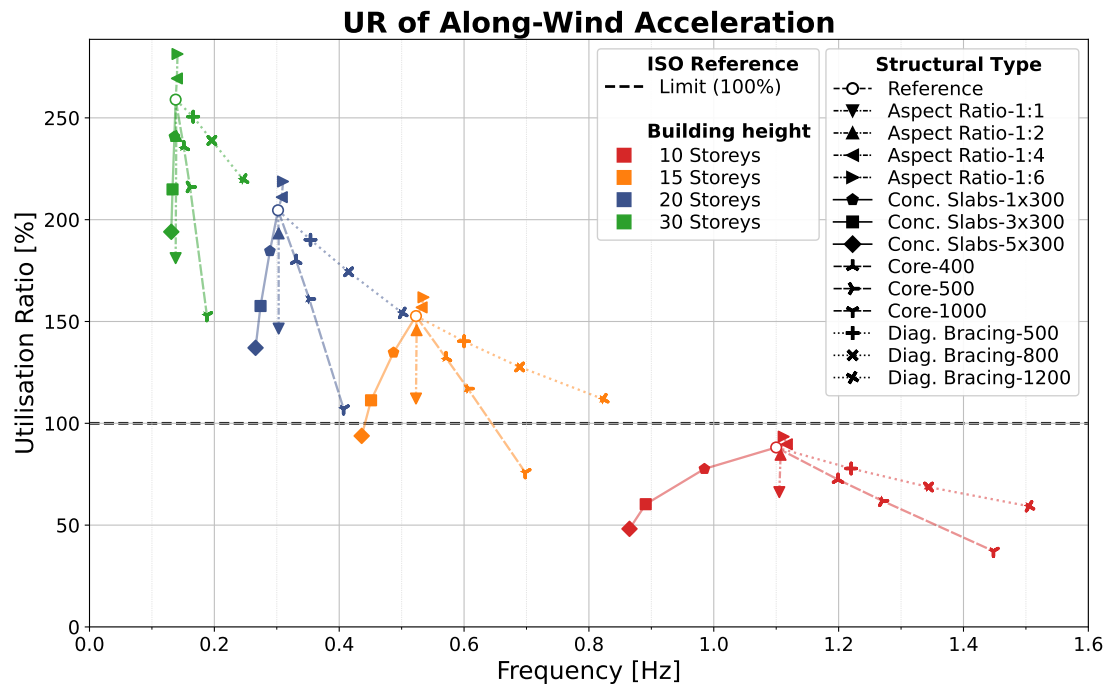


Figure 4.2: Utilisation ratios of ISO 10137 comfort limits for along-wind acceleration.

The reduction in along-wind acceleration is achieved either by adding mass to the upper floors or by increasing structural stiffness through diagonal bracing or increased core thickness. For taller buildings, the influence of floor plan aspect ratio also becomes increasingly significant. Along-wind response was the only response type that exhibited no hybrid modes, even for the tallest buildings. The along-wind acceleration utilisation ratio (UR) with respect to frequency is shown in Figure 4.2.

Adding concrete slabs resulted in a significant reduction in UR, with the effect becoming slightly more pronounced for taller buildings. For the 15-storey building, five slabs of 300 mm thickness resulted in a 59.4% reduction in UR, while for the 30-storey building the reduction increased to 65.0%. However, slab additions also reduced the fundamental frequency and, since most configurations lie below 1 Hz, part of the UR improvement resulted from the shape of the ISO 10137 limit curve.

Diagonal bracing was the most efficient method for increasing the first along-wind natural frequency. The greatest effect was observed for lower buildings, where the frequency increased by 0.42 Hz for the 10-storey building, compared to approximately 0.1 Hz for the 30-storey building. The largest reduction in UR was observed for the 20-storey building, where adding 1.2×1.2 m bracing reduced UR by 50.4%.

Increasing the core thickness also resulted in a clear reduction in UR, with the effect becoming more significant for taller buildings. Increasing the core thickness from 300 mm to 1000 mm for the 30-storey model yielded the highest UR improvement of 106.0%. However, such a configuration is considered unrealistic and is included only to illustrate the trend. For more realistic configurations, increasing the core thickness from 300 mm to 500 mm reduced UR by 26.2% for the 10-storey building and by 42.9% for the 30-storey building.

Changing the aspect ratio had little influence on the first along-wind natural frequency but clearly affected the acceleration response. Elongating the building slightly increased UR, while more square floor plans reduced it more significantly. The influence of aspect ratio became increasingly pronounced with building height. Changing the aspect ratio from 1:3 to 1:1 reduced UR by 21.9% for the 10-storey building and by 77.9% for the 30-storey building.

Overall, along-wind response was more sensitive to additional mass than the across-wind and torsional responses, while also being strongly influenced by increases in structural stiffness through diagonal bracing and increased core thickness. Since increasing core thickness contributes both mass and stiffness, it proved particularly effective in reducing along-wind response. The results also indicate that floor plan geometry becomes increasingly important for taller buildings, where more square floor plans significantly improve occupant comfort performance.

4.2.2 Across-Wind Acceleration

For the reference building, across-wind accelerations were the only response type that remained below the ISO 10137 limit for all four building heights. The largest influence on across-wind accelerations came from changes in core thickness and building aspect ratio. The across-wind acceleration utilisation ratio (UR) with respect to frequency is shown in Figure 4.3.

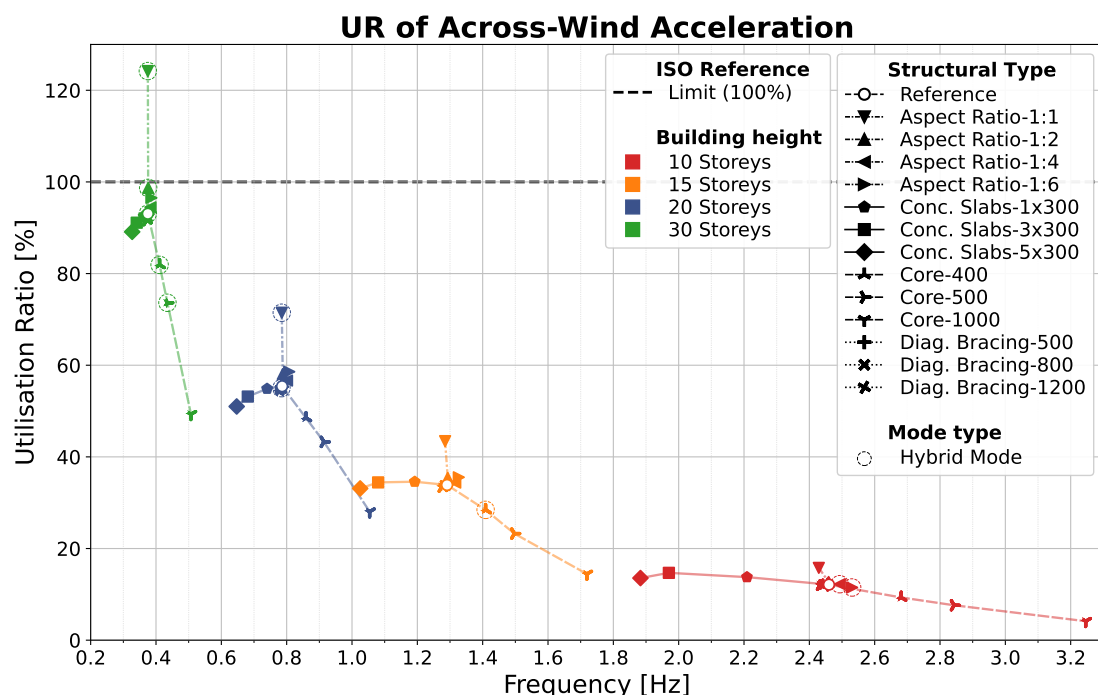


Figure 4.3: Utilisation ratios of ISO 10137 comfort limits for across-wind acceleration.

Increasing the core thickness significantly improved the utilisation ratio (UR), as it was the only parameter that directly increased stiffness in the across-wind direction. The effect became more pronounced with increasing building height. Increasing the core thickness from 300 mm to 500 mm reduced UR by 4.5% for the 10-storey building and

by 19.5% for the 30-storey building. For lower buildings, the increase in core thickness also significantly increased the first across-wind natural frequency, by 0.38 Hz for the 10-storey model, compared to only 0.06 Hz for the 30-storey model.

The aspect ratio had a significant effect only for highly elongated buildings. While aspect ratios between 1:1 and 1:4 resulted in minor changes, increasing the ratio to 1:6 caused a considerable increase in UR, ranging from 3.7% for the 10-storey building to 31.1% for the 30-storey building.

Adding concrete slabs decreased the first across-wind natural frequency, while having a negligible influence on acceleration. For buildings with frequencies below 1 Hz, this resulted in a reduced UR only due to the shape of the ISO 10137 limit curve, rather than due to reduced acceleration.

Since diagonal bracing in the parametric study was applied only on the short sides of the building, it did not contribute significantly to stiffness in the across-wind direction and therefore had a negligible impact on UR.

Overall, the across-wind response was governed primarily by stiffness in the across-wind direction, making increased core thickness the most effective investigated parameter for reducing UR. In contrast to along-wind response, mass addition had little influence on the actual acceleration response and only slightly improved UR due to the frequency-dependent shape of the ISO 10137 limit curve. The increasing sensitivity of taller and more elongated buildings to across-wind response also indicates that floor plan proportions become increasingly important with height. Although not investigated in this study, the observed trends suggest that introducing bracing on the long sides of the building could further improve across-wind performance by increasing stiffness in the governing direction.

4.2.3 Torsional Acceleration

The largest influence on torsional accelerations came from changes in building stiffness and aspect ratio, while the effect of added mass was negligible. Compared to the other response types, torsional acceleration exhibited the highest number of hybrid modes, with their occurrence increasing for taller buildings. The torsional acceleration utilisation ratio (UR) with respect to frequency is shown in Figure 4.4.

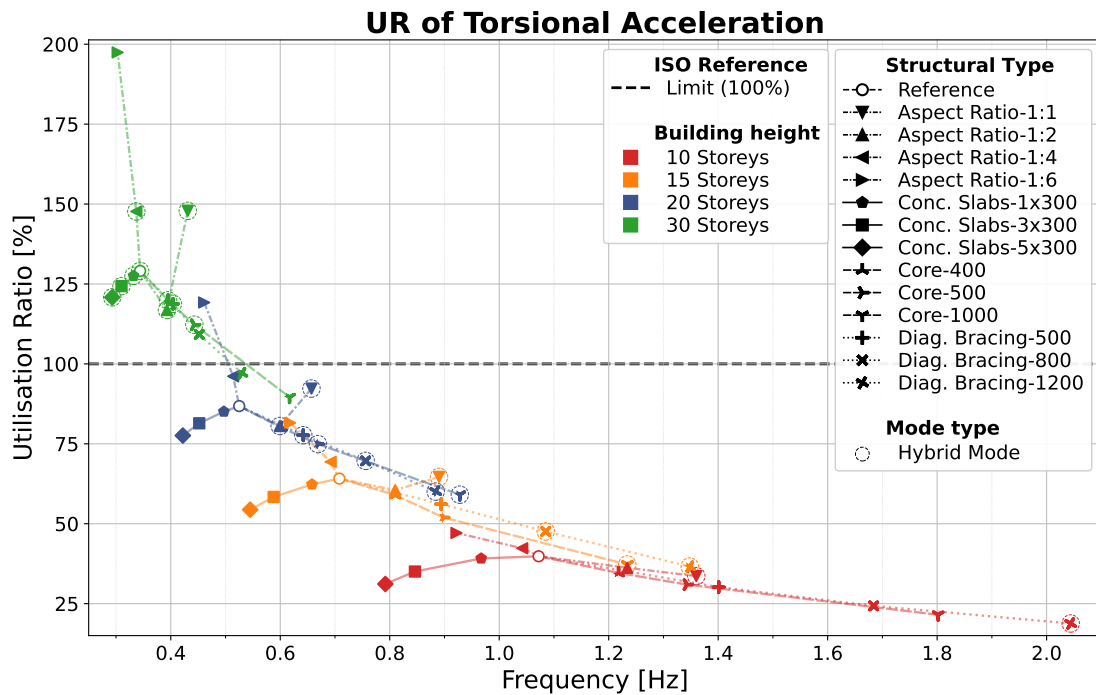


Figure 4.4: Utilisation ratios of ISO 10137 comfort limits for torsional acceleration.

Adding diagonal bracing on the short sides of the building resulted in the greatest improvement in UR, with the effect slightly increasing with building height. Adding 1.2×1.2 m bracing reduced UR by 27.5% for the 15-storey building and by 32.0% for the 30-storey building. Diagonal bracing also significantly increased the torsional fundamental frequency, by 0.97 Hz for the 10-storey model and by 0.41 Hz for the 30-storey model.

Increasing the core thickness had a similar effect to diagonal bracing, although comparable improvements required unrealistically large core thicknesses, such as 1.0 m. For a more realistic thickness of 0.5 m, the UR reduction for the 30-storey building was 16.8%.

Adding concrete slabs had a negligible influence on torsional acceleration, while slightly decreasing the torsional frequency. This resulted in a minor reduction in UR caused only by the shape of the ISO 10137 limit curve.

Changing the aspect ratio resulted in a parabolic trend. For buildings with torsional frequencies below 1 Hz, the lowest UR occurred at an aspect ratio of approximately 1:2.5, while both increasing and decreasing the aspect ratio from this value led to higher UR values, particularly for taller buildings. For buildings with torsional frequencies above 1 Hz, the trend remained parabolic; however, elongated buildings showed a reduced UR, while more square floor plans resulted in an increased UR. Compared to the reference aspect ratio of 1:3, increasing the aspect ratio to 1:6 reduced UR by 4.2% for the 10-storey building but increased it by 68.3% for the 30-storey building.

Overall, the torsional response was governed primarily by the torsional stiffness of the building. For buildings up to approximately 15 storeys, diagonal bracing was the most

efficient solution for reducing torsional response. However, for taller buildings, the influence of core thickness became increasingly significant. Similar to the across-wind response, mass addition had little influence on the actual torsional acceleration response and affected UR mainly through changes in frequency relative to the ISO 10137 limit curve. The results also indicate that torsional response is highly sensitive to floor plan proportions, particularly for taller buildings, where deviations from the optimal aspect ratio of approximately 1:2.5 result in rapid increases in UR.

4.2.4 Combined Wind Acceleration

For the reference building, the combined acceleration values exceeded the office comfort limit according to ISO 10137 for all four storey heights. Although changes in the building configuration significantly improved the results, the figure shows that, starting from the 15-storey reference building, modifying only a single parameter is insufficient to satisfy the ISO limits. Instead, a combination of parameters is required. The combined acceleration utilisation ratio (UR) with respect to frequency is shown in Figure 4.5.

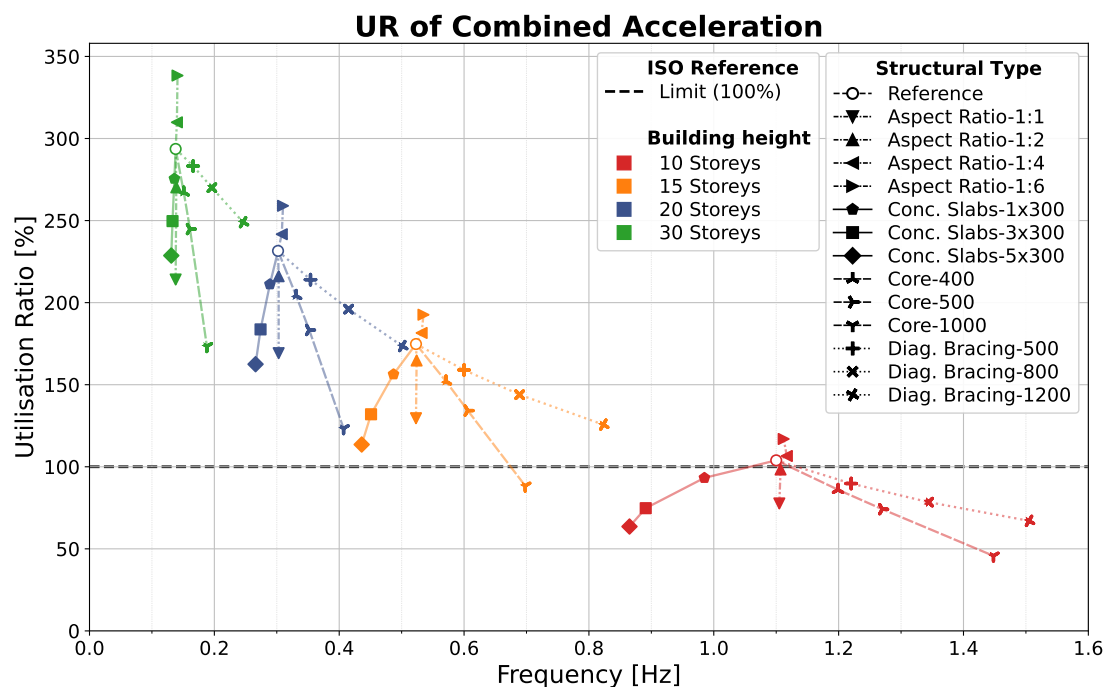


Figure 4.5: Utilisation ratios of ISO 10137 comfort limits for combined acceleration.

Adding concrete slabs primarily increased mass and mainly affected along-wind acceleration. Due to the shape of the ISO 10137 limit curve, the effect of additional slabs was smaller for lower buildings with fundamental frequencies above 1 Hz, while for buildings with frequencies below 1 Hz the influence of height became negligible. For the 10-storey building, with a reference fundamental frequency of 1.1 Hz, adding five 300 mm slabs reduced UR by 40.3%, while for the 15- and 30-storey buildings, both with frequencies below 1 Hz, the reductions were 61.2% and 64.9%, respectively.

Adding diagonal bracing mainly affected along-wind and torsional accelerations. Among

the analysed configurations, diagonal bracing was the most effective method for increasing the natural frequency, although the effect decreased with height. For the 10-storey model, adding 1.2×1.2 m bracing increased the frequency by 0.4 Hz, while for the 30-storey model the increase was reduced to 0.1 Hz. However, for frequencies below 1 Hz, increased frequency may also lead to stricter comfort limits.

Increasing the core thickness increased both mass and stiffness and therefore affected all three acceleration response modes. Increasing the core thickness from 300 mm to 1000 mm for the 30-storey model yielded the highest UR improvement of 120.2%. However, such a configuration is considered unrealistic and is included only to illustrate the trend.

Changes in aspect ratio had a negligible effect on frequency but a clear influence on acceleration response. For along-wind and across-wind responses, floor plans closer to a square shape reduced UR, while more elongated floor plans increased it. In contrast, torsional response showed the lowest UR for aspect ratios close to 1:2.5, while deviations in either direction increased UR. The combined response reflected the influence of all these trends. For the 10-storey building, changing the aspect ratio from 1:3 to 1:1 improved UR by 26.2%, while for the 30-storey building the improvement increased to 79.5%. The opposite trend was observed for elongated buildings, where increasing the aspect ratio to 1:6 resulted in a UR increase of 44.8% for the 30-storey building.

Overall, increasing structural stiffness through diagonal bracing or increased core thickness was the most effective method for reducing combined acceleration response. However, the effectiveness of additional mass was strongly influenced by the shape of the ISO 10137 limit curve, particularly for buildings with frequencies below 1 Hz. Changes in aspect ratio also had a significant influence on acceleration response, especially for taller buildings, while requiring no additional structural material. This suggests that optimisation of the floor plan geometry may provide an efficient early-stage design strategy for mitigating wind-induced accelerations in tall timber–concrete hybrid buildings.

4.3 Evaluation of Governing Acceleration Mode

For all studied configurations of the examined building, along-wind acceleration was the governing response, although the relative influence of across-wind and torsional accelerations varied depending on the configuration. The relationships between along-wind, across-wind, and torsional acceleration utilisation ratios (UR) with respect to frequency are shown in Figure 4.6.

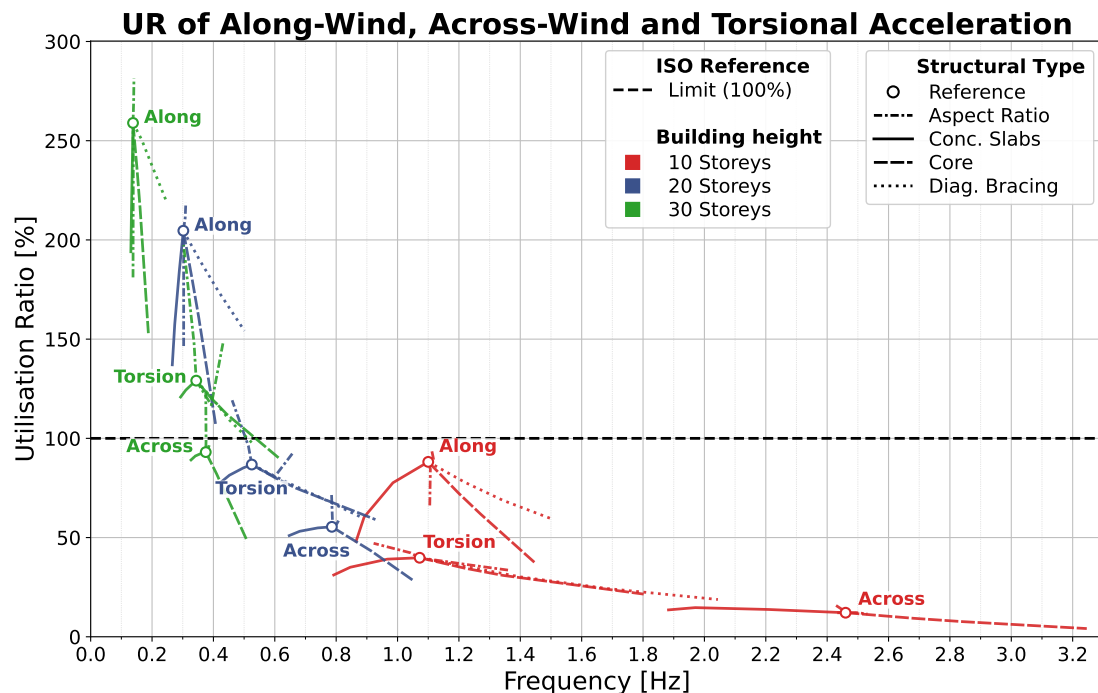


Figure 4.6: Utilisation ratios of ISO 10137 comfort limits for along-wind, across-wind and torsional acceleration.

A clear trend can be observed in which acceleration magnitude increases exponentially with building height. As a result, the absolute increase in along-wind acceleration is greater than for the other response modes. To better compare the governing influence of the different responses, relative differences were evaluated instead of absolute values. The relative difference was calculated as the difference between the along-wind and across-wind or along-wind and torsional UR values, divided by the along-wind UR and expressed as a percentage. The trend was defined as the difference between a specific relative difference and the corresponding relative difference of the reference building. The UR values, relative differences, and corresponding trends are summarised in Table 4.3.

For the 10-storey reference building, the along-wind UR was 54.8% higher than the torsional UR and 86.3% higher than the across-wind UR. For the 30-storey reference building, the difference between along-wind and torsional UR decreased to 50.2%. Meanwhile, the relative influence of across-wind acceleration increased more rapidly, reducing the relative difference to 64.1%.

Adding concrete slabs resulted in the greatest reduction in along-wind UR, thereby in-

Table 4.3: Relative differences between across-wind and along-wind UR, and between torsional and along-wind UR, together with trends in the relative differences for different modifications of the reference building.

Parameter impact on relative difference between the modes				
Storeys	Across / Along		Torsional / Along	
	Relative Dif.	Trend	Relative Dif.	Trend
Reference building				
10	86.3%	-	54.8%	-
20	72.9%	-	57.6%	-
30	64.1%	-	50.2%	-
Adding 5×300 mm concrete slabs				
10	71.8%	-14.5%	35.5%	-19.3%
20	62.8%	-10.1%	43.4%	-14.2%
30	54.1%	-10.0%	37.7%	-12.4%
Changing shaft thickness to 1000 mm				
10	89.0%	+2.7%	42.4%	-12.5%
20	73.9%	+1.0%	44.9%	-12.7%
30	67.8%	+3.7%	41.4%	-8.7%
Adding 1200×1200 mm diagonal bracing				
10	79.6%	-6.7%	68.3%	+13.5%
20	64.8%	-8.1%	61.0%	+3.4%
30	57.7%	-6.4%	55.8%	+5.6%
Changing aspect ratio to 1:6				
10	87.7%	+1.4%	49.6%	-5.2%
20	73.2%	+0.3%	45.5%	-12.1%
30	65.7%	+1.7%	29.9%	-20.3%
Changing aspect ratio to 1:1				
10	76.1%	-10.1%	49.2%	-5.6%
20	51.2%	-21.7%	37.0%	-20.6%
30	31.4%	-32.6%	18.3%	-31.8%

creasing the proportional influence of the other response modes, particularly torsional acceleration. After adding five 300 mm concrete slabs, the relative difference between along-wind and torsional UR decreased by 19.3% to 35.5% for the 10-storey building, while the difference between along-wind and across-wind UR decreased by 14.5% to 71.8%. For the 30-storey building, the corresponding reductions were 12.4% and 10.0%, respectively.

Increasing the core thickness contributes more equally to all three response modes. Therefore, the relative proportions remain closer to those of the reference building. Increasing the core thickness from 300 mm to 1000 mm increased the relative importance of torsional acceleration for the 10-storey building by 12.5%, while the across-wind response became 2.7% less governing. Similar trends were observed for the 30-storey

building, resulting in a relative increase in torsional response of 8.7% and a decrease in across-wind response of 3.7%, respectively.

Diagonal bracing had the greatest influence on torsional acceleration, while having a negligible impact on across-wind acceleration. After adding 1.2×1.2 m bracing on the short sides of the building, torsional acceleration was 13.5% less governing than the along-wind response, with a relative difference of 68.3% for the 10-storey building and 5.6% for the 30-storey building. For across-wind response, the trend was opposite, resulting in a relative increase in importance of 6.7% for the 10-storey building and 6.4% for the 30-storey building.

Changing the aspect ratio had the strongest influence on torsional acceleration for elongated buildings, while along-wind acceleration was most affected for more square floor plans. In both cases, the effect became more pronounced with increasing building height. For the 30-storey building, reducing the aspect ratio from 1:3 to 1:1 significantly decreased the along-wind UR, while across-wind and torsional UR increased, reducing the relative difference to 18.3% between along-wind and torsional acceleration and to 31.4% between along-wind and across-wind acceleration. A similar trend was observed when elongating the building to an aspect ratio of 1:6. However, in this case all UR values increased, with the largest increase occurring in torsional acceleration. For the 30-storey building, this resulted in a relative increase in torsional response of 20.3% and a decrease in across-wind response of 1.7%, respectively.

However, the relative importance of across-wind and torsional response changed significantly depending on the selected design strategy, while along-wind response remained the governing one for all investigated configurations. The greatest variation between response modes was caused by changes in aspect ratio, particularly for taller buildings, where torsional response became closest in magnitude to along-wind response for more square floor plans. For other parameters, additional mass had the greatest impact on relative difference, primarily by reducing along-wind response and thereby increasing the proportional importance of torsional and across-wind accelerations. These results indicate that evaluating tall timber–concrete hybrid buildings based only on along-wind response may underestimate the importance of torsional and across-wind accelerations for certain building configurations.

4.4 Evaluation of Governing Limit State

The evaluation of the optimised configurations indicates that wind-induced acceleration and the ISO comfort limits are the governing design criteria for hybrid timber-concrete high-rise buildings. Across all investigated heights, the ISO 10137 comfort criteria consistently controls the structural response, while the ULS compression and SLS global deflection utilisation ratios (UR) remain comparatively low at lower heights. No tensile stresses, or only negligible tensile stresses caused by lateral loads, were observed within all configurations. Since all configurations were optimised using the same parameters and functions, the resulting UR values and structural systems are relatively similar within each building height. Therefore, the primary significance of the results lies not in the individual subtype differences, but in the overall structural trend observed as building height increases.

Table 4.4: Configuration overview for different storey heights and subtypes.

Storeys	Subtype	Slabs	Bracing	Core. Thick.	V_{conc}/Storey
10	1	-	300×300 mm	300 mm	88.3 m ³
	2	-	-	350 mm	101.5 m ³
	3	1×200 mm	-	300 mm	104.9 m ³
15	1	2×400 mm	1200×1200 mm	300 mm	133.5 m ³
	2	1×400 mm	1200×1200 mm	400 mm	137.7 m ³
	3	3×200 mm	1200×1200 mm	400 mm	147.5 m ³
20	1	5×500 mm	1200×1200 mm	350 mm	206.1 m ³
	2	5×450 mm	1200×1200 mm	400 mm	208.7 m ³
	3	5×500 mm	1200×1200 mm	400 mm	219.3 m ³
25	1	11×400 mm	1500×1500 mm	450 mm	274.0 m ³
	2	9×450 mm	1500×1500 mm	500 mm	275.8 m ³
	3	8×550 mm	1200×1200 mm	600 mm	314.1 m ³
30	1	10×550 mm	1500×1500 mm	700 mm	346.3 m ³
	2	13×550 mm	1500×1500 mm	550 mm	352.4 m ³
	3	11×550 mm	1500×1500 mm	700 mm	361.5 m ³

Table 4.4 displays an overview of all optimised building configurations. The configurations are optimised in terms of UR with respect to ISO comfort limits and concrete volume using the Wallacei plugin in Grasshopper. Three of the best-optimised configurations were chosen for each building height, and the subtypes were selected considering the concrete volume per storey. Table 4.5 presents the different ULS and SLS UR for the building configurations.

At 10–15 storeys, the structures are entirely governed by dynamic serviceability. The ULS compression UR remains between 20–40%, while the ISO comfort limit UR is already around 100%. This indicates that lightweight structural systems are highly susceptible to wind-induced accelerations. Even though the buildings possess sufficient material strength and acceptable global deflections, occupant comfort becomes the deciding factor long before the structural capacity is fully utilised.

Within each height category, the optimised configurations rely on two primary strategies to mitigate wind-induced accelerations: increasing global stiffness and increasing distributed mass. Greater stiffness is achieved through larger bracing dimensions and thicker concrete cores, while distributed slab mass contributes to an increased equivalent mass and therefore reduced accelerations. The high ISO comfort limit UR indicates that both methods are necessary for controlling wind-induced response, particularly as the building becomes taller.

At heights of 20–25 storeys, the optimisation process increases both the use of global stiffness and added mass significantly. Both subtype 1 and 2 in the two building height categories exceed the ISO comfort limit UR by 0.9–2.5%, despite the substantial in-

Table 4.5: ULS and SLS verification results for different building configurations.

Storeys	Subtype	ULS Comp.	SLS ISO 10137	SLS Deflection
10	1	25.5%	97.6%	15.3%
	2	24.3%	94.0%	15.8%
	3	27.8%	97.2%	17.4%
15	1	37.5%	97.1%	18.8%
	2	31.1%	99.0%	17.5%
	3	30.8%	96.5%	17.8%
20	1	60.5%	101.9%	40.2%
	2	55.2%	102.5%	38.8%
	3	56.2%	98.7%	38.3%
25	1	69.4%	100.9%	55.5%
	2	66.2%	101.6%	54.5%
	3	74.3%	98.2%	65.5%
30	1	77.2%	104.0%	88.8%
	2	90.8%	100.4%	92.1%
	3	78.2%	100.4%	87.9%

crease in slab mass and core thickness. Subtype 3 for 25 storeys satisfies the comfort requirement with a UR of 98.2% by combining a thicker 600 mm core with 8×550 mm slabs. Compared to the subtypes for the lower building heights, the improvement in dynamic response comes at the expense of significantly increased dead load and material usage. The ULS compression UR simultaneously increases to 74.3%, illustrating that material strength capacity becomes increasingly influential in structural design at greater heights.

At 30 storeys, extremely large bracing dimensions, thick concrete cores, and substantial distributed slab mass are required to barely meet the ISO comfort limits. Even though subtype 3 includes 11×550 mm slabs, 1500×1500 mm bracings, and 700 mm core thickness, the ISO comfort limit UR is exceeded at 100.4%. The resulting increase in structural dead load causes the ULS compression UR to approach significant levels. For subtype 2, all UR values are above 90% with its comparatively slender core of 550 mm. The structural design is therefore no longer governed solely by wind-induced accelerations, but material strength capacity must also be considered to support the accumulated self-weight of the structure.

Overall, the results demonstrate that dynamic serviceability governs throughout all building heights. However, these measures simultaneously increase the structural dead load, causing compression UR to become increasingly significant. Material strength, on the other hand, can be managed by changing the concrete class or adding reinforcement, and is therefore not critical at this stage. The global deflection UR is, however, of greater significance. The results therefore demonstrate a clear transition from a predominantly serviceability-governed design towards a combined serviceability and

strength-governed structural behaviour as the building height increases for this building layout.

4.5 Optimised Building Configurations

In this section, the optimised building configurations are evaluated based on their structural efficiency in terms of material usage, strength, and mitigation of wind-induced accelerations. The results indicate that timber-concrete hybrid systems gradually lose structural efficiency beyond 20 storeys. Figure 4.7 displays the concrete, timber, and updated timber volumes used per storey for the different optimised building configurations.

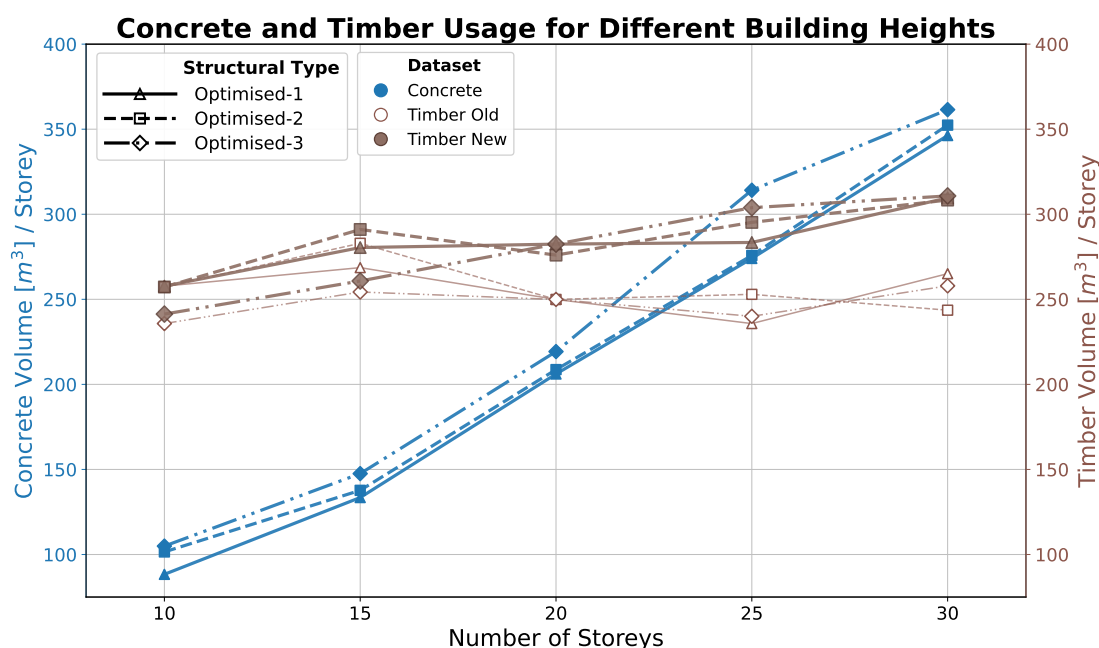


Figure 4.7: Concrete, timber, and updated timber volume used per storey for different optimised building configurations.

The concrete volume increases almost linearly with increasing building height, reflecting the growing demand for stiffness and mass to mitigate wind-induced accelerations. While the timber volume before the added slabs remains relatively constant, the updated timber volumes with the additional slab mass also increase with height. However, this increase is not primarily caused by timber contributing to the lateral stability or added mass of the structure. Instead, the timber volume increases mainly due to the additional dead load generated by the added concrete slabs required to meet ISO 10137 comfort criteria.

At 10–15 storeys, the hybrid system remains structurally efficient. All configurations satisfy the ULS and deflection criteria with large margins, while the UR remains close to or below 100% with respect to the ISO comfort limits. In this height range, the timber-concrete hybrid performs efficiently with relatively small amounts of concrete. Subtype 1 for the 10-storey building only required minimal bracing dimensions of 300×300 mm to satisfy the ISO comfort limits, while subtype 2 for the 15-storey building appears

more balanced in terms of practical dimensions and achieves the lowest UR of 96.5% for the ISO comfort limits with only a moderate increase in concrete volume.

At 20 storeys, the system reaches a transition point where the timber and concrete volumes used per storey approach each other. The need for additional concrete slabs and larger cross-sections also becomes more pronounced. Here, subtype 3 maintains relatively low ULS and deflection UR while keeping the acceleration levels close to the ISO comfort limits. As the concrete volume increases, the timber volume also rises due to the increased column cross-sections to carry the added mass from the 5×500 mm slabs.

At 25–30 storeys, the structural efficiency of the hybrid system begins to decrease. The concrete volume per storey increases significantly and exceeds the timber volume per storey for subtype 3 in order to stay below the ISO comfort limits. This suggests that the hybrid structure gradually behaves more like a concrete building with timber components rather than a true mixed structural system. The 30-storey configurations demonstrate this clearly, where large concrete cores, thick and numerous slabs, and substantial bracing dimensions are all required to satisfy the ISO comfort limits. The increase in concrete volume becomes necessary primarily to fulfil serviceability demands related to wind-induced accelerations.

This raises the question of whether timber-concrete hybrid structures are truly efficient at greater heights. The results indicate that beyond 25–30 storeys, the hybrid concept loses material efficiency because increasingly large concrete elements are required to provide both mass and global stiffness to stabilise the structure. In particular, subtype 2 at 30 storeys reaches a very high ULS compression UR of 90.8% despite having a massive concrete core. This indicates that the additional material does not provide proportional structural benefits.

Overall, the results suggest that the timber-concrete hybrid concept is most structurally efficient up to approximately 20 storeys, corresponding to buildings around 80 m in height in this study. Within this range, the interaction between timber and concrete achieves a favourable balance between stiffness, strength, mass, and dynamic performance. Beyond 25 storeys, the increasing reliance on concrete for vibration control causes both the concrete and timber volumes to rise substantially. The timber increase is primarily a consequence of supporting the additional concrete dead load rather than improving the lateral structural behaviour itself. This indicates a gradual loss of structural efficiency in the hybrid concept at greater heights, where fully concrete systems may become comparatively more competitive from a material efficiency perspective.

4.6 Potential Sources of Error

The examined building was analysed using several simplifications and assumptions that may have influenced the accuracy of the results. Since the current Eurocode does not define procedures for across-wind and torsional actions, the upcoming standard SS-EN 1991-1-4:2026 was used. However, this standard includes limitations regarding building height and floor plan proportions, as described in Section 2.7.2. As some of the lower building models fall outside the recommended height range, a more detailed

analysis of buildings below 25 storeys could improve the reliability of the results.

In addition, all boundary conditions were simplified. In the analysed models, all timber beam, column, and panel connections were assumed to be pinned, while the concrete core connections were assumed to be rigid. No detailed ULS or SLS assessment of the connections was performed. A more refined connection model could influence both the global stiffness of the building and the sizing of structural members.

The foundation was assumed to be perfectly stiff, with pinned connections for the columns and rigid connections to the concrete core. In reality, soil–structure interaction and foundation flexibility may affect the dynamic response of tall buildings.

Furthermore, the optimisation using the genetic algorithm was performed with a smaller population size and fewer generations than recommended by the developers of the algorithm. Increasing the number of parameters, population size, and number of generations could potentially lead to more optimal structural configurations.

4.7 Future Studies

Further studies could investigate how different core geometries and configurations influence the dynamic behaviour and occupant comfort of hybrid timber–concrete high-rise buildings. Variations in stiffness distribution and core shape may affect both global stability and vibration performance. In this project, the core geometry was identical for both the 10-storey and 30-storey buildings. This may not be realistic, since a 30-storey building would likely require a larger core to accommodate additional elevators and vertical circulation systems.

The influence of soil–structure interaction and foundation stiffness could also be studied in greater detail, as these parameters may alter the fundamental frequencies and damping characteristics of the building. Including foundation design and stiffness in the structural model could provide a more realistic representation of the building’s dynamic response.

Another relevant area for future investigation could be the evaluation of a wider range of stabilising systems. Examples include bracing systems placed on all sides of the building, diagrid structures, and hybrid bracing solutions combining timber, steel, and concrete. Comparing these systems could provide further insight into efficient structural strategies for reducing wind-induced accelerations and improving serviceability performance in tall hybrid buildings.

Finally, future studies could apply the full wind load calculation procedure according to SS-EN 1991-1-4:2026. In this project, SS-EN 1991-1-4:2005 together with its national annex was used to compute the along-wind pressure profile. The results may differ if the calculations are instead performed according to SS-EN 1991-1-4:2026.

5 Conclusion

The objectives of this thesis were to investigate the dynamic response of a hybrid timber-concrete high-rise office building. The findings demonstrate that increasing stiffness through bracing or core thickening most effectively reduces combined acceleration. In contrast, additional mass primarily influences along-wind response, with its efficacy depending heavily on the ISO 10137 threshold below 1 Hz. Furthermore, altering aspect ratios in certain building configurations induces conflicting behaviour by mitigating translational actions while increasing torsional response. This highlights the importance of evaluating all response modes simultaneously when optimising tall hybrid buildings.

Along-wind acceleration governed the dynamic response across all analysed configurations. While all response modes increased exponentially with height, their relative influence depended strongly on the structural configuration. Aspect ratio changes and added mass caused the greatest variation. Thus, assessing along-wind response alone can severely underestimate across-wind and torsional actions under certain geometric configurations.

Wind-induced accelerations governed the design of timber-concrete hybrid buildings at all investigated heights, while material strength became critical only at greater heights. Distributed slab mass generally proved more efficient than increasing core stiffness alone. However, at 30 storeys, the substantial concrete demand required to satisfy comfort criteria caused the ULS compression utilisation ratio to become significant as well for this building layout.

The timber-concrete hybrid system was found to be most structurally efficient at heights below 20 storeys, where a balanced material composition provides sufficient strength and stiffness with limited concrete use. Beyond 25 storeys, the increasing dependence on concrete to control wind-induced accelerations reduces the overall efficiency of the hybrid concept.

In summary, all response modes must be evaluated and combined to obtain a complete understanding of the dynamic response of tall hybrid buildings. The governing factor for the analysed configurations was the serviceability limit state. Furthermore, beyond 20 storeys, serviceability requirements lead to a reduction in structural efficiency of the hybrid building concept, as meeting the comfort criteria requires disproportionately large structural dimensions.

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Appendix

A Verifications

A.1 Beams and Columns

This appendix contains the Python script implemented in a Google Colab notebook and used for the beam and column design verification.

✓ Beam Design

- Bending verification with lateral torsional buckling
- Shear verification
- Deformation SLS

Beam 1: inner beam with slabs on each side

Beam 2: facade beam supporting one slab and the facade

```
import numpy as np
digits = 2
```

```
#Floor plan
x_grid_1_m = 4.8      # [m] Habitat 7 floor plan x-distance long
x_grid_2_m = 2.8      # [m] Habitat 7 floor plan x-distance short
y_grid_1_m = 6.2      # [m] Habitat 7 floor plan y-distance long
y_grid_2_m = 4.8      # [m] Habitat 7 floor plan y-distance short

g = 9.81              # [m/s2] Gravity
```

```
# Geometrical properties
h_beam_m = 0.495      # [m] Height of beam
b_beam_m = 0.14*2     # [m] Width of beam
l_beam_m = x_grid_1_m # [m] Span of beam
s_beam_1_m = x_grid_1_m # [m] Spacing 1 of the beam
s_beam_2_m = y_grid_1_m # [m] Spacing 2 of the beam
W_beam_m3 = (b_beam_m*h_beam_m**2)/6 # [m3] Sectional modulus of beam
I_beam_m4 = (b_beam_m*h_beam_m**3)/12 # [m4] Moment of inertia of beam
```

```
# Material parameters
# From Design of Timber Structures, Volume 2, Svenskt Trä
# ASSUMED GL30c
f_mk = 30e6           # [Pa] Bending parallel to grain
f_vk = 3.5e6          # [Pa] Shear
f_rk = 1.2e6          # [Pa] Rolling shear
E_0_05 = 10800e6      # [Pa] Elastic modulus 0.05
E_mean = 13000e6      # [Pa] Mean elastic modulus
G_0_05 = 540e6        # [Pa] Shear modulus 0.05
G_mean = 650e6        # [Pa] Shear modulus 0.05
rho_mean = 430        # [kg/m3] Density
```

```
# Safety factors
# Partial safety factor, permanent load (EN 1990:2002, Table A1.2(A))
gamma_G = 1.35
# Partial safety factor, variable load (EN 1990:2002, Table A1.2(A))
gamma_Q = 1.5
psi_0_office = 0.7     # EKS 12, Table B-1
psi_2_office = 0.3     # EKS 12, Table B-1
psi_0_snow = 0.6       # EKS 12, Table B-1

gamma_m = 1.25         # Partial coefficient for GL (EN 1995-1-1, 2.4.1)
k_mod = 0.8           # Medium load duration, SC-1 (EN 1995-1-1, 3.1.3)
```

```
# Accounts for service class and moisture for deformation, EN 1995-1-1:2004, 3.1.4
k_def = 0.6
```

```
# Size effect
# EC5, Eq.(3.1)
if h_beam_m < 0.6:
    k_h = min((600 / (h_beam_m * 1000))**0.1, 1.1)
else:
    k_h = 1
```

```
# Design values for material parameters
f_md = (f_mk*k_mod*k_h)/gamma_m # [Pa] Bending parallel to grain
f_vd = (f_vk*k_mod)/gamma_m # [Pa] Shear
f_rd = (f_rk*k_mod)/gamma_m # [Pa] Rolling shear
E_mean_fin = E_mean/(1+k_def)
```

```
# Dead load
# ASSUMED [N/m] Dead load of facade + dead load of slab extension along the beam
g_k_facade = 3e3+1.5e3
g_k_floor = 2.89e3 # [N/m2] Dead load of floor build up
#g_k_floor = 25e3*0.55 # [N/m2] Dead load of floor build up

# [N/m] Dead load of floor for beam 1
g_k_floor_1 = g_k_floor * (s_beam_1_m/2 + s_beam_2_m/2)
# [N/m] Dead load of floor for beam 2
g_k_floor_2 = g_k_floor * (s_beam_1_m/2)
g_k_beam = (b_beam_m*h_beam_m*rho_mean * g) # [N/m] Self weight of beam

# Imposed load
q_k_office = 2.5e3 + 0.5e3 # [kN/m2] Live load for office + partition walls
# [N/m] Live load for beam 1
q_k_office_1 = q_k_office * (s_beam_1_m/2 + s_beam_2_m/2)
# [N/m] Live load for beam 1 office + slab extension
q_k_office_2 = q_k_office * (s_beam_1_m/2 ) + 1.5

# Design value for loads
q_d_uls_beam_1 = gamma_G*(g_k_floor_1 + g_k_beam) + gamma_Q*q_k_office_1
q_d_uls_beam_2 = gamma_G*(g_k_floor_1 + g_k_beam + g_k_facade) + gamma_Q*q_k_office_2

print(f"q_d_uls_beam_1 = {q_d_uls_beam_1/1000:.{digits}f}kN/m")
print(f"q_d_uls_beam_2 = {q_d_uls_beam_2/1000:.{digits}f}kN/m")

# Load effects
M_d_uls_beam_1 = (q_d_uls_beam_1*l_beam_m**2)/8
M_d_uls_beam_2 = (q_d_uls_beam_2*l_beam_m**2)/8
#print(f"M_d_uls_beam_1 = {M_d_uls_beam_1/1000:.{digits}f}kNm")
#print(f"M_d_uls_beam_2 = {M_d_uls_beam_2/1000:.{digits}f}kNm")

V_d_uls_beam_1 = (q_d_uls_beam_1*l_beam_m)/2
V_d_uls_beam_2 = (q_d_uls_beam_2*l_beam_m)/2
#print(f"V_d_uls_beam_1 = {V_d_uls_beam_1/1000:.{digits}f}kN")
#print(f"V_d_uls_beam_2 = {V_d_uls_beam_2/1000:.{digits}f}kN")

# Characteristic value for loads
q_k_sls_beam_1 = (g_k_floor_1 + g_k_beam) + q_k_office_1
q_k_sls_beam_2 = (g_k_floor_1 + g_k_beam + g_k_facade) + q_k_office_2

print(f"q_k_sls_beam_1 = {q_k_sls_beam_1/1000:.{digits}f}kN/m")
#print(f"q_k_sls_beam_2 = {q_k_sls_beam_2/1000:.{digits}f}kN/m")
```

```

q_d_uls_beam_1 = 47.00kN/m
q_d_uls_beam_2 = 39.12kN/m
q_k_sls_beam_1 = 32.98kN/m

```

✓ Bending check

EC5, 6.3.3

```

# [m] Effective length, EN 1995-1-1:2004, 6.3.3
l_ef_beam_bend_m = 0.9*l_beam_m + 2*h_beam_m

#Critical bending stress
sigma_m_crit = ((0.78*b_beam_m**2)/(h_beam_m*l_ef_beam_bend_m))*E_0_05 # [Pa]
print(f"sigma_m_crit = {sigma_m_crit:.{digits}f}")

lambda_rel_m = np.sqrt(f_mk/sigma_m_crit) # Relativ slenderness ratio
print(f"lambda_rel_m = {lambda_rel_m:.{digits}f}")

# Factor accounting for the effect of lateral buckling
if lambda_rel_m <= 0.75:
    k_crit = 1
elif lambda_rel_m > 0.75 and lambda_rel_m <= 1.4:
    k_crit = 1.56 - 0.75*lambda_rel_m
else:
    k_crit = 1/(lambda_rel_m**2)
print(f"k_crit = {k_crit:.{digits}f}")

# Design bending moment [Nm]
M_Rd_bend = f_md*W_beam_m3*k_crit
print(f"M_Rd_bend = {M_Rd_bend:.{digits}f}")
#print(f"M_Rd_bend = {(M_d_uls_beam_1/W_beam_m3)/f_md:.{digits}f}")

sigma_m_crit = 251266563.94
lambda_rel_m = 0.35
k_crit = 1.00
M_Rd_bend = 223806.66

```

```

# Utilization ratio
U_bend_1 = M_d_uls_beam_1 / M_Rd_bend
U_bend_2 = M_d_uls_beam_2 / M_Rd_bend

if U_bend_1 > 1 or U_bend_2 > 1:
    raise ValueError(
        f"Bending utilization exceeds 1.0 → U_bend_1 = {U_bend_1:.{digits}f},\
        U_bend_2 = {U_bend_2:.{digits}f}"
    )
print(f"U_bend_1 = {U_bend_1:.{digits}f}")
print(f"U_bend_2 = {U_bend_2:.{digits}f}")

U_bend_1 = 0.60
U_bend_2 = 0.50

```

✓ Shear check

EC5, 6.1.7

```

k_cr = 0.67      # For structural timber or glulam
b_ef_beam_m = k_cr * b_beam_m      # [m] Effective width for shear

# [Pa] Design shear stress for beam 1
tao_d_1 = (1.5*V_d_uls_beam_1)/(b_ef_beam_m*h_beam_m)
# [Pa] Design shear stress for beam 2
tao_d_2 = (1.5*V_d_uls_beam_2)/(b_ef_beam_m*h_beam_m)

print(f"tao_d_1 = {tao_d_1/1e6:.{digits}f}")
print(f"tao_d_2 = {tao_d_2/1e6:.{digits}f}")

```

```

tao_d_1 = 1.82
tao_d_2 = 1.52

```

```

# Utilization ratio
U_shear_1 = tao_d_1 / f_vd
U_shear_2 = tao_d_2 / f_vd

if U_shear_1 > 1 or U_shear_2 > 1:
    raise ValueError(
        f"Shear utilization exceeds 1.0 → U_shear_1 = {U_shear_1:.{digits}f}, \
        U_shear_2 = {U_shear_2:.{digits}f}"
    )
print(f"U_shear_1 = {U_shear_1:.{digits}f}")
print(f"U_shear_2 = {U_shear_2:.{digits}f}")

```

```

U_shear_1 = 0.81
U_shear_2 = 0.68

```

✓ Beam deflection check

Formulas from Svenskt Trä, volume 3

```

# Instant deformation
w_g_inst_bend_1 = (5*(g_k_beam + g_k_floor_1)*(l_beam_m**4))/(384*E_mean*I_beam_m4)
w_g_inst_bend_2 = (5*(g_k_beam + g_k_floor_2 + g_k_facade)*l_beam_m**4)/\
    (384*E_mean*I_beam_m4)

w_q_inst_bend_1 = (5*(q_k_office_1)*l_beam_m**4)/(384*E_mean*I_beam_m4)
w_q_inst_bend_2 = (5*(q_k_office_2)*l_beam_m**4)/(384*E_mean*I_beam_m4)

A_beam_shear = (5/6)*b_beam_m*h_beam_m
w_g_inst_shear_1 = ((g_k_beam + g_k_floor_1)*l_beam_m**2)/(8*G_mean*A_beam_shear)
w_g_inst_shear_2 = ((g_k_beam + g_k_floor_2 + g_k_facade)*l_beam_m**2)/(8*G_mean*\
    A_beam_shear)

w_q_inst_shear_1 = (q_k_office_1*l_beam_m**2)/(8*G_mean*A_beam_shear)
w_q_inst_shear_2 = (q_k_office_2*l_beam_m**2)/(8*G_mean*A_beam_shear)

w_inst_tot_1 = w_g_inst_bend_1 + w_g_inst_shear_1 + w_q_inst_bend_1 + \
    w_q_inst_shear_1
w_inst_tot_2 = w_g_inst_bend_2 + w_g_inst_shear_2 + w_q_inst_bend_2 + \
    w_q_inst_shear_2

```

```

if w_inst_tot_1 > l_beam_m/500 or w_inst_tot_2 > l_beam_m/500:
    raise ValueError(
        f"Deflection exceeds L/500 → w_inst_tot_1 = {w_inst_tot_1:.{digits}f}, \
        w_inst_tot_2 = {w_inst_tot_2:.{digits}f}"
    )

```

```
print(f"w_inst_tot_1 = {w_inst_tot_1*1000:.{digits}f} mm, l/500 = \
      {l_beam_m/500*1000:.{digits}f} mm")
print(f"w_inst_tot_2 = {w_inst_tot_2*1000:.{digits}f} mm")
```

```
w_inst_tot_1 = 7.46 mm, l/500 =      9.60 mm
w_inst_tot_2 = 4.35 mm
```

```
# Final deformation
w_g_fin_bend_1 = w_g_inst_bend_1*(1+k_def)
w_g_fin_bend_2 = w_g_inst_bend_2*(1+k_def)

w_q_fin_bend_1 = w_q_inst_bend_1*(1+psi_2_office*k_def)
w_q_fin_bend_2 = w_q_inst_bend_2*(1+psi_2_office*k_def)

w_g_fin_shear_1 = w_g_inst_shear_1*(1+k_def)
w_g_fin_shear_2 = w_g_inst_shear_2*(1+k_def)

w_q_fin_shear_1 = w_q_inst_shear_1*(1+psi_2_office*k_def)
w_q_fin_shear_2 = w_q_inst_shear_2*(1+psi_2_office*k_def)

w_fin_tot_1 = w_g_fin_bend_1 + w_g_fin_shear_1 + w_q_fin_bend_1 + w_q_fin_shear_1
w_fin_tot_2 = w_g_fin_bend_2 + w_g_fin_shear_2 + w_q_fin_bend_2 + w_q_fin_shear_2
```

```
if w_fin_tot_1 > l_beam_m/300 or w_inst_tot_2 > l_beam_m/300:
    raise ValueError(
        f"Deflection exceeds L/300 → w_fin_tot_1 = {w_fin_tot_1:.{digits}f}, \
          w_fin_tot_2 = {w_fin_tot_2:.{digits}f}"
    )
print(f"w_fin_tot_1 = {w_fin_tot_1*1000:.{digits}f} mm, l/300 = \
      {l_beam_m/300*1000:.{digits}f} mm")
print(f"w_fin_tot_2 = {w_fin_tot_2*1000:.{digits}f} mm")
```

```
w_fin_tot_1 = 10.37 mm, l/300 =      16.00 mm
w_fin_tot_2 = 6.27 mm
```

✓ Column Design

- Compression and flexural buckling

Column 1: inner column with self weight, imposed loads and snow load

Column 2: facade column with self weight, imposed load, facade load and snow load

```
# Material parameters
f_mk_col = 30e6          # [Pa] Bending parallel to grain
f_c0k_col = 24.5e6       # [Pa] Compression parallel to grain
E_0_05_col = 10800e6     # [Pa] Elastic modulus, 0.05 percentile
```

```
# Dead load
g_k_floor = 25e3*0.55    # [N/m2] Dead load of floor concrete
g_k_floor = 2.89e3       # [N/m2] Dead load of floor build up CLT

# [N] Dead load of floor and installation for column 1
g_k_floor_col_1 = (x_grid_1_m*(y_grid_1_m/2 + y_grid_2_m/2))*g_k_floor
# [N] Dead load of floor and installation for column 2
g_k_floor_col_2 = (x_grid_1_m*y_grid_1_m/2)*g_k_floor
g_k_facade_col_2 = x_grid_1_m * g_k_facade      # [N] Facade dead load for column 2
```

```

g_k_beam_col = g_k_beam * x_grid_1_m          # [N] Self weight of beam on column

# Imposed load
# [N] Live load for office for column 1
q_k_office_col_1 = q_k_office * (x_grid_1_m*(y_grid_1_m/2 + y_grid_2_m/2))
# [N] Live load for office for column 2
q_k_office_col_2 = q_k_office * (x_grid_1_m*y_grid_1_m/2)

# Snow load
q_s_k = 0.8 * 1.5e3      # [N/m2] Characteristic snow load
# [N] Snow load for column 1
q_s_k_col_1 = q_s_k * (x_grid_1_m*(y_grid_1_m/2 + y_grid_2_m/2))
# [N] Snow load for column 2
q_s_k_col_2 = q_s_k * (x_grid_1_m*y_grid_1_m/2)

```

```

# -----
# COLUMN CROSS-SECTIONS (bottom → top)
# -----
n_storey_tot = 30      # Total number of storeys
n_d_storey = 5         # Division of storeys
h_col_m = 4           # [m] Height of column
rho_col = 430          # [kg/m3] Density of column

column_sections = {
    "CS1": {"b": 0.19*3, "d": 0.765, "n_storey": n_storey_tot},
    "CS2": {"b": 0.19*3, "d": 0.675, "n_storey": n_storey_tot - n_d_storey},
    "CS3": {"b": 0.165*3, "d": 0.585, "n_storey": n_storey_tot - 2*n_d_storey},
    "CS4": {"b": 0.215*2, "d": 0.495, "n_storey": n_storey_tot - 3*n_d_storey},
    "CS5": {"b": 0.09*4, "d": 0.405, "n_storey": n_storey_tot - 4*n_d_storey},
    "CS6": {"b": 0.14*2, "d": 0.315, "n_storey": n_storey_tot - 5*n_d_storey},
}

```

Reduction for number of loaded floors

EN 1991-1-1, EQ. 6.2

```

results = {}

for name, sec in column_sections.items():

    # -----
    # GEOMETRY
    # -----
    b_col_m = sec["b"]
    d_col_m = sec["d"]
    n_storey = sec["n_storey"]

    A_col_m2 = b_col_m * d_col_m
    I_col = (b_col_m * d_col_m**3) / 12
    W_col_m3 = (b_col_m * d_col_m**2) / 6
    i_col = np.sqrt(I_col / A_col_m2)

    # -----
    # SIZE EFFECT (EC5 3.1)
    # -----
    if d_col_m < 0.6:
        k_h_col = min((600 / (d_col_m * 1000))**0.1, 1.1)
    else:

```



```

k_h_col = 1

# -----
# DESIGN MATERIAL PARAMETERS
# -----
f_cd_col = (f_cd_k_col * k_mod) / gamma_m
f_md_col = (f_md_k_col * k_mod * k_h_col) / gamma_m

# -----
# DEAD LOADS (column self weight updated)
# -----
g_k_col = A_col_m2 * h_col_m * g * rho_col

# -----
# LOAD COMBINATION – COLUMN 1
# -----

alpha_n = (2+((n_storey-1)-2)*psi_0_office)/(n_storey-1)
# print(alpha_n)

# Imposed load as leading variable
q_d_uls_off_col_1 = (
    gamma_G*(g_k_col + g_k_floor_col_1 + g_k_beam_col)*n_storey
    + alpha_n*gamma_Q*q_k_office_col_1*(n_storey-1)
    + gamma_Q*psi_0_snow*q_s_k_col_1
)

# Snow load as leading variable
q_d_uls_snow_col_1 = (
    gamma_G*(g_k_col + g_k_floor_col_1 + g_k_beam_col)*n_storey
    + gamma_Q*q_s_k_col_1
    + alpha_n*gamma_Q*psi_0_office*q_k_office_col_1*(n_storey-1)
)

# Governing design load
N_d_col_1 = max(q_d_uls_off_col_1, q_d_uls_snow_col_1)

# -----
# LOAD COMBINATION – COLUMN 2
# -----

q_d_uls_off_col_2 = (
    gamma_G*(g_k_col + g_k_floor_col_2 + g_k_beam_col + g_k_facade_col_2)*\
    n_storey + alpha_n*gamma_Q*q_k_office_col_2*(n_storey-1)
    + gamma_Q*psi_0_snow*q_s_k_col_2
)

q_d_uls_snow_col_2 = (
    gamma_G*(g_k_col + g_k_floor_col_2 + g_k_beam_col + g_k_facade_col_2)*\
    n_storey + gamma_Q*q_s_k_col_2
    + alpha_n*gamma_Q*psi_0_office*q_k_office_col_2*(n_storey-1)
)

N_d_col_2 = max(q_d_uls_off_col_2, q_d_uls_snow_col_2)

# -----
# BUCKLING (EC5 6.3.2)
# -----
l_e_col_m = h_col_m
lambda_col = l_e_col_m / i_col
lambda_rel = (lambda_col / np.pi) * np.sqrt(f_cd_k_col / E_05_col)

```

```

beta_c = 0.1
k_col = 0.5 * (1 + beta_c * (lambda_rel - 0.3) + lambda_rel**2)

if lambda_rel > 0.3:
    k_c = 1 / (k_col + np.sqrt(k_col**2 - lambda_rel**2))
else:
    k_c = 1

N_Rd = f_cd_col * A_col_m2 * k_c
# -----
# IMPERFECTION MOMENT
# -----
theta_0 = 1 / 200
alpha_h = 2 / np.sqrt(h_col_m)
alpha_m = np.sqrt(0.5 * (1 + 1))

theta_i = theta_0 * alpha_h * alpha_m
e_i = theta_i * h_col_m / 2

M_i_col_1 = N_d_col_1 * e_i
M_i_col_2 = N_d_col_2 * e_i

# -----
# LATERAL BUCKLING (EC5 6.3.3)
# -----
l_ef = 0.9 * h_col_m + 2 * d_col_m
sigma_m_crit = ((0.78 * b_col_m**2) /
                 (d_col_m * l_ef)) * E_0_05_col

lambda_rel_m = np.sqrt(f_mk_col / sigma_m_crit)

if lambda_rel_m <= 0.75:
    k_crit = 1
elif lambda_rel_m <= 1.4:
    k_crit = 1.56 - 0.75 * lambda_rel_m
else:
    k_crit = 1 / (lambda_rel_m**2)

M_Rd = f_md_col * W_col_m3 * k_crit

# -----
# UTILIZATION
# -----
# Checking flexural buckling for column 1, EC5. 6.3.2
U_comp_1 = N_d_col_1 / N_Rd
# Checking flexural buckling for column 2, EC5. 6.3.2
U_comp_2 = N_d_col_2 / N_Rd
# Checking combined bending and axial compression for column 1, EC5. 6.2.4
U_tot_1 = (N_d_col_1 / N_Rd)**2 #+ M_i_col_1 / M_Rd
# Checking combined bending and axial compression for column 2, EC5. 6.2.4
U_tot_2 = (N_d_col_2 / N_Rd)**2 #+ M_i_col_2 / M_Rd

results[name] = {
    "n_storey": n_storey,
    "col_1": {
        "N_d": N_d_col_1,
        "U_comp": U_comp_1,
        "U_total": U_tot_1,
    },
    "col_2": {
        "N_d": N_d_col_2,

```

```

        "U_comp": U_comp_2,
        "U_total": U_tot_2,
    }
}

```

```

for name, r in results.items():

    b = column_sections[name]["b"]
    d = column_sections[name]["d"]
    n_storey = r["n_storey"]

    Nd1 = r['col_1']['N_d']
    Nd2 = r['col_2']['N_d']
    Uc1 = r['col_1']['U_comp']
    Uc2 = r['col_2']['U_comp']
    Ut1 = r['col_1']['U_total']
    Ut2 = r['col_2']['U_total']

    # -----
    # FAILURE CHECK (Compression + Combined)
    # -----
    if (Uc1 > 1 or Uc2 > 1 or Ut1 > 1 or Ut2 > 1):
        raise ValueError(
            f"Failure in {name} "
            f"(b = {b:.3f} m, d = {d:.3f} m, n_storey = {n_storey}) → "
            f"U_comp_col_1 = {Uc1:.3f}, "
            f"U_comp_col_2 = {Uc2:.3f}, "
            f"U_total_col_1 = {Ut1:.3f}, "
            f"U_total_col_2 = {Ut2:.3f}"
        )

    # -----
    # PRINT RESULTS
    # -----
    print("\n=====")
    print(f"{name}")
    print("-----")
    print(f"Cross-section: b = {b:.3f} m, d = {d:.3f} m")
    print(f"Number of storeys carried: {n_storey}")
    print("=====")

    print("\n--- Compression Check ---")
    #print(f"Column 1: N_d_col_1 = {Nd1/1000:.3f}kN")
    print(f"Column 1: U_comp = {Uc1:.3f}")
    #print(f"Column 1: N_d_col_2 = {Nd2/1000:.3f}kN")
    print(f"Column 2: U_comp = {Uc2:.3f}")

    print("\n--- Combined Check (Compression + Bending) ---")
    print(f"Column 1: U_total = {Ut1:.3f}")
    print(f"Column 2: U_total = {Ut2:.3f}")

```

```

=====
CS1
-----
Cross-section: b = 0.570 m, d = 0.765 m
Number of storeys carried: 30
=====

--- Compression Check ---
Column 1: U_comp = 0.879
Column 2: U_comp = 0.650

```

```

--- Combined Check (Compression + Bending) ---
Column 1: U_total = 0.773
Column 2: U_total = 0.422

=====
CS2
-----
Cross-section: b = 0.570 m, d = 0.675 m
Number of storeys carried: 25
=====

--- Compression Check ---
Column 1: U_comp = 0.827
Column 2: U_comp = 0.610

--- Combined Check (Compression + Bending) ---
Column 1: U_total = 0.684
Column 2: U_total = 0.372

=====
CS3
-----
Cross-section: b = 0.495 m, d = 0.585 m
Number of storeys carried: 20
=====

--- Compression Check ---
Column 1: U_comp = 0.875
Column 2: U_comp = 0.643

--- Combined Check (Compression + Bending) ---
Column 1: U_total = 0.766
Column 2: U_total = 0.413

=====
CS4
-----
Cross-section: b = 0.430 m, d = 0.495 m
Number of storeys carried: 15
=====

--- Compression Check ---
Column 1: U_comp = 0.894
Column 2: U_comp = 0.654

```

> .

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A.2 Lateral Deflection

This appendix contains the Python script implemented in a Google Colab notebook and used for lateral deflection verification.

✓ Lateral Deflection Verification

```
#Import libraries
import numpy as np
import math
import matplotlib.pyplot as plt

digits = 3 #Number of digits after coma
```

```
#Input Data
h_storey_m = 4 #[m] Height of the storey
n_storey = 25 #[-] Number of storey
h_building_m = h_storey_m * n_storey #[m] Height of the building
b_building_m = 54 #[m] Width of the building
d_building_m = 18 #[m] Depth of the building (

#Floor plan
x_grid_1_m = 4.8 # [m] Habitat 7 floor plan x-distance long
x_grid_2_m = 2.8 # [m] Habitat 7 floor plan x-distance short
y_grid_1_m = 6.2 # [m] Habitat 7 floor plan y-distance long
y_grid_2_m = 4.8 # [m] Habitat 7 floor plan y-distance short

z_m = np.linspace(0, h_building_m, 21) #[m] Height above the ground
#[m] DEPENDS ON TERRAIN CATEGORY. Current value for category 3. See Table 4.1
z_min_m = 5
#[m] DEPENDS ON TERRAIN CATEGORY. Current value for category 3. See Table 4.1
z_0_m = 0.3
z_max_m = 200 #[m] Defined in EC 1991-1-4 (4.5)
ro_air_kg_m3 = 1.25 #[kg/m^3] Check value with BFS

v_b_0 = 25 #[m/s] The fundamental value of the basic wind velocity

f_1_D_Hz = 0.5 # [Hz] ASSUMED First natural frequency of along wind vibration
k_mode = 1 # ASSUMED Mode shape power exponent, compare to beta_mode_shape

delta_damping = 0.062 # The logarithmic decrement of damping from EC
xi_D = delta_damping/ (2 * np.pi) # Damping ratio for 1 year the along wind vibration

E_concrete_MPa = 33000 # [MPa] Elastic modulus for C30/37 concrete

#Check if z values are in the range defined in EC
if np.any(z_m > z_max_m):
    raise ValueError("z_m exceeds z_max according to EC 1991-1-4")

print("Building is divided into following steps heightwise z_m=", [f"{x:.{digits}f} m" for

Building is divided into following steps heightwise z_m= ['0.000 m', '5.000 m', '10.000 m',
```

› Load calculation

↳ 46 cells hidden

✓

Lateral Deflection Calculation for Elevator Shaft

Moment of inertia for a elevator shaft

$$I_x = \frac{bh^3}{12} - \frac{b_h h_h^3}{12}$$

```
#Elevator shaft properties:
t_core_m = 0.25      # [m] ASSUMED: Thickness of core
b_core_m = 4.8*4      # [m] ASSUMED: Width of core
d_core_m = 4.8        # [m] ASSUMED: Depth of core
I_core_m4 = ((b_core_m + t_core_m) * (d_core_m + t_core_m)**3) / 12 - \
            ((b_core_m - t_core_m) * (d_core_m - t_core_m)**3) / 12

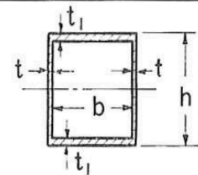
#Midpoint of each vertical interval
force_loc = ((z_m[:-1] + z_m[1:])) / 2
w_array = np.zeros(len(force_loc))
```

Using integration, WITH shear coefficients

THIN - WALLED BOX SECTION

$$K = \frac{10(1+\nu)(1+3m)^2}{(12+72m+150m^2+90m^3) + \nu(11+66m+135m^2+90m^3) + 10n^2((3+\nu)m+3m^2)}$$

$$\text{WHERE } m = bt_1/ht, \quad n = b/h$$



```
def thin_walled_box_K(b, h, t, t1, nu):

    # Dimensionless parameters
    m = (b * t1) / (h * t)
    n = b / h

    numerator = 10 * (1 + nu) * (1 + 3*m)**2

    denominator = (
        (12 + 72*m + 150*m**2 + 90*m**3)
        + nu * (11 + 66*m + 135*m**2 + 90*m**3)
        + 10 * n**2 * ((3 + nu)*m + 3*m**2)
    )

    K = numerator / denominator
    return K
```

```
#Hollow section with shear correction
# 1. Properties (Solid Section Example)
E_concrete_MPa = 33000
G_concrete_MPa = E_concrete_MPa / (2 * (1 + 0.2))
I_core_m4 = ((b_core_m + t_core_m) * (d_core_m + t_core_m)**3) / 12 - \
            ((b_core_m - t_core_m) * (d_core_m - t_core_m)**3) / 12
A_total = ((b_core_m + t_core_m) * (d_core_m + t_core_m)) - \
            ((b_core_m - t_core_m) * (d_core_m - t_core_m))
k_shear = 1 # Shear correction factor
k_shear = thin_walled_box_K((19.2-0.25), (4.8+0.25), 0.25, 0.25, 0.2)

EI = E_concrete_MPa * 1e6 * I_core_m4
```

```

GA_eff = G_concrete_MPa * 1e6 * A_total * k_shear

# 2. Internal Forces (Nodal)
# Calculate Shear (V) at each node by summing segment loads from top down
segment_loads = F_per_part # Total force [N] per 5m segment
V_internal_nodes = np.zeros(len(z_m))
V_internal_nodes[:-1] = np.cumsum(segment_loads[:-1])[:-1]
V_internal_nodes[-1] = 0 # Shear at the very top is zero

# Calculate Moment (M) at each node using average shear per segment
M_internal_nodes = np.zeros(len(z_m))
for i in range(len(z_m)-2, -1, -1):
    V_avg = (V_internal_nodes[i] + V_internal_nodes[i+1]) / 2
    M_internal_nodes[i] = M_internal_nodes[i+1] + V_avg * z_d

# 3. Calculate Rotation (Slope) Increments
# Average moment in each segment
M_avg_segments = (M_internal_nodes[:-1] + M_internal_nodes[1:]) / 2
slope_increments = (M_avg_segments * z_d) / EI

# 4. Calculate Total Deflection (w)
# We integrate the slope and add the direct shear deformation
slope_at_nodes = np.zeros(len(z_m))
deflection_nodes = np.zeros(len(z_m))

for i in range(1, len(z_m)):
    # Cumulative Slope (theta)
    slope_at_nodes[i] = slope_at_nodes[i-1] + slope_increments[i-1]

    # Deflection increment (Average Slope + Direct Shear Strain)
    avg_slope = (slope_at_nodes[i-1] + slope_at_nodes[i]) / 2
    shear_strain = V_internal_nodes[i-1] / GA_eff

    deflection_nodes[i] = deflection_nodes[i-1] + (avg_slope * z_d) + (shear_strain * z_d)

w_top_mm = deflection_nodes[-1] * 1000
print(f"Total along-wind deflection = {w_top_mm:.3f} mm")
#Compare with FEM results
FEM_mm = 301.28 #With stiffeners

print(f"FEM-design:")
print(f"FEM_mm = {FEM_mm:.{digits}f} mm")

rel_error = (w_top_mm - FEM_mm) / FEM_mm * 100
print(f"Relative error = {rel_error:.{digits}f} % " )

Total along-wind deflection = 292.983 mm
FEM-design:
FEM_mm = 301.280 mm
Relative error = -2.754 %

```

✓ Deflection calculation for solid shaft

```

#SOLID SECTION with shear correction

# 1. Properties (Solid Section Example)
E_concrete_MPa = 33000
G_concrete_MPa = E_concrete_MPa / (2 * (1 + 0.2))
I_core_m4 = (b_core_m * d_core_m**3) / 12

```



```

A_total = b_core_m * d_core_m
k_shear = 1.2 # Shear correction factor for solid rectangle

EI = E_concrete_MPa * 1e6 * I_core_m4
GA_eff = G_concrete_MPa * 1e6 * A_total * k_shear

# 2. Internal Forces (Nodal)
# Calculate Shear (V) at each node by summing segment loads from top down
segment_loads = F_per_part # Total force [N] per 5m segment
V_internal_nodes = np.zeros(len(z_m))
V_internal_nodes[:-1] = np.cumsum(segment_loads[:-1])[:-1]
V_internal_nodes[-1] = 0 # Shear at the very top is zero

# Calculate Moment (M) at each node using average shear per segment
M_internal_nodes = np.zeros(len(z_m))
for i in range(len(z_m)-2, -1, -1):
    V_avg = (V_internal_nodes[i] + V_internal_nodes[i+1]) / 2
    M_internal_nodes[i] = M_internal_nodes[i+1] + V_avg * z_d

# 3. Calculate Rotation (Slope) Increments
# Average moment in each segment
M_avg_segments = (M_internal_nodes[:-1] + M_internal_nodes[1:]) / 2
slope_increments = (M_avg_segments * z_d) / EI

# 4. Calculate Total Deflection (w)
# We integrate the slope and add the direct shear deformation
slope_at_nodes = np.zeros(len(z_m))
deflection_nodes = np.zeros(len(z_m))

for i in range(1, len(z_m)):
    # Cumulative Slope (theta)
    slope_at_nodes[i] = slope_at_nodes[i-1] + slope_increments[i-1]

    # Deflection increment (Average Slope + Direct Shear Strain)
    avg_slope = (slope_at_nodes[i-1] + slope_at_nodes[i]) / 2
    shear_strain = V_internal_nodes[i-1] / GA_eff

    deflection_nodes[i] = deflection_nodes[i-1] + (avg_slope * z_d) + (shear_strain * z_d)

w_top_mm = deflection_nodes[-1] * 1000
print(f"Total along-wind deflection = {w_top_mm:.3f} mm")
#Compare with FEM results
FEM_mm = 97.654968 #No stiffeners

print(f"FEM-design:")
print(f"FEM_mm = {FEM_mm:.{digits}f} mm")

rel_error = (w_top_mm - FEM_mm) / FEM_mm * 100
print(f"Relative error = {rel_error:.{digits}f} % " )

```

```

Total along-wind deflection = 95.422 mm
FEM-design:
FEM_mm = 97.655 mm
Relative error = -2.287 %

```

A.3 Dynamic Properties

This appendix contains the Python script implemented in a Google Colab notebook and used for dynamic property verification.

✓ Dynamic Property Verification

```
import numpy as np

# =====
# Bending (Euler-Bernoulli, Fixed-Free)
# Torsion (Saint-Venant, Fixed-Free)
# =====

# -----
# Section definitions
# name, L[m], b[m], h[m], E[Pa], rho[kg/m3], nu, isHollow, t_wall[m]
# -----
elements = [
    ("GlulamBeam", 10.0, 0.27, 0.14, 13e9, 430, 0.20, False, 0.0),
    ("SteelTube", 10.0, 0.45, 0.25, 210e9, 7850, 0.30, True, 0.016),
    ("ConcreteCore", 10.0, 19.2, 4.8, 32.837e9, 2500, 0.2, True, 0.25)
]
```

```
# -----
# FEM Design reference results (manual input)
# mode index 0,1,2 ↔ Mode 1,2,3
# -----
fem_results = {
    "GlulamBeam": {
        "bending": [1.201, 7.465, 20.662],
        "torsion": [np.nan, np.nan, np.nan]
    },
    "SteelTube": {
        "bending": [2.964, 18.404, 50.454],
        "torsion": [62.22, 158.095, np.nan]
    },
    "ConcreteCore": {
        "bending": [0.433, 2.379, 5.665], # stiffener at every 4 m
        "torsion": [3.264, 9.763, 16.156]
        #"bending": [0.432, 2.34, 5.18415], #stiffener at every 10 m
        #"torsion": [3.231, 5.676, 8.883]
    }
}

#"bending": [0.416, 2.291, 5.127], for 1 m wall thickness
#"torsion": [2.96, 5.676, 7.786] for 1 m wall thickness

# -----
# Helper: percent difference
# -----
def percent_diff(fem, ana):
    if fem is None or np.isnan(fem):
        return np.nan
    return 100 * (fem - ana) / fem
```

✓ Bending Euler-Bernoulli

$$\text{Natural Frequency, } f_i = \frac{\lambda_i^2}{2\pi L^2} \sqrt{\frac{EI}{m}}, \quad i=1,2,3\dots \text{ Hertz}$$

✓ Torsion using previous years thesis theory

$$\text{Natural Frequency, } f_i = \frac{\lambda_i}{2\pi L} \sqrt{\frac{CG}{\rho I_p}} = \frac{\lambda_i}{2\pi L} \sqrt{\frac{G}{\rho}} \Big|_{\text{circular section, } C=I_p}, \text{ Hz, } i=1,2,3\dots$$

"For thin-walled closed cross-sections, the Saint-Venant torsional constant J may be expressed as the sum of a membrane (Bredt) and Saint-Venant wall contribution. Under the constant thickness assumption:

$$J = \frac{1}{3} t^3 p + \frac{4A_m^2 t}{p},$$

where p is the section centerline perimeter and A_m is the enclosed area. This formulation is based on classical thin-walled torsion theory (Bredt-Batho), as detailed in open-access engineering sources." — see kupdf.net Bredt-Batho and centerline integral definitions from thin-walled torsion lectures.

$$I_z = \frac{b_o h_o^3 - b_i h_i^3}{12} \quad I_y = \frac{h_o b_o^3 - h_i b_i^3}{12}$$

$$I_p = I_z + I_y$$

```
# -----
# Eigenvalue constants
# -----
lambda_EB = np.array([1.875104, 4.694091, 7.854757]) # bending, fixed-free
lambda_SV = (2*np.arange(1,4) - 1) * np.pi / 2      # torsion, fixed-free
```

```
# -----
# Table header
# -----
print(
    f"\n{'Type':<18s}{'Mode':<8s}"
    f"{'Bend Ana.[Hz]':<15s}{'Bend FEM':<12s}{'ΔBend [%]':<12s}"
    f"{'Tors Ana.[Hz]':<15s}{'Tors FEM':<12s}{'ΔTors [%]':<12s}"
)
print("-"*100)

# =====
# Loop over elements
# =====
for elem in elements:

    name, L, b, h, E, rho, nu, isHollow, t_wall = elem

    # -----
    # 1) Cross-sectional properties
```

```

# -----
if isHollow:
    b_o = b + t_wall
    b_i = b - t_wall
    h_o = h + t_wall
    h_i = h - t_wall

    A = b_o*h_o - b_i*h_i
    Iz = (b_o*h_o**3 - b_i*h_i**3) / 12
    Iy = (h_o*b_o**3 - h_i*b_i**3) / 12
    Jp = Iz + Iy
    #Jp = ((b_o + h_o)**3)*t_wall

    # Saint-Venant torsion constant (thin-wall)
    Bm = b_o
    Dm = h_o
    t = t_wall

    pm = 2*((Bm - t) + (Dm - t))
    Am = (Bm - t)*(Dm - t)

    #Jt = (2*t*Bm**2*Dm**2)/(Bm + Dm)
    Jt = (1/3)*t**3*pm + 2*(2*Am*t/pm)*Am

else:
    A = b*h
    Iz = b*h**3 / 12
    Iy = h*b**3 / 12
    Jp = Iz + Iy

    # Approximate solid torsion constant
    #b1 = max(b, h)
    #h1 = min(b, h)
    #Jt = b1*h1**3 * (1/3 - 0.21*(h1/b1)*(1 - h1**4/(12*b1**4)))

# Shear modulus
G = E / (2*(1 + nu))

# -----
# 2) Analytical bending frequencies (Euler-Bernoulli)
# -----
f_b = (lambda_EB**2) / (2*np.pi*L**2) * np.sqrt(E*Iz/(rho*A))

# -----
# 3) Analytical torsion frequencies (Saint-Venant)
# -----
if isHollow:
    f_t = (lambda_SV/(2*np.pi*L)) * np.sqrt(G*Jt/(rho*Jp))
else:
    f_t = np.full(3, np.nan)

# -----
# 4) Print results
# -----
for n in range(3):

    fem_b = fem_results[name]["bending"][n]
    fem_t = fem_results[name]["torsion"][n]

    db = percent_diff(f_b[n], fem_b)
    dt = percent_diff(f_t[n], fem_t)

```

```

print(
    f"{name:<18s}"
    f"Mode{n+1:<4d}"
    f"{f_b[n]:<15.3f}"
    f"{fem_b:<12.4f}"
    f"{db:<12.2f}"
    f"{f_t[n]:<15.3f}"
    f"{fem_t:<12.4f}"
    f"{dt:<12.2f}"
)

```

Type	Mode	Bend Ana.[Hz]	Bend FEM	Δ Bend [%]	Tors Ana.[Hz]	Tors FEM
GlulamBeam	Mode1	1.243	1.2010	3.42	nan	nan
GlulamBeam	Mode2	7.793	7.4650	4.21	nan	nan
GlulamBeam	Mode3	21.820	20.6620	5.31	nan	nan
SteelTube	Mode1	3.163	2.9640	6.29	63.817	62.2200
SteelTube	Mode2	19.821	18.4040	7.15	191.452	158.0950
SteelTube	Mode3	55.499	50.4540	9.09	319.086	nan
ConcreteCore	Mode1	0.453	0.4330	4.51	3.243	3.2640
ConcreteCore	Mode2	2.842	2.3790	16.28	9.729	9.7630
ConcreteCore	Mode3	7.957	5.6650	28.81	16.216	16.1560

B Calculation Procedure

B.1 Along-Wind, Across-wind, Torsional and Combined Actions and Accelerations

This appendix contains the Python script implemented in a Google Colab notebook and used for along-wind, across-wind, torsional and combined wind action and acceleration computations.

Along-wind, Across-wind, Torsional and Combined Actions and Accelerations

```
#Import libraries
import numpy as np
import matplotlib.pyplot as plt
import pandas as pd

digits = 3 #Number of digits after coma
```

```
#Input data
n_storey = 20 #[-] Number of storey
#Frequencies
n_1_x_Hz = 0.302 #[Hz] Lowest along-wind freq. From FEM-design
f_1_L_Hz = 0.786 #[Hz] Lowest across-wind freq. From FEM-design
f_1_t_Hz = 0.525 #[Hz] Lowest torsional freq. From FEM-design
#Permanent load for 1 storey from FEM-design
g_FEM = 5618.1 #[kN]
#Impossed load for 1 storey from FEM-design
q_FEM = 2607 #[kN]
#Link to exported data from FEM-design for generalised polar mass moment of inertia comp.
url = 'https://github.com/KrisjanisKu/I_z_calcs/raw/refs/heads/main/ref/20_floors.xlsx'
#-----
#Changing height of the building change also github excell file!!!
h_storey_m = 4 #[m] Height of the storey
h_building_m = h_storey_m * n_storey #[m] Height of the building

b_building_m = 54.6 #[m] Width of the building (perpendicular to wind direction)
d_building_m = 18.2 #[m] Depth of the building (paralel to wind direction)

# In FEM-design only load bearing structure is modelled to, get realistic values wind load
# is calculated for real area and before adding load in FEM-design increased by coeficient
# that shows difference between real area of wall and wall in FEM-design
b_in_FEM_design_m = 53.6 #[m] Width of the building in FEM-design
d_in_FEM_design_m = 17.2 #[m] Depth of the building in FEM-design

v_b_0 = 27 #[m/s] The fundamental value of the basic wind velocity

z_m = np.linspace(0, h_building_m, 101) #[m] Vector with height above the ground

#Variables from FEM-design
#-----
zeta_fundamental_mode_shape = 1
#-----

#Logarithmical decrament of structural damping (For aroedynamic damping look further)

delta_s = 0.1 #Assumption that concrete is governing the behavior

z_min_m = 5 #[m] DEPENDS ON TERRAIN CATEGORY. Current value for category 3. See Table 4.1
z_0_m = 0.3 #[m] DEPENDS ON TERRAIN CATEGORY. Current value for category 3. See Table 4.1
z_max_m = 200 #[m] Defined in EC 1991-1-4 (4.5)
ro_air_kg_m3 = 1.25 #[kg/m^3] Check value with BFS

#Check if z values are in the range defined in EC
```



```

if np.any(z_m > z_max_m):
    raise ValueError("z_m exceeds z_max according to EC 1991-1-4")
print(f"Height of the building h_building_m = {h_building_m:.1f} m" )
print("Building is divided into following steps heightwise z_m=", \
      [f"{x:.{digits}f} m" for x in z_m])

```

Height of the building h_building_m = 80.0 m
 Building is divided into following steps heightwise z_m= ['0.000 m', '0.800 m', '1.600 m',

```

# Export data from Github link with corresponding exported excell file
# For each storey height/configuration there are different links and files
# Access to right file needs to be changed manually

```

```

df = pd.read_excel(url, engine='openpyxl')
df.columns = df.iloc[0]
df = df.drop([0, 1])
df = df.drop(columns=['Level'], errors='ignore')
df = df.reset_index(drop=True)
cols_to_fix = ['Mass', 'x', 'y', 'z']
for col in cols_to_fix:
    df[col] = (
        df[col]
        .astype(str)
        .str.replace(',', '.', regex=False) # handle comma decimals
    )
    df[col] = pd.to_numeric(df[col], errors='coerce')
df_cog_sheet = pd.read_excel(
    url,
    sheet_name='2-Total-Level masses',
    skiprows=1
)
def to_float(val):
    return pd.to_numeric(str(val).replace(',', '.'), errors='coerce')
x_cog = to_float(df_cog_sheet.at[1, 'x'])
y_cog = to_float(df_cog_sheet.at[1, 'y'])
z_cog = to_float(df_cog_sheet.at[1, 'z'])
tot_weight_t = to_float(df_cog_sheet.at[1, 'Mass'])
df['Mass_kg'] = df['Mass'] * 1000
df['r_sq'] = (df['x'] - x_cog)**2 + (df['y'] - y_cog)**2
df['Iz_node'] = df['Mass_kg'] * df['r_sq']
Iz_total = df['Iz_node'].sum()
print("x_cog, y_cog:", x_cog, y_cog)
print("Iz_total:", Iz_total)

Iz_per_m = Iz_total / h_building_m
dz = z_m[1] - z_m[0]
print(f"Iz_total:= {Iz_total:.1e} m" )

```

```

x_cog, y_cog: 26.786 8.605
Iz_total: 2772630944.4462028
Iz_total:= 2.8e+09 m

```

Along-wind, across-wind and torsional actions and accelerations

```

#Define parameter for looping
zeta_fundamental_mode_shape = 1
loop_zeta_fundamental_mode_shape = [1.0]

```

```

results_along_acceleration = np.zeros_like(loop_zeta_fundamental_mode_shape)
"""
Code
"""
count = 1
for i, zeta_fundamental_mode_shape in enumerate(loop_zeta_fundamental_mode_shape):
    # print(count)
    count = count+1

    #-----Code-----
    if 0.6*h_building_m < z_min_m:
        z_s_m = z_min_m
    else:
        z_s_m = 0.6*h_building_m
    # The basic wind velocity
    c_dir = 1
    c_season = 1
    v_b = v_b_0 * c_dir * c_season

    # To get acceleration every 1 year, T_a value for 2 years needs to be used
    T_a = 2    #[Years]
    v_b_1y = 0.75 * v_b * np.sqrt(1 - 0.2 * np.log(- np.log((1-1/T_a))))

    z_o_II_m = 0.05 #Reference value
    k_r = 0.19 * (z_0_m/z_o_II_m)**0.07

    #Calculate Roughness factor for reference height
    #According the formula if z < z_min, eq. is computed with z_min
    z_s_eff_m = np.maximum(z_s_m, z_min_m)
    cr_z_s = k_r * np.log(z_s_eff_m/z_0_m)

    # Roughness factor for all z values:
    #According the formula if z < z_min, eq. is computed with z_min
    z_eff_m = np.maximum(z_m, z_min_m)
    cr_z = k_r * np.log(z_eff_m/z_0_m)

    c_o_z = 1                #Effects of terrain orography are neglected
    c_o_z_s = c_o_z          #Same for the reference height

    v_m_z_s_50y = cr_z_s * c_o_z * v_b    #Mean wind velocity at reference height z_s
    v_m_z_50y = cr_z * c_o_z * v_b        #Mean wind velocity at z

    #For comfort calculations:
    v_m_z_s_1y = cr_z_s * c_o_z * v_b_1y  #Mean wind velocity at reference height z_s
    v_m_z_1y = cr_z * c_o_z * v_b_1y      #Mean wind velocity at z

    k_I = 1 # The turbulence factor

    I_v_z_s = 1 / (c_o_z_s * np.log(z_s_eff_m / z_0_m)) #at reference height z_s
    I_v_z = 1 / (c_o_z * np.log(z_eff_m / z_0_m))        #at z

    q_b_50y_Pa = 1 / 2 * ro_air_kg_m3 * v_b**2 #[Pa] The basic velocity pressure

    #For comfort calculations:
    q_b_1y_Pa = 1 / 2 * ro_air_kg_m3 * v_b_1y**2 #[Pa] The basic velocity pressure

    ##### WIND TURBULANCE #####
    #Reference height at which the fundamental value of the basic wind velocity is measured
    h_ref = 10 #[m]

    #The background factor allowing for the lack of full correlation of the pressure on structure
    B_squared = np.exp(-0.05 * (h_building_m / h_ref) + (1 - b_building_m / h_building_m) *

```

```

(0.04 + 0.01 * ( h_building_m / h_ref)))

# Factors are calculated using mean wind speed at the top of the building
v_m_h_50y = v_m_z_50y[-1] =
v_m_h_1y = v_m_z_1y[-1]

gamma_c_50y = 150 * n_1_x_Hz / v_m_h_50y
phi_h_50y = 1 / (1 + (2 * n_1_x_Hz * h_building_m) / v_m_h_50y)
phi_b_50y = 1 / (1 + (3.2 * n_1_x_Hz * b_building_m) / v_m_h_50y)

#For 1 year reference period:
gamma_c_1y = 150 * n_1_x_Hz / v_m_h_1y
phi_h_1y = 1 / (1 + (2 * n_1_x_Hz * h_building_m) / v_m_h_1y)
phi_b_1y = 1 / (1 + (3.2 * n_1_x_Hz * b_building_m) / v_m_h_1y)

F_wind_vibrations_50y = 4 * gamma_c_50y / (1 + 70.8 * gamma_c_50y**2)**(5/6)

#For 1 year reference period:
F_wind_vibrations_1y = 4 * gamma_c_1y / (1 + 70.8 * gamma_c_1y**2)**(5/6)

phi_1_z = (z_m / h_building_m)**zeta_fundamental_mode_shape # at z locations

#Permanent load
# Mass per area computed from FEM-design load
g_perm_kN_m2 = g_FEM / (b_building_m * d_building_m)#[kN/m2]
#Imposed load
psi_2 = 0.3 #Table B-1 EKS12: Offices (B)
q_imposed_quasi_kN_m2 = q_FEM / (b_building_m * d_building_m) * psi_2

#Combined
w_kN_m2 = g_perm_kN_m2 + q_imposed_quasi_kN_m2 #[kN/m2] Mass per area
g = 9.81 #[m/s^2] Gravitational acceleration
#Mass per unit length
m_line = (w_kN_m2 * 1000) * (b_building_m * d_building_m) / g / h_storey_m #[kg/m]
m_building_total = m_line * h_building_m #[kg] Total mass of the building

#Calculation of equivalent mass
#Mass vector with assumption that all floors have the same mass
m_vector = m_line * np.ones_like(z_m)

#Integration:
num = np.trapezoid(m_vector * phi_1_z**2, z_m)
den = np.trapezoid(phi_1_z**2, z_m)

m_e_kg_m = num / den #[kg/m] Equivalent mass

ratio_d_b = d_building_m / b_building_m

if ratio_d_b < 0.2:
    c_f_0 = 2.0
elif ratio_d_b > 10.0:
    c_f_0 = 0.9
else:
    # Figure is logarithmic -> logarithmic interpolation is being used
    x_pts = np.array([0.2, 0.6, 0.7, 1.0, 2.0, 5.0, 10.0])
    y_pts = np.array([2.0, 2.35, 2.4, 2.1, 1.65, 1.0, 0.9])

    c_f_0 = np.interp(np.log10(ratio_d_b), np.log10(x_pts), y_pts)

r_corners_m = 0 #[m] Radius of corners
ratio_r_b = r_corners_m / b_building_m

```

```

if ratio_r_b <= 0.2: #Interpolate
    psi_r = 1 + ratio_r_b / 0.2 * (0.5 - 1)
else:
    psi_r = 0.5
#Check height of the building and compute lambda

if h_building_m > 50:
    lamda_eff_slender_0 = 1.4 * (h_building_m / b_building_m)

elif h_building_m > 15: #Interpolate between 1.4 and 2
    factor_eff_slender = 2.0 - (2.0 - 1.4) * (h_building_m - 15.0) / (50.0 - 15.0)
    lamda_eff_slender_0 = factor_eff_slender * (h_building_m / b_building_m)
else:
    lamda_eff_slender_0 = 2 * (h_building_m / b_building_m)

lamda_eff_slender = min(lamda_eff_slender_0, 70)

#It is assumed that building is fully "filled" solidity ratio = 1
lam_pts = np.array([1.0, 10.0, 70.0])
psi_pts = np.array([0.60, 0.69, 0.915])

#As curve is logarithmic use logarithmic interpolation
loglam = np.log10(lamda_eff_slender)
psi_lambda = np.interp(loglam, np.log10(lam_pts), psi_pts)

c_f = c_f_0 * psi_r * psi_lambda

delta_a_50y = c_f * ro_air_kg_m3 * b_building_m * v_m_z_s_50y / (2 * n_1_x_Hz * m_e_kg_m)
#For 1 year reference period:
delta_a_1y = c_f * ro_air_kg_m3 * b_building_m * v_m_z_s_1y / (2 * n_1_x_Hz * m_e_kg_m)

delta_damping_50y = delta_s + delta_a_50y
#For 1 year reference period:
delta_damping_1y = delta_s + delta_a_1y

R_squared_50y = 2 * np.pi * F_wind_vibrations_50y * phi_b_50y * phi_h_50y / \
    (delta_a_50y + delta_s)
#For 1 year reference period:
R_squared_1y = 2 * np.pi * F_wind_vibrations_1y * phi_b_1y * phi_h_1y / \
    (delta_a_1y + delta_s)

nu_50y = n_1_x_Hz * np.sqrt(R_squared_50y) / np.sqrt(B_squared + R_squared_50y)
#For 1 year reference period:
nu_1y = n_1_x_Hz * np.sqrt(R_squared_1y) / np.sqrt(B_squared + R_squared_1y)

T_period_s = 600 #[s] Represents 10 min time period that is used for v_b_0 definition

k_p_50y = np.sqrt(2 * np.log(nu_50y * T_period_s)) + 0.6 / \
    np.sqrt(2 * np.log(nu_50y * T_period_s))
#For 1 year reference period:
k_p_1y = np.sqrt(2 * np.log(nu_1y * T_period_s)) + 0.6 / \
    np.sqrt(2 * np.log(nu_1y * T_period_s))

#The peak velocity pressure using dynamic peak facotr

#at reference height z_s
q_p_z_s_50y = (1 + 2 * k_p_50y * I_v_z_s) * (k_r * np.log(z_s_eff_m / z_0_m) \
    * c_o_z_s)**2 * q_b_50y_Pa

#at z
q_p_z_50y = (1 + 2 * k_p_50y * I_v_z) * (k_r * np.log(z_eff_m / z_0_m) \
    * c_o_z)**2 * q_b_50y_Pa

#For 1 year reference period:

```

```

#at reference height z_s
q_p_z_s_1y = (1 + 2 * k_p_1y * I_v_z_s) * (k_r * np.log(z_s_eff_m / z_0_m) \
        * c_o_z_s)**2 * q_b_1y_Pa

#at z
q_p_z_1y = (1 + 2 * k_p_1y * I_v_z) * (k_r * np.log(z_eff_m / z_0_m) * c_o_z)**2 \
        * q_b_1y_Pa

cs_cd_50y = (1 + 2 * k_p_50y * I_v_z_s * np.sqrt(B_squared + R_squared_50y)) / \
        (1 + 2 * k_p_50y * I_v_z_s)

#For 1 year reference period:
cs_cd_1y = (1 + 2 * k_p_1y * I_v_z_s * np.sqrt(B_squared + R_squared_1y)) / \
        (1 + 2 * k_p_1y * I_v_z_s)

F_w_dynamic_50y_Pa = cs_cd_50y * c_f * q_p_z_50y
#For 1 year reference period:
F_w_dynamic_1y_Pa = cs_cd_1y * c_f * q_p_z_1y

#Acceleration
q_m_1y = 0.5 * ro_air_kg_m3 * v_m_z_1y[-1]**2      #Mean wind pressure
phi_1_x_z = (z_m / h_building_m)**1.5

sigma_x_z = (3 * I_v_z[-1] * np.sqrt(R_squared_1y) * q_m_1y * b_building_m * c_f * \
        phi_1_x_z) / m_e_kg_m
X_max_z = k_p_1y * sigma_x_z
X_max = X_max_z[-1]

results_along_acceleration[i] = X_max

# print(f"The maximum along-wind acceleration at the top of the building X_max = " )
# print(results_along_acceleration)

#####
#####          TORSIONAL VIBRATIONS          #####
#####

#Torsional damping
delta_a_50y_t = c_f * ro_air_kg_m3 * b_building_m * v_m_z_s_50y / (2 * f_1_t_Hz * m_e_kg_m)
delta_a_1y_t = c_f * ro_air_kg_m3 * b_building_m * v_m_z_s_1y / (2 * f_1_t_Hz * m_e_kg_m)

delta_damping_50y_t = delta_s + delta_a_50y_t
delta_damping_1y_t = delta_s + delta_a_1y_t

v_m_h_50y = v_m_z_50y[-1]
v_m_h_1y = v_m_z_1y[-1]

#according to EN-1991-1-4:2026 Annex F
# Reduced mean wind speed at height h
v_star_m_50y = v_m_h_50y / (f_1_t_Hz * np.sqrt(b_building_m * d_building_m))
v_star_m_1y = v_m_h_1y / (f_1_t_Hz * np.sqrt(b_building_m * d_building_m))

db = d_building_m / b_building_m
beta_M_1 = (db + 3.6) / (db**2 - 5.1 * db + 9.1) + 0.14/db + 0.14
beta_M_2 = (0.044 * db**2 - 0.0064) / (db**4 - 0.26 * db**2 + 0.1) + 0.2
K_M_1 = (-1.1 * db + 0.97) / (db**2 + 0.85 * db + 3.3) + 0.17
K_M_2 = (0.077 * db - 0.16) / (db**2 - 0.96 * db + 0.42) + (0.35 / db) + 0.095
l = max(d_building_m, b_building_m)
S_M_1 = (0.14 * K_M_1**2 * d_building_m * (b_building_m**2 + d_building_m**2)**2
        * 4.5**(2*beta_M_1)) / (np.pi * l**2 * b_building_m**3)
S_M_2 = (0.14 * K_M_2**2 * d_building_m * (b_building_m**2 + d_building_m**2)**2
        * 6**(2*beta_M_2)) / (np.pi * l**2 * b_building_m**3)

```

```

if v_star_m_50y <= 4.5:
    S_M_50y = S_M_1 * (v_star_m_50y / 4.5)**(2*beta_M_1)
elif v_star_m_50y > 4.5 and v_star_m_50y < 6:
    S_M_50y = S_M_1 * np.exp(3.5 * np.log(S_M_2 / S_M_1) * np.log(v_star_m_50y / 4.5))
elif v_star_m_50y >= 6 and v_star_m_50y <= 10:
    S_M_50y = S_M_2 * (v_star_m_50y / 6)**(2*beta_M_2)
else:
    raise ValueError("v_star_m_50y is too high!!!")

if v_star_m_1y <= 4.5:
    S_M_1y = S_M_1 * (v_star_m_1y / 4.5)**(2*beta_M_1)
elif v_star_m_1y > 4.5 and v_star_m_1y < 6:
    S_M_1y = S_M_1 * np.exp(3.5 * np.log(S_M_2 / S_M_1) * np.log(v_star_m_1y / 4.5))
elif v_star_m_1y >= 6 and v_star_m_1y <= 10:
    S_M_1y = S_M_2 * (v_star_m_1y / 6)**(2*beta_M_2)
else:
    raise ValueError("v_star_m_1y is too high!!!")

R_squared_M_50y = np.pi**2 * S_M_50y / (2 * delta_damping_50y_t)
R_squared_M_1y = np.pi**2 * S_M_1y / (2 * delta_damping_1y_t)

t_period_s = 600 #[s] Represents 10 min time period that is used for v_b_0 definition

g_M_prime = np.sqrt(2 * np.log(f_1_t_Hz * t_period_s)) + 0.577 / \
    np.sqrt(2 * np.log(f_1_t_Hz * t_period_s))

if g_M_prime >= 2.8:
    g_M = g_M_prime
else:
    g_M = 2.8

c_dM_50y = g_M * np.sqrt(1 + R_squared_M_50y)
c_dM_1y = g_M * np.sqrt(1 + R_squared_M_1y)

c_M = b_building_m / d_building_m * (0.0066 + 0.015 * (d_building_m / b_building_m)**2) \
    **0.78

q_m_h_50y = 0.5 * ro_air_kg_m3 * v_m_h_50y**2
q_m_h_1y = 0.5 * ro_air_kg_m3 * v_m_h_1y**2

m_M_z_50y = 1.8 * q_m_h_50y * c_M * b_building_m * d_building_m * (z_m / h_building_m) \
    * c_dM_50y
m_M_z_1y = 1.8 * q_m_h_1y * c_M * b_building_m * d_building_m * (z_m / h_building_m) \
    * c_dM_1y

#Accelerations

phi_1_z = (z_m / h_building_m)**zeta_fundamental_mode_shape # at z locations
I_M_integrand = Iz_per_m * dz * (phi_1_z**2)
I_M = np.sum(I_M_integrand)
sigma_a_M_z_1y = (0.3 * ro_air_kg_m3 * v_m_h_1y**2 * b_building_m * d_building_m * \
    h_building_m) / I_M * c_M * np.sqrt(R_squared_M_1y) * phi_1_z[-1] * phi_1_z
a_p_M_z_1y = g_M * sigma_a_M_z_1y #rad/s^2
a_p_M_h_1y = a_p_M_z_1y[-1] #rad/s^2
distance_torsion_m = np.sqrt((b_building_m/2)**2 + (d_building_m/2)**2)
a_p_M_m_s2_1y = a_p_M_z_1y * distance_torsion_m #m/s^2
a_p_M_max_m_s2_1y = a_p_M_m_s2_1y[-1] #m/s^2
# print(f"Torsional acceleration on top of the building {a_p_M_max_m_s2_1y = :.5f} m/s^2")

#####
##### ACROSS-WIND VIBRATIONS #####

```

```
#####

#Across-wind damping
delta_a_1y_l = c_f * ro_air_kg_m3 * b_building_m * v_m_z_s_1y / \
    (2 * f_1_L_Hz * m_e_kg_m)
delta_a_50y_l = c_f * ro_air_kg_m3 * b_building_m * v_m_z_s_50y / \
    (2 * f_1_L_Hz * m_e_kg_m)

delta_damping_50y_l = delta_s + delta_a_50y_l
delta_damping_1y_l = delta_s + delta_a_1y_l

v_m_h_50y = v_m_z_50y[-1]
v_m_h_1y = v_m_z_1y[-1]

#according to EN-1991-1-4:2026 Annex F

db = d_building_m / b_building_m
if db < 3:
    m = 1
    # print(f"db ratio is {db:.{digits}f} only values j=1 will be used")
else:
    m = 2
    # print(f"db ratio is {db:.{digits}f} values j=1;2 will be used")
k_L1 = 0.85
beta_L1 = (db**4 + 2.3 * db**2) / (2.4 * db**4 - 9.2 * db**3 + 18 * db**2 \
    + 9.5 * db - 0.15) + 0.12 / db

n_s1_50y = 0.12 * v_m_h_50y / ((1+0.38*db**2)**0.89 * b_building_m)
n_s1_1y = 0.12 * v_m_h_1y / ((1+0.38*db**2)**0.89 * b_building_m)

if m == 2:
    k_L2 = 0.02
    beta_L2 = 0.28 * db**(-0.34)
    n_s2_50y = 0.56 * v_m_h_50y / (db**0.85 * b_building_m)
    n_s2_1y = 0.56 * v_m_h_1y / (db**0.85 * b_building_m)

S_L_1_50y = (4 * k_L1 * (1 + 0.6 * beta_L1) * beta_L1) / np.pi * \
    (f_1_L_Hz / n_s1_50y)**2 / ((1 - (f_1_L_Hz / n_s1_50y)**2)**2 \
    + 4 * beta_L1**2 * (f_1_L_Hz / n_s1_50y)**2)
S_L_1_1y = (4 * k_L1 * (1 + 0.6 * beta_L1) * beta_L1) / np.pi * \
    (f_1_L_Hz / n_s1_1y)**2 / ((1 - (f_1_L_Hz / n_s1_1y)**2)**2 + 4 \
    * beta_L1**2 * (f_1_L_Hz / n_s1_1y)**2)

if m == 1:
    S_L_50y = S_L_1_50y
    S_L_1y = S_L_1_1y
else:
    S_L_2_50y = (4 * k_L2 * (1 + 0.6 * beta_L2) * beta_L2) / np.pi * (f_1_L_Hz / n_s2_50y)**2 \
        / ((1 - (f_1_L_Hz / n_s2_50y)**2)**2 + 4 * beta_L2**2 * (f_1_L_Hz / n_s2_50y)**2)
    S_L_50y = S_L_1_50y + S_L_2_50y
    S_L_2_1y = (4 * k_L2 * (1 + 0.6 * beta_L2) * beta_L2) / np.pi * (f_1_L_Hz / n_s2_1y)**2 / \
        ((1 - (f_1_L_Hz / n_s2_1y)**2)**2 + 4 * beta_L2**2 * (f_1_L_Hz / n_s2_1y)**2)
    S_L_1y = S_L_1_1y + S_L_2_1y

R_squared_L_50y = np.pi**2 * S_L_50y / (2 * delta_damping_50y_l)
R_squared_L_1y = np.pi**2 * S_L_1y / (2 * delta_damping_1y_l)

t_period_s = 600 #[s] Represents 10 min time period that is used for v_b_0 definition
g_L_prime = np.sqrt(2 * np.log(f_1_L_Hz * t_period_s)) + 0.577 / \
    np.sqrt(2 * np.log(f_1_L_Hz * t_period_s))
if g_L_prime >= 2.8:
    g_L = g_L_prime
else:
    g_L = 2.8
```

```

else.
    g_L = 2.8

q_m_h_50y = 0.5 * ro_air_kg_m3 * v_m_h_50y**2
q_m_h_1y = 0.5 * ro_air_kg_m3 * v_m_h_1y**2

c_dL_50y = g_L * np.sqrt(1 + R_squared_L_50y)
c_dL_1y = g_L * np.sqrt(1 + R_squared_L_1y)

c_L = -0.015 * db**2 + 0.14 * db + 0.03

f_L_z_50y = 3 * q_m_h_50y * c_L * b_building_m * (z_m/h_building_m) * c_dL_50y
f_L_z_1y = 3 * q_m_h_1y * c_L * b_building_m * (z_m/h_building_m) * c_dL_1y
f_L_z_h_50y = f_L_z_50y[-1]
f_L_z_h_1y = f_L_z_1y[-1]

g_perm_kN_m2 = g_FEM / (b_building_m * d_building_m)
psi_2 = 0.3 #Table B-1 EKS12: Offices (B)
q_imposed_quasi_kN_m2 = q_FEM / (b_building_m * d_building_m) * psi_2
w_kN_m2 = g_perm_kN_m2 + q_imposed_quasi_kN_m2 #[kN/m2] Mass per area
g = 9.81 #[m/s^2] Gravitational acceleration
#[kg/m] Mass per unit length
m_line = (w_kN_m2 * 1000) * (b_building_m * d_building_m) / g / h_storey_m
dz = z_m[1] - z_m[0]
m_L = np.sum(m_line * dz * phi_1_z**2)
sigma_a_L_z = (0.5 * ro_air_kg_m3 * v_m_h_1y**2 * b_building_m * h_building_m) / \
    m_L * c_L * np.sqrt(R_squared_L_1y) * phi_1_z[-1] * phi_1_z
a_p_L_z = g_L * sigma_a_L_z
a_p_L_h = a_p_L_z[-1]
# print(f"Across-wind acceleration on the top of the building {a_p_L_h = :.5f}")

#####
acc_W = X_max
acc_L = a_p_L_h
acc_T = a_p_M_max_m_s2_1y
#####

print(f"Along-wind acceleration on the top of the building {acc_W = :.3f} m/s^2")
print(f"Across-wind acceleration on the top of the building {acc_L = :.3f} m/s^2")
print(f"Torsional acceleration on the top of the building {acc_T= :.3f} m/s^2")

```

Along-wind acceleration on the top of the building $acc_W = 0.212 \text{ m/s}^2$
 Across-wind acceleration on the top of the building $acc_L = 0.038 \text{ m/s}^2$
 Torsional acceleration on the top of the building $acc_T = 0.070 \text{ m/s}^2$

✓ Combination of accelerations

```

#Interpolation from Tables= F.4
n_L = f_1_L_Hz
n_M = f_1_t_Hz
n_1 = min(n_L, n_M)

if n_L >= n_M:
    f_LM = n_L / n_M
else:
    f_LM = n_M / n_L

print(f"{n_L = :.3f}, {n_M = :.3f}, {n_1 = :.3f}")
print(f"{f_LM = :.2f}")

```



```

#Interpolation from table
from scipy.interpolate import interp1d

def get_gamma_LM(db, v_ratio, f_LM):
    # --- 1. Define the Data Table Structure ---
    f_cols = np.array([1.0, 1.1, 1.4])

    # db_points: 0.5, 1.0, 2.0
    # Note: Values < 0.5 use 0.5 data; Values > 2.0 use 2.0 data

    table_data = {
        0.5: {
            'v_pts': np.array([0.1, 0.2, 0.3, 0.6, 1.0]),
            'vals': np.array([
                [0.95, 0.61, 0.55], # <= 0.1
                [0.55, 0.67, 0.61], # 0.2
                [0.55, 0.73, 0.67], # 0.3
                [0.79, 0.79, 0.79], # 0.6
                [0.84, 0.84, 0.84] # >= 1.0
            ])
        },
        1.0: {
            'v_pts': np.array([0.1, 0.2, 0.3, 0.6, 1.0]),
            'vals': np.array([
                [0.84, 0.55, 0.55],
                [0.61, 0.55, 0.55],
                [0.55, 0.55, 0.55],
                [0.67, 0.67, 0.67],
                [0.73, 0.73, 0.73]
            ])
        },
        2.0: {
            'v_pts': np.array([0.05, 0.1]),
            'vals': np.array([
                [0.79, 0.55, 0.55], # <= 0.05
                [0.55, 0.55, 0.55] # >= 0.1
            ])
        }
    }

    # --- 2. Helper function to interpolate within a single d/b block ---
    def interp_block(db_key, target_v, target_f):
        data = table_data[db_key]
        # Clamp v_ratio to table limits
        v_clamped = np.clip(target_v, data['v_pts'].min(), data['v_pts'].max())
        f_clamped = np.clip(target_f, f_cols.min(), f_cols.max())

        # First interpolate rows (v_ratio) for each of the 3 f_LM columns
        interp_v = [interp1d(data['v_pts'], data['vals'][i], kind='linear')(v_clamped) for i in range(3)]

        # Then interpolate across the resulting f_LM values
        return interp1d(f_cols, interp_v, kind='linear')(f_clamped)

    # --- 3. Interpolate between d/b blocks ---
    # Clamp db to table limits
    db_clamped = np.clip(db, 0.5, 2.0)

    if db_clamped <= 0.5:
        return interp_block(0.5, v_ratio, f_LM)
    elif db_clamped >= 2.0:
        return interp_block(2.0, v_ratio, f_LM)
    elif 0.5 < db_clamped < 1.0:

```

```

        val05 = interp_block(0.5, v_ratio, f_LM)
        val10 = interp_block(1.0, v_ratio, f_LM)
        return interp1d([0.5, 1.0], [val05, val10])(db_clamped)
    else: # 1.0 to 2.0
        val10 = interp_block(1.0, v_ratio, f_LM)
        val20 = interp_block(2.0, v_ratio, f_LM)
        return interp1d([1.0, 2.0], [val10, val20])(db_clamped)

# --- Input variables ---
n_L = f_1_L_Hz
n_M = f_1_t_Hz
n_1 = min(n_L, n_M)
f_LM = max(n_L, n_M) / min(n_L, n_M)
n_b_v_ratio_50y = n_1 * b_building_m / v_m_h_50y
n_b_v_ratio_1y = n_1 * b_building_m / v_m_h_1y

# Calculate gamma_LM
gamma_LM_50y = float(get_gamma_LM(db, n_b_v_ratio_50y, f_LM))
gamma_LM_1y = float(get_gamma_LM(db, n_b_v_ratio_1y, f_LM))

print(f"Interpolated gamma_LM for 50 years: {gamma_LM_50y:.3f}")
print(f"Interpolated gamma_LM for 1 year: {gamma_LM_1y:.3f}")

n_L = 0.786, n_M = 0.525, n_1 = 0.525
f_LM = 1.50
Interpolated gamma_LM for 50 years: 0.825
Interpolated gamma_LM for 1 year: 0.840

```

GD (6.5)

$I_u(z) = I_v(z)$ (6.10)

```

g_u = 2.9
G_D_50y = cs_cd_50y * (1 + g_u * I_v_z)**2
G_D_1y = cs_cd_1y * (1 + g_u * I_v_z)**2
print(f"G_D_top 50y: {G_D_50y[-1]:.3f}")
print(f"G_D_top 1y : {G_D_1y[-1]:.3f}")

```

```

G_D_top 50y: 2.539
G_D_top 1y : 2.397

```

```

angle_a = np.arctan(d_building_m / b_building_m)

acc_T_x = np.sin(angle_a) * acc_T
acc_T_y = np.cos(angle_a) * acc_T

print(f"{acc_T_x = :.{3}f}, {acc_T_y = :.{3}f}")

#Acceleration combinations using vector summation:
acc_1 = np.sqrt((acc_W + (0.4 * acc_T_y))**2 + ((acc_L * 0.4) + (acc_T_x * 0.4))**2)
print(f"First combination: {acc_1 = :.{3}f}")

acc_2 = np.sqrt(((acc_W * (0.4 + 0.6 / G_D_1y[-1])) + (acc_T_y * gamma_LM_1y))**2 + \
                (acc_L + (acc_T_x * gamma_LM_1y))**2)
print(f"Second combination: {acc_2 = :.{3}f}")

acc_3 = np.sqrt(((acc_W * (0.4 + 0.6 / G_D_1y[-1])) + acc_T_y)**2 + \
                ((acc_L * gamma_LM_1y) + acc_T_x)**2)
print(f"Third combination: {acc_3 = :.{3}f}")

```

```
acc_max = max(acc_1, acc_2, acc_3)
print(f"Maximal acceleration on the top of the building: {acc_max = :.{3}f}")
```

```
acc_T_x = 0.022, acc_T_y = 0.066
First combination: acc_1 = 0.240
Second combination: acc_2 = 0.202
Third combination: acc_3 = 0.211
Maximal acceleration on the top of the building: acc_max = 0.240
```

C Additional Results

C.1 Variations of Top Concrete Slabs

To mitigate the top acceleration of the building, additional mass was introduced in the form of concrete slabs. Two approaches were assessed. First, 300 mm thick concrete slabs were added to the top one, three, and five floors of the 10-, 15-, 20-, 25-, and 30-storey buildings. The same buildings were then analysed with the top three slabs increased to a thickness of 500 mm. This allowed for a comparison of the effectiveness of increasing mass either by adding more slabs or by increasing their thickness. Finally, the influence of mass location was evaluated by placing a single 900 mm thick concrete slab at the top and comparing it with three 300 mm slabs of equal total volume. The same comparison was carried out using a 1500 mm thick slab at the top and three 500 mm slabs.

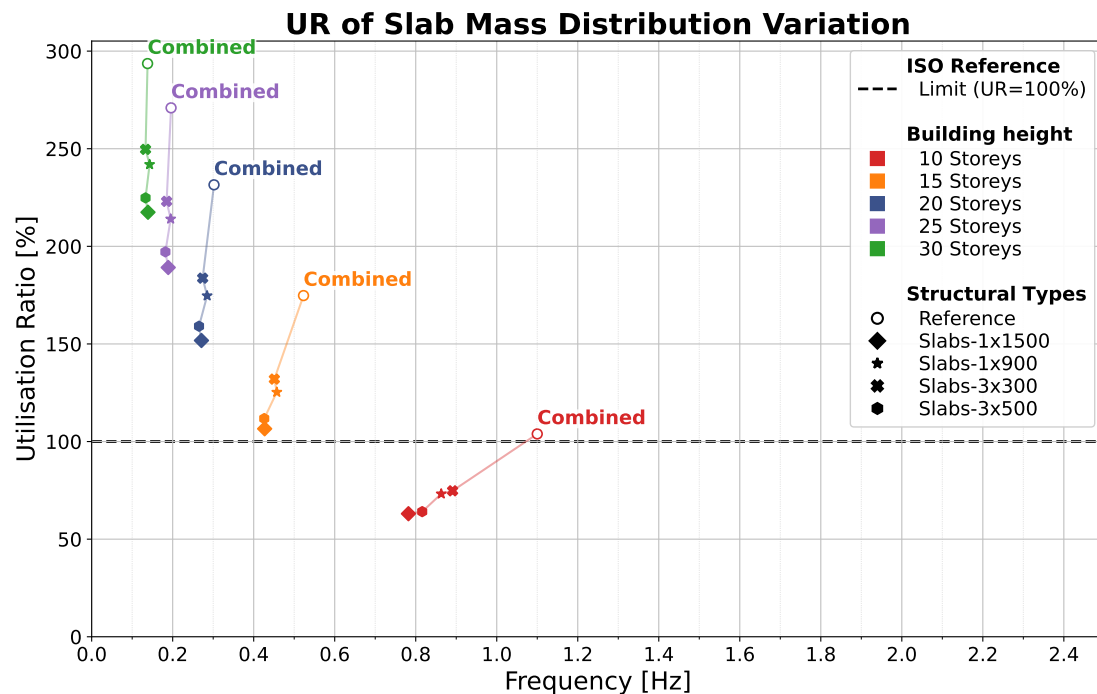


Figure C.1: Utilisation ratios of ISO 10137 comfort limits for combined accelerations of the slab mass distribution variations and reference building. Slabs of 3x300 mm, 1x900 mm, 3x500 mm, and 1x1500 mm.

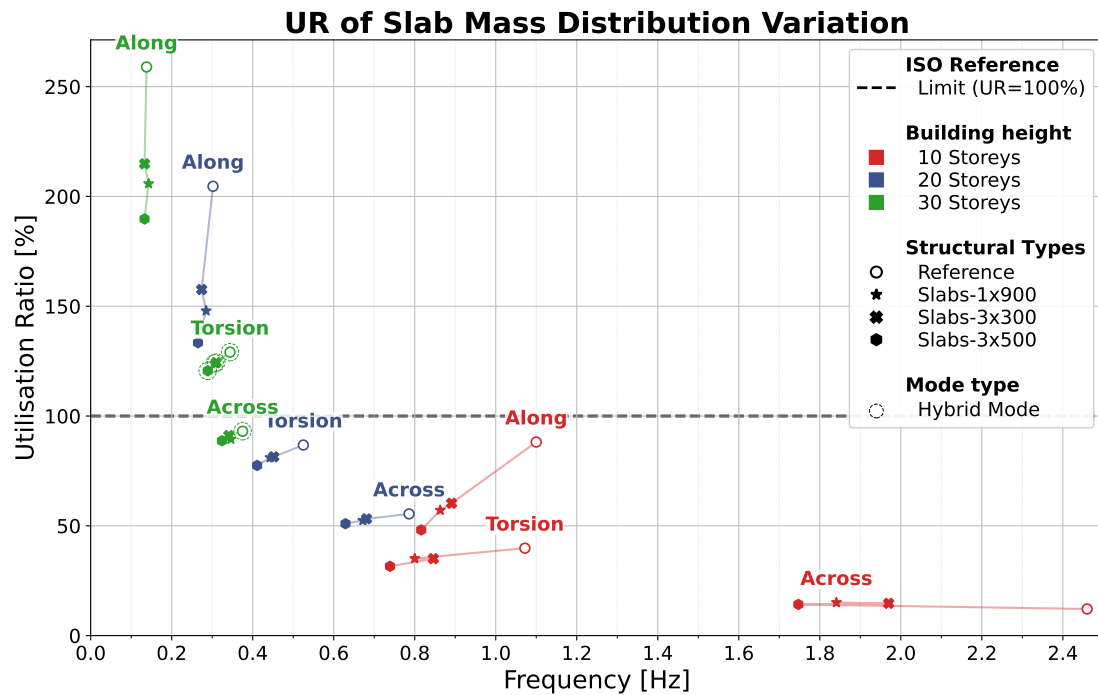


Figure C.2: Utilisation ratios of ISO 10137 comfort limits for along-wind, across-wind, and torsional accelerations of the slab mass distribution variations and reference building. Slabs of 3x300 mm, 1x900 mm, 3x500 mm, and 1x1500 mm.

Figure C.1 shows the combined accelerations for the different slab mass distributions. A slight increase in frequency is observed when mass is added at the top. Adding three 300 mm slabs resulted in a mean reduction in utilisation ratio of 45.6% across the building heights, with the 10-storey building as an exception at 29.1%. For the thicker 500 mm slabs, the corresponding mean reduction was 23.9%, again with a smaller decrease of 10.7% for the 10-storey building.

Concentrating the same concrete volume in a single top slab proved more effective than distributing it over several slabs. The mean difference between the 3x300 mm configuration and the single 900 mm slab was 8%, with a similar difference observed between the 3x500 mm configuration and the single 1500 mm slab.

Figure C.2 shows the along-wind, across-wind, and torsional accelerations for the different mass distributions. A similar trend to that of the combined acceleration results is observed in terms of reduction in utilisation ratio. However, the frequency changes are more scattered, with the largest increases occurring in the across-wind and torsional responses. These increases are more pronounced for the lower buildings. For instance, the across-wind frequency increased by 0.92 Hz for the 10-storey building, accompanied by a slight increase in utilisation ratio.

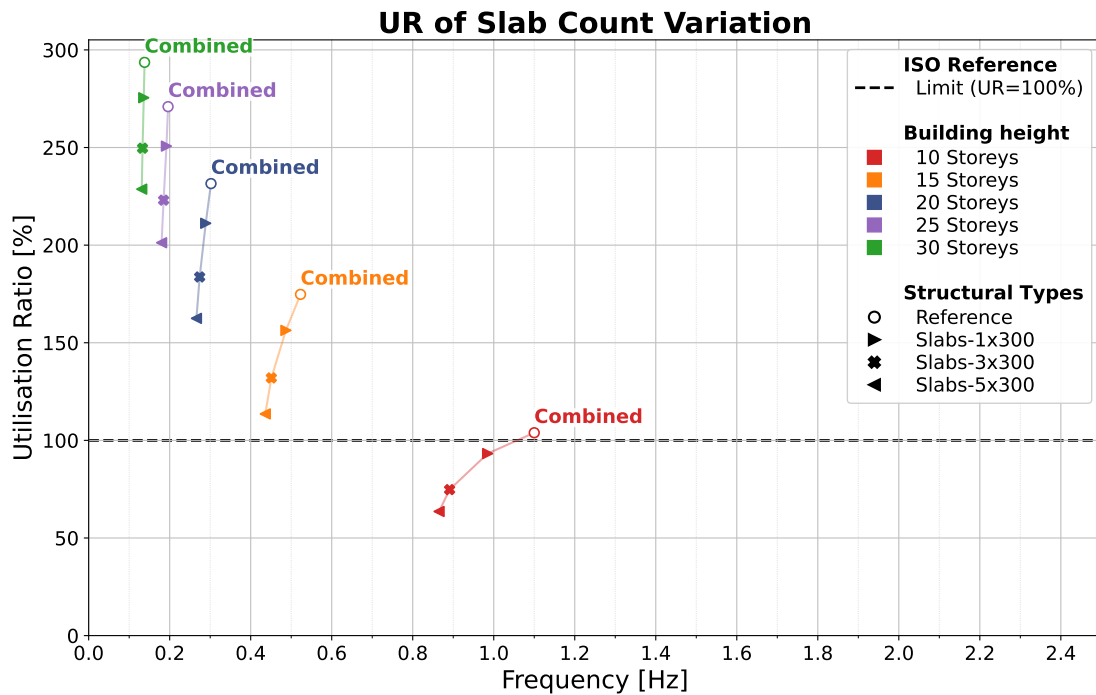


Figure C.3: Utilisation ratios of ISO 10137 comfort limits for combined accelerations of the slab count variations and reference building. The slab count variations are 1, 3, or 5 slabs of 300 mm.

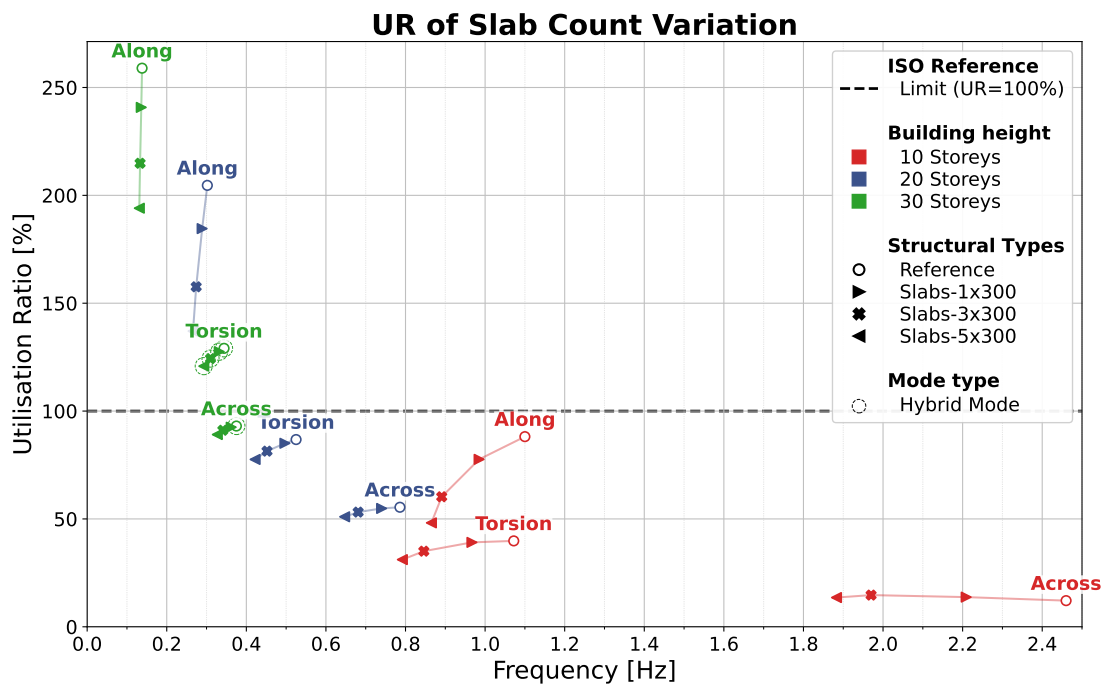


Figure C.4: Utilisation ratios of ISO 10137 comfort limits for along-wind, across-wind, and torsional accelerations of the slab count variations and reference building. The slab count variations are 1, 3, or 5 slabs of 300 mm.

In Figure C.3, the combined accelerations are displayed against frequency for slab count variations. For the 10-storey building, there is a significant decrease in frequency, from

1.1 Hz to 0.87 Hz, while the decrease is less pronounced for the taller buildings. For the 30-, 25-, 20-, and 15-storey buildings, the decrease in utilisation ratio remains relatively constant across building heights between slab counts. The mean reduction for adding one 300 mm slab is 19.3%. The mean reduction between one slab and three 300 mm slabs is 26.3%, and between three and five slabs is 20.6%. The reduction for the 10-storey building is less pronounced than for the other building heights, varying between 18.4% and 10.7% for the different slab counts.

Figure C.2 shows the along-wind, across-wind, and torsional accelerations for the slab count variations and the reference building. The patterns for the different acceleration components are similar to those observed for the combined acceleration. The frequency decreases for the 10-storey building, while the reduction in utilisation ratio is negligible for both across-wind and torsional accelerations. The total reduction in utilisation ratio for along-wind acceleration is 39.9% for the 10-storey building, compared with 65.0% for the 30-storey building. The effects on torsional and across-wind accelerations are less pronounced for the taller buildings.

C.2 Variations of Bracing Types

In this section, the results of the X-bracings and diagonal bracings, and the comparison between the two bracing systems, will be presented.

C.2.1 X-bracing

To increase the lateral stiffness, bracings were introduced on the building's short sides. In this section, X-bracings with cross-sections of 500 × 500 mm and 250 × 250 mm were assessed for the 10-, 20-, and 30-storey buildings. The bracings extended over two floors, and the effects of the different cross-sections were compared.

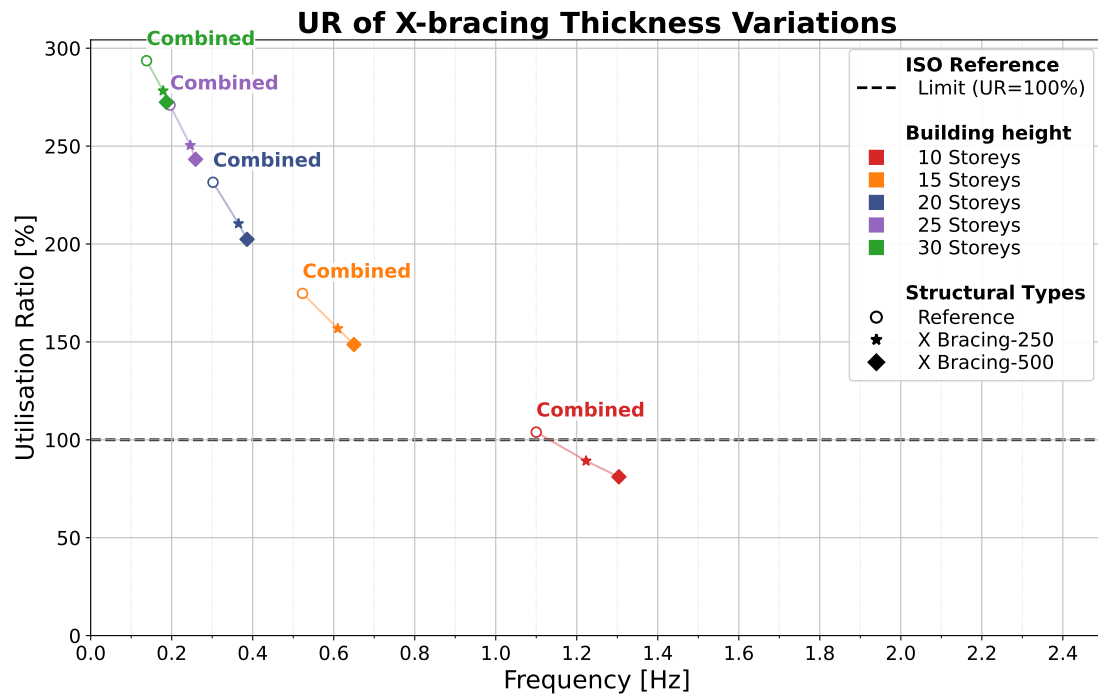


Figure C.5: Utilisation ratios of ISO 10137 comfort limits for along-wind, across-wind, and torsional accelerations of the X-bracing thickness variations and reference building.

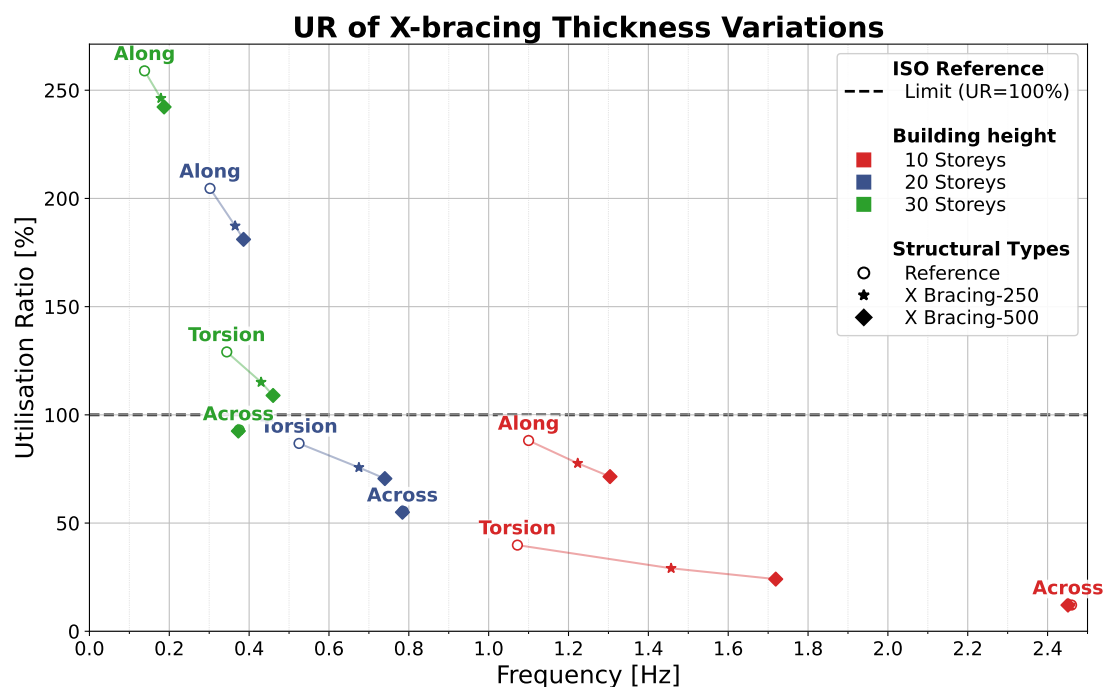


Figure C.6: Utilisation ratios of ISO 10137 comfort limits for along-wind, across-wind, and torsional accelerations of the X-bracing thickness variations and reference building.

Figure C.5 shows the combined accelerations and demonstrates that adding X-bracing on the short sides reduces the utilisation ratio. The mean reduction between the unbraced case and the 250×250 mm cross-section was 17.9%. Increasing the cross-section beyond this led to diminishing returns, particularly for taller buildings. For the 30-storey

building, a larger cross-section resulted in only a 5.9% additional reduction in utilisation ratio, while the timber volume increased by 303 m³ when increasing the cross-section from 250×250 mm to 500×500 mm.

Figure C.6 shows the along-wind, across-wind, and torsional accelerations. The X-bracing on the short sides has no noticeable effect on the across-wind response. A similar trend to that observed in the combined acceleration results is evident: the influence of increasing the cross-section decreases with building height for both along-wind and torsional responses in terms of frequency changes. For instance, the torsional frequency increases by 0.27 Hz for the 10-storey building, compared with only 0.05 Hz for the 30-storey building. Despite the smaller frequency changes in taller buildings, the reduction in utilisation ratio due to larger cross-sections remains relatively consistent across all heights, at around 5%.

C.2.2 Diagonal Bracing

A diagonal bracing system was investigated as a means to increase the building's stiffness and potentially reduce the volume of timber required compared with other bracing configurations. Bracings with cross-sections of 500 × 500 mm, 800 × 800 mm, and 1200 × 1200 mm were positioned in a zig-zag pattern on the short sides of the building, connecting the outer corners every five storeys. Initially, configurations with bracing every three and four storeys were examined; however, as these yielded only negligible decreases in acceleration compared with bracing every 5 storeys, it was determined that a connection every five storeys represents the most efficient use of timber.

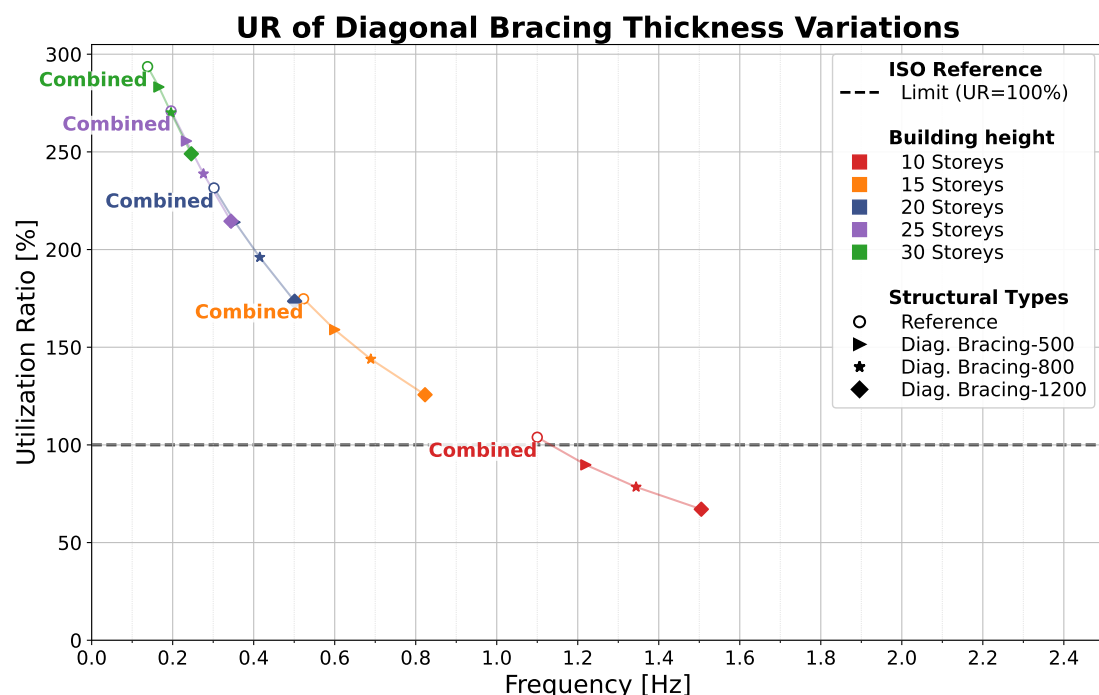


Figure C.7: Utilisation ratios of ISO 10137 comfort limits for combined accelerations for diagonal bracing thickness variations and reference building.

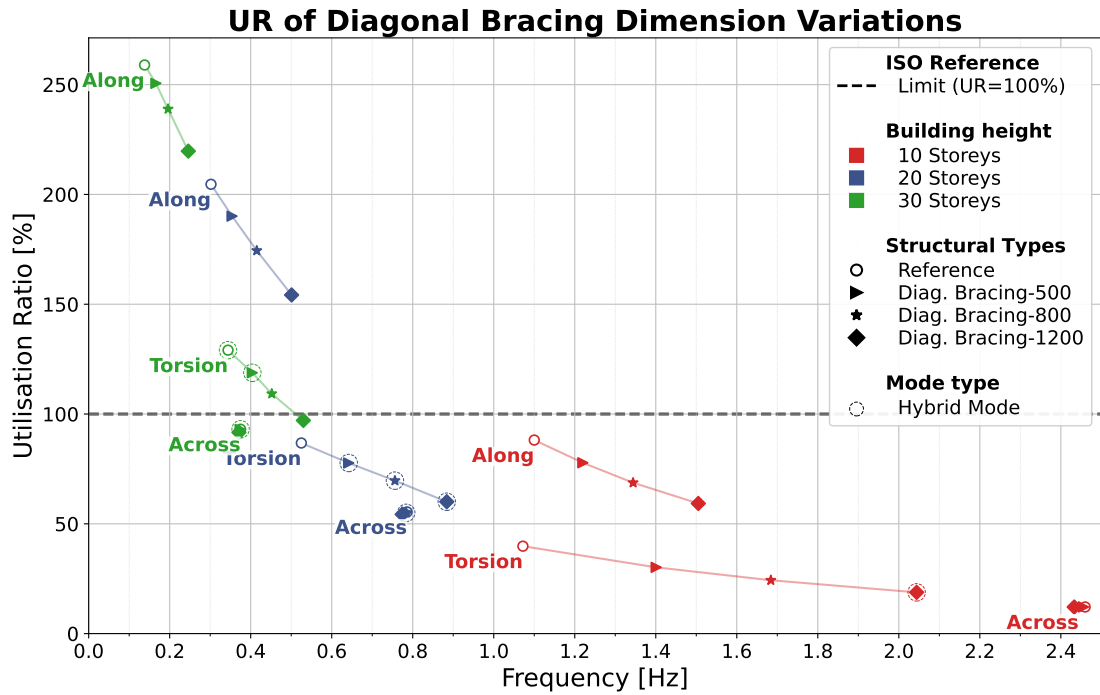


Figure C.8: Utilisation ratios of ISO 10137 comfort limits for along-wind, across-wind, and torsional accelerations for diagonal bracing thickness variations and reference building.

Figure C.7 reveals that the impact of diagonal bracing is most significant for buildings with 15 storeys or more, with a maximum reduction in acceleration of 57.9% for the 20-storey model. While the acceleration decrease is less pronounced for the 10-storey building, this model experiences the highest increase in natural frequency.

An analysis of along-wind, across-wind, and torsional accelerations (Figure C.8) shows that the reduction in torsional acceleration improves with building height, reaching 32.0% for the 30-storey model. For shorter buildings, the bracing has a greater effect on the first torsional frequency; for the 10-storey building, this frequency increases by 0.89 Hz. Overall, diagonal bracing demonstrates the highest efficiency in reducing along-wind acceleration, achieving a 50.4% reduction for the 20-storey model, while having a negligible influence on across-wind acceleration.

