Greening the urban office building

Boosting urban biodiversity by implementation of Vertical Greening Systems

Master thesis

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Boosting urban biodiversity by implementation of Vertical Greening Systems

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Preface

This thesis marks the final step of my Master's degree in Structural Engineering at Delft University of Technology. The research was driven by a growing awareness of the role biodiversity plays in our ecosystems and the urgent need to rethink how we design and adapt our buildings to help restore it. I believe that the creation and implementation of Vertical Greening Systems has only just begun. It is time for humans to stop being parasites within the natural system and to start living in harmony with it.

I would like to express my sincere gratitude to my thesis supervisors at TU Delft, Marc Ottelé and Marco Schuurman, for their valuable insights and guidance throughout the process. After every meeting I left with new insights and a more focused research direction. Besides, it should be noted that your enthusiasm for Vertical Greening Systems and concrete was truly contagious. In addition, I would like to thank my supervisors from Arup. Christa de Vaan, thank you for introducing me to biodiversity in the built environment, equipping me with the tools to quickly familiarise myself with the topic and providing a relevant and practical research challenge. Rogier van Reen, thank you for your input during our concrete-related discussions and our facade-focused exploration on Google Maps. This helped shape my understanding of the technical context.

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Executive summary

Cities today face a combination of global challenges, including climate change, biodiversity loss and rapid urbanization. While urban areas disrupt natural ecosystems, they also hold untapped potential for biodiversity due to their lower management intensity compared to agricultural land. At the same time, the built environment is one of the largest contributors to global biodiversity loss, underscoring the urgent need to integrate nature into cities. Retrofitting existing buildings with nature-based solutions (NBS) offers a more sustainable alternative to demolition and reconstruction, particularly in densely built environments. Among NBS, Vertical Greening Systems (VGS) offer high spatial potential on building envelopes but remain underutilized, partly due to limited knowledge of their biodiversity impact and feasibility on existing buildings. Current methods for assessing structural feasibility are highly detailed and often expensive, therefore not suitable for early design phases. Additionally, VGS suppliers rarely provide insight into biodiversity performance or adaptation to existing structures, complicating system selection.

This thesis addresses the need for a practical framework to support the decision-making of VGS implementation on existing urban office buildings during the feasibility phase of enhancing a building's biodiversity performance. It aims to reduce the knowledge and feasibility gap by structuring the exploration process based on biodiversity indicators, building characteristics and structural potential. The objective is reflected in the main research question:

> "How can a framework be developed to support decision-making on the feasibility and selection of VGS for existing Dutch office buildings aiming to enhance urban biodiversity?"

To answer the main research question, a multidisciplinary research approach was applied, structured around four core components of the developed framework. First, an exploratory literature review was conducted to classify VGS based on key distinctive physical features. This typology serves as a foundation for the entire study. Second, the biodiversity performance of the classified VGS types was assessed through a Multi-Criteria Analysis (MCA). Biodiversity indicators were derived from empirical literature and weighted based on the preferences of selected animal groups acting as ecological stakeholders. This resulted in a relative biodiversity ranking of VGS types. Third, a set of building indicators was developed by analyzing the key features of the VGS types and linking them to architectural characteristics commonly found in Dutch office buildings constructed between 1960 and 1990. This resulted in the definition of four representative building typologies to support suitability assessments. Fourth, the structural feasibility of VGS was addressed by analyzing Dutch design guidelines for existing concrete structures and validating a simplified estimation method for excess capacity in facade-supporting slabs and beams, using a parameter study. These steps together form the basis of the framework that supports VGS selection during the feasibility phase. Finally, the framework is verified through an illustrative case study.

The findings of this thesis show that all VGS types included in the study contribute to urban biodiversity, although to varying extents depending on their physical characteristics. The indicators most relevant to biodiversity performance are plant coverage, plant diversity, substrate size and substrate orientation. For assessing VGS suitability, building height, shape, facade geometry, glass percentage and facade type proved to be key characteristics. While the framework enables general recommendations based on these factors, detailed assessments remain necessary for project-specific implementation. Furthermore, the parameter study validates the use of excess capacity factors as a viable method for estimating structural feasibility in early phases, although some outliers from the parameter study have been translated into practical warnings for the framework. These are mainly related to the design of shear reinforcement and the calculation of its resistance, as this approach has evolved significantly over the years.

The resulting framework enables users to explore the implementation of VGS during the feasibility phase of enhancing a building's biodiversity performance. By relying on basic, broadly applicable parameters, it remains adaptable to a wide range of office buildings, while still offering structured guidance on the feasibility of different VGS types. The framework is built around the expected level of structural knowledge available in the feasibility phase and provides, for each level, either a graph or a formula to estimate structural excess capacity. This estimate can then be compared to the weight ranges identified for the different VGS types, offering insight into their structural feasibility. Based on the building's exterior, relevant characteristics can be derived that allow the building to be classified within one of the predefined office building typologies. These same characteristics are used to identify VGS recommendations for each typology. By combining the structurally feasible options, the architecturally suitable systems, and the biodiversity ranking, the framework supports well-informed decision-making for both the feasibility and selection of VGS implementation. Its application to a hypothetical case demonstrates that the framework functions as intended. Future application to real-world buildings would further validate its robustness and enhance its relevance for practical use, especially for architects, engineers, and municipalities seeking to green the urban environment.

Nomenclature

Abbreviations

Abbreviation	Definition
BAG Viewer	Basisregistratie Adressen en Gebouwen Viewer (Basic Registration of Adresses and buildings)
CC (1 tot 3)	Consequence Class (1 to 3)
EC 2012	Eurocode 2012
GBV 1962	Gewapend Beton Voorschrift 1962 (Reinforce Concrete Code)
GF	Green Facade
LWS	Living Wall System
MCA	Multi-Criteria Analysis
NBS	Nature-based Solutions
SLS	Serviceability Limit State
ULS	Ultimate Limit State
VB 1974	Voorschrift Beton 1974 (Concrete code)
VGS	Vertical Greening Systems
TGB 1955	Technische grondslagen voor bouwvoorschriften 1955 (Technical foundations for building regulations)
WLAI	Wall Leaf Area Index

Symbols

Symbol	Definition	Unit
A_0	Total area of bent-up bars (GBV 1962)	[cm²]
A_0	Required area of vertical stirrups in one plane (VB 1974)	[mm²]
A_0/t	Required area of vertical stirrups per mm at supports (VB 1974)	[mm²/mm]
$A_{a,aanwezig}$	Total area of main reinforcement (VB 1974)	[mm²]
$A_{a,benodigd}$	Theoretically required reinforcement area (VB 1974)	[mm²]
A_b	Total area of vertical stirrups in one plane (GBV 1962)	[mm²]
$A_{b,ben}$	Required area of vertical stirrups within distance y (GBV 1962)	[mm²]
A_s	Total area of main reinforcement	[mm²]
b_{beam}	Width of beam	[mm]
b_{slab}	Width of slab	[mm]
c	Concrete cover	[mm]
C	Coefficient for deflection calculation of VB 1974	[-]
$C_{F,element,code}$	Governing excess capacity in an element for a code edition	[kN/m]
$C_{F,element,code,a}$	Excess capacity in an element for a code edition, based on fundamental load combination 6.10a	[kN/m]
$C_{F,element,code,b}$	Excess capacity in an element for a code edition, based on fundamental load combination 6.10b	[kN/m]

Symbol	Definition	Unit
$C_{F,element,code,SLS}$	Excess capacity in an element for a code edition, under SLS	[kN/m]
$C_{F,ten}$	Excess capacity of a concrete element in tension	[kN/m]
$C_{Rd,c}$	coefficient required for shear resistance according to NEN 870X	[-]
$C_{x,com}$	Excess capacity factor for concrete in compression	[-]
$C_{x,element,code}$	Excess capacity factor for a specific element and a code edition	[-]
$C_{x,ten}$	Excess capacity factor for steel reinforcement in tension	[-]
d	Effective height of cross-section	[mm]
$d_{min,element}$	Required minimum effective height of beam or slab	[mm]
E_a	Modulus of elasticity of steel	[N/mm²]
E_c	Modulus of elasticity of concrete	[N/mm²]
E_cm	Modulus of elasticity from the ultimate limit state derived with short term loading tests	[N/mm²]
E_equi	Assumed modulus of elasticity (NEN 870X)	[N/mm²]
f'_b	Design compressive strength of concrete without significant normal force (VB 1974)	[N/mm²]
f_a	Design tensile strength of steel reinforcement (VB 1974)	[N/mm²]
f_b	Design tensile strength of concrete (VB 1974)	[N/mm²]
f'_b	Design compressive strength of concrete without significant normal force (VB 1974)	[N/mm²]
f_{cd}	Design compressive strength of concrete	[N/mm²]
f_{ck}	Characteristic compressive strength of concrete	[N/mm²]
f_{cm}	Mean compressive strength of concrete (NEN 870X)	[N/mm²]
f_{yd}	Design tensile strength of steel	[N/mm²]
f_{yk}	Characteristic tensile strength of steel	[N/mm²]
$G_{k,j}$	Characteristic value of a permanent action i	[kN/m]
$G_{k,slab}$	Permanent load acting on the slab	[kN/m²]
g_{fac}	Facade weight	[kN/m²]
$g_{k,element}$	permanent load acting on beam or slab	[kN/m]
g_{lw}	permanent load accounting for the weight of lightweight partition walls (VB 1974)	[kN/m²]
h_{beam}	Height of beam	[mm]
h_{st}	Story height	[m]
I	moment of inertia	[m³]
k	coefficient required for shear resistance according to NEN 870X	[-]
k_1	coefficient required for shear resistance according to NEN 870X	[-]
$k_{x,max}$	coefficient used for calculation of minimum reinforcement ratio	[-]
L_{beam}	Length of beam	[m]
L_{slab}	Length of slab	[m]
M_{Ed}	Acting bending moment	[Nmm]
M_{Rd}	Bending moment resistance	[Nmm]
M_u	Ultimate bending moment	[Nmm]
n	Ratio of modulus of elasticity of steel to concrete (GBV 1962)	[-]
n_b	Fitted number of stirrups (GBV 1962)	[-]
$n_{b,ben}$	Required number of stirrups (GBV 1962)	[-]
n_{main}	Number of main reinforcement bars in the cross-section	[-]

Symbol	Definition	Unit
N_s	Design tensile force in steel	[kN]
$q_{k,element}$	variable load acting on beam or slab	[kN/m]
$q_{M,Rd}$	Uniformly distributed design load based on bending moment resistance (NEN870X)	[kN/m]
$q_{Rd,ULS}$	Maximum design load of the ultimate limit state acting on an element	[kN/m]
$q_{V,Rd}$	Uniformly distributed design load based on shear resistance (NEN870X)	[kN/m]
$q_{w,Rd}$	Uniformly distributed design load based on deflection requirements (NEN870X)	[kN/m]
$Q_{k,i}$	Characteristic value of a variable action i	[kN/m]
$Q_{k,slab,code}$	Variable floor load defined by a code edition	[kN/m²]
r	Pearson correlation coefficient	[-]
R^2	Coefficient of determination	[-]
R_k	Characteristic design resistance	[kN]
R_d	Design resistance	[kN]
$s_{d,element}$	Design line load of beam or slab	[kN/m]
$s_{k,element,code}$	Characteristic load acting on an element for a code edition	[kN/m]
s_{main}	Spacing between main reinforcement bars	[mm]
s_{st}	Spacing between stirrups	[mm]
S	Tensile force resisted by shear reinforcement (GBV 1962)	[kgf]
S_0	Tensile force resisted by bent-up bars (GBV 1962)	[kgf]
S_r	Diagonal tensile force resisted by stirrups (GBV 1962)	[kgf]
S_k	Characteristic design load	[kN]
S_d	Design load	[kN]
t_{slab}	Slab thickness	[mm]
T	Acting shear force (GBV 1962)	[N]
T_1	Shear force resisted by the concrete (VB 1974)	[N]
T_d	Acting shear force (VB 1974)	[N]
v_min	Minimum shear capacity of concrete (NEN 870X)	[N/mm²]
V_{Ed}	Acting shear force	[N]
V_{Rd}	Total shear resistance	[N]
$V_{Rd,bent}$	Shear resistance of bent-up bars	[N]
$V_{Rd,bent,max}$	Maximum shear resistance of bent-up bars	[N]
$V_{Rd,c}$	Shear resistance of concrete without shear reinforcement	[N]
$V_{Rd,c,min}$	Minimum shear resistance of concrete without shear reinforcement	[N]
$V_{Rd,st}$	Shear resistance of stirrups	[N]
$V_{Rd,st,max}$	Maximum shear resistance of stirrups	[N]
w_{lim}	Maximum allowed deflection (NEN 870X)	[mm]
x	Height of neutral axis from top of section	[mm]
y	Distance from supports over which shear reinforcement is required	[mm]
z	Internal lever arm	[mm]
α	Angle between the shear reinforcement and the beam axis measured perpendicular to the shear force	[°]
α_{cw}	Coefficient for shear capacity calculation (NEN 870X)	[-]
ξ	Reduction factor applied to unfavorable permanent actions	[-]

Symbol	Definition	Unit
ϵ_{ar}	Fracture strain of steel (VB 1974)	[-]
θ	Angle between the concrete compression strut and the beam axis perpendicular to the shear force	[°]
γ_1	Partial factor primarily addressing uncertainties in calculations (VB 1974)	[-]
γ_C	Partial factor of concrete	[-]
γ_c	Volumetric weight of concrete	[kN/m³]
γ_s	Partial factor applied when self-weight and permanent loads have favorable effect on load-bearing capacity (VB 1974)	[-]
γ_S	Partial factor of steel	[-]
$\gamma_{total,com,code}$	General safety factor of concrete in compression for a code edition	[-]
$\gamma_{total,ten,code}$	General safety factor of steel in tension for a code edition	[-]
$\gamma_{load,code}$	Load safety factor defined by a code edition	[-]
γ_m	Partial factor primarily addressing uncertainties in construction and the level of quality control (VB 1974)	[-]
γ_{mat}	Material safety factor defined by a code edition	[-]
$\gamma_{G,j}$	Partial factor for permanent action i	[-]
$\gamma_{Q,i}$	Partial factor for variable action i	[-]
γ_u	Safety factor for ultimate limit state according to VB 1974	[-]
$\Delta s_{k,code}$	Difference between original and current characteristic load	[kN/m]
μ_{mat}	Mean cube compression strength of concrete	[N/mm²]
ψ_0	Combination factor applied to a variable action to determine its combination value	[-]
ψ_1	Combination factor applied to a variable action to determine its frequent value	[-]
ψ_2	Combination factor applied to a variable action to determine its quasi-permanent value	[-]
ω	Actual reinforcement ratio (VB 1974)	[%]
$\omega_{0,max}$	The maximum reinforcement ratio (VB 1974)	[%]
$\omega_{0,min}$	The minimum reinforcement ratio (VB 1974)	[%]
ρ_l	Main reinforcement ratio (NEN 870X)	[-]
σ_a	Induced tensile stress in steel reinforcement	[N/mm²]
$\bar{\sigma}_a$	Allowable tensile strength of steel reinforcement in bending	[N/mm²]
σ'_b	Induced compressive strength in concrete in	[N/mm²]
$\bar{\sigma}'_b$	Allowable compressive strength of concrete in bending	[N/mm²]
$\bar{\sigma}_b$	Allowable tensile strength of concrete in shear without reinforcement present	[N/mm²]
$\sigma_{b,A}$	Tensile stress in concrete at supports	[N/mm²]
$\bar{\sigma}_{b,max}$	Allowable tensile strength of concrete in shear with reinforcement present	[N/mm²]
τ_1	Design shear strength of concrete (VB 1974)	[N/mm²]
τ_d	Acting design shear stress (VB 1974)	[N/mm²]
ϕ_{main}	Diameter of main reinforcement	[mm]
ϕ_{st}	Diameter of stirrups	[mm]

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1 Introduction

1.1. Problem context

The current generation is facing many interrelated global challenges, including population growth, urbanization, the effects of climate change and biodiversity loss. These originate from the rapid development of the human species that led to humans overusing the plant today [29]. As such, after industrialization, fossil fuels replaced timber as the preferred energy source. Unlike timber, fossil fuels can't be easily grown. Whereas timber can be regrown, fossil fuels are finite and non-renewable [56]. Carbon dioxide and other greenhouse gases trap heat, like a warming blanket around the Earth. Their increase directly leads to global warming [174], a result of human induced climate change. Combined with strong population growth, this problem is worsening, as it leads to increased energy consumption [3].

Another phenomenon unconsciously orchestrated by humans is urbanization, which disrupts the balance of natural ecosystems [63]. The natural system functions as a feedback loop, but cities take without giving back, breaking that cycle [45]. Odum [119] even called cities to be "the only parasites in the biosphere". In 1800, only 2 percent of the world's population lived in rural settlements [175]. From here, urbanization really took flight. Since 2007 more than half of the world's population is urbanized [6, 175]. In addition, the bigger the cities and the taller their buildings, the more heat they trap. The urban heat island effect is a phenomenon where the temperature in the city is higher than in the surrounding rural areas. This is the result of the high heat capacity but low conductivity of building materials, decreased evaporation, urban canyons, anthropogenic heat and air pollution [74].

More people and larger cities also imply less natural spaces and less chances for biodiversity to flourish. In addition, much of the land that is generally considered "green" in the Netherlands is in fact used for agriculture. While it may appear green in color, agricultural land is one of the leading contributors to biodiversity loss [39]. Farming converts natural ecosystems to highly controlled systems and leads to pollution, including greenhouse gas emissions. Over the centuries, not only cities but also agricultural land in the Netherlands have expanded significantly [19]. Together, urban and agricultural development have increasingly occupied space that was once available to natural habitats. As a result, biodiversity has been pushed to the margins. Humanity is considered responsible for the drastic decline in biodiversity over the past 50 years [30]. In 2019, approximately 25% of species were threatened with extinction [103] and the Living Planet Index reports an average decline of around 40% in species populations. Moreover, the built environment sector alone accounts for nearly 30% of biodiversity loss worldwide [81].

In other words, we are hurting the planet and its ecosystems in a rapid and often irreversible way, all without realizing that these ecosystems are crucial to our health and prosperity [29]. Nature provides us with "ecosystem services", which are products and processes that are essential for our well-being [71]. The Millennium Ecosystem Assessment [138] defined them as "the benefits people obtain from ecosystems". Ecosystem services have been categorized into four groups: provisioning services, regulating services, cultural services and supporting services [171]. Examples from regulating services, related to the topics explained earlier, include the regulation and purification of water, air, and soil quality, as well as climate regulation [63]. To simplify, earth's ecosystems provide humans with life's necessities, such as food, fresh water and clean air [138]. Literature shows that biodiversity is essential for ecosystem functioning, including the delivery of those ecosystem services [79, 93, 150]. To sustain the ecosystem services on which we depend, humans must start giving back to the natural feedback loop of urban ecosystems, where the imbalance is most severe.

To summarize, increasing and preserving biodiversity can help address the problems humanity caused. Cities are recognized as areas where biodiversity can thrive due to their high degree of heterogeneity of urban ecosystems, which are characterized by a wide variety of plant and animal species [26, 93]. Unlike agricultural land, urban environments are not as intensively managed, which is beneficial for biodiversity. Consequently, biodiversity is typically greater in urban environments than in agricultural ones [123]. However, space for additional greenery is scarce in modern cities, while buildings are abundant [164]. Moreover, adapting a city rather than demolishing and rebuilding it is the more sustainable approach [64, 161]. Demolishing would only enlarge the problem of pollution and rebuilding would contribute to the problem of resource depletion. Hence the starting point should be to adapt the buildings cities currently house.

1.2. Research problem

Integrating nature into cities can be achieved by the implementation of nature-based solutions (NBS). The IUCN defines NBS as: "actions to protect, sustainably manage and restore natural or modified ecosystems, which address societal challenges effectively, while simultaneously providing human well-being and biodiversity benefits" [72]. NBS that can be applied directly to the building envelope of existing buildings are nest boxes, green roofs and Vertical Greening Systems (VGS). Nest boxes are generally small in size, have low self-weight and are relatively easy to apply, therefore they are not the problem. While green roofs are already well implemented and their effects on biodiversity and urban climate are relatively well documented, vertical built surfaces cover a significantly larger total area in urban areas and the biodiversity impact of VGS is less understood [47, 164].

The implementation of VGS can add a substantial weight to the existing structure, which is not always feasible. A detailed calculation by a structural engineer is required to identify potential structural excess capacity, which will hereafter be referred to as excess capacity in this study. Moreover, implementing VGS is often considered unaffordable [164], not to mention the additional costs of involving a structural engineering firm to assess structural feasibility during an exploratory phase. Beeren [10] and van Uffelen [165] both studied the excess capacity of existing concrete buildings within their research, either to repurpose them or to apply VGS. Beeren [10] calculated the excess capacity of a specific building's facade based on the original structural calculations, drawings and guidelines, combined with the structural guidelines of today. This means that highly detailed, building-specific information is required to perform the excess capacity calculation as done by Beeren [10]. However, Beeren [10] applied a useful concept from van Uffelen's research to quantify excess capacity, which is referred to in this study as the excess capacity factor. The research from van Uffelen [165] introduced an interesting framework for examining the potential for repurposing a building and identifying excess capacity. Within that framework, the safety margin between acting load and resistance is analyzed over time. The reduction in this margin is used by Beeren [10] as a factor to identify excess capacity in structural elements. Van Uffelen's [165] advise often includes technical research, which is actual testing of the concrete's compressive strength at various locations in the building. Both methods introduced by Beeren [10] and van Uffelen [165] are deemed inapplicable in the feasibility phase of implementing green building envelope systems as they require highly detailed information or experimental research, not solving the issue of detailed and expensive calculations. However, the application of an excess capacity factor appears promising.

Moreover, suppliers of VGS provide a diverse set of systems to choose from. This diverse set of systems comes with a broad range of purposes, which increases the complexity of deciding to add a

VGS. When it comes to improving urban biodiversity, most suppliers of VGS do not provide information on their biodiversity potential. Furthermore, most VGS on the market today are designed to be implemented onto new construction and limited information of the implementation onto existing buildings can be found.

1.3. Objective

This research aims to develop a framework to support the decision-making of VGS implementation during the feasibility phase of enhancing a building's biodiversity performance. The framework provides an initial assessment of the feasibility of integrating VGS onto existing buildings and offers insight into their potential impact on urban biodiversity.

1.4. Research scope

Section 1.1 explains that although the biodiversity potential in cities is high, it remains largely underutilized. It also recommends to focus on existing buildings, due to sustainability considerations. Furthermore, Arup notes that while companies aim to make their existing buildings Paris-proof and improve biodiversity performance, they face challenges in selecting the most appropriate nature-based solutions for implementation across their building portfolios. This challenge corresponds to what can be considered the feasibility phase. As discussed earlier, among nature-based solutions applicable to the building envelope, VGS are considered the most relevant due to their high spatial potential and relatively limited implementation to date. Therefore, this research is limited to existing office buildings in urban areas, with a specific focus on supporting the feasibility phase of implementing vertical green systems.

Van Uffelen's research [165] shows that safety margins have decreased over time, suggesting that the greatest excess capacity is most likely found in older buildings. At the same time, both technical knowledge and reliability have increased throughout the years. In particular, during the period from 1910 to 1940, concrete construction was still in an exploratory phase [165]. After World War II, the primary focus was on rebuilding the housing stock [142], while office construction experienced a strong boost in the 1960s [142]. In addition, NEN 8700 [115, App. F.0] recommends new construction requirements to buildings younger than 15 years instead of more favorably requirements for existing construction. To investigate a promising timeframe with considerable structural uncertainties, this study focuses on office buildings constructed between 1960 and 1990.

Structural materials differ in their mechanical behavior, which leads to varying design approaches and material-specific considerations when assessing structural feasibility. Van Uffelen [165] also identified significant differences in the safety margins of concrete and steel reinforcement. Traditionally, Dutch office buildings were constructed using concrete column-beam structures combined with brick masonry facades [142]. As office building heights increased around the 1950s and 1960s, steel and concrete frames with lightweight facades became more common [33]. However, due to the limited fire resistance of steel, these frames had to be encased in fire-resistant materials such as concrete. As a result, steel only became economically viable for buildings of approximately ten stories or more [33]. Moreover, while a structural assessment standard for existing concrete structures is available, such a standard for steel structures is still lacking [115]. Therefore, this study focuses exclusively on existing office buildings constructed from reinforced concrete.

Van Uffelen [165] shows that concrete beams are most likely to govern excess capacity, in comparison to columns. Therefore, structural elements supporting facades and subjected to bending are considered more relevant for examining excess capacity than elements primarily loaded in compression. As a result, the scope is limited to concrete beams and slabs supporting facades.

The framework should aid in optimizing the retrofitting of office buildings with VGS systems to enhance biodiversity. However, no experimental validation of the outcomes is included in this study. This applies both to the feasibility of implementing VGS and to biodiversity improvement. The input and methods used for the framework will be gathered and verified through a desk study and an illustrative case study. Regarding biodiversity enhancements, no guarantees can be made about the actual arrival of species, as this lies beyond the control of both the framework and the building owners. What is within their control, and can be verified by governmental authorities, is whether the necessary actions have been implemented. Therefore, the framework will support building owners in the preliminary exploration of VGS, making it easier to assess both their feasibility and potential impact. This will potentially lower the threshold for implementing green building envelope systems. It should be noted that proper use of the framework requires a basic understanding of structural systems and building materials. Therefore, it is intended to support the professional judgment of those with structural expertise.

1.5. Research questions

Research questions have been formulated to address the above-mentioned problems and to achieve the stated objective within the defined scope. The main research question is formulated as follows:

"How can a framework be developed to support decision-making on the feasibility and selection of VGS for existing Dutch office buildings aiming to enhance urban biodiversity?"

In order to answer the main research question, several guiding sub-questions have been defined, aligned with the structure of the research process. The study begins by identifying various VGS types applicable to the facades of existing buildings, examining their key characteristics in order to classify them. Secondly, the urban biodiversity potential of the classified VGS is investigated. In addition, the suitability of these systems for application on existing facades is assessed. From an architectural perspective, an analysis is conducted of office buildings constructed between 1960 and 1990, in order to define distinguishing characteristics relevant to office building typologies. This is followed by an exploration of historical and current Dutch building regulations, with the aim of identifying an appropriate excess capacity estimation method suitable for the feasibility phase of VGS implementation. As part of this exploration, the origin and use of excess capacity factors is analyzed in detail and subsequently validated through a parameter study. The information gathered on VGS, biodiversity potential, building characteristics and excess capacity estimation is then integrated into a unified framework. The final step involves the verification of this developed framework. The sub-questions are formulated in accordance with the structure of this research process:

1. "What are the main types of VGS applicable to building facades?"
2. "Which features of VGS serve as key indicators of their biodiversity performance?"
3. "Which characteristics of office buildings constructed between 1960 and 1990 indicate their suitability for VGS implementation?"
4. "How can the excess capacity of slabs and beams supporting facades in existing Dutch office buildings be estimated during the feasibility phase?"
   (a) How does the structural assessment of existing buildings differ from that of new construction in Dutch building regulations and guidelines?
   (b) How does the existing method for assessing excess capacity apply excess capacity factors and what are its limitations in the feasibility phase?
   (c) How can the use of excess capacity factors be validated for beams and slabs carrying facade loads in existing Dutch office buildings?
5. "How can biodiversity indicators, building characteristics and an excess capacity assessment be integrated into a framework for the feasibility phase of implementing VGS for biodiversity enhancement?"
6. "How can a framework designed to support the feasibility phase of implementing VGS for biodiversity enhancement be verified?"

1.6. Research methodology and thesis outline

This section presents the research methods used to answer the sub-questions introduced in the previous section. Figure 1.1 shows, for each product of the thesis, the method applied, the research question addressed, and the chapter in which it is discussed. The black arrows in the diagram indicate the chronological sequence of the research steps, illustrating which phases were conducted first and how later steps build on earlier results. The products and their corresponding methods are briefly explained below.

Chapter 2 presents an exploratory literature review on VGS to identify the main types suitable for application on existing facades, in order to answer the first research question. This was supplemented by an expert interview with a facade designer who developed his own VGS with a strong focus on enhancing biodiversity. The result is a classification of VGS based on distinctive key features, which forms a reference throughout the remainder of the thesis.

Chapter 3 focuses on empirical studies to investigate the biodiversity performance of the classified VGS types. First, these systems are analyzed to identify features that indicate their potential for supporting urban biodiversity. This informs the answer to the second research question. The VGS types are then compared in an ordinal-scale multi-criteria analysis (MCA), in which animal groups derived from empirical studies are considered stakeholders. This process results in a biodiversity ranking of the VGS types, which contributes to the development of the VGS implementation framework, supporting the main research question.

Chapter 4 begins with an analysis of the classified VGS types to identify building indicators that influence their suitability. The exploratory literature study is then broadened to include architectural aspects, specifically through an investigation of office buildings constructed between 1960 and 1990 and two expert interviews. The outcome is the definition of four office building typologies that help assess the suitability of different VGS types. This addresses the third research question and informs the VGS implementation framework.

Chapters 5 and 6 aim to answer the fourth research question. Chapter 5 begins with an analysis of Dutch structural design guidelines to understand how existing concrete structures are treated compared to new construction. It then explores how excess capacity factors are derived in an existing method, to gain insight into where structural excess capacity may originate and how these factors can be applied. Chapter 6 validates the use of this method through a parameter study focusing on facade-supporting slabs and beams. The result is a validated approach for estimating excess capacity in these structural elements.

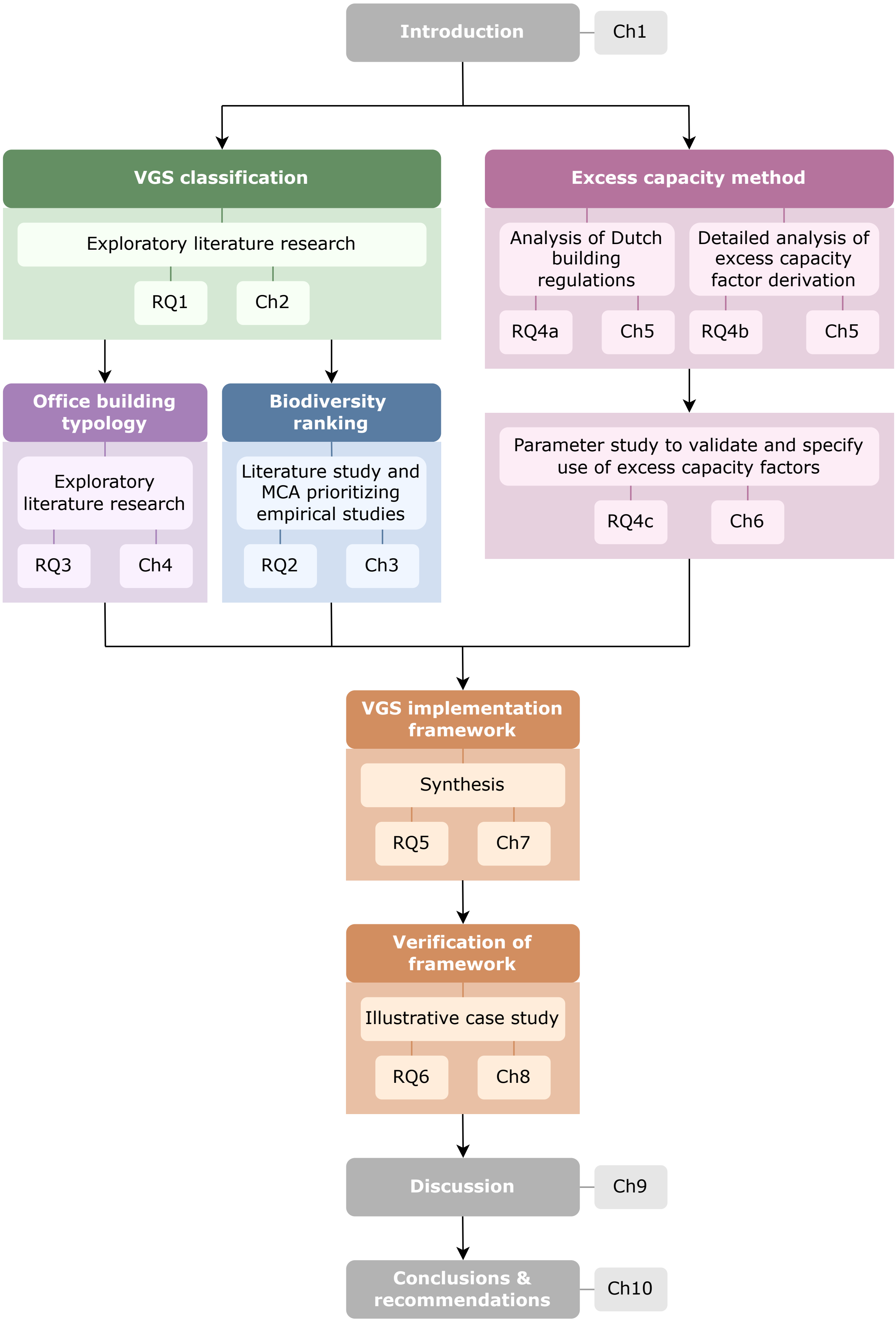


Figure 1.1: Research methodology diagram

2

Vertical Greening Systems

This chapter dives into VGS applicable to building facades. The aim of this chapter is to identify the different design options and classify them according to their key features. In order to limit the study, a guided literature review approach was used, supplemented with online information from VGS producers gain a better understanding of the structural composition and mounting methods of the systems, as most literature focuses on other facets. A detailed explanation of the applied procedure can be found in appendix A.

2.1. Definition

Vertical greening systems have many different names in literature, but the definitions are quite similar. Examples of used nomenclatures for VGS are ‘green walls’ or ‘green wall systems’ [88, 70, 125], ‘green vertical systems’ [133] and ‘vertical gardens’ [9, 127]. They all describe VGS as a type of vegetated wall surface, a system that enables greening a vertical wall or simply as all forms of vegetation for facades. In the context of this study, vertical greening systems are defined as “all types of vegetated wall surfaces, featuring plants rooted in the ground, integrated into the wall itself or embedded in modular panels attached to the facade” [134]. It should be noted that only VGS applicable to building facades are included, which excludes freestanding green systems. Additionally, this study only considers VGS types whose biodiversity performance has been examined in existing studies.

2.2. Classification

The literature indicates that there is no universally adopted classification for VGS. As a result, not only do definitions of VGS vary, but different terms are also used to describe the various system types [23]. This inconsistency is reflected both in the research field and among producers, making it challenging for potential adopters to clearly understand which benefits are associated with which system. Despite this inconsistency, there is one general division that many studies agree with, resulting in two main categories: living walls and green facades [23, 88, 129, 43, 125, 134]. Some base this division on whether the systems are ground-bound or facade-bound [60, 123, 91], others say intensive or extensive systems [129, 128], but most refer to the support structure and growing method as the main distinctive features [43, 134, 88, 85]. Extensive and intensive systems relate to the ease of implementation and the required maintenance afterwards [128], but support structure and growing method are often not properly defined.

This study bases the initial classification on the cultivation method, which is defined as ‘the method to establish and maintain plant growth’. In other words, it refers to the way plants receive their required water and nutrients. Two distinct cultivation methods are identified: either the plants are rooted in the ground or a hydroponic technique is used, with or without a growing medium. The key feature of green facades “is a vegetation cover formed by climbing and hanging plants, growing from ground level” [164]. Conversely, plants implemented in the living wall systems are not in contact with the soil at ground level, but the habitat of the plants is created on the facade itself. Accordingly, these systems are equipped with an irrigation system that provides water and nutrients [23, 43, 125, 134, 124, 96]. Between these systems are green facades that are not rooted directly into the ground, but instead into planter boxes placed at ground level. However, since their weight is not carried by the facade, the full habitat of the plants is not actually established on the facade itself. As the differences between these systems are minimal and as discussed in section 3.2.2, rooting directly into the ground is considered more beneficial for biodiversity. Therefore, these slightly different types of green facades are excluded from this research.

Further down the line, classifications found in literature tend to diverge based on whether the focus lies on quantifying benefits or on identifying physical differences without a specific underlying objective. This study does have such an underlying objective, it aims to study the implementation of VGS on existing office buildings in the Netherlands for biodiversity enhancement. Hence, their biodiversity potential should be derivable. In line with this objective, a classification based on distinctive physical features is proposed, so the comparison with VGS from Dutch producers is easily made. For every classification level a different distinctive physical feature with proper definition is used, such as cultivation method, support structure and growing unit. figure 2.1 presents an overview of the classification. In the following subsections the definitions are provided and key features of the various system types are discussed.

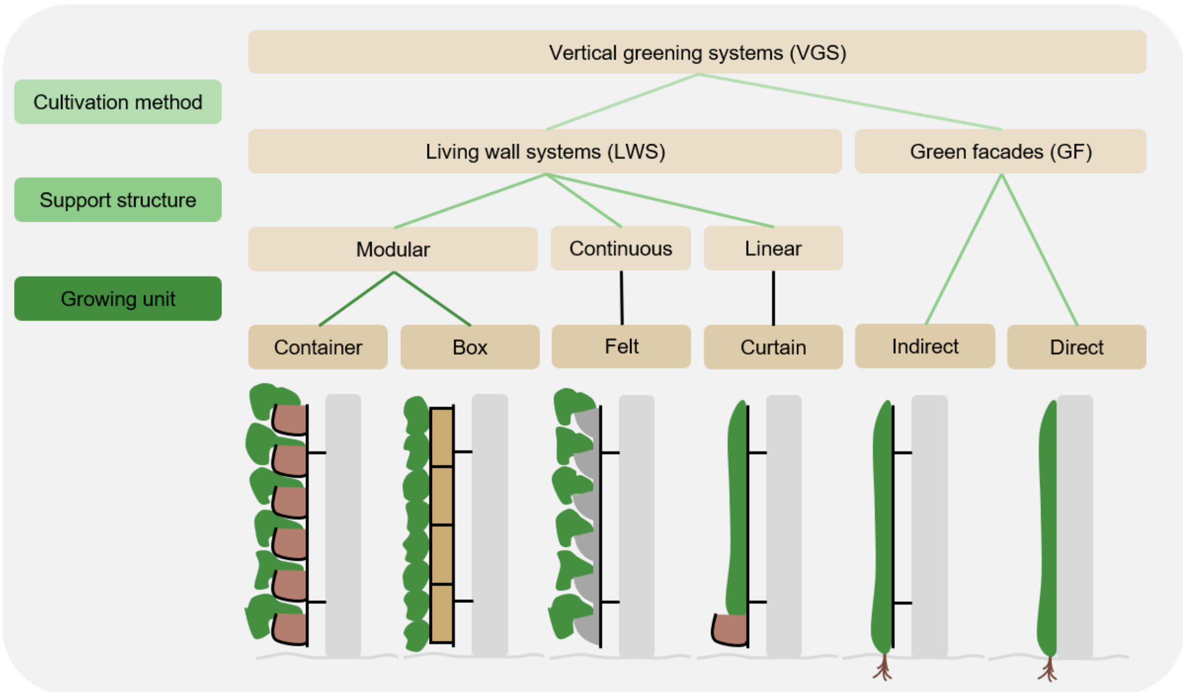


Figure 2.1: Classification system of Vertical Greening Systems

2.2.1. Green facades

An important physical distinction, commonly used in literature, is the support structure. The subsequent classification step is based on this aspect, defined as ‘the physical framework that supports the vegetation system and transfers its load to the building’. The vegetation system consists of the plants and the growing medium and includes the irrigation system and growing units if present. Applying the support structure aspect to green facades results in two system types: indirect and direct green facades [88, 164, 130].

Direct green facades

Plants of direct green facades do not need additional support, since they adhere directly to the wall by adventitious roots or self-adhesive pads [91]. Their self-weight is based solely on the weight of the plant, which results in little debate in literature [96, 164, 34, 124]. The weight of the direct green facade is considered 5 kg/m^2.

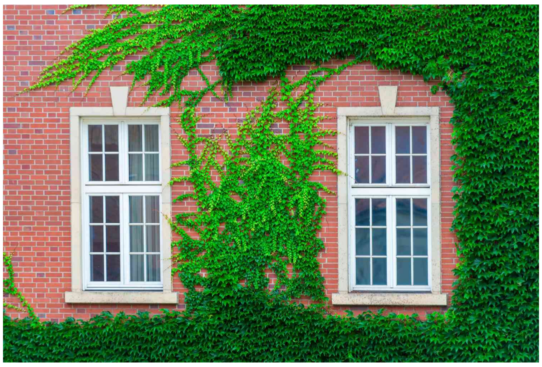


Figure 2.2: Direct green facade [92]

Indirect green facades

Indirect green facades include an additional vertical support system [164] with an air cavity in between the wall and the system. Therefore they are often referred to as double-skin green facades [91, 129, 27, 12, 128]. The support system can either be continuous or modular. Continuous systems are referred to as continuous guides, cables or ropes [25, 125], which is a single support structure along the entire wall [88]. Modular trellises or meshes are rigid lightweight elements mounted on the building wall [129, 134, 120]. The continuous or modular climbing aids are connected to the façade using a combination of elements such as uprights, brackets, anchors, and spacers, depending on the system [67, 50]. In this study, the weight of the indirect green façade is considered to range between 20 and 30 kg/m^2, based on findings in the literature [164, 10, 35]. This estimation is further supported by a supplier of indirect green systems, who states that their system weighs 25 kg/m^2 [67]. Although lighter systems also exist. For example, Ottelé et al. [124] describe a steel grid structure that adds only 1.6 kg/m^2 in addition to the plant weight. Furthermore, auxiliary structures can have their own load-bearing system, meaning that no self-weight needs to be transferred to the façade [143]. This can be advantageous in retrofit applications, especially when the existing building structure is unable to bear the additional load or when the façade is not suitable for direct mounting. However, it does require additional space in front of the building, along with a supplementary superstructure and foundation.

Figure 2.3: Just planted indirect green facade with continuous guides at Naritaweg, Amsterdam (own work)

2.2.2. Living wall systems

Living wall systems employ different support structures, cultivation methods and growing medium. The following classification level is based on the support structure, which can either be linear, continuous or modular [164, 35]. The linear system consists of linear planter boxes filled with substrate and climbing plants supported by a climbing aid. The continuous system features a continuous screen with integrated pockets for plant insertion, mounted along the facade. The modular system uses separate growing units that are mounted onto modular panels, allowing them to be combined or rearranged.

(a) Indirect green facade with continuous guides [67]

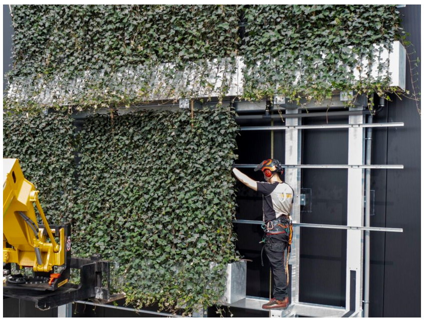
(b) Indirect green facade with modular trellis [67]

Figure 2.4: Indirect green facades

Linear LWS

The linear system is in between green facades and living wall systems as it uses climbers with a similar support system as indirect green facades, but it is rooted in planter boxes around the perimeter of the building or at different heights along the facade [164]. These linear planter boxes are typically filled with an artificial, organic or mixed substrate and are always combined with a hydroponic technique to sustain plant growth [35, 91]. Several studies refer to it as an indirect green facade with planter boxes filled with substrate [130, 23, 88, 25, 134, 96]. Since these planter boxes are facade-bound, thus the full vegetation habitat is realized on the facade, hence nutrients and a watering system is needed and the load of the substrate and planter box is generally also carried by the building, the system is classified as a living wall system in this study [132]. The system name used is curtain system. The components of the curtain system are directly fastened to the facade by brackets and spacers or a substructure of often steel or aluminum is used [120]. The size and material of the containers, the type of substrate, the type and material of the support structure, and the vertical spacing between containers all have a significant influence on the total self-weight of the system. For this reason, a wide weight range of 40 to 140 kg/m^2 is applied. While literature typically cites the lower end of this range [164, 134, 34], both a supplier [97] and a design calculation by Beeren [10] indicate that values around 140 kg/m^2 are not uncommon. The Wallplanter system by Mobilane of 140 kg/m^2 is shown in figure 2.5b [97].

(a) Curtain system of the One Central Park project in Sydney, Australia [67]



(b) Wallplanter by Mobilane [97]

Figure 2.5: Linear living wall systems

Continuous LWS

Continuous systems refer to lightweight cloth or felt systems of permeable and flexible nature, typically composed of multiple layers [43]. The system name applied in this study is felt system. The textile layer is a root proof screen cut in pockets to house plants individually [164]. Shrubs, grasses and perennials are identified as suitable vegetation [88]. The screen is stapled to a base panel (PVC) with a waterproof membrane, which is in turn supported by a frame that protects the wall from humidity and is fixed to the building wall [88, 35]. Fernández-Cañero, Pérez Urrestarazu, and Perini [43] explain that continuous living wall systems mainly consist of pure hydroponic cultures, but the felt pockets can also contain organic substrate. In the pure hydroponic system without substrate, the roots receive all water and nutrients by an often automated irrigation system that is evenly spread by horizontal branches with drippers [123, 130, 43]. The irrigation system is generally installed only at the top of the structure, while the permeable nature of the textile ensures that water and nutrients are distributed throughout the system [125]. In the combined system, part of the nutrients are provided by the organic substrates. Suitable for felt systems are spaghnum moss or topsoil [43].

The type of substrate and the size of the felt pockets have a significant impact on the total self-weight of the system, which explains the wide range of reported values. Literature often suggests higher weights of up to 75 or even 100 kg/m^2 [10, 96], while the maximum weight among four systems from different producers is only 41 kg/m^2 [170, 134]. However, these reported weights generally do not include the weight of water retained in the system, the fastening structure or the vegetation itself. Vertiko states that their complete system, "LivingPANELS", weighs 35 kg/m^2 [169]. The felt system "Plantwall" by DonkerGroep weighs 50 kg/m^2 in total [38]. A sketch of the system and a photo of its fully grown state are shown in figure 2.6. Notably, these commercially available systems all contain only limited amounts of substrate. The well-known felt system by Patrick Blanc, used at the Caixa Forum in Madrid or at the Musée du quai Branly in Paris in figure 2.7, is even substrate-free and weighs approximately 30 kg/m^2 [51]. Therefore, a weight range of 30 to 100 kg/m^2 is applied for felt-based systems.

(a) Plantwall by DonkerGroep in Woerden, the Netherlands [38]

(b) Sketch of Plantwall system by DonkerGroep [170]

Figure 2.6: Continuous living wall system

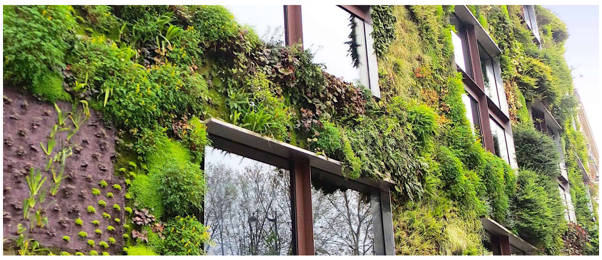


Figure 2.7: Felt system of the Musée du quai Branly in Paris, France [162]

Modular LWS

Modular systems use modular prefabricated panels equipped with organic or inorganic substrate [123]. Currently, there are many variants of modular living wall systems with specific names given to them in literature and in commercial practice [23]. El Menshawy, Mohamed, and Fathy [41] have summed them up as "vessels, trays, flexible bags, planter tiles, wire cages, framed boxes, solid planter boxes with pre-cut holes, or panels". The next classification step is based on the different type of growing units, which is defined as 'the physical structure in which the plants are grown'. This results in three different system types with distinctive characteristics: containers, bags and boxes.

Container

Plants can be cultivated by horizontal containers containing substrate, referred to in literature as trays, vessels or planter tiles [23, 88, 125, 12]. Next to shrubs, grasses and perennials, are succulent plants also listed as suitable for these container systems [88]. Modular trays or vessels are generally composed of horizontally and/or vertically interlocking parts, made of lightweight materials as plastic or metal sheets [88, 12]. They are usually fixed to a horizontal and/or vertical frame that is attached to the building wall. Alternatively, the modular panels can also be directly fixed to the wall with uprights and brackets, without the need for an auxiliary substructure [120]. Planter tiles are generally made from a porous lightweight material, such as low density concrete, ceramics or foam materials [36]. They are designed as modular facade cladding, specifically shaped for plant insertion [164]. These modules can be connected to the wall with standard tile adhesive, mortar and grout or mechanical fastening like screws or make use of an additional substructure to create an offset from the wall. The growing medium in the containers can be either organic or inorganic substrate [130, 125]. Organic fibres (coco peat), sphagnum moss or natural soil are the most common organic substrates [43, 23]. Inorganic substrates, do not provide the plants with any nutrients, which makes the use of an irrigation system crucial for plants survival [43]. Mineral wool or polyurethane foam are examples of inorganic substrates in living wall systems [123], but often producers develop their own formulas to create an optimum growth media [43]. In order to reduce the self-weight of the system, an inorganic substrate can be chosen, since natural soil minerals generally have a higher specific gravity than lightweight inorganic substitutes [70, 35].

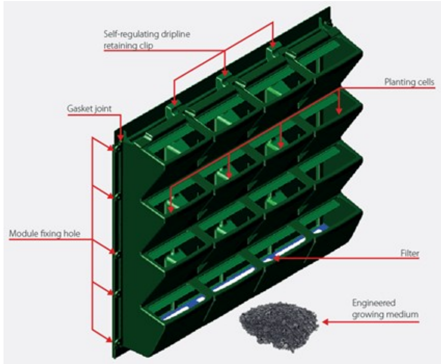
The various components with multiple options of the container systems result in a range of 50 to 150 kg/m^2 [170, 55, 134, 124, 98]. An example of a system weighing 73 kg/m^2 is the “Minigarden” system by Minigarden, made of polypropylene with potting soil as the substrate [94, 170], as shown in figure 2.8. An example of a system that weights 73 kg/m^2 is the “Minigarden” system from Minigarden made of polypropylene with potting soil as substrate [94, 170], shown in figure 2.8. The “Modulogreen” system by Mostert de Winter, weighing 105 kg/m^2, is made of glass fibre-reinforced polypropylene and uses a mixed inorganic and organic substrate, as shown in figure 2.9. Even with the lighter substrate, the Modulogreen system is heavier than the Minigarden system due to the material used for the growing units.

(a) Minigarden by Minigarden in Ho Chi Minh City [94]

(b) Container system by Minigarden [94]

Figure 2.8: Container living wall system

(a) Modulogreen by Mostert de Winter in Delft [99]



(b) Sketch of Modulogreen system by Mostert de Winter [99]

Figure 2.9: Container living wall system

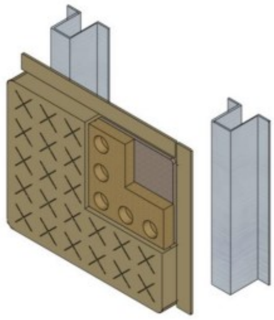
Bag

Flexible bags are specifically useful if a curved surface needs to be greened [88]. They are based on elongated bags connected on their long sides in horizontal direction, consequently suspended as one piece on the long side of the top bag [48]. These waterproof bags, made of flexible polymeric material, are filled with soil and have openings in the vertical plane, into which partially grown trees can be placed [20]. As a result of the system being suspended, the openings close, preventing soil from falling out. This specific flexible bag system with a hanging mechanism is currently not available on the market. Only stacked flexible bags used as green retaining walls are available, not as green facades for buildings [100]. Consequently, this system is left out of scope for this study.

Box

Modular framed boxes often have horizontally planted or grown vegetation as well [164]. These boxes are filled with a semi-rigid artificial substrate, held in place by steel cables or another fastener [126]. The boxes can be directly attached to the wall using uprights and brackets. Alternatively, they can be mounted onto a substructure, such as a waterproof backing board or an aluminum frame, which is then fixed to the wall [96, 120, 14]. Either polyurethane foam or mineral wool is mentioned in literature as a substrate used in these modular elements [123, 35, 143, 170, 43]. Both materials have water-retaining properties, ensuring continuous water availability to plant roots [126]. Systems containing mineral wool typically have a lower self-weight than those using a foam substrate [96]. The self-weight range for box systems with rock wool is estimated at 30 to 80 kg/m^2 [14, 170, 169], while for foam-based systems it ranges from 60 to 120 kg/m^2 [96]. A relatively new and specialized type of box system, called Vertical Meadow, uses a thin layer of rock wool (1 cm) combined with wildflower seeds to create vertical meadows, resulting in an even lighter system of just 25 kg/m^2 [168]. A bare and fully grown version of the Vertical Meadow system is shown in figure 2.11. A more traditional type of box system by Sempergreen, using a thicker layer of rock wool and weighing 45 kg/m^2 [148], is shown in figure 2.10. Based on both literature and practical examples, a self-weight range of 30 to 120 kg/m^2 is assumed for box systems overall. The literature does not distinguish between suitable plant species for box or container systems. Therefore, the same plant species are assumed to be suitable for both types.

(a) Flexipanel by Sempergreen in Stavanger, Norway [32]



(b) Sketch of the Flexipanel by Sempergreen [170]

Figure 2.10: Box living wall system

(a) Meadow Cladding by Vertical Meadow at Gartner facility in Gundelfingen, Germany [168]

(b) Meadow Cladding by Vertical Meadow [168]

Figure 2.11: Box living wall system

2.3. Overview of key features

An important takeaway from this analysis is that VGS have been developed in many different shapes and configurations. The systems described here represent the most common types, though exceptions always exist. The objective is to enable classification of VGS available on the Dutch market based on their distinctive physical features. These distinctive characteristics, along with additional key features of each system, are summarized in table 2.1 for living wall systems and in table 2.2 for green facades.

Main category	Living Wall Systems (LWS)			
Cultivation method	Hydroponic technique with optional growing medium			
Sub-category	**Modular**		**Continuous**	**Linear**
Type of support structure	Modular panels with separate growing units that can be combined or rearranged		Continuous textile screen with integrated pockets for plant insertion	Linear planter boxes with climbing aid
System name	**Container**	**Box**	**Felt**	**Curtain**
		Key features		
Vegetation	Wide range of species, as shrubs, grasses, perennials, succulents		Wide range of species, as shrubs, grasses, perennials	Twining climbers, tendril climbers, rambling shrubs and hanging plants with evergreen or deciduous foliage
Growing medium	(In)organic substrate	Semi-rigid (in) organic substrate	Small amount of organic substrate or none and plants root in felt layer	Soil, (in)organic or mixed substrate
Growing unit	Vessels, trays, planter tiles, cassettes, planter boxes	Steel/plastic panel boxes with front cover, like steel/plastic grid or felt layer	Flexible pockets of multiple textile layers	Horizontal planter boxes
Mounting system	Containers are hooked onto substructure or directly attached to wall with uprights and brackets	Boxes attached directly to wall with uprights and brackets or use substructure like waterproof backing board or an aluminum frame	Screen stapled to base panel with waterproof membrane, in turn connected to a frame that is attached to wall	Planter boxes and climbing aid are fastened to substructure or directly attached to the wall with brackets
Irrigation system	Computerized irrigation, drip line on top of each module		Computerized irrigation, drip line at top of wall	Periodically depending on plants and climate
Growing speed	Fast	Fast	Medium-fast	Medium-slow
Weight (kg/m²)	50-150	30-80 wool 60-120 foam	30-100	40-140

Table 2.1: Overview of Living Wall Systems (LWS) [88, 160, 123, 164, 43, 91, 96, 120, 98, 125]

Main category	**Green Facades (GF)**	
Cultivation method	Rooted into the ground	
Sub-category	**Indirect**	**Direct**
Type of support structure	Climbing aid with air cavity in between the wall and the system	No additional support structure, plants directly adhere to wall by adventitious roots or self-adhesive pads
System name	**Indirect**	**Direct**
Key features		
Vegetation	Twining climbers, tendril climbers, rambling shrubs and hanging plants with evergreen or deciduous foliage	Root climbers and hanging plants with evergreen or deciduous foliage
Growing medium	Ground soil	
Growing unit		
Mounting system	The climbing aid is attached to the wall by uprights, brackets, anchors and spacers	
Irrigation system	Manually, periodically depending on plants and climate	
Growing speed	Medium-slow	Slow
Weight (kg/m^2)	20-30	5-10

Table 2.2: Overview of Green Facades (GF) [164, 160, 123, 67, 130, 91, 125]

3
Biodiversity performance

This chapter explores biodiversity in relation to Vertical Greening Systems. The term biodiversity will first be defined, followed by an explanation of its link to VGS. A literature study on biodiversity performance identifies relevant indicators. These indicators are used as criteria in a Multi-Criteria Analysis (MCA) to evaluate the VGS based on their key features. Relevant animal taxa are considered stakeholders in the MCA, resulting in a biodiversity ranking of the VGS.

3.1. Urban biodiversity

Biodiversity is the common abbreviation for biological diversity [22], which is defined by the Convention on Biological Diversity in 1993 as “the variability among living organisms from all sources including, inter alia, terrestrial, marine and other aquatic ecosystems and the ecological complex of which they are part: this includes diversity within species, between species and of ecosystems” [103]. Notably, this definition does not only include species diversity or genetic diversity, but also ecosystem diversity. It was the first time that an academic definition included ecosystems with both biotic and abiotic components [37]. This is important, because it points towards the fact that the living and nonliving environment must be in harmony in order for biodiversity to remain balanced. These balanced ecosystems are capable of providing ecosystem services, as explained in the introduction. Many services are beneficial to the ecosystems themselves, maintaining the natural system [63].

However, the human species decided to interfere with the balanced natural system and build cities. Cities use the natural system, without providing a feedback loop. A study covering 29 cities from the Baltic Sea region estimated that the areas each city needed for ecosystem support is at least 500 to 1000 times larger than the area of the cities themselves [45]. So humans do not keep the feedback loop of the natural system in tact and in addition, they need more space than the one they already inhabit. Instead of completing the loop, the natural system gets more problems in return, such as human induced climate change, emittance of greenhouse gases and habitat destruction [63]. As explained in the Introduction, biodiversity supports the condition of ecosystems and can contribute to the provision of other ecosystem services that help address the problems previously mentioned. The prevailing view in literature is that VGS have substantial potential to boost urban biodiversity [164, 123, 91, 25, 47, 125, 89, 31]. According to den Hartog [35], key elements to support biodiversity are food, connectivity, reproduction, and shelter. Several studies state that VGS can provide food, reproduction, and shelter opportunities for various animal species [90, 87, 27, 62, 23, 16]. Coherently, there is a growing consensus in the literature that further research is needed to assess the potential of VGS to act as urban wildlife corridors [23, 90, 89]. One empirical study finds that VGS located near existing vegetation are visited significantly more by birds than those with no nearby vegetation [28] and another theoretical study agrees. This suggests that VGS do support urban connectivity in some way and are not only beneficial as exclusive habitat.

3.2. Empirical research on biodiversity performance

The distinctive properties of green facades and living wall systems result in different habitats [35]. Green facades generally provide warm and dry habitats, while living walls provide cooler and damper conditions [87, 27]. These different habitats determine which plant species are suitable, while the combination of habitat and vegetation influences the presence of animal species. Four empirical studies were discovered that researched the relationship between specific animal species and certain VGS, which could provide quantitative evidence for many theoretical studies on the subject of biodiversity performance of VGS. The studies mainly focused on the species richness and species abundance as indicators for the species specific impact of VGS on biodiversity [35]. From the empirical studies and additional supporting sources, key features of VGS that influence their biodiversity performance could be identified.

3.2.1. Results of empirical studies

One research analyzed thirteen felt systems in Bogotá, Colombia, to assess their bird use [16]. No comparison was made between the bird use of a bare wall and the felt system. Additionally, the climate in Bogotá is substantially different from the Dutch climate, which also results in different birds and vegetation. As a result, only general conclusions could be drawn from this study. Interestingly enough, all of the thirteen felt systems were placed on walls close to a national park, which on one hand guaranteed the presence of certain birds and on the other hand did present other suitable nesting sites for these birds. Nevertheless, six nesting events were documented within the living wall samples. This suggests that the structure of felt systems does provide attractive nesting facilities in general.

The second study analyzed arthropods, birds and mammals on twenty green facades and bare walls in Opole, Poland [121]. The research does not make a difference between indirect or direct green facades. The total number of captured insects, the abundance, was higher on the bare walls than on the walls with green facades. Which could be explained by the way the measurements were performed, the yellow sticky plates could be more attractive in the absence of vegetation. No spiders were captured on bare walls, which aligns with the observations done by Madre et al. [87]. The species richness was found to be higher on green facades than on the control walls. Thirteen bird species were observed within the green facades. Four of them were nesting species and the others used the green facades as foraging area. The dense network of vines is mentioned as an important reason for nesting facilities. Furthermore, a very strong and statistically significant correlation between the number of nests and the age of the climbers was established. Additionally, the presence of certain bird species that forage for small invertebrates can serve as an indirect indicator of the local diversity and abundance of insects and spiders. Mammals, like the European hedgehog, brown rat and beech marten were also recorded in the immediate vicinity of the green facades, but only the beech marten was spotted on the branches foraging for food. Nonetheless, the same mammal species were spotted near the control walls, so no conclusions could be drawn on the value of green facades for the biodiversity of mammals.

The third study quantified spider and beetle sightings in eight bare walls, ten climbing plant facades (green facades), six substrate module facades (container system) container and nine felt layer facades (felt systems) across Paris [87]. Beetles and spiders are chosen because they are the one of the most diverse and abundant taxa on earth. Consequently, they are responsible for various functions and ecosystem services, including decomposition, pollination or biological control. Furthermore, their small size is advantageous, as it allows them to maintain stable populations within relatively small habitats. The beetle abundance was significantly lower on the felt systems than on the container system or green facades. There was no significant difference between the latter two. No beetles were captured on the bare walls. The species richness, species abundance of the vegetation in the VGS had a positive effect on the species richness and abundance of the beetles. The spider abundance was substantially different between the different VGS types and showed a significant increase in the following order: bare walls, green facades, felt systems and container systems. The species richness increased in the same order. The high abundance of beetles and spiders in the container systems could be related to the suitable microclimatic conditions, floristic properties and the presence of a sphagnum-based substrate. The sphagnum-based substrate may offer the greatest number of habitats, which could partly be explained by the larger space within the containers. As a result, the habitat created by the container system is also the most structurally diverse one. The high abundance of beetles in the green facades could be explained by a predator-prey relationship between the beetle and the spider, but this is too detailed for the aim of this study. The high species diversity in the container systems could be directly linked to the high plant diversity in that system.

The first part of the fourth study, conducted in Staffordshire, UK, observed birds and snails on twenty-seven green walls and bare walls as control surfaces with similar surroundings [27, 28]. No distinction was made between indirect and direct green facades. Substantially more birds were seen on or around the green facade compared to the bare walls. At locations with bare walls, birds were only found on the roof, while at locations with green facades, birds were found on all areas (surrounding vegetation, wall surface and roof). Hence, ground vegetation becomes more attractive to birds if it is near green facades. That said, birds were always observed on vegetation, like trees or shrubs, but never on the ground. The wall size of the green facade had no significant impact on the bird abundance and species richness. Contrarily, birds on green facades were only spotted on the upper half of the surfaces, regardless of their height. The results suggest that green facades may serve key functions within avian habitats, such as providing nesting sites, shelter, or food. As for the snail population, their species richness and abundance was higher on green facades than on the control walls. Living walls were supposed to be only included in the second part of the fourth study, but some unplanned observations on snail population could be made on the living walls as well. The living walls appeared to be more attractive to snail species, which could be related to great plant diversity, more diverse vegetation structure and the presence of growing media. Furthermore, the leaf litter that can be present in living wall systems could provide hibernation spots and shelter for snails. Together with the irrigation system, the ideal humid habitat is created for snails. Mollusca, the phyla of snails in the taxonomic system [53], are important as they are essential detritivores and act as food sources for birds and small mammals. This could also explain the significant bird sightings on the green facades compared to the bare walls.

The second part of the fourth study analyzed spiders and insects, which represent distinct classes within the taxonomic system, on twenty-nine green façades with control walls and twenty-two living walls without controls [27]. All VGS and their corresponding control walls were situated in the UK, across locations including Staffordshire, London, the Greater London Area, and Stoke-on-Trent. Again no distinction is made between the green facade systems, but the living wall systems could be classified as different systems. Sixteen container systems with different substrates and six box systems with rock-wool units could be identified. The species richness and abundance of spiders were higher on green facades than on the bare walls. Moreover, the species richness and abundance of spiders were positively correlated with plant richness, plant density and age of the wall vegetation. Again no relationship was found between the vegetation surface area and the spider population. Overall, species richness was comparable between green facades and living walls, with living walls showing a slight advantage. Species comparisons showed that certain species were exclusive to either green facades or living wall systems, likely due to differences in environmental and structural composition. Insect abundance and richness was greater on green facades than on their paired control walls. Insect fauna altogether demonstrated little variation in abundance and richness on the living walls, yet individual insect orders displayed significant variation, shaped by specific features of the systems. In addition to total insect abundance and richness, the abundance and richness of the orders Diptera (flies), Coleoptera (beetles), Hymenoptera (e.g. bees and wasps), and Hemiptera (e.g. true bugs and leafhoppers) were also analyzed. These analyses suggest that to enhance Hymenoptera and Hemiptera populations, plant diversity and density should be high, as seen in certain container systems. In contra to the Diptera population that was thriving in less diverse VGS. Hence, the living walls can be designed specifically to enhance or limit specific insect orders. An important conclusion for urban insect biodiversity is that the insect abundance-to-richness ratio was lower on living walls than on green facades, suggesting that living walls tend to attract a more diverse insect fauna.

3.2.2. Indicators of biodiversity performance

The literature study on the biodiversity performance of VGS resulted in the identification of four key indicators. These relate to characteristics of the various VGS. Each indicator will be introduced and its origin explained. In addition, various biodiversity recommendations that can not be attributed to a specific system are also mentioned below.

Plant coverage

The empirical animal biodiversity studies identify plant coverage, also referred to as plant density or foliage density, as an important feature of VGS [16, 27, 121, 87]. Den Hartog [35] states a higher foliage density or a thicker layer of vegetation as the most important design measure regarding the ecological value. Chiquet, Dover, and Mitchell [28] observed a positive correlation between foliage density and spider populations, suggesting that this relationship can be explained by the provision of shelter and protection. Furthermore, two studies that either identify suitable plant types for VGS or analyze their growth performance, also acknowledge vegetation density as an important parameter in shaping suitable habitats for various animal species [160, 69].

Plant diversity

Plant diversity, often described as plant variety, richness, or heterogeneity, is recognized as an important parameter by the empirical biodiversity studies [16, 27, 121, 87], as well as by many theoretical ones [47, 35, 90, 91, 164]. This is partly due to the fact that many animals are highly plant species-specific [35]. As noted by Chiquet [27], attracting certain animal species requires the use of targeted plant species [23]. For instance, certain plant species can serve as host plants for the reproduction of butterfly species [2]. Generally, birds show a preference for evergreen climbers over deciduous ones, particularly those that grow taller [28], although different bird and insect species may be attracted to varying vegetation heights for roosting, nesting, or foraging purposes [2, 11]. Moreover, bird species have different dietary preferences: many urban birds favor shrub-like plants for shelter or berries, grasses for their seeds, and perennials for their floristic properties, which also attract insects that form part of their diet [104, 2, 87, 16]. Beetles and spiders have been observed to show greater abundance in systems with more floristic plant species [87]. Timur and Karaca [160] also emphasize that the type of bird or insect species attracted largely depends on the specific plant species present. Pollinators such as bees, butterflies, and certain birds are particularly drawn to nectar-rich plants with strong floristic characteristics [104]. Additionally, Mayrand [90] recommends the use of native plant species to enhance plant survival and support local biodiversity. This is also emphasized by Law, as noted in section E.3. Law suggests that a combination of high plant diversity and density may provide sufficient habitat complexity for the system to become self-manageable (section E.3). Designing for biodiversity also involves selecting plant species with a variety of functional traits [90], such as combining deciduous and evergreen species and incorporating a mix of fruit- and flower-producing plants [35]. Consequently, systems that promote greater plant diversity are more likely to support diverse animal communities, thereby contributing positively to biodiversity.

Substrate size

The thickness of the substrate layer, or more generally the size of the substrate, is considered a key parameter. This can be explained in two ways. On the one hand, a lager substrate volume enhances root development and healthy root systems are essential for plant performance [35, 125, 47]. However, as Law points out, roots are geotropic, meaning they grow downward (section E.3). This makes the horizontal dimension of the substrate less relevant for rooting space. This explains why the Vertical Meadow system developed by Law uses only a 1 cm thick layer of rock wool, while allowing roots to grow freely in the vertical direction. On the other hand, a lager substrate volume in both directions creates more habitats for various arthropods, which positively affects species richness and diversity [87, 90, 27].

Substrate orientation

In addition, the presence of growing media on horizontal surfaces is recognized as an advantage, as it results in the accumulation of plant litter, a high degree of moisture and horizontal nesting surface. These factors create an attractive habitat for arthropod and mollusk populations [28]. Growing media and litter facilitate opportunities for shelter. Furthermore, part of this litter consists of decaying plant material, which increases the organic matter content of the substrate and can stimulate microbiological activity [90]. Increased microbiological activity has been positively linked to biodiversity and can reduce the need for artificial fertilizers [35]. In addition, both the horizontal substrate surface and accumulated plant litter may serve as resources for nesting birds, which are generally more attracted to elevated locations due to the reduced risk of predation [172, 84, 28]. Hence, the presence of a growing medium is considered beneficial for biodiversity performance, particularly when the substrate is open and positioned horizontally, maximizing litter accumulation and moisture retention.

3.3. Multi-Criteria Analysis on biodiversity performance

These indicators serve as criteria in a MCA of biodiversity performance. The first step involves assessing the VGS based on these indicators, resulting in a ranking of the VGS for each indicator. As ties are possible, the scales may differ between indicators, therefore a standardization step is required. Animal groups or taxa studied in relation to VGS are recognized as stakeholders to inform the weighting process. The final outcome is a ranking of the VGS based on their estimated biodiversity performance.

3.3.1. Assessment using biodiversity indicators

Each VGS type is assessed using a representative variant, chosen as a typical example within its category. The evaluation is based on the five biodiversity indicators identified in the literature study. These indicators are used to analyze and compare the features of each system, resulting in a ranking per indicator. The following list briefly describes the representative variant selected for each VGS type. These variants serve as the basis for the subsequent evaluations and are intended to reflect typical characteristics found within their respective VGS types.

- **Container system**: features high plant diversity and trays containing organic substrate.
- **Box system**: features high plant diversity and vertical boxes containing inorganic substrate.
- **Felt system**: features high plant diversity and substrate-less textile pockets.
- **Curtain system**: features multiple climbing species and horizontal planter boxes containing organic substrate, placed at each floor level.
- **Indirect system**: features multiple climbing species
- **Direct system**: features a single climbing species

An overview of the indicator scores per VGS type is provided in table 3.1, along with the total scores for additional insight.

	Container	Box	Felt	Curtain	Indirect	Direct
High plant coverage	3	4	3	2	2	1
High plant diversity	3	3	3	2	2	1
Large substrate size	2	2	1	3	4	4
Horizontal substrate orientation	4	1	2	4	3	3
Total	12	10	9	11	11	9

Table 3.1: Scores per indicator for each VGS

High plant coverage

Foliage density can be quantified using the Wall Leaf Area Index (WLAI), which indicates the number of square meters of leaf surface per square meter of facade [35]. Den Hartog [35] outlines an approximate ranking of potential plant types for living walls based on increasing WLAI, listing succulents, grasses, perennials, and shrubs in ascending order. Hence the different living wall systems can't be differentiated based on foliage density. The same applies to plant density, which can be expressed by the number of individual plants per square meter of wall surface. One study used a system in which the felt system had a higher plant density than the container system [159], while another study found the opposite [27]. Chiquet [27] also notes that this parameter can be adjusted within the systems and is therefore not system-specific. Climbers have the potential to develop denser foliage than living wall systems, but this strongly depends on favorable ecosystem conditions and requires several years [35]. In contrast, the ecosystem of modular and continuous living wall systems can be actively managed through irrigation systems designed for the selected plant species, and pre-planted modules can be used to accelerate establishment [43]. Even if seedlings are used, continuous and modular living wall systems allow rapid coverage [125]. Therefore, the plant coverage is considered higher for modular and continuous living wall systems than green facades. Jim [69] finds that plants suitable for indirect green facades tend to have denser foliage and higher growth rate than those suited for direct green facades. Linear living wall systems have similar coverage as indirect green facades, since the growing support and suitable plant types are the same.

High plant diversity

Green facades typically incorporate only a few plant species, resulting in mono-specific covers and low diversity, whereas living wall systems often feature a more diverse vegetation cover [164]. This is supported by both Chiquet [27] and Madre et al. [87], who used standard market systems that revealed significant differences in plant diversity, with continuous and modular living wall systems exhibiting notably higher diversity. According to Chiquet [27], continuous and modular living walls can accommodate almost any plant species, including ferns, perennial flowers, low shrubs, vegetables, and herbs. Van Reeuwijk [164] notes that the Caixa Forum in Madrid, classified as a felt system [137], incorporated 250 different plant species. Furthermore, Mayrand et al. [90] identified only eight suitable species for direct green facades, compared to approximately 250 species for continuous and modular living wall systems. Although Lepp [80] listed 30 to 50 species suitable for living curtains and green facades in temperate climates, this number remains considerably lower than the variety suited to continuous and modular living wall systems. Jim [69] reported that, among 130 tropical climbers considered in a growth performance study in a humid subtropical climate, only a limited number were suitable for direct green facades, whereas the majority were appropriate for indirect green facades. This difference in plant diversity is reflected in the representative variants: direct green facades receive the lowest score, while the indirect and curtain systems share second place. The plant diversity of curtain systems is considered comparable to that of indirect green facades, as both rely on similar support structures and are therefore suited to similar plant species. The remaining living wall systems all receive the highest score.

Large substrate size

Indirect and direct green facades share the same cultivation method and growing medium, as plants are rooted in the ground adjacent to the building. Consequently, both facade types receive the same score for substrate size. This score is based on the available rooting space and potential for habitat creation. While linear living wall systems may use a similar growing medium, they differ in cultivation method, as plants are rooted in planter boxes mounted on the facade. It is assumed that planter boxes offer less rooting space and habitat potential compared to soil-based systems. Therefore, linear living wall systems receive a lower substrate size score than green facades, with a score of three. The substrate size of modular living wall systems is considered smaller than that of linear systems. Box and container systems are considered approximately equal in this regard, both receiving a score of two. In felt-based systems, plants can root directly into the felt layers, making the system effectively substrate-less. As a result, these systems receive the lowest substrate size score.

Horizontal substrate orientation

Substrate orientation is a key differentiating factor among living wall systems, influenced primarily by moisture retention, plant litter accumulation, and nesting potential. However, because moisture retention also depends heavily on the specific substrate and drainage types used, which can vary within a single system type, it is not considered a consistent or distinguishing factor when assigning substrate orientation scores [125, 43]. A horizontal substrate orientation is present in green facades, felt systems, curtain systems, and container systems. Box systems contain a substantial amount of substrate, but due to their vertically oriented openings and substrate, they prevent leaf litter from accumulating, resulting in the lowest substrate orientation score. Felt systems, although lacking substrate altogether, are classified as horizontally oriented. This orientation offers increased nesting potential compared to the vertical configuration of box systems, therefore felt systems receive the second lowest score. In contrast, green facades contain only a single substrate surface located at ground level. While this provides some horizontal orientation, it offers limited nesting opportunities for birds, which generally prefer elevated locations to avoid ground predators [28]. Curtain and container systems include multiple horizontal substrate layers distributed across the facade, offering both multiple horizontal substrate levels and higher nesting positions. As a result, these systems receive the highest substrate orientation scores.

3.3.2. Standardization

The assessment of the VGS uses an ordinal scale, which indicates relative performance. No precise measurements are used, hence no exact differences between the ranks can be determined. The VGS are ranked from worst to best, with the possibility of ties. This results in different scales for the various indicators, where either a three-point or a four-point scale is applied. For example, plant coverage resulted in a three-point scale, while substrate size resulted in a four-point scale. The VGS with the lowest performance on an indicator receives a score of one. The VGS with the highest performance receives a score of three or four, depending on the scale used. To preserve relative proportions, the scores are standardized. The method of maximum standardization is applied, ensuring that the highest-performing VGS for each indicator receives a score of 6 [139]. The minimum score is based on the original scale and is deliberately not set to zero, as this would neutralize the effect of weighting and hinder the accurate representation of relative performance. This results in applying equation (3.1) for standardization.

$$x'_i = \frac{x_i}{x_{max}} \cdot 6 \tag{3.1}$$

	Container	Box	Felt	Curtain	Indirect	Direct
High plant coverage	4.5	6.0	4.5	3.0	3.0	1.5
High plant diversity	6.0	6.0	6.0	4.0	4.0	2.0
Large substrate size	3.0	3.0	1.5	4.5	6.0	6.0
Horizontal substrate orientation	6.0	1.5	3.0	6.0	4.5	4.5
Total	20	17	15	18	18	14

Table 3.2: Assessment results of VGS based on standardized scores for biodiversity indicators.

3.3.3. Assignment of impact weights

After standardization, weights are assigned to the indicators, based on their relative impact on biodiversity. Weight determination is carried out systematically for each relevant animal group selected as a stakeholder, as described in the following section. The final biodiversity score is obtained by aggregating the weighted indicators, where the assigned impact weights reflect the relative contribution of each indicator to the overall biodiversity performance.

Relevant animal groups as stakeholders

Literature refers to different taxa within the taxonomic system, which serves as the biological classification framework for living organisms. Taxa are the categories in which organisms are grouped based on shared characteristics, and these are organized into taxonomic ranks. Taxonomic ranks are the hierarchical levels of classification, such as kingdom, phylum, class, order, family, genus, and species. This research focuses on the animal kingdom, which encompasses a wide range of taxa across these ranks. Figure 3.1 presents the animal taxa addressed in the empirical studies within the taxonomic system, illustrating their respective positions and relationships. The darker brown boxes represent examples of animal groups of the taxa that is connected to it. For example, all birds belong to the class Aves, but the term ‘bird’ is more commonly used in literature and is easier to understand. The same applies to spiders and the order Araneae. In addition, a snail is a type of gastropod, but not all gastropods are snails, and snails do not fall under a single taxon at the next taxonomic rank. Diptera, Coleoptera, Hymenoptera, and Hemiptera are highly diverse orders, hence examples of commonly occurring species are provided.

A greater taxonomic distance between species typically indicates fewer similarities, contributing to higher overall species diversity. However, species from different taxa also fulfill various ecological roles that are essential to maintaining a balanced ecosystem. Studies often refer to functional groups such as pollinators, fungivores, detritivores, predators, or prey [27]. Well-known pollinators include bees and butterflies, but certain beetles, ants, birds, and bats can also fulfill this function [46]. In the absence of pollinators, most plant species would be unable to reproduce [35, 86]. As many of these plants serve as essential food sources for other animals, this would lead to significant disruptions within the ecosystem. Chiquet [27] also explains that Diptera, Coleoptera, Hymenoptera, and Hemiptera were specifically selected for their study, as these insect orders serve as key food sources for various vertebrate species, are commonly referred to as bio-indicators, and include many important pollinators. Spiders generally inhabit areas with a high abundance of invertebrates [27]. As such, the presence of spider fauna can

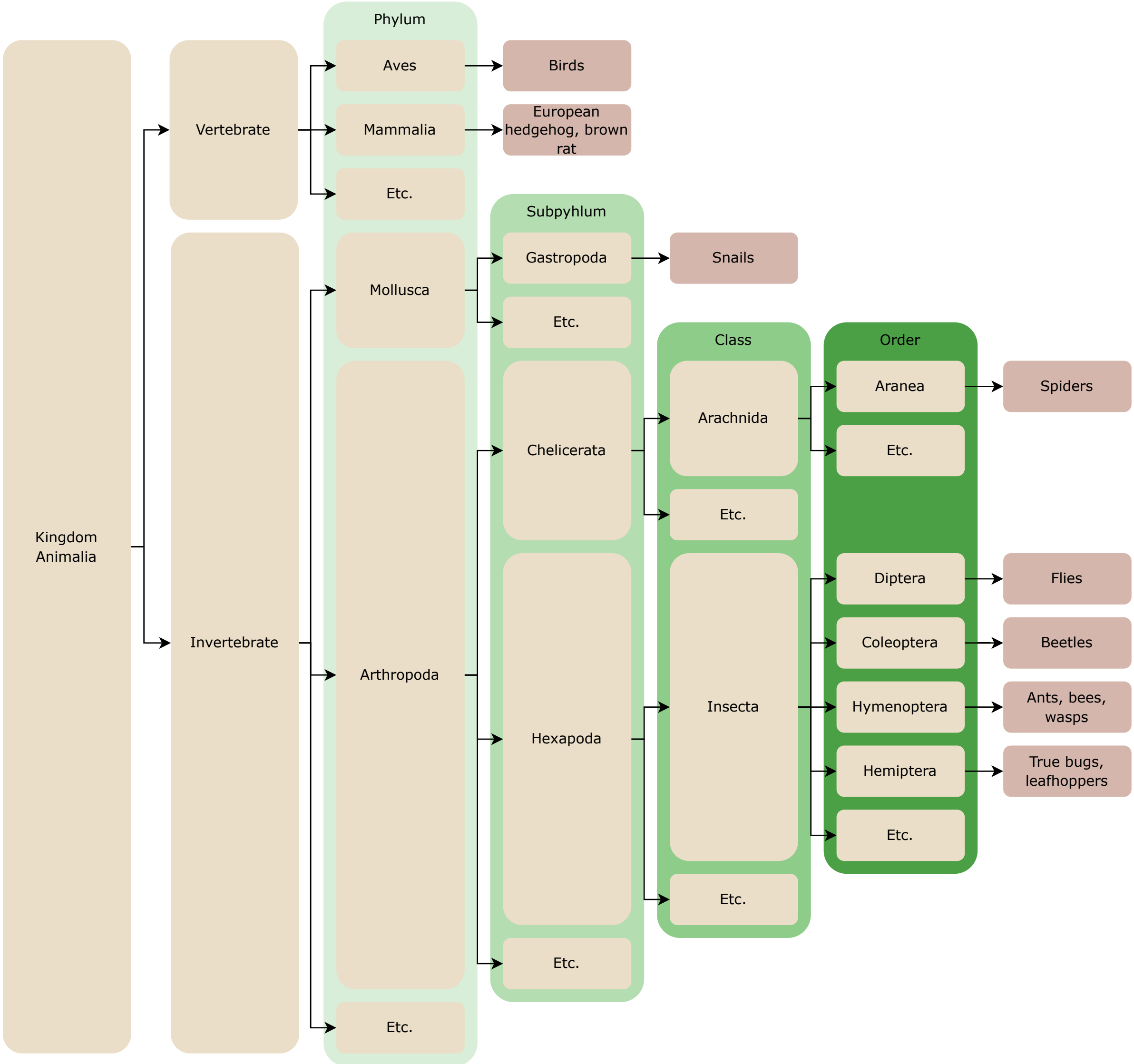


Figure 3.1: Taxonomic system of the Animal Kingdom showing taxa and examples of animals researched in relation to VGS [8, 27, 101, 53, 65, 105, 122, 102]

provide additional insight into the overall biodiversity performance of VGS. Moreover, Madre et al. [87] state that Coleopterans and spiders are among the most diverse and abundant taxa on Earth. Consequently, they contribute to many functions and ecosystem services, such as decomposition, pollination and biological control. Snails have also been identified as providers of various ecosystem services, such as feeding on living and decaying plant material (detritivores) and acting as prey to birds and small mammals [27]. Birds are predators of various invertebrate and many bird species are very familiar with the urban ecosystem. Two of the five indicator species listed for the urban habitat in the province of Zuid-Holland are birds [17]. Similarly, three of the ten indicator species identified for the province of Utrecht are also bird species [7]. Indicator species represent a type of habitat with specific characteristics and are therefore also indicative of other various plants and animals [7]. Consequently, birds can be regarded as relevant taxa in the urban habitat. The stakeholders in this analysis are selected based on the fact that they belong to different taxa, play vital roles in ecosystem functioning, are frequent inhabitants of the urban ecosystem, represent distinct perspectives on biodiversity indicators and are substantiated by findings from empirical studies on VGS. Birds, snails, Diptera, Coleoptera, Hymenoptera and Hemiptera are selected as stakeholders in the analysis.

Determination of impact weights

The more stakeholders consider a biodiversity indicator relevant and agree with the objective it reflects (for example "high" plant coverage), the higher the weight assigned to that indicator. The relationship between an indicator and a stakeholder can be specifically studied in literature, mentioned as an explanation in a study or not addressed at all. Lastly, literature may not explicitly state the relationship, but can provide supporting information from which such a connection can be inferred. Each indicator is examined for its relevance to the stakeholders. Based on how many stakeholders are impacted by an indicator, an impact weight is assigned. All weights together sum up to one. Table 3.3 shows the number of invested stakeholders per indicator and the resulting impact weights.

	High plant coverage	High plant diversity	Large substrate size	Horizontal substrate orientation
Coleoptera	0	0	0.5	0
Hymenoptera	1	1	0	0
Diptera	0	0	1	1
Hemiptera	1	1	0	0
Birds	1	1	0	0
Snails	0	1	1	1
Spiders	1	1	0.5	1
Total	4	5	3	3
Impact weights	0.27	0.33	0.2	0.2

Table 3.3: Determination of impact weights based on the supporting number of stakeholders

The tree and shrub structure have been proven to influence bird responses [28]. In addition, they prefer evergreen over deciduous green facades in winter, this indicates that a higher plant coverage is more attractive to bird populations [28]. Moreover, nesting sites have been observed in both systems with horizontal orientation as in climber plants [27, 132, 172] and box systems with thick vegetation (section E.3), so it cannot be clearly stated that birds have a specific preference for horizontal substrate orientation. Bolhuis [16] explains birds are attracted to nectar-producing flowers, but also to plant species with seeds or fruits, which indicates that plant diversity is also valued by diverse bird populations. No relation between birds and the other indicators has been found in literature. Chiquet [27] state that snails benefit from a high diversity of plant composition, as this can enhance structural heterogeneity. In addition, the presence of rooting medium and leaf litter can improve snail habitats by providing shelter and maintaining humidity. However, specific aspects such as plant coverage of VGS are not directly valued by snails. Spiders prefer a diverse and dense plant coverage [27] in a damp habitat [87]. Furthermore, the presence of a substrate is positively associated with spider abundance [87], while substrate orientation does not appear to be a significant factor. Coleopterans are not particularly selective in their habitat preferences [27], as they mainly require a cool, humid environment with the presence of substrate [87]. The relationship with substrate size was observed only for coleopteran abundance, not for species richness. The same result was found for spiders. Therefore, they are jointly considered as a single stakeholder for this indicator, rather than as two separate ones. Hymenoptera and Hemiptera richness and abundance is positively correlated with plant density and diversity, while dipterans prefer to have access to substrate and leaf litter [27]. The soil and litter layer serves as a crucial habitat for overwintering and the larval or pupal stages of many insect species [27].

3.3.4. Biodiversity performance ranking of VGS

The impact weights are multiplied with the standardized scores to determine the overall biodiversity performance ranking of the VGS, as presented in table 3.4. As an ordinal scale was used, only a performance ranking can be derived from the scores and no conclusions can be drawn about the magnitude of the differences between them. This results in the ranking shown in figure 3.2.

Indicator	Container	Box	Felt	Curtain	Indirect	Direct
High plant coverage	1.2	1.6	1.2	0.8	0.8	0.4
High plant diversity	2.0	2.0	2.0	1.3	1.3	0.7
Large substrate size	0.6	0.6	0.3	0.9	1.2	1.2
Horizontal substrate orientation	1.2	0.3	0.6	1.2	0.9	0.9
Total	5.0	4.5	4.1	4.2	4.2	3.2

Table 3.4: Weighted biodiversity scores for each VGS type

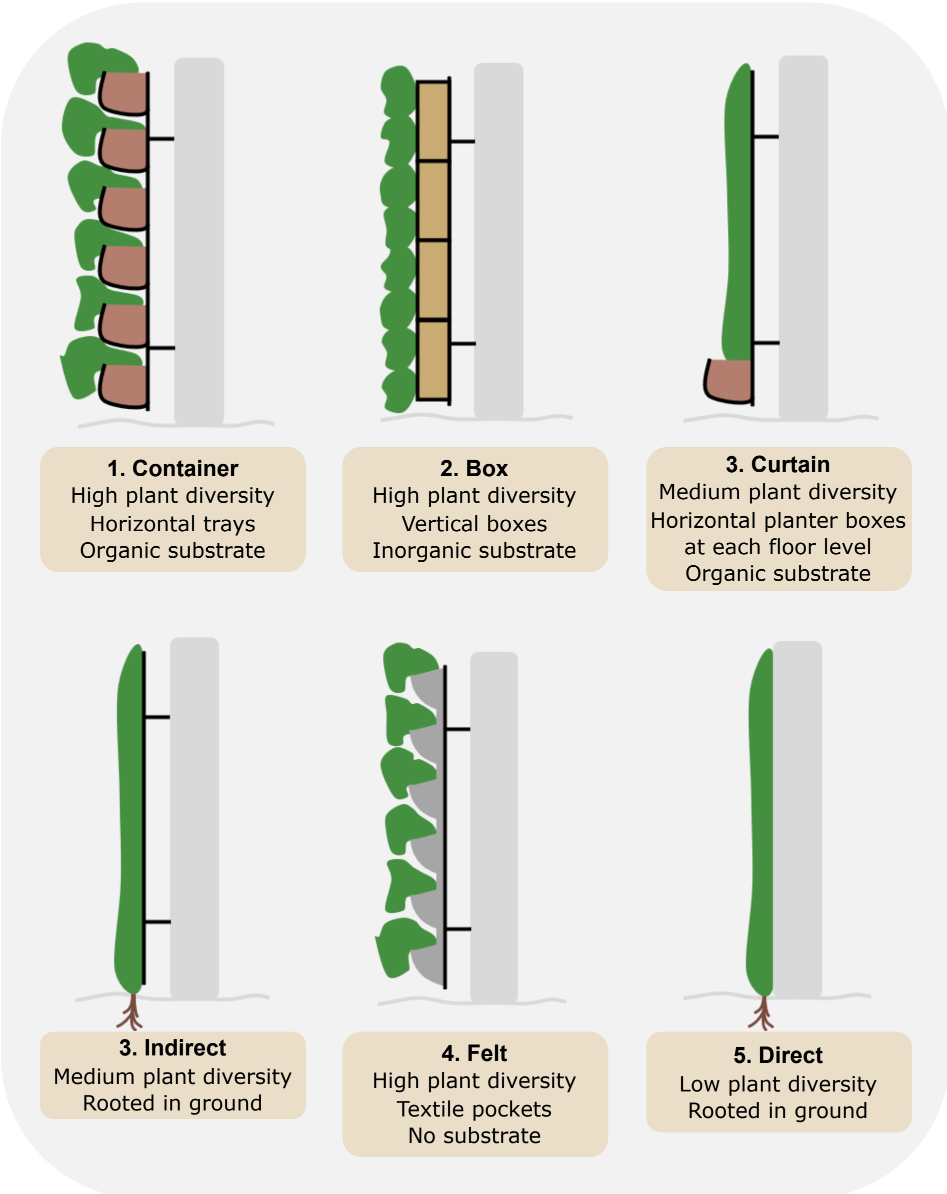


Figure 3.2: Biodiversity performance ranking of VGS

An important disclaimer regarding the analysis of VGS features and relevant animal groups is that most VGS can be specifically designed to target particular animal species. Many species also show a preference for organic over inorganic substrates or are attracted to specific plant species rather than general vegetation types associated with a certain VGS [27, 160]. In this analysis, general distinguishing characteristics were used as the basis for comparison, so not all relevant factors could be taken into account.

Moreover, the applied method did not account for potential interactions between different biodiversity indicators or between different animal species. However, in practice an ecologist can provide additional insight when designing for particular target species and urban ecosystems. Furthermore, most animal species have additional preferences related to abiotic factors, coming from the location of the building and orientation of the facade. Variables such as sunlight availability, wind exposure, human disturbance, habitat connectivity and artificial lighting also significantly influence the attractiveness of a given system for specific animal species [63]. In addition, Law presses the need for a focus on local biodiversity with sufficient plant diversity and complexity, so the systems can manage themselves (section E.3).

4

Existing Dutch office buildings

In order to determine the suitability of implementing VGS onto existing dutch office buildings, the key features of the VGS are discussed. This analysis results in a set of building indicators that can be used to assess the overall suitability of a building for each VGS. The building indicators are linked to general building characteristics, which are analyzed at the level of office building types along with facade types. The result is a description of four office building types, each with a set of common facades and corresponding VGS implementation recommendations.

4.1. Building indicators for VGS suitability

Building indicators are high-level categories of physical building properties that influence the suitability of VGS implementation. They identify which aspects of the building are possibly relevant when evaluating VGS applicability. These indicators were derived by analyzing the key physical features of various VGS types, as discussed in the sections below.

4.1.1. Building height

The height of office buildings can vary significantly, ranging from low-rise constructions to high-rise towers. No specific height limitations have been found in literature for box, container, felt or curtain systems. Naturally, green facades depend on the growth potential of the plant species used. After all, different climbing plants exhibit different maximum attainable heights [69]. Some climbers grow 5 to 6 meters, while others can reach heights of 10 or even up to 25 meters [134, 125, 124]. In addition, the support structure of indirect green facades and curtain systems helps prevent vegetation from falling, which becomes increasingly important in taller buildings [125]. Box systems also offer this safeguard through their front cover, while felt and container systems generally lack such a mechanism. For installations at significant heights, it is therefore recommended to include a suitable safety feature. Based on these considerations and assuming the use of standard system configurations, curtain and box systems are generally recommended for application in high-rise contexts. Regardless of where the systems are installed on the facade, all components must remain accessible for maintenance, whether for pruning or plant replacement. This requirement should always be taken into account when designing a VGS for a specific building.

4.1.2. Building shape

The shape of a building is recognized as an indicator for the suitability of VGS. Buildings are often shaped as a single rectangular volume. However, if the building consists of stacked volumes or is located close to other high-rise buildings, it is likely that more shaded areas are created on the facade. This should be taken into account when selecting an appropriate VGS. Modular and continuous living wall systems provide separate growing units, allowing for a high degree of plant variety. This makes them particularly suitable for dealing with microclimatic variation across a single facade. In contrast, plant species used in green facades are rooted at the base of the building, meaning the same species must typically be used over the full height of the facade. Moreover, stacked building forms often limit green facades to the base section of the building, while larger greening coverage tends to yield greater biodiversity benefits. As such, a modular or continuous living wall system would be more suitable for these types of buildings. Naturally, different VGS types can also be combined. To minimize maintenance in green facades, it is advisable to select plant species whose natural growth height matches the height of the facade. As outlined under the previous indicator, climbing plants differ significantly in their maximum attainable height. The curtain system faces similar limitations in plant variety as green facades, making it less suitable for dealing with microclimatic variation across a single facade, as a single species is typically used over the full height. However, the stacked building shape may facilitate the placement of linear planter boxes. These boxes can potentially rest on an intermediate roof of a lower block, rather than being fastened to the main facade using heavy-duty brackets. The latter option would impose more specific structural demands on the facade, as not every facade can accommodate such brackets. There are several types of brackets available, some of which are specifically designed for externally or internally insulated facades. Certain brackets require a concrete or masonry backing layer to anchor into, while others are compatible with steel or timber structures [83]. If the planter boxes can rest on an intermediate roof, this would add a load to that roof, but generally avoids complex fastening requirements. Naturally, the load-bearing capacity of the roof must be verified to ensure it can support the additional weight at the intended location.

4.1.3. Facade geometry

Many office buildings have planar facades, which are generally suitable for all VGS types. Another facade geometry that is typical of office architecture is one with a curved facade, which can also limit the applicability of VGS. While any VGS can be installed on a curved facade with the help of a tailored substructure, some systems are more easily adapted to such conditions. For starters, direct green facades are not affected by the curvature of the surface they grow on. In addition, the mesh and cable systems used in indirect green facades are versatile and well suited to curved surfaces [43]. Furthermore, an auxiliary substructure supporting the vegetation does not necessarily need to follow the exact shape of the facade. It can deviate from the facade's curvature and remain planar. As with traditional double-skin facades, it is beneficial for maintenance purposes if the distance between the facade and the green layer is large enough to allow human access. This principle is applied in the curtain system shown in figure 4.1. Felt systems have also been successfully applied on curved surfaces, as demonstrated by the curved facade of the Musée du quai Branly in Paris, shown in figure 4.2. In contrast, modular living wall systems are typically less suitable for curved facades, due to the rigid shape of their individual growing units.

Figure 4.1: Curtain system in Natters, Austria [44]

Figure 4.2: Felt system of the Musée du quai Branly in Paris, France [15]

4.1.4. Facade glass percentage

As figure 4.1 and figure 4.2 clearly illustrate, the presence of windows or the glass percentage of the facade is also a determining factor in the choice of system type. The facade in figure 4.1 has a high glass percentage, hence the curtain system is placed in front of the facade. In contrast, the glass percentage in figure 4.2 is sufficiently low to allow the felt system to be applied around the windows. Naturally, windows are placed for various reasons, one of which is particularly important: research has shown that sufficient daylight is essential for human health [167]. Accordingly, the Dutch government has implemented regulations regarding daylight access [95]. Two relevant new terms are introduced: open and closed systems. Felt, container, and box systems are considered closed systems, as they permanently block daylight when installed in front of windows. Consequently, closed systems must be attached only to closed sections of a facade. Furthermore, the anchors or brackets typically used to fasten these systems cannot be mounted into glass. As a result, fully glazed facades, such as glass curtain walls, are generally not suitable for closed systems. Important to realize when these closed systems are placed around windows or especially door openings, is the placement of the irrigation system. Predominantly with modular systems, where the driplines are generally placed above each module, it is important to also take care of the excess water dripping down below.

Green facades and curtain systems are considered open systems, as they can be designed in such a way that they do not fully block incoming daylight. As illustrated in figure 2.4a and figure 2.5a, curtain systems or indirect green facades with continuous guides are often placed in front of windows, as the guides make it easier to direct the vegetation across the glazed facade in a desired way. They can even be used to create a designed green pattern [70]. In contrast, net or grid-shaped climbing aids, as shown in figure 4.1, are typically used to cover entire surface sections, rather than allowing for precise guidance along predetermined lines. Additionally, figure 4.3 illustrates that other creative and functional green designs can also be achieved using these systems. Moreover, climbing or hanging vegetation generally requires regular pruning. This need becomes particularly critical when the vegetation is positioned in front of windows or glazed surfaces, as it may obstruct daylight entry or hinder access for facade maintenance. Furthermore, Jakob Rope Systems [66] indicates that twining climbers can exert significant tensile forces on the guides by twisting around the rope structure, for which pruning serves as the essential corrective measure. Direct green facades are not desirable in front of windows, as they make window cleaning difficult. Moreover, unlike indirect systems, they cannot be guided or shaped by design. Therefore, direct green facades are only recommended for predominantly opaque facades.

Figure 4.3: Green shade roof in Zurich Oerlikon, Zwitserland, with a net climbing aid [67]

Furthermore, a choice must be made between deciduous and evergreen plant species for use in curtain and green facades [125]. Although, combinations of multiple plant species are also possible. Evergreen species can provide shading in summer and protection from wind, snow, and rain in winter [134]. However, since existing facades are usually designed to withstand such environmental conditions and since solar heat gain through windows is often desirable during winter, evergreen vegetation may not always be beneficial there. Depending on facade orientation, deciduous species placed in front of facades can reduce cooling demand in summer while facilitating passive solar heating in winter. Consequently, within the context of Dutch building retrofit strategies, deciduous species are typically favored for placement in front of windows. In the case of closed facade sections, the assessment of heating and cooling effects involves numerous additional parameters and therefore falls outside the scope of this study [164]. Based on these considerations, it is concluded that indirect green facades and curtain systems are the only recommended types for buildings where the facade has a high glazing percentage. The specific type of plant species and support structure should be chosen in accordance with the building's shape and the characteristics of the facade.

4.1.5. Facade type

As mentioned in the previous sections, the type of facade is an important factor to consider before implementing a VGS. This includes not only the facade cladding, but also the composition, layering and structural function of the facade. The modules of the container system may be mounted onto a substructure, such as aluminum angle profiles or a timber frame, which is in turn connected to the wall using wall brackets [99]. A small ventilation layer is always present between the modules and the wall, allowing moisture from rain or irrigation to evaporate. The container modules themselves form a waterproof layer or an additional waterproof backing board is applied [55]. In this way, the modules can serve as the outer leaf of a cavity wall, with insulation placed in the space between the modules and the structural wall [99]. Ideally, the wall is constructed from a mineral-based material such as concrete, masonry or sand-lime brick. However, a timber frame structure is also a viable option. The container modules can also be mounted directly to the back wall using screws, bolts or resin anchors, applied to a structural layer made of plywood, aluminium, steel, concrete or masonry [135]. LLC [83] states that the installation contractor is responsible to select appropriate anchors. One advantage of using a substructure is that the attachment points to the back wall can generally be spaced further apart. As a result, buildings with a column-beam load-bearing structure or a frame structure in the facade are more suitable for substructure-based installation than for direct fastening.

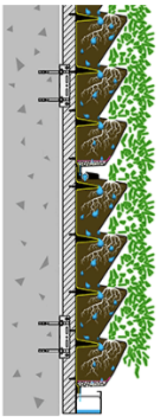


Figure 4.4: Container system attached to a wall by an aluminum substructure and wall brackets [99]

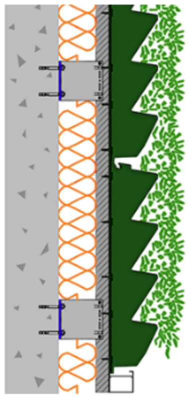


Figure 4.5: Container system attached to a wall by an aluminum substructure and wall brackets with insulation in between [99]

The producer of a box system, 90deGREEN [1], emphasizes that the most important requirement for the façade is its ability to support the load of the VGS. Concrete, steel, or timber are mentioned as examples for suitable base materials for the wall. Figure 4.6 illustrates how this box system is attached to the wall, in a similar way to the container system shown in figure 4.5, using an aluminum angle profile, spacer, anchor, and screw. The producer of the 'Pflanzwand Eva' system makes a similar statement and also explains that their box system can replace the outer leaf of an external thermal insulation composite system [141]. The curtain systems from Mobilane [97] and GSky [54] do not specify particular requirements for the material of the back wall, as the fastening method is either considered outside the scope of the system producer or too dependent on the specific building context. Instead, emphasis is placed on the need for a sufficiently strong load-bearing structure. Jakob Rope Systems [68] developed a special high-load bracket for retrofitting externally insulating facades with a concrete load-bearing inner leaf with indirect green facades. These brackets can handle higher loads than earlier brackets on the market and can be placed without opening up the renovated facade, so it does not compromise the insulating properties. A similar bracket has been developed by Sto [158], but can also be anchored in perforated brick masonry next to concrete.

Köhler [75] describes VGS as a means of protecting the facade, thereby reducing the maintenance required over time. However, all types of VGS demand a certain level of maintenance, with green facades generally requiring the least [130, 129, 128]. Furthermore, vegetation of direct green facades can grow on most facade materials, provided there is an appropriate match between wall moisture levels, wall texture, and the specific plant species [70]. Literature warns against the application of such

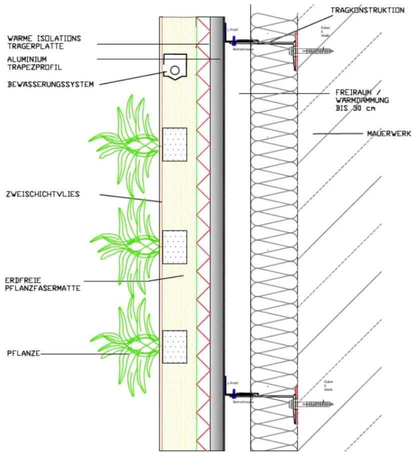


Figure 4.6: Box system attached to a wall by an aluminum substructure and wall brackets [1]

systems on existing facades, particularly on brick walls [43, 134, 123, 28], as these walls may have small cracks that can be penetrated by the aggressive roots of certain woody climber species [70, 131]. However, even if the original wall cladding is in good condition, sucker roots can leave marks on certain materials such as marble or limestone, which become visible once the direct green facade is removed [123, 96]. For this reason, the application of direct green facades is generally not recommended on masonry or marble facades. Summarizing the findings on indirect VGS, all indirect VGS are ideally mounted to a concrete or masonry facade, although steel or timber framework is not excluded. The latter two requires more precision when mounting the system. Additionally, the indirect VGS types have developed anchors or brackets so they can be combined with externally insulated facades as well. Most VGS can be tailored to a facade type by the use of a substructure that can be connected to the load-bearing structure of the building.

4.2. Dutch office architecture

This section provides an overview of Dutch office architecture between 1960 and 1990, highlighting key architectural movements, design principles, and construction trends. Starting with the Functionalism in the early 1960s, the chapter follows the transition through Brutalism and Monumentalism, the influence of the energy crisis in the 1970s, and ends with the emergence of high-tech Modernism and the office districts of the late 1980s and early 1990s. Specific attention is paid to typical facade types and structural systems of each period, which are relevant to the suitability assessment of VGS.

1955-1965
As explained in section 1.4, office construction in the Netherlands experienced a significant boost during the 1960s. This period of optimism led to the development of large office buildings constructed from glass, steel, and concrete, often equipped with air-conditioning and artificial lighting [142]. The modernist business architecture of Functionalism is characterized by box-shaped office buildings with largely identical facades on all sides. These facades are highly transparent and frequently feature visible prefabricated concrete elements, arranged as strip windows. An example of the Functionalism architecture of the 1960s is given in figure 4.7. A key feature of this era is that facades were often no longer load-bearing, leading to the increasing use of curtain wall systems in combination with slender concrete structures [142, 33]. This aligns with de Gunst and de Jong [33], who note in their book on office building design that most post-war office buildings featured steel or concrete frame construction. Due to steel's poor fire resistance, it needed to be covered in stone-like materials. Therefore, most countries opted to use reinforced concrete for office buildings. In the Netherlands, steel became an attractive structural option for buildings with ten or more floors, due to its significantly lower weight compared to concrete [33].

1965-1975
Following the Functionalist period, Brutalism and Monumentalism emerged between 1965 and 1975, as described by Rijksdienst voor het Cultureel Erfgoed [142]. These movements are characterized by large

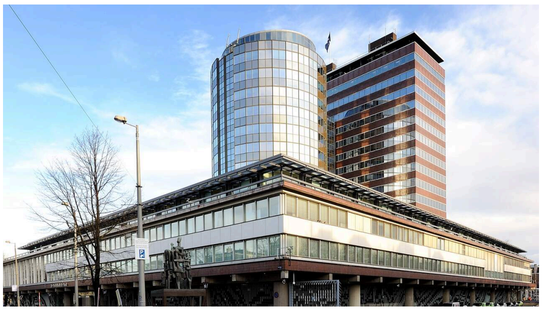


Figure 4.7: Headquarters of De Nederlandsche Bank in Amsterdam, the Netherlands (construction: 1960-1968) [5]

scales, geometric forms, articulated building volumes, experimental construction techniques, rough concrete finishes, and the occasional use of luxurious materials such as natural stone. Office buildings from this period continued to emphasize strong horizontal lines with large glass facades, now combined with articulated volumes of box-shaped masses, intersecting at right angles and stacked on columns. These interconnected forms also reflected the evolving organizational structures within these buildings, which became more open and communicative. As a result, closed and hierarchical cellular offices were increasingly replaced by freely configurable open-plan offices. The interiors of these interconnected forms resembled small, democratic cities. Open-plan offices are well suited to a beam-column structure, as load-bearing walls would limit layout flexibility [34]. In addition to the stacking of geometric volumes, another hallmark of Monumentalism was the sculptural shaping of buildings, creating bold architectural gestures [142]. An example of a typical articulated building volume is shown in figure 4.8, while an example of a sculptural building is presented in figure 4.9.

Figure 4.8: Former office of Centraal Beheer in Apeldoorn, the Netherlands (construction: 1967-1972) [59]

Figure 4.9: Former office of SC Johnson in Mijdrecht, the Netherlands (construction: 1962-1964) [152]

1975-1985

The energy crisis of 1973 and the subsequent recession made insulation and indoor climate control major themes in office construction. Concrete thermal bridges and glass curtain walls from the 1960s no longer met performance requirements. The high costs of heating, climate control, and lighting also marked the end of the full open-plan office. The desire to save energy led to numerous innovations, such as double glazing, climate-controlled windows, mechanical ventilation, glass wool insulation, sandwich panels and climate facades. These developments had a significant impact on architectural design. As a result, sun-reflective glass boxes were increasingly built. The Rabobank office in Utrecht, shown in figure 4.10, was the first example. Internally, combinations of open-plan and cellular offices emerged. As a further consequence of the economic recession, the building sector and office spaces underwent downsizing, and rental offices became increasingly popular. This shift led to the standardization of office design to ensure that buildings could accommodate a wide variety of tenants. These developments align with the architectural movement of Structuralism, characterized by geometric structures, the composition of smaller units and humane architecture. Humane architecture reflects the principle that standardization can enable individual freedom. The former headquarters of the Nederlandse Middenstands Bank (NMB) in Amsterdam, shown in figure 4.11, is an example of Structuralist architecture.

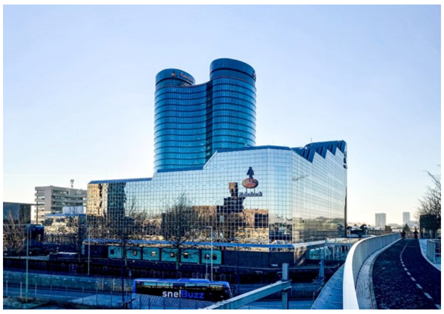


Figure 4.10: Headquarters of Rabobank in Utrecht, the Netherlands (construction: 1980-1983) [58]

Figure 4.11: Former headquarters of the NMB in Amsterdam, the Netherlands (construction: 1979-1987) [4]

1985-1995
The economy began to flourish again in the second half of the 1980s, and as a result, the construction of large high-tech office buildings gained popularity within the architectural movement of Modernism, which is closely related to Functionalism [136]. The building envelope was predominantly defined by glass facades, climate control systems, sun shading, and energy-reducing constructions [142]. High-rise buildings also became increasingly common during this period. Layout flexibility, combined with a column-beam structural system, remained a widely adopted design principle [33]. However, realizing the typical large spans became more challenging in high-rise buildings compared to low-rise structures, as column loads can become substantial. Nonetheless, achieving the desired level of flexibility generally requires spans of at least 7 meters, and preferably between 8 and 9 meters [33]. Both the use of glass facades and high-rise typologies are exemplified in figure 4.12, which shows the former headquarters of Nationale Nederlanden in Rotterdam. In the office rental market, the lifespan of the structural shell was decoupled from that of the interior fit-out, allowing for greater flexibility in responding to changing market demands. This development encouraged the use of neutral office boxes with adaptable interiors. Furthermore, growing office development demands could no longer be accommodated within existing city structures, making the outskirts more attractive due to lower land costs. This gave rise to the phenomenon of the office district: a monofunctional work zone dedicated to office employment, typically located on the urban periphery or along highways [142]. Figure 4.13 shows a typical office building located in such a district. In these office districts, low-rise buildings were prevalent and glass facades were often combined with masonry or natural stone. Figure 4.13 is showing a typical office building from an office district.

Figure 4.12: Former headquarters of Nationale Nederlanden in Rotterdam, the Netherlands (construction: 1986-1991) [163]

Figure 4.13: Former office building 'Dutchport' in Zoetermeer, the Netherlands (completion in 1991) [76]

4.3. Office facade types

Facades come in many different configurations and materials. They are often tailored in collaboration with a facade manufacturer and contractor to meet the design intentions of the architect and structural engineer. While bespoke solutions exist, this study examines a number of standard configurations used in facade construction between 1960 and 1990, subsequently defined as facade types. As explained in section 1.4, the analysis focuses on facades that are not part of the main load-bearing structure. This includes facades supported by floor slabs or facade beams. These facade types are discussed based on building examples and their visible external characteristics or material. The information in this section is based on conversations with structural engineers, supported by literature and online sources. The aim is to provide an overview of the various facade types and their estimated weights used in the period between 1960 and 1990.

4.3.1. Facade configurations

Masonry, and in particular brick masonry, has traditionally been used as a facade material in the Netherlands for both load-bearing and non-load-bearing constructions [33]. The load-bearing brick facade configuration typically consisted of an inner leaf and outer leaf of brick masonry, separated by an air cavity. However, as window sizes increased, the use of masonry in facades became technically challenging. Moreover, the larger scale of many office buildings does not permit the use of brick masonry in a load-bearing capacity [33]. As building dimensions increase, the weight and required structural provisions of load-bearing brickwork result in disproportionately higher construction time and cost. As a result, masonry is typically constructed in segments that rest on the structural frame at each floor or every second floor [33]. These are referred to as self-supporting brick masonry facades. Self-supporting means that the facades can carry and transfer their own weight, as well as that of potential windows in their facade, to load-bearing structural elements, such as a floor, a facade beam or directly to columns or walls.

Koster [77] found that, of 100 Dutch high-rise buildings constructed between 1960 and 1990 with a height above 30 metres, approximately 40% use concrete elements as facade cladding and around 30% feature curtain wall facades. Both facade types allowed for larger window openings and are well suited for high-rise construction, which explains their popularity. A classic curtain wall is defined as a non-structural, continuous facade system that functions as the external envelope of a building [140]. It is mounted on the exterior of the load-bearing structure and is self-supporting. The framework typically consists of aluminum, timber, or steel (the latter used for larger spans), combined with glass cladding. Additionally, plastics, aluminum, and certain types of natural stone can also be used as cladding materials in infill panels [140, 33]. These curtain facades may also incorporate masonry or concrete behind the opaque sections for thermal insulation purposes, which became increasingly important over time, or to provide additional structural support to the curtain walls. The increasing importance of thermal insulation was a direct result of the 1973 energy crisis, as mentioned in section 4.2. In line with this development, Koster [77] notes that the glass percentage in facades decreased from approximately 70% in 1960 to around 40% throughout the 1970s and beyond.

Concrete in facades was used in various ways: as part of the primary load-bearing structure, as a self-supporting system, or only as cladding material [77, 140]. Self-supporting concrete facades generally fall into two categories. In the first, the facade transfers its entire self-weight directly to the foundation, while being horizontally supported by floors or other elements of the main-load bearing structure. In the second, the facade supports only its own weight for one or multiple storeys, transferring the load to intermediate floors, facade beams, or directly to columns. Both concrete facades that form part of the primary load-bearing structure and self-supporting concrete elements can be finished with various cladding materials, such as natural stone or aluminum panels.

Based on the most commonly applied facades during the research period, four main facade configurations can be identified. These configurations differ primarily in how the facade is structurally supported and in the materials used for their construction. The four distinctive facade configurations are listed below.

Classic curtain wall

The main features of the classic curtain wall are its self-supporting nature, a framework made of aluminum, timber or steel, and the use of glass or other lightweight infill panels. Furthermore, from the exterior of the building, typically only the curtain facade is visible. The classic curtain wall provides the sole separation between the interior and exterior climate. Examples include the facades of the WTC tower in Rotterdam, shown in figure 4.14, and the original facades of the WTC building in Amsterdam, shown in figure 4.15.

Figure 4.14: WTC tower in Rotterdam, the Netherlands (construction: 1984-1986) [52]

Figure 4.15: Original facade of the WTC building in Amsterdam, the Netherlands (completion: 1985) [173]

Hybrid curtain wall

The primary distinction between the classic and hybrid curtain wall lies in their structural support. Unlike the classic curtain wall, the hybrid variant is supported from behind by an inner leaf, typically constructed from concrete or masonry, which forms part of the load-bearing structure [77]. Consequently, the supporting system of the hybrid curtain wall is the same as that of the facade type 'structural wall', as will be explained in the following section. Despite this difference in structural systems, hybrid curtain walls often appear indistinguishable from classic curtain walls when viewed from the exterior. Moreover, they are frequently classified as a subtype of curtain wall in the literature. For this reason, the hybrid curtain wall is treated as a distinct facade type in this study. Two examples are provided. Behind the curtain wall of the Delftse Poort building, shown in figure 4.12, lie concrete walls measuring 300 to 400 mm in thickness. Similarly, the walls of the Rabobank headquarters, shown in figure 4.10, consist of concrete sandwich panels with mirrored cladding and relatively small actual windows.

Structural wall

Structural wall refers to a facade type in which the facade is directly attached to a load-bearing wall. While these facades are not the primary focus of this study, they are briefly addressed for completeness. Cavity walls are commonly part of the main load-bearing structure. Initially, both the inner and outer leaves were constructed from brick masonry, as previously mentioned. Over time, concrete was increasingly used for the load-bearing inner leaf, while the outer leaf often remained brick masonry. An example of such a facade is shown in figure 4.11. An alternative to brick masonry is to construct the outer leaf from concrete as well. Prefabricated concrete sandwich panels became increasingly popular from the 1960s and 1970s, with widespread adoption in the 1980s [77]. These panels consist of two concrete layers, an inner and an outer layer, with a thermal insulation core placed in between [140]. During this period, the insulation layer was typically around 80 millimeters thick and commonly made of polystyrene or polyurethane foam [77]. This integrated construction method provides both structural support and enhanced thermal performance in a single prefabricated element. A key feature of structural walls is that the facade is directly supported. Its weight is transferred to the load-bearing wall it is attached to, which eliminates the need for the facade to be self-supporting. As a result, a wide range of facade cladding materials can be applied. Structural walls are typically used when predominantly opaque surfaces are acceptable. When large glazed areas are desired, a beam and column structural system is generally more appropriate as structural system and an alternative facade type may be more suitable.

Self-supporting facades

Self-supporting facades share one key characteristic with classic curtain walls: both are self-supporting and are not part of the primary load-bearing structure. However, they differ significantly in that self-supporting facades can incorporate a wider range of external materials and configurations. Instead of a full facade composed of a consistent framework of mullions and transoms with infill panels, certain sections or even the entire facade may be constructed using different materials and structural configurations. For example, concrete spandrels with optional interior insulation were often combined with curtain walls for the transparent sections of the facade [77]. These spandrels support the curtain wall elements and transfer the facade load typically directly to the primary load-bearing structure due to their stiffness (sections E.1 & E.2). A noteworthy feature of concrete spandrels, and an exception among self-supporting facades, is that they can also function as facade beams by transferring floor loads to the main structural frame. The exact load-bearing configuration of such facades is often difficult to determine based solely on external appearance. For the purpose of this study, concrete spandrels are regarded as facade beams of particular shape. These spandrels may also be clad with various materials such as aluminum panels, natural stone or it can even support masonry. An example of exposed concrete spandrels without additional cladding is shown in figure 4.16.

In addition to concrete, facade spandrels can also be constructed using self-supporting brick masonry, as illustrated by the facade of the building in figure 4.17. These brick spandrels support the weight of the curtain glass facade between them and transfer the facade loads floor by floor to the intermediate floors or facade beams. The entire facade can also be constructed from self-supporting concrete or masonry, incorporating openings for windows. As previously discussed, such facades may be self-supporting from ground level, but masonry is typically vertically and horizontally supported at intermediate floors and lightweight timber or steel frames. The key distinguishing feature between these facade types and structural walls is that they carry only their own self-weight and are therefore not part of the primary structural system. Instead, they are horizontally supported by the building's main structure [140]. It is assumed, for the purpose of this study that concrete as facade element is either self-supporting from ground-level or acting as facade beam in the particular spandrel shape. As previously noted, these concrete elements may also be clad with various facade materials. No distinction is made between the self-weight of glazed sections and curtain wall systems.

In addition to concrete or brick masonry, facade panels made of aluminum, natural stone or other cladding materials can also be supported by a lightweight frame of aluminum or timber (sections E.1 & E.2). These frames can be filled with insulation material, resulting in a very lightweight facade system. Openings can be incorporated for windows, and the self-weight is typically transferred to intermediate floors or facade beams.

4.3.2. Conclusion on facade types

As discussed in the introduction of this section, the analysis focuses on facades supported by floor slabs and facade beams. Based on the preceding sections, the main facade types used during the studied period are self-supporting masonry facades, classic curtain walls and lightweight frames with facade panels. Most concrete facades are sufficiently stiff and strong to transfer their own weight, often directly to the ground or to columns or load-bearing walls. They typically act as structural walls or as facade beams themselves, rather than relying on floor slabs or separate facade beams for support. For this reason, concrete facades are excluded from this study. The weights and assumed build-ups of the facade types considered in the analysis are listed in table 4.1. Since lightweight facades are considered governing in the excess capacity analysis, insulation is excluded from the weight calculation and lightweight variations are used in the calculations of weights.

Facade type	Build-up	Weight [kN/m²]
Classic curtain wall	Lightweight framework with infill panels	1.0
Self-supporting masonry	Brick masonry with lightweight steel or timber inner frame	1.8 + 0.5 = 2.3
Lightweight frame with facade panels	Steel or timber lightweight frame with natural stone cladding	0.5 + 0.8 = 1.3

Table 4.1: Overview of facade types, considered build-up and weight estimation [140, 110, 61] and (sections E.1 & E.2)

4.4. Analysis of office building types

The overlapping building characteristics identified through the analysis of office architecture and the formulation of building indicators have led to the definition of four office building types: Functional boxes, Glass volumes, Interlocking shapes and Sculptural designs. This typology offers a framework for classifying existing office buildings, allowing for tailored recommendations on suitable VGS options based on typical building characteristics. The four office building types and their common facade configurations are discussed in the following sections.

4.4.1. Functional boxes

The term ‘Functional boxes’ denotes box-shaped office buildings that do not feature fully glazed facades, creating many possibilities for implementation of various VGS. The prevailing message of office architecture from 1960 to 1990 was that form should follow function and allow for flexibility. This approach led, in the 1960s, to the design of large box-shaped buildings. By the 1980s, the box-shaped typology remained popular, with its application extending to smaller-scale buildings in office districts on the urban periphery. The facades of these smaller office boxes often features brick masonry, part of a cavity wall in the main-load bearing system. However, the larger scale of many office buildings does not permit the use of brick masonry in a load-bearing capacity [33]. As building dimensions increase, the weight and required structural provisions of load-bearing brickwork result in disproportionately higher construction time and cost. As a result, masonry is typically used in the form of facade panels that are supported floor by floor by the building’s primary structural frame [33]. These are referred to as self-supporting brick masonry facades, examples of this facade type are shown in figure 4.11 and figure 4.17. Another typical facade configuration for these functional box shaped buildings, both high-rise and low-rise, involves prefabricated concrete parapets combined with strip windows. The building in figure 4.16 is a typical functional box building with concrete parapets in the facade. Additionally, strip windows are often combined with natural stone or other lightweight cladding panels, as seen in the facades of the high-rise and low-rise part of the building in figure 4.7. The layers behind the visible facade cladding are difficult to determine based on external characteristics, but they may well consist of concrete elements as well (section E.2). This would make it possible to remove the cladding and attach closed VGS systems directly to the facade structure. As described in section 4.1.5, masonry and concrete are suitable materials for mounting substructures of VGS using various fastening techniques. Only direct green facades are not recommended in combination with natural stone or masonry facades. The building height indicator does not specifically correspond to this facade typology as a whole, since these box shaped buildings may be either high-rise or low-rise. However, their geometric form of planar facades allows all VGS to remain suitable.

Figure 4.16: Former office in Alphen aan den Rijn, the Netherlands (completion in 1984), captured via Google Street View [73]

Figure 4.17: ‘Havengebouw’ in Amsterdam, the Netherlands (construction: 1958-1960) [166]

4.4.2. Glass volumes

Buildings with fully glazed or mirrored facades fall under the typology of ‘Glass volumes’. These buildings are generally designed as a single large volume rather than as stacked sections, as stacking would reduce the architectural impact of the glass or mirrored surfaces. This typology is based on the facade glass percentage indicator, as explained in section 4.1.4. Closed VGS systems, such as box, container and felt systems, are not suitable for this typology. Among the open systems, direct systems are also not recommended due to limited accessibility for maintenance and cleaning. In addition, the brackets for climbing aids in curtain systems typically cannot be mounted onto glass facades. The monolithic design of these buildings, often conceived as one continuous volume, also makes them slightly incompatible with curtain facades. Indirect or curtain-based systems may still be used, but they require adaptations to the building design and substructure of the VGS. This could involve either creating an independent substructure with its own foundation for attaching the climbing aids, or opening up the facade to allow an auxiliary substructure to connect to the primary load-bearing system. The latter option is too complex for the scope of this study. Therefore, only systems with an independent substructure such as curtain or indirect green facades are considered suitable for glass volumes.

4.4.3. Interlocking shapes

The building typology 'Interlocking shapes' refers to structures composed of articulated volumes. These buildings consist of distinct geometric or architectural forms that are connected or stacked together, much like pieces of a puzzle. Their facade materials range from brick masonry and natural stone to aluminium and tiles, typically in combination with windows. Examples of interlocking shapes with brick masonry facades are shown in figure 4.8 and figure 4.11. The building in figure 4.18 features tiled facades, which are often attached to precast concrete panels such as concrete parapets. The main building indicator relevant to this typology is the building shape with stacked volumes as featured building characteristic. As a result, ground-rooted vertical greening systems such as green facades are generally not suitable for offices with interlocking shapes. All living wall systems are in principle suitable for this typology due to their modular nature, which allows them to adapt to stacked building volumes. However, their suitability can vary depending on the glass percentage and the geometry of the facade. Naturally, a high glass percentage makes closed systems such as felt, box, or container types less appropriate. Another important building indicator is facade geometry, since interlocking shapes may include curved facades, which are less compatible with box or container systems. The headquarters of De Nederlandsche Bank, shown in figure 4.11, is a clear example of an interlocking shape featuring both curved and planar facades.

Figure 4.18: Headquarters of Randstad in Amsterdam, the Netherlands (construction: 1987-1990) [24]

4.4.4. Sculptural design

Section 4.2 introduced the sculptural architecture that emerged between 1965 and 1975, exemplified by the expressive design of architect Hugh Maaskant shown in figure 4.9. These buildings were intended to create bold architectural gestures. For this typology, curtain facades combined with a column-beam load-bearing structure proved particularly useful. These lightweight facade systems could accommodate nearly any shape envisioned by the architect. The same construction method was utilized in the SC Johnson office depicted in figure 4.9. However, the facade in this instance did not follow the building's curved form, resulting in a roof that appears to float above the structure. Due to the highly variable heights, geometries, forms, and facade types of sculptural buildings, no single building indicator with a consistent set of characteristics can be directly assigned to this typology.

4.4.5. Office building typology

Figure 4.19 provides a summary of the suitability of various VGS in relation to Dutch office building typologies from 1960 to 1990. Each typology is associated with specific building indicators, derived from an analysis of VGS suitability. For each building type, these indicators correspond to distinct building characteristics. Based on these characteristics, the figure provides recommendations on suitable or unsuitable VGS types. This overview highlights the impact of office architecture on VGS implementation.

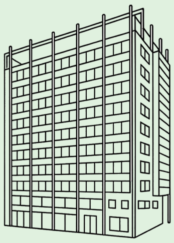
	Building indicator	Building characteristic	VGS suitability
Functional boxes 	**Box-shaped buildings of varying heights and facade materials**		
	1. Facade geometry	Planar facade	All VGS are suitable.
	2. Building height	Low-rise	All VGS are suitable.
		High-rise	Green facades are considered unsuitable, due to the maximum attainable growth height of 25 m.
			Box and curtain systems are recommended, due to their standard integrated safety features that prevent vegetation from falling.
	3. Facade type	Concrete or masonry	Concrete or masonry is preferred for attaching all living wall systems and indirect green facades.
		Marble or masonry	Direct green facades are not recommended as they potentially damage the existing facade.
		High window percentage	Direct green facades are considered unsuitable, as they obstruct window cleaning.
		Continuous strip windows	Direct green facades are considered unsuitable, due to their vertical growing direction.
Interlocking shapes 	**Stacked volumes with varying facade geometries, heights and materials**		
	1. Building shape	Stacked volumes	*In case* of regularly shaded areas on the facade, modular and continuous living wall systems are recommended due to their high plant variety and density.
	2. Facade geometry	Planar facade	All VGS are suitable
		Curved tall facade section	Felt systems are recommended, as they can easily follow facade curvature and have no height limit.
		Curved short facade section	Felt and green facades are recommended, as they can easily follow facade curvature.
	3. Facade type	Concrete or masonry	Concrete or masonry is preferred for attaching all living wall systems and indirect green facades.
		Marble or masonry	Direct green facades are not recommended as they potentially damage the existing facade.
		High window percentage	Direct green facades are considered unsuitable, as they obstruct window cleaning and facade maintenance.
		Continuous strip windows	Direct green facades are considered unsuitable, due to their vertical growing direction.
Glass volumes 	**Varying building shapes with fully glazed or mirrored facades**		
	Facade type	Fully glazed or mirrored facade	Closed systems such as felt, container or box systems are considered unsuitable.
			Direct green facades are considered unsuitable, as they obstruct window cleaning and facade maintenance.
			Fastening of indirect green facades or curtain systems is difficult. Therefore, an independent substructure with a dedicated foundation is recommended.

Figure 4.19: Overview of the office building typology and corresponding impact on VGS suitability

5 Structural feasibility

In order to determine whether adding a VGS to an existing building is feasible, an understanding of its structural feasibility is also required. This chapter begins with an overview of current structural guidelines for existing concrete buildings and explains the differences between evaluating an existing structure and a new one. To further deepen the understanding of how excess capacity originates and to look for an applicable excess capacity method in the feasibility phase, these insights from structural guidelines are complemented by studies on the structural assessment of existing concrete buildings.

5.1. Structural evaluation of existing buildings

This section will outline what current building guidelines state about the structural evaluation of existing buildings. It will also analyze the differences between the structural evaluation of a new building and that of an existing building.

5.1.1. Standards for existing structures

Since April 1, 2012, the Eurocodes with their corresponding national annexes have been mandatory for assessing structural safety under the Building Decree [78]. The Eurocodes are an European initative that originated in the 70s. In 2005, the first set of Eurocodes was released, which replaced the previously used national building guidelines. The Netherlands had to wait with the implementation of the Eurocodes until the release of the new Building Decree. For the assessment of the structural safety of existing buildings, the NEN published the standard document NEN 8700 [78]. This standard is linked to the Eurocodes, as it is based on the NEN-EN 1990 along with the corresponding national annex [115, Voorwoord]. NEN 8700 defines the principles, application rules, and assessment methods for evaluating whether an existing structure meets an adequate level of safety and usability. However, while it outlines these principles, it does not provide detailed assessment methods for determining the current or future safety level of an existing building. The NEN 8700 is an essential addition to the standards for new construction, as the assessment of an existing structure differs significantly from that of new construction in several key aspects [115, 153]:

- The determination of the desired safety level involves considerations of both personal safety and cost-effectiveness. Cost-effectiveness is evaluated differently for existing construction. Increasing the safety level or achieving the standards of new construction generally incurs relatively higher costs in a renovation scenario compared to the design phase of a new building.
- The remaining service life of existing structures is often shorter than the standard reference period of at least 50 or 100 years used in the design of new buildings. This factor also influences the cost-effectiveness assessment of a renovation.
- Measurements can provide additional insights into a building's structural condition.

NEN 8700 applies in cases of building modification or expansion, changes in occupancy or loading conditions, or when a structure has reached the end of its design life [115, Voorwoord]. The standard describes the required safety levels for renovation and condemnation, while defining the fundamental principles for assessing the structural safety of existing buildings, regardless of their function or age. It is important to note that a structure is considered existing upon formal completion of construction. Distinguishing between renovation and condemnation is essential, as each approach corresponds to different safety levels. Renovation is defined as a tangible building alteration, which excludes load changes not accompanied by physical modifications to the building. However, changes in load may necessitate reassessment of the existing structure. In case of a change in function, the recommendation is to apply at least the renovation level to the entire structure [153]. In other words, other changes than physical modifications that belong to the specific term renovation have to adhere to the renovation level. Condemnation level is defined as the legally required minimum level of structural safety, below which a (preliminary) enforcement order and regulatory action by the competent authority must follow [115, Voorwoord]. In practice the following systematic is recommended for the assessment of the structural safety of existing buildings [153]:

1. The new construction level is always the target level.
2. If the new construction level is not met and the costs to comply with it are disproportionate, the renovation level is sufficient.
3. Disproportionate costs to meet the renovation level could, in exceptional cases, justify not implementing measures and considering the condemnation level as sufficient.
4. If the condemnation level is not met immediate measures are required, regardless of the legally acquired level.
5. In the case of renovations, the new construction level remains the target level. However, for economic reasons, reduction to the minimum legal renovation level may be justifiable in certain cases.

The implementation of green roofs or green facades only adds loads to the existing structure, the existing load-bearing system does not change by this addition. Therefore, at least the renovation level will be applied to the entire structure, but the new construction level stays the target level. Consequently, the structure should be assessed against both the renovation and the new construction levels. For renovations, new construction requirements apply unless the building is at least 15 years old, as noted in informative Annex F of NEN 8700 [115, App. F.0]. In that case, the renovation level may be applied if justified by the reasons explained above. NEN 8700 does specify certain exceptions, but these require specific structural and legal justification. In addition to NEN 8700, NEN 8701 and NEN 8702 have been published. NEN 8701 specifies the loads that must be applied in all forms of renovation and in the assessment of whether an existing building structure may need to be condemned. NEN 8702 provides specific information on the evaluation of existing concrete structures for renovation or condemnation.

5.1.2. Differences between NEN-EN 1990 and NEN 8700

The scope of NEN 8700 has been described in section 5.1.1. The NEN 8700 states that the standard is based on NEN-EN 1990 and must be read in conjunction with it [115, par. 1.1]. The standard only includes texts that deviate from the NEN-EN 1990 where necessary for the assessment of the structural safety of existing buildings. NEN-EN 1990 defines the principles and requirements for the safety, serviceability, and durability of structures. It is based on the concept of limit states, used in conjunction with the partial factor method [108, Voorwoord].

Residual service life and reference period

A new term that's introduced in the NEN 8700 is the term “residual service life” (restlevensduur) [115, par. 2.3]. The term can be compared with the term “Design service life” (ontwerplevensduur) used in NEN-EN 1990, which is set to 50 years for office buildings [108, par. 2.3]. Residual service life is defined as the assumed period during which an existing or renovated structure, or part of it, remains usable for its intended purpose. The minimum safety level must not be compromised during this time. The term “residual service life” has officially a different definition than “reference period” (referentieperiode), although they often have the same value. The reference period is the timeframe considered when determining the magnitude of variable loads and it does not necessarily have to match the residual service life, but it must be at least equal to the residual service life. The differences are primarily related to human safety requirements. The reference period used to determine the standard characteristic variable loads set in NEN-EN 1991-1-1 is also 50 years. In previous building standards, such as the TGB-series, the design service life was referred to as the reference period. The NEN 8700 provides a formula to calculate new uniformly distributed variable loads if a different reference period is preferred [115, par. 2.3.2]. When a structure is set for renovation, the residual service life should at least correspond to the remaining part of the original design service life. The minimum value of the residual service life is 15 years. No calculations directly depend on the residual service life, the reference period only depends on it.

Principles of limit state design

For structures that are part of a renovation, a distinction must be made between ultimate limit states and serviceability limit states [115, par. 3.1]. When assessing whether a structure should be condemned, the evaluation should be based solely on ultimate limit states. NEN 8700 states that even in the case of renovations, the legislator only imposes requirements on the ultimate limit states. Limit states must be related to design, calculation or verification situations in accordance with the conditions under which the structure must fulfill its function [115, par. 3.2]. The same limit states are classified as ultimate limit states and serviceability limit states in both NEN-EN 1990 [108, par. 3.3] and in NEN 8700 [115, par. 3.3]. The design and calculation based on limit states must be founded on the use of structural models and load schemes applicable to the relevant limit states. The structural analysis must be done in the same way for existing buildings as for new ones, hence the method described in NEN-EN 1990 must be adhered [108, chap. 5]. This also applies to the partial factor method, which must be used for verification and is outlined in chapter 6 of NEN-EN 1990 [108, chap. 6].

Basic variables

In order to assess a structure, values are required for geometry, material properties, loads and the current condition (such as cracks, deflections and discolorations) [115, chap. 4]. For existing structures, this information can be found in the original building dossier, the standards in effect at the time and results from additional measurements and inspections. Documentation from the building dossier should be interpreted with caution due to potential deviations in practice.

The characteristic value of a load is the most important representative value, as stated in NEN 8700 [115, par. 4.1.2]. Consequently, properties of materials or products must be represented by characteristic values according to NEN 8700 [115, par. 4.2]. For material and product properties not covered by the new building regulations, NEN 8700 allows these values to be derived from the original building regulations or the original building dossier. The standard also highlights that the originally applied safety margins were often incorporated in these representative values and must be maintained. For example, this applies when allowable stresses were used instead of characteristic and design values. Another way to determine the material properties is by conducting measurements on the existing structure. When insufficient statistical data is available to determine the characteristic values of a material or product property, nominal values may be used as characteristic values, or the design values of the property may be determined directly.

Ultimate limit state

NEN 8700 states that the same formulas (6.9a to 6.12b) from NEN-EN 1990 must be used for the determination of loads for the verification of the ultimate limit states [115, par. A1.2.1]. Additionally, the psi factors from NEN-EN 1990 table A1.1, presented here in table 5.1, must be used [115, par. A1.2.2]. The design values of the loads for the ultimate limit states, STR and GEO, in fundamental combinations must be determined in the same manner as in NEN-EN 1990 (formulas 6.10a and 6.10b), except that different partial load factors are specified in table A1.2(B) and (C) of NEN 8700. These partial load factors depend on the consequence class, which are described in Appendix B of NEN-EN 1990. Office buildings are classified in consequence class 2. Formulas 6.10a and 6.10b, as defined in NEN 1990 [108, par. 6.4.3.2], are given in equation (5.1) and equation (5.2). The corresponding partial factors, with a variable load other than wind being the dominant one, are listed in table 5.2. It is assumed that the implementation of VGS on a building will not alter or affect the existing main stability system in a relevant way, hence the stability verification (EQU limit state) is left out of scope. Consequently, wind loads are excluded from this study as well, as they are assumed not to be dominant in STR- and GEO-limit states. Naturally, a later design stage should account for the influence of wind loads and include a stability analysis.

Category	ψ_0	ψ_1	ψ_2
B: Office areas	0.5	0.5	0.3
H: Roofs	0	0	0

Table 5.1: Psi factors for offices from NEN-EN 1990 NB [109, Tabel NB.2 - A1.1]

$$\text{6.10a:} \sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P^* + \gamma_{Q,1}\psi_{0,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\psi_{0,i} Q_{k,i} \tag{5.1}$$

$$\text{6.10b:} \sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P^* + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\psi_{0,i} Q_{k,i} \tag{5.2}$$

Formula	Permanent	Variable
6.10a	$\gamma_{G,j} = 1.3$	$\gamma_{Q,1} = 1.3$
6.10b	$\xi_j \cdot \gamma_{G,j} = 1.15$	$\gamma_{Q,1} = 1.3$

Table 5.2: Partial factors for renovation of CC2 from NEN 8700 with a variable load other than wind being the dominant one [115, tabel A1.2(B) en (C)]

Serviceability limit state

NEN-EN 1990 states that the deformations to be considered in relation to serviceability requirements depend on the nature of the construction [108, par. 6.5.3]. For buildings in the Netherlands, appendix A of the national appendix of NEN-EN 1990 (NEN-EN 1990 NB) states that vertical and horizontal deflections shall be calculated in accordance with NEN-EN 1992 to NEN-EN 1999 using the formulas for load combinations of the serviceability limit state [109, par. A1.4.2]. The NEN-EN 1990 defines three type of load combinations for the serviceability limit states: the characteristic combination (equation (5.3)), the frequent combination (equation (5.4)) and quasi-permanent combination (equation (5.5)) [108, par 6.5.3]. The National Annex of NEN-EN 1990 specifies in which design situation each combination should be used and defines the maximum allowable deflection for that situation [109, par. A1.4.3]. Both the partial factors for material properties, as for the load combinations are taken as 1, unless other standards specify otherwise (NEN-EN 1991 - NEN-EN 1999). As stated earlier in section 5.1.2, NEN 8700 considers the serviceability limit states for public law purposes not applicable [115, chap. 6]. The standard further notes that, from a private law perspective, it may still be useful to apply them [115, par. 3.4].

$$\text{6.14b:} \sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{5.3}$$

$$\text{6.15b:} \sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{5.4}$$

$$\text{6.16b:} \sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \tag{5.5}$$

5.1.3. Differences between NEN-EN 1991-1-1 and NEN 8701

NEN-EN 1991-1-1 provides guidelines for the design, calculation and loading of buildings and civil engineering structures. It covers material densities, self-weight of structures and imposed loads for buildings. NEN 8701 establishes the regulations regarding the applicable loads for renovations and assessment of existing structures. NEN 8701 is intended to be used in conjunction with NEN-EN 1990 to NEN-EN 1999 series.

Loads

The actual value of permanent loads related to self-weight may be determined by calculation, but in certain cases, it may also be established fully or partially through measurement and/or weighing [116, par. 4.2.1]. Volumetric weights of construction materials are provided in appendix A of NEN-EN 1991-1-1. For factory-made elements, such as facades, the properties may be provided by the manufacturer [108, par. 5.2.2].

For the design and calculation of floor structures of a story or roof, the imposed load must be considered as a free load at the most unfavorable part of the floor structure [110, par. 6.2.1]. A separate verification must be done for a concentrated load, which must not be combined with the uniformly distributed loads or other variable loads. For the design and calculation of columns and walls, the variable load on the most unfavorable part of at least one floor must be considered [110, par. 6.2.2]. Office spaces are classified as category B with a uniformly distributed load of 2,5 kN/m and a concentrated load of 3 kN [111, par. 6.3.1.2]. NEN 8701 only states that adjustments in use are possible, referring to the fact that variable loads may be reduced based on actual usage [116, par. 4.3.1]. The self-weight of lightweight partition walls have to be added to the uniformly distributed weight of variable loads [110, par. 6.3.1.2].

5.1.4. Differences between NEN-EN 1992-1-1 and NEN 8702

NEN-EN 1992-1-1 defines the principles and requirements for the safety, serviceability and durability of concrete structures, with specific provisions for buildings. In combination with NEN 8700, NEN 8702 provides the framework for the minimum safety level to be maintained for the structural integrity of a concrete structure in the case of renovation or condemnation [117, Voorwoord]. The standard covers unreinforced, reinforced and prestressed concrete. For a renovation, this standard provides guidelines on the use of properties of original materials, the impact of previously prescribed cover requirements and similar factors. NEN 8702 refers to NEN 8700 for the design and calculation based on limit states and to NEN 8701 for the determination of loads [117, par. 2.1.1]. The standard also states that the assessment of structural safety must take existing structural damage into account [117, par 2.2]. In the case of concrete, this specifically includes cracking, reinforcement corrosion and spalling. In general, this damage inspection consists at minimum of a visual inspection, an investigation into the cause and the determination of any necessary repair measures.

Geometrical data

NEN 8702 and NEN 8700 specify that original geometrical data can be assumed to be true [117, par. 2.3.3]. Diameter, amount and position of reinforcement are also part of geometrical data. Only when there is reason to doubt the accuracy of the original design data or if deviations from the original design have a significant impact, an investigation of the structure must be conducted to reduce uncertainty regarding the geometric data. If no data is available on the applied reinforcement configuration (location and diameter), it must always be determined through an investigation of the structure. Assuming the applied reinforcement based on the regulations in effect during the original design, without verifying its accuracy through structural investigation, is therefore insufficient.

Partial factor method

The method of partial factors as explained in NEN 8700 and NEN-EN 1990 (section 5.1.2) should be applied for concrete structures as well [117, par. 2.4]. The partial factors for materials, in this case for concrete and steel reinforcement, may not be taken lower than those specified in NEN-EN 1992-1-1. Maintaining constant partial factors for materials has been one of the key principles in determining the partial factors for loads in NEN 8700. The partial factors for concrete and steel reinforcement, γ_c and γ_s, for the ULS and SLS design are given in table 5.3.

	γ_c	γ_s
Ultimate limit state (ULS)	1.5	1.15
Serviceability limit state (SLS)	1.0	1.0

Table 5.3: Partial factors for concrete and steel

Material properties

NEN-EN 1992-1-1 explains that the compressive strength of concrete is indicated by concrete strength classes, which are related to the characteristic (5%) cylinder strength f_{ck} or the cube strength $f_{ck,cube}$ in accordance with EN 206-1 [117, par. 3.1.2]. NEN-EN 206 defines the use of raw materials in concrete, establishes requirements for durability and compliance and outlines the responsibilities of all parties involved in the specification, production and application of concrete. The procedure for determining f_{ck} from f_{cm} is also specified in NEN-EN 1992-1-1 [113, par. 3.1.2]. However, formula 3.2 from NEN-EN 1992-1-1, which calculates the compressive strength of concrete at age t is specified to be non applicable by NEN 8702 [117, par. 3.1.2]. Existing buildings are often several years old, whereas this formula uses days as input, which could result in an overestimation of the compressive and tensile strength of concrete. The characteristic strength f_{ck} and the corresponding mechanical properties required for design and calculation are given in Table 3.1 from NEN-EN 1992-1-1 [113, par. 3.1.2].

NEN 8702 specifies how the characteristic values of concrete and steel reinforcement used in existing buildings must be determined. The characteristic values of the material properties should be at least be derived from the original building dossier, determined by means of samples taken from the structure using destructive testing methods, assessed through non-destructive test methods or established by a combination of these three methods [117, par. 3.1]. Nevertheless, if the structure exhibits visible signs indicating a deterioration of material strength, the existing material strength must be determined based on representative samples taken from the structure. The required research methods to determine the material properties of existing concrete and steel reinforcement depends on the available documents, inspection results, required material strength, failure mechanism or failure behavior and the influence of material properties on the calculated structural resistance [117, par. 3.1]. Additionally, the investigation must account for the potential impact of material degradation on material behavior. NEN 8702 also specifies which tests should be conducted and how these should be performed [117, par. 3.2].

If the material properties are derived from the original building dossier, the original parameters must be interpreted and converted into characteristic values in accordance with NEN-EN 1992-1-1. The original safety margins are often incorporated into values of the allowable stresses associated with the material properties from the standards in effect at the time of the original design. As a result, the characteristic properties can often not be directly obtained from the original building dossier. The characteristic values for the most common original material specifications are provided by table 1 and table 2 from NEN 8702 [117, par. 3.1]. If the original concrete strength class cannot be derived from the original building dossier and no tests are conducted on the structure, the lowest cube compressive strength specified in the regulations applicable at the time of construction must be assumed [117, par. 3.2]. The same applies for steel reinforcement [117, par. 3.3]. The GBV 1940, GBV 1950 and GBV 1962 allows lower steel stress than the design rule given in NEN-EN 1992-1-1 for determining f_{yd}. If this design rule is applied and thus a higher value of f_{yd} is used than specified by the standard in effect at the time of construction, NEN 8702 implies that attention must be paid to the required embedment length and to sustainable safety. Sustainable safety refers to the consideration of how an increased load-bearing capacity affects the long-term durability of the structure and the measures needed to ensure structural reliability throughout its remaining service life. NEN 8702 also states that the verification of the serviceability state was missing from GBV 1940 until GBV 1962 [117, par. 3.3]. However, the allowable stress was limited for high-strength steel grades. This was, among other things, an indirect way to control crack width.

Sustainability and concrete cover

NEN-EN 1992-1-1 states that a sustainable structure must meet the requirements for serviceability, strength and stability throughout its design service life, without significant loss of functionality and without requiring extraordinary unforeseen maintenance [113, par. 4.1]. The protection of reinforcing steel against corrosion depends on the density, quality, thickness of the concrete cover and on crack formation. The density and quality of the cover of new concrete are achieved through an appropriate choice of the maximum water-cement ratio and the minimum cement content, which is further detailed in NEN-EN 206-1. The requirements for concrete cover and crack formation are defined in NEN-EN 1992-1-1. NEN 8702 outlines that in order to verify whether the targeted residual service life is feasible, a visual inspection of the concrete structure may be conducted to assess surface conditions [117, par. 4.2]. In addition to this inspection, samples may be taken to determine the likelihood of reinforcement corrosion or the presence of chemical damage. Previous design standards generally recommend a lower concrete cover than current standards. While this does not automatically imply corrosion, it increases the likelihood. If the applied concrete cover is smaller than the minimum concrete cover specified in NEN-EN 1992-1-1, NEN 8702 defines its implications for adhesion [117, par. 4.3]. For the requirements regarding corrosion protection in existing structures, NEN 8702 provides an assessment method [117, par. 4.3]. The values of the minimum cover requirements from past standards are largely documented by TNO. If no other values can be derived from the original building dossier, these may be used as a basis for the assessment. Naturally, if there is reason to doubt the applied concrete cover, it must be determined through measurements on the structure.

Structural evaluation

NEN-EN 1992-1-1 states that a linear-elastic calculation may be performed for both the serviceability and the ultimate limit states. To determine the section forces, a linear calculation may be conducted based on uncracked cross-sections, linear stress-strain relationships and the mean value of the modulus of elasticity [113, par. 5.1]. NEN 7802 aligns with this statement but further emphasizes that the structural assessment must also consider the effects of settlements, modifications to the structure, structural damage or other changes [117, par. 5.1].

Ultimate limit state

The deviating rules provided in NEN 8702 regarding the ultimate limit state apply to the assessment of the safety level of existing structural components that are not being renovated, as defined above [117, par. 6.1]. In other words, NEN 8702 regarding the ultimate limit state applies when the elements of the existing load-bearing structure remain unchanged. In the section on the ultimate limit state, the only deviating rules concern the method for calculating the shear resistance. The shear resistance can be determined either through verification according to NEN-EN 1992-1-1 or through verification using the modified assessment rules provided by NEN 8702. The structural safety may be assessed using the most favorable of the two methods. In the method described by NEN 8702, the contribution of concrete ($V_{Rd,c}$) may be taken into account for the shear capacity if this was also allowed in the original design and achieved the intended safety level according to the original regulations [117, par. 6.2]. Whereas NEN 1992-1-1 does not include the contribution of concrete to the shear capacity of an element with shear reinforcement [113, par. 6.2.1].

Serviceability limit state

NEN-EN 1992-1-1 includes stress limitation, crack control and deflection verification as serviceability limit states [113, par. 7.1]. When an existing building is assessed according to the condemnation level, the serviceability limit state does not have to be taken into account as is stated in section 5.1.2 [117, par. 7.1]. Legally speaking, proof that sustainable safety is ensured is only required when the residual service life exceeds one year, at which point serviceability limit state requirements may play a role. Unlike the serviceability limit requirements defined in NEN-EN 1992-1-1 for new construction, visual inspections of existing structures can identify defects that affect serviceability, such as excessive deformations, reinforcement corrosion and unacceptable crack widths. In case defects are present, an explanation must be provided and their influence on the sustainable safety must be evaluated. Crack widths can be examined through inspection, but can also be assessed numerically using the formulas provided in NEN 8702 for smooth reinforcing steel and in NEN-EN 1992-1-1 for ribbed reinforcing steel [117, par. 7.2]. NEN-EN 8702 states that for the verification of sustainable structural safety, deflection is generally not a critical factor, but it may be desirable for assessing the suitability of an existing building for its intended use [117, par. 7.3]. When checking deflection, information on actual deformations may be included in the assessment. A deflection check based on NEN-EN 1992 may only be used for structures in which reinforcing steel with a characteristic strength of approximately 500 MPa has been applied.

5.1.5. Key findings

In the structural evaluation of existing concrete buildings, the key take-aways are:

- Assessment of new and existing construction differs significantly [115, Voorwoord].

- Two different safety levels are introduced for the evaluation of existing structures: renovation level and condemnation level [115, Voorwoord].
- Original safety margins must be adhered and may not be as explicitly mentioned as the difference between characteristic and design values we use today [115, par. 4.2].
- The same formulas including psi factors must be used for the determination of loads for the verification of ULS and SLS, only the partial load factors are different [115, app. A1.2.1].
- NEN 8700 states that even in the case of renovations, the legislator only imposes requirements on the ultimate limit states [115, par. 3.1]. However, the standard also notes that, from a private law perspective, it may still be useful to apply them [115, par. 3.4].
- Original geometrical data can be assumed to be true, unless there is reason to doubt the accuracy of the original design data or if deviations from the original design have a significant impact [117, par. 2.3.3]. If no data is available on the applied reinforcement configuration (location and diameter), it must always be determined through an investigation of the structure. Assuming the applied reinforcement based on the regulations in effect during the original design, without verifying its accuracy through structural investigation, is therefore insufficient.
- Partial load factors for materials may not be taken lower than those specified in NEN-EN 1992-1-1 [117, par. 2.4].
- NEN 8702 specifies how the characteristic values of concrete and steel reinforcement used in existing buildings must be determined [117, par. 3.1.2].
- NEN 8702 states that to verify whether the targeted residual service life is feasible, a visual inspection of the concrete structure may be conducted to assess surface conditions [117, par. 4.2]. A visual inspection is also recommended for the structural assessment [117, par. 4.2], as stated in both chapters on the ultimate limit state and the serviceability limit state.
- The deviating rules from NEN 8702 regarding the ultimate limit state evaluation only apply to the elements of the existing structure that remain physically unchanged [117, par. 6.1]. The structural safety of the shear resistance may be assessed by the most favorable outcome of the method from NEN 8702 or the method from NEN-EN 1992-1-1 [117, par. 6.2].

5.2. Safety factor analysis

The previous section explains how existing concrete buildings are structurally evaluated, resulting in specific calculations that can determine the allowable load on the building today if detailed information about its structure is known. However, during the feasibility stage these detailed information is not known. Therefore, the following section explores the potential origin of excess capacity by analyzing the structural design codes used between 1960 and the present day. This analysis results in the calculation of excess capacity factors for facades of concrete buildings constructed during the reference period.

5.2.1. From safety margins to excess capacity

Section 5.1.2 outlines the partial load factors used for the evaluation of existing structures. These partial load factors are lower than the load factors required for new construction [108, A.1.3]. This originates from the fact that these buildings have proven to be safe, so part of the uncertainty around safety can be eliminated [13]. This results in lower safety factors, which potentially creates room for a standard amount of excess capacity in existing construction. Van Uffelen [165] looked at the safety margins between the tested strength of construction materials and the characteristic loads on these elements or the allowable stresses in the materials from the GBV 1912 to Eurocode 2012. The compressive strength of concrete and the tensile strength of reinforcement steel have been investigated. The analysis shows a decrease in safety margins over time for both materials, suggesting that existing reinforced concrete structures may be able to sustain higher loads than originally accounted for [165]. This additional capacity is referred to as potential excess capacity. It should be kept in mind that this safety margin existed for a reason. At the time, knowledge of concrete was significantly more limited than it is today and on-site quality control was often poor [165]. As a result, significant variation in the material properties of existing concrete structures may occur, which is why the NEN 87 series emphasizes the importance of on-site investigations [117].

As part of the method used to calculate the excess capacity of specific structural elements, Beeren [10] calculates two safety ratios. The first ratio is based on the safety margin in the current structural guidelines for new construction, Eurocode 2012, compared to the margin used in the guidelines for existing structures, NEN 87 series. The second ratio compares the safety margin of Eurocode 2012 with that of the structural guideline from 1962, GBV 1962. These two safety ratios can be multiplied to become a factor that can be used to calculate excess capacity. Both van Uffelen [165] and Beeren [10] use the most unfavorable situation or factors if given a choice. When specific element calculations are excluded from the analysis, the ratio identified by Beeren [10] can be interpreted as an excess capacity factor. Different excess capacity factors apply to various material and loading combinations: concrete under compression with permanent loads, concrete under compression with variable loads, reinforcing steel under tension with permanent loads and reinforcing steel under tension with variable loads. If the original loading type, such as a distributed or point load, corresponds to the current load type, the excess capacity can be estimated by multiplying the relevant excess capacity factor, determined by the construction year, with the original load. If the loading type differs, the excess capacity must be derived using structural mechanics formulas. For beams, floors, or facades supported by either, it is assumed that reinforcing steel in tension governs the design and that these elements are subjected to distributed loads only. As the actual loads are unknown, suitable partial factors are adopted, as justified in the following sections. Among concrete and steel, the safety margin for reinforcing steel is the lowest, as shown in figure 5.3. Consequently, this results in lower excess capacity factors for steel reinforcement than concrete. In other words, this factor can be used as the governing excess capacity factor for this study. Applying this method to the structural design codes in use from 1962 to the present results in the excess capacity factors presented in table 5.5. Their derivation is presented in the following sections.

5.2.2. Derivation of excess capacity factors

The derivation of excess capacity factors starts with the analysis of safety margins, which results in the identification of a general safety factor γ_{total}. Both reinforcing steel and concrete are considered, resulting in different safety factors for compression and tension. The safety factors can be combined to derive excess capacity factors for concrete and reinforcing steel.

Concrete

Up to and including 1962, only a safety margin can be identified between the mean cubic compression stress from three tested cubes of size 200 mm and the allowable compression forces resulting from the actual self-weight and unfavorable variable load [165, 49]. From 1974, concrete cubes of 150 mm were used to calculate the mean cube compression strength. Therefore, a factor of 1.05 was applied to the mean cube compression strength of concrete tested before this time [165]. The mean cube compression strength of 150 mm cubes is referred to as μ_{mat}. The safety margin from GBV 1912 to GBV 1962 is represented by the safety factor $\gamma_{total,com}$, which is shown in figure 5.1 and calculated by equation (5.6).

$$\gamma_{total,com} = \frac{\mu_{mat}}{\text{allowable compression stress}} \tag{5.6}$$

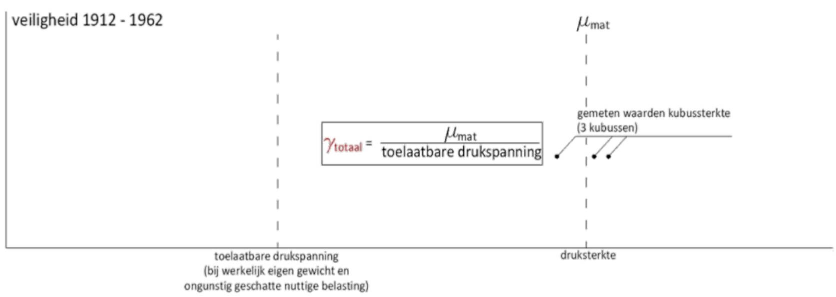


Figure 5.1: Safety factor $\gamma_{\text{total,com}}$ for concrete under axial compression from GBV 1912 to GBV 1962 [165]

From 1974 onward, the use of statics led to the introduction of characteristic values for both material strength and loading [154]. Both the material strength as the loading were assumed to follow a normal distribution. The characteristic value of material strength is defined as the value exceeded by 95% of tested specimens, meaning it is only lower in 5% of cases. Similarly, the characteristic value of a load is the value that is exceeded in 5% of cases. These characteristic values, S_k and R_k, are then adjusted using a material or load factor, γ_{mat} or γ_{load}, to arrive at a design value. Where the design load S_d must not exceed the design resistance R_d. According to van Uffelen [165], the allowable stress of the loaded material can be compared with the characteristic value of the load. Since the characteristic value was not defined for concrete prior to 1974, and the mean cube compressive strength can be approximated for concrete after 1974, the mean cube compressive strength μ_{mat} is used as the basis for the safety factor [165]. This choice also avoids neglecting a significant portion of the safety margin, which would occur if the characteristic strength were used instead. For Eurocode 2012, the mean compressive strength of concrete was determined using cylinder tests instead of cubes [82]. NEN 1992 [113, Table 3.1] specifies the mean cylinder compressive strength, the characteristic cylinder compressive strength, and the characteristic cube compressive strength, but not the mean cube compressive strength. For the purpose of this analysis, it is assumed that the difference between the characteristic and mean values for cylinder strength is equal to the difference between the characteristic and mean values for cube strength, which is 8 N/mm². The safety margin from VB 1974 to Eurocode 2012 is represented by the safety factor $\gamma_{total,com}$, which is shown in figure 5.2 and calculated by equation (5.7).

$$\gamma_{total,com} = \gamma_{load} \cdot \gamma_{mat} = \frac{S_d}{S_k} \cdot \frac{\mu_{mat}}{R_d} = \frac{\mu_{mat}}{S_k} \tag{5.7}$$

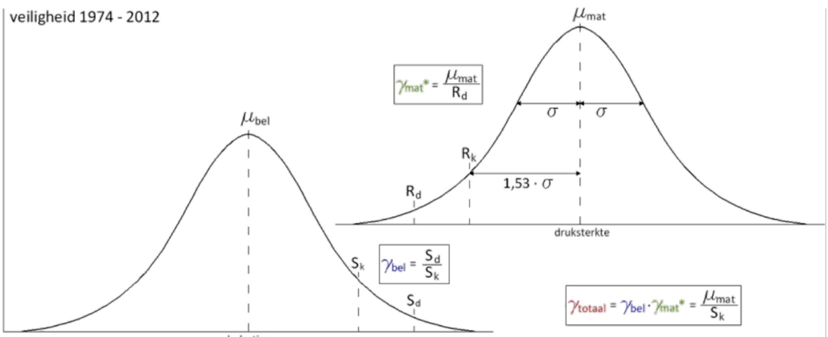


Figure 5.2: Safety factor $\gamma_{total,com}$ for concrete under axial compression from VB 1974 to Eurocode 2012 [165]

Steel reinforcement

The derivation of the safety factor $\gamma_{total,ten}$ for steel reinforcement follows the same approach as for concrete. Until 1962, the safety margin was defined as the ratio between the maximum allowable stress and the average tensile strength, μ_{mat}. This results in the same graph and equation for $\gamma_{total,ten}$ as presented in figure 5.1 and equation (5.6). When applied to tensile strength, the corresponding expression is provided in equation (5.8).

$$\gamma_{total,ten} = \frac{\mu_{mat}}{\text{allowable tensile stress}} \tag{5.8}$$

From 1974 onward, characteristic values and design values were introduced. The previously applied safety margin was split into a load factor and a material factor. VB 1974 states that the characteristic value of tensile strength may be taken as either the yield strength or the 0.2% proof stress [154]. As before, the characteristic values were converted into design values using either a load factor γ_{load} or material factor γ_{mat}. Figure figure 5.2 illustrates this principle, while equation (5.9) provides the calculation of the safety factor, γ_{total}, for reinforcement steel in tension between VB 1974 to Eurocode 2012.

$$\gamma_{total,ten} = \gamma_{load} \cdot \gamma_{mat} = \frac{S_d}{S_k} \cdot \frac{\mu_{mat}}{R_d} = \frac{\mu_{mat}}{S_k} \tag{5.9}$$

Safety factors from 1960 to 2012

The previous section translates the identified safety margins into safety factors, which are based on the applicable structural guidelines and the material strength. The available concrete and steel strengths has increased over time, so in order to compare the safety factors of the code years, materials with similar average strengths, μ_{mat}, are used. Concrete with an average cubic compressive strength of 30 N/mm² (as this corresponds with the highest concrete strength of 1962) and steel with an average tensile strength of 500 N/mm² are chosen. From 1990 onward, two load combinations with separate partial load factors were introduced for permanent and variable loading, as discussed in section 5.2.1. Given the actual load distribution is unknown an assumption must be made. Assuming a load combination of 50% permanent and 50% variable loading for one load combination of both NEN 1990 and EC 2012 yields in the same factor as the partial load factor for the permanent load of the other load combination. The partial load factor for variable loads of the latter load combination is either non existent in NEN 1990 and lower in EC 2012. For this reason, the partial load factor of 1.35 is used in the analysis for NEN 1990 and EC 2012 as γ_{load}. Moreover, the permanent load factor results in the governing excess capacity factor in the next stage of the analysis, which further justifies the use of this factor. Figure 5.3 and table 5.4 show how the safety factor, γ_{total}, both for steel reinforcement and concrete has decreased over the years. The detailed derivation of γ_{total} is provided in appendix B.1.

Code year	$\gamma_{total,com}$	$\gamma_{total,ten}$
1962	4.2	2.2
1974	3.8	2.1
1984	3.8	2.1
1990	3.0	1.9
2012	2.7	1.9

Table 5.4: Safety factors for concrete in compression $\gamma_{total,com}$ and steel in tension $\gamma_{total,ten}$ from GBV 1962 to Eurocode 2012

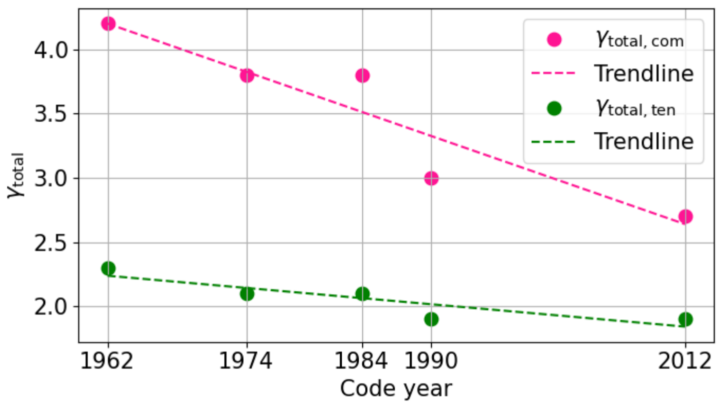


Figure 5.3: Safety factors for concrete in compression $\gamma_{total,com}$ and steel in tension $\gamma_{total,ten}$ from GBV 1962 to Eurocode 2012

Calculation of excess capacity factors

As discussed in section 5.2.1, two safety ratios must be derived and multiplied to yield the excess capacity factor. The first safety ratio, referred to as Safety Ratio 1, can be determined using the safety factors presented in the previous section. With Safety Ratio 1, the safety factors from earlier codes are compared to those of the current code, as the difference between them can be interpreted as excess capacity. In addition, the partial load factor from the current code for new construction must be compared to that of the current code for existing structures, as different partial load factors were introduced by the NEN 87 series [115]. This results in the second safety ratio, referred to as Safety Ratio 2, which is based solely on partial factors for permanent loads. This choice is made because the actual load combination is unknown and partial factors of permanent loads yield the lowest safety ratio, leading to governing excess capacity values. The derivations of Safety Ratio 1 and Safety Ratio

2 are provided in appendix B.2. The excess capacity factors are calculated using equation (5.10) and equation (5.11), where the relevant safety ratios are multiplied, and one is subtracted to express the result as a percentage. This percentage is based on the assumption that excess capacity represents an allowable increase relative to the original characteristic load. Therefore, the excess capacity factor is always applied with respect to the original load. The reasoning behind expressing excess capacity as a percentage of the original load is presented by equation (5.12). The excess capacity factors for concrete and steel are listed in table 5.5 and visualized in figure 5.4. These results confirm the observations already visible in figure 5.3, the excess capacity of the steel reinforcement is lower than that of concrete. Given that this study focuses on floors and facade beams, where steel reinforcement is typically governing, the excess capacity factors for steel reinforcement are considered applicable for the purpose of this analysis.

$$C_{x,com} = \text{Safety Ratio 1,com} \cdot \text{Safety Ratio 2} - 1 = \frac{\gamma_{total,com}}{\gamma_{total,com,2012}} \cdot \frac{\gamma_{load,2012}}{\gamma_{load,87}} - 1 \tag{5.10}$$

$$C_{x,ten} = \text{Safety Ratio 1,ten} \cdot \text{Safety Ratio 2} - 1 = \frac{\gamma_{total,ten}}{\gamma_{total,ten,2012}} \cdot \frac{\gamma_{load,2012}}{\gamma_{load,87}} - 1 \tag{5.11}$$

$$C_{x,ten} = \frac{s_{k,code} + C_{F,ten}}{s_{k,code}} - 1 = \frac{C_{F,ten}}{s_{k,code}} \tag{5.12}$$

Code year	$C_{x,com}$	$C_{x,ten}$
1962	0.66	0.25
1974	0.46	0.14
1984	0.46	0.14
1990	0.14	0.04
2012	0.04	0.04

Table 5.5: Excess capacity factors $C_{x,com}$ and $C_{x,ten}$ per code year

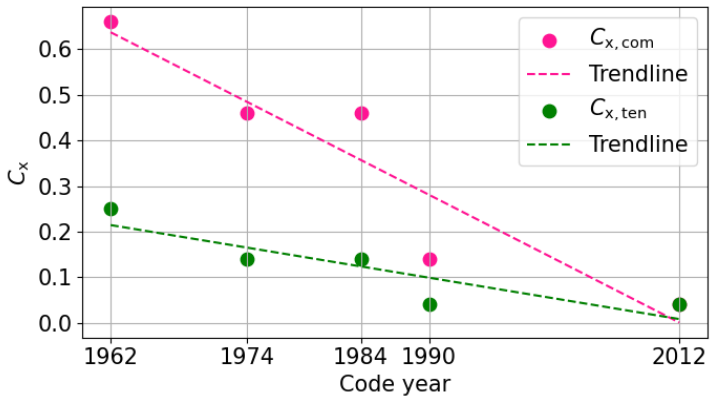


Figure 5.4: Excess capacity factors for concrete in compression $C_{x,com}$ and steel in tension $C_{x,ten}$ per code year

Implementation of excess capacity factors

The previous section presented the calculation of excess capacity factors and proposed the use of the excess capacity factor for reinforcing steel in tension, E_{ten}. As the structural system behind the facade is unknown at the start of the feasibility phase, the only load that can be estimated is that of the facade itself. The assumption made is that the facade is supported by a facade beam or by the floor, which also justifies using the excess capacity factors for reinforcing steel. Typically, the facade beam or the floor also carries additional loads, such as its own self-weight or variable floor loads, but these are assumed to be unknown at this stage. Therefore, only the excess capacity related to the facade load is calculated in this first excess capacity calculation. However, this value is considered a conservative estimate of the total excess capacity of the facade. Table 4.1 provides estimated weights of facades used between 1960 and 1990, based on their external appearance and the expected internal composition. To demonstrate the potential range of excess capacity for these facades, calculations are performed for the lightest and the heaviest facade identified in table 4.1, which are the classic curtain wall and the self-supporting masonry facade. For the calculations, a story height h_{st} of 3 meters is used. The calculations follow equation (5.13) and the results are presented in table 5.6.

$$C_{F,fac} = C_{x,ten} \cdot g_{fac} \cdot h_{st} \tag{5.13}$$

Code year	$C_{F,curtain}$ (kN/m)	$C_{F,masonry}$ (kN/m)
1962	0.74	1.7
1974	0.43	0.99
1984	0.43	0.99
1990	0.12	0.28
2012	0.12	0.28

Table 5.6: Calculated excess capacity per story for a classic curtain wall and a self-supported masonry facade per code year based solely on the self-weight of the facade

5.2.3. Evaluation of excess capacity factors

Focusing on the timeframe of this study, the analysis is limited to the excess capacity factors related to GBV 1962, VB 1974, and VB 1984. Naturally, EC 2012 and the NEN 87 series are also included in the derivation of the excess capacity factors. Several assumptions have been made in the analysis of these factors, which are worth evaluating, as they may significantly influence the determination of excess capacity.

Choice of partial load factors

The evaluation begins with the selection of partial load factors for γ_{load} of NEN 8700 and NEN 1990 in the analysis. As explained in section 5.1.2, these codes employ two fundamental load combinations for the ultimate limit states, each with distinct partial load factors per combination and per code. For the determination of the safety factors, a value of 1.35 is used for γ_{load}, as justified in the previous sections. In the derivation of Safety Ratio 2, the partial load factors for permanent loading are also adopted. Both choices lead to lower excess capacity factors. Ultimately, these excess capacity factors can be multiplied by the originally applied load on an element to estimate the excess capacity of that element. Consequently, the original classification of the load as either permanent or variable can significantly influence the actual resulting excess capacity. If the contribution of variable loads is substantial, the actual excess capacity may be considerably higher than the value derived using the current factor. This raises the question of whether the assumption to use partial factors for permanent loads may be overly conservative. A more accurate estimation of excess capacity could be achieved if the distribution between permanent and variable loads were known and separate excess capacity factors could be applied. Unfortunately, this is typically not the case in the feasibility phase.

Element-type dependence

As explained in section 5.2.2, GBV 1962 does not specify a separate load factor and material factor. Instead, the safety margin between the allowable stress and the mean strength is used to calculate $\gamma_{total,com}$. However, in GBV 1962, the allowable compressive strength depends on the loading type. GBV 1962 distinguishes the following types: axial compression, eccentric compression, bending, shear force, torsion, and combined shear force and torsion [49, Art. 44]. For each loading type, allowable compressive strengths are provided for concrete with and without steel reinforcement. For reinforced concrete of quality K300, the allowable compressive stress varies between 7.5 N/mm² and 10 N/mm², depending on the loading type [49, Art. 44]. For steel reinforcement, only the allowable tensile stress for steel reinforcement in bending is stated. In this analysis, the allowable compressive stress for axial compression was used. This choice is justified as it most closely resembles the mean cube compressive strength and corresponds to the lowest allowable compressive strength in the code. However, these values are inherently linked to specific loading types, which in turn often correspond to particular structural elements. Therefore, this phenomenon is referred to as element-type dependence in this study.

Although this element-type dependence is also reflected in the derivation of $\gamma_{total,com}$ in VB 1974, it originates from a different underlying parameter. For VB 1974, $\gamma_{total,com}$ is derived by multiplying γ_{load} with the mean compressive strength μ_{mat}, and dividing by the design compressive strength R_d. In this case, the design compressive strength is based on the loading type. VB 1974A [154, Art. 204.5.1] provides the following options: bending with a small normal force, bending with a large normal force, or bending with a tensile force. Furthermore, different compressive strengths are defined at the supports. For example, the characteristic compressive strength of concrete is reduced by 25% when subjected to a large normal force [154, Art. 204.5.1]. This difference arises from the fact that the compressive strength of concrete decreases over time, which is particularly relevant for elements subjected to large normal forces, as they rely heavily on the compressive strength of the concrete [18]. In contrast, for elements under bending with small normal forces, the steel reinforcement is typically governing. Therefore, for concrete in bending, the design compressive strength is equal to the characteristic compressive strength according to VB 1974 [154, Art. 204.5.1]. However, in the derivation of the excess capacity factors above, the design compressive strength used corresponds to concrete subjected to a significant axial force, as would be the case in an element such as a column.

The derivation of $\gamma_{total,com}$ of EC 2012 is not dependent on the loading type. Instead, it is based solely on the mean cylinder compressive strength, a partial material factor and a partial load factor [113]. Load type becomes relevant in detailed element-level design, which lies beyond the scope of this analysis. In conclusion, both the derivations of the excess capacity factor in GBV 1962 and VB 1974 are dependent on the type of the loading, which is typically linked to a specific structural element, such as a column or beam. Therefore, it remains uncertain whether the calculated excess capacity factor is universally applicable to all structural elements.

Governing resistance and material influence

As stated in section 5.2.1, it is assumed that steel reinforcement governs the design of slabs and beams. Therefore, it also governs the derivation of the excess capacity of the element. This reflects the prevailing design principle in most structural building standards where in the ultimate limit state design of bending elements the reinforcement must yield before the concrete crushes, such as VB 1974 and EC 2012 [21, 18]. However, while this principle ensures ductile failure, the concrete's strength remains a key contributor to the bending moment capacity, meaning that steel is not the sole determining factor. Moreover, bending moment capacity is not necessarily the only governing limit state. For example, high shear demands may shift the critical resistance check to shear, where both concrete and reinforcement interact in a different way. Additionally, deflection, which is part of the serviceability limit state, often governs the design of slabs. VB 1974 and GBV 1962 prescribe minimum slab thicknesses or beam heights, which may also be decisive in the design [49, 156]. In conclusion, the choice of governing capacity calculation affects the applicable excess capacity factor and in turn the resulting excess capacity. This questions the universal validity of the assumption that steel reinforcement always governs excess capacity calculations of beams and slabs.

Principle of using Excess capacity factors

Equation (5.10) and equation (5.11) subtract 1 to express the results as percentages, representing the excess capacity factors for compression and tension. This subtraction is based on the assumption that the excess capacity is an increase towards the original load and can be based on the original load solely. Since the structure is assumed to remain unchanged, with only additional permanent load being added, this assumption initially appears reasonable. However, both the assumed self-weight of concrete and the prescribed variable loads have changed over time. For example, the volumetric weight of reinforced concrete is taken as 25 kN/m^3 in EC 2012 [110], compared to 24 kN/m^3 in TGB 1955 [61, Tabel I]. Similarly, the variable load on floors including lightweight partition walls is set at 2.5 kN/m^2 in TGB 1955 [61, Tabel II], whereas in EC 2012 the value increases to 2.5 + 1.2 = 3.7 kN/m^2 [110, p. 6.3.1.2] when accounting for the heaviest variant of such walls. Lighter variants of the partition walls are 0.8 or 1.0 kN/m^2 instead of 1.2 kN/m^2 [110, p. 6.3.1.2]. This higher value is used in this study under the assumption that masonry may have been included among the lightweight partition wall types used during the period in question. This challenges the validity of the assumption that excess capacity can be fully attributed to an increase relative to the original load, as part of the calculated capacity may in fact be required to reconcile changes in load definitions over time.

Correct safety margin

Lastly, the safety factors are based on the margin between the material mean strength μ_{mat} and either the characteristic strength R_k or the allowable stress S_k. However, caution is warranted when interpreting this entire margin as structural excess capacity. Safety margins in concrete design serve a critical function, accounting not only for model uncertainty and load variability but also for the natural scatter in material properties and the quality of construction [18, 21]. Especially in earlier decades, material strengths exhibited larger variability, making conservative safety factors essential [165]. Thus, the difference between μ_{mat} and R_k reflects more than just unused capacity, it also captures uncertainty that Dutch building regulations aims to cover explicitly. Moreover, μ_{mat} is not used in calculations according to EC 2012 or the NEN 87 series for the assessment of existing structures, nor is it used in GBV 1962, VB 1974, or VB 1984 for the design of concrete elements [49, 154, 156, 157]. This makes it difficult to directly relate μ_{mat} to structural performance. If the safety margin is limited only to the difference between R_k and S_k, the resulting excess capacity would be significantly lower. Finally, it is important to note that from 1974 onward, the value of μ_{mat} is based on estimations, as outlined in section 5.2.2. As Linssen [82] points out, it cannot be precisely determined from available data. This introduces an additional level of uncertainty in using μ_{mat} as a reference value in the derivation of excess capacity factors and calls for caution when interpreting it as a reliable measure of available excess capacity.

6
Parameter study

Focusing specifically on the excess capacity of facades built between 1960 and 1990, a parameter study is conducted on potential beam-column configurations designed according to the structural design codes of those times. The findings of this study will be used to develop a framework in the next chapter.

6.1. Motivation

The uncertainties described in section 5.2.3 mainly concern how the actual excess capacity relates to the calculated excess capacity factors and whether these factors are applicable in practice. In this context, applicability refers to the extent to which the results are consistent with the principles and outcomes of current design standards for assessing the structural capacity of existing concrete structures, namely the NEN 8700 series and EC 2012. The magnitude of the excess capacity factor is particularly questioned in relation to several aspects: the distinction between variable and permanent load factors, the contribution of steel compared to concrete, the dependence on the type of structural element, and differences in values for the same loads. If the same excess capacity factor can be applied to similar structural elements designed according to the same structural design code, this would provide a useful simplification in the feasibility phase. This is especially the case if the factor can be implemented as straightforwardly as shown in equation equation (5.13). To address these uncertainties, a parameter study is proposed in which the excess capacity is calculated for facades supported by beams or floors that were designed according to earlier structural design codes. These facades can be classified as self-supporting facades, as defined in section 4.3.1. A deliberate choice was made to focus on this facade configuration, as these facades are supported by facade beams or floors rather than by columns or load-bearing walls. Van Uffelen [165] concluded that columns are likely to have sufficient excess capacity and that beams are expected to be governing, as explained in section 1.4. This observation is also reflected in figure 5.4, where the excess capacity factors for concrete in compression are considerably higher than those for steel in tension for earlier codes. Since columns and load-bearing concrete walls primarily rely on the compressive strength of concrete, it is assumed that facades supported by such elements have higher excess capacity than those supported by facade beams or floors. As this study aims to provide a method for estimating the excess capacity of a facade during the feasibility phase, the focus is placed on identifying the governing excess capacity and a potential range rather than the maximum possible value.

6.2. Methodology

The parameter study focuses on two structural systems. System 1 refers to a facade supported by a concrete facade beam, also referred to as the Beam System, shown in figure 6.1. In system 2, the facade is supported by a concrete slab, also referred to as the Slab System, as shown in figure 6.2. The beam or slab from either system is considered a simply supported element subjected to uniform loading. Parameters for both systems are defined and grouped into sets, referred to as combinations. For each combination, the beam or slab of the system is designed according to the applicable structural design codes and literature from the corresponding period. The structural design methods of VB 1974 and VB 1984 show minimal differences in the initial design of reinforced concrete slabs and floors. The only notable change is an increase of 5 mm in the minimum concrete cover for beams and floors [154, 157]. The effect of this minor difference on structural performance is minimal. Furthermore, with the focus on governing excess capacity, VB 1974 is used for both versions. To fully cover the time frame of this study, from 1960 to 1990, two structural codes are therefore included for the design of the systems in the parameter study: GBV 1962 and VB 1974. To ensure comparability, concrete and steel with approximately similar strengths have been used in the designs to allow for a fair comparison of excess capacities across the different codes. Concrete with μ_{mat} of approximately 30 N/mm² and steel with μ_{mat} of approximately 500 N/mm² are chosen. The structural resistance of the designed systems is analyzed using the NEN 8700 series, as detailed in section 5.1. In this way, the excess capacity according to current structural guidelines is determined. Note that some parameters are denoted using different symbols in GBV 1962 or VB 1974 than in EC 2012. To ensure clarity and consistency, this chapter follows the current notation from EC 2012 as much as possible, with definitions given where appropriate.

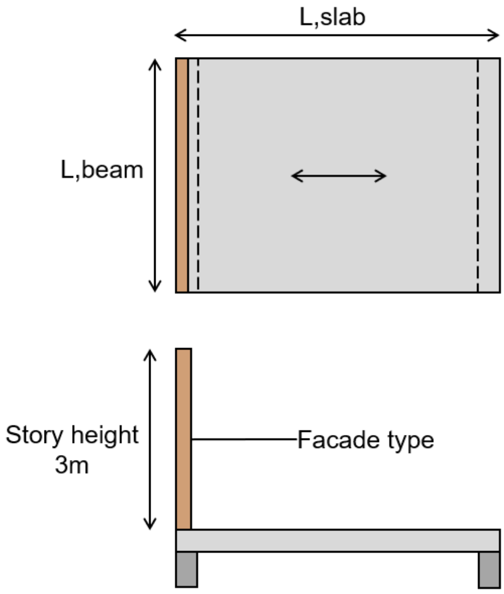


Figure 6.1: Sketch of structural system 1: the Beam System

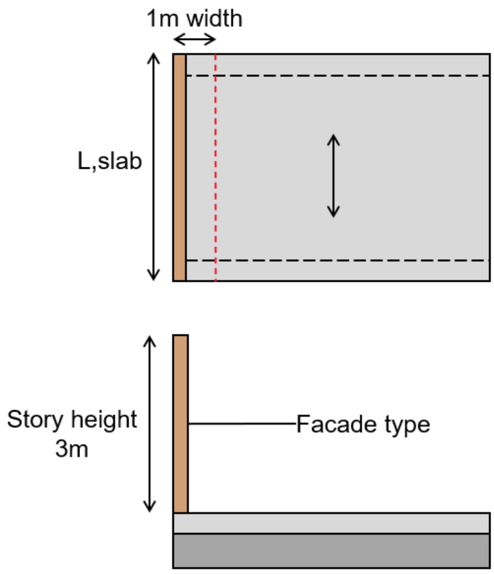


Figure 6.2: Sketch of structural system 2: the Slab System

6.2.1. Parameter combinations

System 1: the Beam System

In system 1 the facade beam supports the floor and the facade. The floor spans in one way and the facade of one story height is supported by the facade beam. One story height is assumed 3 meters. The researched parameters for the Beam System are the length of the beam (L_{beam} or B), the length of the slab (L_{slab} or S) and the Facade weight (q_{fac} or F). The values used for the parameters are listed in table 6.1. The parameters are combined to result in 24 combinations, specified in table D.2.

Parameters				
L_{beam}	B1 = 3.6 m	B2 = 5.4 m	B3 = 7.2 m	
L_{slab}	S1 = 3.6 m	S2 = 5.4 m	S3 = 7.2 m	S4 = 9 m
g_{fac}	F1 = 1 kN/m²	F2 = 2.3 kN/m²		

Table 6.1: Parameters used for the parameter study of system 1

System 2: the Slab System

In system 2 the slab supports the facade of one story height. One story height is assumed 3 meters. The researched parameters for the Slab System are the length of the slab (L_{slab} or S) and the Facade weight (q_{fac} or F). The values used for the parameters are listed in table 6.2. The parameters are combined to result in 8 combinations, specified in table D.4.

Parameters				
L_{slab}	S1 = 3.6 m	S2 = 5.4 m	S3 = 7.2 m	S4 = 9 m
g_{fac}	F1 = 1 kN/m²	F2 = 2.3 kN/m²		

Table 6.2: Parameters used for the parameter study of system 2

6.2.2. Design assumptions

Since multiple valid design options may exist for each parameter combination, it is essential to define design assumptions to ensure consistency and realism in the resulting designs. The overarching design principle adopted in this study is to "Design as they would". To obtain meaningful results regarding excess capacity, the structural designs must be representative of how buildings were actually constructed during the period in question. This requires not only adherence to the formulas prescribed in the historical structural codes, but also informed choices within the flexibility those formulas allow. For any given parameter combination, multiple valid designs are often possible. However, some of these are more plausible or historically consistent than others, based on typical design practices of the time. For example, construction materials such as concrete and steel were relatively expensive during the relevant period, while labor costs played a less dominant role in total construction costs [49] (section E.2). As a result, designers were more inclined to minimize material use. In this study, that assumption is interpreted as optimal material utilization, where the calculated unity checks are intended to closely approach their governing limits. To minimize material use, designers may have preferred using more reinforcement bars with smaller diameters instead of fewer, larger bars. This design principle is also supported by Schrier [146] in his 1965 book on concrete design, where he states that using more reinforcement bars with smaller diameters at closer spacing improves stress transfer. However, this principle also has its limitations. Designing with as little material as possible could, for instance, lead to unrealistic beam proportions in relation to their span, or reinforcement bars with diameters disproportionate to the size of the cross-section. To avoid such outcomes, reasonable parameter ranges have been established to guide the design process.

System 1: the Beam System

The parameter ranges for the beam designs of System 1, according to GBV 1962 and VB 1974, are listed below. The beam height h_{beam} is estimated based on the beam span, as suggested by GBV 1962 [49]. However, for the purpose of this analysis, the range for beam height is not strictly bounded. Since the combinations include extremes in q_{fac} and L_{slab}, this may lead to less conventional heights. The beam width b_{beam} is derived from the beam height [151] and is treated as strictly bounded, in order to maintain proportionate cross-sections. A minimum width of 200 mm is applied, based on the recommendation by Schrier [146], who notes that this is the minimum required to properly place reinforcement bars. The slab thicknesses t_{slab} used for System 1 are initially estimated as described below [151]. However, their final values are based on the design calculations performed for System 2, excluding the weight of the facade. Although floors are designed in accordance with both GBV 1962 and VB 1974, they result in the same applicable thicknesses. The final values of t_{slab} per slab span are listed in table 6.3. The values of h_{beam}, b_{beam} and t_{slab} are rounded to increments of 5 mm.

The range for the main reinforcement bar diameters is based on the diameters that were expected to be available and commonly used in beams at the time [165]. A standard spacing between the main reinforcement bars, s_{main}, is applied under the principle of minimizing material use and ensuring governing excess capacity. This spacing respects the minimum requirement prescribed in GBV 1962, which states that the bar distance must be no less than the maximum of 30 mm or the diameter of the largest bar [49, Art. 34.1]. Furthermore, GBV 1962 requires the use of stirrups in beams and prescribes a minimum diameter of 6 mm [49, Art. 34.3–34.5]. In this study, however, a minimum diameter of 8 mm is adopted. This value was observed in historical designs reviewed by van Uffelen [165] and serves as a practical starting point for most parameter combinations. The maximum spacing of stirrups s_{st} is also regulated in GBV 1962. In this study, increments of 50 mm are applied, with 50 mm also serving as the minimum spacing [42]. The value of the concrete cover c for beams in a dry environment is the same according to GBV 1962 [49, Art. 28] and VB 1974 [155, Tabel B-1].

- $h_{beam} = \frac{1}{12} \cdot L_{beam}$ to $\frac{1}{9} \cdot L_{beam}$
- $b_{beam} = \frac{1}{5} \cdot h_{beam}$ to $\frac{1}{3} \cdot h_{beam}$, with $b_{beam} \geq 200$ mm
- $t_{slab} \geq \frac{1}{35} \cdot L_{slab}$
- $\phi_{main} = 12$ to 32 mm, with $\phi_{main} \in \{12, 14, 16, 18, 20, 25, 32\}$ mm
- $s_{main} = 35$ mm, with $s_{\text{main}} \geq \max(30\text{ mm}, \phi_{main})$
- $\phi_{st} = 8$ to 10 mm, with $\phi_{st} \geq 6$ mm
- $s_{\text{st}} = 50$ to 300 mm, with $s_{\text{st}} \leq \min(300\text{ mm}, \frac{2}{3} \cdot h_{\text{beam}})$
- $c = 20$ mm

L_{slab}	t_{slab}	[mm]
S1	t1	120
S2	t2	175
S3	t3	225
S4	t4	280

Table 6.3: The values of t_{slab} for System 1 combinations are determined according to the corresponding slab span L_{slab}

System 2: the Slab System

The parameter ranges for the beam designs of System 2, according to GBV 1962 and VB 1974, are listed below. The slab thicknesses for System 2 use the same lower bound as System 1, but the final value of t_{slab} is determined by the design calculations, as explained in section 6.3. A minimum slab thickness of 80 mm is prescribed in GBV 1962 [49, Art. 32.4] and VB 1974B [155, Art. 709.1]. The slab width is taken as 1000 mm, as illustrated in figure 6.2. The range for the main reinforcement bar diameters is based on those expected to be available and commonly used in slabs during that period [165]. A range for spacing between the main reinforcement bars, s_{main}, is specified in both GBV 1962 [49, Art. 34.2] and VB 1974 [156, Art. 709.3]. As these specifications differ slightly, both are listed below. For this study, a minimum spacing of 90 mm is adopted, as smaller distances are assumed to have been uncommon. The value of the concrete cover c for slabs in a dry environment is the same according to GBV 1962 [49, Art. 28] and VB 1974 [155, Tabel B-1].

- $t_{slab} \geq \frac{1}{35} \cdot L_{slab}$ with $t_{slab} \geq 80$ mm
- $b_{slab} = 1000$ mm
- $\phi_{main} = 10$ to 25 mm, with $\phi_{main} \in \{10, 12, 14, 16, 18, 20, 25\}$ mm
- $s_{main} \geq 90$ mm, with 25 mm $\leq s_{\text{main}} \leq \min(200\text{ mm}, 2 \cdot t_{slab})$ according to GBV1962
- $s_{main} \geq 90$ mm, with $\max(25\text{ mm}, \phi_{main}) \leq s_{\text{main}} \leq \min(250\text{ mm}, 2 \cdot t_{slab})$ according to VB1974
- $c = 10$ mm

6.2.3. Hypotheses

The selection of parameters was made deliberately to reflect the intended variation in the study. As suggested by section 5.2.2, a larger original load is expected to lead to a higher excess capacity, which supports the use of excess capacity factors. This is especially valid if the excess capacity factor remains constant for similar structural elements designed according to the same code, as it would then allow for a straightforward application in the form of equation (5.13). However, a greater proportion of variable loads relative to permanent loads could also result in increased excess capacity. Unfortunately, the parameter study is not suited to investigate this, as this proportion is not explicitly researched. Moreover, differences in used values for loads across code years may negatively impact excess capacity, as discussed in section 5.2.3. Furthermore, the element type may also influence the excess capacity, as explained in section 5.2.3. However, since both the slab and the beam are considered elements in bending to which the same allowable stress (GBV 1962) and safety factors (VB 1974) apply, no significant difference is expected in this regard. Investigating this potential difference would require a comparison with a column, which is beyond the scope of this study. However, the excess capacity factors $C_{x,com}$ in table 5.5 are derived based on column characteristics. As such, lower values are expected for beams in general. Additionally, as stated in section 5.2.2, it is assumed that the reinforcing steel governs the excess capacity calculation, since it typically governs the design of concrete beams and floors. This assumption implies that the excess capacity factors resulting from this parameter study will more closely resemble the excess capacity factors for reinforcing steel in tension, $C_{x,ten}$, than those derived for concrete in compression, $C_{x,com}$. These expectations can be translated into the following hypotheses:

1. Beams or slabs with a higher original total distributed load, $s_{k,beam}$ or $s_{k,slab}$, are expected to exhibit a higher excess capacity, $C_{F,beam}$ or $C_{F,slab}$.
2. For structural elements of the same type that were designed according to the same structural design code, it is expected that a single excess capacity factor $C_{x,element,code}$ can be applied in a similar manner as equation (5.13).
3. An increase in used values for loads used in earlier codes compared to EC 2012 negatively impacts excess capacity C_F.
4. Excess capacity factors derived from beams or slabs are expected to resemble $C_{x,ten}$ more than $C_{x,com}$.
5. No significant difference is expected between the excess capacity factors C_x corresponding to the Beam System and those corresponding to the Slab System.

6.3. Design according to GBV 1962

This section outlines the design approach and calculations used for the beams of System 1 and the slabs of System 2, based on the guidelines of GBV 1962. It starts by presenting the general design principles and load assumptions in line with the GBV 1962. The following subsections provide a detailed explanation of the calculations related to bending moment capacity, shear capacity, and deflection requirements for both structural systems.

6.3.1. Design principles

GBV 1962 outlines various stages in the loading process of a slab or beam subjected to bending. The design calculations according to the n-method are based on the stage in which the concrete has cracked in the flexural tensile zone and the tensile stress is almost entirely carried by the steel reinforcement. The parameter n represents the ratio between the modulus of elasticity of steel and that of concrete, as shown in equation (6.1) [49, Art. 42.1]. A further increase in load ultimately results in failure, either due to the reinforcement exceeding its tensile strength or the concrete exceeding its compressive strength, depending on the applied reinforcement ratio. An alternative design approach, referred to as the ultimate capacity method, is based on this failure stage. Since this method was introduced for the first time in GBV 1962 and was not suitable for all design situations, the choice has been made to adopt the n-method for the current analysis. The following assumptions apply for the design according to the n-method as described by GBV 1962 [49, Art. 42.1]:

1. Strains in the fibres due to bending vary linearly with the distance to the neutral axis, referred to as Navier's bending theory.
2. Tensile stress resulting from bending are resisted exclusively by the reinforcement, referred to as the assumption of Mörsch.
3. Stresses in the cross-section are linearly related to the corresponding strains.
4. The value of the ratio n shall not exceed 15.

$$n = \frac{E_a}{E_c} \quad \text{[-]} \tag{6.1}$$

6.3.2. Material properties

The material properties used for the design according to GBV 1962 are listed in table 6.4. As presented in table 6.4, GBV 1962 employs not only different symbols but also different units compared to current standards. To facilitate comparison with more recent codes, most of the original units have been converted to those commonly used today. However, certain formulas in GBV 1962 are unit-specific and therefore retain their original units. For that reason, the corresponding values are also listed in their original units in table 6.4. The assumed value for the ratio n is 15, in line with the general recommendation provided in GBV 1962 [49, Art. 42.1]. The allowable tensile strength of steel reinforcement in bending, $\bar{\sigma}_a$, for steel types up to and including QR40 may be increased by 10 N/mm² if the slab thickness exceeds 150 mm according to Schrier [146, par. 6].

Symbol	Value	Unit	Value	Unit	Definition
γ_c	24	kN/m³			Volumetric weight of reinforced concrete
n	15	-			Ratio of the modulus of elasticity of steel to that of concrete
$\bar{\sigma}'_\text{b}$	10	N/mm²	100	kgf/cm²	Allowable compressive strength of concrete in bending
$\bar{\sigma}_\text{b}$	0.8	N/mm²	8	kgf/cm²	Allowable tensile strength of concrete in shear without reinforcement present
$\bar{\sigma}_{b,max}$	2.0	N/mm²	20	kgf/cm²	Allowable tensile strength of concrete in shear with reinforcement present
$\bar{\sigma}_a$	220	N/mm²	2200	kgf/cm²	Allowable tensile strength of steel reinforcement in bending

Table 6.4: Material properties for concrete class K300 and steel reinforcement QR40 according to GBV 1962, retrieved from NEN 8702 [117, Tabel 2], GBV 1962 [49, Art. 42, 44] and TGB 1955 [61, Tabel I]

6.3.3. Loads

The variable and permanent loads acting on the elements are calculated according to the formulas provided in appendix C.1.1. Where, the facade load g_{fac} is taken as F1 or F2, as specified in table 6.1 and table 6.2. The variable floor load $Q_{k,slab}$ is taken as 2.5 kN/m², based on TGB 1955 [61, Tabel II]. This value includes an allowance for lightweight partition walls, in accordance with TGB 1972 [118, p. 2.2.1.1.]. Since no specific load factor is applied under GBV 1962, the total permanent and variable loads acting on the beam are simply summed to obtain the design load [49]. The formula for the design line load $s_{d,beam}$ acting on the beam is provided in equation (6.2) and the formula used to calculate the design load $s_{d,slab}$ acting on the slab is provided in equation (6.3).

$$s_{d,beam} = s_{k,beam} = q_{k,beam} + g_{k,beam} \quad \text{[kN/m]} \tag{6.2}$$

$$s_{d,slab} = s_{k,slab} = q_{k,slab} + g_{k,slab} \quad \text{[kN/m]} \tag{6.3}$$

Where:

- $q_{k,beam} =$ variable load acting on the beam in [kN/m], calculated by equation (C.4)
- $g_{k,beam} =$ permanent load acting on the beam in [kN/m], calculated by equation (C.3)
- $q_{k,slab} =$ variable load acting on the slab in [kN/m], calculated by equation (C.5)
- $g_{k,slab} =$ permanent load acting on the slab in [kN/m], calculated by equation (C.2)

6.3.4. Bending moment capacity

The acting bending moment M_{Ed} is calculated according to equation (C.16). The induced stress in the concrete or the steel are calculated according to control formulas provided by Schrier [146]. It should be noted that assuming a linear stress–strain distribution across the cross-section defines the assumed height of the resultant compressive force, therefore it influences z. The induced compressive stress in concrete is calculated with equation (6.5) and the induced tensile stress in steel is calculated with equation (6.4). The resisting bending moment is determined according to equation (6.6).

$$\sigma_a = \frac{M_{Ed}}{A_s \cdot z} \quad \text{[N/mm²]} \tag{6.4}$$

Where:

- $M_{Ed} =$ acting bending moment in [Nmm], calculated by equation (C.16)
- $A_s =$ total area of main reinforcement in [mm²], calculated by equation (C.19)
- $z =$ inner lever arm in [mm], calculated by equation (C.21)

$$\sigma'_b = \frac{2 \cdot M_{Ed}}{x \cdot b \cdot z} \quad \text{[N/mm²]} \tag{6.5}$$

Where:

- $M_{Ed} =$ acting bending moment in [Nmm], calculated by equation (C.16)
- $x =$ height of neutral axis from top of section in [mm], calculated by equation (C.20)
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $z =$ inner lever arm in [mm], calculated by equation (C.21)

$$M_{Rd} = A_s \cdot \bar{\sigma}_a \cdot z \quad \text{[Nmm]} \tag{6.6}$$

Where:

- $A_s =$ total area of main reinforcement in [mm²], calculated by equation (C.19)
- $\bar{\sigma}_a =$ allowable tensile strength of steel in [N/mm²], retrieved from table 6.4
- $z =$ inner lever arm in [mm], calculated by equation (C.21)

The bending moment capacity can be verified by comparing the induced stresses with the allowable stresses in bending. The unity checks used for this verification are provided in equation (6.7), equation (6.8) and equation (6.9). All unity checks value below 1.0 indicates that the beam has sufficient bending moment capacity. Within unity checks, the values must be expressed in the same units.

$$UC_{a,M} = \frac{\sigma_a}{\bar{\sigma}_a} \tag{6.7}$$

$$UC_{b,M} = \frac{\sigma'_b}{\bar{\sigma}'_b} \tag{6.8}$$

$$UC_{tot,M} = \frac{M_{Ed}}{M_{Rd}} \tag{6.9}$$

6.3.5. Shear capacity

The acting shear force V_{Ed} is calculated according to equation (C.17). Based on this shear force, the diagonal tensile stresses are derived following the method by Schrier [146, par. 25], as shown in equation (6.10). To simplify this calculation, z can be assumed as described below, in accordance with Schrier [146, par. 27]. This represents a conservative assumption, as the actual value of z is typically higher.

$$\sigma_b = \frac{T}{b \cdot z} \quad \text{[N/mm²]} \tag{6.10}$$

Where:

- $T = V_{Ed}$, acting shear force in [N], calculated by equation (C.17)

- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $z =$ inner lever arm in [mm], calculated by $\frac{2}{3} \cdot h_{beam}$ or $\frac{2}{3} \cdot t_{slab}$

The calculated diagonal tensile stress in the concrete σ_b is compared to the allowable tensile stress by calculation of a unity check in equation (6.11). For all combinations of the Slab System, $UC_{b,V} < 1$. Therefore, no inclined tensile reinforcement is required in the slabs. Consequently, the following formulas for the design of shear reinforcement apply only to the beams.

$$UC_{b,V} = \frac{\sigma_b}{\bar{\sigma}_b} \tag{6.11}$$

If:

- $UC_{b,V} < 1$: no inclined tensile reinforcement necessary
- $UC_{b,V} \geq 1$: diagonal stresses must be fully resisted by steel reinforcement, but may never exceed $\bar{\sigma}_{b,max}$

Shear reinforcement design

Therefore, if the first if-statement is satisfied, no shear reinforcement is required to resist the diagonal tensile stresses. Nevertheless, as stated in section 6.2.2, GBV 1962 prescribes that stirrups must always be provided in beams. However, if shear reinforcement is not required for the resistance calculation, these stirrups are not considered in the shear capacity. In such cases, the maximum allowed spacing s_{st} is used for the stirrups in the beam designs considered in this study. If the second if-statement is satisfied, shear reinforcement must be designed to fully resist the diagonal tensile stresses, as prescribed by GBV 1962 [49, Art. 44.7]. In addition, a unity check is performed to verify that the maximum allowable tensile stress in the concrete is not exceeded in the presence of shear reinforcement, as defined in equation (6.12).

$$UC_{b,max,V} = \frac{\sigma_b}{\bar{\sigma}_{b,max}} \tag{6.12}$$

If shear reinforcement is required, two types of shear reinforcement are optional according to GBV 1962 [49]: bent-up bars and stirrups. The distance from the supports over which the shear reinforcement is required is denoted as y. The formula for y is retrieved from [146, par. 27.c] and provided in equation (6.13). Over the distance y, the shear reinforcement must resist the diagonal tensile force S, which is calculated according to equation (6.14). It should be noted that the original units are used in the formulas related to the diagonal tensile force.

$$y = 0.5 \cdot L_{beam} \cdot \frac{\sigma_{b,A} - \bar{\sigma}_b}{\sigma_{b,A}} \quad \text{[m]} \tag{6.13}$$

Where:

- $L_{beam} =$ beam length in [m], defined in table 6.1
- $\sigma_{b,A} = \sigma_b$, tensile stress in concrete at supports in [N/mm²], calculated by equation (6.10)
- $\bar{\sigma}_b =$ allowable tensile strength in concrete without reinforcement in [N/mm²], retrieved from table 6.4

$$S = 35.4 \cdot (\sigma_{b,A} + \bar{\sigma}_b) \cdot b_{beam} \cdot y \quad \text{[kgf]} \tag{6.14}$$

Where:

- $\sigma_{b,A} = \sigma_b$, tensile stress in concrete at supports in [kgf/cm²], calculated by equation (6.10)
- $\bar{\sigma}_b =$ allowable tensile strength in concrete without reinforcement in [kgf/cm²], retrieved from table 6.4
- $b_{beam} =$ beam width in [cm], defined in section 6.2.2
- $y =$ distance from supports over which shear reinforcement is required [m], calculated by equation (6.13)

Bent-up bars

In the first instance, part of the main reinforcement is bent up near the ends of the beam to resist diagonal tensile stresses over the required length. Schrier states that at least one-third of the main reinforcement bars present at the location of the maximum bending moment must be bent up [146, par. 27.a]. However, the GBV 1962 state that two main reinforcement bars must always be continued as corner bars beyond the face of the support [49, Art. 34.9]. In addition, when main reinforcement bars are bent up, it must be verified that sufficient bending moment capacity remains near the supports. As this study considers only distributed loads, this reduction is not expected to be critical. Moreover, such detailed checks are considered beyond the scope of this study. Therefore, instead of bending up at least one-third of the main reinforcement bars, only the number of bars strictly required to resist the diagonal tensile stresses is bent up. This approach is also consistent with the design principle of designing governing cross-sections. The bars are typically bent up at an angle of 45 degrees. However, in exceptional cases involving short, deep beams, they may be bent at a steeper angle to improve shear resistance [146, par. 27.e]. Schrier also notes that in short beams, the use of bent-up bars is often not feasible, in which case stirrups are used to resist diagonal tensile stresses [146, par. 27.a]. This assumption is adopted for beams with lengths equal to B1, as described in table 6.1. When multiple reinforcement bars are bent up, Schrier [146, par. 27.e] advises that they should not be bent in the same plane if that plane is located far from the supports. To address this, GBV 1962 prescribes a maximum spacing between bent-up bars: $a \leq min(500, d)$ [mm] [49, Art. 34.9]. If only one bent-up bar is required, the largest allowable value for a is used in the beam designs. If multiple bent-up bars are needed, the minimum value of of a is set to 50 mm, which corresponds to the minimum of s_{st}. The diagonal tensile force resisted by the bent-up bars, S_0, is calculated using the equation provided in equation (6.15). To verify whether this resistance is sufficient, the unity check defined in equation (6.16) is performed. The unity check is satisfied if $UC_{S0,V} < 1$.

$$S_0 = A_0 \cdot \bar{\sigma}_a \quad \text{[kgf]} \tag{6.15}$$

Where:

- $A_0 =$ total area of bent-up bars in [cm²], calculated by equation (C.24)
- $\bar{\sigma}_a =$ allowable tensile strength of steel in [kgf/cm²], retrieved from table 6.4

$$UC_{S0,V} = \frac{S}{S_0} \tag{6.16}$$

Stirrups

In case $UC_{S0,V} \geq 1$, the resistance of the bent-up bars is insufficient. Schrier [146, par. 27.g] explains that additional bent-up bars can be provided, although it is often more practical to utilize the shear resistance of the stirrups. The diagonal tensile force to be resisted by the stirrups, S_r, is derived from equation (6.17). It should be noted that for beams with length B1, no bent-up bars are used. As a result, the total diagonal tensile force S is resisted entirely by the stirrups, meaning that $S_0 = 0$.

$$S_r = S - S_0 \quad \text{[kgf]} \tag{6.17}$$

Where:

- $S =$ acting diagonal tensile force over distance y in [kgf], calculated by equation (6.14)
- $S_0 =$ diagonal tensile force resisted by bent-up bars in [kgf], calculated by equation (6.15)

Based on the required diagonal tensile force to be resisted, S_r, the required area of stirrup reinforcement within distance y is derived, $A_{b,ben}$. The derivation of $A_{b,ben}$ is provided by equation (C.25). It should be noted that only vertical stirrups are considered for the beam designs of System 1. The total area of stirrup reinforcement in one plane A_b, using single stirrups, is calculated with equation (C.26). Dividing $A_{b,ben}$ and A_b results in the required number of stirrups, $n_{b,ben}$, shown in equation (C.27). The required number of stirrups is rounded up to the nearest whole number. Equation C.28 calculates how many stirrups together with s_{st} and c fit within distance y. The fitted number of stirrups n_b are compared to the rounded required number by the unity check described by equation (6.18). Important to note, the unity check is satisfied if $UC_{Sr,V} \leq 1$.

$$UC_{Sr,V} = \frac{n_{b,ben}}{n_b} \tag{6.18}$$

6.3.6. Deflection

GBV 1962 [49, Art. 45] establishes distinct minimum effective height requirements for beams and for slabs. Additionally, a shared minimum effective height criterion is prescribed, to which both elements must conform. The minimum effective height requirement for beams is given in equation (6.19), for slabs in equation (6.20), and the combined requirement applicable to both in equation (6.21). The effective height itself is calculated according to equation (C.18) and the unity check is provided by equation (6.22).

$$d_{min,beam} = \frac{\sigma_a + 15 \cdot \sigma_b'}{50000} \cdot \frac{p}{p+g} \cdot l_{min} \quad \text{[mm]} \tag{6.19}$$

Where:

- $\sigma_a =$ maximum induced tensile stress in steel [kgf/cm²], calculated by equation (6.4)
- $\sigma_b' =$ maximum induced compressive stress in concrete in [kgf/cm²], calculated by equation (6.5)
- $p = q_{k,beam}$ uniformly distributed variable load in [kgf/m], calculated by equation (C.4)
- $g = g_{k,beam}$ uniformly distributed permanent load in [kgf/m], calculated by equation (C.3)
- $l_{min} = L_{beam}$, beam length in [cm], defined in table 6.1

$$d_{min,slab} = \frac{\sigma_a}{35000} \cdot \frac{p}{p+g} \cdot l_{min} \quad \text{[mm]} \tag{6.20}$$

Where:

- $\sigma_a =$ maximum induced tensile stress in steel in [kgf/cm²], calculated by equation (6.4)
- $p = q_{k,slab}$ uniformly distributed variable load in [kgf/cm²], prescribed in section 6.3.3
- $p = g_{k,slab}$, uniformly distributed permanent load in [kgf/cm²], calculated by equation (C.2)
- $l_{min} = L_{slab}$, slab length in [cm], defined in table 6.2

$$d_{min} = \frac{1}{35} \cdot l_{min} \quad \text{[mm]} \tag{6.21}$$

Where:

- $l_{min} = L_{element}$, beam length or slab length in [cm], defined in table 6.1 and table 6.2

$$UC_d = \frac{\max(d_{min,element}; d_{min})}{d} \tag{6.22}$$

6.4. Design according to VB 1974

This section outlines the design approach and calculations used for the beams of System 1 and the slabs of System 2, in accordance with VB 1974. It begins with the general design principles and load assumptions characteristic of VB 1974. The subsequent subsections detail the calculations for bending moment capacity, shear capacity, and deflection requirements for both systems.

6.4.1. Design principles

VB 1974 introduces the concept of characteristic loads corresponding with the serviceability limit state [18, chap. 3]. Furthermore, a margin must be maintained between the serviceability limit state and the ultimate (or another) limit state. This margin is quantified by the coefficient γ, referred to as the safety factor. In the context of ultimate limit state design, this safety factor is denoted as γ_u. It is composed of multiple partial safety factors, which together form γ_u, as shown in equation (6.23). The partial factor γ_1 primarily addresses uncertainties in calculations, as well as economic, human, and psychological aspects. The factor γ_m accounts for uncertainties in construction and the level of quality control. The factor γ_s is applied when the self-weight and permanent loads have a favorable effect on the load-bearing capacity. VB 1974 acknowledges two different load cases, each associated with a different value of γ_u: the construction stage and the service stage. Since the designs of the parameter study do not include loads that have a favorable effect on structural resistance and the service stage is considered, therefore a safety factor of $\gamma_u = 1.7$ is used [18, par. 3.2.2].

$$\gamma_u \approx \gamma_1 \cdot \gamma_m \cdot \gamma_s \tag{6.23}$$

The ultimate bending capacity of the cross-section is determined by the moment just before failure, referred to as the ultimate bending moment. According to VB1974E [156, Art. 503.1], the flexural strength of a reinforced concrete cross-section is based on the following assumptions:

1. Strains in the fibres due to bending vary linearly with the distance to the neutral axis.
2. Tensile stress resulting from bending are resisted exclusively by the reinforcement, hence concrete is assumed cracked until the neutral axis.
3. Concrete stress above the neutral axis follows the simplified standard bilinear stress diagram, with the plateau at a strain of $0.25\,\%$.
4. The relationship between stress and strain in reinforcing steel is linear within the elastic range, up to the yield point.

6.4.2. Material properties

The material properties used for the design according to VB 1974 are listed in table 6.5. In contrast to GBV 1962, consistent units are used throughout VB 1974 and EC 2012.

Symbol	Value	Unit	Definition
γ_c	24	kN/m³	Volumetric weight of reinforced concrete
γ_u	1.7	-	safety factor for the ultimate limit state
f_a	400	N/mm²	design tensile strength of reinforcing steel
f'_b	18	N/mm²	design compressive strength of concrete in bending without significant normal force
f_b	1.3	N/mm²	design tensile strength of concrete

Table 6.5: Material properties for concrete class B22.5 and steel reinforcement FeB400 according to VB 1974, retrieved from VB 1974E [156, Tabel E-11], TGB 1972 [118, Tabel 1] and Boom [18, Tabel 4.1]

6.4.3. Loads

The variable and permanent loads acting on the elements are calculated according to the formulas provided in appendix C.1.2. Where, the facade load g_{fac} is taken as F1 or F2, as specified in table 6.1 and table 6.2. The variable floor load $Q_{k,slab}$ is taken as 2.0 kN/m², based on TGB 1972 [118, Tabel 3]. It should be noted that the variable loading has decreased compared to GBV 1962, due to the fact that lightweight partition walls are now classified as permanent loads rather than variable ones. Consequently, a variable floor load of 0.5 kN/m² has been reallocated to the permanent floor load, referred to as g_{lw}. This is reflected in the calculation of the permanent floor load in equation (C.7).

Moreover, the safety factor specified in table 6.5 is used in multiplication with the characteristic total load s_k to result in the design total load s_d, shown in equation (6.24) and equation (6.25).

$$s_{d,beam} = s_{k,beam} \cdot \gamma_u = (q_{k,beam} + g_{k,beam}) \cdot \gamma_u \quad \text{[kN/m]} \tag{6.24}$$

$$s_{d,slab} = s_{k,slab} \cdot \gamma_u = (q_{k,slab} + g_{k,slab}) \cdot \gamma_u \quad \text{[kN/m]} \tag{6.25}$$

Where:

- $\gamma_u = 1.7$ [-], safety factor for the ultimate limit state, retrieved from table 6.5
- $q_{k,beam} =$ variable load acting on the beam in [kN/m], calculated by equation (C.9)
- $g_{k,beam} =$ permanent load acting on the beam in [kN/m], calculated by equation (C.8)
- $q_{k,slab} =$ variable load acting on the slab in [kN/m], calculated by equation (C.10)
- $g_{k,slab} =$ permanent load acting on the slab in [kN/m], calculated by equation (C.7)

6.4.4. Bending moment capacity

The acting bending moment M_{Ed} is calculated according to equation (C.16). VB 1974 set requirements on the minimum and maximum reinforcement ratios to make sure the yield strength of steel is always reached prior to reaching the ultimate bending moment [18, par. 6.1]. Subsequently, failure occurs when the concrete in the compression zone crushes. The approved reinforcement ratio is used to calculate the ultimate bending moment M_u, which must exceed the applied bending moment M_{Ed}. This requirement is verified through the unity check defined in equation (6.29).

Minimum and maximum reinforcement ratio

The maximum reinforcement ratio $\omega_{0,max}$ is calculated according to equation (6.26), provided by Boom and Kamerling [18, Tabel 6.2]. A value of ω lower than $\omega_{0,max}$ implies that the concrete crushes in the compression zone before the steel yields.

$$\omega_{0,max} = 64.3 \cdot k_{x,max} \cdot \frac{f_b'}{f_a} \quad \text{[\%]} \tag{6.26}$$

Where:

- $k_{x,max} =$ coefficient in [-], calculated by equation (C.29)
- $f_b' =$ design compressive strength of concrete in bending without significant normal force in [N/mm²], defined in table 6.5
- $f_a =$ design tensile strength of steel in [N/mm²], defined in table 6.5

The minimum reinforcement ratio $\omega_{0,min}$ is calculated according to equation (6.27), provided by Boom and Kamerling [18, par. 6.2.3]. The minimum reinforcement ratio is applied to ensure that concrete crushing occurs before steel failure.

$$\omega_{0,min} = \frac{225}{3.5 + 10^3 \cdot \epsilon_{ar}} \cdot \frac{f_b'}{f_a} \quad \text{[\%]} \tag{6.27}$$

Where:

- $\epsilon_{ar} =$ fracture strain of steel in [-], calculated by equation (C.30)
- $f_b' =$ design compressive strength of concrete in bending without significant normal force in [N/mm²], defined in table 6.5
- $f_a =$ design tensile strength of steel in [N/mm²], defined in table 6.5

Hence, the actual reinforcement ratio ω must fall within the prescribed minimum and maximum limits. The following condition must be satisfied:

$$\omega_{0,min} \leq \omega \leq \omega_{0,max}$$

Ultimate bending moment

The ultimate bending moment M_u is calculated according to equation (6.28), provided by Boom and Kamerling [18, par. 6.2.1]. The unity check defined in equation (6.28) ensures that the design provides sufficient bending moment capacity by verifying that the ultimate bending moment exceeds the acting bending moment.

$$M_u = \omega \cdot f_a(1 - 0.55 \cdot \omega \cdot \frac{f_a}{f_b'}) \cdot b \cdot d^2 \quad \text{[Nm]} \tag{6.28}$$

Where:

- $\omega =$ steel reinforcement ratio [%], calculated by equation (C.31)
- $f_a =$ design tensile strength of steel in [N/mm²], defined in table 6.5
- $f_b' =$ design compressive strength of concrete in bending without significant normal force in [N/mm²], defined in table 6.5
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

$$UC_M = \frac{M_{Ed}}{M_u} \tag{6.29}$$

6.4.5. Shear capacity

The acting shear force V_{Ed} is calculated according to equation equation (C.17). Based on this shear force, the design shear stress τ_d is calculated using the formula provided in VB 1974E [156, Art. 504.1], as shown in equation (6.30).

$$\tau_d = \frac{T_d}{b \cdot d} \quad \text{[N/mm²]} \tag{6.30}$$

Where:

- $T_d = V_{Ed}$, acting shear force in [N], calculated by equation (C.17)
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

The calculated design shear stress τ_d is compared to the design shear stress of concrete, referred to as τ_1 by VB 1974E [156, Art. 504.2.1]. The design shear resistance is considered half of the design tensile strength of concrete according to VB 1974E [156, Art. 504.2.1]. The calculation of τ_1 is provided in equation (C.32). The unity check of the shear stress is provided in equation (6.31), the implications are explained below. For all combinations of the Slab System, $UC_{\tau,V} < 1$. Therefore, no calculated shear reinforcement is required in the slabs. Consequently, the following formulas for the design of shear reinforcement apply only to the beams.

$$UC_{\tau,V} = \frac{\tau_d}{\tau_1} \tag{6.31}$$

If:

- $UC_{\tau,V} < 1$: no calculated shear reinforcement necessary
- $UC_{\tau,V} \geq 1$: shear reinforcement required, only the difference between τ_d and τ_1 has to be taken up by shear reinforcement

Shear reinforcement design

According to VB 1974E [156, Art. 504.2.1], can the flexural compression zone of concrete also transmit shear stresses. Therefore, the difference between τ_d and τ_1 is considered the portion of shear stress that must be resisted by the shear reinforcement. The distance from the supports over which the shear reinforcement is required is denoted as y, calculated by equation (6.32).

$$y = \frac{T_d - T_1}{s_{d,beam}} \quad \text{[m]} \tag{6.32}$$

Where:

- $T_d = V_{Ed}$, acting shear force in [N], calculated by equation (C.17)
- $T_1 =$ shear force resisted by the concrete in [N], calculated by equation (C.33)
- $s_{d,beam} =$ total design load acting on the beam in [N/m], calculated by equation (6.24)

Stirrups

Boom and Kamerling [18] state that bent-up bars yield less favorable results than stirrups, partly due to poor force distribution and high stresses in the bends. Consequently, VB 1974 prescribes the use of vertical or inclined stirrups to provide shear capacity. In the beam designs of this parameter study, vertical stirrups are applied. Consequently, the angle α, defined as the angle between the shear reinforcement and the beam axis measured perpendicular to the shear force, is taken as 90 degrees. In simply supported beams subjected to uniform loading, the shear force varies linearly along the span, with maximum values occurring at the supports and decreasing towards zero at midspan. As described by Boom and Kamerling [18, par. 8.6], the shear reinforcement is densest near the supports and becomes more widely spaced along distance y. Therefore, the required amount of shear reinforcement area per mm at the supports, $\frac{A_0}{t}$, is calculated by equation (6.33), as prescribed by Boom and Kamerling [18, par. 8.6]. Together with the chosen stirrup spacing, s, st, the required area of the vertical stirrups, A_0, is derived, shown in equation (6.34). The selected stirrup area is verified against the required stirrup area in the unity check of equation (6.35). The area of single-legged stirrups, A_b, in one plane is calculated using equation (C.26).

$$\frac{A_0}{t} = \frac{T_d - T_1}{f_a \cdot z} \quad \text{[mm²/mm]} \tag{6.33}$$

Where:

- $T_d = V_{Ed}$, acting shear force in [N], calculated by equation (C.17)
- $T_1 =$ shear force resisted by the concrete in [N], calculated by equation (C.33)
- $f_a =$ design tensile strength of steel in [N/mm²], defined in table 6.5
- $z =$ internal lever arm in [mm], calculated by $0.9 \cdot d$

$$A_0 = \frac{A_0}{t} \cdot s_{st} \quad \text{[mm²]} \tag{6.34}$$

Where:

- $\frac{A_0}{t} =$ required shear reinforcement area per mm at supports in [mm²/mm], calculated by equation (6.33)
- $s_{st} =$ stirrup spacing in [mm], defined in section 6.2.2

$$UC_{st,V} = \frac{A_0}{A_b} \tag{6.35}$$

6.4.6. Deflection

The steel stress corresponding to the serviceability limit state is calculated with equation (6.36). The factor $C = 1.2$ is based on the fact that the maximum bending moment is the bending moment at midspan, as prescribed by VB 1974E [156, Art. 305.3.7].

$$\sigma_a = C \cdot \frac{f_a}{\gamma_u} \cdot \frac{A_{a,benodigd}}{A_{a,aanwezig}} \tag{6.36}$$

Where:

- $C = 1.2$ [-], coefficient prescribed in VB 1974E [156, Art. 305.3.7]
- $f_a =$ design tensile strength of steel in [N/mm²], defined in table 6.5
- $\gamma_u = 1.7$, safety factor for the ultimate limit state, retrieved from table 6.5
- $A_{a,benodigd} =$ theoretically required reinforcement area in [mm²], calculated by equation (C.34)
- $A_{a,aanwezig} = A_s$, total area of main reinforcement in [mm²], calculated by equation (C.19)

The effective height requirements prescribed by VB 1974E are based on the general criterion that the total deflection in the end stage must not exceed $0.004 \cdot L$, as stated by Boom and Kamerling [18, par. 13.4]. The effective height of the cross-section must be equal to or greater than the minimum effective heights defined in equation (6.37) and equation (6.21) for beams and in equation (6.38) and equation (6.21) for slabs. Therefore, the unity check from GBV 1962 can be applied, using the larger of the two as a reference for comparison with the designed effective height d. The unity check of the required effective height is presented in equation (6.22).

$$d_{min,beam} = \frac{\sigma_a}{5 \cdot 10^3} \cdot l_{min} \quad \text{[mm]} \tag{6.37}$$

Where:

- $\sigma_a =$ steel stress in serviceability limit state in [N/mm²], calculated by equation (6.36)
- $l_{min} = L_{beam}$, beam length in [mm], defined in table 6.1

$$d_{min,slab} = \frac{\sigma_a}{7 \cdot 10^3} \cdot l_{min} \quad \text{[mm]} \tag{6.38}$$

Where:

- $\sigma_a =$ steel stress in serviceability limit state in [N/mm²], calculated by equation (6.36)
- $l_{min} = L_{slab}$, slab length in [mm], defined in table 6.2

6.5. Assessment according to NEN 8700 series

The previous two sections, covering the design approaches according to GBV 1962 and VB 1974, described how the Beam System and Slab System were designed. This section outlines the methodology and calculations used to determine the structural resistance of the designed systems, as prescribed by EC 2012 and the NEN 8700 series. The bending moment resistance, shear resistance and deflection requirements are assessed. Excess capacities are derived from the individual resistances, with the lowest value considered the governing excess capacity of each system.

6.5.1. Design principles

The design principles of NEN 870X and EC 2012 have been discussed in detail in section 5.1. As outlined in section 5.1.4, a linear-elastic analysis is performed for both the serviceability and ultimate limit states. Sectional forces are determined through a linear calculation based on uncracked cross-sections, linear stress–strain relationships and tensile stress resulting from bending are resisted exclusively by the reinforcement. Specifically, the simplified bilinear stress–strain diagram for concrete above the neutral axis is used, with a linear elastic relationship up to a strain of $\epsilon_{c3} = 0.175$% [21]. After that, the plastic range starts with a maximum strain of $\epsilon_{cu3} = 0.35$%. This diagram is applicable up to strength class C50/60 [21]. For steel, the relationship between stress and strain is linear within the elastic range, up to the yield point. Ductile behavior is preferred, which requires that the tensile reinforcement yields prior to concrete crushing in the compression zone [21]. While ductile behavior is normally designed for in accordance with EC 2012, this parameter study focuses on existing elements. It is therefore assumed that these elements were originally designed to behave in a ductile manner. In addition, the concept of characteristic loads is more developed compared to VB 1974. This is further elaborated in section 6.5.6.

6.5.2. Material properties

Table 6.6 presents the material properties used in the assessment according to the NEN 8700 series. As explained in section 5.1.4, the assessment in accordance with NEN 8702 is based on applying the partial material factors from EC 2012, in combination with the characteristic strengths of concrete and reinforcing steel as specified in NEN 8702. The material partial factors for both the ultimate limit state and the serviceability limit state are listed in table 5.3, which shows that for the serviceability limit state, the partial factors are equal to 1.0. For the assessment of the designs according to GBV 1962, the characteristic strengths of K300 concrete and QR40 reinforcing steel are retrieved from Tables 1 and 2 of NEN 8702 [117]. For the assessment based on VB 1974, the characteristic strengths of B22.5 concrete and FeB400 steel are used. These characteristic strengths are combined with the corresponding material partial factors to derive the design strengths for the ultimate limit state, as shown in equation (6.39) for concrete and equation (6.40) for steel.

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \quad \text{[N/mm²]} \tag{6.39}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \quad \text{[N/mm²]} \tag{6.40}$$

Symbol	Value	Unit	Definition	Original strength class
γ_c	25	kN/m³	Volumetric weight of reinforced concrete	
f_{ck}	19	N/mm²	characteristic compressive strength of concrete	K300
f_{cd}	12.7	N/mm²	design compressive strength of concrete	K300
f_{ck}	18	N/mm²	characteristic compressive strength of concrete	B22.5
f_{cd}	12	N/mm²	design compressive strength of concrete	B22.5
f_{yk}	400	N/mm²	characteristic tensile strength of steel	QR40
f_{yd}	348	N/mm²	design tensile strength of steel	QR40
f_{yk}	400	N/mm²	characteristic tensile strength of steel	FeB400
f_{yd}	348	N/mm²	design tensile strength of steel	FeB400

Table 6.6: Material properties of the original material strength classes according to NEN 8702, retrieved from NEN 8702 [117, Tabel 1, Tabel 2] and NEN 1991 [110, Tabel A1]

6.5.3. Bending moment resistance

The bending moment resistance of the designed elements is calculated in this section. As ductile behavior is assumed for all elements, the steel stress for the calculation of the bending moment resistance is taken equal to f_{yd}. As explained in section 6.5.1, the simplified bilinear stress-strain diagram is applied for the concrete. From this diagram, the compressive force in the concrete and its point of application can be determined, located at $\frac{7}{18} \cdot x$ from the top of the section [21]. The height of the neutral axis x is calculated by equating the concrete compressive force to the tensile force in the reinforcement, as shown in equation equation (C.36). This value is then used to determine the internal lever arm z, as defined in equation (C.37). Finally, the bending moment resistance M_{Rd} is derived using the internal lever arm and the steel tensile force, as presented in equation (6.41). Based on the bending moment resistance, the corresponding uniformly distributed design load $q_{M,Rd}$ is calculated using equation (6.42).

$$M_{Rd} = N_s \cdot z \quad \text{[kNm]} \tag{6.41}$$

Where:

- $N_s =$ design tensile force in the steel in [kN], calculated by equation (C.35)
- $z =$ internal lever arm in [m], calculated by equation (C.37)

$$q_{M,Rd} = \frac{M_{Rd}}{\frac{1}{8} \cdot L^2} \quad \text{[kN/m]} \tag{6.42}$$

Where:

- $M_{Rd} =$ bending moment resistance in [kNm], calculated by equation (6.41)
- $L = L_{beam}$ or L_{slab}, beam or slab length in [m], defined in table 6.1 and table 6.2

6.5.4. Shear resistance

As discussed in section 6.3.5 and section 6.4.5, different components contribute to the shear capacity, such as concrete ($V_{Rd,c}$), stirrups ($V_{Rd,st}$) and bent-up bars ($V_{Rd,bent}$). All relevant components will be addressed in this section. The formula for the total shear resistance of the element is provided in equation (6.43). However, as specified in the aforementioned sections, not all components are always present or allowed to be included in the total shear resistance. In such cases, the corresponding terms in equation (6.43) are set to zero. Further explanation is provided in the following sections. Based on the total shear resistance, the corresponding uniforly distributed design load $q_{V,Rd}$ is calculated using equation (6.44).

$$V_{Rd} = V_{Rd,c} + V_{Rd,st} + V_{Rd,bent} \quad \text{[N]} \tag{6.43}$$

$$q_{V,Rd} = \frac{V_{Rd}}{0.5 \cdot L} \quad \text{[kN/m]} \tag{6.44}$$

Where:

- $V_{Rd} =$ total shear resistance in [kN], calculated by equation (6.43)
- $L = L_{beam}$ or L_{slab}, beam or slab length in [m], defined in table 6.1 and table 6.2

Shear resistance of concrete

The shear capacity of all slabs was sufficient without shear reinforcement. Therefore, for slabs, only $V_{Rd,c}$ is applicable for V_{Rd}. The shear resistance of concrete without shear reinforcement, $V_{Rd,c}$, is calculated using equation (C.38). In accordance with NEN-EN 1992 [113, par 6.2.2], $V_{Rd,c}$ must not be smaller than the minimum shear resistance without shear reinforcement, calculated using equation (C.39). As a result, the applied concrete shear resistance is taken as the greater of the two:

$$V_{Rd,c} = \max\left(V_{Rd,c};\ V_{Rd,c,min}\right)$$

For beam designs according to GBV 1962, the tensile resistance of concrete is deemed sufficient if $UC_{b,V} < 1$. Subsequently $V_{Rd,c}$ is applicable for V_{Rd}. If this condition is not met, i.e. if $UC_{b,V} \geq 1$, the shear resistance of the concrete is not allowed to contribute to the total shear resistance, consequently $V_{Rd,c} = 0$ in equation (6.43). According to VB 1974, the flexural compression zone of the concrete can also transmit shear stresses. Therefore, the concrete shear resistance $V_{Rd,c}$ is always included in the total shear capacity for beam designs corresponding to VB 1974.

Shear resistance of bent-up bars

Bent-up bars were only included in the beam designs according to GBV 1962. Since NEN 8702 and NEN-EN 1992 do not specify the shear resistance of bent-up bars, the shear resistance is estimated using the formula for inclined stirrups, which function in a similar way. All bent-up bars are assumed to be bent in a 45 degree angle. Therefore, the angle α, defined as the angle between the shear reinforcement and the beam axis measured perpendicular to the shear force, is taken as 45 degrees. In addition, the angle between the concrete compression strut and the axis of the beam perpendicular to the shear force, θ, is also considered 45 degrees. The shear resistance of the bent-up bars is taken as the smaller value of $V_{Rd,bent}$ and $V_{Rd,bent,max}$:

$$V_{Rd,bent} = \min\left(V_{Rd,bent};\ V_{Rd,bent,max}\right)$$

The formulas used to calculate $V_{Rd,bent}$ and $V_{Rd,bent,max}$ are based on the expressions for inclined stirrups as defined in NEN-EN 1992 [113, par. 6.2.3] and are given in equation (C.43) and equation (C.44).

Shear resistance of stirrups

As discussed in section 6.3.5 and section 6.4.5, stirrups are always added to beam designs. Although, they were not always included in the original shear capacity, they are included in all shear resistances of the beam designs. Only vertical stirrups are used for the beam designs, consequently $\alpha = 90$. Similar as for the shear resistance of the bent-up bars, the shear resistance of the stirrups is taken as the smaller value of $V_{Rd,st}$ and $V_{Rd,st,max}$:

$$V_{Rd,st} = \min\left(V_{Rd,st};\ V_{Rd,st,max}\right)$$

The formulas used to calculate $V_{Rd,st}$ and $V_{Rd,st,max}$ are retrieved from NEN-EN 1992 [113, par. 6.2.3] and are provided in equation (C.45) and equation (C.46).

6.5.5. Deflection

The deflection is considered part of the serviceability limit state (SLS). As explained in section 5.1.2, even in case of renovations the legislator only imposes requirements on the ultimate limit states. In addition, the verification of the serviceability limit state is taking into account quite some details, especially short term and long term behavior of construction materials [109, App. A.1.4.3]. In other words, the full verification for deflection as provided in NEN 1990 is not suited for existing buildings constructed between 1960 and 1990. However, from a private law perspective it may still be useful to apply a deflection verification, as stated by NEN 8700 [115, par. 3.4]. For this reason, a conservative approach is used that uses one third of the E modulus from the ultimate limit state derived with short term loading tests, E_{cm}, to calculate the deflection w [21]. The formula to calculate E_{cm} is retrieved from NEN 1992 [113, Table 3.1] and used to calculate E_{equi} in equation (C.47). For maximum deflection, the requirement related to the appearance of the structure is used, prescribed in NEN 1990NB [109, App. A.1.4.3]. The formula used to calculate the maximum allowed deflection is provided in equation (C.48). The maximum deflection and the assumed E modulus are combined in equation (6.45) to calculate the uniformly distributed design load $q_{w,Rd}$.

$$q_{w,Rd} = \frac{E_{equi} \cdot I \cdot w_{lim}}{\frac{5}{384} \cdot L^4} \quad \text{[N/mm]} \tag{6.45}$$

Where:

- E_{equi} = assumed modulus of elasticity in [N/mm²], calculated by equation (C.47)
- $I = \frac{b \cdot h^3}{12}$, moment of inertia in [mm^4]
- w_{lim} = maximum allowed deflection, calculated by equation (C.48)
- $L = L_{beam}$ or L_{slab}, beam or slab length in [m], defined in table 6.1 and table 6.2

6.5.6. Loads

The variable and permanent loads acting on the elements, either slab or beam, are calculated according to the formulas provided in appendix C.1.3. The general variable floor load of 2.5 kN/m² is increased by an allowance of 1.2 kN/m² for lightweight partition walls. This corresponds to the heaviest type of lightweight partition wall specified in NEN 1991 [110, par. 6.3.1.2], with a weight of the partition walls ranging between 2 and 3 kN/m. This conservative assumption results in a higher applied load, which in turn reduces the calculated excess capacity, thereby ensuring that a governing case is considered in the assessment. As a result, the applied variable floor load $Q_{k,slab}$ is taken as 3.7 kN/m². The facade load g_{fac} is taken as F1 or F2, as specified in table 6.1 and table 6.2. The characteristic permanent load acting on the beam, $g_{k,beam}$, is calculated using equation (C.13), while the characteristic permanent load acting on the slab (per meter width), $g_{k,slab}$, is calculated with equation (C.12). The characteristic variable load acting on the beam, $q_{k,beam}$, is determined using equation (C.14), and the characteristic variable line load acting on the slab (per meter width),$q_{k,slab}$, is given by equation (C.15).

6.6. Derivation of excess capacity factors

The previous section outlined the calculations used to determine the maximum uniformly distributed load corresponding to individual resistances, such as bending moment resistance, shear resistance, and deflection. Based on these values, in combination with the load combination formulas corresponding to each limit state and the permanent and variable loads acting on an element according to EC 2012, the governing excess capacity of a specific element according to a specific code is derived, denoted as $C_{F,element,code}$. This calculated excess capacity is then compared with both the total characteristic load according to EC 2012 and the total characteristic load according to the earlier code, to derive the excess capacity factor for that element and code. Two variants of the excess capacity factor are considered.

6.6.1. Derivation of excess capacity

Ultimate limit state

The maxim design load of the ultimate limit state (ULS) that can be applied at an element is defined as $q_{Rd,ULS}$. The value of $q_{Rd,ULS}$ is equal to the smaller of the uniformly distributed design loads resulting from the two individual resistance calculations of the ultimate limit state: $q_{M,Rd}$ and $q_{w,Rd}$. Therefore, $q_{Rd,ULS}$ is derived as follows:

$$q_{Rd,ULS} = \min\left(q_{M,Rd};\ q_{V,Rd}\right) \quad \text{[kN/m]} \tag{6.46}$$

As discussed in section 5.1.2, the NEN 8700 states that the formulas from NEN 1990 must be used for the determination of loads for the verification of the ultimate limit states, except that different partial load factors are defined in NEN 8700. These formulas, as specified in equation (5.1) and equation (5.2), are combined with the psi factor ψ_0 for offices and the partial factors from NEN 8700 to derive formulas to calculate the excess capacity in an element, as shown in equation (6.47) and equation (6.48). Whereas equation (6.47) is based on fundamental load combination 6.10a and equation (6.48) on combination 6.10b, both originate from NEN 1990 [108, par. 6.4.3.2].

$$C_{F,element,code;a} = \frac{q_{Rd,ULS,element} - \gamma_{G;a} \cdot g_{k,element,EC2012} - \gamma_{Q;a} \cdot \psi_0 \cdot q_{k,element,EC2012}}{\gamma_{G;a}} \quad \text{[kN/m]} \tag{6.47}$$

$$C_{F,element,code;b} = \frac{q_{Rd,ULS,element,code} - \gamma_{G;b} \cdot \xi_j \cdot g_{k,element,EC2012} - \gamma_{Q;b} \cdot q_{k,element,EC2012}}{\gamma_{G;b}} \quad \text{[kN/m]} \tag{6.48}$$

Where:

- $q_{Rd,ULS,element,code} =$ maximum design load acting on an element in ULS according to GBV1962 or VB1974 in [kN/m], calculated by equation (6.45)
- $\gamma_{G;a} = 1.3$ [-], partial load factor, defined in table 5.2
- $g_{k,element,EC2012} = g_{k,beam}$ or $g_{k,slab}$, permanent characteristic load acting on element according to EC 2012 in [kN/m], calculated by equation (C.13) or equation (C.12)

- $\gamma_{Q;a} = 1.3$ [-], partial load factor, defined in table 5.2
- $\psi_0 = 0.5$ [-], combination factor, defined in table 5.1
- $q_{k,element,EC2012} = q_{k,beam}$ or $q_{k,slab}$, variable characteristic load acting on element according to EC 2012 in [kN/m], calculated by equation (C.14) or equation (C.15)
- $\gamma_{G;b} \cdot \xi_j = 1.15$ [-], partial load factor, defined in table 5.2
- $\gamma_{Q;b} = 1.3$ [-], partial load factor, defined in table 5.2

Serviceability limit state

The design load in the serviceability limit state acting on an element is denoted by $q_{w,Rd}$. As outlined in section 5.1.2, three serviceability limit states are defined, of which the quasi-permanent combination is applied in this analysis. This combination is typically used for evaluating long-term effects and the appearance of the structure [108, par. 6.5.3]. The quasi-permanent load combination is given in equation (5.5). This expression forms the basis for the derivation of the excess capacity under the serviceability limit state, as shown in equation (6.49).

$$C_{F,element,code;SLS} = q_{w,Rd} - g_{k,element} - \psi_2 \cdot q_{k,element} \quad \text{[kN/m]} \tag{6.49}$$

Where:

- $q_{w,Rd} =$ maximum design load acting on an element in SLS according to GBV1962 or VB1974 in [kN/m], calculated by equation (6.45)
- $g_{k,element} = g_{k,beam}$ or $g_{k,slab}$, permanent characteristic load acting on element according to EC 2012 in [kN/m], calculated by equation (C.13) or equation (C.12)
- $\psi_2 = 0.3$ [-], combination factor, defined in table 5.1
- $q_{k,element} = q_{k,beam}$ or $q_{k,slab}$, variable characteristic load acting on element according to EC 2012 in [kN/m], calculated by equation (C.14) or equation (C.15)

Governing excess capacity

The governing excess capacity is the lowest value derived from the assessed limit states and is denoted by $C_{F,element,code}$ for a specific element and a specific code. Therefore, $C_{F,element,code}$ is derived as follows:

$$C_{F,element,code} = \min\left(C_{F,element,code;a};\ C_{F,element,code;b};\ C_{F,element,code;SLS}\right) \quad \text{[kN/m]} \tag{6.50}$$

6.6.2. Derivation of excess capacity factor

As proposed in section 5.2.2, excess capacity is assumed to represent an allowable increase relative to the original load. Therefore, it should be related to the original load. However, if the original load of the same structure is deemed larger now, this difference takes part of the total excess capacity. Subsequently, this difference is also included in the formula for the excess capacity factor, provided in equation (6.51). If the difference between the original characteristic load and the current characteristic load, $\Delta s_{k,code}$, is entered in the formula, a more easily interpretable formula is derived, given in equation (6.53). Both formulas consist of two ratios. The left-hand ratio, $C_{F,element,code}/s_{k,element,code}$, reflects the excess capacity and will be referred to as the capacity ratio. The right-hand ratio, $s_{k,element,EC2012}/s_{k,element,code}$, will be referred to as the load ratio. The detailed derivation of equation (6.53) is provided in appendix C.5.

$$C_{x,element,code} = \frac{C_{F,element,code} + \Delta s_{k,code}}{s_{k,element,code}} \quad \text{[-]} \tag{6.51}$$

$$\Delta s_{k,code} = s_{k,element,EC2012} - s_{k,element,code} \quad \text{[kN/m]} \tag{6.52}$$

$$C_{x,element,code} = \frac{C_{F,element,code} + s_{k,element,EC2012}}{s_{k,element,code}} - 1 \quad \text{[-]} \tag{6.53}$$

6.7. Results

This section presents the results of the parameter study, focusing on the structural capacity of beam and slab systems designed according to GBV 1962 and VB 1974. The main outcomes include excess capacities (C_F), characteristic loads (s_k) and derived excess capacity factors (C_x and C_{sf}), based on structural assessment in accordance with the NEN 8700 series. The results are structured by system type and design code, with distributions, trends and outliers discussed in relation to governing failure modes. Graphical representations are used to highlight key findings, while full design data and calculation results are provided in the appendices. Where relevant, alternative formulations of capacity factors are introduced to enable comparison with established values, such as $C_{x,com}$ and $C_{x,ten}$.

6.7.1. Design context

The structural combinations analyzed in this chapter were designed according to the procedures of GBV 1962 and VB 1974, as outlined in section 6.3 and section 6.4. The full set of parameter combinations for System 1 and System 2 are listed in appendix D.1, while the corresponding design properties, such as member dimensions and reinforcement diameters, are provided in appendix D.2. These design properties result from an iterative process driven by optimal material utilization, implying that unity checks should approach their governing limits, as explained in section 6.2.2. The identification of the governing excess capacity was also integrated into this iterative approach. Since the iterative process was carried out manually, the full design space was not exhaustively explored as would be possible with automated methods. The complete sets of outcomes, including excess capacities, characteristic loads and excess capacity factors, is presented in appendix D.3.

Note that if the governing excess capacity $C_{F,element,code}$ was found to be negative, it was set to zero for the purposes of analysis. This decision is based on the reasoning that excess capacity, by definition, cannot be negative: either a structural element provides additional capacity beyond the design demand, or it does not. In the latter case, the excess capacity is considered to be zero. Accordingly, the corresponding excess capacity factor $C_{x,element,code}$ was also set to zero in accordance with equation (6.51). This approach has been applied consistently throughout the results presented in appendix D.3 and the analyses that follow.

It should be noted that the deflection criteria assumed in section 6.5.5 resulted in no excess capacity for VB 1974 slab designs with slab lengths of 7.2 or 9 meters. For GBV 1962, only one slab design with a length of 7.2 meters showed no excess capacity, which may be attributed to the stricter deflection requirements prescribed by that code. For all other slab and beam designs, deflection was not governing. As the deflection criteria are based on a conservative approach and given that NEN 8700 imposes no formal requirements on the serviceability limit state, the deflection checks according to NEN 870X are disregarded in this parameter study. This applies to both slabs and beams to ensure a consistent analytical approach. Nevertheless, caution is advised when interpreting the excess capacity of slabs with large spans as deflection may still be a critical factor in practice.

6.7.2. Excess capacity distributions

To explore the relationship between the original total characteristic load and the excess capacity, figure 6.3 and figure 6.4 present $s_{k,element,code}$ versus $C_{F,element,code}$ for both systems and both design codes. As the relation appears to be linear in both graphs, linear regression lines and Pearson correlation coefficients are included to quantify the association. While this initial overview provides insight into the general spread and trends of the data, certain combinations of the beams appear to deviate significantly from the main distribution. A Pearson coefficient of 1 can be described as a perfect positive linear relationship, while a Pearson coefficient of 0 means the parameters are uncorrelated. Therefore the Pearson coefficient provides a measure of strength of linear association between two variables [147]. The formula to calculate the Pearson coefficient is provided in equation (D.1). It should be noted that Pearson coefficients are only suited for datasets without outliers [144]. A formal identification and filtering of these outliers is carried out in section 6.7.7. The cleaned datasets of beams are shown in figure 6.5. For the slabs, an outlier was identified only in the VB 1974 dataset. The corresponding cleaned dataset is shown in figure 6.6. The cleaned datasets indeed display stronger linear relationships, as indicated by higher Pearson correlation coefficients. Unless stated otherwise, the cleaned datasets were used for all subsequent analyses involving excess capacity and excess capacity factors, as the excess capacity factors are calculated using equation (6.53). This was done to avoid further distortion of trends in the results. This does not apply to analyses that are based solely on facade weight data, where no filtering was applied. Furthermore, figures 6.5 and 6.4 show that elements designed according to GBV 1962 generally exhibit higher excess capacity than those designed according to VB 1974.

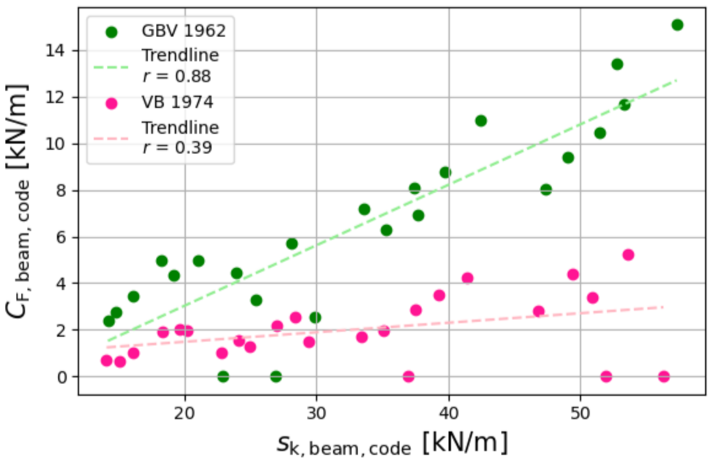


Figure 6.3: Relation between total original characteristic load $s_{k,beam,code}$ and excess capacity $C_{F,beam,code}$ for System 1 with outliers (unclean dataset)

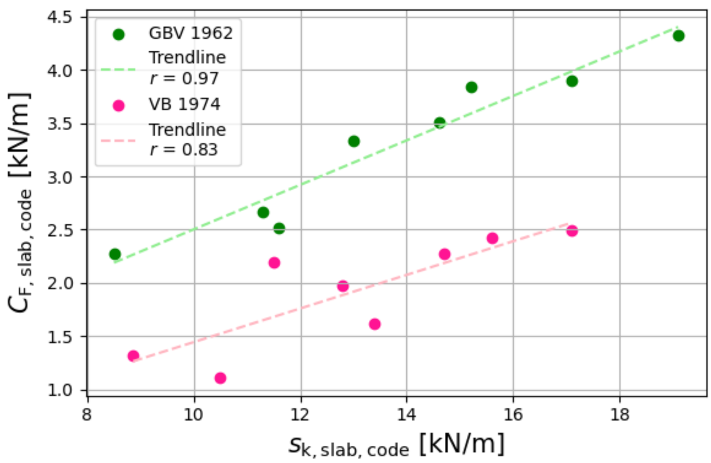


Figure 6.4: Relation between total original characteristic load $s_{k,slab,code}$ and excess capacity $C_{F,slab,code}$ for System 2 with outliers (unclean dataset)

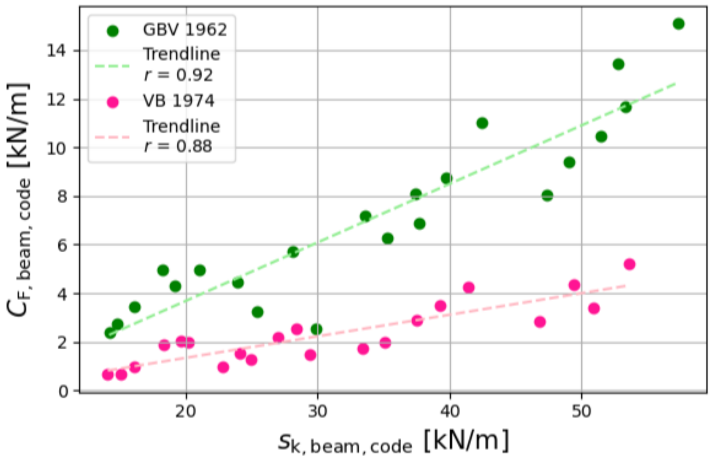


Figure 6.5: Relation between total original characteristic load $s_{k,beam,code}$ and excess capacity $C_{F,beam,code}$ for System 1 without outliers (clean data set)

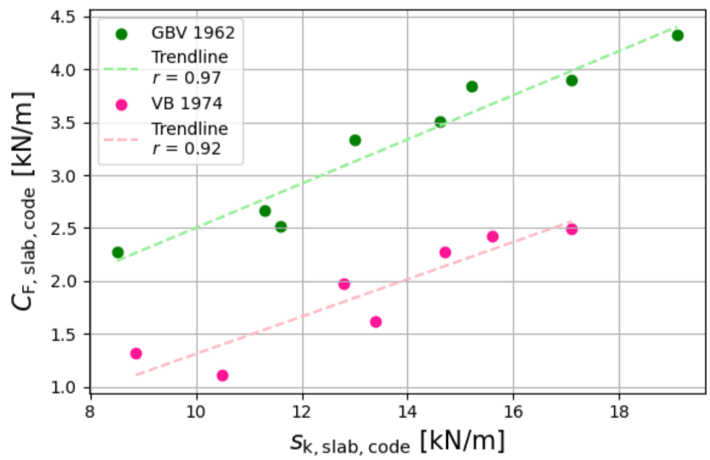


Figure 6.6: Relation between total original characteristic load $s_{k,slab,code}$ and excess capacity $C_{F,slab,code}$ for System 2 without outliers (clean dataset)

The relation between $s_{k,element,EC2012}$ and $s_{k,element,code}$ is also part of the derivation of excess capacity factor $C_{x,element,code}$, as shown in equation (6.53). Therefore their relation is plotted for both systems and design codes in figures 6.7, 6.8, 6.9 and 6.10. The linear regression lines have been plotted in the graphs as well, along with the Pearson correlation coefficient r, which equals 1 for all graphs, indicating perfect positive linear relationships between the parameters on the x- and y-axes. Additionally, a 1:1 line has been included, where $s_{k,beam,VB1974} = s_{k,beam,EC2012}$. The graphs show that all plotted values lie above the 1:1 lines. Furthermore, for higher values of $s_{k,beam,code}$, the distance between the data points and the 1:1 line increases, whereas this behavior is not observed for the slabs.

6.7.3. Beam parameters

The influence of individual beam parameters, such as the weight of the facade and the span of the slab, on both the excess capacity and the characteristic load acting on the beams is analyzed individually. Special attention is given to the excess capacity and the original characteristic load, as these parameters are used in the equation for calculating the excess capacity factor, which is presented in equation (6.51).

Box-plots of $C_{F,beam,code}$ have been created for the varying facade load, F1 and F2, as shown in Figures 6.11 and 6.12. The vertical axes in both figures, representing $C_{F,beam,GBV1962}$ and $C_{F,beam,VB1974}$ respectively, have been set to the same scale to allow for easier comparison. Both figures demon-

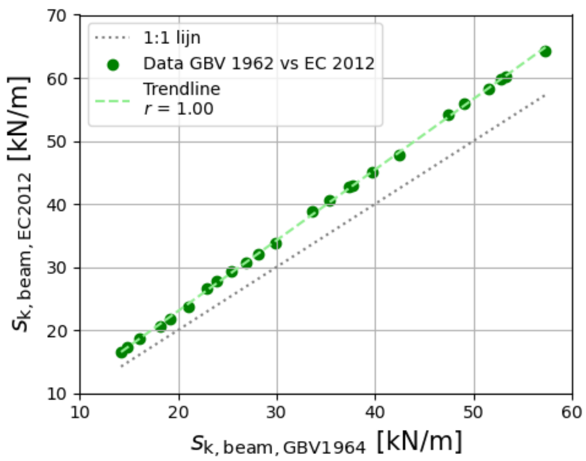


Figure 6.7: Relation between the characteristic load according to EC 2012 $s_{k,beam,EC2012}$ and according to GBV 1962 $s_{k,beam,GBV1962}$ (unclean dataset)

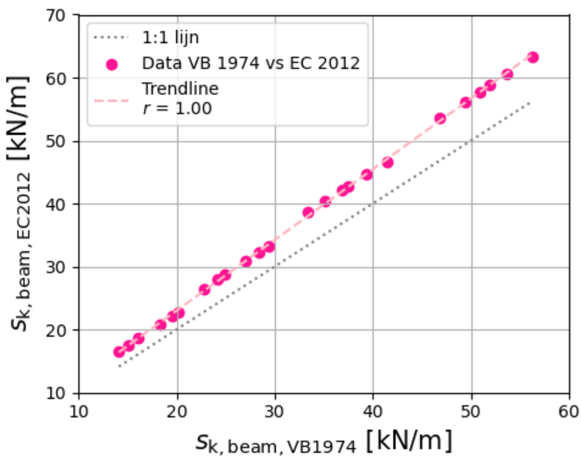


Figure 6.8: Relation between the characteristic load according to EC 2012 $s_{k,beam,EC2012}$ and according to VB 1974 $s_{k,beam,GBV1974}$ (unclean dataset)

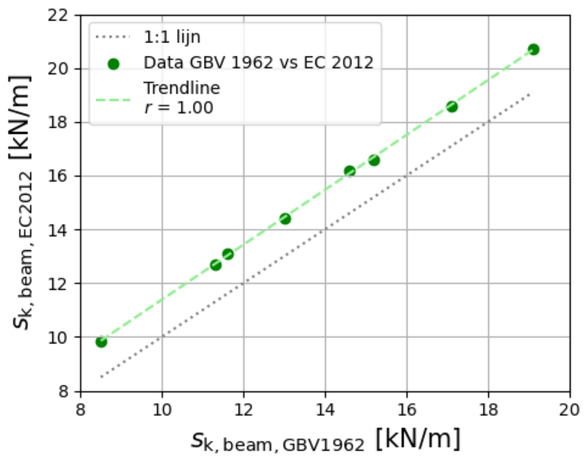


Figure 6.9: Relation between the characteristic load according to EC 2012 $s_{k,slab,EC2012}$ and according to GBV 1962 $s_{k,slab,GBV1962}$ (unclean dataset)

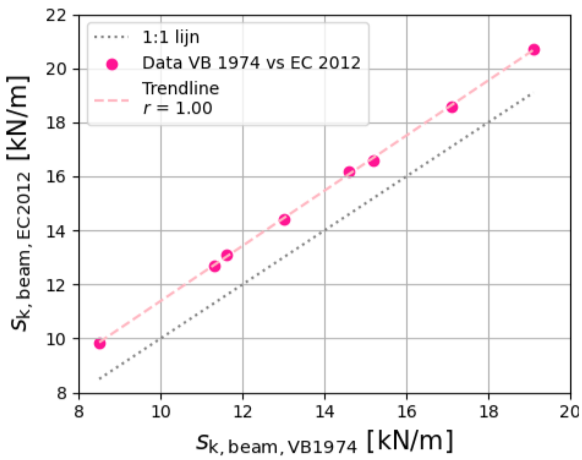


Figure 6.10: Relation between the characteristic load according to EC 2012 $s_{k,slab,EC2012}$ and according to VB 1974 $s_{k,slab,GBV1974}$ (unclean dataset)

strate a general increase in $C_{F,beam,code}$ as the facade load g_{fac} increases. However, the distribution of the values for the beams from GBV 1962 is noticeably more dispersed. The central box in each plot illustrates the interquartile range, which spans from the first quartile (Q_1) to the third quartile (Q_3), and therefore includes the middle fifty percent of the data [149]. The horizontal line within the box indicates the median value, also referred to as the second quartile, which represents the midpoint of the ordered dataset. The whiskers extend to data points that fall within 1.5 times the interquartile range from the quartiles. Values outside this range are classified as statistical outliers and are plotted as individual points. This approach has been adopted for the detection of outliers, provided in appendix D.4.2.

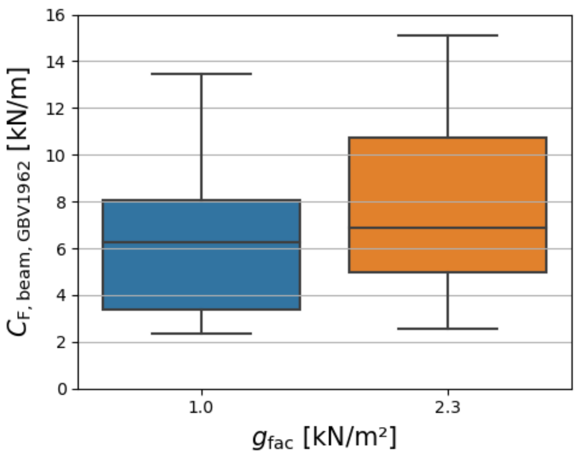


Figure 6.11: Box-plot of $C_{F,beam,GBV1962}$ (clean dataset) for varying facade load g_{fac}

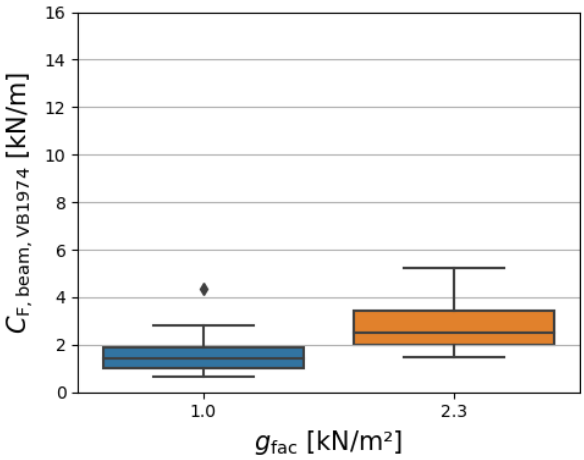


Figure 6.12: Box-plot of $C_{F,beam,VB1974}$ (clean dataset) for varying facade load g_{fac}

To explore the relationship between the excess capacity and slab length, box-plots have been created from the cleaned datasets, shown in figures 6.13 and 6.14. The relationship between the parameters appears to be quadratic. To investigate this further, the values are plotted along with quadratic regression lines in Figures 6.15 and 6.16. The coefficient of determination, R^2, is calculated here, as it provides information about the goodness of fit of a model. In this case, the model refers to the quadratic regression line. The R^2 value is a statistical measure of how well the regression line approximates the actual data [106]. The formula for R^2 is given in equation (D.2). As the graphs show, the variation is quite high, since only one regression line is used to represent the trend across all data. Taking this into account, the quadratic regression line follows the data reasonably well. Since excess capacity is considered an increase on the original characteristic load, the original characteristic load, $s_{k,beam,code}$, has been plotted against the slab length, L_{slab}, as shown in figures 6.17 and 6.18. Again, a quadratic regression line and the corresponding coefficient of determination have been calculated. The R^2 values in figures 6.17 and 6.18 indicate that almost all variation in $s_{k,beam,code}$ is explained by L_{slab}.

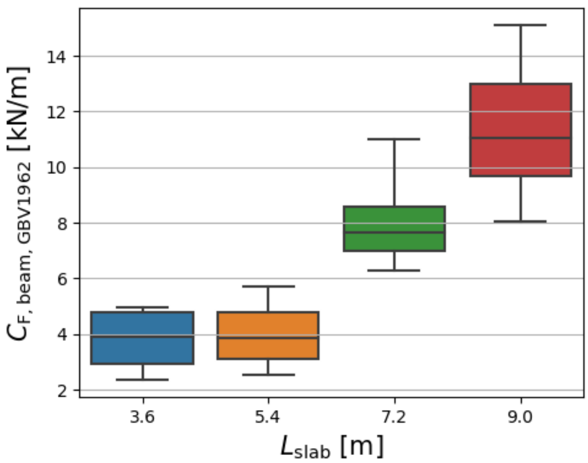


Figure 6.13: Box-plot of $C_{F,beam,GBV1962}$ (clean dataset) for varying slab length L_{slab}

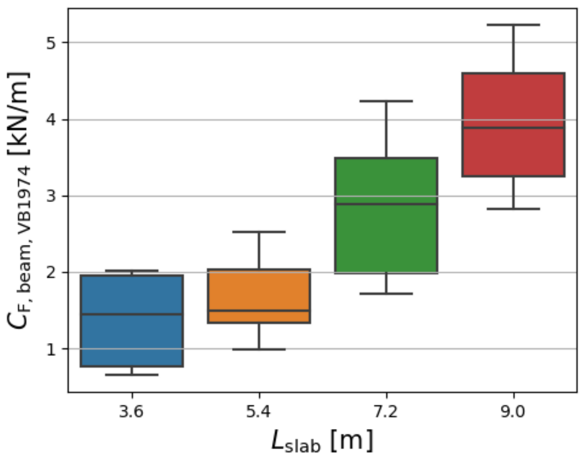


Figure 6.14: Box-plot of $C_{F,beam,VB1974}$ (clean dataset) for varying slab length L_{slab}

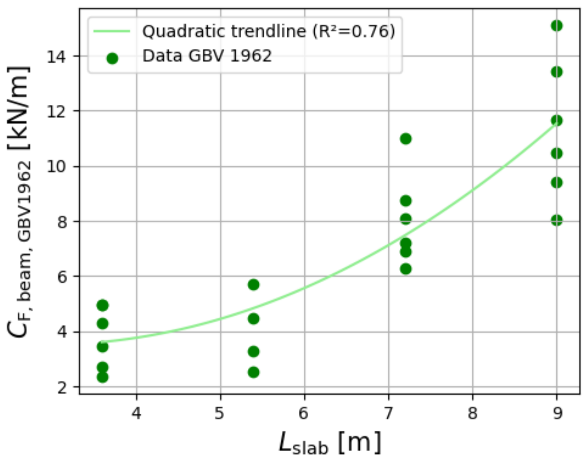


Figure 6.15: Scatterplot of the excess capacity of beams $C_{F,beam,GBV1962}$ (clean dataset) vs. slab length L_{slab}

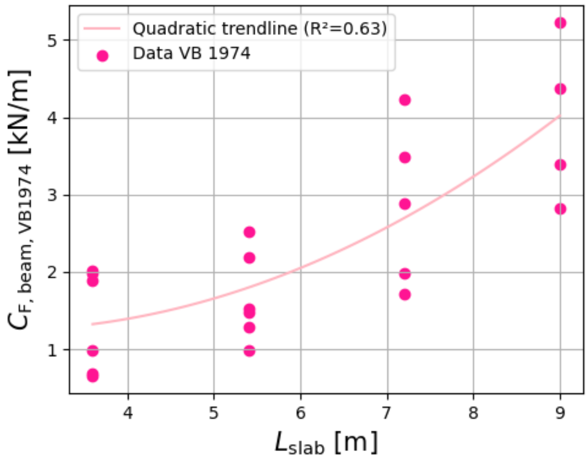


Figure 6.16: Scatterplot of the excess capacity of beams $C_{F,beam,VB1974}$ (clean dataset) vs. slab length L_{slab}

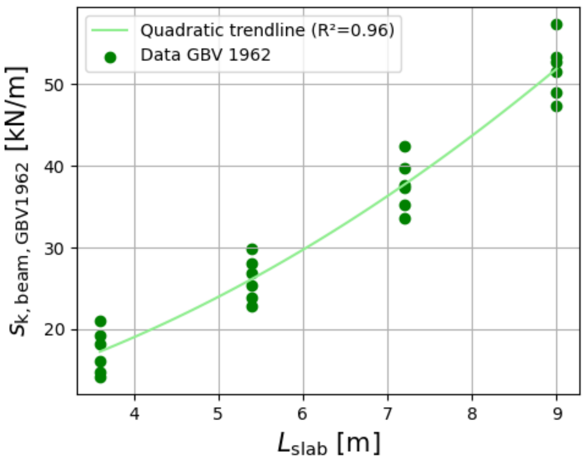


Figure 6.17: Scatterplot of the characteristic load of beams $C_{F,beam,GBV1962}$ (unclean dataset) vs. slab length L_{slab}

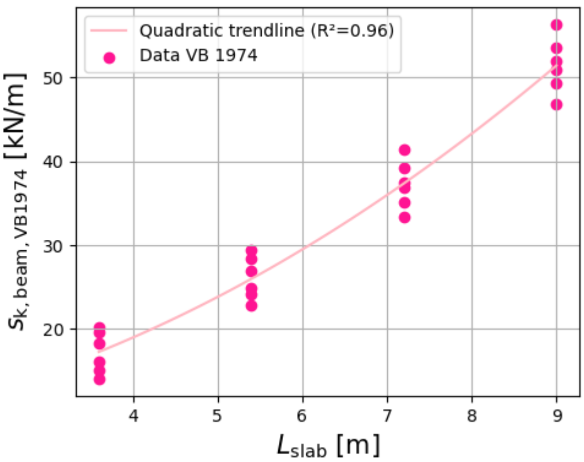


Figure 6.18: Scatterplot of the characteristic load of beams $C_{F,beam,VB1974}$ (unclean dataset) vs. slab length L_{slab}

Besides the slab length, System 1 also includes the beam length as a parameter. The relationship between the excess capacity and the beam length of System 1 is explored by creating box-plots of $C_{F,slab,code}$ and L_{beam}, shown in figures 6.19 and 6.20. The box-plot of the GBV 1962 data shows that the variation increases with beam length, while the median remains approximately constant. Similar behavior is observed in the box-plot of the VB 1974 data, although the box-plot corresponding to a beam length of 7.2 m appears irregular. This may be attributed to the fact that the cleaned dataset was used for this box-plot, in which three data points corresponding to this beam length were removed, as explained in section 6.7.7.

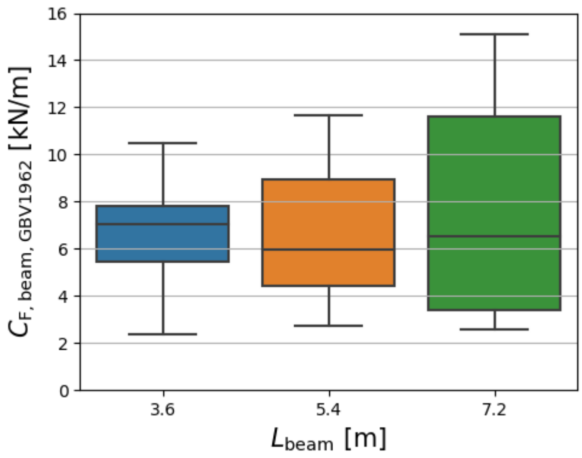


Figure 6.19: Box-plot of $C_{F,beam,GBV1962}$ (clean dataset) for varying beam length L_{beam}

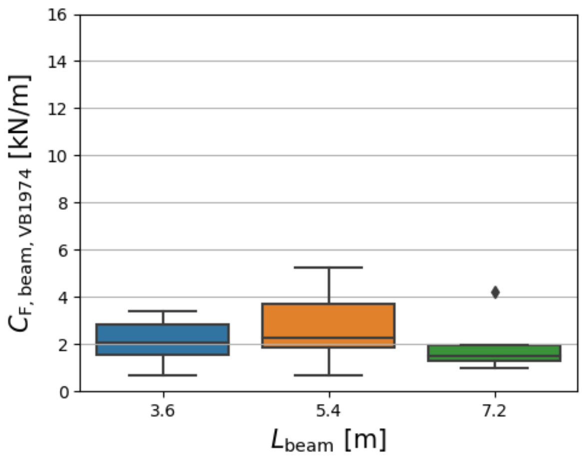


Figure 6.20: Box-plot of $C_{F,beam,VB1974}$ (clean dataset) for varying beam length L_{beam}

6.7.4. Slab parameters

Similar to the analysis for beams, the relevant slab parameters are examined in relation to the excess capacity of the slab, starting with the weight of the facade. Box-plots of $C_{F,slab,code}$ are created for the varying facade loads, F1 and F2, as shown in figures 6.21 and 6.22. An explanation of box-plots is provided in section 6.7.3. Once again, an increase in $C_{F,slab,code}$ is observed as the facade load g_{fac} increases.

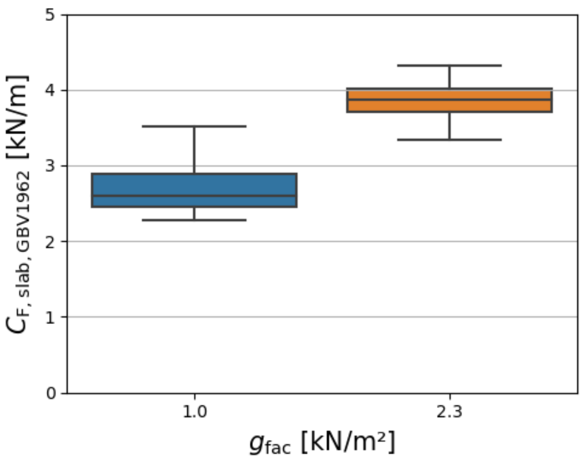


Figure 6.21: Box-plot of $C_{F,slab,GBV1962}$ for varying facade load g_{fac}

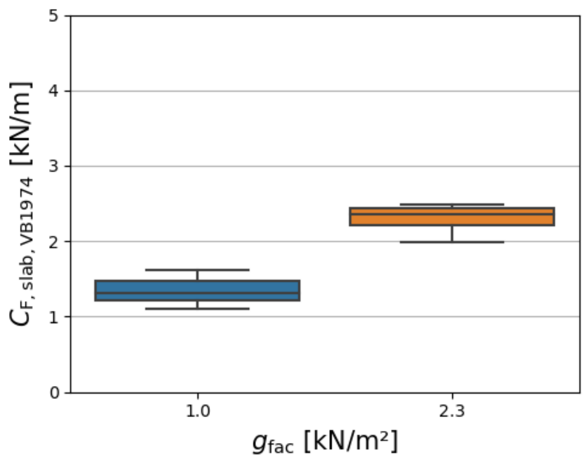


Figure 6.22: Box-plot of $C_{F,slab,VB1974}$ for varying facade load g_{fac} (clean dataset)

The relationship between the excess capacity and the slab length of System 1 is explored by creating scatterplots of $C_{F,slab,code}$ versus L_{slab}, as shown in figures 6.23 and 6.24. These graphs indicate that the slab span of System 2 does not have a significant influence on the excess capacity, although a slight increase can be noticed for increasing slab length. To investigate this further, the original characteristic load is plotted against the slab length in figures 6.25 and 6.26, which reveals a linear relationship for both GBV 1962 and VB 1974.

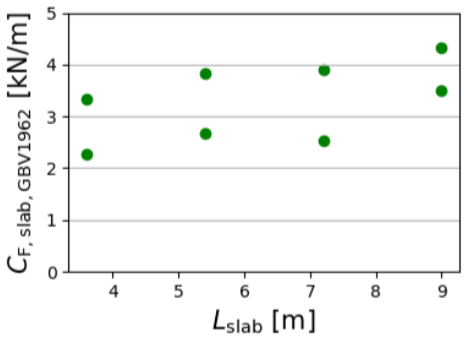


Figure 6.23: Scatterplot of the excess capacity of slabs $C_{F,slab,GBV1962}$ vs. slab length L_{slab}

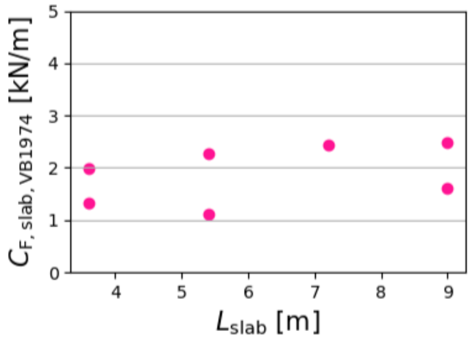


Figure 6.24: Scatterplot of the excess capacity of slabs $C_{F,slab,VB1974}$ vs. slab length L_{slab} (clean dataset)

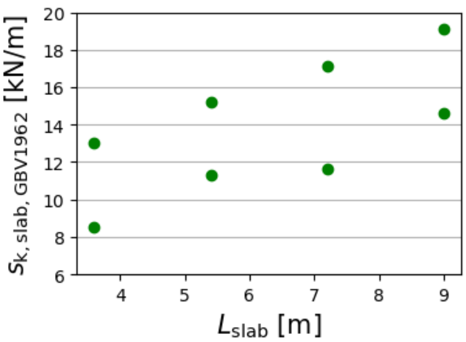


Figure 6.25: Scatterplot of the characteristic load of slabs $C_{k,slab,GBV1962}$ vs. slab length L_{slab}

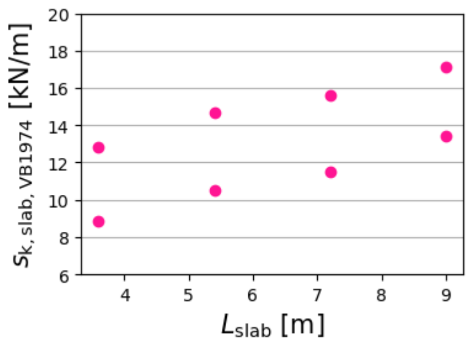


Figure 6.26: Scatterplot of the characteristic load of slabs $C_{k,slab,VB1974}$ vs. slab length L_{slab} (unclean dataset)

6.7.5. Excess capacity factors

The relationship between the excess capacity factor, $C_{x,element,code}$, and the original characteristic load, $s_{k,element,code}$, is examined for both code years and both structural elements. The corresponding graphs are shown in figures 6.27 and 6.28. Linear regression lines have been added to indicate the overall trends. For the beams, the excess capacity factors appear to remain approximately constant, as indicated by the nearly horizontal regression lines. In contrast, the excess capacity factors of the slabs for both codes show a slight decline as the characteristic load increases.

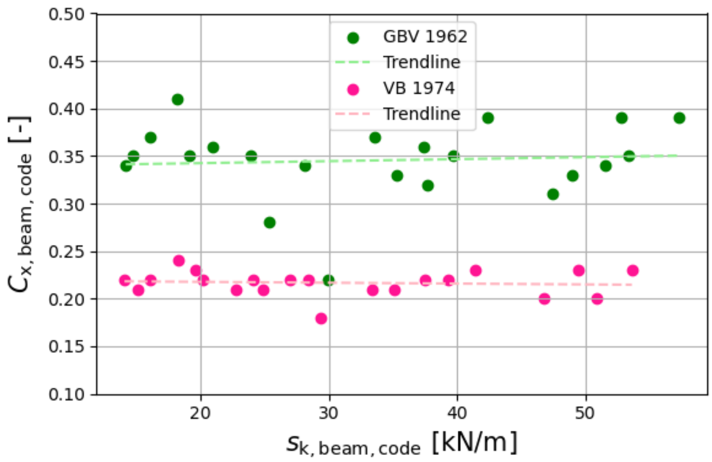


Figure 6.27: Scatterplot of the excess capacity factors of beams $C_{x,beam,code}$ vs. original characteristic load $s_{k,beam,code}$ (clean dataset)

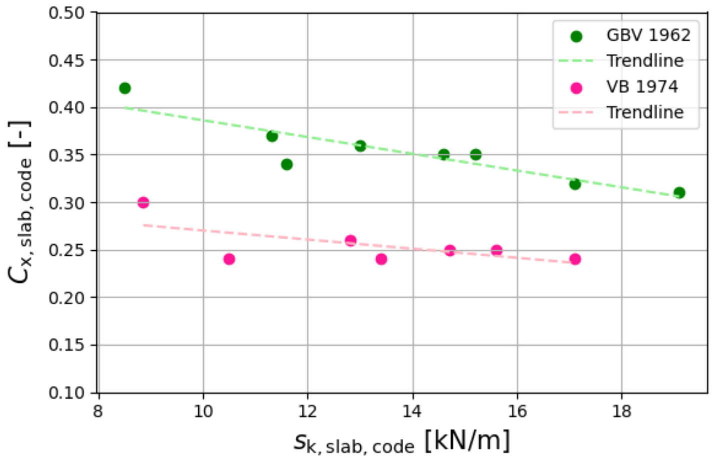


Figure 6.28: Scatterplot of the excess capacity factors of slabs $C_{x,slab,code}$ vs. original characteristic load $s_{k,slab,code}$ (clean dataset)

To further assess the distribution of these values, box-plots of $C_{x,element,code}$ are presented in figures 6.29 and 6.30. The median excess capacity factors of both beams and slabs designed according to GBV 1962 are equal. While there is a small difference between the medians of the VB 1974 designs, the variability in these designs is lower than that of the GBV 1962 designs. All box-plots display at least one outlier. However, it should be noted that the box-plots are based on cleaned datasets, meaning that any visible outliers have either not been statistically or visually identified or were deliberately retained. The outlier in the VB 1974 slab design can be attributed to combination 1 of System 2, which is discussed and deliberately retained in section 6.7.7. The outlier of the GBV 1962 beam designs can be attributed to combination 20 and will be discussed in section 6.8.1.

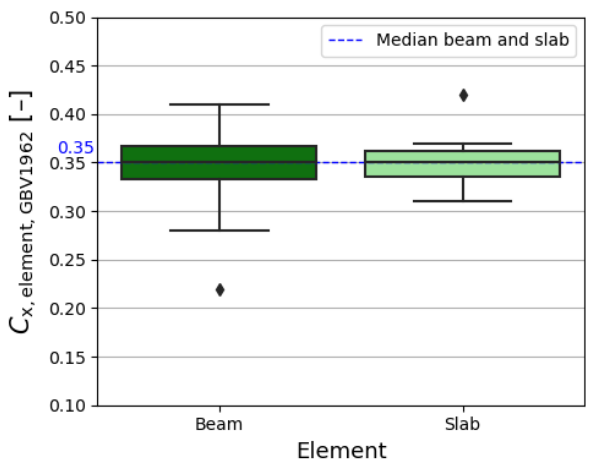


Figure 6.29: Box-plot of the excess capacity factors $C_{x,element,GBV1962}$ for each element according to GBV 1962 (clean datasets)

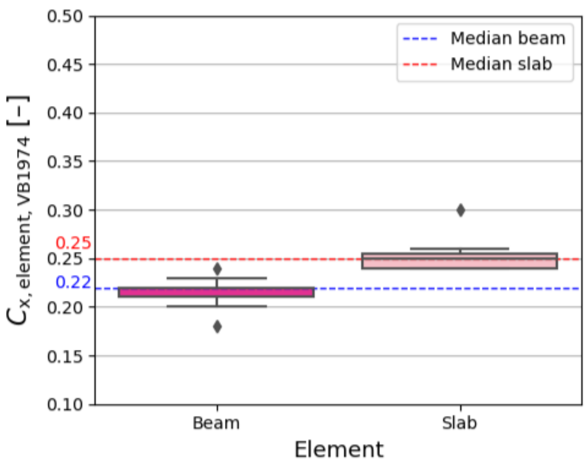


Figure 6.30: Box-plot of the excess capacity factors $C_{x,element,VB1974}$ for each element according to VB 1974 (clean datasets)

Since the box-plots of the beams in both figures show limited skewness, low variation, and a visible mean, histograms of the beam excess capacity factors have been created in figures 6.31 and 6.32 to compare their distribution shapes to a normal distribution. As the graphs show, the histograms of the beam excess capacity factors resemble normal distributions.

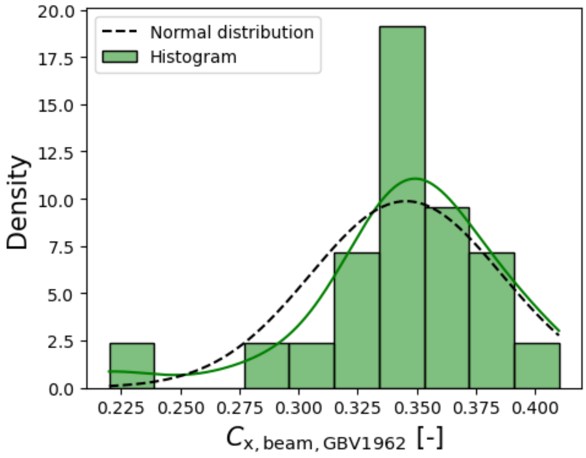


Figure 6.31: Histogram of the excess capacity factors of beams $C_{x,beam,GBV1962}$ with kernel density estimates and normal distribution (clean dataset)

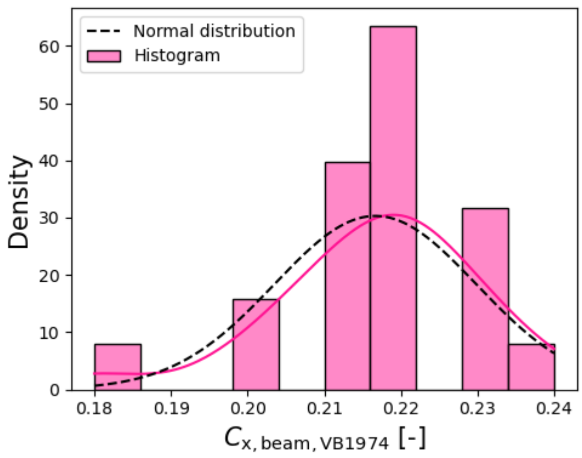


Figure 6.32: Histogram of the excess capacity factors of beams $C_{x,beam,VB1974}$ with kernel density estimates and normal distribution (clean dataset)

6.7.6. Creation of model data

The second hypothesis states that the excess capacity should be constant for structural elements of the same type designed according to the same structural design code. This means that the excess capacity factors calculated with equation (6.53) for the same element under the same code should be constant. To test the validity of this assumption, model data is derived in this section to compare with the data from the parameter study. Excess capacity factors must be selected for each element according to each code. For this analysis, the medians from the box-plots shown in figures 6.29 and 6.30 are used. These excess capacity factors are combined with the uncleaned datasets of $s_{k,element,code}$ and $s_{k,element,EC2012}$ in equation (6.53) to calculate new excess capacities $C_{F,element,code}$. The relationships in the model data between the excess capacity of the elements and the original characteristic load are visualized by scatterplots in figures 6.33 and 6.34. Trendlines and Pearson correlation coefficients have also been calculated, which demonstrate that the relationships in the model data are almost perfectly linear.

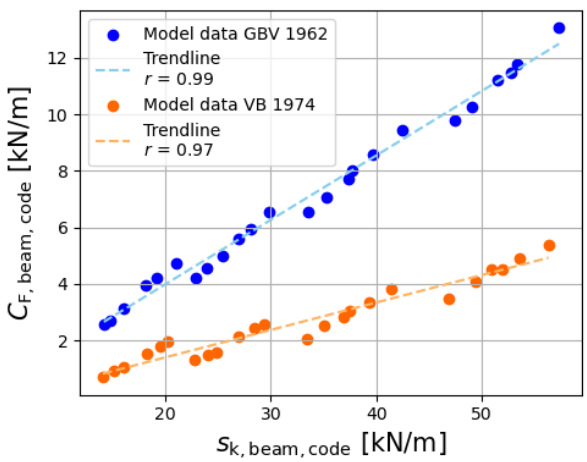


Figure 6.33: Model data showing the relation between the excess capacity $C_{F,beam,code}$ and the original characteristic load $s_{k,beam,code}$

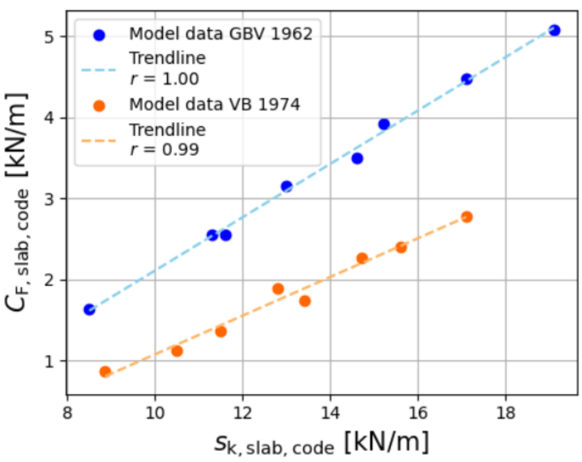


Figure 6.34: Model data showing the relation between the excess capacity $C_{F,slab,code}$ and the original characteristic load $s_{k,slab,code}$

The model data are compared by the data from the parameter study by plotting them in one figure, to check whether the model data approaches the data of the parameter study well enough. These graphs are provided in figures 6.35 and 6.36. The model data fits the cleaned parameter study data quite well. However, for all trendlines, it can be noticed that for higher excess capacities, the model data creates higher excess capacities than the parameter data. For the beam data of GBV 1962 50% of parameter study data falls above the trendline of the model data and for the beam data of VB 1974 this number is 42.9%. For the slab data of GBV 1962 50% of parameter study data falls above the trendline of the model data and for the slab data of VB 1974 this number is 57.1%.

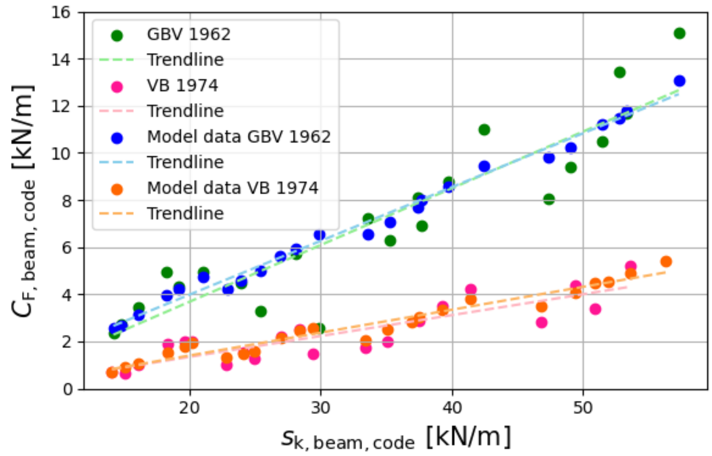


Figure 6.35: Model data and parameter study data showing the excess capacity $C_{F,beam,code}$ and the original characteristic load $s_{k,beam,code}$ (clean parameter study dataset)

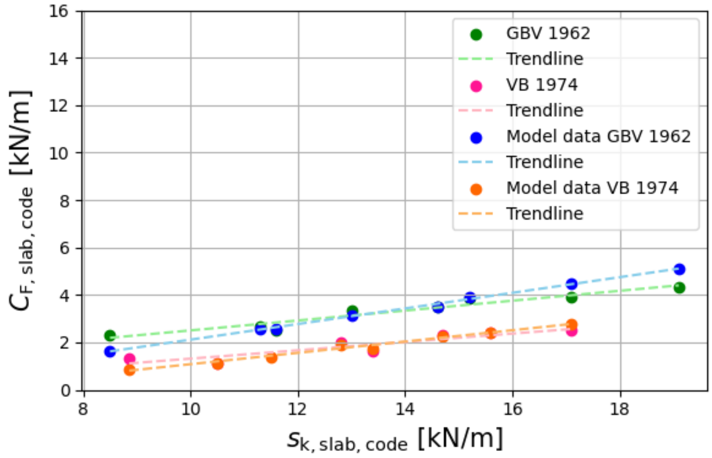


Figure 6.36: Model data and parameter study data showing the excess capacity $C_{F,slab,code}$ and the original characteristic load $s_{k,slab,code}$ (clean parameter study dataset)

6.7.7. Outlier identification and filtering

Outliers are analyzed by visual observations and a univariate statistical analysis on the residuals. An outlier has been defined by Hawkins [57] as “An observation which deviates so much from other observations as to arouse suspicions that it was generated by a different mechanism”. Outliers can mislead the analysis of data, but can also tell relevant information, so they mustn’t be removed without analysis.

Visual observation of figure 6.3 and figure 6.4 and equation (6.51) make the 0 values of $C_{F,beam,code}$ stand out, as the formula is based on the assumption that the presence of $s_{k,element,code}$ should result in excess capacity $C_{F,element,code}$. Moreover, this turns out to be true for the other values. The outliers identified from visual observations have been highlighted in blue for GBV 1962 and in orange for VB 1974 in figure 6.37. No outliers were identified in the slab dataset based on visual observation, as none of the values appeared to deviate significantly from the overall trend. The origin of all identified outliers will be examined in the following subsections to justify whether they should be removed from the dataset to not skew the analysis.

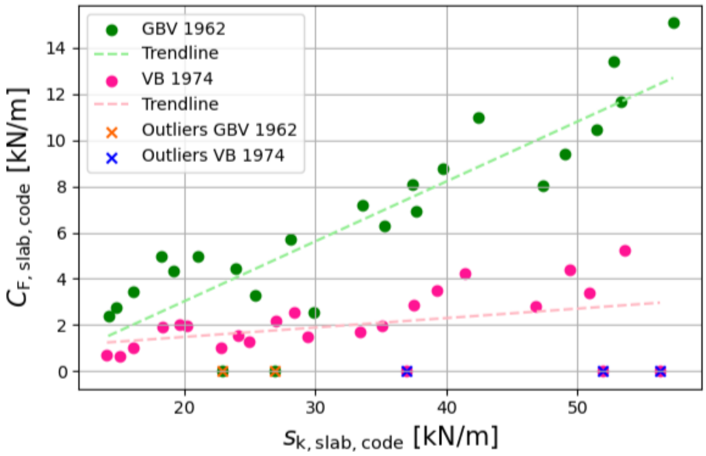


Figure 6.37: Relation between total original characteristic load $s_{k,beam,code}$ and excess capacity $C_{F,beam,code}$ with highlighted outliers based on visual observations

The univariate statistical analysis is performed on the residuals of $C_{F,element,code}$ after removing visually identified outliers. The residuals are defined as being the vertical distance between the linear regression line, which is the expected model, and the values of $C_{F,element,code}$. Equation (D.3) calculates the residuals. Two univariate statistical methods are considered for the analysis: the standard deviation rule or IQR rule [40]. The standard deviation rule can only be applied to data having a normal distribution or approximately symmetrical distribution. Figures 6.38, 6.39, 6.40 and 6.41 show the histograms of the residuals together with kernel density estimates that approximate their probability density functions. Figure 6.38 shows some resemblance to a normal distribution, although it is skewed. Conversely, figures 6.39, 6.40 and 6.41 deviate more strongly and do not appear to follow a normal distribution. Therefore the IQR rule is used as this rule is more robust to skewed data and will calculate separate upper and lower boundaries. The values of the data set must be ordered from smallest to biggest in order to apply the IQR rule [149]. The Inter Quartile Range (IQR) of the residuals is derived and a common value of 1.5 times the IQR is used to calculate the lower and upper bound of the accepted values [149]. These upper and lower bounds have been listed in table D.13, their derivation is provided in appendix D.4.2. No additional outliers were identified through the statistical analysis for beams, but one statistical outlier was found in the slab dataset, which is highlighted in blue in figure 6.42. This outlier is examined in more detail later in this subsection, to assess whether it should be removed in order to avoid skewing the analysis.

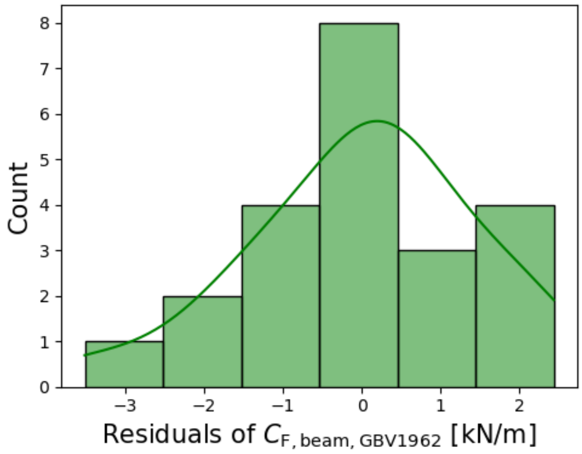


Figure 6.38: Histogram of residuals of $C_{F,beam,GBV1962}$ with kernel density estimates

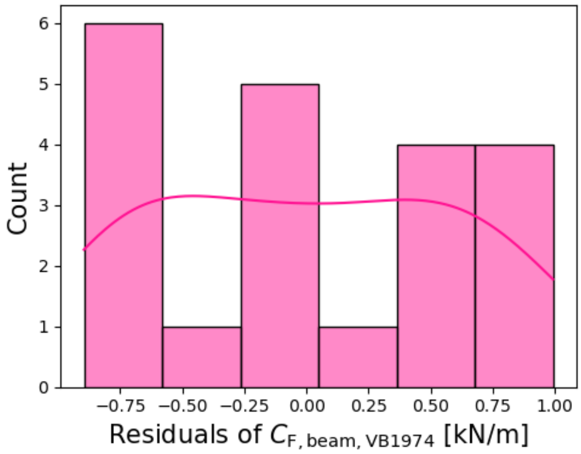


Figure 6.39: Histogram of residuals of $C_{F,beam,VB1974}$ with kernel density estimates

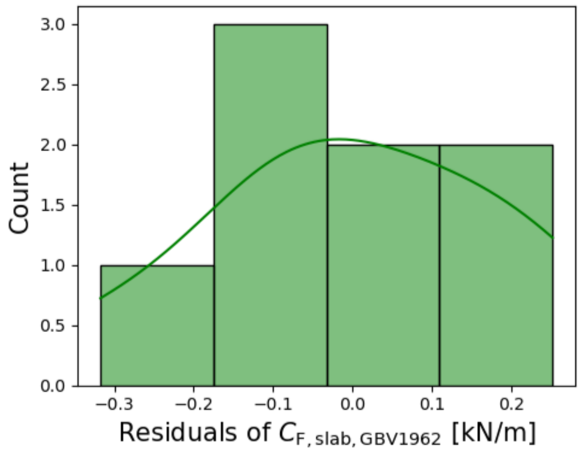


Figure 6.40: Histogram of residuals of $C_{F,slab,GBV1962}$ with kernel density estimates

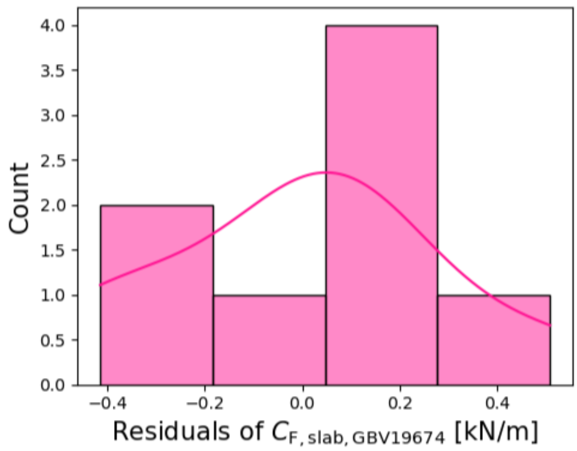


Figure 6.41: Histogram of residuals of $C_{F,slab,VB1974}$ with kernel density estimates

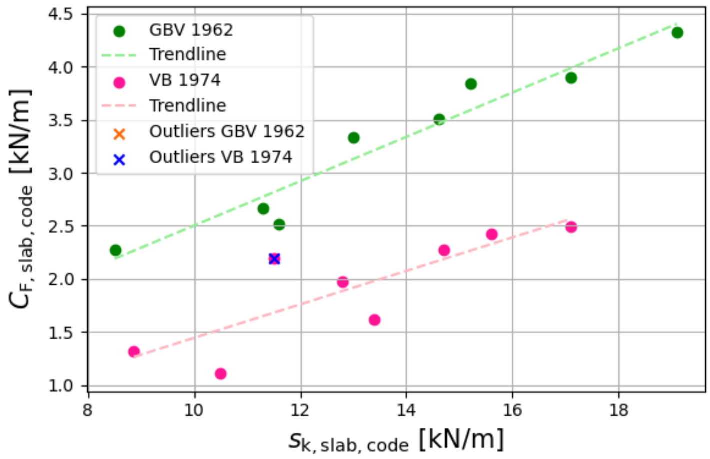


Figure 6.42: Relation between total original characteristic load $s_{k,slab,code}$ and excess capacity $C_{F,slab,code}$ with highlighted outliers based on statistical analysis

Outlier evaluation: Beams GBV 1962 combinations 3 and 4

For combination 3, the excess capacity drops to zero because, according to the calculations, the shear resistance is insufficient. More specifically, the design shear resistance $q_{V,Rd}$ governs the maximum design load of the ultimate limit state $q_{Rd,ULS}$, which causes $C_{F,element,code}$ to fall below zero. This outcome results from the strict application of the design methodology in the parameter study, where a uniform calculation approach was followed. The acting diagonal stress in the concrete slightly exceeds the allowable tensile stress, which causes the unity check of equation (6.11) to exceed 1. As a consequence, all diagonal stresses must be fully resisted by the steel reinforcement. Following the NEN 870X series, this also implies that the shear resistance of the concrete must be excluded from the total shear capacity.

Combination 3 involves a beam with a length of $B1 = 3.6$ m. According to the assumption made in section 6.3.5, this short span means that no bent-up bars are applied, leaving only vertical stirrups in equation (6.43) to resist the shear force. However, a difference exists between the shear forces that must be resisted under GBV 1962 and those under NEN 8700. Under GBV 1962, the diagonal tensile force S that must be resisted by the stirrups is only acting over a distance y, which is approximately 30 mm in this case. The distance y is relatively small due to the combination of a low diagonal tensile stress in the concrete near the supports and the short beam length, both of which directly influence its magnitude in the governing equation. As a result, the diagonal tensile force S is relatively small and the minimum stirrup reinforcement suffices. In contrast, the NEN 8700 series requires the entire shear force to be resisted by vertical stirrups alone. In this case, the stirrups turn out to be insufficient.

In conclusion, while GBV 1962 states that for $UC_{b,V} \geq 1$ the steel must fully resist the diagonal tensile stresses, this resistance only applies over the limited region defined by y. In practice, the concrete still contributes to shear resistance in other parts of the beam. This contribution is excluded under the strict NEN 8700 interpretation. It is expected that including the concrete shear contribution would have led to a positive excess capacity. Therefore, this outlier is justifiably excluded from further analysis, as the shear resistance values compared are not based on fully equivalent assumptions.

The excess capacity of combination 4 also drops to zero, for the same underlying reason as combination 3: vertical stirrups are the only shear-resisting component considered in equation (6.43). This is directly related to the fact that combinations 1 to 8 are based on beam length $B1$, for which bent-up bars are excluded according to the assumptions in section 6.3.5. The remaining combinations with $B1$ in GBV 1962 do not exhibit the same problem. This is explained below to substantiate that combinations 3 and 4 are indeed valid outliers that may be excluded from the analysis.

For combinations 1 and 2, the design shear force is low enough that $UC_{b,V} < 1$. As a result, shear reinforcement is not required according to GBV 1962. However, vertical stirrups are still placed and thus can be included in the shear resistance according to equation (6.43), resulting in a positive excess capacity. In combinations 5 through 8, the acting shear force leads to diagonal stresses in the concrete that are sufficiently high to increase the value of y, and thus the diagonal tensile force S, to a level where the exclusion of the concrete shear resistance from V_{Rd} becomes a valid assumption again. In these cases, the vertical stirrups are adequate to provide the required shear capacity, and the excess capacity remains positive.

Outlier evaluation: Beams VB 1974 combinations 21, 23 and 24

The excess capacity of combinations 21, 23 and 24, based on the designs of VB 1974, also drops below zero due to insufficient shear resistance as determined according to the NEN 8700 series. Similar to combinations 3 and 4 of GBV 1962, the insufficient shear resistance in combinations 21, 23 and 24 of VB 1974 results from the strict and uniform application of the design and assessment methodology throughout the parameter study. Three typical shear design situations can be distinguished in VB 1974, corresponding to increasing levels of shear force acting on beams. Each situation affects both the original design and the assessment differently. These situations are outlined below to clarify why combinations 21, 23 and 24 represent specific cases that justify exclusion from further analysis.

The first situation occurs when the diagonal tensile stress in the concrete, τ_d, is low enough to be fully resisted by the design shear stress resistance of the concrete, τ_1, such that $UC_{\tau,V} < 1$. In this case, no shear reinforcement is strictly required, and stirrups with a maximum spacing of 300 mm are used. These stirrups, however, are not included in the calculated shear capacity under VB 1974. The design shear stress resistance of concrete is generally lower according to EC 2012 than in VB 1974. For example, in combination 23, $\tau_{d,VB1974} = 0.65$ N/mm² while $\tau_{d,EC2012} = 0.40$ N/mm². Although this is a substantial difference, the stirrups are included in the shear resistance according to EC 2012. As a result, combinations in the first situation still show positive excess capacity.

The second situation arises when the diagonal tensile stress slightly exceeds the concrete shear resistance, meaning $UC_{\tau,V} \geq 1$, and shear reinforcement is required. The distance y is then calculated to determine the diagonal tensile force S that must be resisted by the stirrups. In this case, most of the shear force is still carried by the concrete, so only a limited amount of shear reinforcement is needed. Because stirrup spacing is adjusted in steps of 50 mm, the unity check for the required stirrup area, as given in equation (6.35), often remains relatively low. Although the shear stress resistance of concrete is again lower under NEN 870X than under VB 1974, the presence of over-dimensioned reinforcement provides additional capacity, resulting in positive excess capacity for beams falling into this category.

The third situation involves a further increase in shear force. A significant portion of the total shear force must now be resisted by the stirrups. The unity check for the stirrup area in equation (6.35) is relatively high in this case, meaning the stirrup area is closely matched to the design demand. Furthermore, the shear capacity of stirrups is lower under NEN 870X than under VB 1974. While this difference is less pronounced than for concrete, it is still relevant. For instance, in combination 23 the stirrup shear stress resistance is 0.51 N/mm² in VB 1974 but only 0.44 N/mm² under NEN 870X. These differences combined result in a shear resistance that is insufficient to meet the acting shear demand under NEN 8700, which explains the absence of excess capacity.

It is important to note that the parameter study designs are based on actual historical guidelines, but they have also been optimized to result in governing material use and capacity. Therefore, not all VB 1974 beams with high shear forces exhibit this behavior. The outcome for combinations 21, 23 and 24 is the result of a concurrence of factors. These combinations are excluded from the data analysis in order to avoid distortion of the results. Nonetheless, this observation highlights the importance of caution when considering the application of additional loads to VB 1974 beams that already carry high shear forces.

Outlier evaluation: Slabs VB 1974 combination 5

As the slab dataset per code year is relatively small, a single outlier can significantly skew the analysis. Combination 5 is identified as a statistical outlier in figure 6.42, as its residual is substantially higher than that of the other data points, according to the IQR rule. This indicates that the excess capacity is larger than expected for the corresponding characteristic load. This is also reflected in the excess capacity factor of combination 5, which is the highest in the VB 1974 slab dataset, as shown in figure 6.43. The excess capacity of combination 5 is 2.28 kN/m, with a corresponding excess capacity factor of 0.32. Another high excess capacity factor of 0.30 is observed in the same graph and corresponds to combination 1 of VB 1974 designs. Consistently, combination 1 also shows a relative high excess capacity of 1.32 kN/m, although this is less apparent in figure 6.42, as it is the first value displayed.

What stands out when analyzing these combinations is that their unity checks for bending moment capacity are significantly lower than those of the other data points. For combination 1, the unity check is 0.94, and for combination 5 it is 0.93, whereas most other values are close to 1. This strongly correlates with the excess capacity factors: the lower the unity check for bending moment capacity, the higher the excess capacity factor and vice versa. The unity checks for both bending moment and deflection are provided in table D.14, while the excess capacity factors are listed in table D.12. It is important to note that for each design, the governing combination is selected based on the highest unity checks. In most slab combinations, deflection requirements influenced the design, but for combinations 1 and 5, deflection was slightly governing. This slight over-dimensioning of the bending moment capacity can lead to significantly higher excess capacity factors, especially in small datasets such as this parameter study. However, this over-dimensioning results from an idealized optimization based on both unity checks. In reality, particularly in 1974, it is unlikely that such strict optimization was consistently applied. If bending moment capacity had been governing, these combinations would likely have followed the general trend of constant excess capacity factors, as seen in the other data points. To avoid skewing the analysis of excess capacity factors toward higher values that are unlikely to occur in practice, while also avoiding unnecessary reduction of the dataset, only combination 5 is excluded from further analysis.

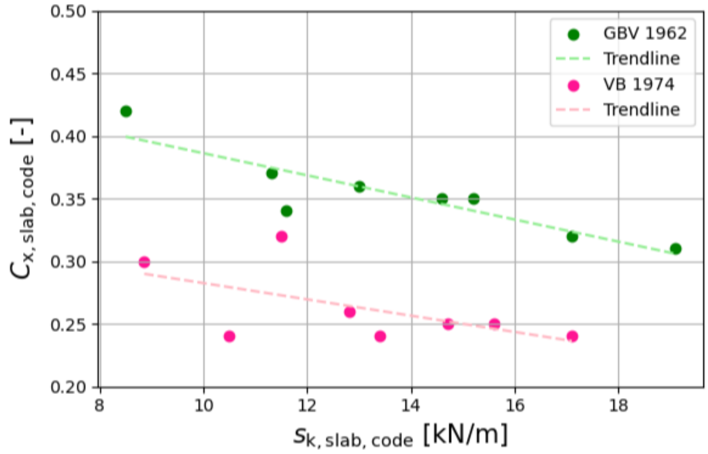


Figure 6.43: Relation between excess capacity factor $C_{x,slab,code}$ and original characteristic load $s_{k,slab,code}$ (unclean dataset)

6.8. Discussion of results

This section provides a detailed interpretation of the results obtained from the parameter study. The behavior of beams and slabs is analyzed separately, taking into account observed patterns, deviations from expected trends and the influence of design parameters such as characteristic load and governing failure modes. Outliers are examined where relevant, especially when they impact the reliability or consistency of the results. Finally, the model data is briefly evaluated to assess how well it captures the trends found in the parameter study and to explore its applicability in practical design scenarios.

6.8.1. Interpretation of beam data

The results for the beam data reveal several consistent patterns. Firstly, a strong correlation is found between $C_{F,beam,code}$ and $s_{k,beam,code}$ for both GBV 1962 and VB 1974, which aligns with expectations. The removal of outliers particularly strengthened this relationship for VB 1974, as these outliers strongly deviate from the expected trend. Section 6.7.7 explains that the outliers in combinations 21, 23, and 24 are caused by high shear stresses in the cross section, calculated using the governing equations. Therefore, these outliers were removed from the dataset for analytical purposes, but they are not considered noise or errors. They represent valid results for beams designed in accordance with VB 1974. For this reason, caution is advised when applying equation (6.51) to beams with high shear forces designed according to VB 1974, as the outcome may result in high excess capacities, while in reality this may not be the case. The outliers in combinations 3 and 4 for designs according to GBV 1962 are less likely to occur in practice. They are the result of the strict application of the GBV 1962 design methodology and of differences in resistance calculation between GBV 1962 and EC 2012. It is likely that, in practice, the strict design sequence of first relying on concrete shear capacity, then using bent up bars, and only using stirrups if necessary was not always followed exactly as done in the parameter study. Moreover, at the time, the lack of computational tools meant that designs were less optimized. Even though in this study the process was performed manually in Excel, many combinations were still tested to reach the final configurations.

Figures 6.7, 6.8, 6.9, and 6.10 show expected behavior, as all plotted values lie above the 1:1 lines. This is consistent with the fact that $s_{k,element,EC2012}$ is always higher than $s_{k,element,code}$, because the volumetric weight of concrete according to EC 2012 is higher than in the earlier codes. In addition, the variable load used in the assessment is governed by a value of 3.7 kN/m², whereas GBV 1964 and VB 1974 apply lower values. Moreover, the data points for the beams deviate further from the 1:1 line as $s_{k,element,code}$ increases, meaning that Δ_{sk} increases. In order to keep the load ratio in equation (6.53) approximately constant, this should be the case. Therefore this behavior is expected. Together with the strong linear correlation between $C_{F,beam,code}$ and $s_{k,beam,code}$ in figure 6.5, which results in an approximately constant capacity ratio from equation (6.53), the excess capacity factors show a near horizontal trendline and approximately normal distributions in figures 6.31 and 6.32.

However, the box-plot of GBV 1962 shows one clear outlier with a considerable distance from the median, as illustrated in figure 6.29. This outlier corresponds to combination 20, which features beam length $B3$, slab length $S2$, and facade weight $F2$. For this combination, the shear resistance governs the determination of excess capacity. Similar to other identified outliers, this low value can be attributed to the strict application of the design guidelines. In combination 20, $UC_{b,V} = 0.99$, which indicates that no shear reinforcement is required, despite a relatively high shear force. Stirrups are applied at the maximum allowable spacing and the assessment according to NEN 870X permits the inclusion of concrete shear resistance. The acting shear stress is 0.79 N/mm², while the allowable concrete shear stress under GBV 1962 is 0.80 N/mm². The calculated design shear resistance of the concrete is only 5.27$\cdot 10^{-7}$ N/mm², and the contribution from stirrups is even lower. Combined, these offer limited excess capacity. The three preceding combinations of $B3$ also rely solely on concrete shear capacity with minimum stirrup reinforcement, while the four subsequent combinations of $B3$, subjected to higher shear forces, are all permitted to use bent-up bars. The shear resistance provided by bent-up bars is significantly higher than that of concrete. As such, combination 20 is an unfortunate case: the acting shear force is high, but not high enough to trigger the use of bent-up bars under the strict design assumptions applied. This highlights the need for careful consideration of shear reinforcement configurations when analyzing GBV 1962 beam designs. Nonetheless, this outlier is not considered to have a significant distorting effect on the overall results.

When considering all outliers of beam designs from GBV 1962 and VB 1974, they provide some warnings for practical use. For GBV 1962 designs, short beam spans ($L_{beam} = 3.6$ m) with medium shear force or in simpler terms, medium floor spans ($L_{slab} = 5.4$ m), should be reason for a structural engineer to look more closely at the shear reinforcement configuration. The medium shear force is the tipping point where shear reinforcement is required, but often the minimum, which could result in lower excess capacities than the overall trend. In addition, a similar tipping point for the same reason, but now with bent-up bars instead of stirrups, can be identified for large beam spans ($L_{beam} = 7.2$ m) of GBV 1962 designs with medium shear force or in simpler terms, medium floor spans ($L_{slab} = 5.4$ m). For VB 1974 there is a clear warning for large beam spans ($L_{beam} = 7.2$ m) with high shear forces, or in simpler terms with large slab spans ($L_{slab} \geq 7.2$ m).

In addition, an overall higher excess capacity in similar elements design according to GBV 1962 than according to VB 1974 is in line with the safety factor analysis in section 5.2.1, which can be concluded from figures 6.5 and 6.4. In addition, the excess capacity $C_{F,beam,code}$ generally increases with increasing facade load g_{fac} according to figures 6.11 and 6.12. The variability of the box-plot corresponding to GBV 1962 is larger than VB 1972, therefore the overall impact of the increase between F1 and F2 is more evident in the graph of VB 1974.

The relationship between the excess capacity in beams and the length of the slabs in System 1 has also been examined. Figures 6.23 and 6.24 show a strong quadratic relationship between $s_{k,beam,code}$ and L_{slab}. This arises from the fact that two linear relationships are effectively multiplied. Specifically, the slab thickness increases in approximately constant steps as the slab length increases in similarly constant steps, shown in table 6.3. Since constant steps can be interpreted as a linear relationship between parameters, this results in a combined quadratic relationship. Examining the formula for g_{beam} in equation (C.3) shows that G_{slab}, which depends solely on slab thickness, is multiplied by L_{slab}, further confirming the quadratic trend. In addition, the near-linear relationship between $s_{k,beam,code}$ and $C_{F,beam,code}$ explains why a quadratic pattern is observed in the scatter plots of $C_{F,beam,code}$ versus L_{slab}. The R^2 value in figure 6.16, corresponding to VB 1974 beam designs, is lower than that for the GBV 1962 beam designs. This may be attributed to the fact that the outliers removed from the VB 1974 dataset correspond to beams with large slab spans. As a result, the relationship appears weaker for designs with higher excess capacity.

Lastly, the median of the excess capacity for varying beam lengths remains approximately constant. However, the variability increases for both GBV 1962 and VB 1974, which can be explained by higher characteristic loads associated with larger beam sizes for longer spans. The irregular data point corresponding to the beam span of 7.2 meters in the VB 1974 dataset can be attributed to the removal of three outliers linked to this beam length, as explained in section 6.7.7.

6.8.2. Interpretation of slab data

The results for the slabs exhibit somewhat different patterns compared to those of the beams. However, given that the slab dataset is only one third the size of the beam dataset, it is more challenging to assess whether certain data points should be classified as outliers or whether the observed behavior represents a fundamentally different pattern relative to the beams or to what was originally anticipated.

Overall, the positive linear trendlines in figure 6.4 and the clear increasing effect of facade weight on excess capacity, as shown in figures 6.21 and 6.22, both indicate that an increase in characteristic load leads to an increase in excess capacity. However, upon closer inspection, another pattern may also be present. Specifically, a slight decline in the excess capacity factor is observed as the characteristic load increases, as shown in figure 6.28. This trend is visible for both codes, even when the outlier corresponding to combination 5 of VB 1974 designs is excluded from the analysis. Before exploring potential causes for this behavior, specific data points that could influence the observed downward trend are reviewed and assessed. This declining trend suggests that excess capacity factors may not be constant for structural elements of the same type. To better understand this, the two ratios in equation (6.53), introduced in section 6.6.2, are analyzed in more detail.

Combination 1 and 5 of the slabs of VB 1974 pointed out an interesting point regarding the capacity ratio, the other outliers too in a different way. Namely, the governing capacity may have a significant influence on the excess capacity. As discussed in section 6.7.7, a governing deflection criteria of VB 1974 slabs results in over-dimensioning for strength, which in turn results higher excess capacity. If the datapoints of combination 1 and 5 are taken out of the graph in figure 6.28, the trendline approximately becomes horizontal. The highest value of $C_{x,slab,GBV1962}$, correspond to combination 1 as well, shown in figure 6.28, but also as outlier in figure 6.29, is steering the decline of the trendline of the beam GBV 1962 data. This data point can be explained by the same behavior as combination 1 of VB 1974, namely deflection being governing ($UC_d = 0.99$), resulting in slight over-dimensioning of the slab ($UC_{a,M} = 0.96$), which can be used as excess capacity. For the other data points of GBV 1962, the governing failure mode is a combination of bending moment capacity and deflection, but then specifically the deflection criteria depending on the steel stress, which is also an underlying parameter of the ultimate bending moment capacity. The $UC_{a,M}$ increases from 0.96 to 0.99 across combinations, as shown in table D.15, which can possibly explain the decline, as the lower values of excess capacity factors generally attribute to lower characteristic loads. This means that for combinations 1 through 5 of GBV 1962 slabs, there is slight over-dimensioning in the main steel reinforcement. This results from the limited range of available bar diameters and the assumed constant reinforcement spacing. The lower excess capacity factors of combinations 6 to 8, approximately 1.33, are most likely representative of the true excess capacity related to ultimate strength.

The relationship between the original and current characteristic loads is examined in figures 6.9 and 6.10, both showing nearly perfect linear relations. These graphs also illustrate $\Delta_{sk,code}$, which represents the difference between the 1:1 reference line and the plotted data points, and remains constant across both slab graphs. In terms of formulas, this implies that the load ratio significantly decreases as the characteristic load increases for slabs. However, since the load according to NEN 870X is always higher than the load according to the earlier codes, the load ratio remains above one. To keep the excess capacity factor constant, the capacity ratio would need to increase proportionally to compensate for the decreasing load ratio. This is not observed, as figure 6.6 shows that although the excess capacity increases slightly with the characteristic load, the increase is not sufficient to maintain a constant excess capacity factor. As a result, the slight decline observed in figure 6.28 is caused by the decreasing load ratio, while the capacity ratio does not increase enough to fully offset this effect. This trend could potentially be reasonable, as the increase in permanent load is likely less effective in generating additional strength for slabs than for beams, since beam dimensions can be adjusted more efficiently by modifying their height and width. However, this effect should be examined further. For now, the observed decline, especially when considering the influence of slightly over-dimensioning the main reinforcement for lower characteristic loads of GBV 1962 and VB 1974, is considered too marginal to be regarded as a significant trend. Still, this should be taken into account when determining an appropriate excess capacity factor for slabs, as will be further explored through model data.

6.8.3. Interpretation of model data

As stated in section 6.7.6, the model data aligns reasonably well with the results of the parameter study. However, for practical application, too many data points still fall above the model data trendline. Across all data, this applies to approximately 50% of the cases, which is consistent with the use of the median excess capacity factors in generating the model data. Particularly at higher excess capacities, the model trendline exceeds the parameter study data for all element types, with the difference being notably larger for slabs designed according to GBV 1962. This observation aligns with the explanation provided in section 6.8.2 regarding the marginal decline in the slab data shown in figure 6.28. Therefore, for practical use, a more conservative estimate of the excess capacity factor is advised.

6.9. Reflection on hypotheses

In this section, the initial hypotheses formulated in section 6.2.3 are briefly revisited in light of the findings from the parameter study. Where relevant, the results are compared to the excess capacity factors of the safety factor analysis to assess the validity and applicability of the assumptions made. This reflection aims to evaluate whether the observed trends support the expected behavior outlined at the outset of this study.

6.9.1. Hypothesis 1

The first hypothesis is as follows:

"Beams or slabs with a higher original total distributed load, $s_{k,beam}$ or $s_{k,slab}$, are expected to exhibit a higher excess capacity, $C_{F,beam}$ or $C_{F,slab}$."

After excluding the outliers, the Pearson correlation coefficients improved substantially, reinforcing the assumed linear relationship underlying this hypothesis. This linear trend is clearly visible in figures 6.5 and 6.6.

6.9.2. Hypothesis 2

The second hypothesis is as follows:

"For structural elements of the same type that were designed according to the same structural design code, it is expected that a single excess capacity factor $C_{x,element,code}$ can be applied in a similar manner as equation (5.13)."

The excess capacity factors for the same structural elements, grouped by design code, are presented in figures 6.27 and 6.28. The trendlines for both beam datasets are approximately horizontal, indicating that the excess capacity factor remains constant across the range of characteristic loads. For the slab datasets, the trendlines exhibit a slight downward slope, as discussed in detail in section 6.8.2. While this suggests a potential reduction in the excess capacity factor with increasing load, the effect is considered marginal within the context of this study. Further research is recommended to assess this behavior more thoroughly, particularly for heavier facade loads not included in the current analysis. For the purposes of this parameter study, it is therefore considered acceptable to use a single excess capacity factor when determining the excess capacity of both beams and slabs. However, as noted in section 6.8.3, a more conservative estimate is recommended to ensure reliability across the full range of loads examined.

6.9.3. Hypothesis 3

The third hypothesis is as follows:

"An increase in used values for loads used in earlier codes compared to EC 2012 negatively impacts excess capacity C_F."

To analyze this hypothesis, it is essential to clarify what is meant by excess capacity in this study. Within the parameter study, excess capacity, denoted as C_F, is defined as the characteristic permanent, uniformly distributed load (in kN/m) that can be added to either a facade beam or a one-meter-wide strip of slab supporting a facade, as calculated according to NEN 870X. The term added means that the original function and loading of the structural element remain unchanged. The permanent and variable loads originally acting on the element, though possibly updated in value, are subtracted with their respective partial safety factors from the total design resistance.

This also implies that if the original loading has increased, such as due to a higher volumetric weight of concrete or increased variable loads in EC 2012, then a larger portion of the element's design resistance is consumed compared to earlier codes. This effect is reflected in the equations used to determine excess capacity. When applying equations 6.47, 6.48, and 6.49, it is evident that lower or equal load values according to EC 2012 result in a higher governing excess capacity. The exact magnitude of this effect depends on the type of load (permanent or variable) and on which of the three equations governs the result. If this increase in excess capacity is then used in equation (6.53), assuming no change in characteristic load ($\Delta s_{k,code} = 0$), the excess capacity factor remains approximately the same, depending on the magnitude of change of the governing excess capacity. This indicates that while the defined excess capacity C_F may decrease when the load difference decreases, the total available structural capacity of the element remains constant. This makes sense, as no modifications are made to the element itself. Under this reasoning, the third hypothesis is considered valid within the scope of this parameter study.

6.9.4. Hypothesis 4

The fourth hypothesis is as follows:

"Excess capacity factors derived from beams or slabs are expected to resemble $C_{x,ten}$ more than $C_{x,com}$."

To compare the excess capacity factors derived from the parameter study with the previously defined values $C_{x,ten}$ and $C_{x,com}$, these factors have been added to the box plots of $C_{x,element,code}$ for both beams and slabs, shown in figures 6.44 and 6.45. These graphs clearly support the hypothesis that the role of steel as a governing factor in the design of beams and slabs also most-likely governs the available excess capacity of these elements. The distance between $C_{x,ten,code}$ and the median value of $C_{x,element,code}$ is even almost equal for both codes.

On average, the excess capacity factors from the parameter study are about 10% higher than those obtained from the safety factor analysis. This difference may be explained by the element-dependence discussed in section section 5.2.3, where it is noted that elements subjected to significant normal forces typically exhibit lower excess capacity factors. Nevertheless, since such elements are generally designed for higher absolute loads, their total excess capacity is still expected to be greater than that of beams and slabs.

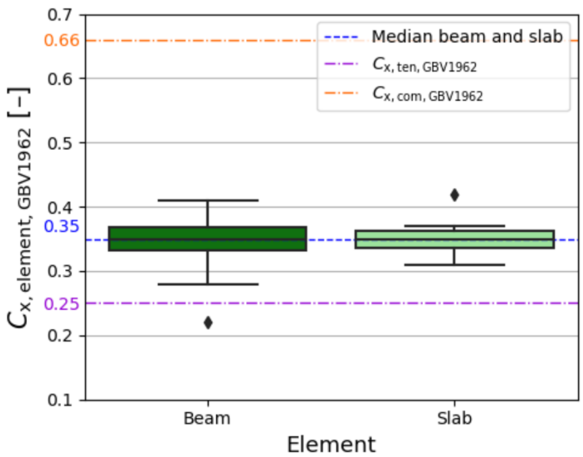


Figure 6.44: Box-plot of the excess capacity factors $C_{x,element,GBV1962}$ for each element according to GBV 1962 (clean datasets) with excess excess capacity factors from safety factor analysis plotted

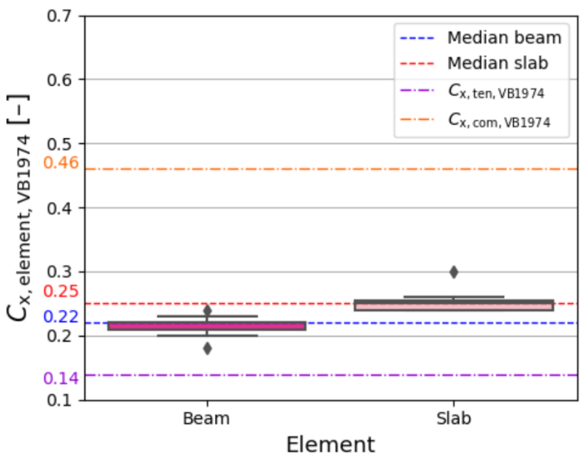


Figure 6.45: Box-plot of the excess capacity factors $C_{x,element,VB1974}$ for each element according to VB 1974 (clean datasets) with excess excess capacity factors from safety factor analysis plotted

6.9.5. Hypothesis 5

The fifth hypothesis is as follows:

"No significant difference is expected between the excess capacity factors C_x corresponding to the Beam System and those corresponding to the Slab System."

This hypothesis is easily validated by figures 6.44 and 6.45. The medians of the excess capacity factors for slabs and beams designed according to GBV 1962 are exactly equal. For VB 1974, a slight difference of 0.03 is observed between the medians of the two element types. This minor deviation can be attributed to the deflection criterion being governing for two out of the eight slab data points. It is likely that, had this not been the case, the excess capacity factors would have matched as well. However, the observed difference is clearly not significant, confirming the validity of the hypothesis.

7

User guide to the VGS implementation framework

In this chapter, the validated method for estimating excess capacity is combined together with the classification of VGS, biodiversity indicators and office building types to create a framework for the feasibility phase of implementing VGS for biodiversity enhancement. This chapter provides a user guide to applying the framework and is designed to be read independently.

7.1. Introduction to the guide

The user guide presents the VGS implementation framework and explains how it can be applied in practice. The framework is developed to support early-stage decision-making during the feasibility phase of integrating Vertical Green Systems (VGS) onto existing Dutch office buildings, with the aim of enhancing urban biodiversity. The structural and architectural components of the framework are based solely on concrete office buildings constructed between 1960 and 1990, in which facade loads are transferred to slab edges or facade beams.

The guide is structured as a combined explanation and user manual. It provides the necessary context, outlines the required inputs and explains each part of the framework to guide potential users through the application process. The intention is to make the framework understandable and usable for those involved in early-stage vertical greening decisions. While a structural engineering background is not required to apply most parts of the framework, especially in situations where detailed structural information is not yet available, further expert involvement will be necessary in later stages to validate the structural feasibility of implementing a VGS. In fact, the user guide includes specific warnings and remarks that highlight when consultation with a structural engineer is recommended, should such situations arise. The framework and accompanying guide are particularly relevant to sustainability consultants, architects, building owners, building managers, project developers and municipal advisors who are exploring VGS options for existing Dutch office buildings. At the feasibility stage, detailed design information is often not yet available, which makes a structured and accessible method like this framework especially valuable. In fact, the framework can already be applied when only the building's exterior is known.

This guide is structured as follows: first, the framework is introduced, followed by a description of the four knowledge levels. Then, the application of each of the three main components, excess capacity estimation, office building suitability and biodiversity performance, is explained in detail. The guide concludes with free decision space on how to integrate these components into a final VGS recommendation.

7.2. Overview of the framework

The VGS implementation framework presented in figure 7.1 serves as a guide during the feasibility phase of integrating VGS onto existing office buildings with the aim to boost urban biodiversity. The framework provides insight into three key components, excess capacity, office building suitability and biodiversity performance, which are briefly explained below. The framework components are intentionally left unnumbered, allowing users to follow a flexible order based on the available information or project context. It is not necessary to complete all three components, users may choose to apply only one or two, depending on the specific objectives or stage of their project. The outcomes of the selected components are brought together in the final step of the framework, resulting in a recommendation for VGS implementation. The black arrows in the diagram indicate information flows between framework steps, showing how outcomes from one step can inform or support another. Each step of the framework is further elaborated in the subsequent sections.

- **The structural feasibility by estimating the excess capacity**
  Structural feasibility is assessed by estimating how much additional weight the building's structure, specifically concrete floor edges and beams supporting facades, can safely carry beyond its original design. This remaining capacity is referred to as the excess capacity. To estimate this capacity, the framework applies a validated method based on excess capacity factors. The resulting estimate can be compared to the weight of various VGS options applicable to facades, in order to identify which systems are likely to be structurally feasible.
- **The office building suitability by analysis of exterior building characteristics**
  The suitability of the existing office building for VGS implementation is assessed by identifying building characteristics that are linked to VGS features in the framework, in order to provide recommendations for system selection.
- **The biodiversity performance of VGS**
  The biodiversity potential of various VGS systems is compared based on animal preferences and VGS features, resulting in a biodiversity ranking of the systems included in the framework.

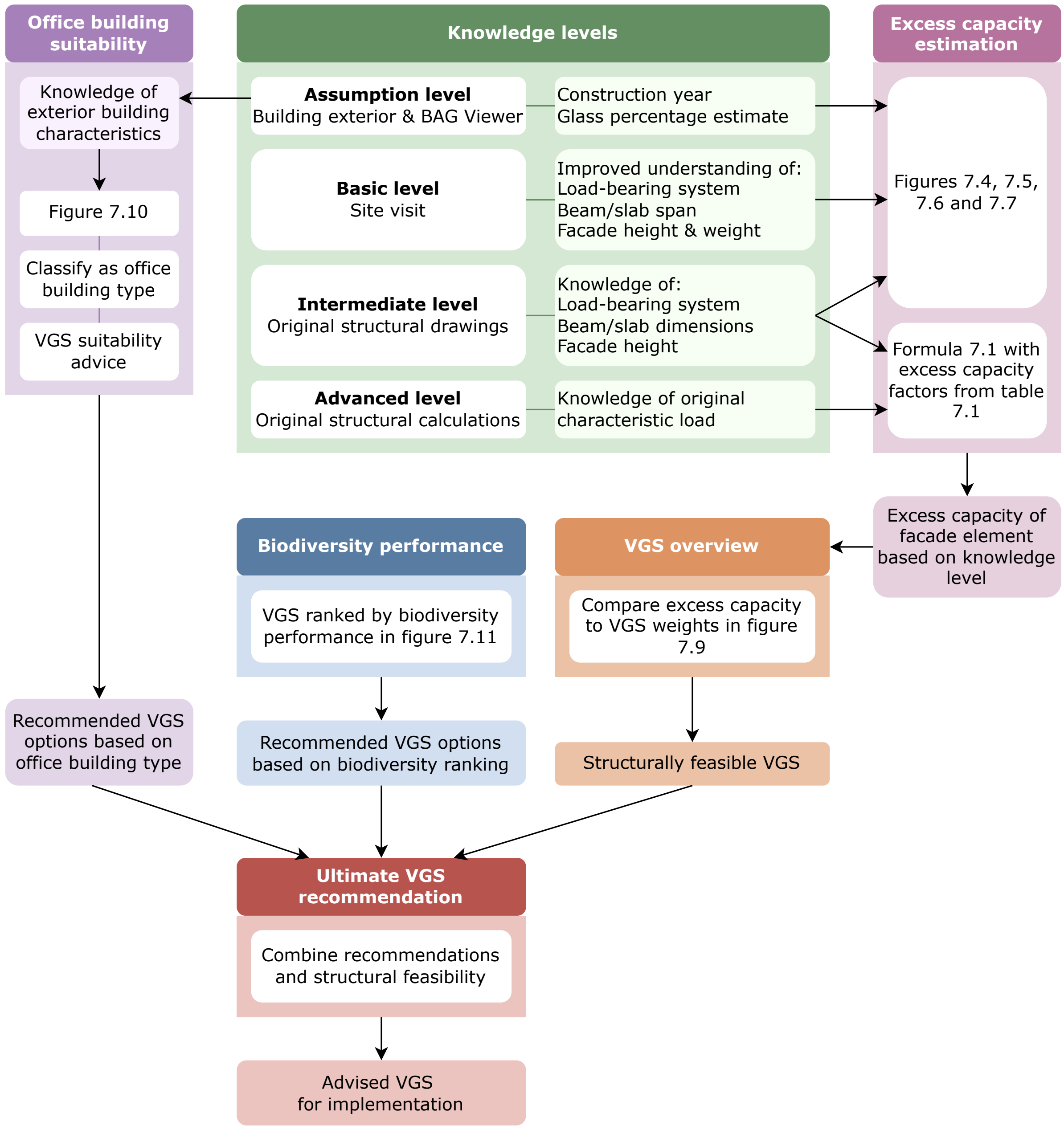


Figure 7.1: VGS implementation framework

7.3. Knowledge levels

The framework is designed to be applied during the feasibility phase. However, this phase can involve various sources of information, each contributing to different levels of knowledge. To account for this, the framework incorporates distinct knowledge levels, each associated with specific sources. Depending on the building in question, these sources may yield different types of information relevant to VGS implementation, such as the construction year, the configuration of the load-bearing system or the weight of the existing facade. It is important to note that a given source does not always result in the same type of knowledge. For example, some buildings visibly display steel columns on the exterior, such as the Havengebouw in Amsterdam, which already reveals substantial information about the structural system. In contrast, other buildings may feature mirrored curtain walls on all sides, creating the impression of a lightweight facade, while actually concealing concrete load-bearing walls, such as the former headquarters of Nationale Nederlanden in Rotterdam. In such cases, a site visit or the inspection of structural drawings may be required to obtain a sufficiently reliable estimation.

Furthermore, in practice, the sources associated with the knowledge levels are not necessarily consulted in the same order as presented in this framework. In some cases, structural drawings may already be available or can be retrieved from online archives, providing more detailed information with less effort than a site visit. However, as frequently emphasized in the NEN 8700 series, a site visit can provide valuable insights into the current condition of the building, something that drawings alone cannot offer [115, 117]. The NEN 8700 series is a set of Dutch standards used to assess the structural safety of existing buildings. Advancing to a higher knowledge level generally results in more accurate and complete information, thereby increasing the reliability of the estimated excess capacity. This is why the specific ordering of knowledge levels and associated sources has been applied in the framework. The four knowledge levels, along with their associated sources, are discussed in the following sections.

7.3.1. Assumption level

The assumption level refers to the stage at which only publicly available online information is considered. The construction year can be retrieved from the BAG Viewer, while the building's exterior can generally be examined using Google Street View or photos available on company websites or other online sources. This typically results in an estimate of the glass percentage of the facade, which in turn can be used to approximate the weight of the existing facade. As the framework includes a full glazed facade or classic curtain facade of 1 kN/m^2 and a lightweight brick masonry facade of 2.3 kN/m^2. Therefore, the glass percentage can help indicate whether the original characteristic load of the facade is closer to that of a glazed or a brick facade. In some cases, the material of the load-bearing structure can also be identified from the exterior, as illustrated in the example discussed in the previous section. This is particularly important in this context, as the excess capacity estimation within the VGS implementation framework is based exclusively on concrete structures. However, the material and the configuration of the load-bearing structure often cannot be determined from online exterior views alone, which is why this information is not included in the assumption level. Fortunately, concrete was the most commonly used construction material for office buildings between 1960 and 1990 [33], meaning the framework is applicable in a large share of cases from this period.

7.3.2. Basic level

The basic level involves conducting a site visit, allowing both the interior and the exterior of the building to be examined up close. This may provide additional insights into the main load-bearing structure, such as visible beams, columns or load-bearing walls. In addition, rough estimates can be made of the spans of structural elements such as slabs or beams. If these spans are clearly visible, they can even be approximately measured. Furthermore, observing and physically accessing the facade from both the inside and outside can offer valuable information. For example, viewing the facade from both sides may allow for an estimation of the material and the approximate thickness of the complete facade build-up.

7.3.3. Intermediate level

The intermediate level involves examining the original structural drawings, a task that benefits from basic architectural knowledge or a general understanding of construction principles. Naturally, these provide definitive information about the location of structural elements, their exact dimensions and the facade height. Since the position of the structural elements is known, the load path can often be derived. Meaning that it is often possible to understand how the building transfers loads down to the ground. In addition, the direction of floor loading is often indicated in the drawings. This means the drawings can show whether the facade is directly supported by a floor edge, a beam or another type of structural element that would make the use of the framework unsuitable in that case. The facade material can often also be identified from existing architectural or structural drawings. However, the exact total weight of the facade cannot be determined from these sources alone.

7.3.4. Advanced level

The advanced level represents the highest knowledge level included in the feasibility phase. Beyond this point, only physical testing can provide further insight into the structure. This level involves a structural engineer examining the original structural calculations, in which the original characteristic load acting on the element is either specified directly or can be accurately derived. These documents also reveal the exact behavior, configuration and dimensions of the main load-bearing system, including whether the facade is supported by a facade beam or by the slab edge. The current characteristic load acting on the element can be directly derived from the dimensions of the structural elements combined with their corresponding volumetric weights and variable floor loads specified in EC 2012 and NEN 8700 series. The combination of the characteristic load and the identified support method provides the most accurate basis for estimating structural capacity within the framework.

7.4. Excess capacity estimation

The excess capacity estimation is structured around the different knowledge levels, as increasing knowledge allows for a more accurate and reliable assessment. Two methods are proposed: a graph-based method for the assumption, basic and intermediate levels, and a formula-based method for the intermediate and advanced level. While the same graphs can be applied across the first three levels, the precision and reliability with which they are used improves as more information becomes available. At the intermediate level, user can choose between the two methods. Users can either apply the graphs with more accuracy based on structural drawings, or, if sufficient input is available, opt to use the formula from the advanced level with the appropriate excess capacity factors. Before introducing the two methods in detail, the next subsection explains which excess capacity factor was used to generate the model data shown in the graphs.

7.4.1. Excess capacity factor for excess capacity estimation

The validated method based on excess capacity factors is used to generate the model data presented in the framework's graphs. Validation was carried out through a parameter study described in chapter 6. As noted in the discussion of the parameter study in section 6.8, a more conservative estimate is recommended for the framework than was used in the parameter study, in order to ensure greater reliability across the full range of examined loads. The model data generated for the parameter study used the median excess capacity factor per element and per historical code. However, this approach resulted in a slope that was overall too steep when compared to the parameter study data, potentially leading to an overestimation of excess capacity at higher loads. To address this, the framework's model data uses the excess capacity factor corresponding to the first quartile of the distributions shown in figures 6.29 and 6.30. Now, instead of 50% of the data being above the chosen excess capacity factor (as with the median), now 75% of the data lies above it. This means that 25% of the designs (from clean datasets) included in the parameter study result in lower excess capacity factors than the value applied in the framework. In other words, the framework does not consistently provide the minimum excess capacity expected in an element, but rather an indicative value. After more detailed analysis by a structural engineer, the actual excess capacity may be lower than the framework suggests. However, in many cases, the framework offers a valuable and sufficiently accurate indication. The selected excess capacity factors are listed in table 7.1. The original characteristic loads of both elements and the characteristic load according to EC 2012, listed in sections D.3.1 and D.3.2, are combined with the excess capacity factors listed in the table below to create model data for the framework by using equation (7.1).

Element	Code	$C_{x,element,code}$
Beam	GBV 1962	0.33
Beam	VB 1974	0.21
Slab	GBV 1962	0.34
Slab	VB 1974	0.24

Table 7.1: Excess capacity factors used for the creation of model data for the VGS implementation framework

The structural elements

The term 'element' refers to the structural component that supports the facade. In this framework, a distinction is made between two typical configurations: (1) the Beam System, which consists of a concrete beam positioned along the facade edge, and (2) the Slab System, in which a concrete floor slab is positioned along the facade edge. In the Beam System, the facade beam is assumed to also support the load of a one-way spanning slab that spans perpendicular to the beam. These configurations exhibit different structural behaviors and therefore separate excess capacity factors have been derived. The Beam System is visualized in figure 7.2 and the Slab System is visualized in figure 7.3. The focus is on calculating the excess capacity of beams and slabs, as literature suggests that concrete columns and walls generally have greater excess capacities [165].

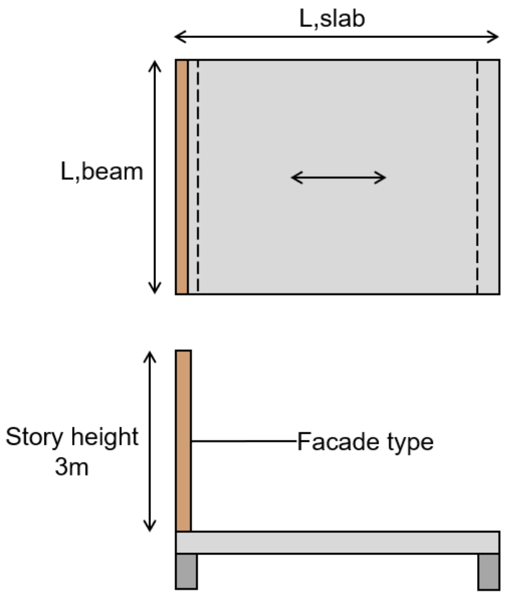


Figure 7.2: Sketch of structural system 1: the Beam System

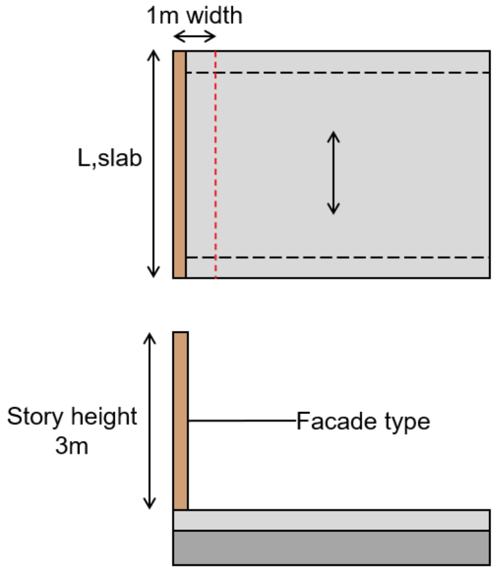


Figure 7.3: Sketch of structural system 2: the Slab System

7.4.2. Graph-based method for excess capacity estimation

For the excess capacity estimation at the assumption, basic and intermediate knowledge levels, separate graphs have been developed for beam and slab systems. The graphs for the beam systems are provided in figures 7.4 and 7.5, while the graphs for the slab systems are shown in figures 7.6 and 7.7. The graphs indicate that greater beam spans and especially larger slab spans for both systems, generally lead to higher excess capacities. The following sections explain how to use these graphs at each knowledge level.

Assumption level

The building's construction year determines which set of graphs applies: figures 7.4 and 7.6 correspond to structures built between 1962 and 1974, while figures 7.5 and 7.7 apply to buildings designed between 1974 and 1990. As such, the construction period guides users to the appropriate pair of graphs for beam and slab systems. However, the load-bearing element of the facade (slab or beam) typically cannot be identified from the exterior, while the associated excess capacity is typically lower for slabs than for beams. For this reason, it is recommended to start with the slab graphs at this stage of limited knowledge.

In addition, the estimated glass percentage can be used as an indicator: a facade weight of 1.0 kN/m² corresponds to a typical curtain wall or glass facade, while a facade weight of 2.3 kN/m² represents a brick masonry facade. The latter is the heaviest facade type considered self-supporting and typically supported by slab edges or facade beams, as described in section 4.3.1. Therefore, if the glass percentage can be estimated, it can serve as a basis for determining the likely excess capacity of facade elements using the provided graphs. In the slab graphs, the facade weight is explicitly indicated. In the beam graphs, two dots of the same color are shown for each beam length: the lower dot corresponds to the lighter facade weight, while the higher dot represents the heavier facade weight.

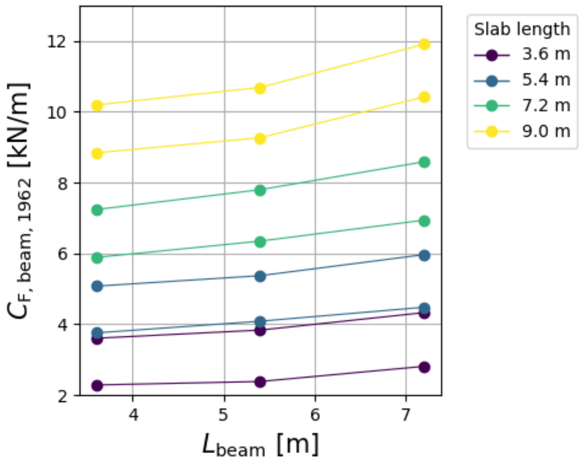


Figure 7.4: Relation between excess capacity $C_{F,beam,GBV1962}$ and the beam length L_{beam} with highlighted Slab length L_{slab} (framework dataset)

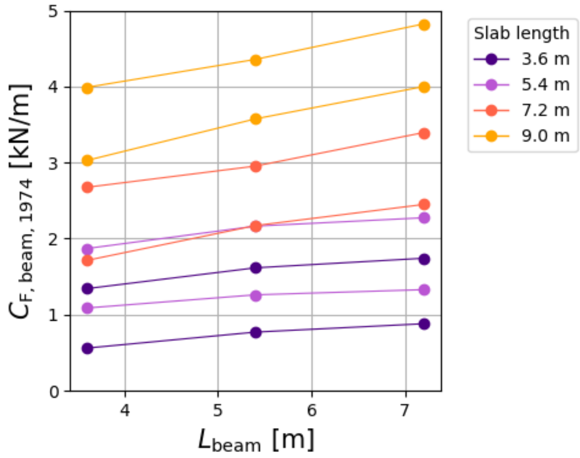


Figure 7.5: Relation between excess capacity $C_{F,beam,VB1974}$ and the beam length L_{beam} with highlighted Slab length L_{slab} (framework dataset)

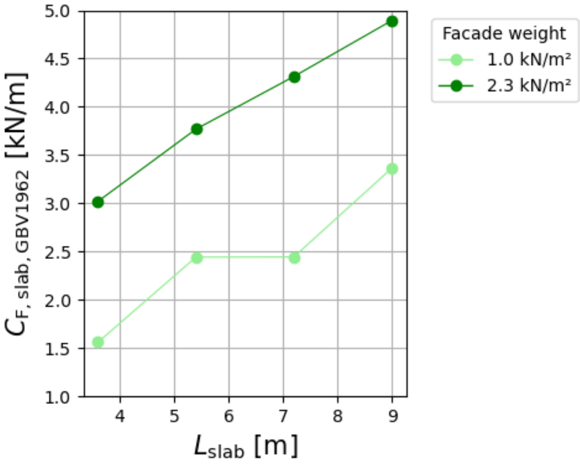


Figure 7.6: Relation between excess capacity $C_{F,slab,GBV1962}$ and the slab length L_{slab} (framework dataset)

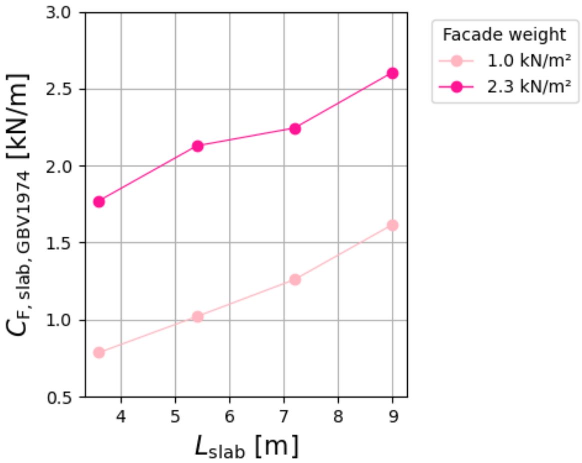


Figure 7.7: Relation between excess capacity $C_{F,slab,VB1974}$ and the slab length L_{slab} (framework dataset)

Since the slab span is often unknown when only the building exterior is available, it is recommended to use a conservative estimate, corresponding to the smallest slab span of figures 7.6 and 7.7. The beam graph can still be used to provide insight into what the derived excess capacity from the slab graph would imply for the configuration of a potential beam design, since a conservative value is appropriate at this stage. In this regard, it should be noted that the excess capacity values for the smallest slab span of VB 1974 are not necessarily lower than those for the smallest beam and slab span configuration in the VB 1974 beam designs.

Basic level

It is uncertain what additional information a site visit may provide, as this depends on the specific building and whether structural elements are clearly visible. Typically, a site visit improves understanding of the load-bearing system, potential beam and/or slab spans, the story height and the facade weight. In particular, estimated beam and slab spans can be used in the graphs shown in figures 7.4, 7.6, 7.5, and 7.7 to refine the excess capacity estimation and potentially avoid relying on the most conservative values corresponding to the lowest beam and/or slab span. However, since the specific element supporting the facade is likely still unknown, it is recommended to primarily use the slab graphs as the basis for the excess capacity estimation and to use the beam graphs for reference.

Furthermore, the values shown in these graphs are based on a story height of 3 meters, with each structural element assumed to support one story of facade, also 3 meters. If the actual story height is larger, the corresponding facade weight on the structural element will generally be higher. This means that not only the glass percentage and facade material, but also the story height can influence the assumed facade weight used in the graphs of the excess capacity estimation.

Intermediate level

Structural drawings are particularly informative because they reveal the location of structural elements. Even without knowing the exact load path as originally designed, it becomes possible to determine whether a beam is present at the facade's location or in which direction the floor spans. As a result, the drawings often provide sufficient information to decide whether the slab or beam graph should be used for the excess capacity estimation.

In addition, facade drawings allow for a more detailed determination of the facade weight. The dimensions of all structural elements are also available, eliminating the need for assumptions. This means that, rather than relying on a conservative excess capacity estimate from the graphs, users can apply the actual slab and/or beam spans taken directly from the drawings. As a result, the excess capacity can be derived more accurately from the graphs, based on the specific configuration of the building.

7.4.3. Formula-based method for excess capacity estimation

For the excess capacity estimation at the intermediate or advanced level, equation (7.1) can be used in combination with the excess capacity factors provided in table 7.1. The other two parameters in the formula are derived differently depending on whether the intermediate or advanced level is applied, as they correspond to different sources of information. The following sections explain this in more detail.

$$C_{F,element,code} = (C_{x,element,code} + 1) \cdot s_{k,element,code} - s_{k,element,EC2012} \quad \text{[kN/m]} \qquad (7.1)$$

Where:

- $C_{F,element,code} =$ the excess capacity for a specific element, based on a specific historic code in [kN/m]
- $C_{x,element,code} =$ the excess capacity factor for an element type, based on a specific historic code in [-], defined in table 7.1
- $s_{k,element,code} =$ the original characteristic load for a specific element, based on a specific historic code in [kN/m]
- $s_{k,element,EC2012} =$ the current characteristic load for a specific element, based on the EC 2012 in [kN/m]

Intermediate level

As explained in the above sections, the dimensions of the structural elements and the facade weight can be determined with more certainty. Based on these values and the volumetric weights of materials (originally and currently), both the original and current characteristic loads can be calculated. These loads can then be combined with the appropriate excess capacity factor and used in equation (7.1) to directly calculate the excess capacity. The formulas to calculate the original and current characteristic loads are assumed to be known by the structural engineer or they can be found in section 6.3.3 for GBV 1962, in section 6.4.3 for VB 1974 and in section 6.5.6 for EC 2012 and NEN870X. The inputs for these formulas, such as the volumetric weight of concrete, lightweight variants of facade systems, and uniformly distributed variable floor load, are provided in table 7.2 to streamline the calculation process.

The minimum value of the current uniformly distributed variable floor load is 2.5 kN/m². For lightweight partition walls, an additional load between 0.8 and 1.2 kN/m² is added, referred to as q_{lw} in table 7.2. This additional load depends on the weight of the lightweight partition walls originally used. The values in the graphs of the framework are based on the heaviest lightweight partition walls and therefore reflect the maximum increase in variable floor load. However, if the original structural calculations are available, a more accurate value for q_{lw} can potentially be derived. The structural engineer is referred to paragraph 6.3.1.2(8) of NEN 1991 [110] to determine their own value for q_{lw}. Still, continuing to use the highest value for this additional variable load is the recommended conservative approach and corresponds to the values shown in the framework graphs.

Symbol	Value	Unit	Definition
γ_c	24	kN/m³	Volumetric weight of reinforced concrete according to GBV 1962 and VB 1974
γ_c	25	-	Volumetric weight of reinforced concrete according to EC 2012
$Q_{k,slab}$	2.5	kN/m²	variable load acting on the slab according to GBV 1962
$Q_{k,slab}$	2.0	kN/m²	variable load acting on the slab according to VB 1974
g_{lw}	0.5	kN/m²	permanent load accounting for the weight of lightweight partition walls according to VB 1974
$Q_{k,slab}$	2.5 + q_{lw}	kN/m²	variable load acting on the slab according to Ec 2012
q_{lw}	0.8; 1.0; 1.2	kN/m²	variable load accounting for the weight of lightweight partition walls according to EC 2012

Table 7.2: Material and load properties, retrieved from VB 1974A [154], TGB 1972 [118, Tabel 1 & 3], TGB 1955 [61, Tabel I & II] and NEN 1991 [110, Tabel A1]

Advanced level

Instead of estimating the original characteristic load based on assumed volumetric weights and floor loads, as is done at the intermediate level, a structural engineer can directly derive the original characteristic load, $s_{k,element,code}$, from the original structural drawings and calculations. The current characteristic load acting on the element, $s_{k,element,EC2012}$, can be determined in the same manner as at the intermediate level. As with the structural drawings used at the intermediate level, the additional variable load q_{lw} for lightweight partition walls can potentially be derived more accurately based on the original structural calculations. A structural engineer may therefore choose to apply a lower value, although this represents a less conservative approach. Together, the original and current characteristic loads, in combination with the excess capacity factors provided in table 7.1, can be used in equation (7.1) to calculate the excess capacity estimation.

7.4.4. Limitations and practical warnings of the excess capacity estimation

General limitations

While the excess capacity estimation method provides a structured and scalable approach, several limitations should be acknowledged. First, the framework relies on historical codes (GBV 1962 and VB 1974) and assumes simplified, uniformly distributed load conditions on facade beams and slab edges. Consequently, the method is only suitable for buildings constructed between 1962 and 1990, prior to the introduction of a new Dutch concrete building standard. In addition, buildings that deviate from typical concrete office construction, for example in terms of load-bearing structure or load distribution, may not align with the assumptions underlying the method. Specifically, the excess capacity estimation is based on a parameter study of basic concrete slabs spanning in one direction and rectangular concrete beams with ribbed steel reinforcement, both subjected only to uniformly distributed loads. It does not account for specific floor or beam types, such as hollow-core slabs (referred to differently in historical codes), I-beams, or point loads. Second, the excess capacity factors used in the graph-based method represent conservative estimates (based on the first quartile). These enhance general reliability but may still over- or underestimate the actual capacity in individual cases. Third, an accurate determination of structural capacity always requires the input of a structural engineer. The framework is not intended to replace engineering judgment but to support early-stage decision-making, when detailed calculations are typically not yet desired. Finally, the formula-based method assumes accurate knowledge of both original and current loads, which may not always be obtainable from available documentation. As a result, this method is not always applicable. Therefore, the outcomes of the excess capacity estimation should always be interpreted as indicative and further validation by a structural expert is essential in later design stages.

Practical warnings for the user

The following practical warnings highlight areas of uncertainty for the user and indicate situations in which consultation with a structural engineer is advised.

- If the building was constructed between 1962 and 1974 and facade-supporting beams have been identified, a warning applies to short beam spans (approx. 3.6 m) and large beam spans (approx. 7.2 m), both in combination with medium floor spans (approx. 5.4 m). For both beam lengths, there is a tipping point, occurring with medium floor spans, at which the reinforcement design and calculation method changes under GBV 1962. This shift results in a significant reduction of excess capacity, according to resistance calculations based on the current concrete codes (NEN 8700 series), as this calculation method has remained unchanged. However, it should be noted that this outcome follows from highly optimized structural designs based on historic codes, which were likely not applied with the same level of optimization in practice. Therefore, this does not mean that all excess capacity values associated with these beam configurations are by definition invalid, but rather that they should be treated with caution and ideally verified by a structural engineer in such cases.
- If the building was constructed between 1974 and 1990 and facade-supporting beams have been identified, a warning applies to large beam spans (approximately 7.2 m) in combination with large floor spans (approximately 7.2 to 9 m). These beams are heavily loaded and the (shear) strength of reinforced concrete beams is significantly lower under the current concrete design guidelines than under the historic VB 1974 code. Together with specific design and calculation methods for large beams, this can result in no excess capacity, even though the framework would suggest a high excess capacity for such configurations. Therefore, this should be considered a serious warning and a reason to have the configuration reviewed by a structural engineer. It should again be noted that such zero-capacity outcomes stem from highly optimized designs, which were likely not applied to this extent in practice. This does not mean that all concrete designs with this configuration lack excess capacity, but it does call for careful evaluation.
- Lastly, it is important to note that the parameter study and the resulting excess capacity estimation in the framework do not account for deflection criteria of floors or beams. The NEN 8700 series for existing structures does not legally require a deflection check, but it is reasonable to assume that building users expect, for example, their furniture to remain level and doors to close properly. Therefore, it is recommended to assess deflection in a later design stage or at least remain aware of potential deflection issues and visually inspect them during a site visit.

Interpretation notes for the structural engineer

This subsection provides more detail on the general beam warnings described above and explores their potential causes. The design and calculation of shear reinforcement in GBV 1962 and VB 1974 differ significantly from current practice. These differences are substantial enough to cause notable deviations in excess capacity compared to the expected trends within the framework. A brief explanation of how shear reinforcement was designed according to GBV 1962 and VB 1974 is provided below, followed by a more in-depth analysis of the underlying causes of the practical warnings outlined above. The full step-by-step design approaches of GBV 1962 and VB 1974, the resistance calculations according to the NEN 8700 series and the analysis of outliers that resulted in practical warnings can be found in chapter 6.

- **GBV 1962**
  The total diagonal tensile stress resulting from the shear force is initially resisted by the allowable diagonal tensile strength of the concrete. When this is insufficient, a distance y from the end of the beam is derived over which the shear reinforcement must resist the total shear force in that region, meaning the tensile strength of the concrete is not considered within that zone. The shear reinforcement initially consists of bent-up bars, which are main reinforcement bars bent-up near the ends of the beam. If these are not sufficient, vertical stirrups are added to the shear capacity

calculation. As a result, bent-up bars and possibly additional stirrups (placed at a closer spacing than the minimum used over the rest of the beam) are only applied over the specific distance y near the end of the beam.

- **VB 1974**
  According to VB 1974, the shear force is initially resisted by the concrete. If this is not sufficient, the portion of the shear force that exceeds the concrete's capacity over a certain distance y must be resisted by shear reinforcement, which in this case consists only of vertical stirrups. In the remaining part of the beam, the minimum required stirrup spacing is applied.
- **NEN 8700 series**
  The shear calculations according to NEN 8700 series only take stirrups into account and assume a uniform stirrup spacing. Bent-up bars are interpreted as inclined stirrups with an assumed spacing and any variation in stirrup spacing along the beam is not considered. As a result, the stirrup and/or bent-up bar spacing at the end of the beam is used in the resistance calculation of NEN 8702 and EC 2012. For the GBV 1962 designs, the concrete shear resistance is only included if the tensile resistance in the original design was deemed sufficient without shear reinforcement, while for the VB 1974 the shear capacity of the concrete can always be taken into account.

The differences between historical and current shear design approaches, as described above, can lead to discrepancies in excess capacity calculations. These discrepancies become particularly relevant in specific beam-span configurations observed in the parameter study. The following section highlights the structural origins of the practical warnings introduced earlier and explains why certain combinations of beam and floor spans may require special attention.

- **Short-span beams of GBV 1962**
  In the parameter study, it is assumed that short-span beams (3.6 m) do not contain bent-up bars, as these shorter beams are more likely to lack sufficient bending moment capacity near the supports. When the applied shear force exceeds the concrete's allowable shear resistance, a critical distance y from the beam end is calculated over which vertical stirrups must carry the excess shear. At the tipping point described in the previous subsection, the concrete shear capacity is just slightly insufficient, and due to the short beam span, this results in a small distance y over which the shear reinforcement must act. This means that, under historical codes, the required shear reinforcement only needs to resist a small portion of the total shear force. However, the NEN 8700 series does not account for this limited distance y. Instead, it assumes the provided stirrup reinforcement must resist the full shear force over the entire length of the beam. As a result, stirrups that were originally sufficient for a small region under GBV 1962 may appear insufficient under the current code.
  It is important to note that a small value for y implies that, in the historical method, most of the beam relies on the concrete and minimum stirrup reinforcement to resist shear. This type of composite behavior is not accounted for in the current parameter study, as it cannot be directly derived from either the historical or current codes and would require further research. Therefore, applying strict code-based calculations may lead to conservative warnings, where the estimated excess capacity is significantly lower than the framework would otherwise suggest. In such configurations, a more detailed shear assessment by a structural engineer is strongly recommended.
- **Long-span beams of GBV 1962**
  The tipping point for the long-span beams is also associated with the moment when the shear stress nearly becomes high enough to require additional shear reinforcement. At this point, the acting shear force is substantial, but still just below the threshold that would trigger the design of additional shear reinforcement. As a result, only the concrete shear resistance and stirrups at maximum spacing are used. However, the allowable concrete shear resistance according to GBV 1962 is significantly higher than that allowed under the current NEN 8700 series. This discrepancy, combined with the use of minimum stirrup reinforcement, results in lower excess capacity than expected for this beam configuration according to the framework. Therefore, this scenario represents a practical warning that warrants careful evaluation by a structural engineer.
- **Long-span beams of VB 1974**
  When long-span beams are combined with large floor spans, the resulting shear forces at the

supports can become considerable. In designs based on VB 1974, shear resistance is provided by both concrete and stirrups. The concrete is fully utilized and the stirrup unity checks closely approach the design limits. While the shear capacity of the stirrups is already lower under the NEN 870X series than under VB 1974, the reduction in concrete shear capacity is even more significant. Consequently, a reinforcement configuration that was acceptable under VB 1974 may now prove insufficient, resulting in no excess capacity. Therefore, beams subjected to high shear forces should be thoroughly assessed by a structural engineer.

7.5. VGS overview

To determine whether the estimated excess capacity is sufficient for the implementation of a VGS, the weights of various VGS types are compared with the excess capacity estimation obtained in the previous step of the framework. The VGS types included in the framework have been classified in chapter 2 based on their main distinguishing physical characteristics, allowing market-available systems to be classified and compared accordingly. This classification plays a key role in the building suitability and biodiversity performance step of the framework, where the systems' key features are used to provide implementation recommendations and a biodiversity ranking. These aspects are explained in more detail in their respective sections. A visual overview of the VGS classification used in the framework is provided in figure 7.8.

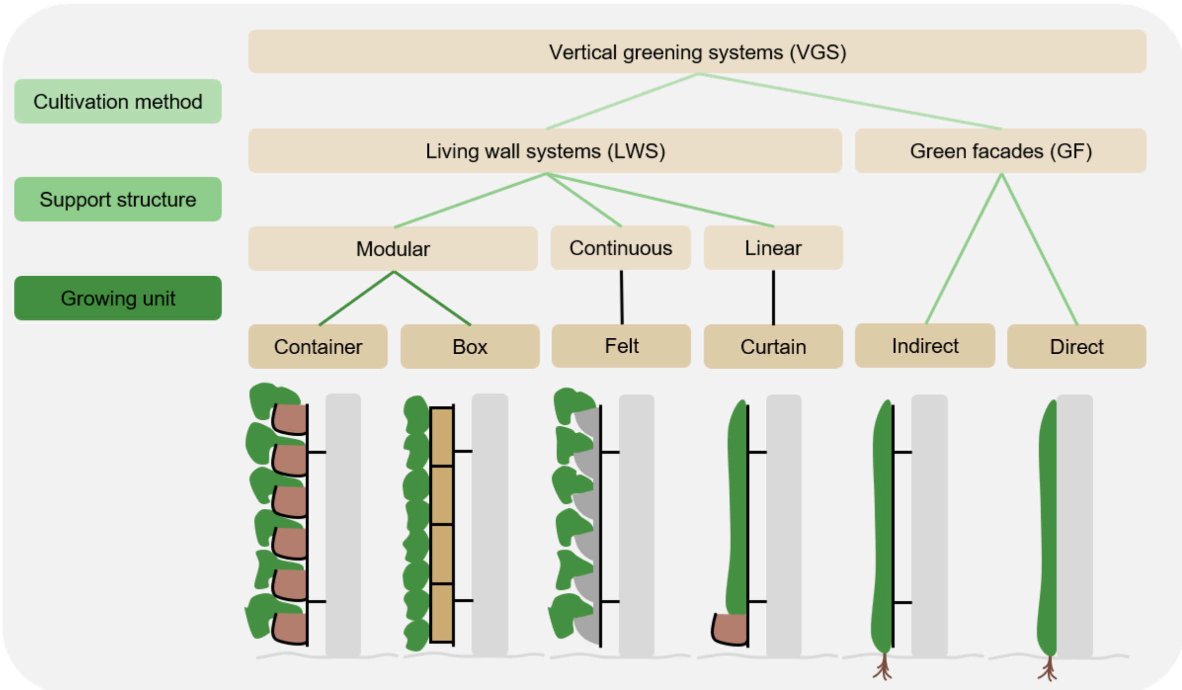


Figure 7.8: Classification system of Vertical Greening Systems

7.5.1. VGS types

To gain a better understanding of the various VGS types, their key features have been summarized in table 7.3 for the living wall systems and in table 7.4 for the green facades. For further background and explanation of the information presented in these tables, it is recommended to consult chapter 2.

7.5.2. VGS weights

To determine whether the estimated excess capacity is sufficient for the implementation of a VGS, the weights of the VGS are compared with the excess capacity estimation obtained in the previous step of the framework. These weights are visualized in figure 7.9. They are multiplied by a story height of 3 meters, consistent with the parameter study and the values for excess capacity in the other graphs of the framework, to express them in [kN/m]. This allows for a direct comparison with the excess capacity

Main category	Living Wall Systems (LWS)			
Cultivation method	Hydroponic technique with optional growing medium			
Sub-category	**Modular**		**Continuous**	**Linear**
Type of support structure	Modular panels with separate growing units that can be combined or rearranged		Continuous textile screen with integrated pockets for plant insertion	Linear planter boxes with climbing aid
System name	**Container**	**Box**	**Felt**	**Curtain**
		Key features		
Vegetation	Wide range of species, as shrubs, grasses, perennials, succulents		Wide range of species, as shrubs, grasses, perennials	Twining climbers, tendril climbers, rambling shrubs and hanging plants with evergreen or deciduous foliage
Growing medium	(In)organic substrate	Semi-rigid (in) organic substrate	Small amount of organic substrate or none and plants root in felt layer	Soil, (in)organic or mixed substrate
Growing unit	Vessels, trays, planter tiles, cassettes, planter boxes	Steel/plastic panel boxes with front cover, like steel/plastic grid or felt layer	Flexible pockets of multiple textile layers	Horizontal planter boxes
Mounting system	Containers are hooked onto substructure or directly attached to wall with uprights and brackets	Boxes attached directly to wall with uprights and brackets or use substructure like waterproof backing board or an aluminum frame	Screen stapled to base panel with waterproof membrane, in turn connected to a frame that is attached to wall	Planter boxes and climbing aid are fastened to substructure or directly attached to the wall with brackets
Irrigation system	Computerized irrigation, drip line on top of each module		Computerized irrigation, drip line at top of wall	Periodically depending on plants and climate
Growing speed	Fast	Fast	Medium-fast	Medium-slow
Weight (kg/m²)	50-150	30-80 wool 60-120 foam	30-100	40-140

Table 7.3: Overview of Living Wall Systems (LWS) [88, 160, 123, 164, 43, 91, 96, 120, 98, 125]

Main category	**Green Facades (GF)**	
Cultivation method	Rooted into the ground	
Sub-category	**Indirect**	**Direct**
Type of support structure	Climbing aid with air cavity in between the wall and the system	No additional support structure, plants directly adhere to wall by adventitious roots or self-adhesive pads
System name	**Indirect**	**Direct**
Key features		
Vegetation	Twining climbers, tendril climbers, rambling shrubs and hanging plants with evergreen or deciduous foliage	Root climbers and hanging plants with evergreen or deciduous foliage
Growing medium	Ground soil	
Growing unit		
Mounting system	The climbing aid is attached to the wall by uprights, brackets, anchors and spacers	
Irrigation system	Manually, periodically depending on plants and climate	
Growing speed	Medium-slow	Slow
Weight (kg/m^2)	20-30	5-10

Table 7.4: Overview of Green Facades (GF) [164, 160, 123, 67, 130, 91, 125]

values derived in the previous step.

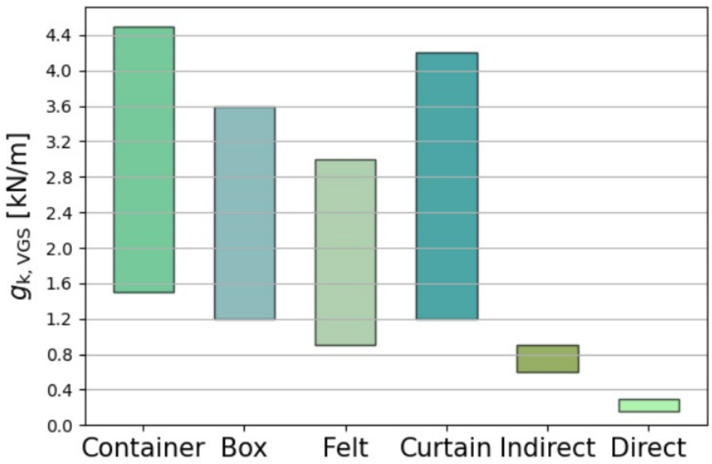


Figure 7.9: The weights of the VGS in [kN/m], considering a story height of 3 m, retrieved from tables 7.3 and 7.4

7.6. Office building suitability

Within this part of the framework, the existing office building in question is assessed for its suitability for various VGS options. This assessment is based on the assumption level of the identified knowledge levels, where the building's exterior is essentially the only input considered. From the exterior, certain building characteristics can be derived, such as building height and facade shape. A set of building characteristics derived from to the building in question can then be used in figure 7.10 to classify the building under a specific office building typology. These typologies are based on common architectural features observed during the studied construction period, rather than on the exact construction year. In figure 7.10, building characteristics are linked to specific indicators used to assess the suitability for VGS implementation. The figure suggests an order in which the building indicators for the Functional Boxes and Interlocking Shapes should be assessed, as some indicators are more restrictive than others. For some building indicators, multiple building characteristics are presented as different options, although in certain cases, a combination of these options may apply. For example, within the facade type category of the Functional Boxes and Interlocking Shapes, a building may feature continuous strip windows as well as opaque sections made of concrete, meaning that the corresponding recommendations for both should be taken into account. In other cases, the options are mutually exclusive. For instance, within the facade geometry of Interlocking Shapes, the facade may be either planar, curved and tall, or curved and short, but not more than one at the same facade. Finally, the figure provides a set of recommendations on which VGS types are most suitable for the building in question.

Naturally, it is possible that a building does not fit into one of the predefined office building types, as these types were developed for office architecture with clear, distinctive physical features. In such cases, all relevant building characteristics from figure 7.10 can still be considered and the user can determine the appropriate combinations themselves. Additionally, a more in-depth explanation of the office building typology and the reasoning behind the VGS suitability recommendations can be found in chapter 4.

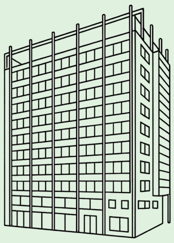
	Building indicator	Building characteristic	VGS suitability
Functional boxes 	**Box-shaped buildings of varying heights and facade materials**		
	1. Facade geometry	Planar facade	All VGS are suitable.
	2. Building height	Low-rise	All VGS are suitable.
		High-rise	Green facades are considered unsuitable, due to the maximum attainable growth height of 25 m.
			Box and curtain systems are recommended, due to their standard integrated safety features that prevent vegetation from falling.
	3. Facade type	Concrete or masonry	Concrete or masonry is preferred for attaching all living wall systems and indirect green facades.
		Marble or masonry	Direct green facades are not recommended as they potentially damage the existing facade.
		High window percentage	Direct green facades are considered unsuitable, as they obstruct window cleaning.
		Continuous strip windows	Direct green facades are considered unsuitable, due to their vertical growing direction.
Interlocking shapes 	**Stacked volumes with varying facade geometries, heights and materials**		
	1. Building shape	Stacked volumes	*In case* of regularly shaded areas on the facade, modular and continuous living wall systems are recommended due to their high plant variety and density.
	2. Facade geometry	Planar facade	All VGS are suitable
		Curved tall facade section	Felt systems are recommended, as they can easily follow facade curvature and have no height limit.
		Curved short facade section	Felt and green facades are recommended, as they can easily follow facade curvature.
	3. Facade type	Concrete or masonry	Concrete or masonry is preferred for attaching all living wall systems and indirect green facades.
		Marble or masonry	Direct green facades are not recommended as they potentially damage the existing facade.
		High window percentage	Direct green facades are considered unsuitable, as they obstruct window cleaning and facade maintenance.
		Continuous strip windows	Direct green facades are considered unsuitable, due to their vertical growing direction.
Glass volumes 	**Varying building shapes with fully glazed or mirrored facades**		
	Facade type	Fully glazed or mirrored facade	Closed systems such as felt, container or box systems are considered unsuitable.
			Direct green facades are considered unsuitable, as they obstruct window cleaning and facade maintenance.
			Fastening of indirect green facades or curtain systems is difficult. Therefore, an independent substructure with a dedicated foundation is recommended.

Figure 7.10: Overview of the office building typology and corresponding impact on VGS suitability

7.7. Biodiversity performance

The biodiversity performance does not take specific building characteristics as input. Instead, it is based on the preferences of key animal groups considered important indicators of urban biodiversity, as identified primarily through empirical studies on VGS in chapter 3. VGS types have been evaluated based on specific biodiversity indicators, key system features that signal their potential to support urban biodiversity. These indicators include plant diversity, plant coverage, substrate size and the orientation of the substrate. This analysis forms the basis for the biodiversity ranking of representative VGS types, as shown in figure 7.11. The detailed analysis that lead to the biodiversity ranking can be found in chapter 3. Within each VGS type, a wide range of design variations exists. These variations also lead to different system weights, as illustrated in figure 7.9. To allow for meaningful comparison, the biodiversity ranking is based on representative variants of each VGS type, with descriptions included in figure 7.11.

It is important to note that the system with the highest weight within a given VGS type or representative variant does not necessarily offer the greatest biodiversity potential. This is because system weight is largely influenced by factors such as the materials used for growing units, substrates and mounting systems, aspects that are not identified as biodiversity indicators nor as distinctive system features in this framework. Therefore, the framework focuses on general system features that define a representative variant of each VGS type. This approach underpins the biodiversity ranking provided in figure 7.11.

Lastly, the ranking is presented on an ordinal scale. This means it should be interpreted as follows: for example, the representative variant of the container system has a greater biodiversity potential than that of the box system. A rank of 1 represents the highest biodiversity potential, while a rank of 5 indicates the lowest, relative to the other systems in the ranking. In this way, the biodiversity ranking provides insight into the relative biodiversity performance of the different VGS types, supporting the selection of the most suitable VGS option in the next step of the framework.

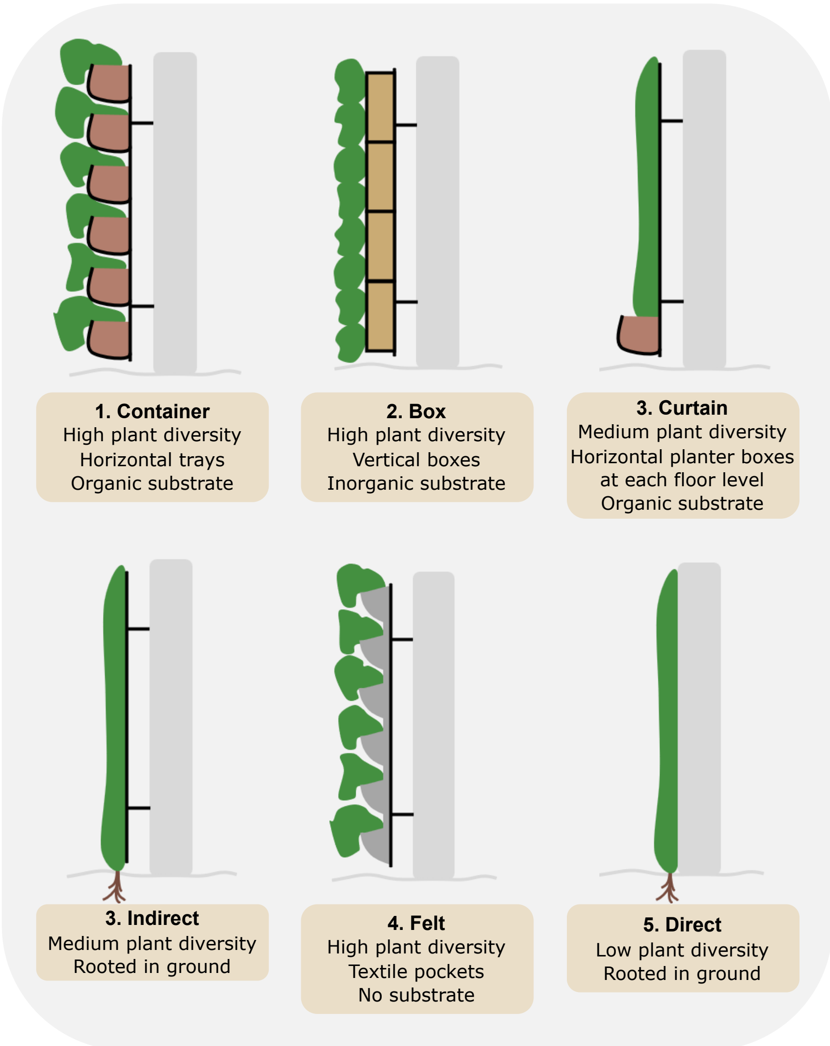


Figure 7.11: Biodiversity performance ranking of VGS

7.8. Ultimate VGS recommendation

The structural feasibility, combined with the recommendations based on building and biodiversity aspects, provides greater insight into the suitability of specific VGS options and serves to guide the feasibility phase. No specific method is prescribed for this step in the framework, as the building itself is a highly variable factor that cannot be fully assessed at this stage. This step is therefore entrusted to the user, with the confidence that the framework offers sufficient guidance to develop a well-substantiated recommendation for the feasibility phase.

8 Verification

The steps of the VGS implementation framework are verified through application to an illustrative case study, based on a hypothetical building. The aim is to evaluate whether the framework functions as intended and whether it aligns with the original research objective.

8.1. Introduction of illustrative case study

The case study uses a hypothetical building, which is first described in detail. Subsequently, the framework is applied step by step, followed by a reflection on its performance. In order to verify all steps of the framework, specifically the different knowledge levels, the associated knowledge at each level is assumed to be known during its respective step. Nevertheless, a full description of the hypothetical building is provided at the start of the analysis. Since the structural design of the building is known, the actual excess capacity can be calculated in detail according to the NEN 8700 series. This calculated value is presented at the start of the case study to allow for a later comparison with the outcomes of the excess capacity estimation. The detailed derivation of the actual excess capacity is explained in sections 6.5 and 6.6.1.

It should be noted that the permanent loads used for the hypothetical building are slightly higher than the permanent facade and floor loads used in the parameter study. This difference results from the assumption in the parameter study that only governing excess capacities were calculated, meaning finishing layers and insulation were disregarded in those calculations. By doing so, the loads do not exactly comply with the assumed loads in the parameter study, as considered common in practice. In addition, the floor design is based on design calculations for the bending moment capacity and consequently on resistance calculations to resemble the original design approach of the historic code as much as possible. The design calculations for the bending moment capacity have been provided in the following section and the resistance calculations of GBV 1962 are specified in section 6.3. It should be noted that design calculations are only available for bending moment capacity, not for shear capacity or deflection criteria.

8.2. Description of hypothetical building

The hypothetical building is a five-story office building located in an urban setting with planar facades, constructed in 1966. It features a concrete-framed structural system, which is not visible from the exterior. Columns are spaced at 6-meter intervals in both directions and the floors span parallel to the facade, supporting it at each floor level. The floors consist of one-way spanning solid concrete slabs, which are simply supported. The facade is 15 meters high and primarily constructed of classic red brick masonry, with approximately 60% glazed areas in the form of continuous strip windows. The building owner seeks to improve the building's biodiversity performance and has engaged an advisory firm with structural engineers to explore possible interventions, one of which is the implementation of a VGS. A more detailed look into the design variables of the structural elements results in the following:

- Variable floor load: $q_{k,GBV1962} = 2.5$ kN/m²
- Variable floor load: $q_{k,EC2012} = 3.7$ kN/m²
- Volumetric weight of concrete according to GBV 1962: $\gamma_c = 24$ kN/m³
- Volumetric weight of concrete according to EC 2012: $\gamma_c = 25$ kN/m³
- Facade weight: $g_{fac} = 1 \cdot 0.4 + 2.3 \cdot 0.6 + 0.2 = 2.0$ kN/m²
- Ceiling and floor finishes: 0.5 kN/m²
- Story height: 3 m
- Slab thickness: $t_{slab} = 210$ mm
- Main reinforcement diameter: $\phi_{main} = 16$ mm
- Number of reinforcement bars in 1 meter slab width: 9
- Excess capacity of the floor edge: $C_{F,slab,NEN870X} = 4.8$ kN/m

8.2.1. Design calculations

The required minimum effective height of the cross-section is calculated using equation (8.1), derived from [146, par. 3]. The required minimum reinforcement area is determined with equation (8.2), derived from [146, par. 3]. These design calculations are used to design the hypothetical slab. Subsequently, its combined bending moment capacity and shear capacity is verified with the equations provided in section 6.3.

$$d_{min} = \alpha \cdot \frac{M}{b} \quad \text{[mm]} \tag{8.1}$$

Where:

- $\alpha =$ coefficient in [-], calculated by equation (C.23)
- $M = M_{Ed}$, acting bending moment in [Nmm], calculated by equation (C.16)
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2

$$A_{min} = \frac{\bar{\sigma}'_b \cdot \eta}{2 \cdot \bar{\sigma}_a} \cdot b \cdot h \quad \text{[mm²]} \tag{8.2}$$

Where:

- $\bar{\sigma}'_b =$ allowable compressive strength of concrete in bending in [kgf/cm²], retrieved from table 6.4
- $\eta =$, coefficient, calculated by equation (C.22)
- $\bar{\sigma}_a =$ allowable tensile strength of steel in [kgf/cm²], retrieved from table 6.4
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $h = h_{beam}$ or t_{slab}, beam or slab thickness in [mm], defined in section 6.2.2

8.3. Application of the framework

This section demonstrates how the VGS implementation framework can be applied to a hypothetical office building. This application example focuses first on the excess capacity estimation for each knowledge level of the framework. After that, the results are compared to VGS system weights, followed by the analysis of office building suitability and the final recommendation.

8.3.1. Application of Assumption level

At the Assumption level, the available knowledge is limited to information derived from the building's exterior and BAG Viewer, resulting in only the construction year and an estimate of the glass percentage being known. In this case, the building was constructed in 1966 and has an estimated glass percentage of approximately 60%. The exterior shows brick masonry with strip windows in the facades, suggesting that the facades are likely non-load-bearing.

Excess capacity estimation

For the excess capacity estimation, the framework refers to figures 7.4, 7.5, 7.6, and 7.7. Since the structural element supporting the facade is unknown, both the beam and slab graphs for GBV 1962 are considered. Additionally, as the span of the element is not specified, a conservative assumption is made by focusing on the shortest span. Furthermore, the building exterior suggests that the floor heights are rather standard than generous, so a typical floor height of 3 meters is assumed. Given that slabs show generally lower excess capacities than beams, the evaluation begins with the slab graph of figure 7.6. Since the original characteristic load consists of a significant amount of both glass and masonry, the expected excess capacity lies between the light green and dark green trend lines in the figure. The light green line represents a fully glazed facade, while the dark green line represents a fully closed masonry facade. The midpoint between these two lines is proposed as a reasonable estimate at the shortest slab span, resulting in an excess capacity of approximately 2.3 kN/m. When this value is compared to the beam graph in figure 7.4, it closely aligns with the lowest excess capacity observed for beams with the shortest beam and slab span configuration. Therefore, the same value is retained for use in the next step of the framework.

8.3.2. Application of Basic level

At the Basic level, additional information is obtained through a site visit. While the exact insights gained depend on the specific building, the framework suggests that such a visit typically improves the understanding of the load-bearing system, the spans of slabs and/or beams, and the height and weight of the facade. A better understanding of the load-bearing system implies that structural elements such as beams or columns may be identified within the building, providing clues about structural spans. For the hypothetical building, columns are visible inside, but no beams can be observed. However, this does not necessarily mean beams are absent, they may be concealed by interior finishes. Based on the visible column placement and the strip window arrangement, the span of the facade-supporting element is estimated to be 6 meters. This same distance can be approximated in the perpendicular direction. Additionally, the floor-to-floor height of each level is estimated at 3 meters. As the facades consist of brick masonry on the exterior, considered the heaviest type in the framework, combined with a lightweight inner frame, the facade weight estimation is not expected to be further improved in this case. The glass percentage could be refined further if the initial estimation based on the building exterior at the assumption level was inaccurate.

Excess capacity estimation

For the excess capacity estimation, the framework refers again to figures 7.4, 7.5, 7.6, and 7.7. Since the structural element supporting the facade is unknown, both the beam and slab graphs for GBV 1962 are considered. The evaluation again starts with the slabs, as these show lower values for excess capacity and involve fewer parameters. Based on the estimated span of 6 meters and assuming a value between the two facade weight categories, figure 7.6 indicates an excess capacity of 3.2 kN/m. For the beam graph, a configuration with a 6 meter beam span, a 6 meter slab span and a 60% glass percentage yields an estimated excess capacity of approximately 5.6 kN/m. Since the 6 meter slab span is more difficult to read accurately in the graph, users may opt for the safer choice of using the 5.4 meter slab span instead, which corresponds to approximately 5.0 kN/m at 60% glass coverage. To remain fully on the conservative side, the lower estimation of the slab system of 3.2 kN/m is adopted.

8.3.3. Application of Intermediate level

At the Intermediate level, the original structural drawings are available, from which information about the load-bearing system, the dimensions of the elements and the facade height can be obtained. In this case, the structural drawings indicate the direction of the floor span and do not show any facade beams or other structural elements supporting the facade apart from the floor itself. Therefore, it can be concluded that the hypothetical building makes use of the slab system. In addition, the floor thickness and type are indicated on the drawing, revealing that a solid concrete slab of 210 mm is used. However, the facade weight cannot be specified further, as the drawing provides no additional information other than the use of masonry brick, which is similar to that used in the framework already. The facade height can be confirmed at 3 meters, which matches the earlier assumption.

Excess capacity estimation

For the excess capacity estimation, the framework refers to either the graphs from the Basic level or to the formula from the Intermediate level. Application of the graphs from the Basic level results in the same estimation of excess capacity as before, as this was related to the slab system as well. It should be noted that in calculating both characteristic loads, the conservative weights have been used, as no specific information is yet available regarding potential finish layers of either the facade or the floor. Estimating the excess capacity by calculation is done with equation (7.1) (also provided below), resulting in an estimated excess capacity of 3.0 kN/m.

$$C_{F,element,code} = (C_{x,element,code} + 1) \cdot s_{k,element,code} - s_{k,element,EC2012} \quad \text{[kN/m]}$$

Where:

- $C_{x,slab,GBV1962} = 0.34$, as specified in table 7.1
- $s_{k,slab,GBV1962} = 24 \cdot 0.21 \cdot 1 + 2.5 \cdot 1 + 1.8 \cdot 3 = 12.9$ [kN/m], calculated according to equation (6.3)
- $s_{k,slab,EC2012} = 25 \cdot 0.21 \cdot 1 + 3.7 \cdot 1 + 1.8 \cdot 3 = 14.4$ [kN/m], calculated according to summation of $g_{k,slab}$ and $q_{k,slab}$ as provided in equations C.12 and C.15

$$C_{F,slab,GBV1962} = (0.34 + 1) \cdot 12.9 - 14.4 = 3.0 \quad \text{[kN/m]}$$

8.3.4. Application of Advanced level

At the Advanced level, the original structural calculations are available, from which the original characteristic load can be applied in order to exactly fill in the formula provided in equation (7.1). This results in an excess capacity of 3.2 kN/m, which is considered the most reliable outcome, as it is based on the most building-related knowledge.

$$C_{F,element,code} = (C_{x,element,code} + 1) \cdot s_{k,element,code} - s_{k,element,EC2012} \quad \text{[kN/m]}$$

Where:

- $C_{x,slab,GBV1962} = 0.34$, as specified in table 7.1
- $s_{k,slab,GBV1962} = 24 \cdot 0.21 \cdot 1 + 2.5 \cdot 1 + 2.0 \cdot 3 = 13.5$ [kN/m], calculated according to equation (6.3)
- $s_{k,slab,EC2012} = 25 \cdot 0.21 \cdot 1 + 3.7 \cdot 1 + 2.0 \cdot 3 = 15.0$ [kN/m], calculated according to summation of $g_{k,slab}$ and $q_{k,slab}$ as provided in equations C.12 and C.15

$$C_{F,slab,GBV1962} = (0.34 + 1) \cdot 13.5 - 15.0 = 3.2 \quad \text{[kN/m]}$$

8.3.5. Application of practical warnings

As the hypothetical building employs standard office construction with solid rectangular concrete columns, beams and floors, the excess capacity estimation method provided by the framework can indeed be applied. In addition, the 6-meter spans in both directions, already known from the site visit at the Basic level, do not specifically indicate short or long beam spans, so no specific warnings from the framework need to be considered. However, since deflection checks are not included in the framework, this limitation should be noted during the site visit and kept in mind for subsequent design stages.

8.3.6. VGS Overview

The subsequent step is to compare the estimated excess capacity with the VGS weights of figure 7.9. All VGS types are considered possible with every excess capacity estimation, however not all variations within the VGS types. For this building, the excess capacity estimation does not pose any limitations on the feasible VGS.

8.3.7. Office building suitability

Certain building characteristics can be derived from the exterior: five stories, planar facades, 60% glass coverage and brick masonry for the opaque sections. These characteristics are used in figure 7.10 to classify the building within a building typology. They clearly identify the building as a functional box, where the facade geometry, building height and facade height are key indicators for VGS suitability. The building characteristics are evaluated as follows:

- The planar facades do not impose any limitations on the suitability for VGS implementation.
- The five story-characteristic of the building relates it more to low-rise than high-rise structure, which do not result in limitations or specific recommendations.
- The continuous strip windows make the building unsuitable for direct green facades, as their vertical growing pattern does not align with horizontally oriented window bands. Moreover, it is generally undesirable for climbing plants to grow across glazed surfaces.
- Direct green facades are also considered unsuitable for buildings with a high glass percentage.
- The masonry facade is also considered unsuitable for direct green facades, while being preferred by living wall systems.

This makes living wall systems the most suitable VGS type according to the building typology analysis.

8.3.8. Ultimate VGS recommendation

The excess capacity estimation, combined with the VGS overview, indicates that all VGS types are structurally feasible. The analysis of building characteristics recommends the implementation of living wall systems. Within that category, the container system is the most recommended based on the biodiversity ranking, while the box system is second in place. Therefore, according to the VGS implementation framework, the final recommendation for this hypothetical building at the feasibility phase is to further investigate the implementation of a container system or box system. However, as more specific information becomes available about the building or potential VGS options, the suitability and biodiversity performance should be reassessed to refine the feasibility assessment. In addition, the exact excess capacity should always be determined by a structural engineer in later stages, prior to the implementation of a VGS.

8.4. Reflections on framework performance

This section provides a reflection on the overall performance and usability of the framework. The aim is to evaluate how effectively the framework supports decision-making during the feasibility phase of VGS implementation and the accuracy of the excess capacity estimation. The key reflection points are outlined below:

- The framework includes all essential components as defined in the research objective. It offers a structured approach to assess both the structural feasibility, office building feasibility and the potential biodiversity impact of VGS on existing buildings during the feasibility phase.
- Although the framework does not provide detailed insight into the absolute biodiversity potential of each VGS type, the ranking does indicate their relative potential impact.
- The excess capacity estimation can be performed using the limited information expected during the feasibility phase, while remaining adaptable to the specific type of information available.
- The knowledge levels and corresponding excess capacity estimation graphs work together in a structured and insightful way. They were also easy to implement and reliability is increased with each level.
- The excess capacity estimation increases overall as more detailed knowledge of the structure becomes available, yet consistently remains well below the actual excess capacity in the floor edge. Based on this example, the estimation appears to be conservative and safe.
- The weight ranges in the VGS overview are less intuitive, as it is unclear what the upper and lower bounds represent or how they should be interpreted.
- The analysis of office building characteristics is less structured than the excess capacity estimation and slightly more subjective, but it still leads to well-founded recommendations.

9 Discussion

This section reflects on the main findings of the research and critically examines the methods, assumptions and limitations that may have influenced the results. The discussion is structured around the research questions, allowing each question to be addressed in relation to the corresponding outcomes and interpretations. This section aims to provide a balanced perspective on the strengths and weaknesses of the study and its potential contribution to the field of VGS implementation in existing buildings.

9.1. VGS Classification - RQ1

The first research question is as follows:

1. “What are the main types of VGS applicable to building facades?”

Based on the findings of this study, six main types of VGS suitable for application on building facades are identified and presented in figure 2.1. Their corresponding key features are listed in tables 2.1 and 2.2. Chapter 2 highlights that while various alternative terms for VGS appear in the literature, their definitions are often similar. However, the challenge becomes more significant once classifications are introduced. Not only is there no universally accepted classification system, but the definitions of similar terms often vary, and the same systems are referred to using different terminology. This inconsistency complicates the practical application of VGS, particularly when attempting to associate specific benefits or drawbacks with a given system. To address this, a clear and consistent classification based on distinguishable physical characteristics was developed for this study. The classification is designed to support the practical application of the implementation framework: once a suitable VGS type has been identified through the framework, users can match practical systems to the defined distinctive key features, namely, cultivation method, support structure and growing unit.

The classification was developed through an exploratory literature review, initiated from a comprehensive master’s thesis on VGS completed in 2023 [164]. As a result, much of the foundational literature dates from before this thesis. Additional sources were consulted selectively, focusing on areas where further detail was needed. While this approach may limit exposure to the latest developments, particularly innovations not yet embedded in research, it aligns with the scope of the study. The primary aim is to support biodiversity and only systems whose biodiversity potential has been assessed were included. An additional literature search on this topic was also conducted. Moreover, newer VGS types are often not yet examined empirically for their biodiversity performance, making their inclusion less suitable within the biodiversity-focused scope of this thesis.

Another limitation stems from the inconsistencies in the literature regarding VGS definitions, which also complicate the determination of system weights, an essential aspect of this study. While weights of more traditional systems such as direct facades are consistently reported, more complex systems vary widely. As newer systems are developed, many producers appear to prioritize reducing structural weight, which often results in reported weights that fall below those in the literature. To reflect this variability, the study uses weight ranges to represent the diversity of options within each VGS type.

Finally, the absence of a universal classification system is likely due to the wide variety of VGS developed globally over several decades, each with unique variations. Consequently, it is inevitable that some systems will not fit neatly into the six types identified in this study. Although further refinement or expansion of the classification is possible, this would go beyond the objective of this research: to provide a practical, biodiversity-oriented classification for VGS application on Dutch building facades. This also explains why studies with different aims often use alternative classification approaches.

9.2. Urban biodiversity potential - RQ2

The second research question is as follows:

2. “Which features of VGS serve as key indicators of their biodiversity performance?”

The biodiversity indicators identified in this study are presented in section 3.2.2 and include: plant coverage, plant diversity, substrate size and substrate orientation. While it is evident that many more aspects of VGS could influence their attractiveness to specific animal groups, this study aimed to establish a general biodiversity approach grounded in empirical evidence rather than assumptions or theoretical expectations, which are commonly found in the literature. The reliance on theoretical studies often leads to divergent conclusions, as many do not specify which species or climatic context their assumptions are based on [91, 47, 134]. This lack of specificity makes comparison difficult and can result in misleading generalizations. By focusing on empirical studies, this research seeks to mitigate those uncertainties. However, this approach is not without limitations. Not all VGS types identified in this thesis have been included in every empirical study, nor are the same animal groups consistently considered and the methodologies vary. Consequently, not every VGS is represented in the literature exactly as it is classified in this study, and not every animal group used in the MCA has been empirically linked to each system.

To address this challenge, the study shifts focus from system-level comparisons to feature-level biodiversity indicators. These indicators are defined as key features of VGS that indicate their biodiversity potential. This approach makes it possible to carefully apply findings from one system to others. If a certain feature is known to support a particular animal group in one system, it is assumed that similar systems with the same feature may offer comparable benefits. This assumption is admittedly a simplification, given the specificity of animal preferences. As ecological interactions are influenced by local conditions such as microclimate, species interactions and urban context, biodiversity performance cannot be wholly predicted by isolated physical features. Even in empirical studies, conclusions are not entirely independent of contextual factors such as climate and the presence of other species.

Additionally, the analysis shows that many animal groups exhibit plant-specific preferences, often at the species level, while others are more generally influenced by microclimatic factors. A key distinction between the systems lies in their planting strategy, green facades primarily use climbing plants, whereas living wall systems incorporate a broader range of plant types. Nevertheless, no clear preference for either planting type was identified across animal groups in the empirical literature reviewed [16, 87, 27, 121]. The plant-specific preferences are indirectly captured through broader features like plant coverage and diversity, already included in the selected biodiversity indicators.

Finally, it should be noted that the selection of empirical studies introduces some geographic limitations. Of the five studies used, one was conducted in Bogotá, Colombia, and focused solely on felt systems [16]. Due to the difference in climate and limited system scope, this study was only used for its insights on nesting behavior of birds in felt systems. The remaining studies were conducted in temperate European climates, offering greater applicability to the Dutch context.

9.3. Office building typology - RQ3

The third research question is as follows:

3. “Which characteristics of office buildings constructed between 1960 and 1990 indicate their suitability for VGS implementation?”

Building characteristics relevant to the suitability of VGS implementation are presented in figure 4.19. Both sides of the equation, the VGS systems and the buildings themselves, were analyzed to identify coherent and practical matching characteristics. The key physical features of VGS, as identified in chapter 2, were examined to derive building characteristics that could serve as indicators for implementation feasibility. However, as outlined in the methodological approach in appendix A, the academic literature discussing the application of VGS on existing buildings proved limited. To compensate for this, the study expanded its scope to include an expert interview and online sources, particularly from VGS producers. While these sources helped fill certain gaps, they also presented challenges. Most producers provide little concrete information on the fastening method of VGS, often redirecting such inquiries to contractors or stating it is too dependent on the building context. This reflects an underlying industry assumption that VGS are often implemented in new construction, where both facade and structural components can be customized to accommodate a chosen system. Nevertheless, some producers noted that their systems or anchoring mechanisms can be adapted to existing surfaces or are designed to be broadly compatible with most facade types [1, 141, 97, 54]. While such claims are encouraging for the retrofit context, they are often presented without detailed specifications or conditions, raising questions about their implementation. For one specific VGS type many mounting options remain, as one VGS type is still a collective term of more detailed systems, which subsequently results in not many building indicators. The classification used is too general to go into more depth and specify more building indicators. Moreover, the scope of this study did not include an exhaustive market survey of available VGS products. A dedicated market analysis could provide deeper insights into the diversity of mounting systems and fastening techniques. Contacting producers might help in this regard.

From an architectural perspective, building characteristics such as overall shape and facade materials typical for office buildings constructed between 1960 and 1990 were derived from literature. However, the available literature on architectural features from this period, particularly regarding facade types, was limited. To address this gap, a supplementary analysis was conducted using visual studies of representative office buildings [166, 173, 52, 59, 4] and consultations with experienced structural engineers (sections E.1 and E.2). While this approach provided valuable practical insights, it also introduces a level of subjectivity. The selected buildings may not fully represent the entire building stock and the interpretation of visual features is inherently dependent on the observer’s background. To provide an example, discussing the same building with different structural engineers resulted in different takes on the expected facade and building structure behind it.

The combination of literature, online field observations and expert opinion led to the definition of four building typologies. This number was intentionally kept limited to maintain practical applicability across a wide range of buildings. Although this makes the classification more accessible, it may also oversimplify the variation in the existing stock. More nuanced typologies might have captured subtle but relevant differences, yet this was not considered necessary, given that the suitability of VGS (in this study) is determined by a few dominant building characteristics, introduced as building indicators. For example, a fully glazed facade may rule out most VGS options, rendering other building properties irrelevant in the initial assessment. Conversely, a box-shaped office building may be suitable for nearly all VGS, shifting the focus to other criteria such as facade height or material. While this logic strengthens the usability of the typology approach, it also risks overlooking complex interactions between building features, especially in less conventional cases. Therefore, although the typologies provide a solid basis for guiding VGS selection, they should be applied with an awareness of their limitations and the potential need for project-specific refinement.

9.4. Excess capacity method - RQ4

The fourth research question is as follows:

4. “How can the excess capacity of slabs and beams supporting facades in existing Dutch office buildings be estimated during the feasibility phase?”

An extensive review was conducted on the official procedures for assessing existing buildings as outlined in the NEN 8700 series, in conjunction with the Eurocode 1990 series [115]. This methodology served as the foundation for the parameter study, in which illustrative beams and slabs were designed according to the standards GBV 1962 and VB 1974 and their excess capacity was subsequently derived. From this analysis, the official method was combined with an alternative approach using excess capacity factors to enable a quicker estimation of structural reserve capacity, one that is more suited to the early feasibility phase. The validated method allows for a simplified estimation: by adding one to the excess capacity factor, multiplying the result with the original characteristic load and subtracting the current characteristic load of the element, both expressed in kN/m.

However, several assumptions were made in the parameter study that influence the applicability of its results to all existing structures of the studied period. A key design principle applied was “design as they would”, meaning the structural elements were designed with a focus on realism and consistency, reflecting practices from the respective periods. This included optimizing material usage with high unity checks. At the same time, the parameter study aimed to determine the governing excess capacity. These objectives, realism, consistency, optimization and governing excess capacity, are not always fully compatible and required compromises. For instance, while structural elements were designed according to historical methods, modern optimization techniques were not available in the design periods considered. Moreover, the structural design books from the studied periods often included both design and resistance calculation procedures. A preliminary comparison showed that using the design calculations resulted in larger cross-sections, thus higher excess capacity, than using resistance calculations. For this reason, the resistance-based calculations were adopted, as the objective of estimating governing excess capacity was prioritized over replicating historical design procedures that were possibly applied. Nonetheless, while the resulting designs comply with the relevant code provisions and meet required unity checks, they may not fully reflect how each individual slab or beam from that period was actually designed.

Several simplifications were also made in the modeling assumptions. For example, floor systems were assumed to span in one direction only, although bidirectional spanning floors are also discussed in both the 1962 and 1974 codes [49, 156]. While the structural behavior at the slab edges may not differ significantly, this simplification still introduces a limitation to the practical applicability of the results. Similarly, all loads were assumed to be uniformly distributed, even though the codes require point load checks as well [49, 154]. The excess capacity factor method is only valid for a single load type, either uniformly distributed or point load. In real scenarios where a combination of load types was considered in the original design, the method might no longer yield reliable estimates. Furthermore, structural elements were modeled as simply supported, despite the fact that slabs in practice are often continuously supported, which generally results in lower bending moments. Applying the derived excess capacity factors to other support conditions without further research may therefore not be valid. Lastly, a material strength discrepancy exists between the two design standards used. The mean material strength (μ_{mat}) in GBV 1962 is 1.5 N/mm² higher than in VB 1974 [49, 154]. However, this difference was not incorporated into the excess capacity comparison, potentially skewing the comparative analysis.

The results of the parameter study, as discussed in section 6.8, indicate a clear upward trend between the original characteristic load and the derived excess capacity. However, several outliers, for both GBV 1962 and VB 1974 elements, raise questions about the consistency of the simplified method. These deviations can be mostly attributed to the strict adherence to historical design methods and the optimization of unity checks, both of which were done to maintain internal consistency within the study, but are most likely not necessary attributed to the “Design as they would” principle. For slabs, the trend in excess capacity factors shows a slight decline as the characteristic load increases. It remains unclear whether this is a meaningful pattern or a result from the design method. For instance, optimizing designs to unity checks of 0.99 may not accurately reflect typical practices of the time. Likewise, optimizing for both bending moment capacity and deflection simultaneously may not align with conventional design approaches. In practice, it is more likely that slabs were designed primarily for bending capacity, with deflection checks carried out afterwards. However, the unity checks for GBV 1962 slabs, as shown in table D.15, suggest that this is not the main issue. Instead, it appears that the additional weight used to increase the original characteristic load does not proportionally increase the excess capacity in slabs as it does in beams. This may be due to the geometric limitations of slabs compared to beams, as beams can be shaped and widened more effectively to resist bending moments. What can be concluded at this stage is that the excess capacity factors for slabs appear particularly sensitive to small variations in unity checks and overall design assumptions.

Beams, in contrast to slabs, were studied using a broader set of design variables, resulting in greater variation in the outcomes but an overall consistent trend in the excess capacity factor. However, the differences in shear reinforcement detailing and the calculation of shear capacity between the historic codes and the current NEN 8700 series introduce notable outliers. These deviations underpin several practical warnings discussed in section 7.4.4. In particular, the GBV 1962 beam designs exhibit a tipping point in shear capacity: as shear stress increases just beyond the limit at which concrete alone is no longer sufficient, but additional reinforcement is not yet required, the estimated excess capacity can drop sharply or deviate from the expected trend. Once the stress increases to the point where shear reinforcement becomes mandatory under GBV 1962, the capacity aligns with the trend again. This tipping point might not occur in real buildings if the original beam designs were not optimized to such extremes, something that warrants further investigation. Additionally, a detailed comparison of the two shear capacity methods (GBV 1962 and NEN 870X) indicates that the calculated shear stress to be resisted by the shear reinforcement may differ. Under GBV 1962, the shear force resisted by reinforcement is calculated over a specific distance y from the beam end, with variable stirrup spacing in that part of the beam [49]. For the remainder of the beam, the concrete shear capacity is relied upon, in combination with stirrups at maximum spacing. In contrast, the NEN 8700 series assumes a uniform distribution of shear capacity components and stirrup spacing along the full beam length [117, 113]. This difference in methodology can lead to significant mismatches in excess capacity estimation, particularly in configurations at the identified tipping points. Within the framework, this tipping point is reflected in the practical warnings related to specific combinations of beam and slab spans. These warnings emphasize the importance of a more detailed evaluation of the shear reinforcement by a structural engineer in such cases.

The VB 1974 beam designs exhibit more concerning outliers, particularly in configurations where large excess capacities would typically be expected due to high shear forces. In these cases, the concrete is fully utilized and the stirrup unity checks approach their design limits. However, both the stirrup and, more notably, the concrete shear capacities are lower under the current NEN 870X series than under VB 1974. As a result, reinforcement configurations that were considered sufficient under the historical code may prove inadequate when assessed using current standards, leading to an absence of excess capacity. This discrepancy underscores the importance of critically evaluating such configurations in practice. The framework therefore explicitly advises that beams subjected to high shear demands (or in simpler terms: long-span beams with large floor spans) be thoroughly reviewed by a structural engineer to ensure that the existing design still meets safety and performance requirements under present-day guidelines.

9.5. Developing the framework - RQ5

The fifth research question is as follows:

5. “How can biodiversity indicators, building characteristics and an excess capacity assessment be integrated into a framework for the feasibility phase of implementing VGS for biodiversity enhancement?”

This research question integrates the various outcomes derived from the previous research questions. While most of these components have been discussed individually in the preceding sections, the biodiversity ranking has not yet been addressed. This ranking was developed by applying a MCA, using animal groups as stakeholders to translate the biodiversity indicators into a comparative assessment of VGS types. The MCA process will be discussed below. Additionally, the framework as a whole will be critically reflected upon.

9.5.1. Biodiversity ranking of VGS

The aim of using the MCA is to compare the biodiversity performance of different VGS types and to provide recommendations within the framework for selecting suitable systems when the goal is to enhance urban biodiversity. In this analysis, the identified biodiversity indicators are used as criteria and each representative variant of the VGS types is evaluated using an ordinal scale. This means that the scores are relative rather than absolute: for each indicator, the analysis only reveals whether, for example, a container system performs better than a box system, not how much better. The ordinal scale was chosen because the representative variants are not exact products, but simplified representations of broader system categories. The scores are therefore based on the typical biodiversity potential associated with key physical characteristics of the systems, which correspond to the biodiversity indicators, rather than on measured performance data from specific implementations. An ordinal scale was chosen because the representative variants represent broader categories that include a variety of specific systems. The scores are therefore based on the potential performance associated with the key physical features of these systems, rather than a single defined product. Furthermore, the diversity in design among VGS types makes direct, quantitative comparison difficult. In addition, biodiversity is too complex to define clear minimum and maximum potential scores for each indicator, as different animal species have varying preferences regarding the biodiversity indicators. To reflect multiple perspectives, relevant animal groups from different taxa in the urban ecosystem were selected as stakeholders to assign weights to the biodiversity indicators. These weights were derived from empirical studies. However, the availability of such studies is limited, particularly in relation to birds, which are often considered indicator species in Dutch urban ecosystems. Data on Dutch bird species in relation to VGS, specifically, remains unfounded. While the empirical studies used were conducted in comparable European climates, such as in Paris and Staffordshire [27, 87], these are not identical to the Dutch context. So while the MCA provides multiple perspectives from key animal groups, it remains a simplification of the complex urban ecosystem. Its recommendations offer structured guidance for the feasibility phase, but do not capture the full spectrum of biodiversity preferences.

9.5.2. VGS implementation framework

The framework integrating biodiversity indicators, building characteristics and structural excess capacity assessment is presented in figure 7.1. The framework is based on anticipated knowledge levels regarding the structural features of a building, where higher levels of knowledge allow for more targeted and reliable estimations. The estimated excess capacity is then compared to the weights of the various VGS types to determine feasibility. While this comparison can, in some cases, exclude certain VGS from further consideration, the applied weight ranges are generally broad enough to prevent premature exclusions. The Office building suitability step relies solely on the building exterior as input, while the Biodiversity performance step is based entirely on the biodiversity indicators of the VGS types, requiring no building-specific input.

This framework is intentionally built upon basic attributes regarding buildings, VGS characteristics and biodiversity potential. This approach ensures broad applicability, particularly in early project stages. However, it also comes with a trade-off: the outcomes often remain quite open-ended, leaving many VGS types still viable at the end of the process. In some cases, a single feature, such as a fully glazed facade, could effectively override the outcomes of all other steps, even though a building owner or designer might be open to modifying or replacing that facade in a future renovation. For this reason, the framework deliberately avoids being overly prescriptive, preserving a wide range of options and flexibility for the user. This raises the question of whether such an open-ended framework is truly useful. A stricter, more exclusionary process could systematically narrow down VGS options at each step. However, this might prematurely eliminate viable solutions, particularly in cases where a building owner is open to structural or architectural modifications and is seeking broad insights on what options to consider. In practice, the initial idea for this framework was developed in collaboration with Arup, with a specific focus on its application during the feasibility phase. A more rigid or deterministic version of the framework may be more appropriate for a subsequent design phase, once feasibility has been confirmed and more precise constraints are known. However, a more deterministic approach would require reducing complexity, for instance, by treating certain VGS or building types as ambassadors for broader groups, which risks overlooking the variation and nuance across real-world cases.

9.6. Verification of framework - RQ6

The sixth research question is as follows:

6. "How can a framework designed to support the feasibility phase of implementing VGS for biodiversity enhancement be verified?"

The framework has been verified through an illustrative case study, in which it was applied step-by-step using a hypothetical office building. This application demonstrated that the framework functions as intended: it guides the user through the feasibility phase by structuring limited input data, providing intermediate outputs and resulting in a well-founded VGS recommendation. The case study also confirmed that the framework includes all necessary components outlined in the research objective, an assessment on structural feasibility, architectural feasibility and a general insight in biodiversity potential of VGS. While the framework proved usable and insightful in this context, its verification remains limited in scope, as it has not yet been tested on real buildings or by external users. Therefore, further testing is required to fully validate its robustness and practical value.

9.7. Concluding synthesis - Main RQ

The main research question guiding this thesis is:

> "How can a framework be developed to support decision-making on the feasibility and selection of VGS for existing Dutch office buildings aiming to enhance urban biodiversity?"

To answer this question, a framework was developed that integrates biodiversity indicators, building characteristics and an excess capacity assessment. Each component was derived from a guided sub-question and discussed in the sections above. The resulting framework enables users to explore VGS implementation during the feasibility phase of enhancing a building's biodiversity performance. By building on basic, broadly applicable parameters, the framework remains adaptable to a wide variety of office buildings, while still offering structured guidance on the feasibility of different VGS types. This open-ended character is intentional: it reflects the practical uncertainties and decision space in feasibility studies. Rather than excluding options too early, the framework encourages informed exploration, while highlighting which factors, such as structural limits or biodiversity goals, may ultimately constrain design choices. As such, the framework serves not as a definitive decision tool, but as a structured support tool that bridges urban biodiversity ambition and technical feasibility in the earliest stages of renovation projects. Further refinement of the framework could involve developing more deterministic approaches for specific building typologies or VGS system types, but such extensions are likely more appropriate in later design phases when detailed data becomes available.

10 Conclusions & Recommendations

This chapter presents the conclusions derived from the research. It summarizes the key findings resulting from the various analysis performed throughout the study. Following the conclusions, several recommendations are proposed to support future work in the field of biodiversity enhancing retrofit strategies.

10.1. Conclusions

The conclusions from this research are outlined below:

- **All VGS types included in this study contribute to urban biodiversity.**
  Chapter 3 points out that all VGS included in this research are attractive to one or multiple key animal groups of the urban ecosystem.
- **Plant coverage, plant diversity, substrate size and substrate orientation are considered key features indicating the urban biodiversity potential of VGS.**
- **Building height, building shape, facade geometry, facade glass percentage and facade type are considered key building characteristics indicating the suitability of VGS implementation during the feasibility phase.**
- **Given the building-related uncertainties in the feasibility phase and the variation in VGS types, only general exterior suitability recommendations are possible, conclusive assessments require more detailed input.**
- **The use of excess capacity factors offers a viable approach for estimating the excess capacity of beams and slabs supporting facades during the feasibility phase.**
  Chapter 6 validates the use of an excess capacity factor.
- **An open-end framework is proposed to guide VGS exploration during the feasibility phase to allow for decision space and practical uncertainties.**

10.2. Recommendations for further research

To further enhance the applicability and the robustness of the proposed framework, several directions for future research can be derived. These recommendations aim to address current limitations and to provide more detail to investigated topics.

- Conducting a more extensive statistical analysis to better evaluate the safety level associated with the proposed excess capacity estimation method.
- The statistical reliability of the parameter study could be improved by expanding the dataset, particularly for slabs with high characteristic loads, where an unexpected slight decline in excess capacity factors was observed with increasing characteristic load. A larger dataset would allow for a more detailed investigation of this trend.

- Outliers observed in the parameter study suggest that excess capacity in slabs is particularly sensitive to governing failure modes and design assumptions. Their geometric limitations, compared to the more efficient load-bearing shape of beams, may contribute to lower excess capacity. Future research is recommended to further investigate these effects and better understand the excess capacity of slabs.
- Outliers observed in the parameter study indicate that differences in shear reinforcement design and resistance calculation between historical and current codes have a notable impact on the estimated excess capacity of beams. Future research could examine these differences in more detail to better understand their influence on capacity estimations and further refine the framework.
- In two of the buildings included in the study with curtain facades, load-bearing facades were found behind them. This suggests that load-bearing walls may have been used more frequently for facades than initially concluded from the literature. Additionally, many VGS types are closed systems suited for closed walls, which are often associated with load-bearing structures. Although load-bearing walls are not expected to be a governing factor, including them in the framework would improve its completeness. It is therefore recommended to include load-bearing walls in a similar parameter study to evaluate their excess capacity. This would also make the framework more broadly applicable, for instance to residential buildings, which often incorporate load-bearing walls.
- The framework is currently applicable only to concrete office buildings constructed between 1962 and 1990. However, as safety margins have decreased over time, extending the scope to include more recent buildings would be valuable. In newer constructions, this reduction in safety margins makes it increasingly uncertain whether implementation of VGS is possible.
- To enhance the applicability of the framework, it is recommended to develop excess capacity factors for other structural materials, such as steel and possibly timber.
- The framework currently focuses solely on Vertical Greening Systems, while green roofs have also demonstrated strong potential for enhancing biodiversity [47]. Moreover, since the variable loads on roofs have remained consistently low over time, the structural excess capacity may be limited [61, 108, 118]. Including green roofs in future studies could expand the framework's applicability.
- The framework as a whole could be validated by applying it to actual case studies rather than illustrative ones.
- Empirical studies on the biodiversity impact of VGS, especially on the relationship between birds and living wall systems, are currently lacking, even though these systems offer significant potential. Birds form an important part of the Dutch urban ecosystem [7, 17] and with growing requirements for nature based solutions in new construction and even renovations [145], investigating this relation becomes particularly relevant.
- As biodiversity performance cannot be captured in a single score, expert interviews could provide additional insights and serve as a basis for validating the biodiversity ranking.
- A market study focused on VGS systems available in the Dutch market could help specify system weights and provide more detailed information on fastening techniques, allowing for a more precise match with various facade configurations. In addition to online brochures, producers could be contacted directly to obtain further technical information.
- To extend the research on existing office buildings from the studied period, archival information could be used to gain more insight into the structural systems and facade configurations of that time, for which limited information is currently available.

References

[1] 90deGREEN. *90deGREEN, der Garten für die Wand*. Online brochure, 90deGREEN. Accessed on 28 april, 2025. 2018. URL: https://www.90degreen.com/90degreen/.

[2] Noor Aghina, Ingrid Sloots, and Timo Roeke. *Natuurinclusief ontwikkelen*. 2023. URL: https://synchroon.nl/nieuws/synchroon-lanceert-gids-natuurinclusief-ontwikkelen/.

[3] Md. Mahmudul Alam et al. "Relationships among carbon emissions, economic growth, energy consumption and population growth: Testing Environmental Kuznets Curve hypothesis for Brazil, China, India and Indonesia". In: *Ecological Indicators* 70 (2016). Navigating Urban Complexity: Advancing Understanding of Urban Social – Ecological Systems for Transformation and Resilience, pp. 466–479. ISSN: 1470-160X. DOI: https://doi.org/10.1016/j.ecolind.2016.06.043. URL: https://www.sciencedirect.com/science/article/pii/S1470160X16303685.

[4] Alberts & Van Huut. *Organic architecture with wellbeing in mind*. Accessed on April 30, 2025. 2017. URL: https://www.discoverbenelux.com/alberts-van-huut/.

[5] ANP. *De Nederlandsche Bank overweegt goudvoorraad te verhuizen*. Accessed on April 29, 2025. Feb. 2016. URL: https://www.nu.nl/economie/4219753/de-nederlandsche-bank-overweegt-goudvoorraad-te-verhuizen.html.

[6] Arup. *Cities Alive; Rethinking green infrastructure*. Tech. rep. Arup, 2024. URL: https://www.arup.com/insights/cities-alive-rethinking-green-infrastructure/.

[7] Jurrian Bakker. "(Natuurinclusief) bouwen aan de toekomst van icoonsoorten in stedelijk gebied". Eindscriptie van de Hogeschool. Almere, Netherlands: Aeres Hogeschool Almere, 2021.

[8] Barnes and Robert D. *Arthropod | Definition, Examples, Characteristics, Classes, Groups, & Facts*. Aug. 1998. URL: https://www.britannica.com/animal/arthropod.

[9] Brad Bass and Bas Baskaran. "Evaluating rooftop and vertical gardens as an adaptation strategy for urban areas". In: (2003).

[10] Daan Beeren. "Verticale Groensystemen op de voorgevel van de faculteit Civiele Techniek en Geowetenschappen". Bachelor's Thesis. Delft, Netherlands: Delft University of Technology, 2024.

[11] Joep van Belkom and Tanja Martens. *Toolbox natuurinclusief bouwen*. https://www.nijmegen.nl/diensten/bouwen-en-wonen/natuurinclusief-bouwen/. 2023. (Visited on 10/04/2024).

[12] Ahmet B. Besir and Erdem Cuce. "Green roofs and facades: A comprehensive review". In: *Renewable and Sustainable Energy Reviews* 82 (2018), pp. 915–939. ISSN: 1364-0321. DOI: https://doi.org/10.1016/j.rser.2017.09.106. URL: https://www.sciencedirect.com/science/article/pii/S1364032117313680.

[13] J.G. Bijlaard. *Toepassing van NEN 8700 bij betonreparatie en -onderhoud*. Tech. rep. TNO, 2012. URL: https://www.tno.nl/media/5800/toepassing_van_nen_8700_bij_betonreparatie_en_onderhoud.pdf.

[14] Biotecture. *Biotecture Living Green Wall Specification*. Accessed on April 23, 2025. 2025. URL: https://www.biotecture.uk.com/design-and-specify/specifications-and-compliance/specifications-and-drawings/.

[15] Patrick Blanc. *Quai Branly Jacques Chirac Museum*. Accessed on 24 april, 2025. 2006. URL: https://www.verticalgardenpatrickblanc.com/realisations/paris/quai-branly-jacques-chirac-museum.

[16] E.I. Bolhuis. *Bird use of living walls in the city of Bogotá, Colombia*. 2016. URL: https://repository.udca.edu.co/handle/11158/494.

[17] B van den Boogaard, R.G. Verbeek, and J.D. Buizer. *Icoonsoorten Zuid-Holland*. Tech. rep. Rapportnr. 19-015. Culemborg, nl: Provincie Zuid-Holland, July 2019.

[18] G.H. van Boom and J.W. Kamerling. *Construeren in gewapend beton 2*. Netherlands: Elsevier, 1978.

[19] Jules F.F.P. Bos, A. (Bert) L. Smit, and Jaap J. Schröder. "Is agricultural intensification in The Netherlands running up to its limits?" In: *NJAS - Wageningen Journal of Life Sciences* 66 (2013). On careful livestock farming, pp. 65–73. ISSN: 1573-5214. DOI: https://doi.org/10.1016/j.njas.2013.06.001. URL: https://www.sciencedirect.com/science/article/pii/S1573521413000183.

[20] Karen Botes and Christina Breed. "Outdoor living wall systems in a developing economy: A prospect for supplementary urban food production?" In: *Acta Structilia* 28.2 (Nov. 2021), pp. 143–169. DOI: 10.18820/24150487/as28i2.6. URL: https://doi.org/10.18820/24150487/as28i2.6.

[21] C.R. Braam and P. Lagendijk. *Constructieleer Gewapend Beton*. Vol. 7. AEneas, 2011.

[22] Jürgen Breuste et al. *Making Green Cities: Concepts, Challenges and Practice*. Cham, Switzerland: Springer Nature Switzerland AG, 2023. DOI: 10.1007/978-3-030-37716-8.

[23] Marta Brković Dodig, Mina Radic Sibinovic, and Thomas Auer. "Green Facades and Living Walls—A Review Establishing the Classification of Construction Types and Mapping the Benefits". In: *Sustainability* 11 (Aug. 2019). DOI: 10.3390/su11174579.

[24] Christiaan de Bruine. *Herinrichting Hoofdkantoor Randstad Diemen*. Accessed on May 4, 2025. 2025. URL: https://www.qwa.nl/projecten/randstad/.

[25] Rosmina A. Bustami et al. "Vertical greenery systems: A systematic review of research trends". In: *Building and Environment* 146 (2018), pp. 226–237. ISSN: 0360-1323. DOI: https://doi.org/10.1016/j.buildenv.2018.09.045. URL: https://www.sciencedirect.com/science/article/pii/S0360132318306036.

[26] Nathalie Butt et al. "Opportunities for biodiversity conservation as cities adapt to climate change". In: *Geo Geography and Environment* 5.1 (Jan. 2018). DOI: 10.1002/geo2.52. URL: https://doi.org/10.1002/geo2.52.

[27] Caroline Chiquet. "The Animal Biodiversity Of Green Walls In The Urban Environment". PhD dissertation. Staffordshire, England: Staffordshire University, 2014. URL: https://core.ac.uk/download/pdf/43609063.pdf.

[28] Caroline Chiquet, John W. Dover, and Paul Mitchell. "Birds and the urban environment: the value of green walls". In: *Urban Ecosystems* 16.3 (Nov. 2012), pp. 453–462. DOI: 10.1007/s11252-012-0277-9. URL: https://doi.org/10.1007/s11252-012-0277-9.

[29] Eric Chivian et al. "Biodiversity: its importance to human health". In: *Center for Health and the Global Environment, Harvard Medical School, Cambridge, MA* 23 (2002).

[30] Rebecca Clausen and Richard York. "Global biodiversity decline of marine and freshwater fish: A cross-national analysis of economic, demographic, and ecological influences". In: *Social Science Research* 37.4 (2008), pp. 1310–1320. ISSN: 0049-089X. DOI: https://doi.org/10.1016/j.ssresearch.2007.10.002. URL: https://www.sciencedirect.com/science/article/pii/S0049089X07000786.

[31] Rebecca Collins, Marije Schaafsma, and Malcolm D. Hudson. "The value of green walls to urban biodiversity". In: *Land Use Policy* 64 (2017), pp. 114–123. ISSN: 0264-8377. DOI: https://doi.org/10.1016/j.landusepol.2017.02.025. URL: https://www.sciencedirect.com/science/article/pii/S0264837716310778.

[32] CPE. *SemperGreenwall*. Accessed on July 7, 2025. 2025. URL: https://www.cpe.be/nl/onze-merken/sempergreenwall.

[33] D.D. de Gunst and T. de Jong. *Typologie van gebouwen: Planning en ontwerp van kantoorgebouwen*. Delft, Nederland: Delft Universitaire Pers, 1989.

[34] J.J. Dekker. "The effect of applying greenery systems on a building's sustainability performance". Master's Thesis. Rotterdam, Netherlands: Delft University of Technology, 2024.

[35] Dieuwertje den Hartog. "Valuing vertical green". Master's Thesis. Utrecht, Netherlands: Delft University of Technology, Wageningen University, 2022.

[36] Elizabeth Devon Deutsch-Aboulmahassine. *Modular, wall-mounted plant growing system*. 2009. URL: https://www.freepatentsonline.com/7627983.html.

[37] Sandra Díaz and Yadvinder Malhi. "Biodiversity: Concepts, Patterns, Trends, and Perspectives". In: *Annual Review of Environment and Resources* 47.1 (Sept. 2022), pp. 31–63. DOI: 10.1146/annurev-environ-120120-054300. URL: https://doi.org/10.1146/annurev-environ-120120-054300.

[38] DonkerGroep. *Groene gevel*. 2025. URL: https://donkergroep.com/nl/donker-groen/groenaanleg/groene-gevel/.

[39] Nigel Dudley and Sasha Alexander. "Agriculture and biodiversity: a review". In: *Biodiversity* 18.2-3 (2017), pp. 45–49. DOI: 10.1080/14888386.2017.1351892. URL: https://doi.org/10.1080/14888386.2017.1351892.

[40] P.K. Dunn. *Scientific Research and Methodology*. Accessed on June 9, 2025. 2021. URL: https://bookdown.org/pkaldunn/Book/.

[41] Adel Samy El Menshawy, Abdelaziz Farouk Mohamed, and Nayera Mahmoud Fathy. "A comparative study on green wall construction systems, case study: South valley campus of AASTMT". In: *Case Studies in Construction Materials* 16 (2022), e00808. ISSN: 2214-5095. DOI: https://doi.org/10.1016/j.cscm.2021.e00808. URL: https://www.sciencedirect.com/science/article/pii/S2214509521003235.

[42] G. van Engelen, M.M. Vola, and H.P. Zuidwijk. *Berekening dwarskracht*. Accessed on May 23, 2025. Cement Online. 2005. URL: https://www.cementonline.nl/artikelen/berekening-dwarskracht.

[43] Rafael Fernández-Cañero, Luis Pérez Urrestarazu, and Katia Perini. "Chapter 2.1 - Vertical Greening Systems: Classifications, Plant Species, Substrates". In: *Nature Based Strategies for Urban and Building Sustainability*. Ed. by Gabriel Pérez and Katia Perini. Butterworth-Heinemann, 2018, pp. 45–54. ISBN: 978-0-12-812150-4. DOI: https://doi.org/10.1016/B978-0-12-812150-4.00004-5. URL: https://www.sciencedirect.com/science/article/pii/B9780128121504000045.

[44] Christian Flatscher. *ASI Reisen Headquarters*. Accessed on 24 april, 2025. 2020. URL: https://www.world-architects.com/en/architecture-news/works/asi-reisen-headquarters.

[45] Carl Folke et al. "Ecosystem Appropriation by Cities". In: *Royal Swedish Academy of Sciences* 26.3 (1997). URL: https://citeseerx.ist.psu.edu/document?repid=rep1&type=pdf&doi=c0b9c12521a88716daa19daabcf1d8b5de44f41a.

[46] Forest Service | US Department of Agriculture. *Who Are the Pollinators?* URL: https://www.fs.usda.gov/managing-land/wildflowers/pollinators/who-are-the-pollinators.

[47] Robert A. Francis and Jamie Lorimer. "Urban reconciliation ecology: The potential of living roofs and walls". In: *Journal of Environmental Management* 92.6 (2011), pp. 1429–1437. ISSN: 0301-4797. DOI: https://doi.org/10.1016/j.jenvman.2011.01.012. URL: https://www.sciencedirect.com/science/article/pii/S0301479711000132.

[48] Yutaka Fukuzumi and Individual. *US5579603A - Plant growing method for greening wall surfaces*. Apr. 1995. URL: https://patents.google.com/patent/US5579603A/en.

[49] Gewapend-Betoncommissie van het Koninklijk Instituut van Ingenieurs, Afdeling voor Bouw- en Waterbouwkunde. *Gewapend-Betonvoorschriften GBV 1962. NEN 1009*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1962.

[50] Green Cable by Carl Stahl ARC GMB. *Greenery with rope systems and wall brackets*. Accessed on April 22, 2025. 2025. URL: https://greencable.eu/greenery-with-rope-systems-and-wall-brackets/.

[51] greenroofs. *Caixa Forum Museum Vertical Garden*. Accessed on July 7, 2025. 2025. URL: https://www.greenroofs.com/projects/caixa-forum-museum-vertical-garden/.

[52] Groosman. *Beurs WTC Rotterdam*. Accessed on May 5, 2025. 2025. URL: https://groosman.com/beurs-wtc/.

[53] Hawaii Island Rat Lungworm Working Group, Daniel K. Inouye, and University of Hawaii. *Taxonomy: Classification of slugs and snails*. Tech. rep. University of Hawaii. URL: https://pharmacy.uhh.hawaii.edu/documents/rat-lungworm/4.Taxonomy_Classification_lesson.pdf.

[54] GSky. *Exterior Climbing Vine System*. Online brochure, GSky. Accessed on 28 april, 2025. 2025. URL: https://www.gsky.com/cads-specs.

[55] GSky. *Exterior System Versa XT Trays*. Online brochure, GSky. Accessed on April 23, 2025. 2025. URL: https://www.gsky.com/cads-specs.

[56] John T Hardy. *Climate change: causes, effects, and solutions*. John Wiley & Sons, 2003.

[57] Douglas M. Hawkins. *Identification of outliers*. Vol. 11. Springer, 1980.

[58] Heijmans. *Heijmans to revitalise Rabobank HQ in Utrecht*. Accessed on April 30, 2025. 2021. URL: https://www.heijmans.nl/en/news/heijmans-to-revitalise-rabobank-hq-in-utrecht/.

[59] Het grote verhaal van Apeldoorn. *Opening Centraal Beheer*. Accessed on April 29, 2025. 2022. URL: https://www.hetgroteverhaalvanapeldoorn.nl/gebeurtenissen/opening-centraal-beheer.

[60] Jutta Hollands and Azra Korjenic. "Evaluation and Planning Decision on Façade Greening Made Easy—Integration in BIM and Implementation of an Automated Design Process". In: *Sustainability* 13.16 (2021). ISSN: 2071-1050. DOI: 10.3390/su13169387. URL: https://www.mdpi.com/2071-1050/13/16/9387.

[61] HoofdCommissie voor de Normalisatie in Nederland (HCNN). *Technische grondslagen voor bouwvoorschriften TGB 1955. N 1055*. Netherlands: Centraal NormalisatieBureau (CNB), 1955.

[62] M.E.C.M. Hop and J.A. Hiemstra. "CONTRIBUTION OF GREEN ROOFS AND GREEN WALLS TO ECOSYSTEM SERVICES OF URBAN GREEN". In: *Acta Horticulturae* 990 (May 2013), pp. 475–480. DOI: 10.17660/actahortic.2013.990.61. URL: https://doi.org/10.17660/actahortic.2013.990.61.

[63] Felice Hunnekens. "Biodiversity in Building Environments: An In-Depth approach to stimulate biodiversity at a building Scale". Master's Thesis. Utrecht, Netherlands: Utrecht University, 2021.

[64] Satu Huuhka. "Understanding demolition". In: *Buildings and Cities* 4.1 (Jan. 2023), pp. 927–937. DOI: 10.5334/bc.398. URL: https://doi.org/10.5334/bc.398.

[65] Iowa State University. *Order Coleoptera - Beetles*. 2025. URL: https://bugguide.net/node/view/60.

[66] Jakob Rope Systems. *4 points for successful greening*. Accessed on 28 april, 2025. Oct. 2021. URL: https://www.jakob.com/au/en/news/4-points-for-successful-greening.

[67] Jakob Rope Systems. *Catalog GreenSolutions G2 Technical*. Online brochure, Jakob Rope Systems. Accessed on 22 april, 2025. 2025. URL: https://www.jakob.com/ch/en/applications/greening.

[68] Jakob Rope Systems. *GreenBlock: High-load bracket for ETICS facades*. Accessed on 28 april, 2025. Aug. 2023. URL: https://www.jakob.com/be/en/news/greenblock-high-load-bracket-for-etics-facades.

[69] C.Y. Jim. "Assessing growth performance and deficiency of climber species on tropical greenwalls". In: *Landscape and Urban Planning* 137 (2015), pp. 107–121. ISSN: 0169-2046. DOI: https://doi.org/10.1016/j.landurbplan.2015.01.001. URL: https://www.sciencedirect.com/science/article/pii/S0169204615000031.

[70] C.Y. Jim. "Greenwall classification and critical design-management assessments". In: *Ecological Engineering* 77 (2015), pp. 348–362. ISSN: 0925-8574. DOI: https://doi.org/10.1016/j.ecoleng.2015.01.021. URL: https://www.sciencedirect.com/science/article/pii/S0925857415000348.

[71] Henk Jonkers and Marc Ottelé. *Introduction to Cross Over module Sustainable Cities: Eco-engineering solutions for climate resilient and healthy cities*. CEGM2001, Cross over module Sustainable cities, The master Civil Engineering, Delft University of Technology. 2023.

[72] Nadja Kabisch et al. *Nature-based solutions to climate change adaptation in urban areas: Linkages between science, policy and practice*. Springer Nature, 2017.

[73] Kadaster. *BAG Viewer*. Accessed on April 30, 2025. 2025. URL: `https://bagviewer.kadaster.nl`.

[74] Laura Kleerekoper, Marjolein Van Esch, and Tadeo Baldiri Salcedo. "How to make a city climate-proof, addressing the urban heat island effect". In: *Resources, Conservation and Recycling* 64 (2012), pp. 30–38.

[75] Manfred Köhler. "Green facades—a view back and some visions". In: *Urban Ecosystems* 11.4 (May 2008), pp. 423–436. DOI: `10.1007/s11252-008-0063-x`. URL: `https://doi.org/10.1007/s11252-008-0063-x`.

[76] Koopstek. *Dutchport*. Accessed on April 30, 2025. 2025. URL: `https://koopenhuurstek.nl/54-9252/dutchport`.

[77] F.B. Koster. "RENOVAIR gevelsysteem; ontwerp tweede-huidgevel voor toepassing op verouderde hoogbouw uit de utiliteitssector". Master's Thesis. Eindhoven, Netherlands: Eindhoven University of Technology, 2007.

[78] Rik Kwakman. "Timber on top. A parametric exploration of CLT vertical extensions in the Rotterdamse Laag". Master's Thesis. Delft, Netherlands: Delft University of Technology, 2024.

[79] Gaëtane Le Provost et al. "The supply of multiple ecosystem services requires biodiversity across spatial scales". In: *Nature Ecology & Evolution* 7 (Nov. 2022). DOI: `10.1038/s41559-022-01918-5`.

[80] Nicholas Lepp. "Planting Green Roofs and Living Walls". In: *Journal of Environmental Quality - J ENVIRON QUAL* 37 (Nov. 2008). DOI: `10.2134/jeq2008.0016br`.

[81] Ho Ning Li. *Accelerating action for biodiversity: what the built environment sector needs to do*. Nov. 2021. URL: `https://www.gresb.com/nl-en/accelerating-action-for-biodiversity-what-the-built-environment-sector-needs-to-do/`.

[82] Jacques Linssen. *Sterkte? Welke sterkte?* Accessed on May 20, 2025. 2018. URL: `https://www.cementonline.nl/artikelen/sterkte-welke-sterkte`.

[83] LiveWall LLC. *LiveWall Components*. Accessed on 23 april, 2025. 2022. URL: `https://livewall.com/technical/components/#installer-sourced`.

[84] LiveWall LLC. *Living Wall Invites Beneficial Insects and Birds*. Mar. 2019. URL: `https://livewall.com/portfolio-items/birds-in-livewall/`.

[85] Alessandro Lo Faro et al. "Vertical Greenery Systems for Building Stock". In: *Proceedings of the 11th International Conference of Ar.Tec. (Scientific Society of Architectural Engineering)*. Ed. by Rossella Corrao et al. Cham: Springer Nature Switzerland, 2025, pp. 488–501. ISBN: 978-3-031-71867-0.

[86] Nadine Maarhuis. *The Pollinators: Tom van de Beek*. Oct. 2024. URL: `https://re-generation.cc/pionier/tom-van-de-beek-the-pollinators/`.

[87] Frédéric Madre et al. "Building biodiversity: Vegetated façades as habitats for spider and beetle assemblages". In: *Global Ecology and Conservation* 3 (2015), pp. 222–233. ISSN: 2351-9894. DOI: `https://doi.org/10.1016/j.gecco.2014.11.016`. URL: `https://www.sciencedirect.com/science/article/pii/S2351989414000869`.

[88] Maria Manso and João Castro-Gomes. "Green wall systems: A review of their characteristics". In: *Renewable and Sustainable Energy Reviews* 41 (2015), pp. 863–871. ISSN: 1364-0321. DOI: `https://doi.org/10.1016/j.rser.2014.07.203`. URL: `https://www.sciencedirect.com/science/article/pii/S1364032114006637`.

[89] Flavie Mayrand and Philippe Clergeau. "Green Roofs and Green Walls for Biodiversity Conservation: A Contribution to Urban Connectivity?" In: *Sustainability* 10.4 (Mar. 2018), p. 985. DOI: `10.3390/su10040985`. URL: `https://doi.org/10.3390/su10040985`.

[90] Flavie Mayrand et al. "Chapter 3.13 - Vertical Greening Systems as Habitat for Biodiversity". In: *Nature Based Strategies for Urban and Building Sustainability*. Ed. by Gabriel Pérez and Katia Perini. Butterworth-Heinemann, 2018, pp. 227–237. ISBN: 978-0-12-812150-4. DOI: https://doi.org/10.1016/B978-0-12-812150-4.00021-5. URL: https://www.sciencedirect.com/science/article/pii/B9780128121504000215.

[91] Alexandra Medl, Rosemarie Stangl, and Florin Florineth. "Vertical greening systems – A review on recent technologies and research advancement". In: *Building and Environment* 125 (2017), pp. 227–239. ISSN: 0360-1323. DOI: https://doi.org/10.1016/j.buildenv.2017.08.054. URL: https://www.sciencedirect.com/science/article/pii/S0360132317304018.

[92] Luke Miller. *Is It Safe to Plant Climbing Vines On Your House?* Accessed on April 23, 2025. 2024. URL: https://www.familyhandyman.com/article/vines-on-house/.

[93] Sarah Milliken. "Chapter 1.2 - Ecosystem Services in Urban Environments". In: *Nature Based Strategies for Urban and Building Sustainability*. Ed. by Gabriel Pérez and Katia Perini. Butterworth-Heinemann, 2018, pp. 17–27. ISBN: 978-0-12-812150-4. DOI: https://doi.org/10.1016/B978-0-12-812150-4.00002-1. URL: https://www.sciencedirect.com/science/article/pii/B9780128121504000021.

[94] Minigarden. *Installation manual*. Online brochure, Minigarden. Accessed on July 10, 2025. 2025. URL: https://minigarden.nl/minigarden-assortiment/minigarden-downloads/.

[95] Ministerie van Binnenlandse Zaken en Koninkrijksrelaties. *Besluit bouwwerken leefomgeving*. Accessed on 25 april, 2025. June 2018. URL: https://www.bblonline.nl/docs/wet/bbl.

[96] Ashraf Mir. "Green facades and building structures". Master's Thesis. Delft, Netherlands: Delft University of Technology, 2011.

[97] Mobilane. *Architect folder - Wallplanter*. Online brochure, Mobilane. Accessed on April 22, 2025. 2025. URL: https://mobilane.com/products/wallplanter/.

[98] Mobilane. *MobiPanel green wall*. Accessed on April 23, 2025. 2025. URL: https://mobilane.com/products/mobipanel-green-wall/.

[99] Mostert De Winter. *Modulogreen De Techniek*. Accessed on 28 april, 2025. 2022. URL: https://www.mostertdewinter.nl/nl/groene-gevels/modulogreen-de-techniek/.

[100] Flex MSE. *Sustainable vegetated wall system - Flex MSE*. Nov. 2024. URL: https://flexmse.com/.

[101] P. Myers et al. *ADW: Arthropoda: CLASSIFICATION*. 2025. URL: https://animaldiversity.org/accounts/Arthropoda/classification/#Arthropoda.

[102] Nagwa. *Lesson Explainer: Kingdom Animalia: Invertebrates*. URL: https://www.nagwa.com/en/explainers/413191401075/#:~:text=Kingdom%20Animalia%20is%20divided%20into,Arthropoda%2C%20Mollusca%2C%20and%20Echinodermata.

[103] United Nations. *Convention on Biological Diversity: Article 2. Use of Terms*. Accessed: 2024-10-04. 1992. URL: https://www.cbd.int/convention/articles/?a=cbd-02.

[104] Nest Natuurinclusief. *Natuurinclusief bouwen en ontwerpen gemakkelijk gemaakt | Nest*. Apr. 2025. URL: https://nestnatuurinclusief.nl/natuurinclusief-bouwen-ontwerpen-kennis-tools/.

[105] NCBI Taxonomy. *Taxonomy browser (Diptera)*. URL: https://www.ncbi.nlm.nih.gov/Taxonomy/Browser/wwwtax.cgi?id=7147.

[106] Newcastle University. *Coefficient of Determination, R-squared*. Accessed on June 10, 2025. URL: https://www.ncl.ac.uk/webtemplate/ask-assets/external/maths-resources/statistics/regression-and-correlation/coefficient-of-determination-r-squared.html.

[107] Normcommissie 351 001 "Technische Grondslagen voor Bouwconstructies". *NEN 6700. Technische grondslagen voor bouwconstructies - TGB 1990 - Algemene basiseisen*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 2005.

[108] Normcommissie 351 001 ”Technische Grondslagen voor Bouwconstructies”. *NEN-EN 1990+A1+A1/C2. Eurocode: Grondslagen voor het constructief ontwerp*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2019.

[109] Normcommissie 351 001 ”Technische Grondslagen voor Bouwconstructies”. *NEN-EN 1990+A1+A1/C2/NB. Nationale bijlage bij NEN-EN 1990+A1:2006+A1:2006/C2:2019 - Eurocode: Grondslagen voor het constructief ontwerp*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2019.

[110] Normcommissie 351 001 ”Technische Grondslagen voor Bouwconstructies”. *NEN-EN 1991-1-1+C1+C11. Eurocode 1: Belastingen op constructies - Deel 1-1: Algemene belastingen - Volumieke gewichten, eigen gewicht en gebruiksbelastingen voor gebouwen*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2019.

[111] Normcommissie 351 001 ”Technische Grondslagen voor Bouwconstructies”. *NEN-EN 1991-1-1+C1+C11/NB. Nationale Bijlage bij NEN-EN 1991-1-1+C1+C11: Eurocode 1: Belastingen op constructies - Deel 1-1: Algemene belastingen - Volumieke gewichten, eigen gewicht en gebruiksbelastingen voor gebouwen*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2019.

[112] Normcommissie 351 001 ”Technische Grondslagen voor Bouwconstructies”. *NEN-EN 1992-1-1+C2/NB+A1. Nationale bijlage bij NEN-EN 1992-1-1+C2 Eurocode 2: Ontwerp en berekening van betonconstructies - Deel 1-1: Algemene regels en regels voor gebouwen*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2020.

[113] Normcommissie 351 001 ”Technische Grondslagen voor Bouwconstructies”. *NEN-EN 1992-1-1+C2+A1. Eurocode 2: Ontwerp en berekening van betonconstructies - Deel 1-1: Algemene regels en regels voor gebouwen*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2015.

[114] Normcommissie 351 86 ”Ontwerp en berekening van betonconstructies”. *Technische grondslagen voor bouwconstructies - TGB 1990 - Voorschriften Beton. Constructieve eisen en rekenmethoden (VBC 1990)*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1991.

[115] Normcommissie 351001 ”Technische Grondslagen voor Bouwconstructies”. *NEN 8700+A1. Beoordeling van de constructieve veiligheid van een bestaand bouwwerk bij verbouw en afkeuren - Grondslagen*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2020.

[116] Normcommissie 351001 ”Technische Grondslagen voor Bouwconstructies”. *NEN 8701+A1. Beoordeling van de constructieve veiligheid van een bestaand bouwwerk bij verbouw en afkeuren - Belastingen*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2020.

[117] Normcommissie 351001 ”Technische Grondslagen voor Bouwconstructies”. *NEN 8702. Beoordeling van de constructieve veiligheid van een bestaand bouwwerk bij verbouw en afkeuren - Betonconstructies*. Netherlands: Nederlands Normalisatie-instituut (NEN), 2023.

[118] Normcomssie 351 01 ”Technische grondslagen voor bouwvoorschriften”. *Technische grondslagen voor de berekening van bouwconstructies - TGB 1972. Algemeen gedeelte en belastingen*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1972.

[119] E.P. Odum. *Fundamentals of Ecology*. Philadelphia, PA: W.B. Saunders, 1971.

[120] Ozge Ogut, Nerantzia Julia Tzortzi, and Chiara Bertolin. “Vertical Green Structures to Establish Sustainable Built Environment: A Systematic Market Review”. In: *Sustainability* 14.19 (Sept. 2022), p. 12349. DOI: `10.3390/su141912349`. URL: `https://www.mdpi.com/2071-1050/14/19/12349#fig_body_display_sustainability-14-12349-f005`.

[121] Grzegorz Oloś. “Green facades support biodiversity in urban environment – A case study from Poland”. In: *Journal of Water and Land Development* (Jan. 2024), pp. 257–266. DOI: `10.24425/jwld.2023.148450`. URL: `https://yadda.icm.edu.pl/baztech/element/bwmeta1.element.baztech-c49f02a8-bada-40ee-b325-7941dde3609d`.

[122] Tiziana Oppedisano, Govinda Shrestha, and Silvia I. Rondon. “Chapter 9 - Hemipterans, other than aphids and psyllids affecting potatoes worldwide”. In: *Insect Pests of Potato (Second Edition)*. Ed. by Andrei Alyokhin, Silvia I. Rondon, and Yulin Gao. Second Edition. Academic Press, 2022, pp. 167–187. ISBN: 978-0-12-821237-0. DOI: `https://doi.org/10.1016/B978-0-12-821237-0.00010-X`. URL: `https://www.sciencedirect.com/science/article/pii/B978012821237000010X`.

[123] Marc Ottelé. "The green building envelope: Vertical Greening". PhD dissertation. Delft, The Netherlands: Delft University of Technology, 2011. URL: https://repository.tudelft.nl/record/uuid:1e38e393-ca5c-45af-a4fe-31496195b88d.

[124] Marc Ottelé et al. "Comparative life cycle analysis for green façades and living wall systems". In: *Energy and Buildings* 43.12 (2011), pp. 3419–3429. ISSN: 0378-7788. DOI: https://doi.org/10.1016/j.enbuild.2011.09.010. URL: https://www.sciencedirect.com/science/article/pii/S0378778811003987.

[125] S A Palermo and M Turco. "Green Wall systems: where do we stand?" In: *IOP Conference Series: Earth and Environmental Science* 410.1 (Jan. 2020), p. 012013. DOI: 10.1088/1755-1315/410/1/012013. URL: https://dx.doi.org/10.1088/1755-1315/410/1/012013.

[126] Foam partner. *Vertical gardening systems*. URL: https://www.foampartner.com/en/foam/industrial-foam/greening-systems/vertical-gardening-systems/.

[127] Steven W Peck et al. "Greenbacks from green roofs: forging a new industry in Canada". In: (1999).

[128] Gabriel Pérez et al. "Green vertical systems for buildings as passive systems for energy savings". In: *Applied Energy* 88.12 (2011), pp. 4854–4859. ISSN: 0306-2619. DOI: https://doi.org/10.1016/j.apenergy.2011.06.032. URL: https://www.sciencedirect.com/science/article/pii/S030626191100420X.

[129] Gabriel Pérez et al. "Vertical Greenery Systems (VGS) for energy saving in buildings: A review". In: *Renewable and Sustainable Energy Reviews* 39 (2014), pp. 139–165. ISSN: 1364-0321. DOI: https://doi.org/10.1016/j.rser.2014.07.055. URL: https://www.sciencedirect.com/science/article/pii/S1364032114005073.

[130] Katia Perini. "Greening the Building Envelope". In: *Bioclimatic Approaches in Urban and Building Design*. Ed. by Giacomo Chiesa. Cham: Springer International Publishing, 2021, pp. 401–414. ISBN: 978-3-030-59328-5. DOI: 10.1007/978-3-030-59328-5_21. URL: https://doi.org/10.1007/978-3-030-59328-5_21.

[131] Katia Perini and Marc Ottelé. "Designing green façades and living wall systems for sustainable constructions". In: *International Journal of Design & Nature and Ecodynamics* 9.1 (2014), pp. 31–46.

[132] Katia Perini et al. "Greening the building envelope, facade greening and living wall systems". In: *Open Journal of Ecology* 01.01 (Jan. 2011), pp. 1–8. DOI: 10.4236/oje.2011.11001. URL: https://doi.org/10.4236/oje.2011.11001.

[133] Katia Perini et al. "Vertical greening systems and the effect on air flow and temperature on the building envelope". In: *Building and environment* 46.11 (2011), pp. 2287–2294.

[134] Katia Perini et al. "Vertical greening systems, a process tree for green façades and living walls". In: *Urban Ecosystems* 16.2 (Sept. 2012), pp. 265–277. DOI: 10.1007/s11252-012-0262-3. URL: https://doi.org/10.1007/s11252-012-0262-3.

[135] Plant Box. *The cleverly simple vertical garden*. Online brochure, Green Tech. Accessed on April 22, 2025. 2025. URL: https://www.green-tech.co.uk/green-roofs-and-living-walls/green-walls-and-ivy-screens/plantbox-green-wall.

[136] Ingeborg Postema. *Modernistische bouwstijl*. Accessed on April 30, 2025. 2022. URL: https://exterieur.architectenpunt.nl/spotlight/modernistische-bouwstijl.

[137] Greenroofs.com pronatur. *Caixa Forum Museum Vertical Garden*. 2022. URL: https://www.greenroofs.com/projects/caixa-forum-museum-vertical-garden/.

[138] Walter V Reid et al. *Ecosystems and human well-being-Synthesis: A report of the Millennium Ecosystem Assessment*. Island Press, 2005.

[139] E.A. Reinshagen. "Formalisering van toekenning van gewichten binnen een Multi Criteria Analyse". Master's Thesis. Enschede, Netherlands: Universiteit Twente, 2007.

[140] C. Rentier, J. Reymers, and M.W.R. Salden. *Jellema 4B - Omhulling*. Thiememeulenhoff, Jan. 2005.

[141] Richard Brink. *Vertikale Begrünung*. Online brochure, Richard Brink. Accessed on 28 april, 2025. 2025. URL: https://www.richard-brink.de/pflanzsysteme/vertikale-begruenung/pflanzwand-adam.html?gad_source=5&gclid=EAIaIQobChMI8bm4sdj6jAMVebKDBx0xJgRDEAAYAyAAEgJDUvD_BwE.

[142] Rijksdienst voor het Cultureel Erfgoed. *Kantoorgebouwen in Nederland 1945-2015: cultuurhistorische en typologische quickscan*. 2013. (Visited on 10/04/2024).

[143] Benjamin Riley. "The state of the art of living walls: Lessons learned". In: *Building and Environment* 114 (2017), pp. 219–232. ISSN: 0360-1323. DOI: https://doi.org/10.1016/j.buildenv.2016.12.016. URL: https://www.sciencedirect.com/science/article/pii/S0360132316305108.

[144] Veronique Scharwachter. *Pearson Correlatiecoëfficiënt (r) Berekenen en Interpreteren*. Accessed on June 9, 2025. 2022. URL: https://www.scribbr.nl/statistiek/pearson-correlatie/.

[145] Maximw Schreurs and Carien van der Wal. *Puntensysteem natuurinclusief bouwen*. 2021. URL: https://openresearch.amsterdam/nl/page/49666/puntensysteem-natuurinclusief-bouwen.

[146] W. van der Schrier. *Bouwen in gewapend beton*. Netherlands: Argus, 1965.

[147] P. Sedgwick. *Pearson's correlation coefficient*. Accessed on June 9, 2025. 2012. URL: https://www.bmj.com/content/345/bmj.e4483.full.

[148] Sempergreen. *Verticale tuinen. Binnen & buiten*. Online brochure, Sempergreen. Accessed on July 10, 2025. 2025. URL: https://sempergreenwall.com/nl/flexipanel/.

[149] Abir Smiti. "A critical overview of outlier detection methods". In: *Computer Science Review* 38 (2020), p. 100306. ISSN: 1574-0137. DOI: https://doi.org/10.1016/j.cosrev.2020.100306. URL: https://www.sciencedirect.com/science/article/pii/S1574013720304068.

[150] Santiago Soliveres et al. "Biodiversity at multiple trophic levels is needed for ecosystem multifunctionality". In: *Nature* 536.7617 (Aug. 2016), pp. 456–459. DOI: 10.1038/nature19092. URL: https://doi.org/10.1038/nature19092.

[151] T.G.M. Spierings, R.Ph. van Amerongen, and H Millekamp. *Jellema 3 - Draagstructuur*. Thiememeulenhoff, 2004.

[152] Petra Starink. *Boemerang beschermd rijksmonument*. Accessed on April 29, 2025. 2015. URL: https://www.architectuur.nl/nieuws/boemerang-beschermd-rijksmonument/.

[153] R. Steenbergen, T. Vrouwenvelder, and N. Scholten. "Veiligheidsfilosolofie bestaande bouw. Toepassing en interpretatie NEN 8700". In: *Cement* 4 (2012), pp. 8–16.

[154] Stichting Commissie Voorschriften Beton. *Voorschriften Beton. Deel A, Gemeenschappelijk gedeelte*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1974.

[155] Stichting Commissie Voorschriften Beton. *Voorschriften Beton. Deel B, In het werk gestort beton, Aanvullende bepalingen*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1974.

[156] Stichting Commissie Voorschriften Beton. *Voorschriften Beton. Deel E, Gewapend beton, Aanvullende bepalingen*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1976.

[157] Stichting voor Onderzoek, Voorschriften en Kwaliteitseisen op het gebied van beton (CUR-VB). *Voorschriften Beton VB 1974/1984. NEN 3880*. Netherlands: Nederlands Normalisatie-Instituut (NEN), 1984.

[158] Sto. *Cable trellis for living green walls - StoFix Iso-Bar ECO*. Accessed on 28 april, 2025. 2021. URL: https://www.sto.com/en/portfolio/facade-finishes/cable-trellis-for-green-walls/cable-trellis-for-green-walls.html.

[159] Petra Thorpert et al. "Exploring the Integration Between Colour Theory and Biodiversity Values in the Design of Living Walls". In: *Frontiers in Ecology and Evolution* 10 (Apr. 2022). DOI: 10.3389/fevo.2022.804118. URL: https://doi.org/10.3389/fevo.2022.804118.

[160] Özgür Burhan Timur and Elif Karaca. "Vertical Gardens". In: *Advances in Landscape Architecture*. Ed. by Murat Özyavuz. Rijeka: IntechOpen, 2013. Chap. 22. DOI: 10.5772/55763. URL: https://doi.org/10.5772/55763.

[161] Saija Toivonen et al. *Real Estate and Sustainable Crisis Management in Urban Environments: Challenges and Solutions for Resilient Cities*. Taylor & Francis, 2024.

[162] Travel France Online. *Quai Branly Museum Vertical Garden*. Accessed on April 23, 2025. 2025. URL: https://www.travelfranceonline.com/quai-branly-museum-vertical-garden/.

[163] Trouw. *Nationale-Nederlanden stapt in kredieten*. Accessed on April 30, 2025. 2014. URL: https://www.trouw.nl/nieuws/nationale-nederlanden-stapt-in-kredieten~bbca4320/?referrer=https%3A%2F%2Fwww.google.com%2F.

[164] L.H. van Reeuwijk. "Green facades for a resilient and liveable built environment: A holistic multi-criteria approach for selecting vertical greening systems". Master's Thesis. Delft, Netherlands: Delft University of Technology, 2023.

[165] Koen van Uffelen. "Herbestemming van een monument in beton; een protocol voor het constructief beoordelen van een betonconstructie uit 1910-1940". Master's Thesis. Eindhoven, Netherlands: Eindhoven University of Technology, 2012.

[166] Peter Veenendaal. *Havengebouw, Amsterdam (1951-1960)*. Accessed on May 2, 2025. 2025. URL: https://dudok.org/1960/08/10/havengebouw-amsterdam-1951-1965/.

[167] Gertjan Verbaan, DGMR, and NEN normwerkgroep daglicht. "Daglight is gezond". In: *BOUWFYSICA* 2 (2022). URL: https://www.nen.nl/media/wysiwyg/Daglicht_-_Artikel_Bouwfysicablad_Verbaan.pdf.

[168] Vertical Meadow. *Meadow Cladding, Living Facade System. A simple and safe living wall system to bring biodiversity into urban environments*. Online brochure, Vertical Meadow. Accessed on July 7, 2025. 2023. URL: https://www.verticalmeadow.com/vertical-meadows-products/.

[169] Vertiko GmbH. *Vertiko Markenbroschüre*. Online brochure, Vertiko. Accessed on April 23, 2025. 2025. URL: https://vertiko.de/kontakt-faq-downloads/#download-section.

[170] J.H.M. Wagemans. "Modularity of Living Walls". Bachelor's Thesis. Delft, Netherlands: Delft University of Technology, 2016.

[171] Ken J. Wallace. "Classification of ecosystem services: Problems and solutions". In: *Biological Conservation* 139.3 (2007), pp. 235–246. ISSN: 0006-3207. DOI: https://doi.org/10.1016/j.biocon.2007.07.015. URL: https://www.sciencedirect.com/science/article/pii/S0006320707002765.

[172] Rosewood Living Walls. *Living walls attract birds*. Apr. 2022. URL: https://www.rosewoodlivingwalls.co.uk/living-walls-attract-birds/.

[173] WTC Amsterdam. *Van pionier tot icoon: de geschiedenis van WTC Amsterdam*. Accessed on May 5, 2025. 2025. URL: https://wtcamsterdam.com/nieuws-en-events/de-geschiedenis-van-wtc-amsterdam/.

[174] Kelvin O. Yoro and Michael O. Daramola. "Chapter 1 - CO2 emission sources, greenhouse gases, and the global warming effect". In: *Advances in Carbon Capture*. Ed. by Mohammad Reza Rahimpour, Mohammad Farsi, and Mohammad Amin Makarem. Woodhead Publishing, 2020, pp. 3–28. ISBN: 978-0-12-819657-1. DOI: https://doi.org/10.1016/B978-0-12-819657-1.00001-3. URL: https://www.sciencedirect.com/science/article/pii/B9780128196571000013.

[175] Xing Quan Zhang. "The trends, promises and challenges of urbanisation in the world". In: *Habitat international* 54 (2016), pp. 241–252.

A

VGS literature review method

As introduced in chapter 2, an exploratory literature study was conducted to investigate the main VGS types, their biodiversity potential and possible preferred building characteristics. This appendix outlines the research approach.

A.1. Research approach

The exploration starts with the master thesis by van Reeuwijk [164], as it includes an extensive literature analysis of VGS. Specifically, the sources used in the chapter on VGS design alternatives and the paragraph on biodiversity were examined. From this initial step the following sources are consulted:

- Perini [130]
- Brković Dodig, Radic Sibinovic, and Auer [23]
- Manso and Castro-Gomes [88]
- Ogut, Tzortzi, and Bertolin [120]
- Hollands and Korjenic [60]
- Medl, Stangl, and Florineth [91]
- Bustami et al. [25]
- Francis and Lorimer [47]
- Pérez et al. [129]
- Fernández-Cañero, Pérez Urrestarazu, and Perini [43]
- Köhler [75]
- Pérez et al. [128]
- Jim [70]
- Palermo and Turco [125]
- Perini et al. [134]
- Ottelé et al. [124]
- Perini et al. [132]
- Mir [96]
- Wagemans [170]
- Ottelé [123]
- Milliken [93]
- den Hartog [35]

- Mayrand et al. [90]
- Thorpert et al. [159]
- Jim [69]
- Madre et al. [87]
- Chiquet [27]
- Mayrand and Clergeau [89]
- Collins, Schaafsma, and Hudson [31]

The relevant references from these sources were used to continue the analysis, the backward snowball effect. From this research step the following sources are employed:

- Perini et al. [133]
- Peck et al. [127]
- Chiquet, Dover, and Mitchell [28]
- Timur and Karaca [160]
- Hop and Hiemstra [62]
- Riley [143]
- Besir and Cuce [12]
- Botes and Breed [20]

To gain further insight into the application of VGS on existing buildings, the weight of different VGS types and their relationship with biodiversity, additional literature research was conducted using Google Scholar. The following search queries were used: “green facades” AND “existing facade”, and “green facades” AND “biodiversity”. In addition, online searches were conducted via Google to obtain information from VGS producers, using various names of the classified VGS types as search terms. Besides these search results, relevant theses on VGS, obtained through other channels, were also reviewed. Lastly, the literature research was supplemented by an expert interview with a facade designer who developed his own VGS with a strong focus on enhancing biodiversity (section E.3). The useful sources identified in this step are listed below:

- Lo Faro et al. [85]
- Oloś [121]
- Bolhuis [16]
- Walls [172]
- LLC [84]
- Beeren [10]
- van Reeuwijk [164]
- Mobilane [98]
- Mobilane [97]
- Biotecture [14]
- Vertiko GmbH [169]
- Jakob Rope Systems [67]
- El Menshawy, Mohamed, and Fathy [41]
- 90deGREEN [1]
- Sto [158]
- GSky [54]
- GSky [55]

- Vertical Meadow [168]

B

Calculations of safety factor analysis

B.1. Calculation of safety factor

B.1.1. Safety factor of concrete in compression

$$\gamma_{total,com,GBV1962} = \frac{\mu_{mat}}{S_k} = 4.2 \quad \text{[-]} \tag{B.1}$$

Where:

- $\mu_{mat} = 31.5$ [N/mm²], mean cubic compression stress of concrete, retrieved from GBV 1962 [49]
- $S_k = 7.5$ [N/mm²], allowable compression stress of concrete, retrieved from GBV 1962 [49]

$$\gamma_{total,com,VB1974} = \gamma_{load} \cdot \frac{\mu_{mat}}{R_d} = 3.8 \quad \text{[-]} \tag{B.2}$$

Where:

- $\gamma_{load} = 1.7$ [-], load factor, retrieved from VB 1974A [154]
- $\mu_{mat} = f_{cm} = f_{ck} + 7.5 = 30$ [N/mm²], mean cubic compression stress of concrete, retrieved from VB 1974A [154]
- $R_d = f_b = 0.75 \cdot f_{bk} = 13.5$ [N/mm²], design value for concrete compression strength, retrieved from VB 1974A [154]

$$\gamma_{total,com,VBC1990} = \gamma_{load} \cdot \frac{\mu_{mat}}{R_d} = 3.0 \quad \text{[-]} \tag{B.3}$$

Where:

- $\gamma_{load} = 1.35$ [-], load factor, retrieved from TGB 1990 [107]
- $\mu_{mat} = f_{ck} + 8 = 33$ [N/mm²], mean cubic compression stress of concrete, retrieved from VBC 1990 [114]
- $R_d = 0.72 \cdot f_{ck}/\gamma_m = 0.72 \cdot 25/1.2 = 15$ [N/mm²], design value for concrete compression strength, retrieved from VBC 1990 [114]

$$\gamma_{total,com,EC2012} = \gamma_{load} \cdot \frac{\mu_{mat}}{R_d} = 2.7 \quad \text{[-]} \tag{B.4}$$

Where:

- $\gamma_{load} = 1.35$ [-], load factor, retrieved from NEN 1990 [108]
- $\mu_{mat} = f_{ck,cube} + 8 = 33$ [N/mm²], mean cubic compression stress of concrete, retrieved from NEN 1992 [113]

- $R_d = f_{ck,cube}/\gamma_m = 16.7$ [N/mm²], design value for concrete compression strength, retrieved from NEN 1992 [113]

B.1.2. Safety factor of steel reinforcement in tension

$$\gamma_{total,ten,GBV1962} = \frac{\mu_{mat}}{\bar{\sigma}_a} = 2.3 \quad \text{[-]} \tag{B.5}$$

Where:

- $\mu_{mat} = 500$ [N/mm²], mean tensile stress of steel, retrieved from GBV 1962 [49]
- $\bar{\sigma}_a = 220$ [N/mm²], allowable tensile stress of steel, retrieved from GBV 1962 [49]

$$\gamma_{total,ten,VB1974} = \gamma_{load} \cdot \frac{\mu_{mat}}{R_d} = 2.1 \quad \text{[-]} \tag{B.6}$$

Where:

- $\gamma_{load} = 1.7$ [-], load factor, retrieved from VB 1974A [154]
- $\mu_{mat} = 500$ [N/mm²], minimum tensile stress of steel, retrieved from VB 1974A [154]
- $R_d = f_a = 400$ [N/mm²], design value for steel tensile strength, retrieved from VB 1974A [154]

$$\gamma_{total,ten,VBC1990} = \gamma_{load} \cdot \frac{\mu_{mat}}{R_d} = 1.9 \quad \text{[-]} \tag{B.7}$$

Where:

- $\gamma_{load} = 1.35$ [-], load factor, retrieved from TGB 1990 [107]
- $\mu_{mat} = 500$ [N/mm²], minimum tensile stress of steel, retrieved from VBC 1990 [114]
- $R_d = f_s = f_{s,rep}/\gamma_m = 400/1.15 = 348$ [N/mm²], design value for steel tensile strength, retrieved from VBC 1990 [114]

$$\gamma_{total,ten,EC2012} = \gamma_{load} \cdot \frac{\mu_{mat}}{R_d} = 1.9 \quad \text{[-]} \tag{B.8}$$

Where:

- $\gamma_{load} = 1.35$ [-], load factor, retrieved from NEN 1990 [108]
- $\mu_{mat} = 500$ [N/mm²], minimum tensile stress of steel, retrieved from NEN 1992 [113]
- $R_d = f_{yd} = f_{yk}/\gamma_m = 400/1.15 = 348$ [N/mm²], design value for steel tensile strength, retrieved from NEN 1992 [113]

B.2. Calculation of safety ratios

B.2.1. Calculation of Safety Ratio 1

The first safety ratio, comparing the safety factors of earlier codes to the safety factor of the current code, is defined as follows:

$$\text{Safety Ratio 1} = \frac{\gamma_{total}}{\gamma_{total,2012}} \tag{B.9}$$

Equation B.9 can be applied to safety ratios related to compression or safety ratios related to tensile forces, which results in Safety Ratio 1,com and Safety Ratio 1,ten. The safety factors used for the calculation are based on concrete with μ_{mat} of around 30 N/mm² and steel of μ_{mat} of 500 N/mm², which can be found in table 5.4. The calculation is performed in table B.1.

Code	Safety Ratio 1,com	Safety Ratio 1,ten
GBV 1962	$4.2/2.7 = 1.6$	$2.3/1.9 = 1.2$
VB 1974	$3.8/2.7 = 1.4$	$2.1/1.9 = 1.1$
VB 1984	$3.8/2.7 = 1.4$	$2.1/1.9 = 1.1$
VBC 1990	$3.0/2.7 = 1.1$	$1.9/1.9 = 1.0$
EC 2012	$2.7/2.7 = 1.0$	$1.9/1.9 = 1.0$

Table B.1: Calculation of Safety Ratio 1 for concrete and steel reinforcement of codes from 1962 to 2012

B.2.2. Calculation of Safety Ratio 2

The second safety ratio, between EC 2012 and NEN 87 series (part of equation (5.10) and equation (5.11)), is defined as follows:

$$\text{Safety Ratio 2} = \frac{\gamma_{load,2012}}{\gamma_{load,87}} \tag{B.10}$$

Safety Ratio 2 is calculated for permanent loads and variable loads for formula 6.10 and 6.10b, which are shown in equation (5.1) and equation (5.2). Where $\gamma_{load,2012,perm}$ and $\gamma_{load,87,perm}$ are defined by the factor multiplied by $G_{k,j}$ in equation (5.1) and equation (5.2). Where $\gamma_{load,2012,var}$ and $\gamma_{load,87,var}$ are defined by the factor multiplied by $Q_{k,1}$ in equation (5.1) and equation (5.2). For the EC 2012 values, ψ_0 is taken from table 5.1, while the partial safety factors (for 6.10b combined with ξ_j) are obtained from table B.2. For the NEN 87 values, ψ_0 is taken from table 5.1, while the partial safety factors (for 6.10b combined with ξ_j) are obtained from table 5.2. The calculation is shown in table B.3.

Formula	Permanent	Variable
6.10a	$\gamma_{G,j} = 1.35$	$\gamma_{Q,1} = 1.5$
6.10b	$\xi_j \cdot \gamma_{G,j} = 1.2$	$\gamma_{Q,1} = 1.5$

Table B.2: Partial factors of CC2 from NEN 1990 NB with a variable load other than wind being the dominant one [109, table NB.4-A1.2(B)]

Safety Ratio 2	Permanent	Variable
6.10a	$1.2/1.3 = 1.04$	$1.5 \cdot 0.5/1.3 \cdot 0.5 = 1.15$
6.10b	$1.35/1.15 = 1.04$	$1.5/1.3 = 1.15$

Table B.3: Calculation of Safety Ratio 2 for equations 6.10a and 6.10b and both permanent and variable loads

C

Parameter study calculations

C.1. General design calculations

C.1.1. Calculation of design loads according to GBV 1962

The permanent loads acting on the slab used in the design of beams in GBV 1962 are calculated according to equation (C.1), while the permanent loads acting on the slab used in the designs of slabs are calculated according to equation (C.2). The permanent loads acting on the beam of System 1 are calculated according to equation (C.3). The variable loads acting on the beam of System 1 are calculated according to equation (C.4). The variable loads acting on the slab of System 2, $q_{k,slab}$, are calculated by multiplying $Q_{k,slab}$ by the standard slab width of one metre, resulting in a line load with units of kN/m, as shown in equation (C.5).

$$G_{k,slab} = \gamma_c \cdot t_{slab} \quad \text{[kN/m²]} \tag{C.1}$$

$$g_{k,slab} = G_{k,slab} \cdot b_{slab} + h_{st} \cdot g_{fac} \quad \text{[kN/m]} \tag{C.2}$$

$$g_{k,beam} = \gamma_c \cdot h_{beam} \cdot b_{beam} + h_{st} \cdot g_{fac} + 0.5 \cdot L_{slab} \cdot G_{k,slab} \quad \text{[kN/m]} \tag{C.3}$$

$$q_{k,beam} = 0.5 \cdot Q_{k,slab} \cdot L_{slab} \quad \text{[kN/m]} \tag{C.4}$$

$$q_{k,slab} = Q_{k,slab} \cdot b_{slab} \quad \text{[kN/m]} \tag{C.5}$$

Where:

- $\gamma_c =$ volumetric weight of concrete in [kN/m³], retrieved from table 6.4
- $t_{slab} =$ slab thickness in [m], retrieved from section 6.2.2
- $G_{k,slab} =$ permanent load acting on the slab in [kN/m²], calculated by equation (C.1)
- $b_{slab} = 1.0$ [m], slab width, retrieved from section 6.2.2
- $h_{beam} =$ beam height in [m], retrieved from section 6.2.2
- $b_{beam} =$ beam width in [m], retrieved from section 6.2.2
- $h_{st} = 3$ [m], story height, defined in section 6.2.1
- $g_{fac} =$ facade weight in [kN/m²], prescribed in table 6.1 or table 6.2
- $L_{slab} =$ slab length in [m], prescribed in table 6.1 or table 6.2
- $Q_{k,slab} = 2.5$ [kN/m²], variable load acting on the slab, prescribed in section 6.3.3

C.1.2. Calculation of design loads according to VB 1974

The permanent loads acting on the slab used in the design of slabs in VB 1974 are calculated according to equation (C.7). The permanent loads acting on the beam of System 1 are calculated according to equation (C.8). The variable loads acting on the beam of System 1 are calculated according to equation (C.9). The variable loads acting on the slab of System 2, $q_{k,slab}$, are calculated by multiplying $Q_{k,slab}$ by the standard slab width of one metre, resulting in a line load with units of kN/m, as shown in equation (C.10).

$$G_{k,slab} = \gamma_c \cdot t_{slab} + g_{lw} \quad \text{[kN/m²]} \tag{C.6}$$

$$g_{k,slab} = G_{k,slab} \cdot b_{slab} + h_{st} \cdot g_{fac} \quad \text{[kN/m]} \tag{C.7}$$

$$g_{k,beam} = \gamma_c \cdot h_{beam} \cdot b_{beam} + h_{st} \cdot g_{fac} + 0.5 \cdot L_{slab} \cdot G_{k,slab} \quad \text{[kN/m]} \tag{C.8}$$

$$q_{k,beam} = 0.5 \cdot Q_{k,slab} \cdot L_{slab} \quad \text{[kN/m]} \tag{C.9}$$

$$q_{k,slab} = Q_{k,slab} \cdot b_{slab} \quad \text{[kN/m]} \tag{C.10}$$

Where:

- $\gamma_c =$ volumetric weight of concrete in [kN/m³], retrieved from table 6.5
- $t_{slab} =$ slab thickness in [m], retrieved from section 6.2.2
- $G_{k,slab} =$ permanent load acting on the slab in [kN/m²], calculated by equation (C.6)
- $b_{slab} = 1.0$ [m], slab width, retrieved from section 6.2.2
- $g_{lw} =$ permanent load accounting for the weight of lightweight partition walls in [kN/m²], prescribed in section 6.4.3
- $h_{beam} =$ beam height in [m], retrieved from section 6.2.2
- $b_{beam} =$ beam width in [m], retrieved from section 6.2.2
- $h_{st} = 3$ [m], story height, defined in section 6.2.1
- $g_{fac} =$ facade weight in [kN/m²], prescribed in table 6.1 or table 6.2
- $L_{slab} =$ slab length in [m], prescribed in table 6.1 or table 6.2
- $Q_{k,slab} = 2.0$ [kN/m²], variable load acting on the slab, prescribed in section 6.4.3

C.1.3. Calculation of design loads according to NEN 8700 series

The permanent load acting on slabs are calculated with equation (C.11) or equation (C.12), either in kN/m² or kN/m. The permanent load acting on the beams of System 1 are calculated with equation (C.13). The variable loads acting on either beam or slab are calculated with equation (C.14) and equation (C.15).

$$G_{k,slab} = \gamma_c \cdot t_{slab} \quad \text{[kN/m²]} \tag{C.11}$$

$$g_{k,slab} = G_{k,slab} \cdot b_{slab} + h_{st} \cdot g_{fac} \quad \text{[kN/m]} \tag{C.12}$$

$$g_{k,beam} = \gamma_c \cdot h_{beam} \cdot b_{beam} + h_{st} \cdot g_{fac} + 0.5 \cdot L_{slab} \cdot G_{k,slab} \quad \text{[kN/m]} \tag{C.13}$$

$$q_{k,beam} = 0.5 \cdot Q_{k,slab} \cdot L_{slab} \quad \text{[kN/m]} \tag{C.14}$$

$$q_{k,slab} = Q_{k,slab} \cdot b_{slab} \quad \text{[kN/m]} \tag{C.15}$$

Where:

- $\gamma_c =$ volumetric weight of concrete in [kN/m³], retrieved from table 6.6
- $t_{slab} =$ slab thickness in [m], retrieved from section 6.2.2
- $G_{k,slab} =$ permanent load acting on the slab in [kN/m²], calculated by equation (C.11)
- $b_{slab} = 1.0$ [m], slab width, retrieved from section 6.2.2
- $h_{beam} =$ beam height in [m], retrieved from section 6.2.2
- $b_{beam} =$ beam width in [m], retrieved from section 6.2.2
- $h_{st} = 3$ [m], story height, defined in section 6.2.1
- $g_{fac} =$ facade weight in [kN/m²], prescribed in table 6.1 or table 6.2
- $L_{slab} =$ slab length in [m], prescribed in table 6.1 or table 6.2
- $Q_{k,slab} = 3.7$ [kN/m²], characteristic variable load acting on the slab, prescribed in section 6.5.6

C.1.4. Calculation of structural effects

M_{Ed} is calculated according to equation (C.16):

$$M_{Ed} = \frac{1}{8} \cdot s_d \cdot L^2 \quad \text{[kNm]} \tag{C.16}$$

Where:

- s_d can be either $s_{d,beam}$ or $s_{d,slab}$ in [kN/m], design load, calculated by equation (6.2) or equation (6.3)
- L can be either L_{beam} or L_{slab} in [m], beam length or slab length

V_{Ed} is calculated according to equation (C.17):

$$V_{Ed} = \frac{1}{2} \cdot s_d \cdot L \quad \text{[kN]} \tag{C.17}$$

Where:

- s_d can be either $s_{d,beam}$ or $s_{d,slab}$ in [kN/m], design load, calculated by equation (6.2) or equation (6.3)
- L can be either L_{beam} or L_{slab} in [m], beam length or slab length

C.2. Capacity calculation according to GBV 1962

C.2.1. Calculations for the bending moment capacity according to GBV 1962

The parameter d is defined as the distance from the extreme compression fibre to the centroid of the tensile reinforcement [146, par. 3.a], or in other words, as the effective height of the cross-section [113]. The value of d is calculated according to equation (C.18):

$$d = h_{beam} - c - \phi_{st} - 0.5 \cdot \phi_{main} \quad \text{[mm]} \tag{C.18}$$

Where:

- $h_{beam} =$ beam height in [mm], retrieved from section 6.2.2
- $c =$ concrete cover in [mm], retrieved from section 6.2.2
- $\phi_{st} =$ diameter of the stirrups in [mm], retrieved from section 6.2.2
- $\phi_{main} =$ diameter of the main reinforcement [mm], retrieved from section 6.2.2

The parameter x is calculated according to equation (C.20) provided by Schrier [146, par. 4], which is defined as the height of the neutral axis from the top of the beam or slab. The total area of the main reinforcement A_s is used as input for x, which is calculated by equation (C.19).

$$A_s = n_{main} \cdot \frac{1}{4} \cdot \pi \cdot \phi_{main}^2 \quad \text{[mm²]} \tag{C.19}$$

Where:

- $n_{main} =$ the number of main reinforcement bars in the cross-section
- $\phi_{main} =$ diameter of the main reinforcement [mm], retrieved from section 6.2.2

$$x = \frac{n \cdot A_s}{b_{beam}} \left(-1 + \sqrt{1 + \frac{2 \cdot b_{beam} \cdot h_{beam}}{n \cdot A_s}} \right) \quad \text{[mm]} \tag{C.20}$$

Where:

- $n =$ Ratio of the modulus of elasticity of steel to that of concrete, retrieved from table 6.4
- $A_s =$ total area of main reinforcement in [mm²], calculated by equation (C.19)
- $b_{beam} =$ beam width in [mm], retrieved from section 6.2.2
- $h_{beam} =$ beam height in [mm], retrieved from section 6.2.2

Parameters x and d can be used to calculate z according to equation (C.21).

$$z = h_{beam} - d - \frac{1}{3} \cdot x \quad \text{[mm]} \tag{C.21}$$

Where:

- $h_{beam} =$ beam height in [mm], retrieved from section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)
- $x =$ height of the neutral axis from top of the element in [mm], calculated by equation (C.20)

C.2.2. Design calculations for the bending moment capacity according to GBV 1962

Two coefficients, η and α, required in the design calculations for the bending moment capacity according to GBV 1962, are calculated as follows:

$$\eta = \frac{n \cdot \bar{\sigma}'_b}{\bar{\sigma}_a + n \cdot \bar{\sigma}'_b} \quad \text{[-]} \tag{C.22}$$

Where:

- $n = 15$, ratio of the modulus of elasticity of steel to that of concrete in [-], retrieved from table 6.4
- $\bar{\sigma}'_b =$ allowable compressive strength of concrete in bending in [kgf/cm²], retrieved from table 6.4
- $\bar{\sigma}_a =$ allowable tensile strength of steel in [kgf/cm²], retrieved from table 6.4

$$\alpha^2 = \frac{6}{\bar{\sigma}'_b \cdot \eta \cdot (3 - \eta)} \quad \text{[-]} \tag{C.23}$$

Where:

- $\bar{\sigma}'_b =$ allowable compressive strength of concrete in bending in [N/mm²], retrieved from table 6.4
- $\eta =$, coefficient, calculated by equation (C.22)

C.2.3. Calculations for the shear capacity according to GBV 1962

The shear reinforcement is designed as described in section 6.3.5. Shear reinforcement design starts with implementing bent-up bars. The total area of the bent-up bars A_0 is calculated according to equation (C.24).

$$A_0 = n_{bent} \cdot \frac{1}{4} \cdot \pi \cdot \phi_{main}^2 \quad \text{[mm²]} \tag{C.24}$$

Where:

- n_{ben} is the number of bent-up bars in the cross-section
- $\phi_{main} =$ diameter of the main reinforcement in [mm], retrieved from section 6.2.2

The total required area of stirrup reinforcement within distance y, $A_{b,ben}$, is derived by equation (C.25).

$$A_{b,ben} = \frac{S_r \cdot \sqrt{2}}{\bar{\sigma}_a} \quad \text{[cm²]} \tag{C.25}$$

Where:

- S_r diagonal tensile force resisted by stirrups in [kgf], calculated by equation (6.17)
- $\bar{\sigma}_a =$ allowable tensile strength of steel in [kgf/cm²], retrieved from table 6.4

The cross-sectional area of a single stirrup, A_b, is calculated with equation (C.26).

$$A_b = 2 \cdot \frac{1}{4} \cdot \pi \cdot \phi_{st}^2 \quad \text{[mm²]} \tag{C.26}$$

Where:

- $\phi_{st} =$ diameter of the stirrups in [mm], retrieved from section 6.2.2

The required number of stirrups $n_{b,ben}$ are calculated according to equation (C.27). It should be noted that $n_{b,ben}$ is rounded up to the nearest whole number.

$$n_{b,ben} = \frac{A_{b,ben}}{A_b} \quad \text{[-]} \tag{C.27}$$

where:

- $A_{b,ben} =$ total required area of stirrup reinforcement within distance y in [mm²], calculated by equation (C.25)
- $A_b =$ cross-sectional area of a single stirrup in [mm²], calculated by equation (C.26)

The number of stirrups that fit within distance y, n_b, is calculated with equation (C.28).

$$n_b = \frac{y - c}{s_{st}} + 1 \quad \text{[-]} \tag{C.28}$$

Where:

- $y =$ distance from supports over which shear reinforcement is required [m], defined in section 6.2.2
- $c =$ concrete cover in [mm], retrieved from section 6.2.2
- s_{st} stirrup spacing in [mm], retrieved from section 6.2.2

C.3. Capacity calculation according to VB 1974

C.3.1. Calculations for the bending moment capacity according to VB 1974

The ultimate bending moment is determined based on the assumption that steel yields before the concrete in the compression zone crushes, as explained in section 6.4.4. To ensure this ductile failure mechanism, VB 1974 introduces both minimum and maximum reinforcement ratios. The parameter $k_{x,max}$ is used in the equation for calculating the maximum reinforcement ratio $\omega_{0,max}$, as shown in equation (6.26). The value of $k_{x,max}$ is obtained from equation (C.29), as prescribed in VB 1974E [156, Art. 503.2].

$$k_{x,max} = \frac{1}{1 + \frac{f_a}{500}} \quad \text{[-]} \tag{C.29}$$

Where:

- $f_a =$ design tensile strength of reinforcing steel in [N/mm²], defined in table 6.5

The fracture strain of steel, ϵ_{ar}, is set at twice the minimum strain after failure reduced by the necking component, as prescribed by Boom [18, p. 6.2.3]. The corresponding formula for calculating ϵ_{ar} is given in equation (C.30).

$$\epsilon_{ar} = 2 * A10 - A5 \quad \text{[-]} \tag{C.30}$$

Where:

- $A10 = 10$ minimum strain after failure of FeB400 in [-], retrieved from VB1974A [154, Tabel A-10]
- $A5 = 14$ minimum strain after failure with necking component of FeB400 in [-], retrieved from VB1974A [154, Tabel A-10]

The steel reinforcement ratio ω is calculated as follows:

$$\omega = \frac{A_s}{b \cdot d} \cdot 10^2 \quad \text{[\%]} \tag{C.31}$$

Where:

- $A_s =$ total area of main reinforcement in [mm²], calculated by equation (C.19)
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

C.3.2. Calculations for the shear capacity according to VB 1974

The shear stress that concrete can resist is denoted by τ_1, and is defined in VB 1974E as half the design tensile strength of concrete [156, Art. 504.2.1]. The corresponding formula for calculating τ_1 is given in equation (C.32).

$$\tau_1 = 0.5 \cdot f_b \quad \text{[N/mm²]} \tag{C.32}$$

Where:

- $f_b =$ design tensile strength of concrete in [N/mm²], defined in table 6.5

The shear force resisted by the concrete is denoted by T_1 and calculated by equation (C.33).

$$T_1 = \tau_1 \cdot b_{beam} \cdot d \quad \text{[N]} \tag{C.33}$$

Where:

- $\tau_1 =$ design shear strength of concrete in [N/mm²], calculated by equation (C.32)

- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

C.3.3. Calculations for the maximum deflection according to VB 1974

The theoretically required reinforcement area $A_{a,benodigd}$ is calculated with equation (C.34). For this formula, the internal lever arm z is approximated as $0.9 \cdot d$, as prescribed by Boom and Kamerling [18, p. 6.2.1].

$$A_{a,benodigd} = \frac{M_{Ed}}{z \cdot f_a} \quad \text{[mm²]} \tag{C.34}$$

Where:

- $M_{Ed} =$ acting bending moment in [Nmm], calculated by equation (C.16)
- $z =$ internal lever arm in [mm], calculated by $0.9 \cdot d$
- $f_a =$ design tensile strength of steel in [N/mm²], defined in table 6.5

C.4. Resistance calculation according to NEN 8700 series

C.4.1. Calculations for the bending moment resistance according to NEN 8700 series

The height of the neutral axis from the top of the element x is calculated with equation (C.36), according to Braam and Lagendijk [21, par. 3.2]. The design tensile strength of steel is multiplied with de main reinforcement area in equation (C.35) to calculate the design tensile force in the steel.

$$N_s = f_{yd} \cdot A_s \quad \text{[N]} \tag{C.35}$$

Where:

- $f_{yd} =$ design tensile strength of steel in [N/mm²], defined in table 6.6
- $A_s =$ total area of main reinforcement in [mm²], calculated by equation (C.19)

$$x = \frac{N_s}{\frac{3}{4} \cdot f_{cd} \cdot b} \quad \text{[mm]} \tag{C.36}$$

Where:

- $N_s =$ design tensile force in the steel in [N], defined in equation (C.35)
- $f_{cd} =$ design compressive strength of concrete in [N/mm²], defined in table 6.5
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2

The height of the internal lever arm z for the calculation of the bending moment resistance is calculated with equation, according to Braam and Lagendijk [21, par. 3.2].

$$z = d - \frac{7}{18} \cdot x \quad \text{[mm]} \tag{C.37}$$

Where:

- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)
- $x =$ height of the neutral axis from the top of the element in [mm], calculated by equation (C.36)

C.4.2. Calculations for the shear resistance according to NEN 8700 series

Calculations for the shear capacity of concrete without shear reinforcement

The concrete shear capacity without shear reinforcement is calculated using equation (C.38).

$$V_{Rd,c} = (C_{Rd,c} \cdot k(100 \cdot \rho_l \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp})b \cdot d \quad \text{[N]} \tag{C.38}$$

Where:

- $C_{Rd,c} = \frac{0.18}{\gamma_c}$, coefficient in [-], prescribed by NEN 1992NB [112, par. 6.2.2]
- $k =$ coefficient in [-], calculated by equation (C.41)
- $\rho_l =$ main reinforcement ratio in [-], calculated by equation (C.42)
- $f_{ck} =$, characteristic compressive strength of concrete in [N/mm²], defined in table 6.5
- $k_1 = 0.15$, coefficient, prescribed by NEN 1992NB [112, par 6.2.2]
- $\sigma_{cp} = 0$, as no normal force, prescribed in NEN 1992 [113, par. 6.2.2]
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

The minimum of the concrete shear capacity without shear reinforcement, $V_{Rd,c,min}$, is calculated with equation (C.39).

$$V_{Rd,c,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b \cdot d \quad \text{[N]} \tag{C.39}$$

Where:

- $v_{min} =$ minimum shear capacity of concrete in [N/mm²], calculated by equation (C.40)
- $k_1 = 0.15$, coefficient, prescribed by NEN 1992NB [112, par 6.2.2]
- $\sigma_{cp} = 0$, as no normal force, prescribed in NEN 1992 [113, par. 6.2.2]
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

$$v_{min} = 0.035 \cdot k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} \quad \text{[N/mm²]} \tag{C.40}$$

Where:

- $k =$ coefficient in [-], calculated by equation (C.41)
- $f_{ck} =$, characteristic compressive strength of concrete in [N/mm²], defined in table 6.5

$$k = 1 + \sqrt{\frac{200}{d}} \quad \text{with } k \leq 2.0 \quad \text{[-]} \tag{C.41}$$

Where:

- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

$$\rho_l = \frac{A_{sl}}{b \cdot d} \quad \text{with } \rho_l \leq 0.02 \quad \text{[-]} \tag{C.42}$$

Where:

- $A_{sl} =$ specific reinforcement area assumed equal to A_s in [mm²], specified in NEN 1992 [113, par 6.2.2], calculated by equation (C.19)
- $b = b_{beam}$ or b_{slab}, beam or slab width in [mm], defined in section 6.2.2
- $d =$ effective height of the cross-section in [mm], calculated by equation (C.18)

Calculations for the shear capacity of bent-up bars

The shear capacity of bent-up bars, $V_{Rd,bent}$, is calculated with the formula for inclined stirrups as prescribed in NEN 1992 [113, par 6.2.3]. The expressions used to determine $V_{Rd,bent}$ and $V_{Rd,bent,max}$ are provided in equation (C.43) and equation (C.44).

$$V_{Rd,bent} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd}(\cot\theta + \cot\alpha)\sin\alpha \quad \text{[N]} \tag{C.43}$$

Where:

- $A_{sw} = A_0$, cross-sectional area of the bent-up bars in [mm²], calculated by equation (C.24)
- $s = a$, spacing between bent-up bars in [mm], defined in section 6.3.5
- $z =$ internal lever arm in [mm], calculated by equation (C.37)
- $f_{ywd} = f_{yd}$, design tensile strength of steel in [N/mm²], defined in table 6.6
- $\theta = 45$, as defined in section 6.5.4
- $\alpha = 45$, as defined in section 6.5.4

$$V_{Rd,bent,max} = \alpha_{cw} \cdot b \cdot z \cdot \nu_1 \cdot f_{cd} \cdot \frac{(\cot\theta + \cot\alpha)}{1 + \cot^2\theta} \quad \text{[N]} \tag{C.44}$$

Where:

- $\alpha_{cw} = 1$, coefficient in [-], prescribed in NEN 1992NB [112, par 6.2.3]
- $b = b_{beam}$, beam width in [mm], defined in section 6.2.2
- $z =$ internal lever arm in [mm], calculated by equation (C.37)
- $f_{cd} =$ design compressive strength of concrete in [N/mm²], defined in table 6.6
- $\theta = 45$, as defined in section 6.5.4
- $\alpha = 45$, as defined in section 6.5.4

Calculations for the shear capacity of stirrups

The shear capacity of stirrups, $V_{Rd,st}$, is calculated with the formula for vertical shear reinforcement as prescribed in NEN 1992 [113, par 6.2.3]. The expressions used to determine $V_{Rd,st}$ and $V_{Rd,st,max}$ are provided in equation (C.45) and equation (C.46).

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot\theta \quad \text{[N]} \tag{C.45}$$

Where:

- $A_{sw} = A_0$, cross-sectional area of the bent-up bars in [mm²], calculated by equation (C.24)
- $s = s_{st}$, stirrup spacing in [mm], defined in section 6.2.2
- $z =$ internal lever arm in [mm], calculated by equation (C.37)
- $f_{ywd} = f_{yd}$, design tensile strength of steel in [N/mm²], defined in table 6.6
- $\theta = 90$, as defined in section 6.5.4

$$V_{Rd,max} = \alpha_{cw} \cdot b \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot\theta + \tan\theta) \quad \text{[N]} \tag{C.46}$$

Where:

- $\alpha_{cw} = 1$, coefficient in [-], prescribed in NEN 1992NB [112, par 6.2.3]
- $b = b_{beam}$, beam width in [mm], defined in section 6.2.2
- $z =$ internal lever arm in [mm], calculated by equation (C.37)
- $\nu_1 = 0.6 \cdot (1 - \frac{f_{ck}}{250})$, coefficient in [-], prescribed in NEN 1992NB [112, par 6.2.3]
- $f_{cd} =$ design compressive strength of concrete in [N/mm²], defined in table 6.6

- $\theta = 90$, as defined in section 6.5.4

C.4.3. Deflection capacity calculations according to the NEN 8700 series

The elastic modulus of concrete applied in the deflection analysis is calculated according to equation equation (C.47).

$$E_{equi} = \frac{1}{3} \cdot E_{cm} = \frac{1}{3} \cdot 22 \cdot \left(\frac{f_{cm}}{10}\right)^{0.3} \quad \text{[N/mm²]} \tag{C.47}$$

Where:

- $f_{cm} = f_{ck} + 8$, mean concrete compression strength in [N/mm²], prescribed in NEN 1992 [113, Table 3.1]

The deflection limit w_{lim} used in the parameter study is calculated according to equation (C.48).

$$w_{lim} = \frac{1}{250} \cdot L \quad \text{[mm]} \tag{C.48}$$

Where:

- $L = L_{beam}$ or L_{slab}, beam or slab length in [mm], defined in table 6.1 and table 6.2

C.5. Derivation of excess capacity factor formula

The derivation below inserts equation (6.52) into equation (6.51) to derive at equation (6.53).

$$C_{x,element,code} = \frac{C_{F,element,code} + \Delta s_{k,code}}{s_{k,element,code}}$$

$$\Delta s_{k,code} = s_{k,element,EC2012} - s_{k,element,code}$$

$$C_{x,element,code} = \frac{C_{F,element,code} + s_{k,element,EC2012} - s_{k,element,code}}{s_{k,element,code}}$$

$$C_{x,element,code} = \frac{C_{F,element,code}}{s_{k,element,code}} + \frac{s_{k,element,EC2012}}{s_{k,element,code}} - 1$$

$$C_{x,element,code} = \frac{C_{F,element,code} + s_{k,element,EC2012}}{s_{k,element,code}} - 1$$

D

Parameter study input and output

D.1. Combinations

The parameters listed in table D.1 and table D.3 are combined into parameter sets for each system. This results in 24 combinations for System 1 and 8 combinations for System 2. These combinations are presented in table D.2 and table D.4, with numbering provided to facilitate reference throughout the analysis.

Parameters				
L_{beam}	B1 = 3.6 m	B2 = 5.4 m	B3 = 7.2 m	
L_{slab}	S1 = 3.6 m	S2 = 5.4 m	S3 = 7.2 m	S4 = 9 m
g_{fac}	F1 = 1 kN/m²	F2 = 2.3 kN/m²		

Table D.1: Parameters used for the parameter study of system 1

Combination	Beam	Slab	Facade
1	B1	S1	F1
2	B1	S1	F2
3	B1	S2	F1
4	B1	S2	F2
5	B1	S3	F1
6	B1	S3	F2
7	B1	S4	F1
8	B1	S4	F2
9	B2	S1	F1
10	B2	S1	F2
11	B2	S2	F1
12	B2	S2	F2
13	B2	S3	F1
14	B2	S3	F2
15	B2	S4	F1
16	B2	S4	F2
17	B3	S1	F1
18	B3	S1	F2
19	B3	S2	F1
20	B3	S2	F2
21	B3	S3	F1
22	B3	S3	F2
23	B3	S4	F1
24	B3	S4	F2

Table D.2: Combinations of system 1: Beam System

Parameters				
L_{slab}	S1 = 3.6 m	S2 = 5.4 m	S3 = 7.2 m	S4 = 9 m
g_{fac}	F1 = 1 kN/m²	F2 = 2.3 kN/m²		

Table D.3: Parameters used for the parameter study of system 2

Combination	Slab	Facade
1	S1	F1
2	S1	F2
3	S2	F1
4	S2	F2
5	S3	F1
6	S3	F2
7	S4	F1
8	S4	F2

Table D.4: Combinations of system 2: Slab system

D.2. Design input

D.2.1. Design input: System 1

Comb.	h_{beam} [mm]	b_{beam} [mm]	ϕ_{main} [mm]	n_{main}	ϕ_{st} [mm]	s_{st} [mm]	n_{bent}	a [mm]
1	300	210	12	4	8	200	0	0
2	305	220	14	4	8	200	0	0
3	345	220	14	4	8	200	0	0
4	360	225	16	4	8	150	0	0
5	405	225	16	4	8	100	0	0
6	430	225	16	4	8	100	0	0
7	430	280	16	5	8	50	0	0
8	465	280	16	5	8	50	0	0
9	400	225	16	4	8	250	0	0
10	490	225	16	4	8	300	0	0
11	495	235	18	4	8	300	1	500
12	560	235	18	4	8	300	1	500
13	560	290	18	5	8	300	1	500
14	625	290	18	5	8	300	1	500
15	630	300	20	5	8	300	2	322
16	680	300	20	5	8	300	2	329
17	485	290	18	5	8	300	0	0
18	615	300	16	6	8	300	0	0
19	595	300	20	5	8	300	0	0
20	680	300	20	5	8	300	0	0
21	830	300	20	5	8	300	1	500
22	935	315	20	5	8	300	1	500
23	970	355	20	6	8	300	1	500
24	1040	355	20	6	8	300	1	500

Table D.5: Design properties per combination of System 1: the Beam System (GBV 1962)

Comb.	h_{beam} [mm]	b_{beam} [mm]	ϕ_{main} [mm]	n_{main}	ϕ_{st} [mm]	s_{st} [mm]
1	350	165	12	3	8	200
2	345	210	12	4	8	200
3	410	170	14	3	8	250
4	375	220	14	4	8	250
5	370	225	16	4	8	200
6	405	225	16	4	8	200
7	410	235	18	4	8	150
8	440	235	18	4	8	150
9	450	220	14	4	8	300
10	565	220	14	4	8	300
11	550	225	16	4	8	300
12	630	225	16	4	8	300
13	520	290	18	5	8	300
14	575	290	18	5	8	200
15	600	340	18	6	8	150
16	640	340	18	6	8	150
17	635	225	16	4	8	300
18	640	235	18	4	8	300
19	650	245	20	4	8	300
20	750	245	20	4	8	300
21	755	300	20	5	8	300
22	835	300	20	5	8	300
23	865	355	20	6	8	200
24	930	355	20	6	8	200

Table D.6: Design properties per combination of System 1: the Beam System (VB 1974)

D.2.2. Design input: System 2

Comb.	t_{slab} [mm]	ϕ_{main} [mm]	n_{main}	s_{main} [mm]
1	125	12	6	153
2	150	12	7	129
3	240	12	8	112
4	240	14	8	110
5	255	16	8	108
6	320	16	9	94
7	380	18	9	106
8	405	20	8	104

Table D.7: Design properties per combination of System 2: the Slab System (GBV 1962)

Comb.	t_{slab} [mm]	ϕ_{main} [mm]	n_{main}	s_{main} [mm]
1	140	10	7	131
2	140	12	7	129
3	210	12	8	112
4	220	14	8	110
5	250	16	8	108
6	260	18	8	106
7	330	18	8	106
8	320	20	9	90

Table D.8: Design properties per combination of System 2: the Slab System (VB 1974)

D.3. Design output

D.3.1. Design output: System 1

Comb.	$C_{x,beam,GBV1962}$ [–]	$s_{k,beam,GBV1962}$ [kN/m]	$s_{k,beam,EC2012}$ [kN/m]	$C_{F,beam,GBV1962}$ [kN/m]
1	0.34	14.2	16.6	2.37
2	0.41	18.2	20.6	4.95
3	0.00	22.9	26.7	0.00
4	0.09	26.9	30.7	0.00
5	0.37	33.6	38.8	7.20
6	0.32	37.7	42.9	6.90
7	0.31	47.4	54.2	17.0
8	0.34	51.5	58.3	17.7
9	0.35	14.8	17.3	2.73
10	0.35	19.2	21.7	4.32
11	0.35	23.9	27.7	4.47
12	0.34	28.1	32.0	5.70
13	0.33	35.3	40.6	6.27
14	0.35	39.7	45.0	8.76
15	0.33	49.0	55.9	9.42
16	0.35	53.3	60.2	11.7
17	0.37	16.1	18.6	3.45
18	0.36	21.0	23.6	4.95
19	0.28	25.4	29.3	3.27
20	0.22	29.9	33.8	2.55
21	0.36	37.4	42.8	8.10
22	0.39	42.4	47.8	11.0
23	0.39	52.8	59.8	13.4
24	0.39	57.3	64.3	15.1

Table D.9: Excess capacity factor, original characteristic load, excess capacity factor and characteristic load according to EC 2012 of System 1: the Beam System (GBV 1962)

Comb.	$C_{x,beam,VB1974}$ [–]	$s_{k,beam,VB1974}$ [kN/m]	$s_{k,beam,EC2012}$ [kN/m]	$C_{F,beam,VB1974}$ [kN/m]
1	0.22	14.1	16.5	0.69
2	0.24	18.3	20.8	1.89
3	0.21	22.8	26.5	0.99
4	0.22	27.0	30.8	2.19
5	0.21	33.4	38.7	1.71
6	0.22	37.5	42.7	2.88
7	0.20	46.8	53.6	2.82
8	0.20	50.9	57.6	3.39
9	0.21	15.1	17.5	0.66
10	0.23	19.6	22.1	2.01
11	0.22	24.1	27.9	1.53
12	0.22	28.4	32.2	2.52
13	0.21	35.1	40.3	1.98
14	0.22	39.3	44.6	3.48
15	0.23	49.4	56.2	4.38
16	0.23	53.6	60.5	5.22
17	0.22	16.1	18.6	0.99
18	0.22	20.2	22.7	1.98
19	0.21	24.9	28.8	1.29
20	0.18	29.4	33.3	1.47
21	0.05	36.9	42.2	0.00
22	0.23	41.4	46.7	4.23
23	0.12	51.9	58.8	0.00
24	0.10	56.3	63.3	0.00

Table D.10: Excess capacity factor, original characteristic load, excess capacity factor and characteristic load according to EC 2012 of System 1: the Beam System (VB 1974)

D.3.2. Design output: System 2

Comb.	$C_{x,slab,GBV1962}$ [–]	$s_{k,slab,GBV1962}$ [kN/m]	$s_{k,slab,EC2012}$ [kN/m]	$C_{F,slab,GBV1962}$ [kN/m²]
1	0.42	8.5	9.83	2.28
2	0.36	13.0	14.4	3.33
3	0.37	11.3	12.7	2.67
4	0.35	15.2	16.6	3.84
5	0.34	11.6	13.1	2.52
6	0.32	17.1	18.6	3.90
7	0.35	14.6	16.2	3.51
8	0.31	19.1	20.7	4.32

Table D.11: Excess capacity factor, original characteristic load, excess capacity factor and characteristic load according to EC 2012 of System 1: the Slab System (GBV 1962)

Comb.	$C_{x,slab,VB1974}$ [–]	$s_{k,slab,VB1974}$ [kN/m]	$s_{k,slab,EC2012}$ [kN/m]	$C_{F,slab,VB1974}$ [kN/m²]
1	0.30	8.86	10.2	1.32
2	0.26	12.8	14.1	1.98
3	0.24	10.5	12.0	1.11
4	0.25	14.7	16.1	2.28
5	0.32	11.5	13.0	2.19
6	0.25	15.6	17.1	2.43
7	0.24	13.4	15.0	1.62
8	0.24	17.1	18.6	2.49

Table D.12: Excess capacity factor, original characteristic load, excess capacity factor and characteristic load according to EC 2012 of System 1: the Slab System (VB 1974)

D.4. Equations for result interpretation

D.4.1. Data analysis

The Pearson correlation coefficient r is calculated in Python according to the formula provided in equation (D.1).

$$r = \frac{\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_{i=1}^{n}(x_i - \bar{x})^2}\sqrt{\sum_{i=1}^{n}(y_i - \bar{y})^2}} \tag{D.1}$$

Where:

- $n =$ number of data points
- $x_i =$ observed value of the independent variable (e.g., $s_{k,element,code}$) for data point i
- $y_i =$ observed value of the dependent variable (e.g., $C_{F,element,code}$) for data point i
- $\bar{x} =$ mean of the x values
- $\bar{y} =$ mean of the y values

The coefficient of determination R^2 is calculated in Python according to the formula provided in equation (D.2) [106].

$$R^2 = 1 - \frac{\sum_{i=1}^{n}(e_i)^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2} \tag{D.2}$$

Where:

- $e_i =$ residual for data point i, calculated by equation (D.3)
- $y_i =$ observed value of the dependent variable (e.g., $C_{F,element,code}$) for data point i
- $\bar{y} =$ mean of the y values

D.4.2. Outlier detection

Calculation of residuals

The residuals between the linear regression line and the values of $C_{F,element,code}$ have been analyzed to identify statistical outliers. The residuals are calculated according to equation (D.3).

$$e_i = y_i - \hat{y}_i \tag{D.3}$$

Where:

- $y_i =$ true value of $C_{F,element,code}$
- $\hat{y}_i =$ predicted value of $C_{F,element,code}$ based on the linear regression line

Calculation of outlier boundaries

According to the IQR method, upper and lower bounds for outlier detection are calculated. The interquartile range (IQR) is determined using equation (D.4). Data points falling below $Q_1 - 1.5 \cdot IQR$ or above $Q_3 + 1.5 \cdot IQR$ are commonly classified as outliers. The corresponding upper and lower bounds are listed in table D.13 and are often referred to as the whiskers of the box-plot.

$$\text{IQR} = Q_3 - Q_1 \tag{D.4}$$

Where:

- $Q_1 =$ first quartile, the value below which 25% of the data falls
- $Q_3 =$ third quartile, the value below which 75% of the data falls

Element	Code	Lower Bound [kN/m]	Upper Bound [kN/m]
Beam	GBV 1962	-3.65	3.47
Beam	VB 1974	-2.15	1.99
Slab	GBV 1962	-0.40	0.43
Slab	VB 1974	-0.51	0.46

Table D.13: Lower and upper boundaries for outlier detection using the 1.5×IQR rule, per element type and design code.

UC values of slabs

The unity checks of the bending moment capacity and the deflection requirement of the slabs designed according to VB 1974 are listed in table D.14. For the designs of GBV 1962, the unity checks are provided in table D.15.

Comb.	UC_M	UC_d
1	0.94	0.95
2	0.97	0.97
3	0.99	0.97
4	0.98	0.90
5	0.93	0.98
6	0.98	0.98
7	0.99	0.99
8	0.98	0.97

Table D.14: The outputs of the unity checks for bending moment capacity UC_M and deflection requirement UC_d of the beams designed according to VB 1974, calculated with equations (6.29) and (6.22)

Comb.	UC_a, M	$UC_{b,M}$	UC_d
1	0.96	0.75	0.99
2	0.97	0.77	0.86
3	0.98	0.62	0.77
4	0.98	0.75	0.78
5	0.98	0.84	0.97
6	0.99	0.80	0.78
7	0.99	0.76	0.81
8	0.99	0.83	0.76

Table D.15: The outputs of the unity checks for the steel stress $UC_{a,M}$ and concrete stress $UC_{b,M}$ of the bending moment capacity and deflection requirement UC_d of the beams designed according to GBV 1962, calculated with equations (6.8), (6.7) and (6.22)

E

Results from interviews

E.1. Interview: Rogier van Reen

E.1.1. Permission to use statement

I, Rogier van Reen, confirm that I participated in a conversation with Lisanne Kluft on 17-01-2025 in the context of her Master's thesis research on the building suitability, structural feasibility and biodiversity potential of Vertical Greening Systems. I have provided permission to record the conversation.

During this conversation, I provided information based on my professional expertise and experience as structural engineer. I have reviewed the statements derived from the conversation, as included in the text below, and hereby approve their use as a cited source.

I understand that the conversation content may be referenced in the final thesis and I give my consent for this use.

Signed,

Name: Rogier van Reen

Position/Affiliation: Senior Structural Engineer, Arup

Date: 09-07-2025

Signature:

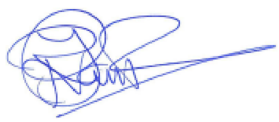


Figure E.1: Permission to use statement signed by Rogier van Reen

E.1.2. Key insights

The following list of statements have been derived from the conversation on January 17:

- It is assumed that the curtain wall has a weight of 1 kN/m².
- Many low-rise office buildings have brick masonry facades. These are often combined with a steel load-bearing structure of columns and beams, and a lightweight inner leaf made of timber or metal stud (0.5 kN/m²), while the brick masonry serves as a lightweight facade material (18 kN/m²).
- Brick masonry can also function as a load-bearing material. In that case, a slightly heavier type of brickwork is typically used for the inner layer (20 kN/m²) and the lightweight brickwork is used as outer layer (18 kN/m²). An example of a low-rise office building with brick masonry is shown

in figure E.2.

- Calcium silicate brick is rarely used in office buildings, except perhaps as a non-load-bearing inner wall. It is very labor-intensive, as a large amount of brickwork is required, and the material is relatively heavy. Timber frame or metal stud walls are therefore more common alternatives, both typically weighing around 0.5 kN/m².
- The Arup office is cladded with natural stone as sandwich element with a load-bearing concrete inner leaf.
- Single-leaf concrete facades are virtually never seen in the Netherlands, there is almost always a cavity wall or sandwich panel, especially from the 1960s and 70s onwards.
- Prefabricated sandwich panels are typically never non-load-bearing, that would be illogical, given their substantial weight and strength.
- A common facade type in office buildings consists of a concrete spandrel panel combined with a strip of glazing. An example is shown in figure 4.16. The presence of the glass strip indicates that the facade is non-load-bearing. The spandrel panel typically spans between columns due to its stiffness. Typical build-up of this facade includes plasterboard (12.5 mm; 0.15 kN/m²), insulation (not considered here) and a concrete panel. A thickness of 150 mm is required to ensure workability, structural stiffness and resistance to wind loads.
- Since the lower bound is leading in the case of estimating excess capacity, the insulation layer in facades can be excluded from the weight calculation. Alternatively, a general safety factor of 0.9 can be applied to all values to remain on the conservative side.

Figure E.2: Office building in Alphen aan den Rijn, the Netherlands, captured via Google Street View

E.2. Interview: Dirk-Jan Kluft

E.2.1. Permission to use statement

I, Dirk-Jan Kluft, confirm that I participated in conversations with Lisanne Kluft on 18-01-2025 and 2-05-2025 in the context of her Master's thesis research on the building suitability, structural feasibility and biodiversity potential of Vertical Greening Systems.

During these conversations, I provided information based on my professional expertise and experience as structural engineer. I have reviewed the statements derived from the conversations, as included in the text below, and hereby approve their use as a cited source.

I understand that the interview content may be referenced in the final thesis and I give my consent for this use.

Signed,
Name: Dirk-jan Kluft.

Position/Affiliation: directeur-eigenaar

Date: 3-7-2025.

Signature:

Figure E.3: Permission to use statement signed by Dirk-Jan Kluft

E.2.2. Key insights

January 18, 2025

The following list of statements have been derived from the conversation on January 18:

- It is assumed that the curtain wall has a weight of 1 kN/m².
- From the outside, it is not always possible to determine whether masonry is part of a load-bearing wall or simply applied as a self-supporting facade layer. I would expect the building in figure E.2 having a load-bearing wall.
- The facade of aluminum panels or natural stone can have potential different build-ups from outside to inside, for example aluminum layer-insulation-concrete, aluminum-insulation-aluminum, natural stone-insulation-concrete, natural stone-insulation-aluminum. The aluminum on the inside can be an aluminum frame, but also an aluminum layer.
- Facades can be self-supporting, meaning they transfer their weight to a facade beam or floor slab, as is the case with masonry. Brickwork masonry facades must be anchored to an inner frame for horizontal support, which is typically a heavyweight inner leaf as masonry is considered a heavy facade material. However, a aluminum or timber frame is technically possible and would result in a more lightweight facade. At the Havengebouw in Amsterdam, shown in figure 4.17, the brick facade is supported by an angle bracket or similar structural element, so it can cover the floor elements for example.
- However, some facades transfer their weight directly to columns due to their sufficient stiffness, such as concrete spandrels.
- The WTC in Amsterdam and Rotterdam both use classic curtain walls, as shown in figure 4.14 and figure 4.15.

May 5, 2025
The following list of statements have been derived from the conversation on May 5:

- The former headquarters of NMB in Amsterdam, as shown in figure 4.11, employs load-bearing concrete walls with a brick masonry facade.
- The pictures of the interior of Randstad's headquarters in Amsterdam reveal visible columns in the facade, suggesting a column structure that supports facade beams, which in turn carry the floors. An example of such a picture is provided in figure E.4. The wall build-up could potentially consist of a concrete inner leaf with insulation, followed by an approximately 10 cm thick concrete outer leaf, forming a sandwich panel. The outer surface included embedded ceramic tiles, which were pressed into the concrete during construction. The panels were usually produced by Schokbeton in Zwijndrecht.
- At the considered studied period (1960-1990), construction materials were expensive, so it was more efficient to use smaller reinforcement bars placed closer together to match the required reinforcement area more precisely, rather than using larger bars and overspending on material. Closer bar spacing also reduced the amount of concrete needed. Today, however, labor costs are generally higher than material costs, which means that minimizing time on site, rather than minimizing material use, has become a greater priority. As a result, modern construction often favors faster methods and standardized reinforcement layouts, even if they possibly require more material.
- Concrete floor slabs were typically 10 to 15 cm thick, due to the shorter spans commonly used.

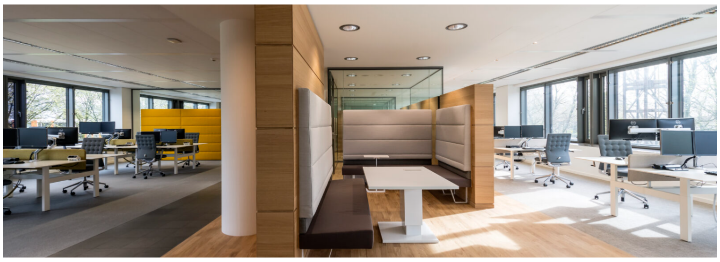


Figure E.4: Interior of the headquarters of Randstad in Amsterdam, the Netherlands (construction: 1987-1990) [24]

E.3. Interview: Alistair Law

E.3.1. Permission to use statement

I, Alistair Law, confirm that I participated in a conversation with Lisanne Kluft on 1-07-2025 in the context of her Master's thesis research on the building suitability, structural feasibility and biodiversity potential of Vertical Greening Systems. I have provided permission to record the conversation.

During this interview, I provided information based on my professional expertise and experience as facade desginer and founder of Vertical Meadow, a Vertical Greening System. I have reviewed the relevant content from the conversation, as included in the text below, and hereby approve the use of this material as a cited source.

I understand that the interview content may be referenced in the final thesis and I give my consent for this use.

Signed,
Name: Alistair Law

Position/Affiliation: Director Vertical Meadow

Date: 08/07/2025

Signature:

Figure E.5: Permission to use statement signed by Alistair Law

E.3.2. Key insights

Lisanne Kluft 0:00: Why is biodiversity so important in your opinion?

Alistair Law 0:03: I guess for me, it kind of all started that I've been working with Arup and Arup has normally forced me to be in quite urban locations. And because of that, and I'm and we're quite lucky in London, London does have lots of parks. There's kind of been this, for me a personal nature deficiency that there is not enough green space, there's not enough space where you can kind of disappear and recuperate. Actually I did a masters in something like 2011 and part of me was really interested in how nature changed people's kind of mental or like the connections between nature, mental health and seeing kind of green spaces. And I wrote a thesis on this and kind of did a lot of research into what I call environmental psychology. So how how place, how space changes, how one feels. And so that for me built up this connection between how we're building and how nature working together can make people feel better and kind of. And then as I kind of started and that was before I even started thinking about reinventing green wall systems and then I guess through that process, through support from Arup, we kind of spoke to ecologists and they're like, oh, just use some wildflower seeds. And so we just took some wildflower seeds and it worked. Wildflower started growing and then ecologists were like, wow, we've never seen these amazing species which are normally weeds, kind of unwanted plants, but growing vertically and people are like, that's incredible. Having those species are are key to the natural world and what I learned over time is that actually wildflower meadows are really the kind of essence of the natural world, like they kind of provide the habitat for insects. These insects are the bedrock of the natural world and we've forgotten them. And there's a good book called Silent Earth and it's a book about, yeah, these incredible insects that are are disappearing at a crazy scale. Actually meadows are the kind of the only way we can, yeah, they're one of the only ways we can support them. And we've lost in the UK almost 97% of our meadows in 100 years. It became for me like a forgotten hero.

Alistair Law 4:00: And I guess what I do notice and we're using some tech is that on our walls the kind of edible, it's teeming with insects. We're recreating quite, how do you say, species rich habitats, which even trying to achieve in the natural world is quite difficult. We don't have the benefits of the soil and all that soil kind of ecology.

Lisanne Kluft 4:26: Yeah, yeah, that was my question.

Alistair Law 4:27: Yeah, no. So soil ecology is really important, but I kind of question in the vertical whether it is really important because soil is heavy. It's actually in terms of efficiency of like growing and nutrients and stuff like that if you're if you're kind of thinking something to go on a building, you want it to be light and you want it to be thin. And so we kind of do everything from the I guess from the stems outwards. We I think we provide exceptional habitats and we're kind of measuring that. Whereas we don't do the soil health bit, but I kind of question in an urban environment whether we can easily incorporate it in a low carbon way, so our living walls are maybe 1/3 of the weight of other living walls, which means we can green many more things with much lower structure. I kind of question this. I know a lot of people talk about soil, but for me, soil is actually what we need to protect in our rural world and kind of look at. Not that we shouldn't protect it in the any existing soil, but I don't think we should be putting back soil when we're actually going to put a back a lot of concrete to support that soil.

Lisanne Kluft 5:48: So you mostly focus on the species diversity and the...

Alistair Law 5:53: Exactly, and the habitats that provides, so the habitats of the plants rather than what happens below ground. And yeah, it seems as though we're the only people in the world that are are kind of focusing on wildflowers. One of our challenges is always wildflowers don't always look great in winter. So we have this challenge with clients who want something that looks great all the time, whereas in reality we have to accept that you cut back in it and that's where our cladding system is all a metal system again. But yeah, I guess that's so the biodiversity bit kind of grew on me and now I'm kind of, also I I have young kids and stuff like that. The kind of actually the reason why I'm sure maybe you're a Arup and why I've kind of maintained my contact is the purpose side kind of I guess one can earn money doing lots of things, but actually can one earn money by changing how the world is and kind of improving the world? That's a conscious choice and and I think for me that's the really important bit of what we do is. It's trying to shape perceptions, perceptions on what is what is natural, but also what is what is good. So pretty much every living wall system out there in the world is just a load of pot plants, often grown in Holland, brought to the UK, mainly non-native, often grown in peat. So they're often like I read a statistic that 40% of the UK's plants is so 40% of the substrate used in UK plants is from peat and you're like, well, where?

Lisanne Kluft 7:56: Where do we get that?

Alistair Law 7:57: Yeah, exactly. When do we get that? And are we going to get that in 100 years? Are we going to get that in 150 years? So yeah, I kind of, there is some interesting things I think and and I'm not saying we're we're kind of got it perfect, but I also think kind of measurement is important. So we're starting to use both pollinator, we've got a bee acoustic sensor, so for bumblebees. So we're testing all the latest tech on biodiversity to kind of actually. Not, not to greenwash and say actually our system is great. It's like actually on our walls we had this, this, this and this. To prove it.

- START SLIDES –

Lisanne explains VGS types included in research.

Alistair Law 15:42: Yeah, so ours is none of those.

Lisanne Kluft 15:45: Yeah, that's the interesting part. I also saw this. I would say that it's kind of a box system, but then without the substrates.

Alistair Law 15:55: It's maybe a box, yeah. So without the kind of, yeah, I guess it's a version of the box system, but it's yeah, I mean with the substrates, we still have the substrates, but we don't obviously pot any plants in there. It grows from seed in place.

Lisanne Kluft 16:19: Yeah, 'cause you you see them, but where do you see them in again?

Alistair Law 16:25: So if I show you, yeah, so if I show you this, let me just see. So yeah, so our system is like this with we have mineral wool and seed paper on the front of the mineral wall.

Lisanne Kluft 18:20: Yes. OK. Yeah. So that was a little bit of the introduction and if you look at because what is important as you already mentioned is the weight of the system, especially the the heavyweight systems that are currently still there. I have some examples to to share as well. But it's very interesting because I focused on these. At first I wanted like clear numbers, but then if you look at the examples from or if you look at literature and then you compare those with actual products, then it's quite hard on the websites of products to kind of derive the weight some some now and then. It's quite interesting why it's not so clear, yeah.

Alistair Law 21:04: Yeah. And and I think they deliberately do that. So our square meter is 25 kg per square meter. We often are lighter by 1/2.

Lisanne Kluft 22:10: Yeah, so that's the the choice why eventually I eventually presented or used the ranges instead of a number is because there were quite some differences between those. I wanted to include all variations within one system. Naturally there isn't just one for one type.

Alistair Law 22:18: Yeah, I agreed.

Lisanne Kluft 22:31: So yeah, and then I wanted to talk to you about some examples and where the aim is to evaluate their biodiversity potential, because in the end the framework provides some advice on which VGS should choose based on architecture feasibility, structural feasibility and biodiversity aim. So yeah, so I derived already a few months ago some key features and are just kind of interesting to know what features do you think are important from those systems if you focus on biodiversity?

Alistair Law 26:39: I guess the issue I have with all living walls, which is kind of why Vertical Meadow came out, is that they're typically using non-native species and often there's, depends, but I guess plant diversity, so let's just say. Let's take Semper Green, where they had 70 plants per square meter or something like that. But are they in three types? Are they in five types? Are they in ten types? Are they 20 types? What's the species diversity in those?

Lisanne Kluft 27:20: Yeah, yeah, they're they're often saying that everything is possible, but I think. I saw on a different website I saw around 15 species they can choose from or yeah, depends.

Alistair Law 27:32: Yeah. Exactly. And then most importantly, obviously by the way, which is quite a big thing, but kind of like, yeah, I mean if you're trying to replicate a meadow. So what is success? I guess what if you speak to ecologists, you talk, they call what we call that a mixed sward. So having lots of different plants with lots of different leaf types that are cohabiting in the same space and that's what I find problematic typically of living walls that often you have stripes or you don't have lots of. I mean like if I show you our walls, we have maybe 10 species in the gap coming out of there and then one will grow, one will die back, the next one will grow, the other one will die back. So you get quite complex habitat, which is what is essential for kind of biodiversity.

Alistair Law 28:47: So lots of different plant types growing together is kind of really important. Making sure that they are locally relevant, because it's a bit like me if I grow, I don't know, like a spider plant, indoor spider plant on my desk. If I grow that on a living wall outside in London. It will have limited biodiversity benefits, whereas if I grow something that is a native where species around them are relevant. And I think that's the issue with these living more companies. They have, let's say, 15 plants you can choose from, but there are 15 plants you can choose from the whole of the Netherlands.

And perhaps for the whole of the same 15 parts will be the Netherlands, the UK, France, etcetera. Because they know they're robust, they know they're cheap, they know they're, whereas they might not be native and therefore they might be not country native, but they might not be locally relevant and so um. That's where kind of biodiversity is complex. It's very local, so that's where in the UK they've just started biodiversity net gain rules where you have to put more biodiversity, but and there's some offsetting requirements. But people are unlike carbon offsetting where it's not really accepted to carbon offset in Africa if you're based in in Holland, let's just say or the Netherlands. In nature it's even worse in nature. It's kind of like what putting nature back in the Amazon. Great, but you just destroyed the whole of the UK habitat of this. And yes, it's great you've put some back in the Amazon, but that's not going to the effects of the destruction are very local and that's why I guess go back to it when I look at all these different options. And I let's just say, curtain, indirect and direct are pretty much the same support mechanism. Direct is right on the building whereas curtain and indirect is you're growing roughly the same species. And the issue, what you've hinted to in your slide. Is that wisteria that you showed the beautiful purple? That's great, but it's only got a moment once in the year where it flowers. So that's the problem also with kind of living walls. I mean, it's kind of what are you trying to support, if you're trying to support insects and probably the most important bees, spiders, aphids, all of that other things are equally important. But if you want pollinator species, which is what a lot of people talk about because they talk about crop, they talk about the importance of kind of having lots of bees to make sure we can feed ourselves like. Though often living walls are not diverse enough to have some flowering in the beginning of the season and flowering all through the season, which is where kind of curtain, indirect and direct, they're often monocultures. You can have a bit of diversity, but I guess container box and felt allows you to have more diversity. And I guess our system is probably taking it a step further where you can have multiple species in the same literally in the same space, whereas container, box and felt you've got a zone of one plant, whereas ours you don't have that, you have a cohabitation. Also, so you're aware on the container box and felt, a lot of them are using biocontrols for spraying walls or pesticides like aphids?

Lisanne Kluft 33:06: Not included in my research, but I can imagine that this often happens.

Alistair Law 33:11: People don't want to talk about it, but they're effectively some people will spray the living walls to get rid of aphids. And aphids are so key as part of the food chain, that because their worry is that it will damage the plants. Whereas actually, when I look at our walls and I see a caterpillar that has eaten a whole plant, for me, that's success.

Alistair Law 33:37: It's like we've got to not choose the insects we think are good, which are not. Because if you have a very diverse wall, then the caterpillars will eat perhaps one species, but all the other species are there. And they're offering food for other things. And so actually diversity is the key and biodiversity is the key in the sense of offering lots of food sources for different things.

Lisanne Kluft 33:58: Yeah, that's also what I included in my research is that animals are quite species specific. So indeed a butterfly is attracted to one plant species to become a butterfly.

Alistair Law 34:14: Yeah. Exactly. To eat. Yeah, exactly. To lay their eggs, etcetera.

Lisanne Kluft 34:32: And others have similar have similar preferences. So that's why indeed the the species diversity is quite important and if you have multiple ot more species than you attract more different kinds of, yeah, insect species as well.

Alistair Law 34:58: Yeah and if they're local, then it's more valuable. So making all of these solutions locally relevant is probably the most important bit and avoiding spraying or putting in water. So sometimes they put in effectively insects in the water to kind of kill other insects. As long as, yeah, for me, I think if the wall was diverse enough then that would, well, our walls, they self-managed. We don't have to put any insect kind of controls.

Lisanne Kluft 35:51: You do have nutrients and water irrigation, right?

Alistair Law 35:55: We put nutrients, yeah. We put in much lower. So typically we're at maybe 1/5 because wildflowers are poor nutrient plants and low water plants as well. So yeah, we're in a position where we can put 1/5 of what others use in sense of nutrients.

Alistair Law 36:21: And even the curtain, indirect and direct, like any managed landscape, you would want to do it. Because you would want to add water by irrigation systems on every system you have on that list. Because as you know, 35 degrees today we have not had rain in the UK for maybe two or three weeks.

Lisanne Kluft 37:13: Yeah, first I wanted to ask you before I'll show you the the features that I thought were important if you would rank these systems based on their biodiversity potential, how would you rank them? Or are they rankable?

Alistair Law 37:41: Yeah, no, I think I would say just because of the plant diversity, the container box of felt has the highest potential. Um, yeah, that's how I would. Um, based on the plants you'd pick. But the potential's there. I think they're not using their potential, but I think they could.

Lisanne Kluft 38:03: Yeah, yeah. We're we're thinking about the the optimum design that they should be choosing.

Alistair Law 38:12: Exactly. If you could, yeah, if if they could choose native relevant plants and have a good kind of species mix like plant mix in that kind of abroad, then I think.

Lisanne Kluft 38:27: Instead of the aesthetic.

Alistair Law 38:28: Yeah, I think they need to move from aesthetics into nature.

Lisanne Kluft 38:31: OK, well, I thought of a few more, only four. It's not very complicated. So naturally, plant diversity. Plant coverage was also mentioned by quite some literature and I think it also kind of the the I talked about it the other day because we wanted to include like more of like plant quality or the like the structure and you can basically divide those between substrate size, coverage and diversity because you have a larger, if you have a substrate, if you have a larger substrate, then you have larger rooting space and if you have more planned coverage. This can also provide shelter for various insects and yeah, just natural structure in your plants and by plant diversity is, yeah, quite obvious I think by now.

Alistair Law 39:37: I disagree on the substrate size and orientation so we can have enormous plants like 1 meter high plants growing on one centimeter thick. I mean the root structure just spreads kind of all across it and actually roots are geotropic, so they grow vertically. So actually substrate volume is not relevant.

Lisanne Kluft 40:06: OK, interesting point, yeah.

Alistair Law 40:09: Yeah, no, it's something that we're kind of often surprised by. But yeah, basically we can grow enormous plants on very kind of thin substrates.
Alistair Law 40:21: So yeah, let me, let me just show you now.
Alistair Law 40:44: So yeah, it's very sun is shining a lot today, but you can see here. Can you see here this is so this is this is growing on one centimetre thick.

Lisanne Kluft 40:54: Yeah, that's crazy.

Alistair Law 40:59: And you can see the thickness is here. So my finger is the thickness and you can see it's maybe half a metre thick.

Lisanne Kluft 41:04: Yeah, so that that really proves that you that like nutrients and water is more important than the rooting space.

Alistair Law 41:21: Yeah, exactly. So we're very thin and you get this huge, I guess, yeah, huge volume. I mean, as you're on those butterflies all over it, but there's bees all over it. So you can see all the bumblebees. So yeah, I guess that's the just to on the substrate point and kind of it's for me not something I've noticed as kind of relevant. I totally, I think your plant coverage bit is quite important. When you see our walls, they have very kind of, I think, yeah, lots of coverage. But for me it's more about, I think you hinted that it in your explanation to me it was kind of more about this. It's what I say that ecologists talk about mixed sward. They talk about complex dense vegetation because coverage for me says actually if I cover the whole of this wall or all of that wall with greenery, then that's success. Whereas actually if I create a thing that was in it, but let's just say it's one species. Well, that links to plant diversity, but let's just say it's not very thick, whereas out there you're seeing a thickness of plants which is very big, which if it was that thick a plant and let's just say it's grass versus that thick of kind of complexity, it's very different. So I think plant coverage must be about kind of. I don't know what the words to use, but you had you said exactly earlier on kind of right words. It's about this kind of. I see almost density and kind of complexity.

Lisanne Kluft 43:21: Yeah. There were different words for the plant coverage and eventually I just chose this one. But I think, yeah, it's more maybe more density as well. So like how many plants per square meter...

Alistair Law 43:26: Well, it it could be plants per square meter, but for me it's more of that density, is less about plants per square meter because like I can't tell you how many plants we have per square meter. We have thousands like kind of the shoots coming out everywhere. So it's it's more about for me about, but more kind of. I have complexity. I'll put the words of mixed sward in the chat.

Lisanne Kluft 44:30: The last part the the substrate orientation is also quite in contrast with your design I think because I kind of state that horizontal substrate orientation could be helpful for nesting sites for birds, but also plant litter accumulation, which could also be used by insects as well. So yeah, I think that's more like a vertical structure thing that you create.

Alistair Law 45:23: Yeah, I think, I totally agree. It's interesting if you asked me about what if you wanted to green cities, I would say horizontal greening on green roofs should be a first thing you do. As in every green roof, like planting horizontally should be better or is better. So it's a balance between when you're trying to do it on the vertical. I'm just less convinced, but because again, if you have good structure in your plants, we've had nesting on our walls, you can have other things on it.

Lisanne Kluft 46:03: Yeah, you have birds nesting on Vertical Meadow?

Alistair Law 46:04: Yep. Yeah. Yeah. So yeah, it could. And if you have more branching things, yeah, that kind of happens. So yeah, interesting kind of question.

Lisanne Kluft 46:21: It was hard because I think that the yeah, least amount of research is done on birds like in relation to the vertical greening system. So I only had like 2 actually studies on that front so it wasn't as substantial as the plant diversity and the plant coverage and substrate size, but it's just another feature that could be helpful, but it's not as important. So that is also why I included. First I just derived possible indicators that could be important, but then secondly I looked at specific species.

Lisanne Kluft 47:16: So the species where I looked at and what they think is important and I think what what the hard part is what they did with this research, is that they looked at those things separately instead of as a whole. So you kind of miss out on that. Yeah, kind of the combination of diversity and coverage, what you said like the complex system. So I'm gonna think about how I can somehow include this more. But this is kind of how I tried to give the animal groups a voice to give them, yeah, to kind of to make them stakeholders in the multi criteria analysis that I did.

Alistair Law 47:55: Yeah, yeah, definitely.

Alistair Law 49:28: My big take away is that as I said, the potential of all of them is better even for curtain, indirect and direct. I guess less direct because direct limits your species type, because direct one needs to have sucking types of things whereas curtain and indirect you can grow lots of different kind of species on it. So as long as you've got species diversity, then I think all of them can kind of meet your criteria quite well. I kind of question your kind of thing about, I mean if you want to provide homes for birds, then probably, you can also apply a bird box as a way of doing it.

Alistair Law 51:34: I think for me kind of work out, I I think if you really wanted to get back to what's the best one, I don't think there's an absolute answer. I think I'm always trust that like Arup ecologists and stuff like that. There is an expression in the landscape world of right plant, right place. It was written years ago and I think it's right system, right place. I mean is that it needs to be locally relevant. think I'd say container film box can do all of that. And I think the results are slightly skewed by the horizontal kind of potential versus the vertical. I think that's really relevant on greening of roofs, but I think it's less relevant on walls.