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Optimal Vertical Extension

A study on costs and environmental impact for structural engineers

Master of Science Thesis

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Optimal Vertical Extension

By

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PREFACE

This is the final report written in the context of my master's thesis, as a completion of my postgraduate studies in Civil Engineering at TU Delft, master track Building Engineering and specialization Structural Design. The purpose of this thesis is to take a step further the academic research on the particular subject of vertical extension in building renovation. What inspired me to choose this topic, was a presentation during my studies about Karel Doorman building in Rotterdam. The addition of sixteen storeys on top of an existing building triggered my curiosity and scientific interest. It was my firm belief from the beginning that this approach could form a practical solution for some of the main problems plaguing mankind and societies, such as man-made climate change and urbanization. The current research revealed indeed benefits related to both costs and environmental impact, a fact that affirmed my initial choice to deal with this subject and strengthens more my confidence for future research.

Fortunately I was not alone in this trip. I would like to take this opportunity to express my gratitude to all these people who stood by me and helped me to complete this challenging thesis successfully. First of all, I would like to thank all my colleagues at IMd Raadgevende Ingenieurs for being always friendly and willing to help me, and for showing always interest on my progress. Especially, I would like to say how lucky and grateful I feel for having Pim Peters as my supervisor at IMd. Pim thank you for being so generous and inspiring. Special thanks also to Tana Bakal and Saskia Kieboom for their valuable assistance.

Furthermore, I would like to express my sincere gratitude to the graduating committee. In the first place, professor Rob Nijsse for teaching me how to approach problems as an engineer, working simple and efficient. Then, special thanks to professor Aad van der Horst for his constructive feedback and critical comments on improving the quality of this final research report. Last but not least, I would like to thank Henk Jonkers, for his guidance, crucial input and positive attitude in the course of this thesis.

Moreover, I would like to thank all these people from the construction world in the Netherlands who participated in my research and were willing to share their knowledge and experiences. At first, the ones I interviewed, Michiel Visscher from Royal Haskoning DHV, Rob Doomen and Ming-Chen Ku from Pieters Bouwtechniek, Ruud Kersten from LSI Project Investment, Michel Schamp from Aronsohn and Jan van der Windt from Zonneveld Ingenieurs. I sincerely want to thank Gert van Appeldoorn from BAM Advies & Engineering, for his assistance and all the information he provided me with, and for the very good collaboration. His contribution on the cost analysis was priceless.

Someone ever said that true friends support you because they want to see you succeed. In this case I am proud to have some true friends both in Greece and the Netherlands, who have been supporting me mentally and practically. Special thanks to Angela and Tana for being always there. I would be ungrateful to forget my teammates at Punch, for their tolerance and support. Labrini thank you as well.

Finally, words are poor to express my gratitude to my family, my parents Antonis and Litsa, my brother Dimitris, and my partner in life Nikos, for their understanding, tolerance, encouragement and boundless love.

Maria Papageorgiou,

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ABSTRACT

The motivation to adapt a building in terms of an upward vertical extension lies within the need for urban renewable programs. Although this practice has significant supports to make the built environment more sustainable, there are also barriers which invariably concern costs. In the case of vertical extension projects, the constructional and structural implications are greater than for other renovation projects. Hence, the major objective of this study is to identify the *design parameters* that outline the optimal vertical extension in building renovation with respect to costs and environmental impact, and alongside, to compare the design options of *reuse and vertical extension* and *demolition and new structure*. For these purposes, a crossed research was carried out, including a literature review, the analysis of existing case studies, interviews, and at the end a comparative analysis, based on the design of a new case study.

First, the literature review presents the 'state of the art' of vertical extension projects from the structural engineer's standpoint. Then, the first list of critical design parameters were identified, on the basis of the analysis of the case studies and the interviews. The last part of this thesis, consists of the investigation of a new case study, an existing vacant building in the Hague, called Astoria. A preliminary structural design was carried out for different scenarios of vertical extension, which were defined with respect to the extent of interventions in the existing structure. Then, a global cost analysis and a Life Cycle Assessment are performed for both design options, presenting the results in comparative graphs.

As a result of the structural analysis, the foundation and the shear concrete core were figured out as the critical structural elements during the vertical extension. Moreover, environmental impact and costs analyses highlighted the addition of 4 storeys as the optimal vertical extension for Astoria; further vertical extension requires interventions in the existing structure, which makes the project economically unviable. Concluding, this research indicates five principal design parameters that define the optimal vertical extension, namely, the municipal policies, the foundation of the existing building, the absence or not of testing methods, the floor system of the new block, and, last but not least, a feasibility study, which reveals the relationship between value and costs. For future projects, it is advised to introduce these parameters in the initial design stages, as a quick scan, in order to investigate and optimize the possibilities for vertical extension. As an outcome of the Astoria comparative case study, it is further suggested to introduce vertical extension as a normal practice in building renovation, regarding the multiple benefits and perspectives that have been revealed for both developers and society on the basis of environmental impact and construction costs.

Keywords: vertical extension, design parameters, LCA, environmental impact, structure, reuse, construction costs, structural assessment, case study

1 INTRODUCTION

1.1 MOTIVATION

The man-made climate change since the last quarter of the 20^{th} century has been unprecedented. Economic growth and urbanization are the key drivers of change in modern, developed countries (Douglas J. , 2006). By 2050, almost 70% of the global population will be living in urban areas (United Nations, 2008). It is well understood that carbon emissions reduction is one of the most crucial strategies to mitigate the adverse impact of climate change. There is also a growing concern in the planning regime to incorporate the reduction of carbon emission in urban development (Yung & Chan, 2012). Buildings have a direct impact on the environment, ranging from the use of raw materials during construction, maintenance and renovation to the emission of harmful substances throughout the building's life cycle (Balaras, Droutsa, Dascalaki, & Kontoyiannidis, 2005). Buildings are responsible for more than 40 percent of global energy use and one third of global greenhouse gas emissions, both in developed and developing countries (UNEP, 2009). At the same time, the global shortage of materials is an increasingly pressing issue. At world level, civil works and building construction consumes 60% of the raw materials extracted from the lithosphere. From this volume, building represents 40%, in other words 24% of these global extractions (Bribián, Capilla, & Usón, 2011). Provided all the above, building sector is an obvious place to look in order to minimize energy consumption and carbon emissions (Feifer, 2011).

Governments (and institutes) around the world have identified the implications of climate change and have developed, the last decades, an international policy framework in order to inhibit its negative course. The European Commission, in particular, is looking at cost-efficient ways to make the European economy more climate-friendly and less energy consuming. Much has been achieved since the EU adopted its first package of climate and energy measures in 2008. The EU is now well on track to meet the 2020 targets for greenhouse gas emissions reduction (at least 20 per cent to 1990 levels) and renewable energy (increase of share to 20 per cent) and significant improvements have been made in the intensity of energy use (goal of 20 per cent improvement) thanks to more efficient buildings, products, industrial processes and vehicles (European Commission, 2014).

The concept of low carbon cities is closely linked to sustainable development and is arguably one of the most critical sustainability challenges facing the world in recent decades (Yung & Chan, 2012). The Brundtland Commission provided the often-quoted definition of sustainable development as "development that meets the needs of the present without compromising the ability of future generations to meet their own needs" (World Commission on Environment and Development, 1987). The reuse of existing buildings to suit the needs of the present and future generations, while avoiding demolition and reconstruction is one of the most sustainable form of urban development (Yung & Chan, 2012). Due to long building life cycles, maintenance and deconstruction management are likewise major levers to cope with resource efficiency and to enable closed-loop material cycles (Volk, Stengel, & Schultmann, 2014). Especially in industrialized countries with low new construction rates, activities of the construction sector increasingly shift to building modifications, retrofits and deconstruction of existing buildings (Penttilä, Rajala, & Freese, 2007; Mill, Alt, & Liias, 2013).

With the implementation of energy efficient design, the initial embodied energy of building materials becomes a more important consideration (Thormark, 2006). Although energy used in construction and making construction materials, during the life cycle of a building, varies between 10 and 20 per cent of total energy use of the building – unlike the energy consumption of buildings where the use and operational phase will consume the major share of energy – that does not mean that the resource consumption of the production and transport of construction materials should not be analyzed and taken into account (United Nations, 2010). Although the embodied environmental impacts of buildings are smaller than the environmental impacts in the operation phase, the embodied environmental impacts may be significant when the different time frame is considered. With the growing interest in sustainable buildings such as the zero-energy building, the embodied environmental impacts of buildings will become increasingly important (Jang, Hong, & Ji, 2015). In particular, extending the life of an existing building through reuse can lower material, transport and energy consumption and pollution and thus make a significant contribution to low carbon reduction and sustainability (Bullen, 2007). Adapting a property as opposed to constructing a new building helps to reduce energy consumption, pollution and waste (Douglas J. , 2006).

The motivation to adapt a building in terms of an upward vertical extension lies in the need for urban renewable programs. As pointed out by Remøy and van der Voordt (2014), while researching the drivers for adaptive reuse in London, New York, Toronto, Melbourne, Perth and Tokyo, 'sustainability aims, obsolete office buildings and a tight housing market were the most important conversion drivers' (Remøy & Voordt, 2014). These drivers are also present in Dutch real estate markets (Remøy & Voordt, 2014). Extensions are very interesting type of intervention since they instantly produce new, commercially valuable surface, which could compensate the costs of energy optimization (Cukovic-Ignjatovic & Ignjatovic, 2006).

As it is already aforementioned, over the last few decades, concern has been growing about resource efficiency and the environmental impact of material consumption (Pacheco-Torgal, 2014). One of the methodologies used to assess the environmental impacts of a given material is known as life cycle assessment (LCA) and includes the complete life cycle of the product, process or activity, i.e., the extraction and processing of raw materials, manufacturing, transportation and distribution, use, maintenance, recycling, reuse and final disposal' (SETAC , 1993). LCA is the most appropriate and accepted method used to provide a holistic assessment of the environmental impacts associated with a building and building materials (Cole, 1998; Junnila, Horvath, & Guggemos, 2006; Horne, Grant, & Verghese, 2009). Because LCA takes a comprehensive, systemic approach to environmental evaluation, interest is increasing in incorporating LCA methods into building construction decision making for selection of environmentally preferable products, as well as for evaluation and optimization of construction processes (Asdrubali, Baldassarri, & Fthenakis, 2013). Moreover, life cycle studies had focused on the quantification of energy and materials used and wastes released into the environment throughout the life cycle (Sharma, Saxena, Sethi, Shree, & Varun, 2011).

1.2 STATEMENT OF THE PROBLEM

However, the concept of adaptive reuse has significant supports as a positive strategy to make the built environment more sustainable, there are also barriers which invariably concern cost, such like: no economic benefit for the building owner's in reuse, extensive and costly refurbishment, ongoing maintenance costs higher than a new building, and maintaining the structural integrity of older buildings may be difficult (Bullen, 2007). From the other side, in general practice, the refurbishment and reuse of existent buildings is not always sustainable (Fontana, 2012).

In the case of vertical extensions, the constructional and structural implications are greater than for other renovation projects. Extra loading on the existing building may result to strengthening work of structural elements, fire safety requirements as well as fire and sound insulation might increase due to current regulations. Provided that costs are an inhibitory factor for owners, designers, investors, and in general different parties that get involved in such projects, more intense research should be directed to this direction.

Among the stakeholders on the real estate market, there is a general lack of knowledge about transformation processes (Wilkinson, Remøy, & Langston, 2014). Some research is directed last years towards the direction of gaining more knowledge on the field of vertical extension projects, however, regarding merely the structural design and implications related to these projects. Best practices, construction methods and processes have been developed, all of them associated with strengthening of the existing structure and the structural design of the new structure on top. The load bearing capacity of the existing structure (foundation, columns, walls, beams, floors) is unquestionably a first indicator for the amount of square meters we can add on top of it, and furthermore, structural alterations and interventions can lead to an increase of this load bearing capacity, and consequently to an increase of the amount of extra square meters. At this point, researchers should make the next step and explore other aspects of vertical extension projects as well. As mentioned in the motivation of this thesis, sustainable design and construction have an important role to play in helping to avoid increased vulnerability to the range of impacts arising from climate change and to manage risks through adaptation (Department for Communities and Local Government, 2012). The three facets of sustainable development, economic, social and environmental sustainability, should be incorporated in future researches. This is the rationale behind the current thesis that will try to draw new paths in the field of vertical extension projects, consider different features and methods of approach in order to be a useful tool for the different stakeholders involved in such projects. Due to the broad field of this subject, this subject will focus on the economic and environmental sides of a project and the emphasis will be put on materials and construction. More specifically, the main objective will be to research the parameters that outline the optimal vertical extension in building renovation with respect to costs and environmental impact. These parameters will be revealed during the literature review and the design of the case study as well.

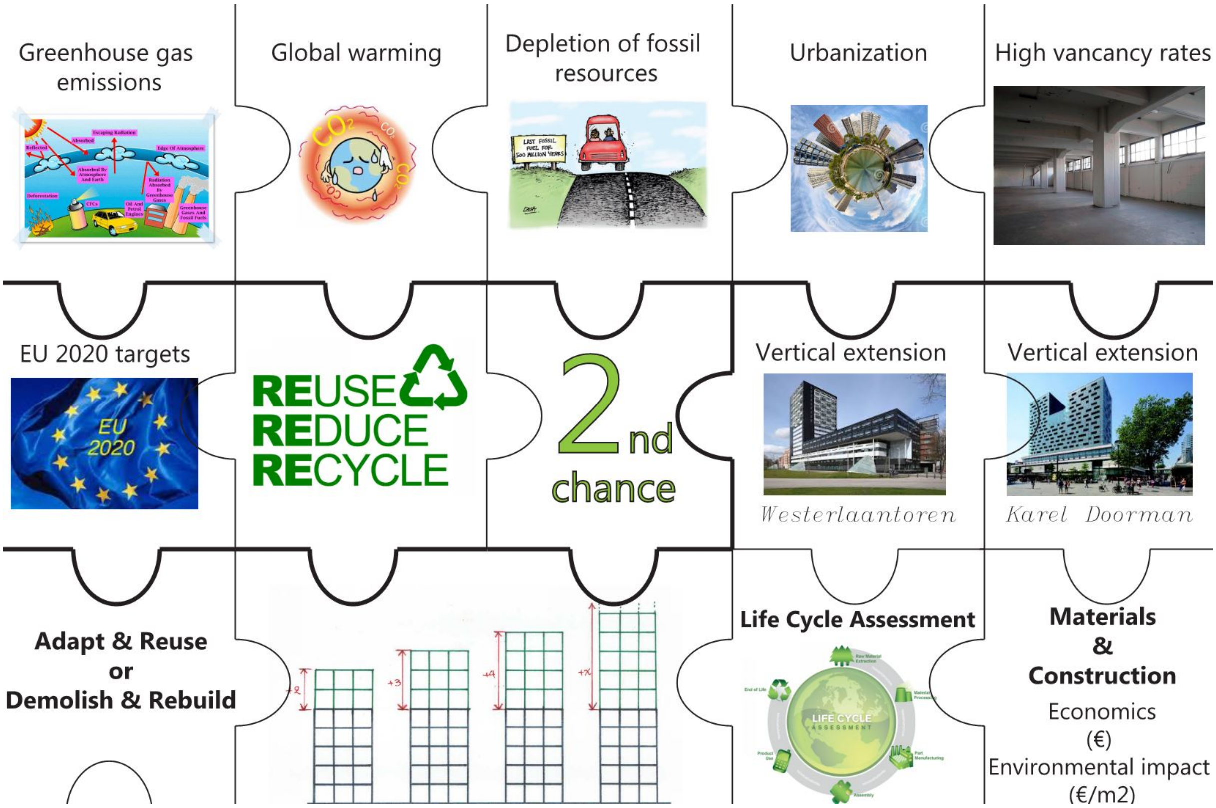

Figure 1-1 Motivation and problem statement

1.3 RESEARCH QUESTION

Which design parameters define the optimal vertical extension in building renovation with respect to costs and environmental impact?

1.4 RESEARCH OBJECTIVES

Literature review

- To highlight the predominance of reuse against demolition for coping with structurally vacant office buildings and to research the structural possibilities and limitations of vertical extensions in sustainable building adaptation deriving from structural as well as functional characteristics of office buildings.
- To describe the design approach and phases of vertical extensions in building renovation understanding the 'state of the art' behind this practice.
- To analyze a certain number of example case studies of vertical extension projects, focusing on specific aspects, trying to illustrate design and construction principles, point out common practices, relate them with the corresponding theory, and distinguish the parameters that defined eventually the extent of the vertical extension.
- To research and argue why a Life Cycle Assessment (LCA) is the best approach to assess environmental impact.

Design case study

- To investigate which load-bearing components are most critical and require the most drastic strengthening measures when the amount of square meters is increased.
- To examine what is the influence of the addition of extra storeys on the environmental impact and the costs of the building and conclude to an optimal vertical extension for this specific building.
- To identify from the design procedure the parameters that play the most important role in defining the optimal vertical extension of an existing structure.
- To compare the two design options for the Astoria building, reuse and vertical extension on the one hand, and demolition and new structure on the other hand, in terms of environmental impact and construction costs, and indicate the best option.
- To draw some useful directives that could be used from developers and decision-makers in building renovation and adaptation projects when having to deal with structurally vacant office buildings, with regard to possibilities for vertical extension.

1.5 METHODOLOGY

The different stages of a master thesis and the sequence of these stages are both important factors and can be helpful tools in the course of such a project. The current thesis consists out of four main stages: the introduction, the literature review, the design of a case study and the conclusions.

At the outset, time was spent in order to gain knowledge on the field of interest, to identify relationships between the different related variables, to pinpoint possible lack of knowledge or gaps in the literature, to observe the application of the theory in practice and potential implications connected to it and in general to identify the need for further research on the field of building renovation and vertical extension projects specifically. All the aforementioned constitute the introductory part of this thesis and help the reader to form an overview of the chapters following, the research question, the goals and the methodology to achieve them.

The stage that follows the introduction is the one of the literature review. This research focuses on specific goals, providing the necessary knowledge that will serve the purposes of the design case study and facilitate the following stages. The main research objectives of this phase are already discussed previously in this report. Additionally, the added value of the example case studies in the literature research should be mentioned. The aim is to highlight and confirm the outcomes of the literature review, with the intention of underlining links between theory and practice. A distinction of the main parameters that influence a vertical extension project will be made in the literature review and an effort will be done thereafter in order to pinpoint these parameters in the example case studies.

The final goal of the research is to define the parameters that outline the optimal vertical extension with regard to the costs and the environmental impact. In order to validate the parameters discerned from the literature review and the example case studies, a single case study will be designed. This will be the third stage of the thesis. The usefulness of a single case study is certainly not to draw generic conclusions, but to develop a suggested approach and highlight the most significant parameters of this approach. At the same time, the results of the structural, LCA and cost calculations of this case study will be interpreted in order to draw some conclusions for this case study itself and not in order to create a general theory.

At the last stage of the research, the author will hopefully be able to answer the main research question, and the research objectives will be met. At this stage, the author's critical insight is essential so that to understand and evaluate what went right, what wrong, which are the possibilities for further research and what is the contribution of this research in the scientific community and the professional field as well.

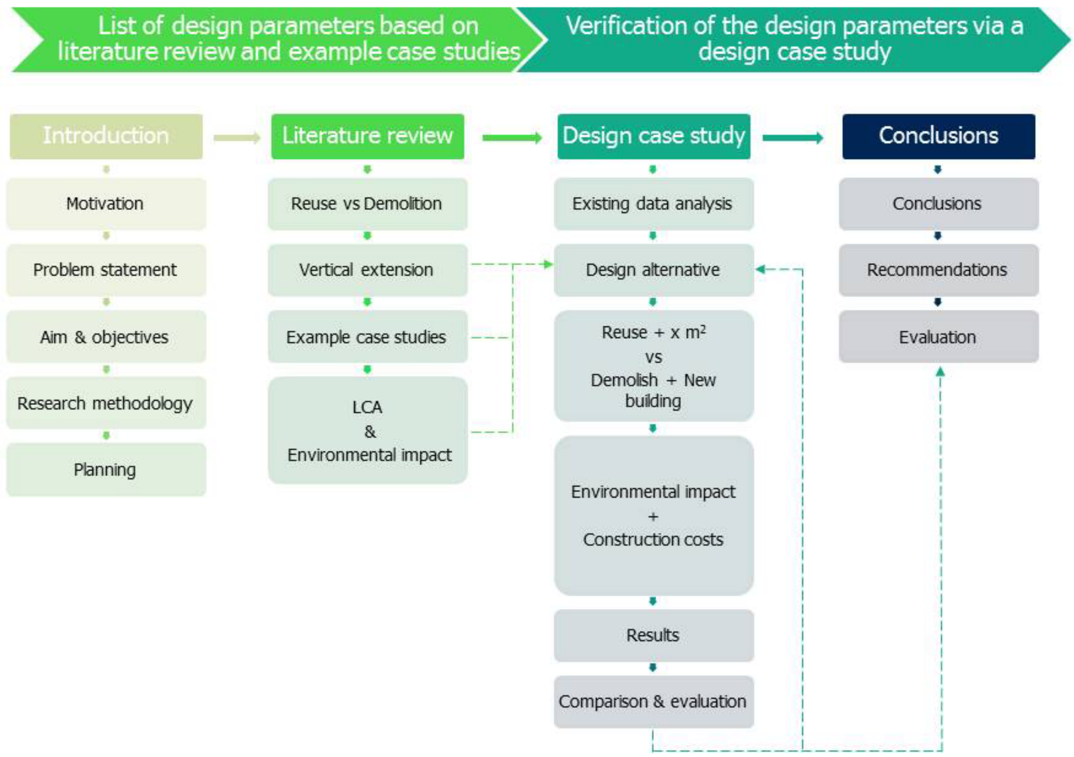

Figure 1-2 Research methodology

1.6 EXAMPLE CASE STUDIES

The example case studies are integrated in the literature review as a means to support and validate the literature research. The projects chosen are in their majority recently completed. Redevelopment, transformation or change of function are the main concepts behind them. Despite the fact that the current thesis deals with change of function of an office building to residential, the example studies do not all fulfill this criterion. Although, the different design approaches are examined and the dominant parameters is distinguished for all of them. Afterwards, it could be assessed whether the function parameter influences decisions which might define the final result. These projects are the following:

- De Karel Doorman, residential building, Rotterdam
- Groot Willemsplein, office building, Rotterdam
- Westerlaantoren, mixed use, Rotterdam
- Maritime Hotel, mixed use, Rotterdam
- St. Jobsveem, residential building, Rotterdam

The example case studies will be assessed on the following aspects:

1. General information
   Parties involved, location, year, former and post function.

2. Building renovation design process
   Vertical extension strategy, assessment existing structure, new structure, construction technique and fire safety.
3. Sustainability and costs in the design process
   How sustainability was integrated in the design process?
   Environmental impact and costs.
4. Design parameters
   Which were the parameters that defined the amount of extra squared meters?
5. Problems and problem-solving

Given the fact that the research question is so particular, interviewing the different parties involved in the aforementioned projects is also part of the literature review, especially for these projects where not a lot of information are available from publications, articles or reports.

1.7 THE DESIGN CASE STUDY

A design case study is used to validate the outcomes of the literature review. Through the involvement of IMd Raadgevende Ingenieurs in this research, an opportunity arises to work on a real-time study case. It is a project in the city center of The Hague for the renovation and vertical extension of the existing office building Astoria. The Archipel Designers (Archipel Ontwerpers) designed the conversion of the office building into an apartment building and IMd Raadgevende Ingenieurs is commissioned for the structural design. The existing building is analyzed in order to verify its current state, the condition of the building elements, the load bearing capacities and all other parameters that emerge from the literature review, in order to define the optimal amount of storeys to be added on top of it.

Literature review

2 Adaptive Reuse vs. Demolition

2.1 Introduction

The change to reuse and adaptation of buildings is a trend that has been clearly charted by many experts during the 2 last decades (Gallant & Blickle, 2005; Ball R. , 2002; Bon & Hutchinson, 2000; Kohler & Hassler, 2002). The importance of this trend is that extending the useful life of existing buildings supports the key concepts of sustainability by lowering material, transport and energy consumption and pollution (Douglas J. , 2002). Over the last decade vacancy levels in office markets worldwide are unprecedented (Remøy & Voordt, 2014). In the Netherlands, about 7 million square meters (15%) was vacant in 2013, whereas 3-8% is regarded 'normal vacancy', necessary to provide for mutations in the market. Half of the vacant office space is structurally vacant, i.e. vacant for 3 or more years (Remøy H. , 2010). This surplus in the office stock was the driven force behind the interest for adaptation in the Netherlands.

As buildings appreciate in years their operational and commercial performance decreases until eventually they fall below the expectations of owners and occupiers (Haakinen, 2007). Property owners have four possible strategies for dealing with vacant office buildings: consolidation, adaptation or upgrading, demolition and construction, and conversion to new functions (Wilkinson, Remoy, & Langston, 2014). Most owners choose for consolidation, which is translated in either doing nothing and wait for better times, or reaching for new tenants. Though smaller renovations are performed every 5 years (Douglas J. , 2006) at some point the building requires major adaptations (Wilkinson & Remøy, 2011). However, in markets with high levels of vacancy or with location obsolescence, there is a risk that the positive effect of adaptation will be less than the costs of intervention (Wilkinson, Remøy, & Langston, 2014). Another alternative for copying with structural vacancy is demolition and new construction. Responding to the declining performance of the buildings, has resulted in decisions to purely demolish and redevelop buildings based on economic grounds (Pearce, 2004). This kind of interventions have interesting dynamics since they create possibilities to meet the future users' needs. However, there are two reasons why demolition and new construction is not the best option: on the one hand redevelopment takes time and causes income delay, and on the other hand, if the building is technically in a good state, redevelopment is a waste of resources and conflicts with global aims for sustainable development (Remøy & Voordt, 2014). The decision to demolish may be premature if it ignores the residual utility and value of buildings that could be optimized by adapting and refurbishing using the process of adapting reuse (Ellison, Sayce, & Smith, 2007).

Failing to optimize buildings can result in their residual lifecycle expectancy not being fully exploited, which is not sustainable use of built stock (Bullen & Love, 2010). Finally, conversion is the fourth way of coping with structural vacancy. As Remøy and van der Voordt (2014) suggest, conversion sustains a beneficial and durable use of the location and building, implies less income disruption than redevelopment and can have high social and financial benefits.

2.2 DECISION-MAKING CRITERIA

Table 2-1 lists various options for property owners, along with the benefits and drawbacks, as they have been summarized by Wilkinson, Remøy, & Langston (2014).

Table 2-1 Options for property owners for structurally vacant buildings (Wilkinson, Remøy, & Langston, 2014)

Option	Benefits	Drawbacks
Maintain in current state (consolidate)	Preserves the property Sustains existing use Ensures ongoing service and lifespan	Requires maintenance costs though no incomes are generated
New tenancy – better study of the market	Find a suitable tenant, may insure ongoing beneficial use of the property	May be time consuming to find a user for a structurally vacant building; requires maintenance, refurbishment and incentives
Mothball	Minimizes running costs, such as cleaning, heating and lighting	Costly to keep safe and secure; vulnerable to vandalism and squatting, dust and dirt accumulation and dampness in the building; no rental income
Anti-squat	Minimizes running costs, secures the building against squatting and vandalism	Exposed to wear and tear, inhabitation may influence possible tenancy negatively
Dispose	Realizes asset/site value, reduces management and operating costs	Loss of potentially useful asset, price may not correspond to book value
Demolition and new building	New building tailored to meet users preferences	Disruptive and expensive, delay if income, location characteristics cannot be influenced
Adapt and renovate	Enhances the physical and economical characteristics of the building, delays deterioration and obsolescence, reduces the likelihood of redundancy, sustains the building's long-term beneficial use	Disruptive and expensive, extended lifespan is unlikely to be as great as a new building, upgraded performance cannot wholly match that of a new building, location characteristics cannot be influenced
Convert	Enhances and alters the physical and economic characteristics if the building, prevents deterioration and obsolescence, sustains the building's long-term beneficial use, sustains social coherence in the area	Disruptive and expensive, market uncertainty, location characteristics may not suit new function, building costs may be out of control, new rental function may not be the core business of the owner

2.3 TOOLS AND INSTRUMENTS

As result of research, a number of different tools and instruments has been developed to evaluate the potentials and the feasibility of buildings' conversion projects, on the basis of a variety of aspects, such as the physical and functional characteristics of the building, the location, the organizational and market aspects (Hek, Kamstra, & Geraedts, 2004; Zijlstra, 2006; Geraedts & van der Voordt, 2007; Hofmans, Schopmeijer, Klerkx, & van Herwijnen, 2007) and can be used at the different phases of such projects. Three of these tools will be presented in the following. The first highlights the architectural value of an existing building. The last two consider inter alia, an extensive list of physical and technical factors related to the building, and hence, useful from the structural engineers' standpoint.

2.3.1 Architectural Value

Wilkinson, Remoy and Langston (2014) analyze the Architectural Value and refer to it more as a method rather than a tool. Interventions in a building should be preceded by a study of a building's contectual aspects (i.e. original commission, location and architect), next to a study of the building's architecture, in order to decide the potential changeability of the building.This method assumes three levels of time: commencing, ageing and continuing. Within these layers of time, the building elements, space, structure, substance and services are studied. The technical lifespan and the technical state of the building are important, as technical decay is often seen as the most important aspect of the ageing of the bilding, and thus, it is also important for the continued life of the building. Analyzing buildings with these aspects in mind, new possibilities are created, offering possibilities for a different way of living, working and recreating. By studying the possibilities before starting the desing process, buildings can be kept for continuation instead of being lost in decay.

2.3.2 The Transformation Potential Meter

In order to be able to measure the transformation potential both at location and at building level, the 'transformation potential meter' was developed by Geraedts and van der Voordt (2003, 2007). The five-steps method includes an analysis of the local market and critical characteristics of the location and the building(s), an economic feasibility study and a check on a number of risk factors from a functional, architectonical, juridical and technical point of view. Using veto criteria and gradual criteria, the method shows which features of the location and the building favor successful transformation, and which hinder it. According to experts in the field of real estate, the transformation prospects of the current supply of the office buildings depend primarily on three factors:

the duration of the vacancy, the reason for vacancy (market, location or building) and the municipal policy. In addition, the supply of transformed offices into housing must be in line with the demand of future tenants, with regards to both the location (residential) and the features of the building.

Step 0 is an inventory of the unoccupied office space. Step 1 is a Quick Scan of the transformation potential of this stock, using a limited number of veto criteria with respect to Market, Location, Building and Organization. When a project meets one or more of these criteria it does not have sufficient transformation potential, resulting in a NO GO decision. Step 2 is a feasibility scan with a number of appropriate criteria, showing the features of the location and the building (Table 2-2) that lend themselves to transformation and which do not. This leads in step 3 to the assignment of an overall score expressing the transformation potential of the building(s) in question, varying from non-transformable to highly suitable for transformation. Depending on the results, step 3 leads either to a NO GO decision or to further refinement of the feasibility study in two subsequent phases: step 4 (a financial feasibility scan) and step 5 (a risk assessment checklist). Depending on the nature of the project involved, step 5 may come before step 4. The five steps of the transformation potential meter are presented in Table 2-3.

Table 2-2 Step 2b - Appraisal of suitability of an office building for transformation to residential housing with reference to features of the building itself

BUILDING						
Aspect		Gradual criterion		Data source	Appraisal	
Functional					Yes	No
1	Year of construction or renovation	1	Office building recently built (< 3 years)	Year of construction		
		2	Recently renovated as offices (< 3 years)	Year of renovation		
2	Vacancy	3	Some office space still in use	e.g. NEPROM		
		4	Building unoccupied < 3 years	ditto		
3	Features of new dwelling units	5	≤ 20 -person units (50 m2 each) can be made	≤ 1000 m2 useful area		
		6	Layouts suitable for local target groups cannot be implemented	Design sketch		
4	Extendability	7	Not horizontally extendable (neighboring buildings)	On-the-spot investigation		
		8	No extra storeys (pitched roof or insufficient load-bearing capacity)	On-the-spot investigation		
		9	Basement cannot be built under building	Inspection and/or estate agent		
Technical						
5	Maintenance	10	Building poorly maintained/looks in poor condition	External visual inspection		
6	Dimensions of skeleton	11	Office depth < 10 m	Estate agent or inspection		
	Module of façade determines placing of walls	12	Module of support structure < 3.60 m	On-site or estate agent		
		13	Distance between floors > 6.00 m	On-site or estate agent		
7	Support structure (walls, pillars, floors)	14	Support structure is in poor/hazardous condition	On-site inspection		
8	Façade	15	Cannot be made to blend with surroundings or module > 5 m	On-site or estate agent		
	External spaces dependent on target group	16	Façade (or openings in façade) not adaptable	On-site inspection		
	Protected monuments: limits on adaptation	17	Windows cannot be reused/opened	Inspection/new design		

9	Installations	18	Impossible to install (sufficient) service ducts	Inspection/new design		
Cultural						
10	Character	19	No character in relation to surrounding buildings	On-site inspection		
	cf. Location, 'Tone of neighborhood'	20	Impossible to create dwellings with an identity of their own	Inspection/new design		
11	Access (entrance hall/lifts/stairs)	21	Unsafe entrance, no clear overview of situation	Inspection/new design		
Legal						
12	Environment	22	Presence of large amounts of hazardous materials	On-site or municipality		
	Exposure to sunlight, air and noise	23	Acoustic insulation of floors < 4 dB	Inspection/new design		
	pollution, hazardous materials	24	Very poor thermal insulation of outer walls and/or roof	On-site or municipality		
		25	< 10% of floor area of new units gets incident daylight	On-site inspection		
13	Requirements of Bouwbesluit (Dutch official rules and standards for the building industry)	26	No lifts in building (> 4 storeys), no lifts can be installed	On-site or estate agent		
	concerning access and escape route	27	No (emergency) stairways	Inspection/new design		
		28	Distance of new unit from stairs and/or lift ≥ 50 m	Inspection/new design		
	Maximum possible (weighted) Building score = 28 x 3 = 84			Total number of Yes's for Building:		x
				Default weighting:	3	=
				Building score:		B
				Maximum possible Building score (28x3):	84	

Table 2-3 The five steps of the transformation potential

Transformation Potential Meter			
Step	**Action**	**Level**	**Outcome**
Step 0	Inventory market supply of unoccupied offices	Stock	Location of unoccupied offices
Step 1	Quick Scan: ignition appraisal of unoccupied offices using veto criteria	Location Building	Selection or rejection of offices for further study; GO / NO GO decision
Step 2	Feasibility scan: further appraisal using gradual criteria	Location Building	Judgment about transformation potential of office building
Step 3	Determination of transformation class	Location Building	Indicates transformation potential on 5-point scale from very good to NO GO
Further analysis (optional, and nay be performed in reverse order if so desired):			
Step 4	Financial feasibility scan using design sketch and cost-benefit analysis	Building	Indicates financial/economic feasibility
Step 5	Risk assessment checklist	Location Building	Highlights areas of concern in transformation plan

2.3.3 ABT-Quickscan

ABT, a multidisciplinary consultancy firm in structural engineering, has developed a tool for assessing the conversion capacity of existing buildings. During this quick scan, two aspects are considered as the most important: the possible new functions in the building and the costs of conversion. The ABT-Quickscan consists of three steps: inspecting the building (collecting information), controlling (tests based on the standards and legislation), and appraising (evaluating the technical state, functionality, flexibility and adaptability, architectural, historical and 'visual and emotional' quality of the building. The method is assessing, on the basis of knowledge and experience of the engineers, five aspects of the building (structure, façade, fixed interior and installations) and three of the location (condition, quality and legislation) (Hofmans, Schopmeijer, Klerkx, & van Herwijnen, 2007). As it can be seen in the 'tree diagram' structure of the method (Figure 2-1), the building is forming the central element and the location is seen as the sixth of the building's attributes.

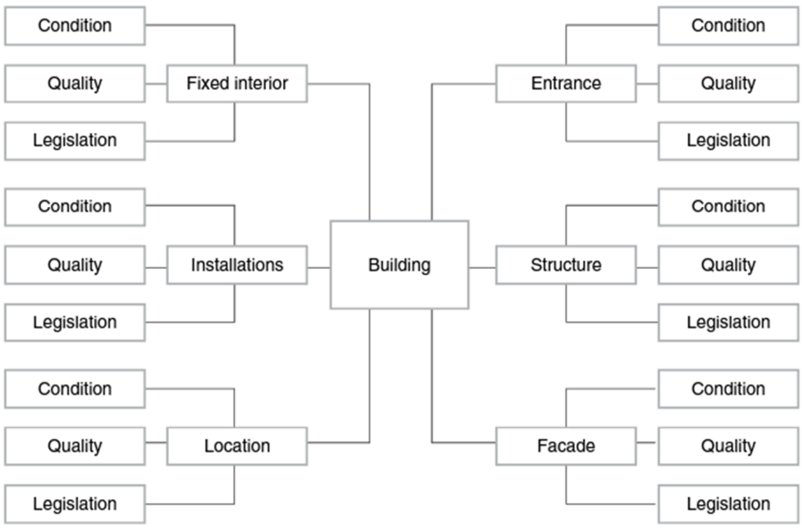

Figure 2-1 Tree diagram ABT-Quickscan method

2.4 SUSTAINABILITY FACTORS

Adaptive reuse enhances the longer-term usefulness of a building and is therefore a more sustainable option than demolition and rebuilding (Bullen, 2007). The positive benefits for adaptive reuse identified during the research of Bullen (2007) also support the tenets of sustainability and include:

- reducing resource consumption, energy and emissions;
- extending the useful life of buildings;
- being more cost effective than demolition and rebuilding;
- reclaiming embodied energy over a greater time frame;
- creating valuable community resources from unproductive property;
- revitalizing existing neighborhoods;
- reducing land consumption and urban sprawl;
- enhancing the aesthetic appeal of the built environment;
- increasing the demand for retained existing buildings;
- retaining streetscapes than maintain sense of place; and
- retaining visual amenity and cultural heritage.

The existing building stock has the greatest potential to lower the environmental load of the built environment significantly within the next 20 or 30 years (Bromley, Tallon, & Thomas, 2005; Rovers, 2004; Balaras, Dascalaki, & Kontoyiannidis, 2004). Adaptive reuse extends the building's life by avoiding the creation of demolition waste and saving the embodied energy, is well recognized as contributing to reducing low carbon emissions, mitigating climate change and hence achieving development (Yung & Chan, 2012)

Demolition is often selected when the life expectancy of an existing building is estimated to be less than a new alternative; despite any improvements that adaptive reuse may inject (Douglas J. , 2002). Undoubtedly, the life cycle expectancy of the materials of a new building exceeds the one of the materials of an older building. This affects directly the maintenance costs of the adapted building, which will be higher than these of a new building. However, as Bullen & Love (2010) suggest, adaptive reuse can offer a more efficient and effective process of dealing with buildings than demolition. This is because it is deemed to be safer as it reduces the amount of disturbance due to hazardous materials, contaminated ground and the risk of falling materials and dust. Particularly, working on site is also more convenient because the existing building offers a work enclosure that reduces interruptions from extreme weather conditions. In the same direction, transformation is a much more environmentally efficient way to achieve the same result than are demolition and rebuilding (Itard & Klunder, 2007). Itard and Klunder (2007) found from a renovation study that adapting buildings for a new use minimizes construction waste, used fewer materials and probably uses less energy than demolition and rebuilding. Evidence clearly suggests that the opportunities created by adaptive reuse outweigh those presentes by demolition and reconstruction (e.g., Ball R. , 1999; Brand , 1994; Cooper, 2001; Douglas J. , 2002; Kohler & Hassler, 2002).

2.5 COST FACTORS

There has been an extensive discussion on the relative costs, the benefits, the constrains and the risks of reuse versus demolition and new build. There is a growing perception that it is cheaper to convert old buildings to new uses that to demolish and rebuild (e.g., Ball R., 2002; Department of the Environment and Heritage, 2004; Douglas J. , 2002; Geraedts & van der Voordt, 2007). Hall (1998), Douglas (2006), and Kohler and Yang (2007) have proffered that the costs of reusing buildings are lower than the costs of demolition. It is potentially cheaper to adapt than to demolish and rebuild, inasmuch as the structural components already exist and the cost of borrowing is reduced, as contract periods are typically shorter (Shipley, Utz, & Parsons, 2006).

According to Wilkinson, Remøy and Langston (2014), the financially most interesting strategy for coping with structurally vacant office buildings, comparing conversion to consolidation, adaptation and demolition with new construction, can be found by calculating the net present value (NPV) for these different strategies. Muller, Remøy and Soeter (2009) calculated the NPV for a sample of structurally vacant office buildings in Amsterdam. Since many of the stakeholders in the market are sceptical of building conversion, a worst-case scenario was calculated for the conversion building costs, and a best-case scenario (from the investors' perspective) was used for calculating the value of the structurally vacant office buildings. The NVP calculations revealed that in 40% of the cases studied conversion proved to be financially viable. Calculating a best-case scenario would conclude conversion to be the optimum strategy for a higher percentage of the structurally vacant office buildings. Moreover, the financial feasibility of the conversion could be additionally enhanced by extending the building horizontally or vertically or by adding a commercial programme like retail or leisure functions to the ground floor of the building. The possibilities depend on the location and the building; e.g. the purchasing price of apartments and the rental price of offices differ according to location, so the conversion potential in some locations was much higher than in other locations. In addition to that, it was demonstrated that a vertical extension could be possible for a large amount of the existing office buildings as these were more sturdily constructed than the standard apartment buildings, and so most office buildings could be extented vertically with one or two floors.

Table 2-4 shows the estimated range of total investment costs (acquisition and building costs) for the transformation of existing (office) buildings to student accommodation, per dwelling unit and per m^2 of GFA, compared with the costs of comparable new buildings. The data are based on a large number of projects carried out by the housing association Stadswonen in Rotterdam, collected by Vrij (2004) and indexed by Geraedts and van der Voordt (2007).

Table 2-4 Expected investment costs per dwelling unit and per m2 GFA for student accommodation

Type of construction project		Type of budget	Costs per unit	Costs per m^2 GFA
Transformation	Much demolition and modification	Acquisition budget for student unit	10.000-15.000	
		Residual budget for renovation costs	27.000-33000	540-660
	Much reuse (including façade)	Acquisition budget for student unit	20.000-25000	
		Residual budget for renovation costs	21.000-26.000	420-540
New construction		Student unit	36.000-39.000	720-780
		Social housing		890-970
		Luxury flat		1.100

On the other hand, recent research on the adaptive reuse of heritage buildings has proven that there is a consensus amongst stakeholders from the building industry that the economic viability of the new operating use has been the key hurdle to successful adaptive reuse (Yung & Chan, 2012). In the same research, the difficulty in achieving cost efficiency is highlighted, while, conserving the historic value of the building is challenging and incurs extra costs (an approximation is given of additional 30% to the costs and double time for project completion). However, a heritage consultant raised this issue, "Adaptive reuse is an expensive investment, if people only count the economic return and overlook the intangible non-economic values, then the economic efficiency seems to equal to zero" (Yung & Chan, 2012).

2.6 Conclusions

In this Chapter the rhetorical question 'adaptive reuse or demolition and rebuild' has been researched in the existing literature, taking into consideration sustainability and costs factors. Certainly, there is no single answer for such a question. The literature suggests to the stakeholders different criteria and instruments to be implemented in the decision-making phase. The final choices are always dependent on the particularities and the issues related to every building.

As far as sustainability is concerned, there is a clear trend in the literature that supports the dominance of adaptive reuse against demolition and rebuilding, however, solely considering environmental issues and excluding social and economic criteria.

Looking at the economic factors, the conversion of structurally vacant office buildings is presented to be a promising strategy in comparison to consolidation, adaptation and demolition with new construction. However, possibilities depend highly on location and building. The literature distinguishes net present value (NPV) calculation as a reliable method to make choices based on financial arguments. With regard to redevelopment of heritage buildings, the uncertainty about the economic viability and the difficulty in achieving cost efficiency are underlined, not disregarding the non-economic values of such projects.

3 VERTICAL EXTENSION

3.1 INTRODUCTION

The starting point for a vertical extension project, as for every conversion project, is the assessment of the existing building. The analysis of the existing building is an on-going task that is repeated many times as design and construction proceeds. The steps of this procedure are described in ISO 13822:2010 and are summarized in the following flowchart:

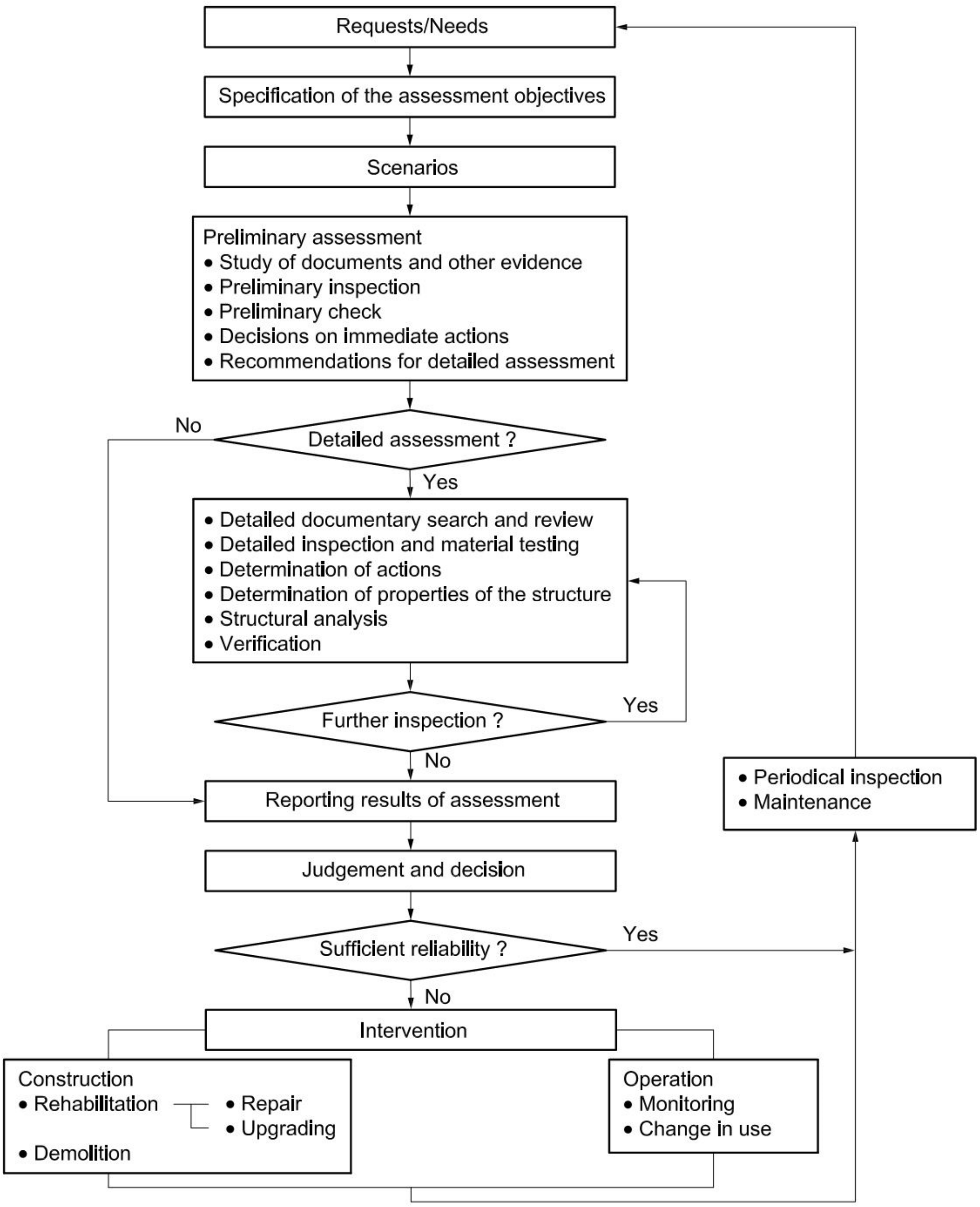

Figure 3-1 Flowchart for the general assessment of existing structures (SAMCO, 2006)

The main stages of this process will be presented in this chapter, and are summarized in the following:

1. assessment levels;
2. review and analysis of available documents;
3. investigation of the strength of the existing structure;
4. soil investigation;
5. determination of the new loads, norms and standards;
6. methods of structural analysis; and
7. structural modifications and vertical extension.

3.2 DEFINITIONS

The definitions describing the different building measures are vague and not applicable in a universal level. Giebeler (2009) attempted to define the various terms and distinguish them from each other. Among the various definitions Giebeler (2009) suggests, are also the ones describing the vertical extension works in existing buildings. These are presented in the following:

Total refurbishment. Demolition measures during total refurbishment projects are very extensive. The demolition returns the building more or less to its loadbearing carcass. The primary structure remains essentially unaltered. Typical measures include the complete replacement of the infrastructure and the upgrading of all building components to meet the requirements of the latest legislation and standards.

Conversion. Conversions always affect the structure of a building. They extend the concept of refurbishment to interventions in the load bearing members and/or the interior layout. In conversion projects it is therefore essential to appraise the existing loadbearing structure. Total refurbishment measures almost always involve conversion work, meaning that many construction projects are best described by using more than one term, "total refurbishment plus conversion". Changes to the structure always require structural calculations, which must also take into account the existing building fabric. This makes early, often destructive, investigations of the materials and methods used unavoidable, e.g. cutting open a concrete slab to establish the position and nature of the reinforcement.

Extensions/Additions. An extension is a new structure that is directly connected with the use of the existing building. The planning work should consider that conversion work at the junction with the existing building is usually unavoidable and therefore structural issues are involved. A frequent cause of problems is the differential settlement that can occur between the old and the new parts of the building, especially in the following cases:

- different foundation levels;
- building the foundations for the new works in the region of the previous excavation;
- building the foundations in different soils/adding extra storeys to only part of the existing building (subsequent settlement); and,
- dewatering measures for the new works, e.g. lowering the water table.

Note: It should be mentioned that vertical extension is mentioned in the literature also with the terms vertical expansion or vertical phasing.

3.3 Assessment levels

The assessment of existing structures can be carried out with methods of varying sophistication and effort. The core objectives are to analyze the current load carrying capacity and to predict the future performance with a maximum of accuracy and a minimum of effort. Unduly conservatism but also too lax restrictions should be avoided (Rücker, Hille, & Rohrmann, 2006). In general assessment procedures can be classified into three groups: measurement based assessment, model based assessment and non-formal assessment. It is recommended to start the assessment with simple but conservative low level methods and, in case the assessment failed, move on with more refined upper levels.

SAMCO association, proposes in the 'Guideline for the Assessment of Existing Structures' (SAMCO, 2006) the assessment levels as described below:

<u>Level 0: Non-formal qualitative assessment:</u> Assessment, based on experience of the engineer, is mostly used for a pre-evaluation of the structure. One is able to evaluate visual deterioration effects like corrosion of steel members or visual signs of damage (cracks, spalling).

<u>Level 1: Measurement based determination of load effect:</u> Assessment of serviceability by measurement of performance values and comparison with threshold values. There is no structural analysis carried out. The threshold values can be given in codes or individually specified.

<u>Level 2: Partial factor method, based on document review:</u> Assessment of load-carrying capacity and serviceability using information from design, construction and inspection

documentation. Structural analysis is generally carried out using simple methods. Safety and serviceability verification is based on partial factors.

Level 3: Partial factor method, based on supplementary investigation: Assessment of load-carrying capacity and serviceability using information from site specific detailed non-destructive investigations. Structural analysis is carried out using refined methods and detailed models. Safety and serviceability verification is based on partial factors.

Level 4: Modified target reliability, modification of partial factors: Verification of the load–carrying capacity with site-specific modified partial safety factors. Structural properties as well as external circumstances can influence the safety measure. Practically, modifying of partial factors is carried out for groups of structures with similar structural behavior or load influences.

Level 5: Full probabilistic assessment: Assessment, taking into account all basic variables with their statistical properties. Structural reliability analysis is used directly and instead of partial factors. Uncertainties are modelled probabilistically.

3.4 AVAILABLE INFORMATION

The structural evaluation of the existing building is one of the first phases of a vertical extension project, essential in its success. The examination is complete and accurate to determine the existing conditions and the process to implement. The current state of the building should be defined through information regarding the age of construction, the architectural and structural drawings, and reports with structural design calculations, where amongst other, material specifications and standards used can be found. These information is well documented most of the times at the archives of the local authorities. Furthermore, information such as geotechnical surveys, cone penetration tests and foundation advice are rarely available at the local authorities. In this case, it is advisable to contact the contractor or the engineering consultancies, which also keep archives of the realized projects.

In case the aforementioned information is not available, there is a variety of methods and tests in order to specify the technical characteristics of the existing structure. It is worth to mention at this point, that these methods and tests are not applicable exclusively in case of missing available information but also in the opposite case, so that to verify the information available. In this case, on site-measurements and visual inspections accompany the conversion measures during all phases of the work.

3.5 INVESTIGATING THE STRENGTH OF EXISTING STRUCTURES

Investigating the existing structure is one of the most significant phases of the feasibility study at the beginning of every project. Visual and technical assessments are used in order to indicate the quality and the mechanical properties of the load bearing systems. There is a variety of testing and research techniques which help to clarify the condition of the existing structure. To assess physical and strength properties in the building diagnostics the non-destructive testing (NDT) methods are widely used (RUNKIEWICZ, 2009). In particular, non-destructive tests are used to assess: compression strength and tensile strength, homogeneity, size and distribution of honeycombing and cavities in concrete, concrete-concrete connections and steel-wood connections in nods, stiffness, thickness and destruction of elements. Another category is the destructive testing (DT) methods, where the material is broken down in order to determine mechanical properties. Destructive tests are best when used together with non-destructive tests.

3.5.1 Non-Destructive Testing Methods

Runkiewicz (2009), reviewed the non-destructive testing methods and summarized them in the following categories:

- **sclerometric methods**, which are based on the measuring of hardness of the near-surface layer of the material;
- **acoustic methods**, which consist in measuring, among others, speed and other characteristics of propagation of longitudinal and transverse waves in the material (e.g. impact-echo method);
- **radiological methods**, which use, among others, the absorption of X-rays and gamma rays passing through the material and their parameters of dispersion and suppression;
- **electric and electromagnetic methods,** which use electric and dielectric properties and characteristics of electric field (in the material in its proximity);
- **semi-non-destructive methods,** for materials in the structure (e.g. pull-out method);
- **complex methods,** using several testing methods.

Some of the mostly used test tasks for building diagnosis have been reviewed and summarized by Flohrer (2010) in the following table:

Table 3-1 Test tasks for building diagnosis (Flohrer, 2010)

Test problem	Test method	Aim of investigation
Concrete compressive strength	Bouncing hammer (Schmidt hammer) and destructive testing of drilling cores	Categorization of supplied concrete into classes of compressive strength
Surface tensile strength	Test of tensile strength	Application of composite layers on old concrete surfaces
Concrete cover, determination of diameter of reinforcement	Cover meter, radar (deep reinforcement)	Assessment of the durability and the load-carrying capacity
Position and alignment of reinforcement	Cover meter, radar, radiography	Assessment of the durability and the load-carrying capacity
Detection of defects inside concrete, structural modifications	Radar, ultrasonic echo, impact–echo	Assessment of homogeneity of massive elements
Determination of the thickness of the structure, depth of installation parts or defects	Impact–echo, ultrasonic echo, radar	Unilaterally accessible structural elements, displacement bodies inside concrete, steel installation parts. Detection of insulating layers or dividing layers, multilayer components
Layer composition of wall and floor	Radar and further minor destructive testing (e.g. endoscopy)	For the large-scale stock-taking within the building diagnosis
Moisture content of the elements	Microwaves, radar, capacitive methods	Determination of the moisture content of elements and building materials
Location of tendons (lateral position, depth position)	radar	Reliable detection of tendons as a pre-study in advance of other investigations of the tendons or for repair work
Compaction faults inside tendons of post-tensioning	Ultrasonic echo	Contribution to the stability analysis of a pre-stressed concrete structure
Active corrosion of reinforcement	Potential difference method	Assessment of the durability and the stability
Cracks of tension wire cracks	Magnetic field method	Investigation of pre-stressed concrete elements with regard to possible cracks of tension wires
Glued laminated timber beams	Ultrasonic echo	Investigation of glulam beams with regard to structure or delaminations

The main of the testing methods, that are used in the construction industry, summarized in the prior table are further presented in Appendix A.

3.5.2 Destructive Testing Methods & Laboratory Tests

The non-destructive testing methods are combined, in most cases, with destructive testing techniques, in order to derive the most reliable conclusions for the mechanical and physical properties of the reinforced concrete and the steel reinforcement bars.

Extraction of concrete cylindrical specimens

The extraction of concrete cylindrical specimens (Figure 3-2), cores, is necessary to perform laboratory tests, such as: characteristic compressive and tensile strength, carbonization depth, chloride and sulphate content, chemical analysis, cracking etc..

Figure 3-2 Concrete core cutting (Skill Gulf, 2015)

Pull-off test

The concrete adhesion (pull-off) strength and mode of failure of a coating from a concrete substrate are, also, important performance properties. The pull-off adhesion test determines the maximum perpendicular force that a surface area can bear before a temporarily stuck plug of material is detached. This type of test is assisted by the NEN-EN 12504-3: Testing concrete in structures – Part 3: Determination of pull-out force.

Rebar exposure

Partly demolition and exposure of the underlying reinforcement bars is often a destructive testing technique when information from structural drawings is not available, but also to verify the existing structural drawings. When there is no sufficient information about the existing reinforcement, it is common to remove a number of steel rebars, so as to perform laboratory tensile strength tests.

3.6 SOIL INVESTIGATIONS

Soil investigations reveal the actual state of the subsoil with regard to the soil parameters and the underground watertable level. The project engineer sets a strategy for the soil investigations based on completeness of the available data. The archived geotechnical reports and on site investigations have to be researched in order to conclude whether these provide enough information for the new approach of the building. In case the existing information is not enough, the missing parameters are specified and the most common techniques are chosen in order to get this information.

Cone Penetration Tests (CPTs) provide a rapid and cost effective means in order to measure and quantify the geotechnical properties and characteristics of the sediments (Figure 3 9). Normally, CPTs are executed in the perimeter of the existing building. These results are not fully indicative of the underground condition of the foundation. In case there is a foundation plan of the existing piling foundation, the more expensive option of performing a CPT from indoor is chosen. A few locations are chosen for this reason in order to have the most representative results. However old CPTs are many times available from the time the building was originally built, the low costs of the CPT technology and the expected different soil characteristics, due to eventual soil compaction and settlements, are main driving forces to perform new CPTs.

Kyakula, Kapasa , & Opus (2006) have set out the different considerations related with soil investigations in vertical extension of reinforced concrete. Soil investigation must be undertaken to determine (1) the bearing capacity of the soil, (2) its settlement rate and (3) the position of the water table. One of the easiest methods is to dig trial pits and visual inspections carried out, then samples with minimum disturbance are collected for subsequent laboratory testing. Where possible, drilling should be undertaken as this enables one to obtain undisturbed samples from which settlement rate and bearing capacity may be obtained. For soils that loosen, such as sand and gravel, a plate-bearing test can be used to determine the bearing capacity of the soil in-situ and designing of the static loads on spread footings. If the strength of the soil is not adequate for the increased loading, it is necessary to improve on the foundations by introducing piles or enlarging the footing and reinforcing it betters to sustain the increased loading.

All the aforementioned, are certainly proportional to the stage of development of the project as well as to a risk analysis that takes into account the convergence of the available information. If both the archived information and the information of current databases converge to the same underground conditions then probably the risk to be taken of not making a new soil investigation is not so significant.

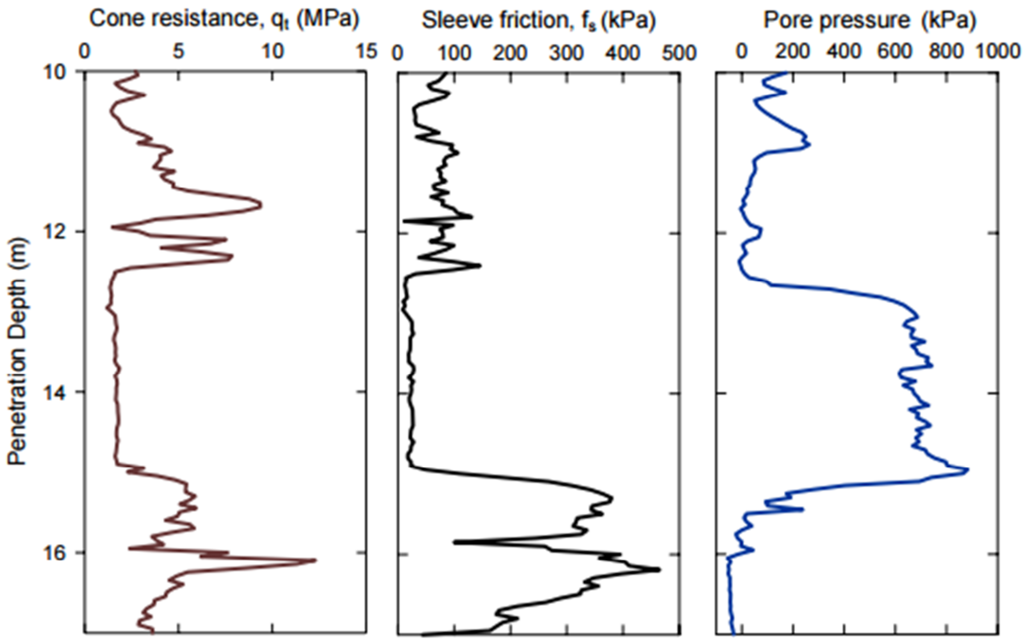

Figure 3-3 Profiles of cone resistance, sleeve friction and pore pressure of CPT

3.7 Reliability verification

3.7.1 General

ISO 13822 states that the verification of an existing structure should normally be carried out to ensure a target reliability level that represents the required level of structural performance. Current codes or codes equivalent to ISO 2394 that have produced sufficient reliability over a long period of application may be used. Former codes that were valid at the time of construction of an existing structure should be used as informative documents.

From the structural engineer's standpoint, the European and Dutch standards used during the design process, differ according to the following cases:

- For new buildings
  - Eurocode: NEN-EN 1990 – Basis of structural design
- For assessment of existing structures in case of renovation and disapproval
  - Eurocode: NEN-EN 1990 – Basis of structural design
  - Eurocode: NEN 8700 – Basic Rules
  - Eurocode: NEN 8701 – Loads

According to NEN 8700, when assessing an existing building in case of renovation or disapprove, deviations from the rules for new buildings (NEN-EN 1990) are possible, with regard to:

- partials factors;
- design or remaining service life and reference period;
- the characteristic values of loads (actual use);
- the extra loads taken into account (snow, wind, thermal loads), and
- the method of determining the strength.

When applying the derogations from the rules for new buildings, NEN 8700 assumes that one is initially guided by administrative and economic principles (proportionality).

There are fundamental differences between the assessment of existing structures and the design of new structures, which affect the requirement on the structural performance and thus may affect the used target reliability in individual cases. The differences are as follows (ISO 13822, 2010):

- economic considerations: the cost between acceptance and upgrading the existing structure can be very large, whereas the cost of increasing the safety of a structural design is generally very small, consequently conservative generic criteria are used in design but should not be used in assessment,
- social considerations: these include disruption (or even displacement) of occupants and activities, also heritage values, considerations that do not affect the structural design, but assessment,
- sustainability considerations: reduction of waste and recycling, considerations of less importance in the design of new structure, but in assessment.

3.7.2 Definitions

Design service life, is the assumed period during which a structure or part of it is to be used for its intended purpose, including the foreseen maintenance, but without any radical restoration being required (NEN-EN 1990 article 1.5.2.8).
Remaining service life, is the assumed period during which a current or converted construction or a part of it is to be used for its intended purpose (NEN 8700 article 1.5.2.8a)
Reference period, is the time period chosen and used as the basis for statistical evaluation of variable loads and if necessary for accidental loads (NEN 8700 article 1.5.3.15).
The term 'remaining service life' should be clearly distinguished from 'reference period'. The remaining service life is meant the period within which the minimum safety must not be exceeded. The reference period is the period which plays a role in the determination of magnitude of the variable loads, and this does not need to be equal to the remaining

service life. The differences are primarily related to the requirements concerning human security (NEN 8700 article 1.5.2.8a Note 1).

Characteristic value of a load (F_k), is the main representative value of a load. (Note: To the extent that a characteristic value may be defined on statistical grounds, they shall be selected in accordance with a prescribed probability to be exceeded to the unfavorable side during a 'reference period', taking into account the design service life of the structure and the time period of the design situation.)

Accidental load, is a load usually of short duration but of significant magnitude, of which the probability of occurrence during the design service life of the structure is low.

3.7.3 Partial Factors & Limit states

According to NEN-EN 1990 (Annex C4), there are diverse reliability methods available for calibration of partial factors (limit state) for the equations for the design and calculation, and their interrelationships. The partial load factors are derived using the reliability index β that belongs to the refurbished structure. The reliability index β is dependent on the reference period and the reliability class (RC). The NEN-EN 1990 (Annex B) prescribes that the three Reliability Classes RC1, RC2 and RC3 should be associated as one with the Consequence Classes CC1, CC2 and CC3. The magnitude of the partial load factors for the existing structures differ from the factors for new buildings, and this is because of the difference in the reference periods which affects directly the reliability indexes. A structure has to be calculated for two different limit states:

- **Ultimate Limit States (ULS)**, which are related to the safety of the people, and/or the safety of the structure.
- **Serviceability Limit States (SLS)**, which are related to the functioning of the structure, or structural members, under normal use, the comfort of the people, and the appearance of the structure.

The design situations applicable should be chosen taking into account the circumstances under which the structure must fulfill its function (Table 3-2).

Table 3-2 Design situations according to NEN-EN 1990 (article 3.2)

Design situations		Verifications
Persistent	Normal use	ULS, SLS
Transient	Execution, temporary conditions applicable to the structure, e.g. maintenance or repair	ULS, SLS
Accidental	Normal use	ULS
	During execution	ULS
Seismic	Normal use	ULS, SLS
	During execution	ULS, SLS

3.7.4 Ultimate Limit States

The Eurocode explicitly establishes 6 different ultimate limit states: EQU – Equilibrium, STR – Structural, GEO – Geotechnical, FAT – Fatigue, UPL – Uplift, and HYD – Hydraulic heave. A concrete upper structure has to be checked for EQU and STR.

a. EQU: Loss of static equilibrium of the structure or any part of it considered as a rigid body, where:
   - minor variations in the value or the spatial distribution of actions from a single source are significant, and
   - the strengths of construction materials or ground are generally not governing.

$$E_{d,dst} \leq E_{d,stb}$$

Where:
$E_{d,dst}$ is the calculation value of the destabilizing load effect.
$E_{d,stb}$ is the calculation value of the stabilizing load effect.

b. STR: Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs.

$$E_d \leq R_d$$

Where:
E_d is the calculation value of the load effect, such as internal forces, moments, or a vector which represents a number of internal forces or moments.
R_d is the calculation value of the corresponding resistance.

The formulas for the load combinations of the ultimate limit states, as presented in NEN-EN 1990 (article 6.4), are summarized in Table 3-3.

Table 3-3 Overview table with the formulas of the load combinations for the ULS (EQU, SRT)

Combinations of actions		
Combination	**Reference NEN-EN 1990**	**General expression**
Fundamental (for persistent and transient design situations)	6.10 (EQU)	$\sum_{j\geq1} \gamma_{G,j} G_{k,j}\text{"} + \text{"}\gamma_P P\text{"} + \text{"}\gamma_{Q,1} Q_{k,1}\text{"} + \text{"}\sum_{i>1} \gamma_{Q,1} \psi_{0,i} Q_{k,i}$
	6.10 a/b (SRT)	$\begin{cases} \sum_{j\geq1} \gamma_{G,j} G_{k,j}\text{"} + \text{"}\gamma_P P\text{"} + \text{"}\gamma_{Q,1} \psi_{0,1} Q_{k,1}\text{"} + \text{"}\sum_{i>1} \gamma_{Q,1} \psi_{0,i} Q_{k,i} \\ \sum_{j\geq1} \xi_j \gamma_{G,j} G_{k,j}\text{"} + \text{"}\gamma_P P\text{"} + \text{"}\gamma_{Q,1} Q_{k,1}\text{"} + \text{"}\sum_{i>1} \gamma_{Q,1} \psi_{0,i} Q_{k,i} \end{cases}$

		$0,85 \leq \xi_j \leq 1,00$ for unfavorable permanent actions G
Accidental (for accidental design situations)	6.11	$\sum_{j\geq 1} G_{k,j}" + "P" + "A_d" + "(\psi_{1,1} or\ \psi_{2,1})Q_{k,1}" + "\sum_{i\geq 1} \psi_{2,i}Q_{k,i}$
Seismic (for seismic design situations)	6.12	$\sum_{j\geq 1} G_{k,j}" + "P" + "A_{Ed}" + \sum_{i\geq 1} \psi_{2,i}Q_{k,i}$
$\gamma_{G,j}$ Partial load factor for dead load $G_{k,j}$ Characteristic value of dead load $\gamma_{Q,1}$ Partial load factor for dominant imposed load $\Psi_{0,1}$ Factor for combination value of dominant imposed load $G_{k,1}$ Characteristic value of dominant imposed load $\gamma_{Q,i}$ Partial load factor for dead load $\Psi_{0,i}$ Factor for combination value of imposed load i $Q_{k,i}$ Characteristic value of imposed load i ξ Reduction factor for unfavorable dead load G		

NEN-EN 1990 and NEN 8700 address different values for the partial load factors, in case of new building and refurbishment respectively, only for the ultimate limit state of 'Internal failure or excessive deformation - STR' (Table 3-4). The values for the partial load factors in case of the ultimate limit state of 'Loss of static equilibrium - EQU' coincide, and are presented in Table 3-5.

Table 3-4 Comparison of the partial load factors (γ) for the ultimate limit state STR as addressed by NEN 8700 and NEN-EN 1990.

Partial load factors – Refurbishment / New building				
Load combinations	Persistent loads		Dominant variable load other than wind	Variable wind load normative
	Unfavorable	Favorable		
(According 6.10a)	$\gamma_{GJ,sup}$	$\gamma_{GJ,inf}$	$\gamma_{Q,1}$	$\gamma_{Q,1}$
CC1a/b	1,15 / 1,20	0,90 / 0,90	1,10 / 1,35	1,20 / 1,35
CC2	1,30 (1,20) / 1,35	0,90 / 0,90	1,30 / 1,50	1,40 / 1,50
CC3	1,40 (1,20) / 1,50	0,90 / 0,90	1,50 / 1,65	1,60 (1,50) / 1,50
(According 6.10b)	$\xi\gamma_{Gj,SUP}$	$\gamma_{Gj,INF}$	$\gamma_{Q,1}$	$\gamma_{Q,1}$
CC1a/b	1,05 / 1,10	0,90 / 0,90	1,05 / 1,35	1,10 / 1,35
CC2	1,15 / 1,20	0,90 / 0,90	1,15 / 1,50	1,30 / 1,50
CC3	1,25 (1,20) / 1,30	0,90 / 0,90	1,50 / 1,65	1,60 (1,50) / 1,65
The values in parentheses may be applied only building for which an environmental permit for the construction authorized under Building Decree 2003 or earlier.				

Table 3-5 Partial load factors for ultimate limit state EQU (NEN-EN 1990)

Partial load factors – Refurbishment & New building					
Permanent and transient design situations	Permanent loads		Dominant variable load	Variable loads simultaneously with the predominant	
	Unfavorable	Favorable		Main (if present)	Other
According 6.10 (Table 3-3)	1,1 $G_{k,j,sup}$	0,9 $G_{k,j,inf}$	1,5 $Q_{k,1}$		**1,5 $\Psi_{0,I}\,Q_{k,I}$ (I>1)**

3.7.5 Serviceability Limit States

The verification of the serviceability limit states should be based on criteria that are related to the deformations, the vibrations and damage which is likely to work disadvantageously on the appearance, the durability or the functioning of the building. NEN 8700 and NEN-EN 1990 define the same values for the partial load factors and the material partial factors, $\gamma_Q=\gamma_G=1{,}0$ and $\gamma_M=1{,}0$ respectively.

Combinations of actions		
Combination	**Reference NEN-EN 1990**	**General expression**
Characteristic	6.14b	$\sum_{j\geq 1} G_{k,j}\text{" + "}P\text{" + "}Q_{k,1}\text{" + "}\sum_{i>1}\psi_{0,i}Q_{k,i}$
Frequent	6.15b	$\sum_{j\geq 1} G_{k,j}\text{" + "}P\text{" + "}\psi_{1,1}Q_{k,1}\text{" + "}\sum_{i>1}\psi_{2,i}Q_{k,i}$
Quasi-permanent	6.16b	$\sum_{j\geq 1} G_{k,j}\text{" + "}P\text{" + "}\sum_{i\geq 1}\psi_{2,i}Q_{k,i}$
$G_{k,j}$ Characteristic value of dead load $\Psi_{0,1}$ Factor for combination value of dominant imposed load $G_{k,1}$ Characteristic value of dominant imposed load $\Psi_{0,i}$ Factor for combination value of imposed load i $Q_{k,i}$ Characteristic value of imposed load i		

3.7.6 Remaining Service Life

In broad terms the approach according to NEN 8700 regarding the remaining service life is as follows:

- In case of refurbishment (in Dutch: verbouw), a building is subject to new building requirements, unless 15 years of the design service life have elapsed.
- If the building exists already for 15 years, the remaining service life is equal to the original design service life minus the elapsed time, with a minimum of 15 years.

The requirements with regard to the remaining service life of a structure are fully prescribed in NEN 8700 (article 2.3.1).

3.7.7 Reference period & Characteristic values of variable loads

In outline, some of the requirements prescribed in NEN 8700 (article 2.3.2) with regard to the reference period of a refurbished structure, are:

- In case of refurbishment, the reference period should be at least equal to the remaining service life according to NEN 8700 (article 2.3.1).
- The reference period used to determine the variable loads in order to assess whether or not the performance of a building distinguishes from the disapproval, differentiates for consequence class CC1a and for the consequence classes CC1b, CC2 and CC3.
- The characteristic values of the variable loads for buildings in NEN-EN 1991 are generally based on a reference period of 50 (or 100) years. If different reference periods are used, from the reference period of 50 years, the extreme values of uniformly distributed loads may be adjusted. Therefore, in a number of cases, rules are included in the applicable standard in the series of standards NEN-EN 1991, such as:
  - snow loads (NEN-EN 1991-1-1-3/Annex D)
  - wind loads (NEN-EN 1991-1-1-4/article 4/Note 4)
  - thermal loads (NEN-EN 1991-1-1-5/Annex A.2)
- Since, in case of refurbishment, the remaining service life has a different value than the design service life, the considered reference life can also deviate from that of a new building. The methodology that is applied for new buildings for a period that deviates from the 'standard' 50 years, can fully be applied for the assessment of an existing structure. In case NEN-EN 1991 indicates no rules, such as for floor loads, a different formula can be used (NEN 8700 article 2.3.2).

3.8 Determination of new loads

Undoubtedly, determining the new loads during a vertical extension is one of the most critical stages. Some of the following changes should be taken into consideration:

- increased wind loading, as a consequence of the increased height;
- change of function of the building, resulting different imposed loads;
- adjusted accidental and characteristic variable loads, arising from a different reference period.

3.8.1 Wind Load

The most common lateral load is the wind load. Wind load against a building builds up a moment at the foundation. This moment leads to compression and tension forces in the piles. Using basic mechanics, and a simple mechanical scheme, one can realize that an increase in the height of the building, results to an increase of the wind loads, and therefore, an increased moment at the foundation of the building (Figure 3-4). It can be seen that the increase of the moment is proportionally much larger than the increase of the height of the building and this is justified by the fact that in the formula for calculating the moment ($M=1/2*q_w*l^2$) the height of the building (L) is in the power of 2. This increase should be taken up either from the potential extra load bearing capacity of the foundation or from new foundation constructed.

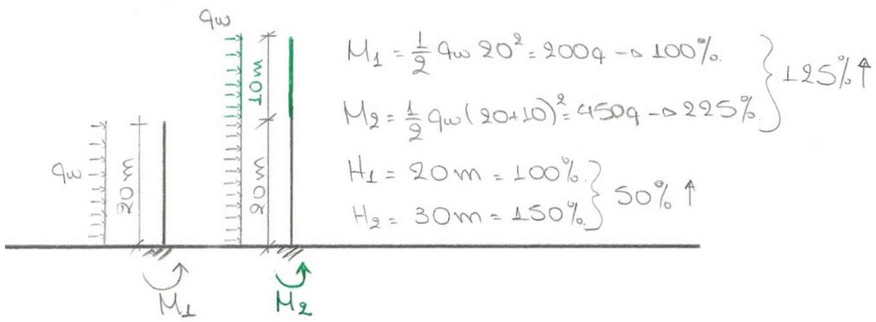

Figure 3-4 Schematic representation of the increased moment at the foundation of a building due to an increase in the building height

3.8.2 Change of Function & Imposed Loads

An eventual change of function, during a vertical extension, could lead to higher or lower loads. A favorable change of function could be converting offices or, even better, an old warehouse into residences. The practice has shown, as it is explained in the examined case studies in Section 2.3, that such a transformation is favorable for vertical extension projects by 'creating' extra load bearing capacity for the existing structure. On the other hand, an adverse alteration, than the ones just described, could have unfavorable effects. Increasing the imposed loads on the existing structure already before the vertical extension should be avoided. In that case, it is likely to end up with a much higher increase of applied loads on the foundation, loads that might exceed the load bearing capacity of the foundation piles, structural elements which cannot be reinforced or modified.

3.8.3 Accidental Loads

NEN 8701 (article 4.4.1) prescribes the following general requirements for the accidental loads in case of refurbishment.

- In case that in the original design of the structure, no or limited accidental loads have been taken into account, the structure can often only meet the rules for the new construction with costs that are proportionate to the decrease of the risk. In those cases, it is allowed, when assessing whether the structure should be rejected, to only take into account the initial accidental loads. In case of refurbishment, in the aforementioned cases, it is allowed to leave apart other accidental loads than the ones originally considered, if this can be justified on the basis of the principle of proportionality.
- For the assessment of the resistance against the accidental loads, the actual state of the structure or the taken measures should be weighed against the relative costs (proportionality principle) of the higher degree of safety. Simple measures which significantly decrease the risk should always be taken. If the costs do not differ significantly for those of a new building, the structure will have to withstand the accidental loads. Also, in case that the impact of the accidental loads is particularly large and socially unacceptable, the accidental loads should be taken into account.
- In the case of the accidental loads, 'fire' should always be considered regarding buildings.

The subject of the characteristic variable loads in building renovation is addressed in Section 2.2.4.

3.9 METHODS OF STRUCTURAL ANALYSIS

Structural performance shall be analyzed using models that reliably represent the loading on the structure, the behavior of the structure and the resistance of its components. The analytical model should reflect the actual condition of the existing structure (ISO 13822, 2001). SAMCO association, describes clearly in the Guideline for the Assessment of Existing Structures, the three methods of structural analysis and these are presented in the following.

3.9.1 3.4.1 Simple analysis methods

For lower assessment levels it is often effective to calculate load effects with basic conservative methods with simple structural models, provided that the approximately large uncertainty is regarded with an adequate safety measure. Typical simple analysis methods are among others space frame and grillage analysis combined with a simple load distribution and linear elastic material behavior, which result in a lower bound equilibrium solution.

3.9.2 Complex analysis methods

In case low level assessment failed, refined load effect calculation methods need to be accomplished. Refined methods include mainly finite element analysis and non-linear methods such as yield line analysis, where these may lead to higher capacities. Particularly a specified modeling of the material behavior such as time-variant behavior (e.g. shrinkage and creeping of RC structures) and the consideration of interactions between material components (e.g. bond, tension stiffening in RC) will uncover hidden capacity reserves and reduce conservatism. Applying full probability safety verification, stochastic finite elements can be used to model the structure. The difference to conventional finite element models is that the stochastic elements take the spatial correlation of the random variables into account.

3.9.3 Adaptive models

To avail new information about the structural behavior within assessment, for example from long term monitoring, models need to be updated allowing for the new information. Adaptive models are able to update automatically structural variables (e.g. stiffness parameters) using measurement data such as a change in displacements, strains or damage values (e.g. crack width).

3.10 STRUCTURAL INTERVENTIONS

The structural assessment of an existing structure under the new loads of a vertical extension, might reveal the need for strengthening of the existing load bearing system. There is a variety of strengthening methods for the different structural elements (columns, beams, walls, floors, foundation) and several of these methods will be presented in the following. The choice of the most appropriate method varies from project to project and is dependent on the critical insight of the structural engineer.

3.10.1 Reinforced concrete columns

Jacketing methods with reinforced concrete (RC), steel plates or carbon fiber (CF) sheets has been widely used to repair or strengthen the RC columns (Fukuyama, Higashibata, & Miyauchi, 2000).

Fiber Reinforced Polymers

Strengthening RC columns using fiber reinforced polymers (FRP) is a technique being frequently used to seek the increment of load carrying capacity and/or ductility of such compression members (Rocca, Galati, & Nanni, 2008). Existing studies have shown that the use of FRP materials restore or improve the column original design strength for possible axial, shear, or flexure and in some cases allow the structure to carry more load than it was designed for (Parvin & Brighton , 2014). FRP sheets or encasement can be used to increase the axial load carrying capacity of the column with minimal increase in the cross-sectional area (Figure 3-5). Confinement consists of wrapping the column with FRP sheets, prefabricated jacketing, or in situ cured sheets with fiber running in circumferential direction. Confinement is less effective for rectangular and square than circular shape RC columns due to the confining stresses that are transmitted to the concrete at the four corners of the cross-section (Figure 3-6).

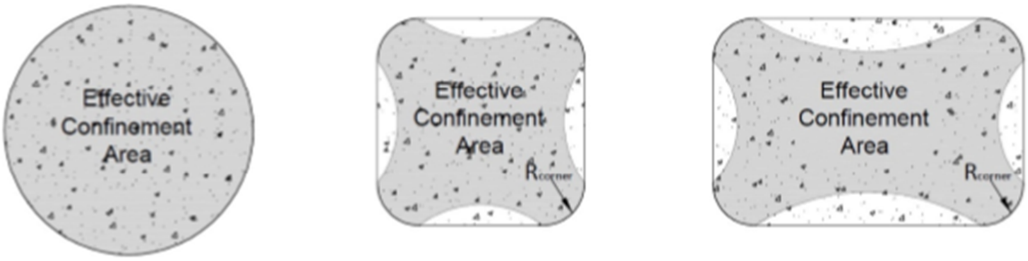

Figure 3-5 Effective confinement areas in circular, square and rectangular columns (Parvin & Brighton , 2014).

Figure 3-6 Applying FRP sheets on circular reinforced concrete columns.

Circularization and FRP confinement

Strengthening square RC columns by circularization and FRP (fibre reinforced polymer) confinement is a technique suggested by Pham , Doan & Hadi (2013). Circularization is a technique where segmental circular concrete covers made of different concrete strengths (40 MPa, 80 MPa and 100 MPa) are used to change a square column to a circular column (Figure 3-8). The different loading conditions of the experiments are shown in Figure 3-7; concentrically and eccentrically loaded columns and beams loaded in shear. The experimental results of their research demonstrated that using high strength concrete (HSC) for the additional covers to strengthen existing square reinforced concrete (RC) columns provides higher load-carrying capacity than covers made of normal strength concrete. The HSC covers and the concrete cores worked as a composite material to failure.

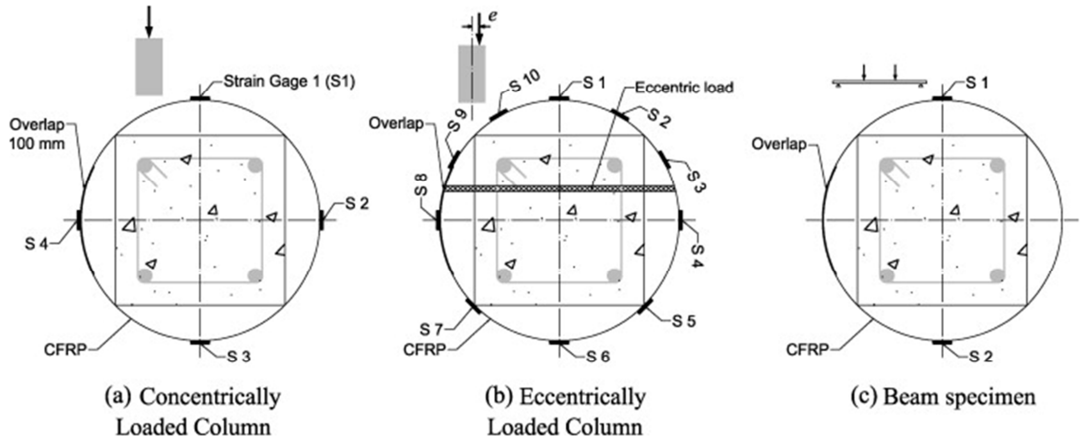

Figure 3-7 Details of loading conditions and gauge locations (Pham , Doan , & Hadi , 2013)

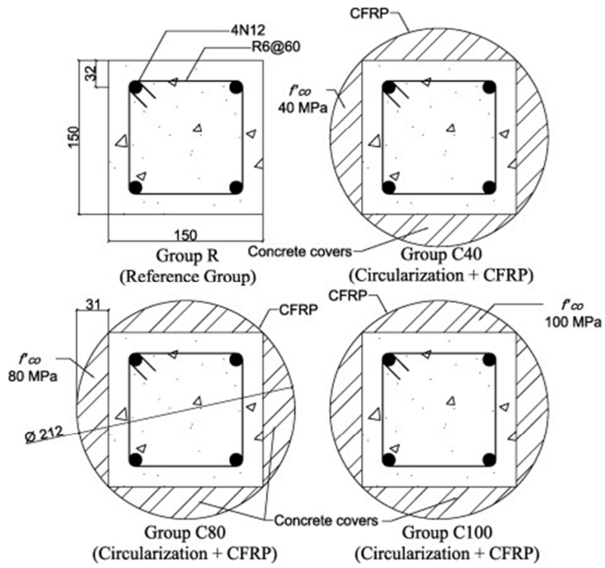

Figure 3-8 Cross sections of specimens during the experiments of Pham , Doan & Hadi (2013)

Steel jacket technique

Strengthening of RC columns using steel jacket technique is commonly used in order to increase their capacity to sustain the applied loads. This method includes actually strengthening of columns with the use of steel angles connected by horizontal strips (Figure 3-9). Experimental research has proven this method to be effective since it increases the column capacity to a minimum of 20% (Belal, Mohamed, & Morad, 2014). In the same vein, other scientific research has demonstrated that the main parameter was the type of the external strengthening (Issa, Elzeiny, Aly, & Metwally, 2008). For the steel jacket the variables were the size of corner angles and the spacing between the steel plates and from the experimental study, it was concluded that increasing the area of corner steel angles and decreasing the spacing between the steel pattern plates of steel jackets increase the ultimate carrying capacity, and ductility of strengthened columns ((Issa, Elzeiny, Aly, & Metwally, 2008).

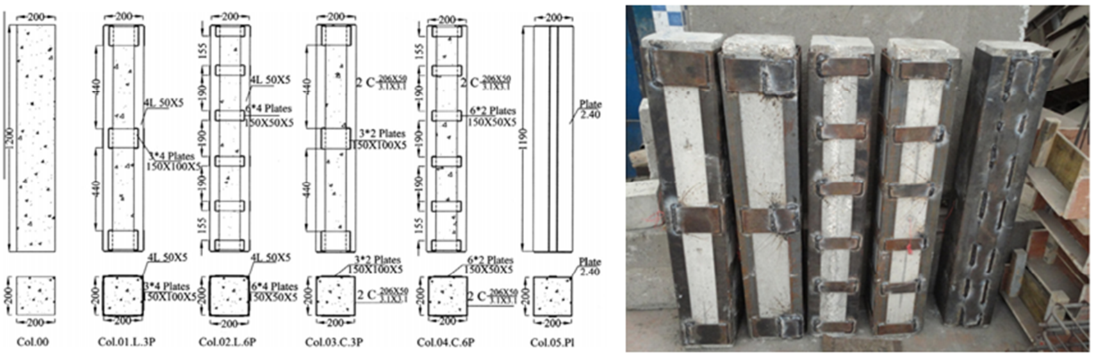

Figure 3-9 Different steel jacket (steel cages) configurations (Belal, Mohamed, & Morad, 2014)

Concrete jacket with additional reinforcement

Adding new concrete jacket with additional reinforcement, by encasing the existing column is an alternative way to increase its load bearing capacity. The additional concrete should preferably enclose the column, and should be provided with closed stirrups (FIP Commission on practical conctruction, 1991). Enlarging the cross section of a column, that column becomes stiffer increasing the moment of inertia and reducing the internal stresses, even when larger forces are applied on it due to increased imposed loads (Relker, 2013).

Attention should be given on the interface between the old and the new concrete. Even if a fully satisfactory interface can be obtained, the strengthened column will not behave as a homogeneous concrete column with the same cross section. The additional concrete will shrink more than the old concrete, so there is the risk of cracking in the new concrete. Unloading the column during the strengthening work may, to some extent, compensate for this.

Figure 3-10 Reinforced concrete column concrete jacketing with additional reinforcement

3.10.2 Reinforced concrete beams

For the reinforced concrete beams the applied strengthening methods are in their principle similar to the ones applied for the reinforced concrete columns, i.e. jacketing with concrete and use of fiber reinforced polymers (FRP). The book "Repair and strengthening of concrete structures" (1991) gives an extensive description of the possibilities to increase the capacity of RC beams, in terms of simple mechanics. The moment capacity of a beam can be limited by compressive failure of the concrete (over-reinforced beam) or by yielding of the reinforcement (under-reinforced beam). In both cases, the moment capacity can be improved by increasing the effective depth, i.e. by adding concrete in the compression zone. The most effective way to strengthen an under-reinforced beam, however, is to add new tensile reinforcement, especially if this also means increasing the effective depth. The extra reinforcement in the tensile zone, should be surrounded with concrete, which provides the reinforcement with protection against corrosion and fire, and which transfers shear forces to the old beam. In the following the different techniques used nowadays, so as to achieve the aforementioned goals, will be discussed.

Concrete jacketing

Concrete jacketing of RC beams, has been considered as one of the important methods for strengthening and repairing of such elements. Jacketing of RC beams is done by enlarging the existing cross section with a new layer of concrete that is reinforced with both longitudinal and transverse reinforcement. The experimental results clearly demonstrated that jacketing can enhance structural properties for the RC beams (Raval & Dave, 2013). From Raval's and Dave's reasearch (2013), it can be observed that jacketing of RC beams may always be employed as one of the very promising technique for enhancing the performance of the beams in case of change in use of the structure. Moreover, the use of dowel connectors & bonding agent with micro-concrete as well use of micro-concrete alone have emerged as better techniques of jacketing RC beams as compared to other jacketing alternatives employed in the present investigation (Figure 3-12).

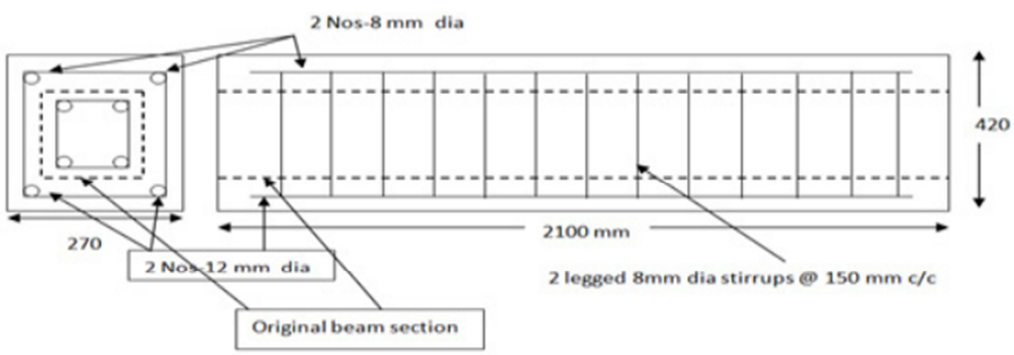

Figure 3-11 Reinforcement detail of jacketed RC beam

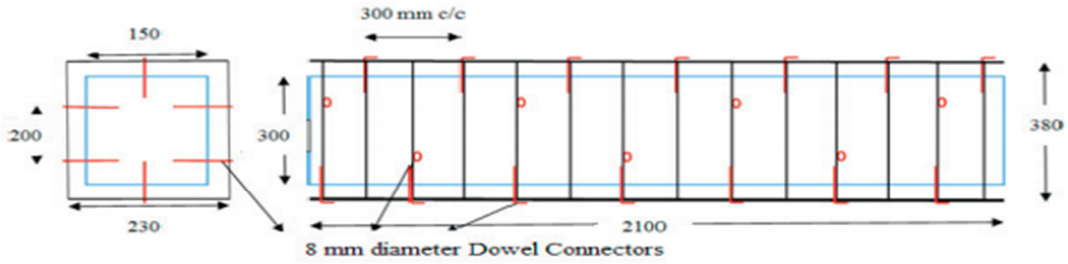

Figure 3-12 Location of dowel connectors on jacketed RC beam

Carbon Fibre Reinforced Polymer (CFRP)

Shear strengthening of reinforced concrete beams with carbon fibre reinforced polymer (CFRP), is a technique that according to experimental research is able to increase the shear capacity of RC beams. Tests showed that it is beneficial to orientate the fibres in the CFRP sheets at 45˚ (see Figure 3-13, last configuration) so that they are approximately perpendicular to the shear cracks (Bukhari, Vollum, Ahmad, & Sagaseta, 2010).

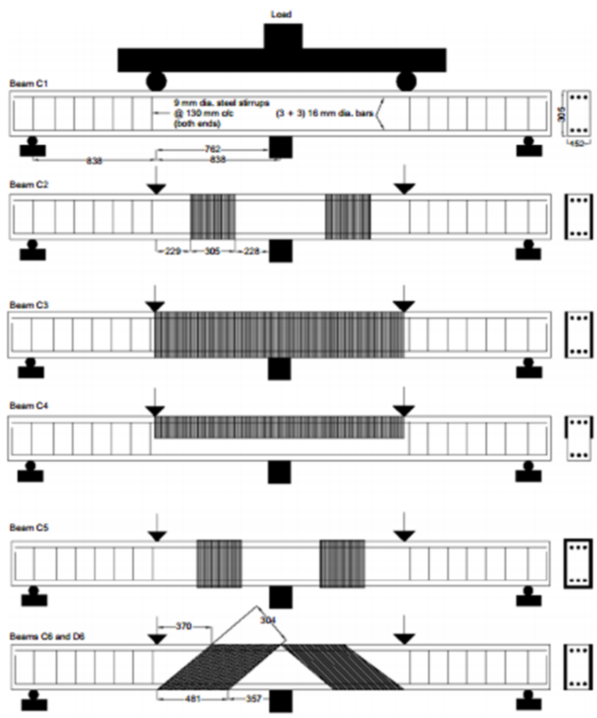

Figure 3-13 Beam configuration details from shear strengthening with CFRP (Bukhari, Vollum, Ahmad, & Sagaseta, 2010)

3.10.3 Slabs

The load bearing capacity of the slabs can be limited by the moment capacity or by the shear capacity. Normally, the bending moment capacity is decisive but in heavily loaded thick slabs supported on columns or in slabs with highly concentrated loads, the punching shear capacity may also be critical. In a serviceability limit state deformations are often decisive for the load to be carried.

Banu and Taranu (2010) reviewed the traditional solutions for strengthening reinforced concrete slabs and, as a first step, started from separating the strengthening techniques for RC slabs (1) with cut-outs and (2) without cut-outs. In case new openings are required by the new design, the structural engineer should examine the effects of these openings on the structural integrity of the slab and whether these can be accommodated without strengthening. Supposing this is not possible, extra provisions should be taken into consideration for the parts of the slab located around the openings. These provisions would include either proper detailing of additional reinforcing steel in the slab or beams, or, increase of the thickness of the slabs around the opening.

Tarek Alkhrdaji (2004) has distinguished the different methods to structurally upgrade concrete structures, and slabs in particular, such as span shortening, external composites, externally bonded steel, external or internal post-tensioning systems, section enlargement, or a combination of these techniques.

Fiber Reinforced Polymer (FRP)

Fiber reinforced polymer (FRP) systems are high-strength, lightweight reinforcement in the form of paper-thin fabric sheets, thin laminates, or bars that are bonded to concrete members with epoxy adhesive to increase their load bearing capacity. The strips can be overlapped at the corners of the opening, making strengthening in two directions simpler, and does not interfere with the floor surface as much as anchored steel plates. In addition to FRP, more innovative strengthening systems exist and emerge from the research, such as, steel reinforced polymer composites (SRP) and glass fibre reinforced polymer (GFRP), that may be used as externally bonded reinforcement. GFRP appears to be a promising technique since test results have shown that the strengthened slabs seems to increase the load-carrying capacity by 29%, 21% and 12% over that of the control specimen for diagonal, parallel and surround strengthening respectively (Choi, Park, Kang, & Cho, 2013). In Figure 3-14 the different configurations can be seen.

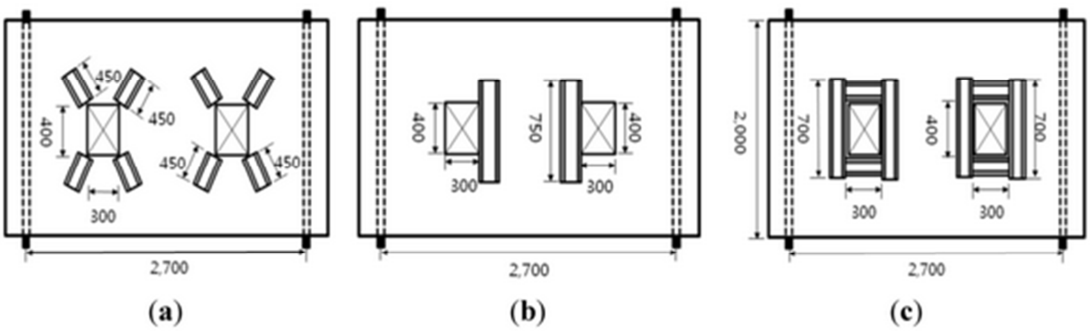

Figure 3-14 Different configurations of (a) diagonal (b) parallel and (c) surrounding strengthening with GFRP of slab opening

Bonded steel elements

Bonded steel elements, is another strengthening technique for slabs, where steel elements are glued to the concrete surface by a two-component epoxy adhesive to create a composite system and improve shear or flexural strength. In addition to epoxy adhesive, mechanical anchors typically are used to ensure that the steel element will share external loads in case of adhesive failure. Because overlapping of the plates is difficult, this method works best when strengthening is required in only one direction (Figure 3-15). The steel elements can be steel plates, channels, angles, or built-up members.

Figure 3-15 Roof slab strengthened with a combination of FRP and steel plates.

In case no new openings are foreseen for the new design, the techniques used to increase the load bearing behavior of an existing slab include:

- cement grout;
- ferrocement cover;
- section enlargement;
- external plate bonding; and,
- external post-tensioning.

These techniques are thoroughly presented by Banu & Taranu (2010) in a review of the traditional solutions for strenthening reinforced concrete slabs.

3.10.4 Walls

The load bearing capacity of a wall is, like that of a column, limited by its strength and, in case of slender walls, by its stiffness. Similarly with the aforementioned structural elements, the techniques for strengthening reinforced concrete wall are summarized in three main categories, that include the use of concrete jacketing, externally bonded steel strips and fiber reinforced polymers (FRP).

Concrete jacketing

Concrete jacketing is used either by means of high performance concrete reinforced with high strength steel mesh (Figure 3-16) increasing the structural resistance, deformation capacity and ductility (Marini & Meda, 2009), or with cement mortar reinforced by glass fiber reinforced plastics (GFRP) grids (Figure 3-17), to improve significantly the lateral load-carrying capacity (Corradi , Borri, Castori, & Sisti, 2014).

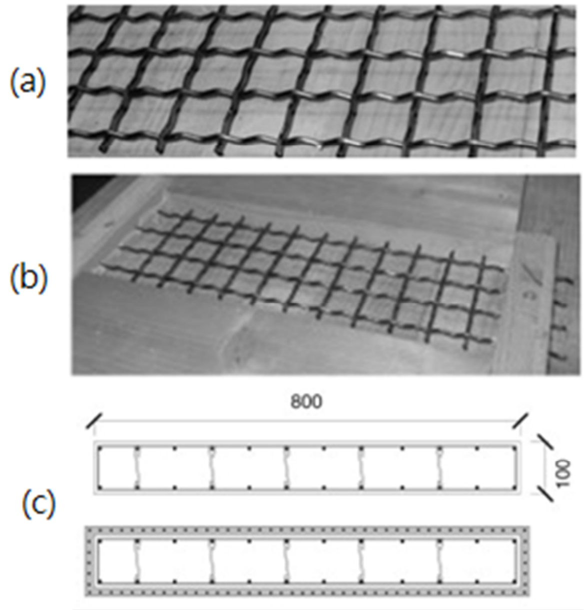

Figure 3-16 (a) High strength steel mesh made of bent wires (b) detail of the high strength steel mesh prior to the high performance fiber concrete cast (c) cross section of the experimental model (Marini & Meda, 2009)

Figure 3-17 Application of mortar jacketing and detail of connection between anchors and GFRP grid (Corradi , Borri, Castori, & Sisti, 2014)

Externally bonded steel strips

Externally bonded steel strips are applied in symmetrical configurations on both sides of a wall. A research focused on the effect of using bonding steel strips enhancing strength and increasing ductility of the non-seismic detailed shear walls revealed that all the steel strip configurations (Figure 3-18) improved the lateral strength, energy dissipation capacity and deformation capacity of the shear deficient RC wall significantly (Altin, Kopraman, & Baran, 2013).

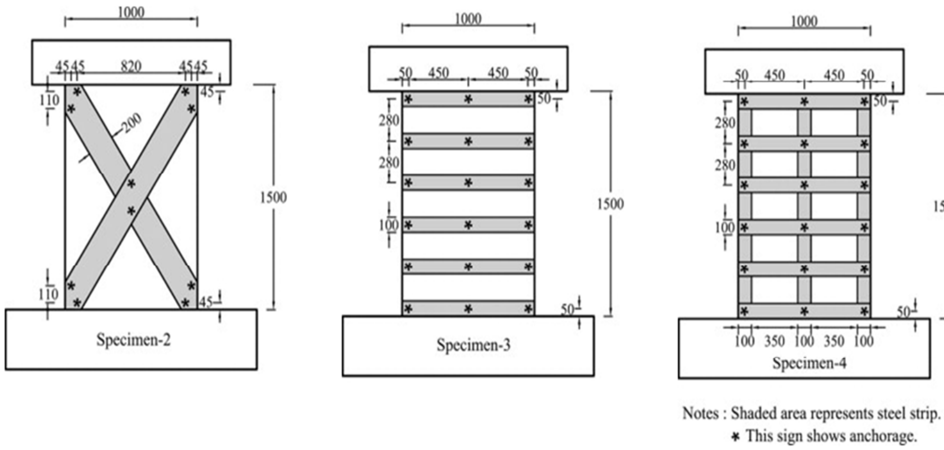

Figure 3-18 Different strengthening schemes (Altin, Kopraman, & Baran, 2013)

Fibre-reinforced polymers (FRP)

The successful application of FRP to strengthen solid concrete walls has been achieved in several studies (Antoniades, Salonikios, & Kappos, 2005; Dan, 2012). All of them performed a rehabilitation of structural walls using externally bonded FRPs to increase the flexural and/or shear strength, stiffness and energy dissipation. FRPs are able to strengthen such walls by redistributing the stresses, allowing the wall to recover almost its full capacity before the opening was created, if not more (Mohammed, Ean, & Malek, 2013; Li & Qian, 2012; Todut, Dan, & Stoian, 2015).

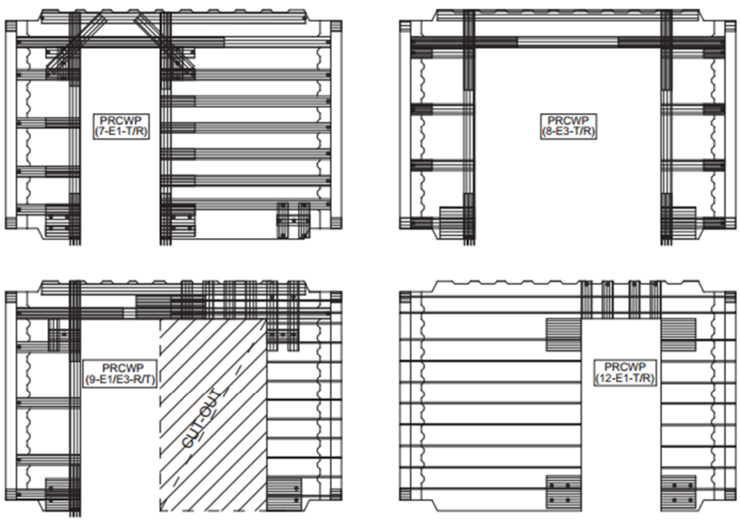

Figure 3-19 Different strategies using carbon fiber reinforced polymers (Todut, Dan, & Stoian, 2015)

Composites, such as fiber reinforced polymers (FRP), carbon fiber reinforced polymers (CFRP) or glass fiber reinforced polymers (GFRP) are also applied for strengthening of unreinforced masonry walls (URM) in order to improve the flexural capacity (Ehsani, 2005).

3.10.5 Foundation

Soil conditions affect in a great extend the foundation type and the construction technique used. In the Netherlands, the low-lying alluvial terrain has resulted in urban and industrialized areas being concentrated around waterways and ports, with considerable depths of soft and loose compressible alluvial soils, and water tables at the ground surface (Hertlein & Davis, 2007). Driven piles were traditional, starting with timber and then proceeding to steel and later to pre-cast concrete as design loads became heavier. With these increasing loads, one of the most economical and sure foundation type in the Netherlands was the pre-cast concrete driven pile in its various forms.

According to Tishkov, Ponomarenko and Ivasyuk (2013), who examined the strengthening of pile foundations during reconstruction of buildings and structures, the methods employed to strengthen the foundations can be divided into three groups:

1. by varying the characteristics and properties of the bed soils (grouting, silicification, thermal and electrochemical stabilization, freezing, etc.), which is a too expensive method and does not guarantee that the results required will be obtained;
2. by varying the design of the foundation (expansion by the installation of monolithic yokes, reduction, built-up, placement of additional piles beneath the foundation, etc.); and,
3. by redistributing the active forces (installation of monolithic girdles, unloading frames, transfer of forces onto neighboring components, etc.).

The method of strengthening by some group or other is selected, depending on the type of foundation, characteristics of the soil bed, and active forces.

3.10.6 Stability

The stability of the structure is provided by the transfer of the lateral loads, caused by wind, design, local or global imperfections, to the foundation. The horizontal (floor diaphragms) and vertical structural elements ensure a smooth transfer of the lateral loads.

Horizontal elements

The floors and roof of a building in addition to resisting gravity loads, are also generally designed to act as diaphragms. A floor diaphragms is an horizontal system (roof, floor or other membrane or horizontal bracing) acting to transmit lateral forces to vertical resisting elements. In this respect, they are required to distribute lateral forces to the main elements of horizontal resistance, such as frames and shear walls, and also to tie the structure together so that it acts as a single entity. The robustness and redundancy of a structure is highly dependent on the performance of the diaphragms. In order to improve this performance the following strengthening techniques are generally applied:

- Improvement of the joint detailing when and where this does not fulfill the strength requirements arising from the new loads. The longitudinal joints between the floor slabs should be able to transfer the concentrated shear stresses and the peripheral tie reinforcement is designed to take up the tensile forcing arising from in-plane bending, ensure the shear capacity in joints and create continuity (Hordijk & Lagendijk, 2014).

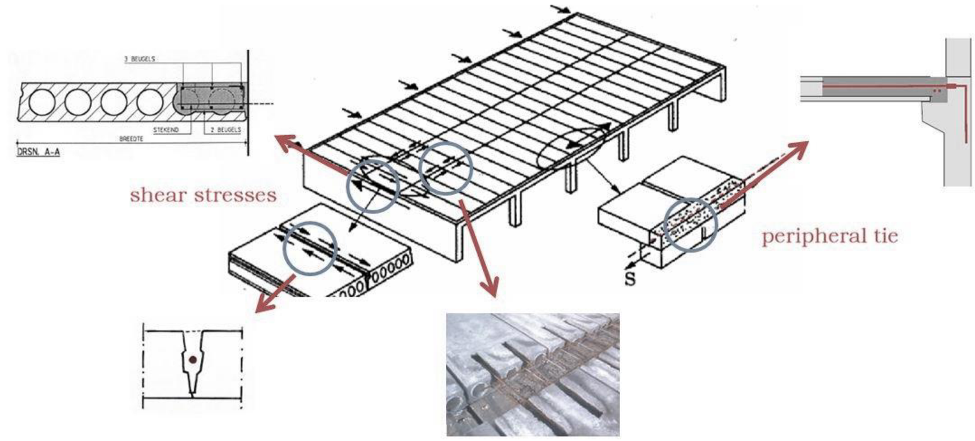

Figure 3-20 Longitudinal and peripheral joints in hollow core floor slabs

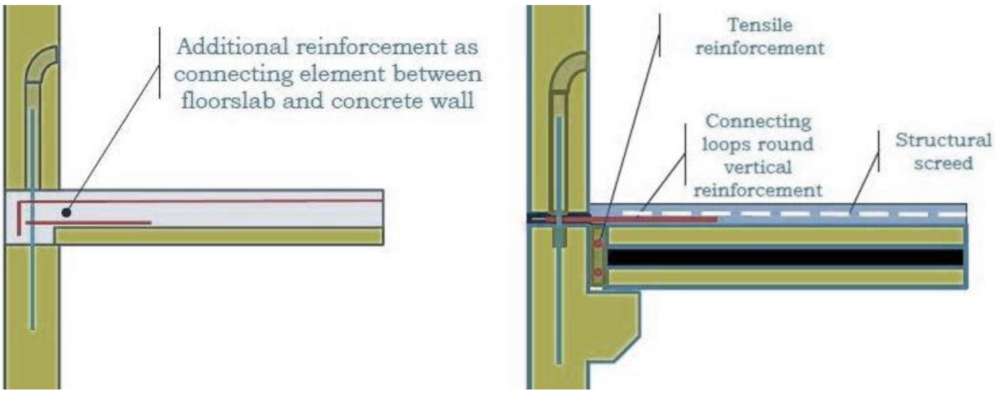

Figure 3-21 (Left) Detail connection plank floor to wall (Right) Detail connection hollow core slab with wall (Hordijk & Lagendijk, 2014)

- Use of structural topping (reinforced concrete layer) on hollow core slabs (Figure 3-21). This method increases the load bearing capacity, the stiffness of the floor, the diaphragm action, and the fire resistance. Moreover, other significant properties of the floor are improved, such as the water tightness and the sound insulation.

Vertical elements

There are several methods to approach the stability of a vertical extension project. Structural design, and stability design in particular, rely on the creativity and the insight of the structural engineer in the project. The structural engineer can combine some of the basic stability systems, ending up with an innovative structural design. The main principles and basic rules of the stability systems will be presented in the following.

- Increase of stiffness. Using a basic schematic representation of a shear core one can see that the displacement at the top of a structure are relative to the moment of inertia (Figure 3-22).

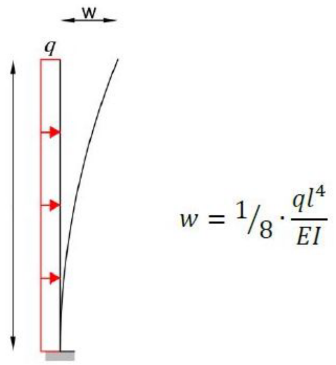

Figure 3-22 Schematic representation of a shear core under lateral loads

- Converting the stability system. Sometimes the stability of the existing building is provided by connections fixed to the foundation or moment resisting frames (Figure 3-23), techniques that are normally applied for building up to 12 m and 20 m respectively. In these cases, the increase of height due to a vertical extension requires a different stability system such as shear cores and/or shear walls, or load bearing façade elements (Figure 3-24).

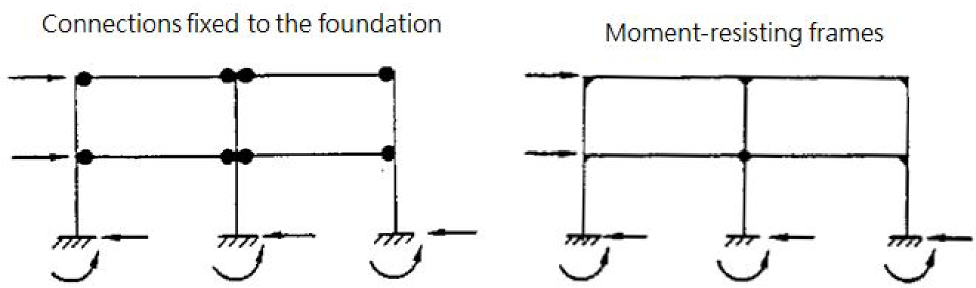

Figure 3-23 Stability systems for low rise buildings (left)connections fixed to the foundation (right) moment resisting frames

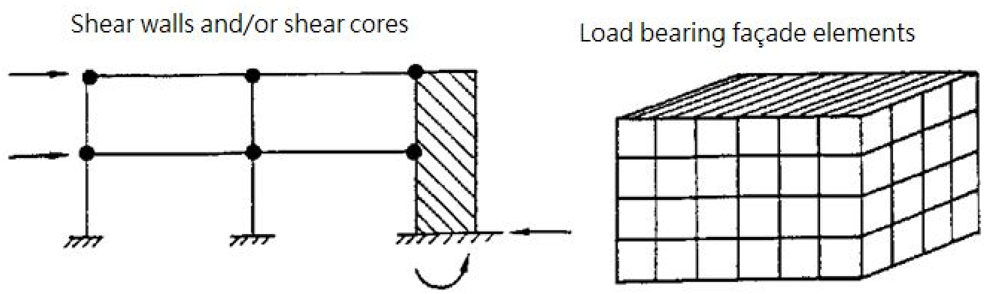

Figure 3-24 Stability systems (left) shear walls and/or shear cores (right) load bearing facade elements

- Introduce bracings or other stabilizing elements to take the extra loads and work together with the existing stability system. Supposing the stability of the existing building is provided by shear core, the solution could be to combine the shear core with a table structure, that results to reduction of the moment in the core and the deflection of the core by the reaction force from the table structure, (Figure 3-25 left) or with an outrigger structure, that has the same results due to the reaction moment of the outrigger (Figure 3-25 right).

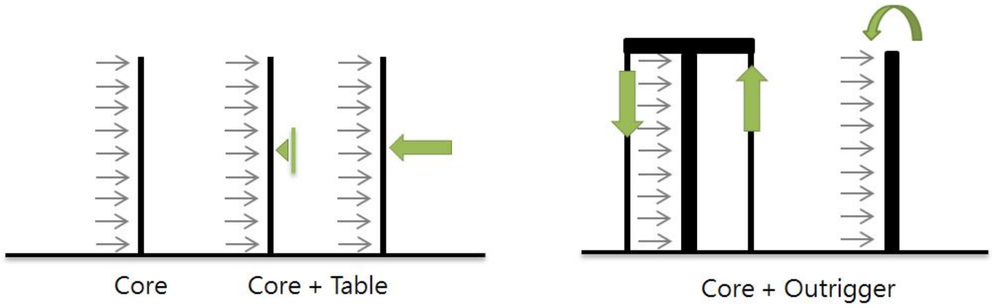

Figure 3-25 (Left) Combination of shear core with table structure (Right) Combination shear core with outrigger structure

3.11 VERTICAL EXTENSION WITH STEEL

According to Herbin (2010) there is a number of advantages to consider steel as the best material choice to refurbish an existing building. These are summarized in the following aspects.

- *Architectural diversity and freedom of design*
  The use of steel offers potential for adaptation and transformation, thanks to long spans, large and adaptable floors, internal and free volumes. These qualities also allow in the future answering the evolution of needs and uses.
- *Ease and speed of construction*
  The two compelling arguments in favor of steel are ease and speed of construction. Made to the factory, the steel structure is easy to store in the site and very rapid to be assembled. Steel frames are also suited where access is difficult.
- *Lightness*
  The lightweight technical solutions of steel enable the limitation of loads (on existing structure), the reduction of number and sections of columns, the impact on basement because any weight reduction is important.
- *Precision and reliability of steel products*
  Usually manufactured on numerical control (NC) machines, steel frames are precise and, therefore, cause few complications on site. The ratio strength/size is

excellent. Dimensional and mechanical characteristics of steel products are guaranteed and a wide range of forms and colors is proposed.

- *Thermal comfort*

  Steel refurbishment offers the possibility of adding thermal insulation externally and upgrading the performance in energy savings.
- *Compliance with Fire regulations*

  Steel is non-flammable and does not contribute any additional fire load. Today, European legislation is more realistic and fire engineering raise the obstacles to the use of the steel.
- *Longevity of steel structures*

  Easy to maintain, steel structures are undemanding in terms of maintenance. Protection of elements against corrosion (or fire) involved well-known and reliable methods. At last, steel is better able to withstand earthquakes.
- *Environmentally friendly*

  On construction site, the disturbance to the neighborhood is reduced and time is short; i.e. less waste and less noise. Steel sections are also produced entirely from recycled materials. As steels are infinitely recyclable, buildings at the end-of-life can be easily and cleanly deconstructed or dismantled.

3.12 CONCLUSIONS

In Chapter 3 the different aspects that relate to the structural design of a vertical extension have been discussed. The steps of the assessment of an existing building are summarized as a guide to the structural engineer, highlighting attention points and available options. To summarize, the assessment of an existing structure consists of the following stages:

- ✓ As a first step the objectives of the assessment of the existing building in terms of its required future structural performance should be specified.
- ✓ It is recommended to start the assessment with simple but conservative low level methods and in case the assessment fails, move on with more refined upper ones.
- ✓ A strategy is drawn, according to the changes in structural conditions and actions with regard to the methods of data acquisition. Existing archived documents are studied and analyzed. Only if conclusions are difficult to be reached. Material testing and further investigation is performed. A variety of non-destructive methods are presented in section 3.5.
- ✓ The reliability verification is carried out following a semi-probabilistic approach that is based on the limit states principles, namely the ultimate limit state (ULS) and the serviceability limit state (SLS). Partial safety factors are established. The

Eurocode is applied in vertical extension projects for the assessment as well as the structural design; both the NEN-EN 1900 (new buildings) and NEN 8700 (renovation). The latter acquires lower safety factors.

- ✓ Determining the new loads for the vertical extension is done in accordance to the future use and new height. The future function can have a favorable, or not favorable, influence on the vertical actions. At the same time the increase of the wind load figures one of the most significant factors.
- ✓ The structural analysis starts using simple structural models combined with simple load distribution and linear elastic material behavior. In case this low method fails, refined load effect calculation methods need to be accomplished such as FE analysis and non-linear methods. The possibility for more advanced adaptive modeling is introduced is case of long term monitoring.
- ✓ The possibilities for structural interventions on an existing structure are broad and various solutions are presented in section 3.10. However, the cost and risk associated with each of the interventions should be estimated. The aim of "minimum intervention", which makes as much use as possible of the existing materials in the structure applies for most existing structure of normal occupancy and use.
- ✓ Considering the stability of the structure as one of the most critical features, considering the sharp increase of the wind loads, the conversion of the stabilizing system, or the introduction of extra stability elements are suggested as first step solutions, so as to avoid strengthening.
- ✓ Last but not least, the advantages of the application of steel for the new part of the building, are presented.

These steps will be the roadmap for the design case study of this thesis. Emphasis should be given to the time devoted to the engineering part in order to investigate the actual capacity of the existing structure, hidden or not. In depth investigation of the available information and the use of testing methods, may reveal new possibilities in favor of the involved stakeholders and the project itself.

4 EXISTING CASE STUDIES

In this chapter, five already completed case studies are presented and analyzed. Emphasis is given on the steps considered in Chapter 3 in order to verify the existing literature and the way it is applied in practice. Available documents and interviews conducted by the author are used as sources of information. The goal of this chapter is to conclude to the design parameters that affected the amount of extra storeys added during the vertical extension for every project separately. At the end, it will be investigated whether these parameters coincide or converge, and could, potentially, form a list that could be used in future vertical extension projects. Certainly, the number of five case studies is not sufficient to draw conclusions based on statistics and make generalizations. Yet, the projects have been chosen randomly, and an indication could be given about possible trends.

So as to facilitate the reading of this chapter and its coherency, all case studies are assessed on the following points:

- General information of the project
- Existing structure (*Available data, Existing load bearing system, Existing foundation, Tests for strength and existing reinforcement*)
- Vertical extension (*Load bearing capacity of the existing building, Structural design of the new block*)
- Sustainability and costs in the design process
- Design parameters

4.1 DE KAREL DOORMAN

Introduction

The information presented in the following originates partly from the collaboration of ir. Michiel Visscher, one of the main structural engineers contributed to the final phase of the structural design of the Karel Doorman building and specialist in sustainable building design with focus on material use, who was kind enough to be interviewed and explained some of the sustainability aspects of the design. Moreover, main source of the information related to the structural design is the conference article 'Ultra-Light Weight Solutions for Sustainable Urban Densification' (Hermens, Visscher, & Kraus, 2014).

Figure 4-1 Karel Doorman building in Rotterdam (Photo: © Ossip van Duivenbode)

General information

Table 4-1 'De Karel Doorman' project - General information

Location	Karel Doormanstraat, Rotterdam
Project	Karel Doorman
Client	DW Nieuwbouw
Former function	Shopping center
New function	Shopping center ∣Residential building
Architectural design	Ibelings van Tilburg Architecten
Structural design	Royal Haskoning DHV
Contractor	van Wijnen Dordrecht
Originally built	1948-1951
Redevelopment	Late 1970s & 1990s
Vertical extension started - completed	~2003 - 2013

During World War II the city center of Rotterdam was almost completely destroyed. In the years after the city center was rebuilt. The building called Ter Meulen (Figure 4-2) was designed by Dutch architects *Van den Broek & Bakema* in the famous Dutch modernistic style. It was realized between 1948 and 1951. Originally shops were placed in the basement, the ground floor and the 1st floor. The 2nd floor was housing offices and the canteen. This floor was intended to be used as a salesroom too in the future. In that case the offices and canteen would be replaced to a new-to-be-built 3rd floor. In the structural design of the pile foundation and the superstructure this expansion was already taken into account. The design comprised an open floor plan made possible by a structural system of columns and beams providing lateral stability, so no structural walls were necessary.

Figure 4-2 Karel Doorman - The original building 'Ter Meulen' (Hermens, Visscher, & Kraus, 2014)

In the late '70's two (instead of one) extra floors were placed on top of the original building. This was possible by using relatively light weight floors. However during the '90's the retail marked changed and the formula of shops decayed more and more. Especially the 2nd floor and above became empty.

The owner asked Dutch architect Ibelings van Tilburg to investigate the possibilities for this location: demolition and new construction or preservation of the existing building in a new context. Because of the few (modern) monuments existing in the city center, the architect chose for the 2nd option. Their suggestion was placing a large block of apartments above the building (Figure 4-3). Through this urban densification the liveliness of the city center was to be enhanced, especially in the evenings.

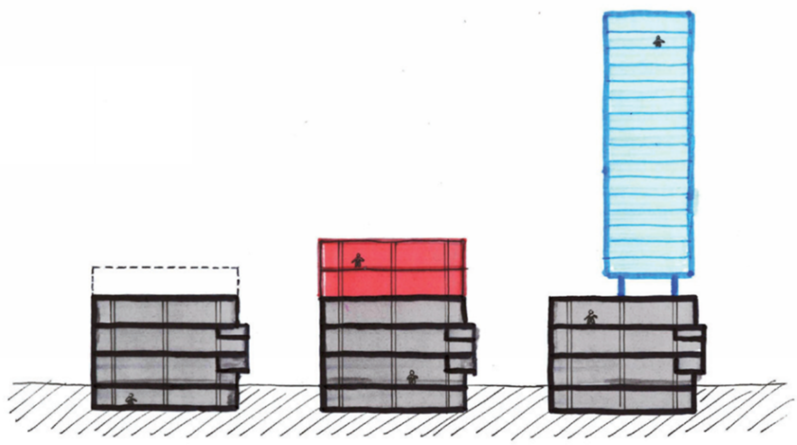

Figure 4-3 Karel Doorman - Development of building Ter Meulen (Hermens, Visscher, & Kraus, 2014)

The challenge was to keep the existing building as original as possible, by adding the new 16 stories with apartments truly on top of the existing building, using the existing load bearing system of columns and pile foundation.

The solution to this question was found by a combination of three approaches:

- The analysis of the load bearing system and its existing and unrevealed load bearing capacities.
- Using an ultra-light weight building system for the new apartment building on top.
- Separation of the vertical load bearing from the horizontal load bearing.

Existing structure

Available Data. The existing building was well documented: gravity load calculations and stability calculations, concrete dimension and reinforcement calculations and drawings of reinforcement were available. Also the pile plan, the geotechnical survey and advice and a report on the installation and testing of a test pile were available, together with a calendering drawing of the installation of the piles.

Existing Load Bearing System. The load bearing system was completely cast-in-situ concrete. The columns and beams did provide the lateral stability of the building through rigid frame action. The column grid was 8 x 10 meters. Because of the rigid frame action the columns are almost similar in dimension on all floors: round 850 mm in the basement to round 800 mm in the 2nd floor. The intended compressive strength of the columns was 250 kgf/ cm^2 which can be compared to a C14/17 strength according to Eurocode. The main beams are 600 x 850 mm with an intended compressive strength of 200 kg/cm^2. The existing structure was checked and designed according to the national codes for new buildings of that time. The NEN 8700 did not exist at that time, except for some memos of this standard.

Existing Foundation. The foundation was designed with reinforced prefabricated concrete piles, with a shaft dimension of square 380 mm and a + shaped pile tip of 760 mm. The calendering showed that there had been a great amount of soil densification due to the installation of the piles: in a group of 8 piles the last 25 blows on the pile caused a settlement of 200 mm in the first, down to only 40 mm in the last pile of the group. This was a strong indication that the bearing capacity of the piles was much larger than the originally intended 70 tons (or 900 kN according to present codes).

Tests. First inspections (visual and with a Schmidt Hammer) indicated that the quality of the construction and thus the concrete strength was very good. In combination with experience and literature the first starting point was a present concrete strength of C28/35 for the columns. In a later stage, cylinders were drilled and tested from 18 different columns, giving a real concrete strength of even 40,9 N/mm^2. To be able to recalculate the capacity of the existing piles as accurate as possible new cone penetration tests (CPT's) were made, inside the building right next to the pile groups, thus measuring the soil densification: the load bearing capacity according to present codes was 1.600 up to 2.000 kN.

Differences in Settlements. The new block is placed on only two of the three existing column lines, causing differences in settlements up to 25 mm between the columns. These implied deformations cause bending moments in the beams and thus in the columns. These bending moments were calculated smaller than the minimum required bending moments in the supported columns (in conformity with the structural code), so they did not reduce the vertical capacity.

Check of Existing Reinforcement. In several places parts of existing columns, floors and beams were removed because of the renovation. In all those places the found reinforcement was compared with the original drawings. No deviations were found, giving good confidence in the original construction.

What-if Analyzes. Sensitivity analyses were performed for the unforeseen situation in which reinforcement would be (partly) absent in crucial elements like columns and piles. The residual safety was calculated to be sufficient. The same was done for the case of broken piles in a pile group, with the same positive result.

Vertical extension

Load Bearing Capacity of the Existing Building. The solution for the challenge to place the 16 stories truly on top of the existing building was found by separating the horizontal loads from the vertical, for the new expansion as well as for the existing building: 2 concrete stability cores were added (for staircases, elevators and ducts) with a section of 7 by 9 meters and wall thickness of 0,4 meters. These were not only used for the new building, but also the floors of the existing building were rigidly connected to the new stability cores.

In the existing building the structural load bearing system thus changed from a system with rigid frame action, with bending moments in the beams and columns caused by horizontal loads, to a system with supported columns, only having to carry vertical loads (Figure 4-4). By eliminating those bending moments the load bearing capacity of the columns increased from about 5.000 kN to about 10.000 kN without any structural modification of those columns.

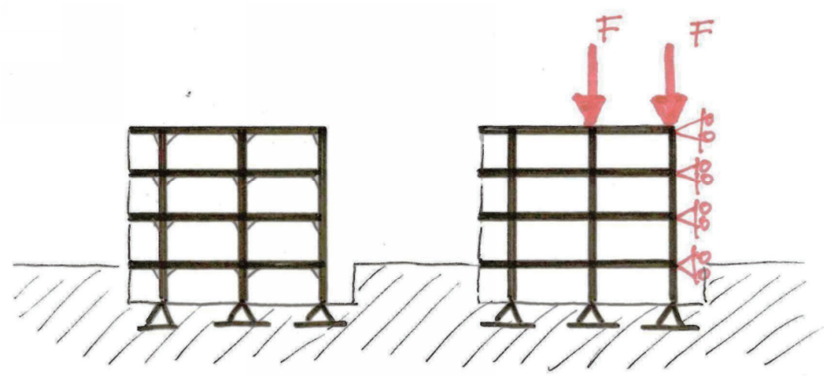

Figure 4-4 Karel Doorman - Structural stability scheme before (left) and after (right) (Hermens, Visscher, & Kraus, 2014)

With a weight of maximum 250 kg/m^2 for the apartments (all inclusive, per GFA) and an extreme live load of 175 kg/m^2 on one floor and 70 kg/m^2 on all other floors (load combination factor 0,4 according to Dutch Code), it was now possible to realize the 16 within those extra 5.000 kN. The pile-groups had a new load bearing capacity of more 8 x 1.600 kN 12.800 kN, which was more than the acting design force in the new situation.

Structural Design of the New Apartment Block. The optimal column grid for the new apartment building was chosen to be 4 x 6 meters. In the lowest new floor steel transfer beams in two directions are used to transfer the new column grid (perimeter columns and middle columns) to the column grid of 8 x 10 meters in the existing building.

With the small footprint of the stability cores and the lightweight structure overturning uplift due to wind loads could be likely. For that reason the foundation plate below the new stability cores is 10 x 16 meters. All new piles have been placed near the perimeter of the foundation plate. In that way tension forces in Serviceability Limit State were prevented. In Ultimate Limit State the tension forces in the piles are up to 600 kN, for which the piles are placed deeply into the sand layer more than 25 meters below ground level.

In order to stay within the available (released) load bearing capacity, the 16 apartment floors can weigh only 250 kg/ m^2. That is roughly 1/5th of the weight of standard Dutch concrete apartment buildings. The acoustic isolation demands however are very high in the Netherlands and in this case a value of 10 dB higher than the governmental demands is used in order to prevent user-complaints.

Therefore the ultra-light-weight structure is built up as follows (Figure 4-5):

- steel columns and beams
- wooden floor system with a 55 mm concrete topping
- a double separated metal stud and gypsum wall system between the apartments
- a wooden facade (exterior wall)
- glass cladding on the outside

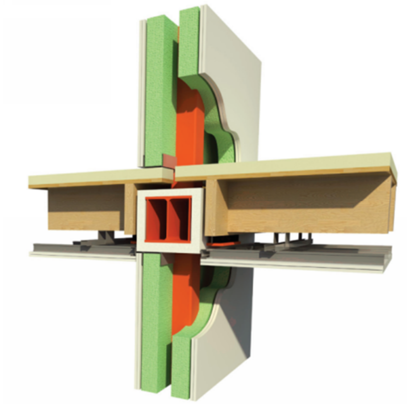

Figure 4-5 Karel Doorman - Detail of separated floors and walls (Hermens, Visscher, & Kraus, 2014)

Sustainability & costs in the design process

Ir. Michiel Visscher talked about how sustainability emerged at the end of the design process, about the initial intention of the design team, the achieved result, with an ultra-light weight solution for the vertical extension, and the driving force behind the whole project. His approach on evaluating the building is more holistic and reflective back on the 10-year-long project.

Sustainability, by means of tools or Life Cycle Assessment, was not considered during the design process, but emerged as one of the main values after the completion of the project. 'In any case, at the time this project started there were no sustainability tools available, such like BREEAM, or no sustainability labels', mentions Visscher. The initial driven force behind this innovative design was to make the project feasible by using as much as possible of the existing structure, with the minimum extent of structural interventions. During the design stages the different parties involved were more focused on how to deal with the various technical issues rather than making the building sustainable. However, ir. Visscher explains that after the completion of the project, in the attempt to describe or present the project to others, Royal Haskoning DHV realized that the Karel Doorman building was much more than a light weight vertical extension. The design team, having an overview of the project and the knowledge earned through these years, conceived the true value of the building. The principal idea of vertical extension could be applied all over the world and could form a potential solution for sustainable urban densification of several big cities, building over infrastructure and other civil engineering works.

Michiel Visscher approaches sustainability not in the sense of numbers but as something really simple: 'If you can explain it to your parents, and convince them for what is sustainable, then the rest is just numbers. Lists and numbers are undoubtedly helpful, however, they block often a designer's creativity, not allowing to think for choices and possibilities out of the box'. Visscher believes that the industry has to change and look what is beneficial for the society as a whole, i.e. a more holistic view is necessary and a different approach of sharing profit.

As far as the costs are concerned, it was explained that since the client was sharing the same faith and vision with Royal HaskoningDHV, about the right approach and success of the project, more risks were taken at the first stages of the engineering part. The time devoted to the investigation of the hidden, or not, structural capacity of the existing structure was actually tripled, compensating, however, in a positive territory, with construction costs and profit from maximizing the number of apartments.

Design parameters

Karel Doorman building project is a unique project where various simple and existing structural and architectural elements are used, but the unique composition of these elements has led to an integral structural challenge. The behavior at the system level turned out to be completely different than at component level: if you change a part of your design or detailing, the behavior of the entire system changes. This means that the design of light buildings asks for a different design approach: in addition to one's own professional input and responsibility, a shared responsibility approach needed for the overall system.

- From a clearly technical point of view, the load bearing capacity of the foundation was the parameter that determined the amount of extra square meters added on top of the existing building, and it was reached to its maximum.
- The alteration of the structural system was also critical for the realization of the vertical extension.
- The testing methods revealed higher concrete quality.
- The risks taken, in terms of time and money invested in order to investigate the ultra-light weight solution. This was determinant for the realization such an extensive vertical extension.

4.2 Groot Willemsplein

Introduction

The information presented in the following, regarding the Groot Willemsplein building, has been retrieved by two interviews conducted by the author, one with the structural engineer of Pieters Bouwtechniek engineering consultancy, ir. Ming-Chen Ku, and the second with the developer of the project from LSI Project Investment N.V., Ruud Kersten. The structural drawings of both the old and the new structure were found, after extensive research, at the Department of Building Permits, Urban Development Section, of the Municipality of Rotterdam. Additionally, the article 'Optopproject profiteert van verborgen geschiedenis' (Beerda, 2012), was a source of valuable information during the research.

Figure 4-6 Groot Willemsplein building

General information

Table 4-2 Groot Willemsplein building - General information

Location	**Willemsplein, Rotterdam**
Project	Groot Willemsplein
Client	LSI Project Investment N.V.
Former 1st function	Distillery
Former 2nd function	Office building
Current function	Office building │Horeca
Architectural design	DAM & Partners Architecten
Structural design	Pieters Bouwtechniek, Delft
Contractor	Slavenburg, Capelle aan den IJssel
Originally built	1946
Started - Completed	2011 - 2013

At the foot of the Erasmus Bridge, in June 2013, the building Groot Willemsplein is delivered. The former distillery from 1946, is thoroughly renovated for the second time. The building at the Willemsplein (Willem Square) will serve as modern and sustainable office building for Joulz, electric utility company.

The building was built in 1946 for the former distillery *N.V. Blankenheym & Nolet's* and in the late 70's transformed into an office for *Smit Tak Internationale*. After several years of vacancy, the building has been given a new purpose. The idea was to vertically extend the building by placing on top of it 3 extra floors.

Existing structure

Available Data. The structural engineers of Pieters Bouwtechniek had at their disposal many reports of load and stability calculations and structural drawings with concrete dimensions and reinforcement. Moreover, information about the pile plan and the geotechnical reports of that time. The study of the existing structure started from carefully looking at this available information and proceeding with a variety of tests.

Existing Load Bearing System. The existing concrete structure has been standing there since 1946. The prefabricated concrete façade was placed at the end of the '70's, replacing the original masonry façade of the distillery. It was only when the structural engineers started to examine the structural drawings that they became aware of the unexpected structural reserves the building had. Despite the fact that the original concrete strength class was relatively low (B15), the concrete structure was designed and reinforced for a load of 15 kN/m^2. That was due to the first former function of the building. This is exactly the reason why adding 3 extra floors on top of the existing building was possible. The columns and beams did provide the lateral stability through the rigid connections and the moment resisting frame. The column grid was 6800 x 5150 mm^2. The columns are almost similar in dimension; for the part of the building from '40's (axis G to N, Figure 4-7): 700 x 1000 mm^2 in the ground floor, 700 x 700 mm^2 in the 1^{st} floor and 600 x 600 mm^2 for the rest of the floors. For these columns no strengthening was necessary in order to withstand the increased axial loads from the vertical extension. For the part of the building from the '70's (axis E and F, Figure 4-7) the columns were square 450 mm and 500 mm. The floor was spanning 1720mm in between the secondary beams of 350 x 500 mm^2 and , which transferred the loads to the main beams of 350 x 500/600 mm^2 (higher close to the columns) which were supported by the columns (Figure 4-8). The structural grid and the positioning of the columns of the second floor can be seen in Figure 4-7.

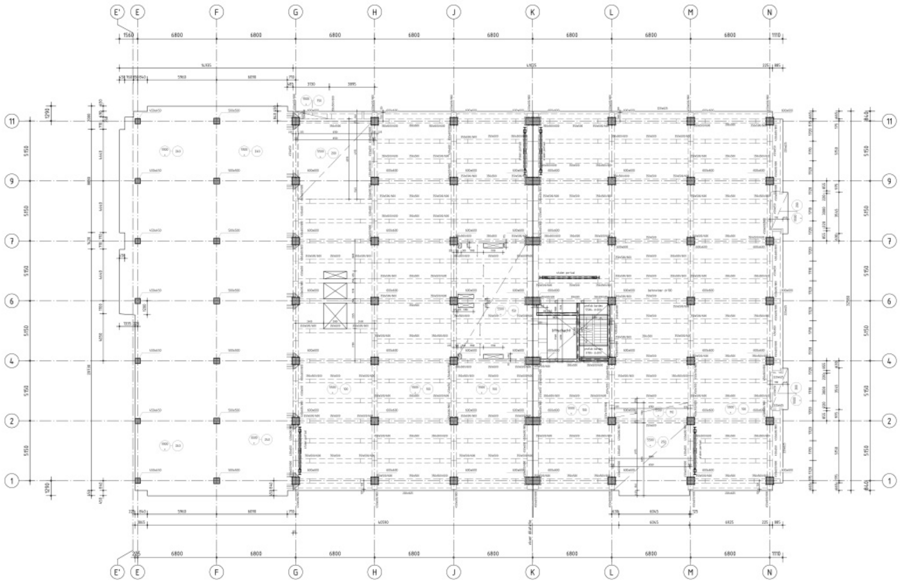

Figure 4-7 Groot Willemsplein - Structural drawing of the second floor, existing structure, Groot Willemsplein building

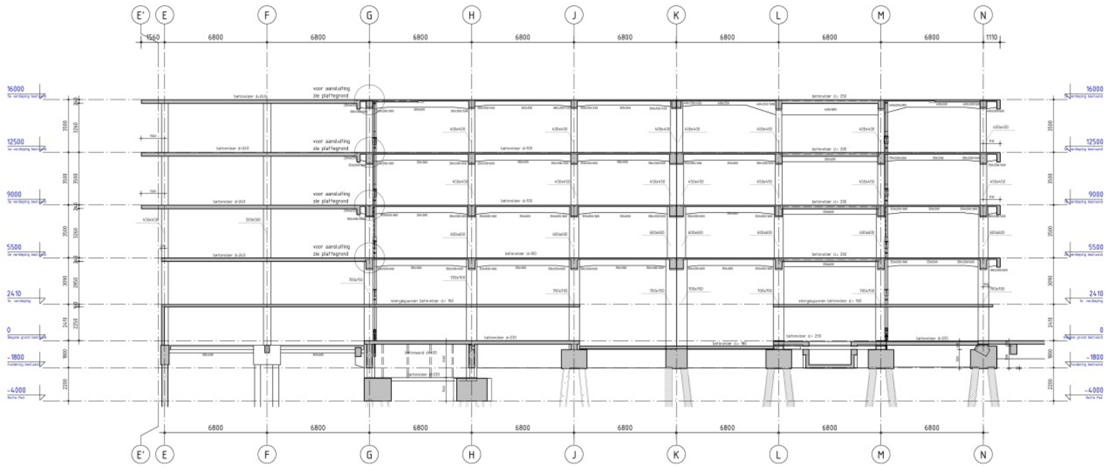

Figure 4-8 Groot Willemsplein - Section of the existing structure on axis 2

Existing Foundation. The foundation out of 1940's was designed by prefabricated reinforced concrete piles in square and circular shapes, with enlarged toes. Besides, the piles from the 1970's do not have the outdated method of the enlarged toe. The different types can be found in Figure 4-9. The load bearing capacity of the existing piles was varying from 620 up to 1200 kN. The majority of the piles has been placed with a slope varying from 1:10 to 1:3 (Figure 4-9). This slope caused problems afterwards and limited the possible positions of the new piles. The piles from the 1940's were placed mostly in groups of 5 or 6 piles under the columns and were connected with a pile cap. The ones from the 1970's are in groups of 2 or 3 piles also under the columns.

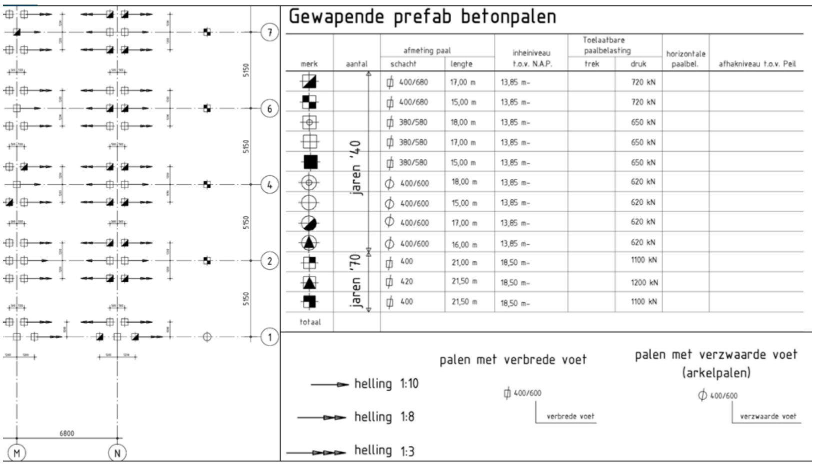

Figure 4-9 Groot Willemsplein - (Left)Table with the existing pile types, including dimensions, compressive strength, and slope (Right) Part of the existing foundation plan, including pile types, distances, and slopes.

Tests for Strength and Existing Reinforcement. The existing structure has been tested in terms of strength and quality in order to confirm the validity of the drawings, even during the construction. The team had to be convinced that the concrete elements were reinforced according to the drawings. For instance, the unfixed prestressed tendons of a prestressed concrete floor added in the 1970's on the ground floor had been tested and found in good condition in order to reuse that floor.

Vertical extension

Load Bearing Capacity of the Existing Building. In order to proceed with the vertical extension, the building was stripped down almost to its structural skeleton. Elements and later additions such like, separation walls, technical rooms and a later added floor of the office building have been totally removed as debris. The only element left untouched was the prefabricated concrete façade, a typical sample of the façades constructed in the end of the 1970's (Figure 4-10). Even a large part of the first floor has been removed in order to create an atrium in the middle of the office building.

Figure 4-10 Groot Willemsplein building stripped down to its structural skeleton with untouched facade (pic. retrieved from www.skyscrapercity.com)

The fact that the structure had been calculated at the first place for the load of 15 kN/m^2 was decisive for the development of the project. This reserve load bearing capacity allowed the addition of the extra square meters on top of the existing structure. In order to achieve that, significant structural interventions were necessary and these have been introduced in the most smart and efficient way.

Structural Design of the New Block. The new headquarters of the energy company named Joulz would be housed in the existing building, whilst the three extra floors added on top would be rented separately. The latter created the need for an apart access to the new part of the building, which was solved by 2 new concrete cores (Figure 4-11). These cores provide stability for the new part. Moreover, the contractor Slavenburg decided to remove the beams and floors around the existing staircases and fix the floors on the new concrete floors. In this way the structural system is transformed from a moment resisting frame to one with shear cores (Figure 4-11).


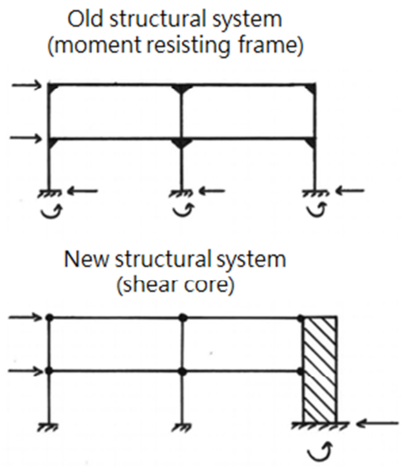

Figure 4-11 Groot Willemsplein - (Left) Construction of the new stability cores (Right) Schematic representation of the new and old structural systems

Slavenburg, constructed under the new cores a foundation that consists out of screw injection piles and heavy pile caps. Thereafter, a lightweight steel structure and hollow core slabs of 6,80m length for the large spans formed the structure of the three new floors that came on top, of approximately 1700 m^2 each. More piles were placed under the point supported floors of an aisle that was added on a later stage, right under the existing columns. The columns located in the new atrium were thickened because their buckling length became too large after having demolished the floor.

Sustainability & costs in the design process

Breeam was the tool used to measure the sustainability of the project. In reuse projects the goal in Breeam score is really high, i.e. 57% up to 65%. That was also the aim of the employer LSI Project Investment N.V.. During the construction steps were taken to reduce the impact on the environment, for example through innovative construction methods. The building was largely renovated and at the same time all useful parts such as the structural skeleton and part of the façade have been maintained. The new part of the building is designed relatively lightweight, so that the amount of material added on the building would be as little as possible. In this way, the environmental impact from the material use was limited. Additionally, debris and concrete aggregates were used where concrete work was needed. The greatest profit is actually the fact that the building is redeveloped and that at the last moment the total demolition was avoided.

Design parameters

The Groot Willemsplein project was rather particular because of the strict timeframes during the decision-making and the design phase. Ruud Kersten explained how the parameters that defined the amount of extra square meters added on top of the existing building, unlike other projects, were determined. These parameters were mostly related to the decision making phase and less with the design phase.

LSI bought the existing building just before the financial crisis began. Therefore, the building was vacant for many years and LSI was looking for a client interested in moving in the building, so that the redevelopment could start. This client was Joulz, a specialist in energy infrastructure, so as to accommodate the new headquarters of the company and should be delivered within two years; the decision-making stage was very constrained. After that, the idea of vertically extend the building, in order to increase the profit, looked appealing to LSI and a new part of negotiations started with Joulz, who had already signed the contract for the existing building.

In addition, there were restrictions from the municipality of Rotterdam and the urban planning related to the amount of office square meters that were allowed to be built in this area on the city. The total area of the new building should not exceed the 14.000 m^2 approximately, in order to get the building permit.

To conclude, the parameters that affected the vertical extension, were:

- timeframe restrictions, and,
- municipal policy, i.e. urban planning regulations from the city of Rotterdam.

4.3 WESTERLAANTOREN

Introduction

This project is analyzed with the help and the collaboration of Aronsohn Consulting Engineers. The senior project manager, ir. Michel Schamp explained the structural design of the tower during an interview carried out. Besides, the article 'Een tweede leven voor de Westerlaantoren' (Schamp, 2010) was used as a source of information, where the structural design of the building is presented more into detail.

Figure 4-12 Westerlaantoren building

General information

Table 4-3 Westerlaantoren project - General information

Location	Westerlaan 10-65, Rotterdam
Project	Westerlaantower (Westerlaantoren)
Client	Maarsen Groep
Former function	Office building
Current function	Retail | Offices | Residences
Architectural design	Ector Hoogstad Architects
Structural design	Aronsohn Constructies Raadgevende Ingenieurs
Contractor	Dura Vermeer Bouw Rosmalen
Originally built	1959
Completed	Mid 2012

In the '60's, Vopak requested from the municipality of Rotterdam and managed to acquire permission in order to build a high-rise building up to 70 m height. That was an exception according to the urban planning of the specific district, due to the status and the great importance of Vopak for the harbor of Rotterdam. This is how the building originally exists. The renovation of the Westerlaantower is a project of Calandstraat CV, which consists out of Maarsen Groep, Brouwershoff Beheer and Royal Vopak NV. The design originates from the architects Ector Hoogstadt, who converted the old office building to a wonderful building where office and residential functions are combined. Aronsohn structural engineers were responsible for the structural design of the renovation, and moreover, to accomplish the challenge of increasing the height of the building to approximately 70 m. Besides, an underground parking garage should be designed for both the employees of the offices and the residents of the apartments. The final design is composed of a commercial level on the ground floor, offices from the 1st up to the 10th floor and residences from the 11th up to the 19th floor. On the roof one can find some installations and the installation for the maintenance of the façade. In the basement is located a passage to the parking garage, the storages for the residences and the HVAC systems for the offices.

Existing structure

Available data. The case of the Westerlaantoren is a particular one since the original structural design was done by Aronsohn Constructies Raadgevende Ingenieurs and consequently all reports and drawings were directly available.

Existing Load Bearing System. The existing structure is made out of in situ casted concrete floors, beams, columns and cores. The existing tower has an almost square floor plan, where the floor area is larger from the 1st up to the 4th floor because of cantilevered floors. Above the 4th floor and up to the 16th floor the floor areas are more or less identical with a surface of approximately 32,5 x 32,5 m^2. The roof is in a height of around 61 m from the ground level.

The vertical loads are carried by 220 mm thick in situ casted floors, resting on concrete beams which bring the loads to 16 columns in the inner-ring and 20 columns in the outer-ring. The layout is shown in Figure 4-13. The dimensions of the concrete columns are 550 x 1000 mm² in the inner-ring and 500 x 750 mm² in the outer-ring. Underneath the 4th floor the columns are larger; 700 x 1000 mm² and 500 x 860 mm² respectively. The stability of the building is ensured by the concrete core which has a clear stronger and weaker direction. The bottom four floors have got four concrete slabs in the corners which also carry some of the horizontal loads.

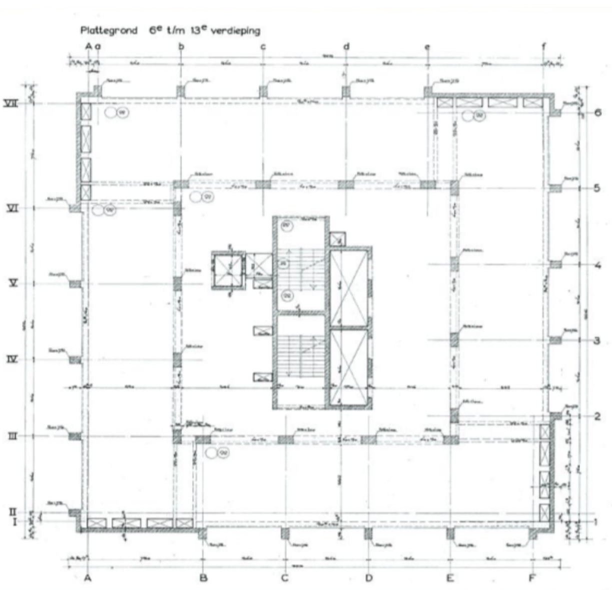

Figure 4-13 Westerlaantoren - Typical structural floor plan of existing building from the 6th up to the 13th level.

Existing Foundation. Underneath the building a reinforced concrete plate is placed with a thickness of 2 m. This plate transfers all the vertical and horizontal loads to the pile foundation. In totally there are 370 piles with a square section of 400 x 400 mm² and a reinforced footing of 530 x 530 mm².

Tests. The first thing that had to be done was to get more information about the capacity of the piles. According to the existing cone penetration test (CPT) a foundation engineering consultancy, Tjaden Ground mechanics, recalculated the allowable load bearing capacity of the piles in line with the current standards. This research showed that the foundation piles had a lot of spare capacity. The original load bearing capacity of the piles was calculated approximately 1000 kN. After the recalculation, with the existing cone penetration test, it was clear that in the ultimate limit state the allowable load bearing capacity of one pile could reach values up to 2000 kN. This difference was mostly because of the differences in standards, the older standards were more conservative compared to the current standards. The maximum load in the ultimate limit state on the piles has been calculated 1450 kN, so according to the new calculation the load would not exceed the allowable load bearing capacity of the piles.

In the pile plan (Figure 4-14) one can see that the piles are placed too close to each other, especially under the core and inner-ring columns; center to center distance 1450 mm, approximately 2,7D. The option of soil compaction was one of the possible solutions and therefore they had to research the ground layers. Unfortunately, the possibilities for new cone penetration tests were limited. Eventually, two new cone penetration tests were made, which gave a better image compared to the existing tests, but obviously these were not representative for all piles.

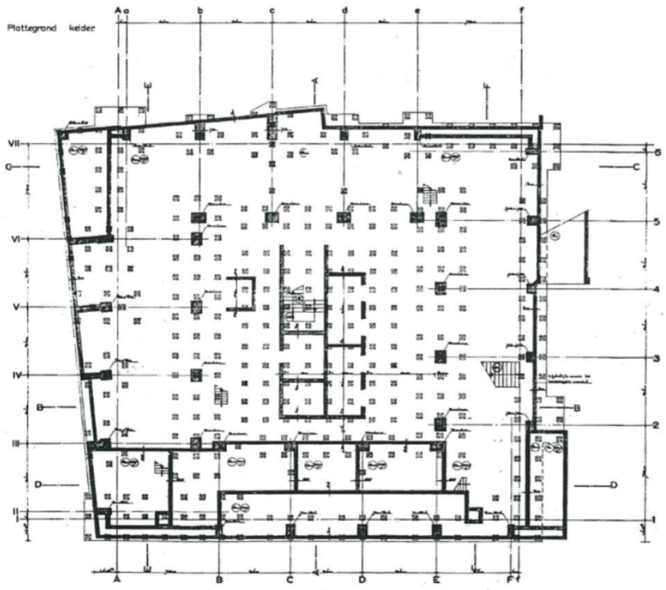

Figure 4-14 Westerlaantoren - Basement floor plan with pile locations indicated

As far as the upper structure is concerned, ir. Michel Schamp explained, that the validity of the information mentioned in the reports and drawings was tested during the demolition of the two upper existing floors of the tower and of large parts of the low-rise building in 2002. Aronsohn did not perform any further tests. However, in 2008 Dura Vermeer and Maarsen Groep wanted to research one more time the feasibility for their point of view. For the purpose of this, cylindrical concrete specimens have been taken and compression tests were performed. Moreover, special inspections have been done in order to check whether the reinforcement specified in the drawings was also applied in the concrete elements. Eventually, it was proven that the concrete was stronger than the K300, which was the qualification of the time of construction, that meant 300 kg/cm^2 compressive strength, which is comparable with the concrete B30. This is present concrete strength class C28/35. From the measurements it was concluded that the concrete was actually comparable with that of B35, given the fact that concrete became stronger. The quality of the steel was QR240 in terms of that period which corresponds to the steel FEB220 nowadays. These rebars were also smooth in comparison with the ribbed rebars of nowadays. That was actually a typical structural steel. There was also steel QR40 or FEB400 as called now. SGS Intron was the company that inspected the structure and made the tests and they certified that the concrete quality was fulfilling the requirements.

Vertical extension

Load Bearing Capacity of the Existing Building. To increase the height of the tower from 61m to 76 m and to make the floors suitable for the different functions (offices and residences) is was decided to strip down the building to its structural skeleton and to demolish the floors 15th to 17th, as well as the cantilevered floors from the 1st to the 4th floor.

Given the large thickness of the foundation plate, it was clear from the beginning that placing extra piles was not an option in order to increase the capacity of the foundation. At first, the bearing capacity of the piles should be clarified. Secondly, all the additional provisions should be focused on spreading and as much as possible equally dividing the extra loads over the piles. It became soon apparent that without additional structural measures, the stresses in the concrete structure and the allowable loads of the piles would be exceeded. Eventually, the investigations done and the measures taken are summarized in the following:

- The existing piled foundation is recalculated on the basis of the current geotechnical standards.
- The walls of the concrete core in the basement have been strengthened and new walls were added (Figure 4-15), to evenly divide the vertical loads on the piles underneath the core and the columns in the inner-ring and to increase the capacity of taking over moments from the wind loads.
- The walls of the concrete core above ground level are strengthened to be able to carry the increasing wind loads.
- The concrete slabs in the corners on the first four floors are extended to the 20th floor.
- An outrigger structure is applied in order to increase the stiffness of the building.

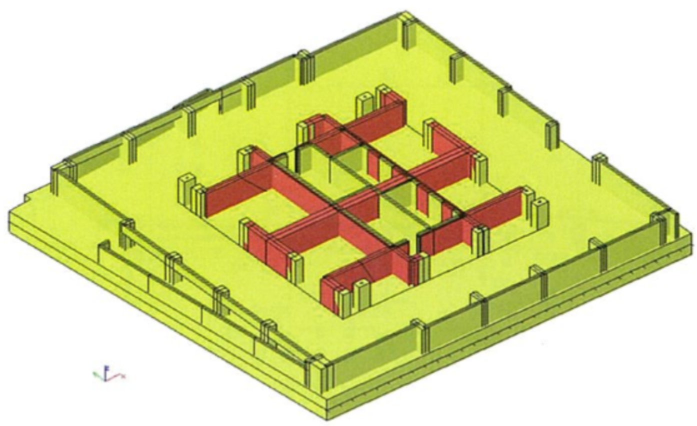

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Figure 4-15 Westerlaantoren - In red color the added walls, in the inner-ring and the core, for more evenly distributed vertical loads

Consequently, the additional structural provisions were limited to the concrete structure. In the strong axis of the concrete core it was obvious from the beginning that the extension was not possible without additional structural provisions. This finding held also for the weak axis of the concrete core, where the stresses in the concrete exceeded even more the allowable values. As aforementioned, concrete walls were added in between the columns of the inner-ring and the concrete core, so that to distribute more evenly the increased vertical loads over the piles. This measure, in combination with the new ground floor, created a box-structure with a height of 4500mm. The increased stiffness of the structure in the basement was consequently favorable in order to carry and transfer the moment from the wind loads to the piles. Nonetheless, the thickness of the added concrete walls was restricted by the architectural design and the solutions mentioned above were not enough to provide the required stiffness to the structure. Thereafter, the solution of the outrigger structure was introduced.

Structural Design of the New Block. The vertical extension of the tower consists out of 5 new floors and a steel domed roof. The lightweight balconies are fixed on the structure with the use of a steel structure. The increase in the height of the tower is translated in an increase of 14%, with respect to the old situation, for the total vertical loads in the ultimate limit state (ULS). As far as the horizontal wind loads are concerned, the new floors and the balconies will lead to an increase of 60% at the moment on the foundation relating to the original one. This original moment of the 61m building, according to the current standards, is 97000 kNm and consequently the increased moment reaches the 157000 kNm.

This outrigger structure will connect the core to the inner-ring columns, involving in this way the columns in the structural system that reacts to the horizontal loads. In such a way, there will be axial forces in the columns, which will generate an adverse moment compared to the moment generated from the horizontal loads (Figure 4-16). This counteracting moment will have as a result not only the reduction of the moment acting on the lower part of the core but also the significant reduction of the horizontal deformations.

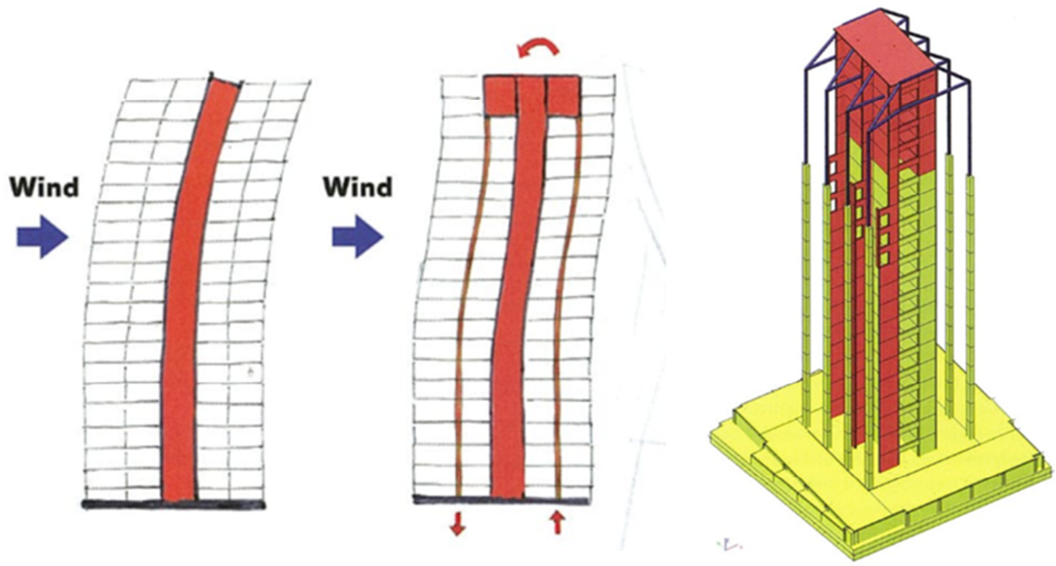


Figure 4-16 Westerlaantoren - Stability scheme with (mid) and without (left) the outrigger structure – (Right) 3D view of the stability system

The effectiveness of an outrigger system depends on several parameters, such as the height where the outrigger will be positioned, the stiffness of the core, the columns and the outrigger itself. As a general rule, an outrigger is most effective when it is positioned at the 2/3 of the height of the core. In this specific design, it was decided for the outrigger to be placed on top of the building; there it does not disturb in a great extend the floor plans of the residences.

Although the position of the core is not the optimum, it offers a great advantage to the structural system. As one can see in Figure 4-17, the zero point of the moment distribution diagram in close to the connection of the existing and the new structure. That means that the tension forces in the concrete walls around this "transition point" are kept in low values and consequently, the connections are designed are practical connections.

Moreover, an interesting point is that the stiffer the outrigger and the columns become with respect to the core, the larger the counteracting moment resulting from the outrigger, and hence the reduction of the moment on the foot of the core. Given the fact that the outrigger and the new columns are made out of steel, one of the most important parameters was the cross-sectional area of the steel profiles. Eventually, stiffness was the normative parameter and larger profiles were used, which were not necessary from the strength point of view.

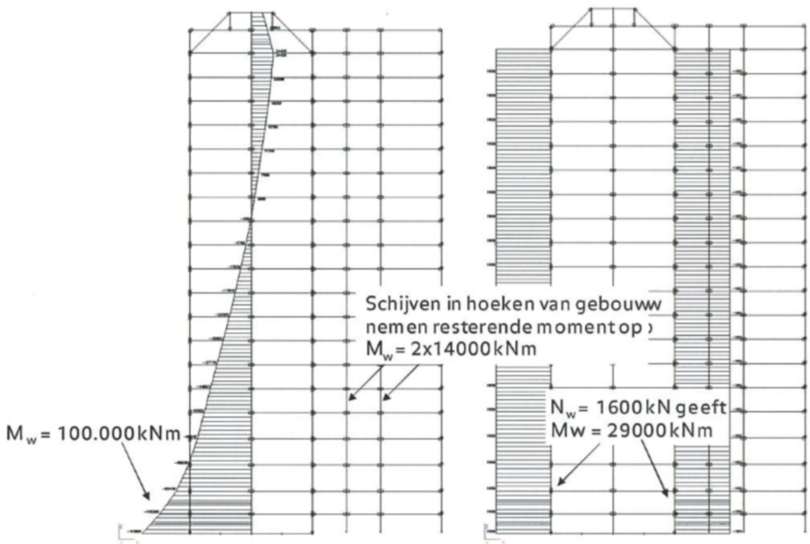


Figure 4-17 Westerlaantoren - (Left) Moment line after structural provisions (Right) Counteracting moment due to axial forces in the columns

Sustainability & costs in the design process

"Sustainability was not one of the primary goals of the project, but came as the consequence of certain choices", says ir. Michel Schamp. "In 2005-2006 we researched in depth the case of demolishing completely the tower and build something new in the same place. Of course that was something that the municipality did not want, but still we looked it up. This research revealed that this option would have been much more expensive. So it was much cheaper to reuse the structure." This cost-related fact was the keystone for reusing the structure. Moreover, in case of demolishing the existing building, it was not possible to obtain again a permission from the municipality to construct a high-rise building. Thus, demolition was not profitable at all for the owner of the building. After having concluded that reusing the existing structure was more economical in comparison with demolition and new structure, the option of exploiting the full potential of the height restrictions was researched. The strength margins of the structure, both foundation and upper structure, were sufficient so that to carry the extra loads. The feasibility study was only done from the structural point of view. Consequently, the amount of the extra square meters and the total height of the building were determined by the costs, at a first sight, as well as by the particularity of the building permit.

Looking at the sustainability and costs, this time from the material use standpoint, the stabilizing system was chosen so as to be economical in terms of time, costs and amount of materials. The first idea was to add an extra reinforced layer of concrete on top of the existing floors. This way the stiffness would have been increased against the horizontal loads. This solution turned to be too expensive and time-consuming and eventually was declined.

Design parameters

- The foundation was the most critical structural element. The thickness of the foundation plate was not allowing for extra piles. Even if the issue of the building permit did not exist, it would only be possible to build one more floor without designing the balconies, based on the load bearing capacity of the piles.
- The chosen structural system of the outrigger was the most cost-effective solution. The outrigger structure predominated the solution of the extra reinforced concrete layer on top of the existing floor.
- Municipal policy, i.e. the urban planning restrictions from the municipality of Rotterdam regarding the height of the building.
- The testing methods revealed higher concrete quality.
- A feasibility study proved reusing the existing building to be more cost-effective than demolition and new built.

4.4 ZEEMANSHUIS | MARITIME HOTEL

Introduction

The Zeemanshuis project is an interesting case that has grabbed the attention of the technical press in the Netherlands. Zonneveld Engineering Consultancy and Bias Architects have communicated this project with two extensive and enlightening articles: ' Uitbreiding Zeemanshuis, Rotterdam – Boorplatform in Havenstad' (van der Windt,

2003) and 'Tweede stalen doos over het Zeemanshuis' (Wind, 2004). These articles are used as valuable sources of information together with the information retrieved for the interview of J.P. van der Windt, director and project leader of Zonneveld Engineering Consultancy.

Figure 4-18 Zeemanshuis / Maritime Hotel building

General information

Table 4-4 Zeemanshuis project - General information

Location	Willemskade 13, Rotterdam
Project	Zeemanshuis | Maritime Hotel
Client	Foundation Zeemanshuis Rotterdam
Former function	Seafarers shelter
Current function	Hotel
Architectural design	Bias Architecten
Structural design	Ingenieursbureau Zonneveld | Viets, Harskamp
Contractor	Hijbeek B.V., Zwijndrecht
Originally built	1951-1953
1st extension	January 1998 - December 2000
2nd extension	September 2003 - May 2004

The Zeemanshuis was built gradually, starting in the early 1950's with the part on the Willemskade (from now on mentioned as "first wing") and following with an horizontal extension of the building, in 1960, on the Westerstraat (from now on mentioned as "second wing"), where a second entrance was provided. It is a typical sample of the architecture of the buildings constructed after the Second World War in Rotterdam designed by the architects Nefkens and Buys. Traditionally, the Zeemanshuis offered to the seafarers, who had to stay for a few days in Rotterdam, a low-priced shelter. The need for such an accommodation still exists, even though nowadays the ships moor closer to the quays and they have on board excellent facilities. Considering the fact that the subsidies were decreased, and eventually stopped in 2005, the Foundation of the Zeemanshuis looked for new incomes. The new concept included Maritime Hotel (Figure 4-18), a low budget hotel for tourists and businesspersons. In order to exploit as much as possible the potency of the beautiful location of the building, on the river side and close to the Erasmus bridge, it was decided to vertically extend the part of the building on the Willemskade side with three extra floors that would accommodate thirty hotel rooms. Three years after the completion of the first extension, the Foundation decided to proceed with one more vertical extension of three floors, this time for the other part of the Maritime Hotel, at the Westerstraat side. Both extensions have interesting aspects, mostly because of the different requirements that emerged due to the fact that the building was a monument for the municipality of Rotterdam.

Existing structure

Available Data. As mentioned before the original building was constructed in two phases; at first the part at the Willemskade side and at a later stage the part at the Westerstraat side. The documentation of the first wing was not good, i.e. no structural drawings and reports were available showing the reinforcement of the concrete elements and the dimensions of them. Only architectural drawings were found in the archive. From the other hand, the second wing was well documented in structural drawings and reports from the original construction as well as from later structural alterations.

Existing Load Bearing System. The two existing parts of the building have two different structural systems totally separated from each other. The rationale behind is however almost similar for both. Concrete buildings with the vertical loads running from the floors to the concrete beams and then transferred to the concrete columns which carry them to the foundation. The stability of the first wing is ensured through the stiff connections in between the columns and the beams which create a moment resisting frame and also through shear walls. The second wing has a different stabilizing system and the stability is provided from a shear core.

Existing Foundation. The load bearing capacity of the foundation of the first wing was really difficult to be determined because of the lack of information of the existing structure. As far as the second wing is concerned, a recalculation of the existing piles showed that there was no reserve load bearing capacity in order to carry extra loads. For this reason, for both vertical extensions were founded on their own new foundation.

Tests. The structure of the first wing was left totally untouched; no extra loads were added on top of it, so no tests were made there to research the load bearing capacity and the quality of the concrete. On the contrary, the available information of the second wing, challenged the engineers to research the structural capacity of the structure and the quality of the concrete. Zonneveld engineers relied on the accuracy of the drawings regarding the reinforcement applied in the concrete elements and did not perform extra tests in order to verify it. However, cylindrical concrete specimens were taken and tested for the compressive strength of the concrete, which was found to be strong enough.

Vertical extension

Load Bearing Capacity of the Existing Building. There were several main issues with respect to the building that affected in a great extend the approach of the two vertical extension projects and limited the possible solutions. At first, the fact that the first wing of the building was characterized as a monument by the municipality of Rotterdam. This meant that the new design should not influence a lot the architecture of the existing building. Secondly, the existence of the artwork "Zeeman aan stuurrad met Madonna" (Sailor on wheel with Madonna) of the artist Bart van der Smiton on the façade of the building. Because of the artwork, the façade of the building is shifted around 0,8 m behind the building-line. Bias architects respected the monumental value of the building as well as the artwork and at the same time they took advantage of the available space due to the recess of the façade. This space offered literally the space in order to keep the extra floors separate from the existing building. The aforementioned reasons in combination with the absence of available data for the existing structure led to the decision to structurally separate the new block (Figure 4-19).

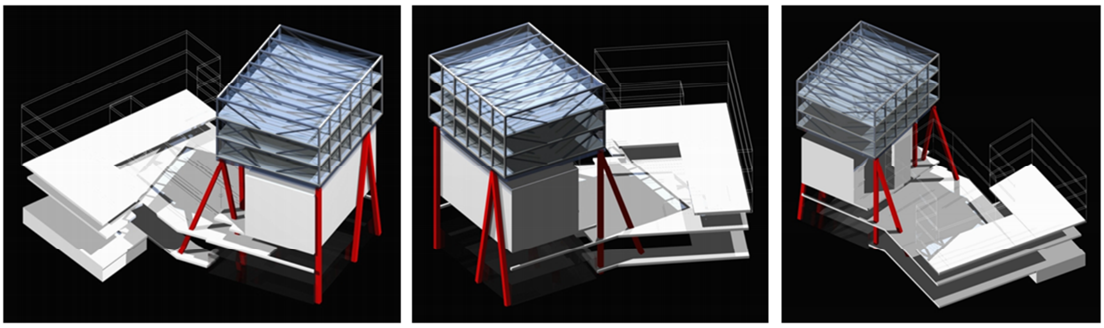


Figure 4-19 Zeemanshuis - 3D impression of the structure of the first vertical extension

The recalculation of the upper structure and the foundation of the second wing showed that the extra capacity was less than 5% of the total extra loads. For the purpose of this calculations the NEN 6702 was used, which was the national standard for technical principles for building structure. Taking into account these results, the structural engineers decided to transfer the vertical loads with a totally independent structural system.

Structural Solutions of the New Blocks. The many preconditions and the special features of the building on the one hand made the structural design complex but on the other hand limited the optional solutions. The first new part of the building consists out of three extra floors; the elevators and the staircase are extended upwards to the new part. The structural design was inspired by a drilling platform which is standing on high legs. The whole extension is supported by four leg structures, two in the front and two at the back, which are placed right in front of the existing façades (Figure 4-19). For the columns of these structures, circular hollow sections (CHS) are chosen, filled with concrete for fire safety reasons. To ensure the stability of the structure, seven out of the eight columns are set oblique in order for them to transfer to the foundation, not only the vertical loads, but the horizontal wind loads as well (Figure 4-20). The oblique columns, of each of the four leg structures, are founded on one single foundation block, so that the piles under these block do not have to undertake much horizontal loading, at least only the strictly necessary.

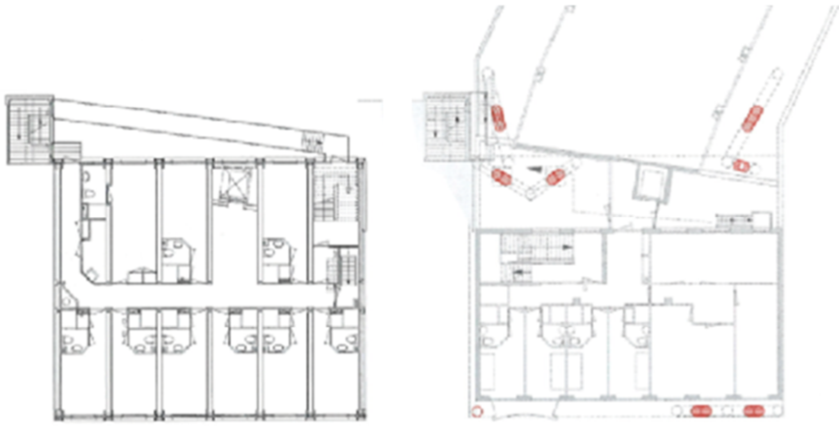
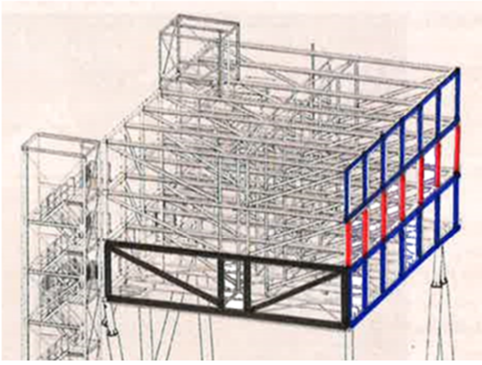


Figure 4-20 Zeemanshuis - (Left) Typical floor plan of the first extension (Right) Position of the eight oblique columns

As far as the upper structure is concerned (Figure 4-21), the floors and the roof rest on the one-storey-high Vierendeel girders that are located on the first and on the third floor of the extension part, in the front and rear façades. In between the aforementioned girders, hinged columns are constructed. The span in between the front and the rear façade ensure trusses in the separation walls. The top and bottom edges of these trusses are integrated hat beams, on which the floors of the hollow core plates are resting. The lower floor of the extension is placed at a certain distance from the roof of the existing part and with the use of diagonals is designed as a rigid disc, which transfers the horizontal loads to the leg structures.



* Vierendeel girders

* Hinged columns

* Trusses between the front and the rear façade

Figure 4-21 Zeemanshuis - Upper structure of the first extension

The foundation consists out of steel tube piles internal driven in the existing basement. These are positioned through drilled holes in the basement floor and the ground floor, so that the pile driver should not be settled in the basement. The foundation blocks were placed on top of the piles and then the legs on top of these blocks. The leg structures were supported permanently by the existing roof. The exact position of the lowest rigid disc of the extension is set by a calculation. It should be taken into account that after constructing all floors, the lowest floor would be subject to a rotation around the vertical axis and a horizontal displacement of approximately 30 mm. The Vierendeel girders have been manufactured in the factory so that the connections could be welded and form rigid nodes, and at the design stage, it was already taken into account that these girders could be transported as a whole.

The second phase of the vertical extension of the Maritime Hotel concerns the part on the Westerstraat where the building is extended also with three extra floors. At this second phase, a different approach was chosen. Because of lack of space, the solution of the first extension with the oblique columns that were taking over the vertical as well as the horizontal loads was not an option. Zonneveld structural engineers and Bias architects decided to use "vertical rigid disks" as the main structural elements of the new structure.

In the southwest transverse direction, a grid of steel profiles with rigid connections is placed close to the existing façade. This rigid framework is kept exposed out of the old and new façades, changing considerably the appearance of the old façade. Unlike the façade on the Willemskade, that was not a problem, since this façade was not protected as a typical architectural illustration of a certain period. The grid is dense with nine columns every 2040mm (Figure 4-22). In the northeast transverse direction, a steel framework with three columns is formed. One of these columns runs through the five existing floors and two of them through the recreation room on the ground floor. The connection between these columns and the existing structure is made with the use of felt and adhesive sealant in order to achieve flexibility. The columns are founded on their own foundation. In the longitudinal directions, there was the need for the steel structure to span over 27 m. That was achieved with the use of three stories high trusses, two along the façades and one hidden in the corridor wall (Figure 4-22). Remarkable of this second vertical extension is that the stability of the new part is provided by fixing the new structure on the existing concrete core.

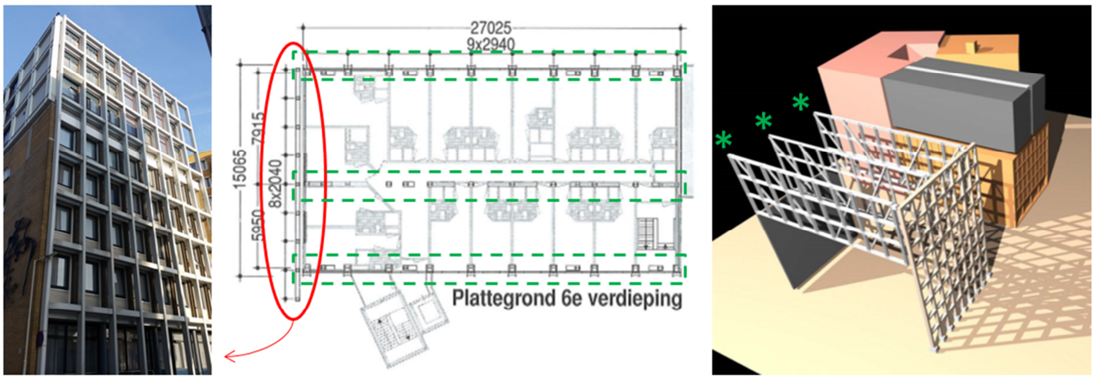


Figure 4-22 Zeemanshuis - Structural design of the second extension

Sustainability & construction costs in the design process

Sustainability in the form of Life Cycle Assessment was not incorporated in the design process of the two vertical extensions. Originally, the motivation behind the decision of keeping the existing structure and the vertical extension was based on the monumental value of the building. Unfortunately, the existing structures did not offer extra load bearing capacity to the extent that they could contribute in less material use.

The construction costs were the main parameter during the feasibility study of the first vertical extension. A number of alternatives we examined and the most cost-effective solution was chosen.

Design parameters

- Municipal policy, i.e. the height was restricted from the urban planning.
- Absence of data for the existing structure.

4.5 ST. JOBSVEEM

Introduction

St. Jobsveem is a particular building, given its construction time and original function. Pieters Bouwtechniek Engineering Consultancy was the only source of information. What follows is the outcome of an interview with Rob Doomen, structural engineer and project leader of this project, and of all the documentation that Pieters Bouwtechniek had in possession regarding the building.

Figure 4-23 St. Jobsveem building

General information

Table 4-5 St. Jobsveem project - General information

Location	Lloydkwartier, Rotterdam
Project	St. Jobsveem
Client	BAM Vastgoed, BAM Volker bouwmaatschappij
Former function	Warehouse
Current function	Residences │ Retail
Architectural design	Mei Architecten & Stedenbouwers
Structural design	Pieters Bouwtechniek Delft
Contractor	BAM Vastgoed, BAM Volker bouwmaatschappij
Originally built	1913
Started - Completed	June 2005 – April 2007

The success of renovating historic buildings depends on the manner in which original features are combined with the new ones. During the renovation of the St. Jobsveem, a monumental warehouse from 1913, in Rotterdam, the building itself owned still some technical possibilities, which were valuable assets and were used to upgrade it into a luxury apartment building. The original building was part of the original environment. However, the changed surroundings, and the change of function demands a new image, without affecting the value and strength of the building. An ambitious developer and a good architectural design have insured that extensive modifications were allowed to the monument. The result is a building that fits nice to its surroundings, where the age of nearly 100 years is but an added value.

Existing structure

Available Data. One of the main obstacles during the preliminary design process was the lack of available information about the existing building. There were barely any structural or architectural drawings, reports and calculations. Some years prior to the specific project, a student of Delft Technical University, Martijn Veltkamp, made an extensive research on this building, trying to determine the structural capacity of the existing columns. The outcome of this research and of the research that Pieters Bouwtechniek engineers conducted is a foundation plan, a typical floor plan and some sketches of the structure. Therefore, the time invested to investigate the actual state of the building exceeded the typical range and more tests had to be executed.

Existing Load Bearing System. The original warehouse is approximately 130 m long, 25 m wide and 30 m high. The structure consists out of structural columns, placed in a grid of 5 x 5 m^2, and load bearing façades around it. The columns are made out of cast iron and support the steel floor beams. The façades are made out of masonry and on the water side concrete frameworks support them. This extra reinforcement was necessary due to the cantilevers of the loading balconies, up to 3,5 m (Figure 4-31). The existing façades are closed and ensure for the stability of the building. The floors consist of timber joists bearing diagonally mounted floorboards of 36 mm thickness. The timber joist is resting on the steel floor beams. These steel beams are protected against fire from both sides with the use of timber beams and from the bottom with cement shell (Figure 4-24).

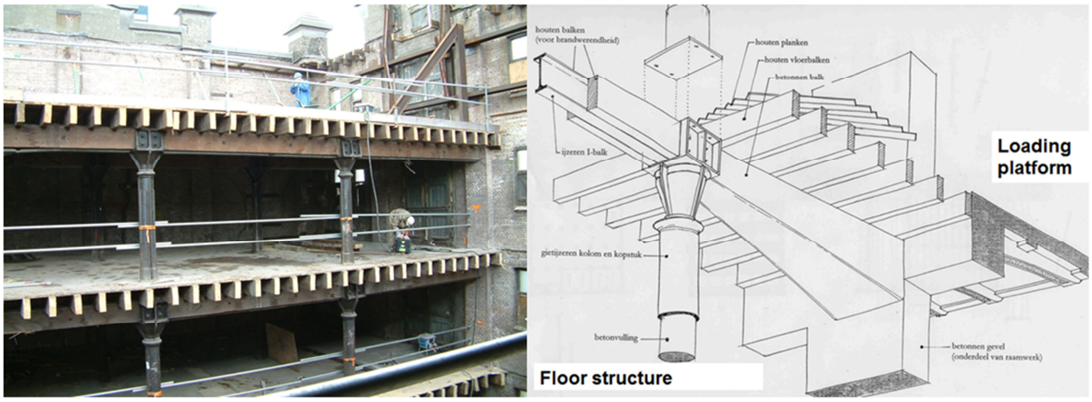


Figure 4-24 Jobsveem - (Left) Cross section of the building exposing the structure (Right) 3D sketch of the loading platforms (retrieved from the thesis of Martijn Veltkamp)

Existing Foundation. The building is founded on timber piles with concrete pile caps. The number and the position of the piles was known from the original foundation plan (Figure 4-25). The actual condition and the load bearing capacity of the piles was determined with the means of onsite tests. No other data was available such as CPT's and geotechnical surveys.

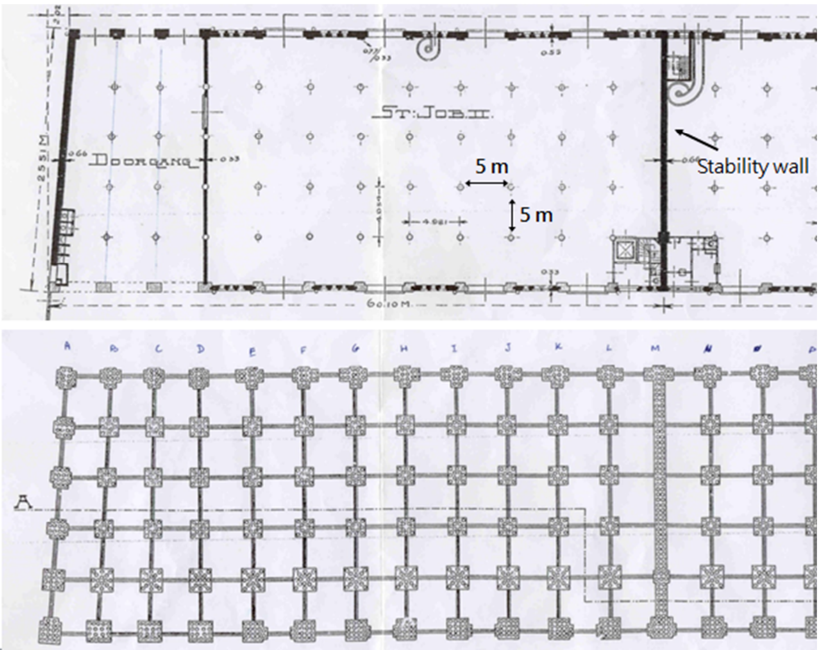


Figure 4-25 Jobsveem - Original foundation plan

Tests. Prior to the design of the building, Pieters Bouwtechniek engineers made visual and technical assessments of the existing structure. The overall technical condition of the building proved to be good. However, the visual inspection revealed at several locations cracks over the whole height of the building. These issues, as well as the residual load bearing capacity of the foundation, were examined first. The lack of information about the existing foundation and upper structure created the need for tests on several structural elements and parts of the building. As far as the foundation is concerned, it was first excavated, at certain locations, up to a depth of approximately 3 m and then from a certain amount of piles a segment was cut. At the position of this segment a jack was placed and by applying force in the pile and the residual capacity of the pile was estimated. This proved to be decisive for the overall loading. The allowable extra load for each pile was 145 kN and most of the pile caps had 12 to 14 piles. By digging at the position of the piles, a visual inspection was also possible so that to ensure the piles were not rotted. Moreover, cylindrical segments were taken from the concrete walls in order to test the quality and the strength of the concrete (Figure 4-26). The loading platforms have been tested as well onsite, by applying higher loads than the design loads, and also, by sloping parts of the concrete in order for the reinforcement to be revealed (Figure 4-26). The results of the loading tests showed that the load bearing capacity was increased in comparison with the design load.

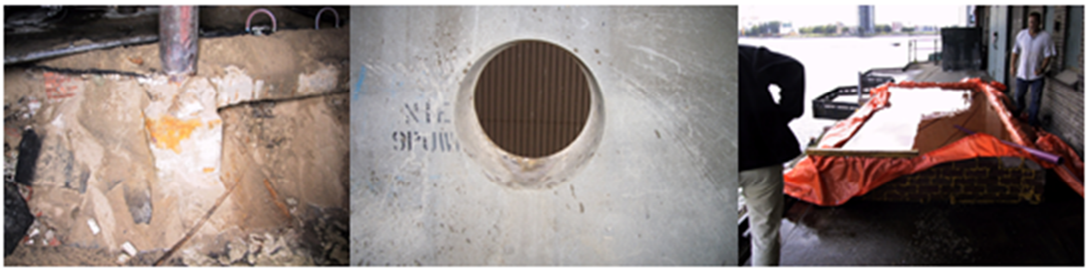


Figure 4-26 Jobsveem - Quality testing (Left) Digging and visual inspections (Mid) Concrete cylindrical specimens (Right) Load test loading platforms

As for the steel profiles of the structure, their dimensions have been measured onsite, by taking small samples of the cross sections the thickness was defined, and eventually a calculation of the respective strength was done. For the floor structure, the load bearing capacity of the steel beams proved to be normative whereas the capacity of the cast iron columns was not decisive. Furthermore, on the basis of the existing drawings, the cause of the cracking in the masonry walls was carefully examined. Thus, the load bearing capacities of all structural elements were known and the normative value was distinguished. The decisive structural element for the residual load bearing capacity of the whole structure was the foundation. Undoubtedly, during the later stages of the design and the construction as well, more loading tests were done and several parts of the structure were partly stripped.

Vertical extension

Load Bearing Capacity of the Existing Structure. Before the selection of the new structural system and the vertical extension, it was essential to determine the residual load bearing capacity of the existing structure. All the structural elements were carefully tested as previously described. The decisive elements for the floor structure were the steel beams, whereas, for the whole structure, the foundation piles. Floors can carry an extra load of approximately 900 kg/m^2 and each timber pile an extra load of 145 kN. This was converted directly to the possibility of adding 2 extra floors on top of the existing building. Nevertheless, the monumental identity of the building itself did not allow to exceed the height of the existing façade. Eventually, one floor was rebuilt and an extra floor was added (Figure 4-27). The existing roof was replaced with a new constructed light-weight floor and the rest of the residual capacity was used so that to improve the characteristics, such as sound insulation, of the existing floors, something that was required to do based on the new function of the building (Figure 4-27).

Structural Design of the New Block. The existing roof of the warehouse is demolished. At the same place, an extra floor is placed in order to accommodate the penthouses which are expected to increase the profit of the building. This extra floor has a light structure, consisting of steel frames and timber frames for the roof and the façade elements (Figure 4-28).

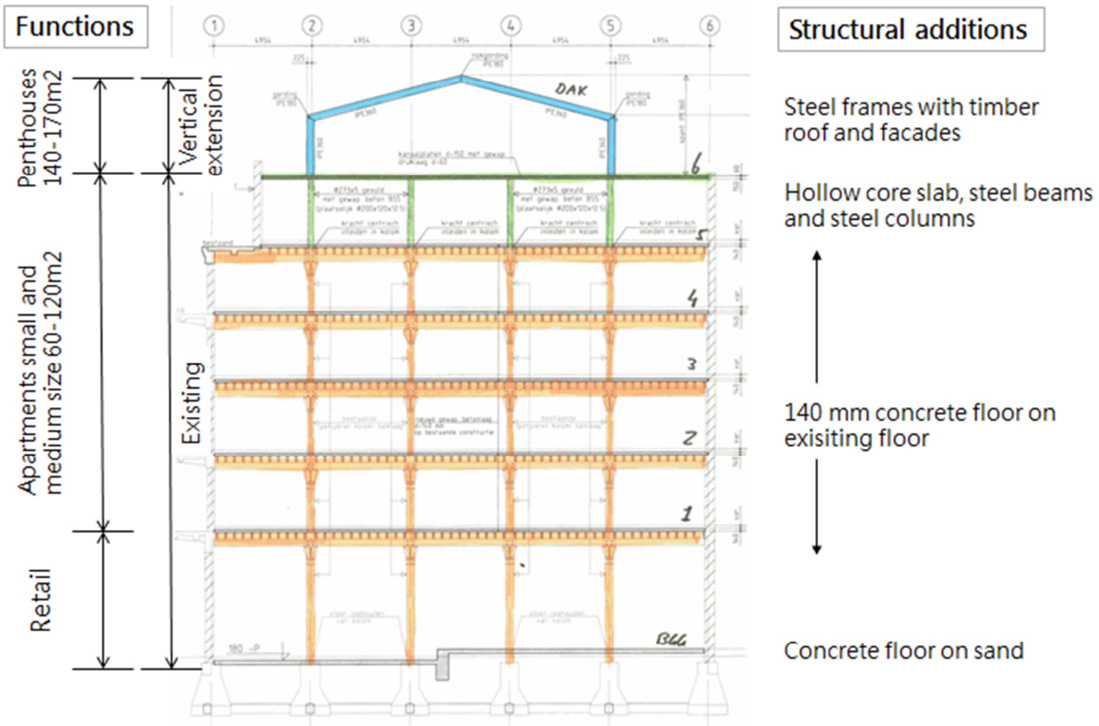


Figure 4-27 Jobsveem - Schematic representation of the new functions, both old and new parts of the building and the adequate structural provisions used to improve the building's performance

Figure 4-28 Jobsveem - Roof structure

The most important intervention though, is the creation of three new atria which cut literally the building in totally separate parts. This was necessary due to the daylight regulations for the apartments attached to the atria. The atria are approximately 9 m wide and the floors are removed over the whole height of the building. In this way, there have been created 3 locations for the entrances, through the atria, and the apartments are places in between the atria (Figure 4-29 and Figure 4-30).

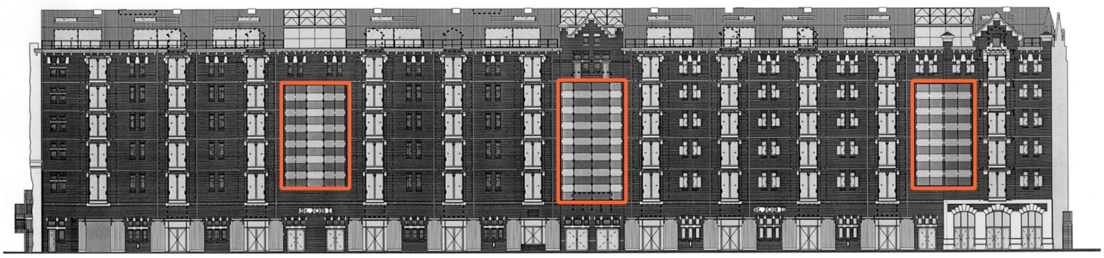


Figure 4-29 Jobsveem - Front view with the new atria

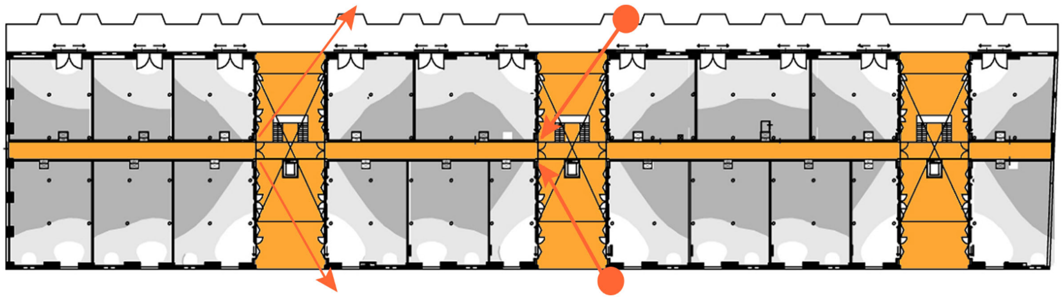


Figure 4-30 Jobsveem - Floor plan after the creation of the three atria due to daylight requirements

A lot of time and money were invested in order to make something special in the atria. The cantilever structures of the loading balconies are demolished and replaced by concrete beams with integrated steel profiles, that ensure the connection with the existing balconies and the stability frameworks (Figure 4-31). The recesses of the atria are covered with spectacular glass façades consisted of glass panes which are hanging on prestressed steel tendons. These tendons are 42 mm thick and 15 m long and are spanning in between the composite beams of 1,25 m height, on top and at the bottom. In the case of extreme wind the force in the tendons can reach the 220 kN, causing a deformation of more than 300 mm.

Figure 4-31 Jobsveem - Loading platforms structural alterations

Given the fact that the existing floors used to function as diaphragms for the stability of the whole building, one can realize that cutting the building in 4 individual masses created the need to stabilize these separate parts in the transverse direction (Figure 4-32). This is achieved by placing steel bracings at both sides of each atrium. In order to avoid steel bars running in front of the apartments' windows, the final choice is to create big frameworks with horizontal elements on each floor, using HE steel profiles (Figure 4-33). These steel structures are left exposed, emphasizing their robust character which refers directly to the harbor character of the surrounding area. For the aforementioned frameworks, HE1000A steel profiles are used for the ground floor and the first floor, and from the second floor up to the roof HE600A profiles. The nodes of the frameworks are welded in the factory and are connected to the concrete floors of each level through welded hairpins. Underneath these frameworks, new foundation piles are added and carry the forces to the ground.

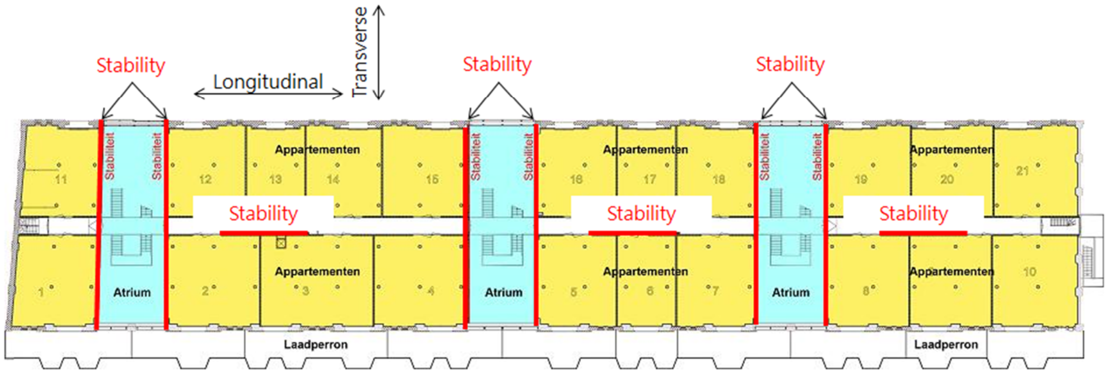


Figure 4-32 Jobsveem – Location of the stabilizing frameworks in the transverse and the longitudinal directions

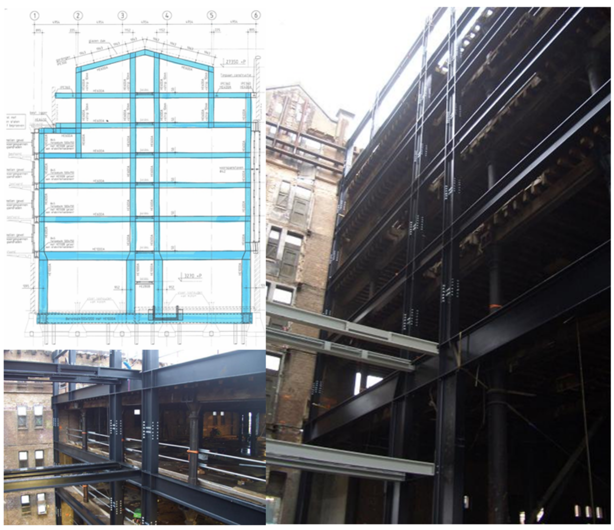


Figure 4-33 Jobsveem - Stabilizing system in the transverse direction

Figure 4-34 Jobsveem - Stabilizing system in teh longitudinal direction

Furthermore, a false floor is designed for above the existing floor, to accommodate the technical installations, and an extra layer of concrete (140 mm) on top of the installations so that to improve the acoustic characteristics of the existing floors (Figure 4-27).

Sustainability & construction costs in the design process

Sustainability came as a consequence of the renovation of the Jobsveem, considering the fact that the residual load bearing capacity of the existing structure was used at its outmost, with barely any structural provisions for the strengthening of the existing elements. The massive building was given another 50 years of life. Life Cycle Assessment was not used during the design. Pieters Bouwtechniek has introduced the LCA method the last 5 years in its projects, and is using it mostly after the request of one of the involved parties.

Construction costs played an important role in the decision making process. The first idea for the renovation of Jobsveem, before it had been listed as a monument, was to completely cover it with a bigger volume, adding up to 5 extra floors. This idea was considered to be very optimistic and totally uneconomical, and so was rejected by the owner. The monumental value of Jobsveem and the philosophy of having the optimal relation between the value of the final result and the construction costs, were the two parameters that affected generally the outcome of the renovation.

Design parameters

The amount of extra square meters added on top of the Jobsveem building have been defined by the residual load bearing of the existing structure itself. The owner of the building wanted to have the most profitable relation in between the final value of the extra square meters and the costs of adding these extra square meters. Eventually, it was decided not to apply strengthening measures for the existing structural elements, and consequently, only one extra floor came on top of Jobsveem. Additionally, the parameter which played the most important role was the fact that the building was a national monument and it was not allowed to significantly interfere to the image of the existing building. The extent to which the existing façade could be changed was very limited.

Therefore, the final option was not based on what was possible from a technical point of view; it was a combination of the different restrictions and particularities, the main of which were based on:

- The fact that the building was a national listed monument
- The best relation between costs and value for the owner of the building.
- Testing methods revealed the critical elements.

4.6 CONCLUSIONS

At first, it should be mentioned once more, that the sample of five case studies is not sufficient from the statistics point of view, in order to generalize the results and conclusions. Though, the selection of the projects was random, and considering that the amount of vertical extension projects in the Netherlands is limited compared to new buildings, the conclusions could be deliberated as useful information for future vertical extension projects.

In this chapter, five already completed vertical extensions case studies have been presented. The structural elements and systems of both the existing and new building have been analyzed, in an attempt to understand which were these factors during the design process that determined the amount of added square meters on top of the existing building. Undoubtedly, throughout the design stages of a project, a huge amount of different factors appear that affect, more or less, the decisions and the choices. However, there are some specific parameters that determine the boundaries of the design. These design parameters are briefly summarized in Table 4-6.

Reflecting on Table 4-6 one can notice that, in three out of five cases, municipal policies determined the decision about the vertical extension. It is in the opinion of the author, that this is a factor open to manipulation as well as negotiation. Vertical extension could be used as a solution for the big cities where only little, if not zero, unbuilt plots are available. The scientific research can help in this direction, by arguing about the benefits of this strategy. In any case, a different way of thinking and designing shall be adopted ahead the man-made climate change that has been unprecedented. Furthermore, the building's foundation played a decisive role in two out of five cases, which means that it is probably a factor to be considered for future projects. At last, in two out of the five case studies a feasibility study was carried out at the beginning of the project and revealed the most profitable option for the vertical extension regarding the economic considerations of the venture.

Table 4-6 Summary table of the design parameters that affected the amount of extra square meters added in the five vertical extension case studies

Building name	Design parameters
Karel Doorman	▪ Load bearing capacity of the foundation ▪ Risks taken; time and money invested for the light-weight solution ▪ Testing revealed higher concrete quality ▪ Alteration of structural system
Groot Willemsplein	▪ Timeframe restrictions ▪ Municipal policy; restrictions from the urban planning regulations (m^2 office area)
Westerlaantoren	▪ Type of existing foundation (2 m thick plate) ▪ Municipal policy; restrictions from the urban planning regulations (height of the building) ▪ Testing revealed higher concrete quality ▪ Feasibility study
Zeemanshuis	▪ Municipal policy; restrictions from the urban planning regulations (height of the building) ▪ Absence of data for the existing structure
St. Jobsveem	▪ National listed monument ▪ Feasibility study ▪ Testing revealed critical elements

To sum up, the aforementioned design parameters deviate significantly from each other. This leads to the conclusion that the special features of every project affect in a different way the project itself. The location and the building are certainly of great importance, but imponderables are always present throughout a design process. Effort should be directed towards solutions less invasive and hazardous for the environment and the society as a whole.

The field of vertical extension projects, from the academic and scientific point of view, is still to be explored. Further research should be directed towards vertical extension as a strategy to redevelop existing buildings, proving statistically and scientifically the benefits of this strategy, and suggesting methods and techniques to optimize it.

5 Life Cycle Assessment

5.1 Background

Undeniably, the construction industry is one of the most energy-consuming trades. A critical factor is the effect of the required activities on the exhaustion of natural resources. Construction output is largely tangible and, hence, calls for significant extraction of raw materials. Besides, the procedures produce noteworthy greenhouse gas emissions either directly through on-site construction, or indirectly from manufacturing and transporting the necessary materials (G.K.C.Ding, 2014). Moreover, it should be noted that the operation and the demolition phase of a construction project should be, also, considered as substantially energy wasting (Santori & Hestnes, 2007). Therefore, research has turned into making the industry more eco-friendly, in order to reduce its environmental impact, regardless of the undertaken project.

In a same vein, the novelist and green business guru John Elkington has convincingly argued that future market success will often depend upon a company's ability to satisfy the three-pronged fork of profitability (Profit), environmental quality (Planet), and social justice (People) (Elkington, 1997). Life Cycle Assessment (LCA) is an approach which addresses the environmental aspects and impacts of a product system (ISO 14040:2006).

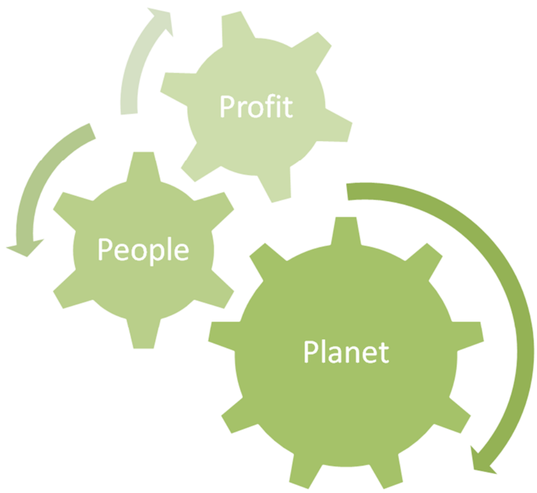


Figure 5-1 Triple bottom line: The 3 Pillars (3 P's)

5.2 LIFE CYCLE ASSESSMENT IN CONSTRUCTION

Definitions

In that scope, during the last decades Life Cycle Assessment (LCA) has fostered, in response to the need for a holistic approach to improving the energy efficiency of construction industry. In essence, LCA is a system that conducts continuous analysis of environmental performance during the complete life cycle of a project (Ciambrone, 1997). It was not until late 1990s that a standard framework was established, by the International Standards Organization (ISO) (ISO 14040:2006). According to International Standard ISO 14040, LCA is a 'compilation and evaluation of the inputs, outputs and the potential environmental impacts of a product system throughout its life cycle'. The Code of Practice by the Society of Environmental Toxicology and Chemistry describes LCA as 'a process to evaluate the environmental burdens associated with a product, process, or activity by identifying and quantifying energy and materials used and wastes released to the environment; to assess the impact of those energy and materials used and releases to the environment; and to identify and evaluate opportunities to affect environmental improvements' (SETAC , 1993).

Building Life-Cycle Stages

Every product or process goes through a variety of stages in its lifetime. Each stage is composed of a number of activities. For industrial products, these stages can be broadly defined as material acquisition, manufacturing, use and maintenance, and end-of-life. In case of buildings, these stages are more fully delineated as: materials manufacturing, construction, use and maintenance, and end of life (Bayer C. , Gamble, Gentry, & Joshi, 2010) (Figure 5-2).

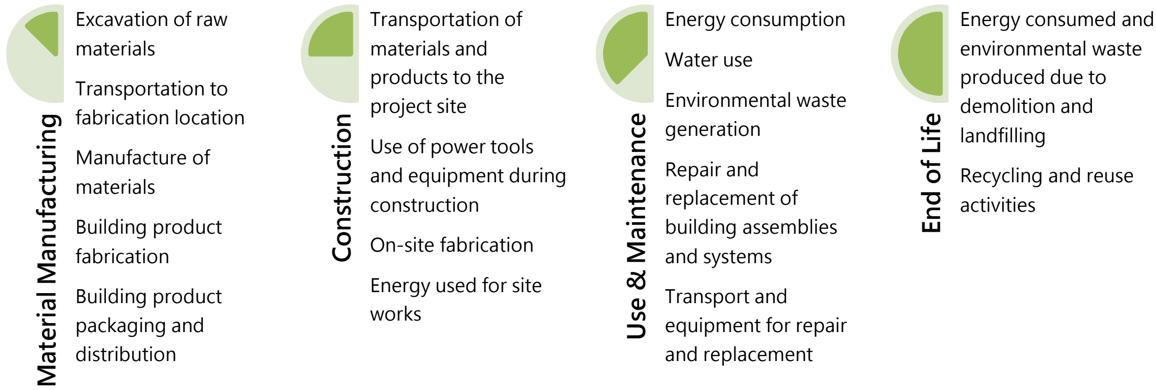


Figure 5-2 Life-cycle stages of a building process

Steps of the LCA process

LCA contains four fundamental steps (Figure 5-3), which are repeated throughout the life-cycle of the project: the goal and scope definition, the life cycle inventory analysis (LCI), the life cycle impact assessment (LCIA), and, ultimately, the life cycle interpretation (ISO 14040:2006).

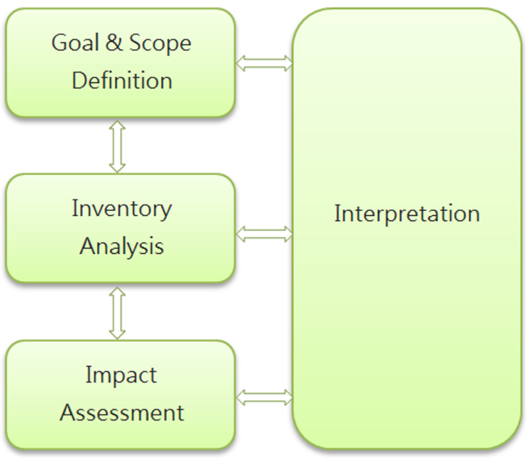


Figure 5-3 Life Cycle Assessment framework according to ISO 14040

The first stage consists of conceptual strategic planning of the life cycle assessment, data collection methodology, and identification of system limitations. The second stage comprises data collection, management, and analysis, as well as quantification of the energy flows produced from the necessary for the completion of the project procedures and materials. During the life cycle impact assessment stage the results are weighted, normalized, and classified in order to evaluate the environmental impact of the project.

Finally, LCA includes a decisive stage, which improves and assesses the results of the previous two stages, recommends specific actions for enhancement of the structure's energy efficiency, and provides with feedback for future reference (Bauman & Tillman, 2004; Cabeza, Rincóna, Vilariño, Pérez, & Castell, 2014).

The aforementioned stages constitute a sophisticated system that aims to assist in the decision making process, since achieving higher levels of sustainability is a top priority for every construction project (Sharma, Saxena, Sethi, Shree, & Varun, 2011; Singh, Berghorn, Joshi, & Syal, 2011). To accomplish so, LCA quantifies the energy consumption of the construction product, in an attempt to improve its performance (Bayer C. , Gamble, Gentry, & Joshi, 2010).

5.3 LCA ON MEASUREMENT OF ENVIRONMENTAL IMPACT

The Netherlands

LCI data are the heart of any LCA analysis. These data are region-specific because the fuel mix and the methods of production often differ from region to region. LCI data for building materials produced and used in the Netherlands in particular, are registered in the National Environmental Database (in Dutch: 'de Nationale Milieudatabase'). This specific national database is managed by the Dutch Construction Quality Association (in Dutch: 'Stichting Bouwkwaliteit - SBK').

Environmental Impact Effect Categories

During the different stages and processes of a building's life-cycle, valuable resources are wasted and harmful environmental pollutions are produced, affecting in a negative way the biotic and abiotic environment. These negative effects have been gathered and categorized under some main groups named 'environmental impact effect categories', which are involved in the LCA methodology. A number of the aforementioned categories are listed in Table 5-1, presenting what type of effects the production and application of both construction materials and civil engineering constructions can have on the environment.

Quantification of the Environmental Impact

Being aware of the main negative effects, of the variety of processes during a building's life-cycle, on the environment, the need to quantify these effects arises, as a means to manage and, eventually, minimize them. One of the methods developed for the quantification of the environmental impact is expressing it in real costs, i.e. in a single monetary value. This way of expressing the environmental impact gives an idea of how much a structure or process costs to society if the damage to the environmental is not included in the sales price (considered as external costs). Such costs will gradually become internal costs as a consequence of governmental regulations (Figure 5-4). This is consistent with the 'polluter-pays' principle which is the commonly accepted practice that those who produce pollution should bear the costs of managing it to prevent damage to human health or the environment.

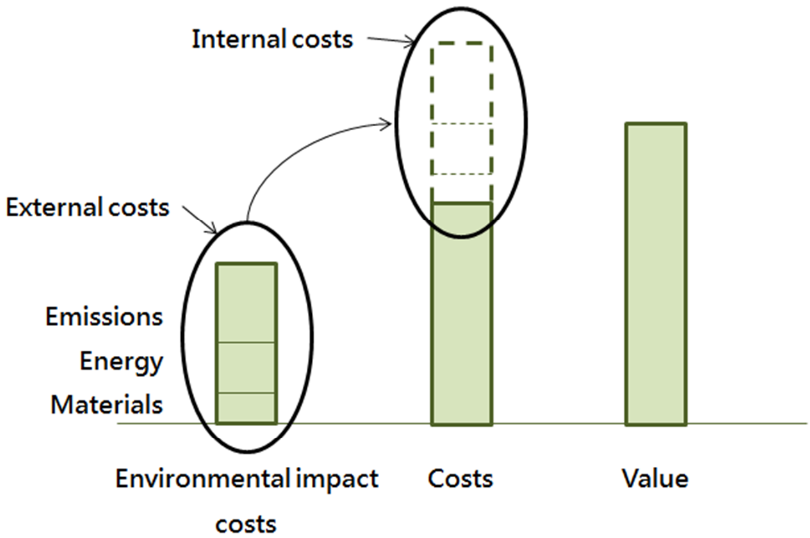


Figure 5-4 Schematization of the environmental impact costs as external and internal costs in the total costs of a building

In the Netherlands, the Dutch Building Decree 2012 (in Dutch: 'Bouwbesluit 2012') prescribes that all houses and office buildings built after the 1st January 2013, with a total user surface larger than 100 m^2, must include an environmental impact calculation (in Dutch: 'Milieuprestatieberekening'). The method to be followed for the calculation of the environmental impact of buildings and civil engineering works is set in an extensive report (in Dutch: 'Bepaling van de Milieuprestaties van Gebouwen en GWW-werken - MPG') by the Dutch Construction Quality Association (in Dutch: 'Stichting Bouwkwaliteit - SBK'), always in accordance with the European standards EN15804 and EN 15978. In Figure 5-5 is found the sequence of the different methodologies and instruments used in order to result to the environmental impact calculation. The single monetary value which comes as a result of the aforementioned method and is representing the environmental costs of a particular process is named Environmental Cost Indicator – ECI-value (in Dutch: 'Milieu Kosten Indicator' – MKI-waarde). In the calculation of the ECI-value only the 11 main environmental impact effect categories, and their respective weight factors, are taken into account while the 5 additional categories are described separately (in m3, kg, or MJ equivalents) (Table 5-1). The unit of the ECI-value is the Euro (€), whereas the result of the calculation itself is mentioned frequently as shadow prices (in Dutch: 'schaduwprijzen').

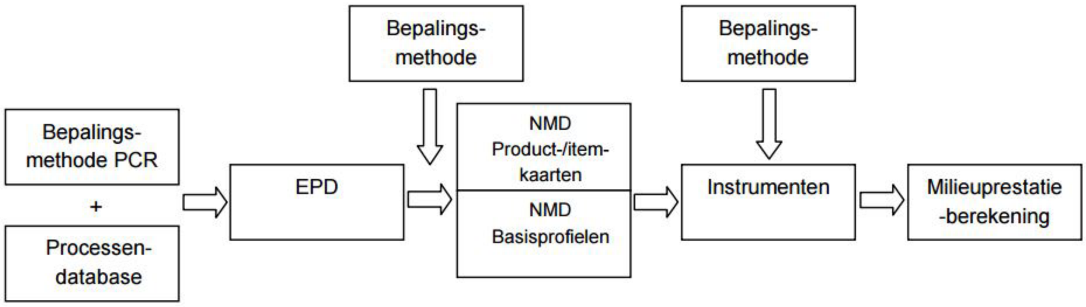


Figure 5-5 Determination method, Environmental Product Declaration, National Environmental Database and tools for the purpose of Environmental Impact Calculation (Stichting Bouwkwaliteit, 2014)

Table 5-1 List of environmental impact effect categories used for the calculation of the ECI-value and their equivalent units

N°	Environmental Impact Effect Category	Abbreviation	Unit*	Weight factor (€/kg equiv.)
1	Global Warming Potential	GWP100	Kg CO_2 equiv.	0,05
2	Ozone layer Depletion Potential	ODP	Kg CFC 11 equiv.	30
3	Human Toxicity Potential	HTP	kg 1,4-DCB equiv.	0,09
4	Freshwater Aquatic Eco-Toxicity Potential	FAETP	kg 1,4-DCB equiv.	0,03
5	Marine Aquatic Eco-Toxicity Potential	MAETP	kg 1,4-DCB equiv.	0,0001
6	Terrestrial Eco-Toxicity Potential	TETP	kg 1,4-DCB equiv.	0,06
7	Photochemical Oxidation Potential	POCP	kg Ethene equiv.	2
8	Acidification Potential for soil and water	AP	kg SO_2 equiv.	4
9	Eutrophication Potential	EP	kg $(PO_4)^{3-}$ equiv.	9
10	Abiotic Depletion Potential for non-fossil resources	ADP-elements	kg Sb equiv.	0,16
11	Abiotic Depletion Potential for fossil fuels	ADP-fossil fuels	MJ, net calorific value	0,16
12	Water use		m^3	
13	Dangerous waste		kg	
14	Non dangerous waste		kg	
15	Total non-renewable energy		MJ	
16	Total renewable energy		MJ	

** Expressed per functional or declared unit*

From 'cradle' to 'grave'

LCA is a process used to determine the environmental impact from 'cradle' to 'grave' of a product or service. 'Cradle' stands for the first stage of a building life-cycle (Figure 5-2 and Figure 5-6), the manufacturing of materials, and 'grave' for the end-of-life stage.

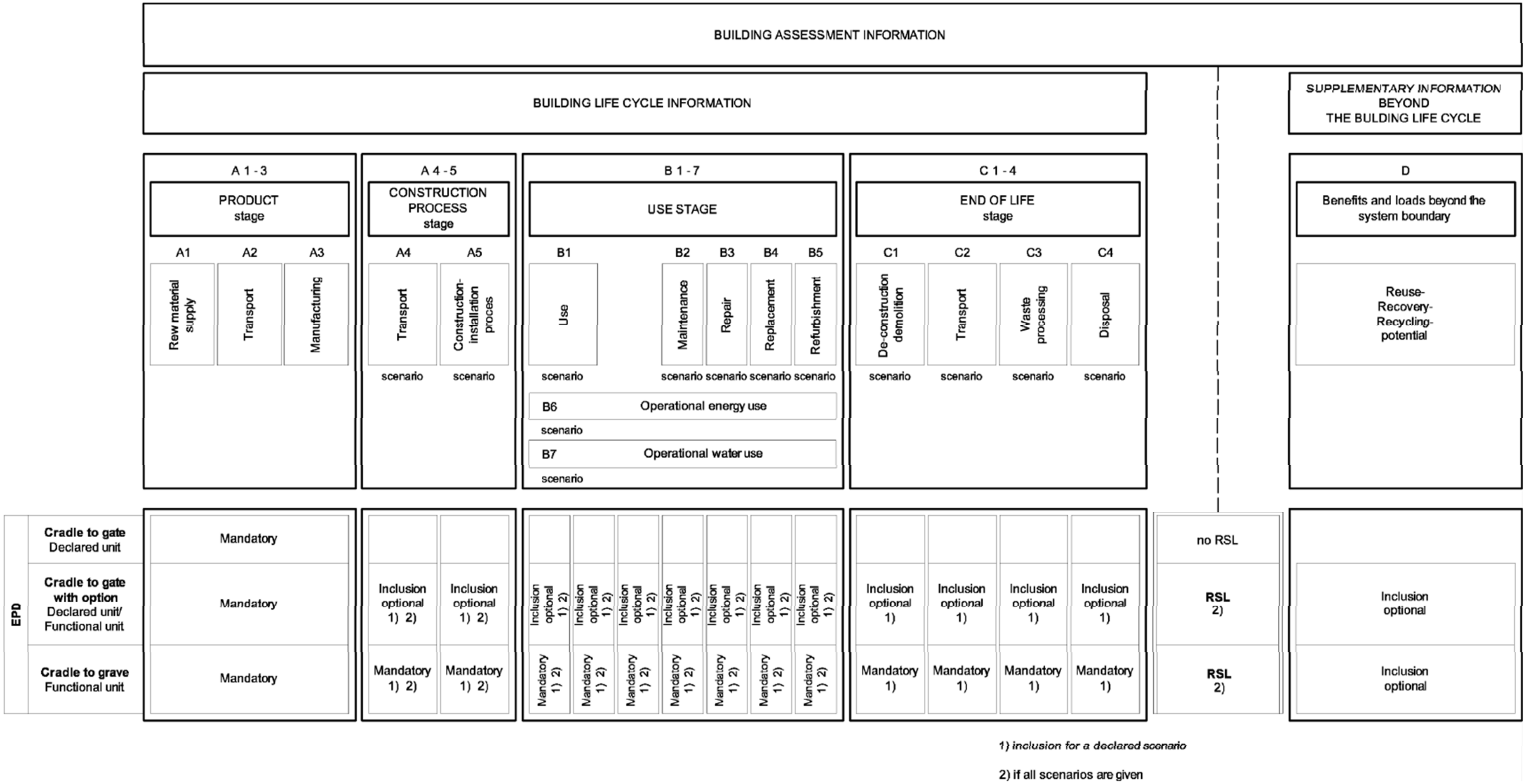


Figure 5-6 Types of EPD with respect to life cycle stages covered and life cycle stages and modules for the building assessment

5.4 TOOLS MEASURING ENVIRONMENTAL IMPACT

As described in the report 'Bepaling van de Milieuprestaties van Gebouwen en GWW-werken (MPG) - Geactualiseerde versie 2015' published by the Dutch Construction Quality Association, the validated tools, in the Dutch market, for calculating the environmental performance of buildings and civil engineering works are the following:

For civil engineering works:
- DuboCalc (Rijkswaterstaat)

For buildings:
- GPR Gebouw en GPR Bouwbesluit (W/E adviseurs)
- DGBC-tool (Dutch Green Building Council)
- MRPI MPG-software (MRPI)
- DUBOkeur® (NIBE)
- DuCo-tool (IMd Raadgevende Ingenieurs & Bouwen met Staal)

5.5 LIMITATIONS AND FUTURE DEVELOPMENT

Life Cycle Assessment models are inherently interwoven with some generic limitations and some additional contributing factors that render LCA quite challenging. Firstly, any model is limited to be as efficient as its input data, and in the case of energy flows throughout the life cycle of a project, the quantification of energy flows is a significantly complicated task (Cabeza, Rincóna, Vilariño, Pérez, & Castell, 2014; Singh, Berghorn, Joshi, & Syal, 2011). Besides, the necessary fundamental assumptions regarding the projected lifespan, and some additional subjective factors enhance its intricacy. Moreover, each project requires site-specific treatment, in order to take into consideration various local and, possibly, unique parameters (Kohler & Moffatt, Life-cycle analysis of the built environment, 2003). Another element that challenges LCA consistency is the various diverse scenarios regarding the operation stage of the project, i.e. a residential building may be renovated to non-residential. Furthermore, supplementary complexity is added to the model from the insertion of recycled materials with unknown properties. Finally, LCA needs to integrate socio-economic impacts as well, in order to provide the decision makers with more consistent and integrated recommendations (G.K.C.Ding, 2014).

In response to these limitations researchers suggest using a dynamic LCA that is more responsive to user preferences, and more adaptive to systemic alterations (Erlandsson & Borg, 2003; Collinge, Landis, Jones, Schaefer, & Bilec, 2013). Conclusively, LCA is still in its developmental stage (Singh, Berghorn, Joshi, & Syal, 2011), regarding the construction industry, but as the literature suggests, it is already expected to enable the numerous stakeholders to accomplish adequate environmental sustainability, along with operational efficiency and user satisfaction (Hellweg, Demou, Scheringer, McKone, & Hungerbühler, 2005; Assefa, et al., 2007).

5.6 CONCLUSIONS

In this chapter an overview is given of the state-of-art in the Netherlands regarding the LCS on measurement of the environmental impact. These information are meant to be used in the design of the case study of Astoria building in the following chapters.

Astoria case study

6 STRUCTURAL ANALYSIS

6.1 INTRODUCTION

6.1.1 Motivation

An empty building originally built in 1983, right opposite of a centrally located railway station in the Hague, is undoubtedly a great opportunity for a real estate company. The remaining service life of the building and its good location worth an attempt to research the possibilities for reusing it. From the financial perspective, the higher the gross floor area (GFA) a centrally located building has, the higher the possible revenue. From the sustainability perspective, reusing an existing structure that has not reached its end-of-life, is consistent with the European policies and the trend of the construction industry. Consequently, the possibilities for vertical extension will be investigated, with the purpose of revealing the parameters that influence the final decision for the amount of extra storeys built on top of the existing building.

6.1.2 Assessment objectives & strategy

At the outset, the objectives of the project shall be clearly specified as well as the goals of the assessment of the existing building. The particular project deals with the transformation of an old and empty office building into a residential building, investigating at the same time the possibilities for vertical extension. The purpose of the current thesis is to investigate the design parameters that determine the optimal vertical extension in terms of costs and environmental impact.

The strategy set at the beginning of the design phase is influenced in a great extent by the example case studies analyzed in the literature review and the successful techniques applied there. The process of optimizing the amount of extra square meters is based on the following approaches:

- Functionality of the existing and new part of the building with the aim of attaining a positive impact on the vertical extension.
- Analysis of the existing load bearing elements and estimation of the residual load bearing capacities, focusing on the critical elements.
- Use of light-weight solution for the structure of the new block on top (steel structure and light-weight floor system).
- Optimization of the structural design of the new block, for the best distribution of the extra load over the existing elements.

- Application of the NEN 8700, where lower partial load factors are applied, in order to, possibly, extent the margins of the overcapacity. NEN 8700 will be introduced when calculations according to NEN 1990 reach the limits.

6.1.3 Scenarios

The design phase will be composed of different scenarios. Starting point is the existing structure and its load bearing capacity in the current state (Scenario 0). Scenario 1 includes the vertical extension without any strengthening of the critical structural elements. Scenario 2 consists of vertical extension with the minimum of structural strengthening. The idea here is to try to distribute the loads more evenly over the existing foundation and this case does not include severe interventions and works. Scenario 3 contains extensive works in order to strengthen the existing structural elements and reach the maximum of vertical extension. The four scenarios are schematically presented in Figure 6-1. Furthermore, construction costs and shadow costs calculations will be performed for all scenarios. The results of these calculations will be compared in order to draw conclusions for the impact of the extra levels added, on the costs and the shadow costs.

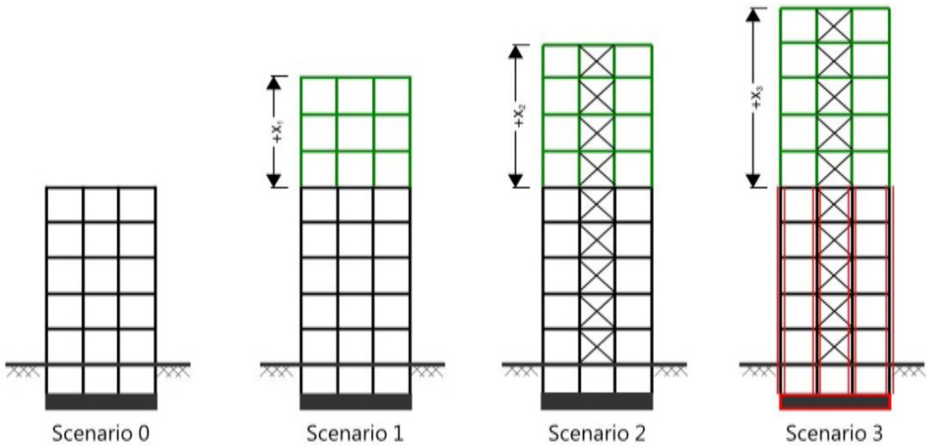


Figure 6-1 Schematic representation of the four design scenarios "Reuse & Vertical extension"

6.1.4 Assessment levels

In section 3.3, the assessment levels regarding an existing structure are presented. In particular, the assessment of the particular structure will be performed in two levels:

Level 1 : Linear static analysis employing basic structural models

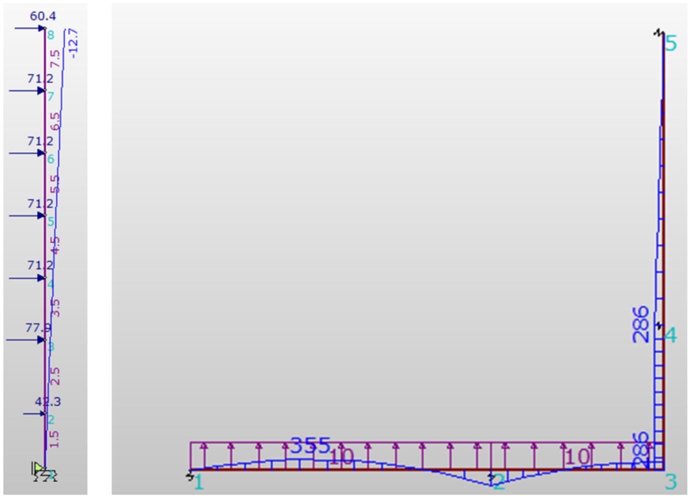


Figure 6-2 Structural analysis based on basic structural models

Level 2 : Non-linear static analysis using refined model (Finite Element Method)

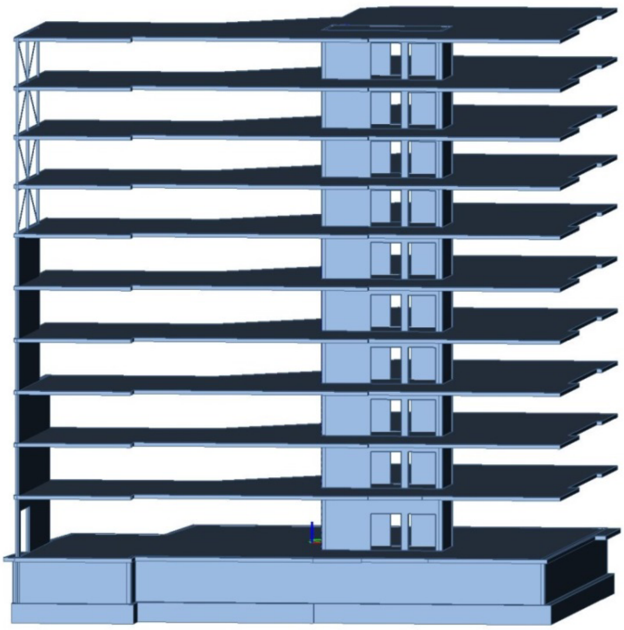


Figure 6-3 Structural analysis based on Finite Element Model (FEM)

The transition from the first level assessment to the upper level will be done in case the former assessment fails. For both assessment levels, the data for the determination of the load effects are acquired from the existing documents and the current standards (Eurocode), and the verification of the results is semi-probabilistic, using the partial safety factors.

6.1.5 Boundary conditions

Considering this study is conducted in the frame of a master thesis, and taking into account the time restrictions, boundary conditions are set and simplifications are made. As follows, the research focuses on specific aspects of the design providing the opportunity to study these aspects more in depth. The boundary conditions are formed as follows:

- The research focuses exclusively on the structural skeleton of the existing building.
- The building is stripped down to its structural skeleton and the bare existing structure is the starting point of the design.
- The maximum allowable height of this particular building, according to the zoning plan of the municipality of the Hague, is 26 m. This limitation will be neglected, in the attempt to overcome outdated formalities and policies that hinder possibly more sustainable solutions. However, it is concluded from the very beginning of the design phase that municipal policies is one of the main design parameters that affect the amount of extra square meters added on top of a building, as verified by the literature review.
- In the same direction, the restrictions related to the minimum number of parking spaces the building should provide is not being researched.
- The vertical transportation is certainly an aspect to look at when dealing with a vertical extension project. In this thesis, no attention is paid to this aspect.
- The existing and new parts of the building will function as student accommodation, i.e. residential use, except for the ground floor which will be used for commercial purposes. Residential use has lower variable loads and that works in favor of vertical extension.

6.2 STARTING POINTS EXISTING & NEW STRUCTURE

In the current section, the starting points related to the properties of the structural materials (existing and new), as well as the various boundary conditions set by the Eurocode, are set.

Eurocode		
Reliability Class	RC2	
Consequence Class	CC2	
Design Service Life Class	3	50 years
Functional Class	A	residential
Fire safety	120 min.	h>13m, no reduction possible (residential)
Material properties		
In-situ concrete	B22.5	existing structure
	C30/37	new structure
Steel	S235	new columns, beams, floor
	FeB400	reinforcement existing structure
	FeB500	reinforcement new structure

6.3 Existing building

6.3.1 General information

Originally built in 1983, 'Astoria' is a former office building that has been standing vacant for the past years. The building is located in the Hague, opposite to Den Haag Holland Spoor railway station, at the crossroads of the Stationsplein and the Stationsweg streets (Figure 6-4). *Geste Groep*, a real estate company, took the initiative in order to give this building a second chance. Primary, the idea is to convert the old office building into a residential building for student accommodation, and to vertically extend it, to create extra exploitable space. *IMd Raadgevende Ingenieurs*, is commissioned, for the first time in 1996, with the purpose of researching the possibilities for vertical extension. In this design case study the goal is to define the optimal amount of the extra square meters that could be added on top of the existing building, with regard to the construction costs and the environmental impact of the materials used, in terms of the shadow price.

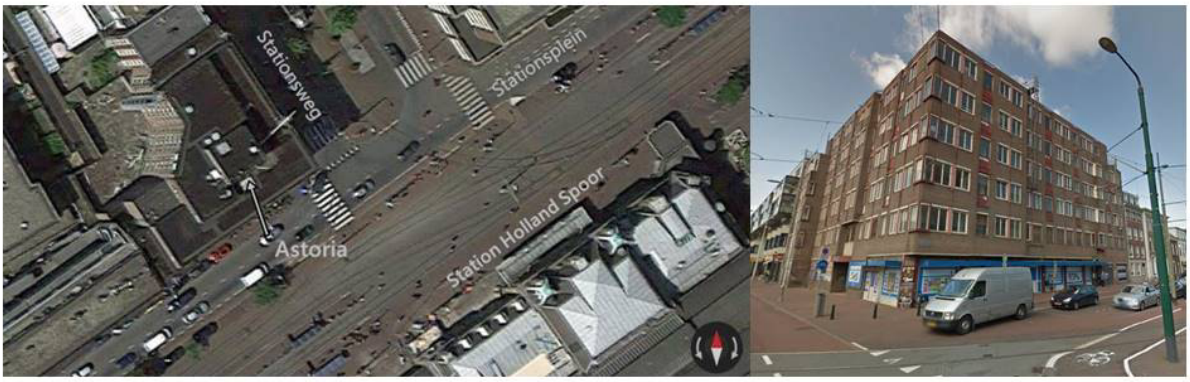


Figure 6-4 Project location and street view current state

6.3.2 Existing structure

Available data

The original structure of the building was been designed and calculated by IMd Raadgevende Ingenieurs in 1983, at that time called Ingenieursbureau Molenbroek, and therefore, all the structural drawings and technical, structural and geotechnical, reports were directly available in the archive of the company. A full list of these drawings and reports can be found in the Appendix B.1.

The information found on the available documents and drawings is the starting point for the analysis of the existing structural system. Concrete and reinforcement steel quality, compressive and tensile strength, concrete cover for the various elements, reinforcement bars are some of the basic properties of the existing structure.

Existing load bearing system

The load bearing system is formed by cast in-situ concrete, except for the precast concrete lintels. The plot of the building is approximately 34m x 31m, it consists of a basement, 7 floors on top of it and a roof, with a total height of 25,4m. The floor slabs act as diaphragms and transfer the lateral loads to a shear core and two shear walls (one per direction), that provide the lateral stability to the building. The gravity loads are carried by the floors and transferred directly to the columns, and thereafter to the foundation plate. The main axis grid is 7,2 x 7,2 meters. There are two main categories of columns, one along the façades (outside columns) and a second category spread over the main layout (inside columns). The outside columns are 300 x 500 mm and are set every 3,6 m. The inside columns are placed every 7,2 m and are ⧄500 mm at the overall floors, ⧄550mm at the ground floor, and Ø650 mm at the basement. The large inside columns have been designed with column heads so as to avoid punching shear failure. The concrete quality according to the structural drawings is B22.5, which is compared to C16/20 concrete quality of nowadays . The steel quality of the reinforcement bars used is FeB400 and FeB500.

Figure 6-5 Floor plans ground and first floor - Current state

Existing foundation & underground condition

The geotechnical report of *Osiris-Cesco Surveys and Site Investigations* in 1981, revealed a medium-dense to dense sand layer from approximately 2,0 to 2,5 m up to the depth of 12 to 13 m. The foundation of the building on a concrete plate was combined with soil improvement (drainage and soil compaction), in order to have the minimum settlements and differential settlements. The bottom of the foundation floor is placed at 4.0-N.A.P. (Normaal Amsterdams Peil or Amsterdam Ordnance Datum) and has a thickness of 1,2 m. The shape properties and dimensions of the reinforcement bars can be found in the corresponding drawing 210W.

Geomet Advies was commissioned in 2009 to perform more cone penetration tests (CPTs) due to the forthcoming vertical extension. Based on the results of these tests, Geomet composed a technical report in which a methodology is suggested for the calculation of underground's pressure on the foundation plate depending on the design value of the vertical load (Appendix B.2). This methodology will be used in order to estimate the residual load bearing capacity of the existing foundation.

6.3.3 Overview old and new norms & standards

The existing building has been designed around 1983 according to the old Dutch norm NEN 3850 "Technische grondslagen voor de berekening van bouwconstructies – TGB 1972" and NEN 2880 "Voorschiften Beton VB 1974" . There are many differences in between the design standards and the current ones regarding the safety factors, the vertical and horizontal actions on the structure and the computation methodologies. These differences can explain deviations from the former calculations. In Table 6-1 the differences in the safety factors and the computation of the variable actions amongst the norms are presented and in Table 6-2 an example calculation is presented just to emphasize the consequences of the norms on the unity checks of the structural elements.

Table 6-1 Differences between NEN3850 and the Eurocode related to variable loads and safety factors

Variable load percentages according to NEN 3850		
Roof	100%	
Top floor	100%	
Second upper floor	90%	
Third upper floor	80%	
Forth upper floor	70%	
Fifth upper floor	60%	
Sixth upper floor	50%	
All other floors	40%	
Variable load factor according to Eurocode		
ψ	0,5 (residential function)	
	The factor is applied to the variable loads of all floor levels except the roof. For the roof the extreme value of the variable load is applied (ψ=1)	
Partial load safety factors according to NEN 3850		
Concrete structures	γ=1.7*	
Steel structures	γ=1.5*	
Ground	γ=1.0*	
* The safety factors were applied to permanent and variable loads.		
Partial load safety factors according to NEN-EN 1990		
Permanent loads	γ=1.35	Load combination 6.10a
	γ=1.20	Load combination 6.10b
Variable loads	γ=1.50	Load combination 6.10a
	γ=1.50	Load combination 6.10b

Partial load safety factors according to NEN 8700		
Permanent loads	γ=1.20	Load combination 6.10a
	γ=1.15	Load combination 6.10b
Variable loads	γ=1.30	Load combination 6.10a
	γ=1.30	Load combination 6.10b
Material safety factors according to Eurocode		
Concrete structures	γ=1.50	
Steel structures	γ=1.15	

Table 6-2 Unity checks comparison amongst NEN 3850, NEN-EN 1990 and NEN 8700 for column K5

	Permanent	Variable NEN 3850	Variable Eurocode
Representative loads (kN)	3082	934	1200
	NEN 3850	**NEN-EN 1990**	**NEN 8700**
Design loads	=1.70*(3082+934)= 6827 kN	=1.2*3082+1.5*1200= 5498 kN	=1.15*3082+1.3*1200= 5104 kN
Concrete safety factor	1.7	1.5	1.5
Ultimate bearing capacity	7028 kN	7368 kN	7368 kN
Unity check	=6827/7028= **0.97**	=5498/7368= **0.75**	=5104/7368= **0.70**

6.3.4 Load bearing capacity of the existing structure

Introduction

The analysis and investigation of the existing structure and its various structural properties is the starting point of a vertical extension. This is achieved, in the normal practice, using both calculations and testing methods. In this design case study, no destructive or non-destructive methods will be applied. This results to an estimation of the load bearing capacity of the existing structure exclusively by means of calculations.

In order to make the process more efficient, the focus will be put on these structural elements that will be critical during the vertical extension, i.e. the elements that are mostly affected from the extra gravity and lateral loads. Analyzing the structural system of the building, one can distinguish the main structural elements that contribute to the smooth transfer of gravity and lateral loads to the foundation. These are the: floor slabs, columns, stability walls and the shear core. The process of adding extra floors on top of the existing building does not influence the concrete floor slabs, so these will be excluded from the critical elements. In any case, changing the function of the building from office to residential use, lowers the variable loads and that has a positive effect.

In the following the basic steps and the results of the analysis of the existing structure are presented. The detailed calculations are fully included in the Appendix B for the sake of comprehensiveness.

Basic structural assessment

In this first level of assessment, the calculations are based on basic structural models, i.e. mechanical schemes that represent the main structural members. The structure of the building is checked in terms of strength and stability for the load combinations:

- ULS_1 =1.35*Perm.Loads +1.50*Var.Loads*ψ_0 +1.50*Wind
- ULS_2 =1.20*Perm.Loads +1.50*Var.Loads +1.50*Wind (normative)
- ULS_3 =0.90*Perm.Loads +1.50*Wind
- SLS =1.00*Perm.Loads +1.00*Var.Loads +1,00*Wind

In Figure 6-6 an overview of the checks that have been performed per element is presented. The unity checks highlight the critical elements, that will fail first during the vertical extension and the increase of vertical and lateral loads.

Columns → Compression
→ Punching shear
Basement wall → Compression
→ Tension (Stability walls)
Foundation → Bending reinforcement
→ Punching shear

Unity checks closer to 1,0
↓
Critical elements

Figure 6-6 Performed checks of structural elements

The concrete columns have been modelled as pinned in both ends (Figure 6-7) in order to check the compressive strength and the applied reinforcement.

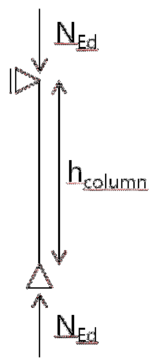


Figure 6-7 Basic structural schematization concrete columns

The structural model used for the analysis of the stability wall and the concrete core, is a bar, that has the adequate stiffness of the structural element it represents and is supported by a spring (Figure 6-6). The spring support represents the elastic foundation. For the complete methodology and the elaborate version of the aforementioned calculations see Appendix B.8.

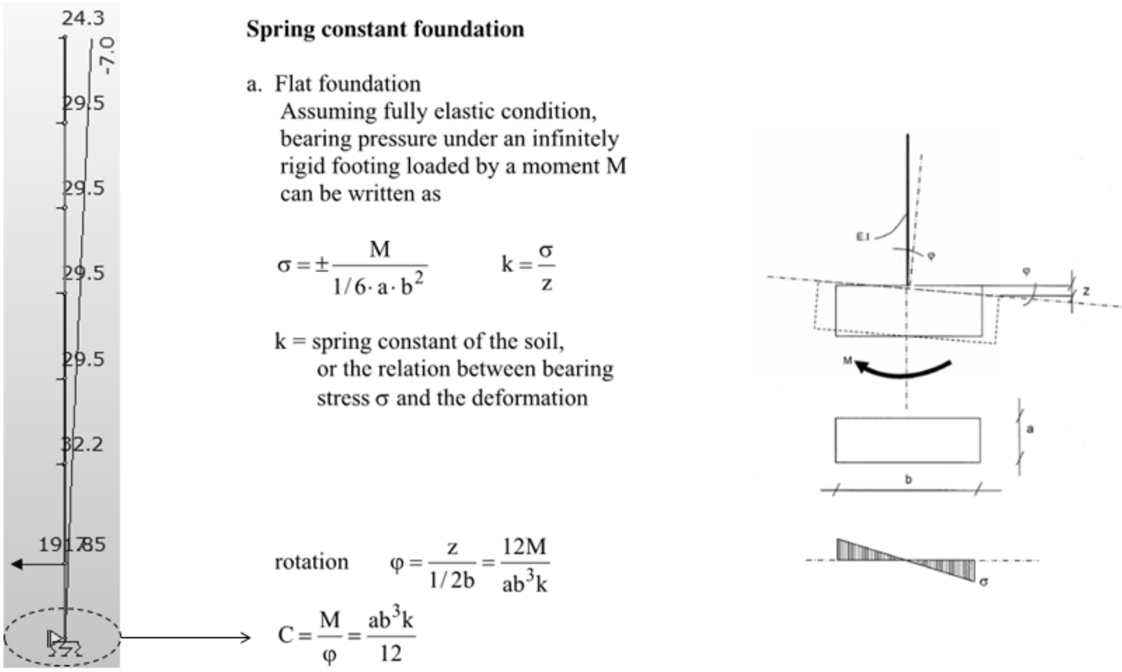


Figure 6-8 (Left) Basic structural schematization stability walls and core (Right) Spring constant foundation

These basic structural models consist a part of the first level structural assessment. Provided a structural member fails, a refined model will be introduced.

Actions on existing structure

The analysis of the existing structure starts with calculating the gravity loads applied on the different structural elements. All the information about the self-weights of the structural elements regarding the existing structure are summarized in table X Appendix B.3. Some of the main actions are presented in the following table.

Table 6-3 Permanent and variable actions on existing structure

	Thickness (mm)	Loads (kN/m^2)		Factor ψ_i		
		Permanent	Imposed	Ψ_1	Ψ_2	Ψ_3
Basement						
Concrete floor	1200	28,80				
Finishing layer	50	1,00				
Variable loads			2,00			
Total		29,80	2,00	0,7	0,7	0,6
Ground floor						
Concrete floor	250	6,00				
Finishing layer	50	1,00				
Variable loads			5,00			
Total		7,00	5,00	0,4	0,7	0,6

Other floors						
Concrete floor	230	5,52				
Finishing layer	50	1,00				
Ceiling and installations		0,50				
Light separation walls			0,80			
Variable loads			1,75			
Total		7,02	2,55	0,4	0,5	0,3

The gravity load calculations are carried out according to both NEN 1990 and NEN 8700. This differs the partial load factors for the ultimate limit space (STR) as discussed in the literature review (section 3.7.4, Table 3-4). The differences will be presented and discussed in the coming sections.

Concrete columns

The concrete columns are mainly responsible to carry the vertical actions applied on the structure to the foundation. The principle according to which the influence area for each column is calculated is presented in Figure 6-9, using as an example column K12. The influence area for each column can be found in the Appendix B.5, in the calculation of the design normal forces for all the columns. An overview of the columns, their code and corresponding position on the floor plan is presented in Figure 6-10.

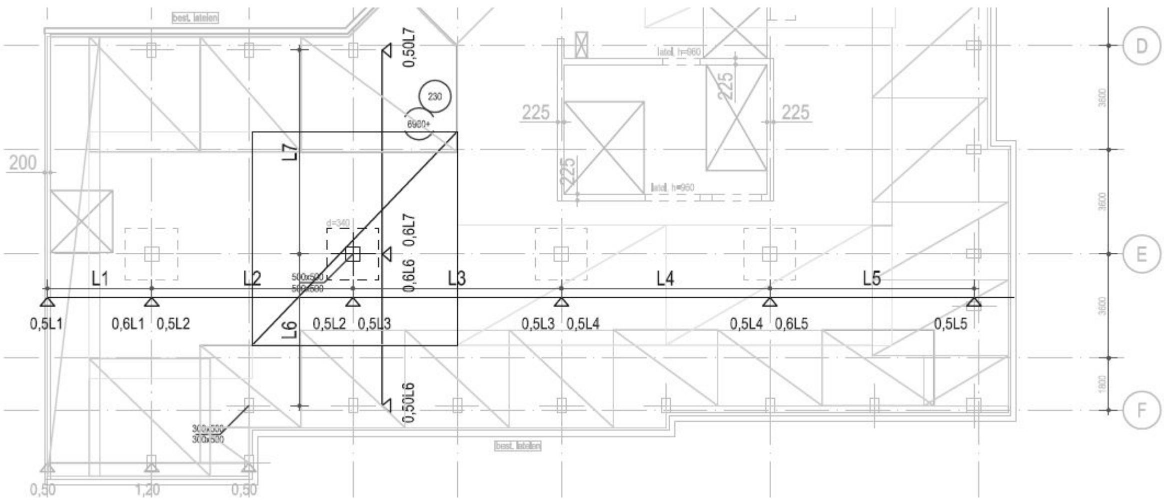


Figure 6-9 Calculation of the influence area for column K12

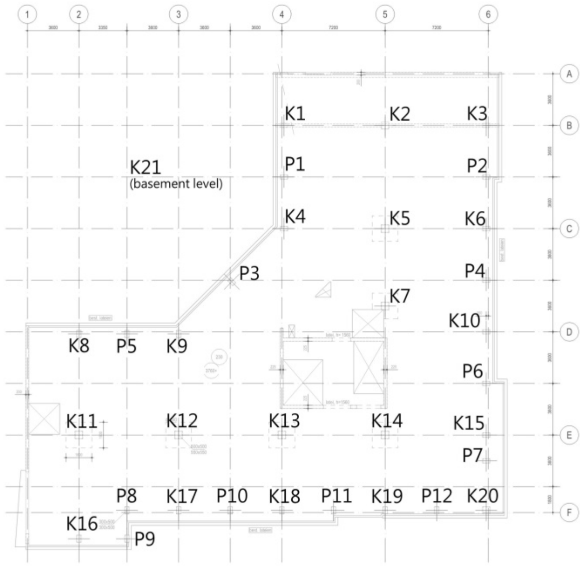


Figure 6-10 Columns' codes and positions - Current state

After having carried out the calculation of the design normal forces applied on the existing columns due to the gravity loads, a comparison is made between the aforementioned values and the ultimate axial load bearing capacities of the different columns. The calculation of the ultimate load bearing capacities, is performed based on the information presented on the archive drawings regarding the reinforcement bars, the concrete and steel qualities.

Since the columns are subject only to axial compressive forces and no bending, the ultimate load that such a column can carry is given by the formula,

$$N_{ult} = \frac{0{,}85 f_{ck}}{\gamma_c}(A_c - A_s) + \frac{f_y}{\gamma_s} A_s$$

Where:
f_{ck}=16 N/mm^2, characteristic cylinder strength of concrete C16/20
γ_c= 1.50, concrete material factor
A_c= concrete sectional area in mm^2
A_s= steel sectional area in mm^2
f_y= 400 N/mm^2 , characteristic yield strength of steel FeB400
γ_s= 1.15, steel material factor

The columns are organized in categories according to their size and reinforcement. The results are presented in Appendix B.6. The unity check of the applied axial load capacity divided by the ultimate axial load bearing capacity reveals the critical columns, that will first have to be checked when adding extra floors (Table 6-4, Table 6-6, Table 6-8). Since the dimensions of the columns differ, mainly in between the ground floor and the basement, the unity check is carried out for a great extent of columns. The calculations are carried out both for NEN 1990 and NEN 8700 partial load factors. The summary tables of the unity checks are presented in the following.

Table 6-4 Unity check existing columns basement floor - Existing situation – NEN 1990

	Column code	Column type	Applied axial load (kN)	Residual axial load capacity (kN)	Unity check
Basement level	K21	4	724	6644,2	0,10
	K4	3	3318	4892,3	0,40
	K5	1	5126	2241,9	0,70
	K7	1	3125	4242,5	0,42
	K8	2	1801	5566,9	0,24
	K9	3	3205	5006,2	0,39
	K11	1	4247	3120,5	0,58
	K12	1	4747	2620,3	0,64
	K13	1	2805	4562,4	0,38
	K14	1	2779	4588,4	0,38

Table 6-5 Unity check existing columns basement floor - Existing situation – NEN 8700

	Column code	Column type	Applied axial load (kN)	Residual axial load capacity (kN)	Unity check NEN 8700	Unity check NEN 1990
Basement level	K21	4	670	6697,7	0,09	0,10
	K4	3	3092	5118,8	0,38	0,40
	K5	1	4772	2595,8	0,65	0,70
	K7	1	2909	4458,9	0,39	0,42
	K8	2	1677	5690,4	0,23	0,24
	K9	3	2982	5228,4	0,36	0,39
	K11	1	3958	3409,8	0,54	0,58
	K12	1	4415	2952,7	0,60	0,64
	K13	1	2612	4756,0	0,35	0,38
	K14	1	2588	4780,1	0,35	0,38

Table 6-6 Unity check columns ground floor - Current state – NEN 1990

	Column code	Column type	Applied axial load (kN)	Residual axial load capacity (kN)	Unity check
Ground floor	K4	8	2019	1668,7	0,55
	K5	5	4179	1224,5	0,77
	K6	6	1288	1801,0	0,42
	K7	5	2605	2797,9	0,48
	K8	6	1025	2064,4	0,33
	K9	8	1883	1804,9	0,51
	K10	6	1288	1801,0	0,42
	K11	5	3546	1857,4	0,66
	K12	5	3957	1445,9	0,73
	K13	5	2339	3064,6	0,43
	K14	5	2317	3086,2	0,43
	K15	6	1120	1969,6	0,36
	K16	6	1397	1692,2	0,45

K17	6	1127	1962,9	0,36
K18	6	1127	1962,4	0,36
K19	6	932	2157,7	0,30
K20	7	857	1954,9	0,30
P1	6	1445	1644,4	0,47
P2	6	1332	1757,7	0,43
P3	8	1374	2313,9	0,37
P4	6	1288	1801,0	0,42
P5	6	1445	1644,4	0,47
P6	6	1426	1663,8	0,46
P7	6	1096	1993,6	0,35
P8	6	879	2210,1	0,28
P9	6	353	2736,3	0,11
P10	6	1130	1959,5	0,37
P11	6	1053	2036,6	0,34
P12	6	1053	2036,6	0,34

Table 6-7 Unity check columns ground floor - Current state – NEN 8700

Ground floor

Column code	**Column type**	**Applied axial load** (kN)	**Residual axial load capacity** (kN)	**Unity check NEN 8700**	**Unity check NEN 1990**
K4	8	1886	1802,0	0,51	0,55
K5	5	3893	1510,3	0,72	0,77
K6	6	1206	1883,4	0,39	0,42
K7	5	2429	2974,3	0,45	0,48
K8	6	960	2129,4	0,31	0,33
K9	8	1756	1932,0	0,48	0,51
K10	6	1206	1883,4	0,39	0,42
K11	5	3310	2093,3	0,61	0,66
K12	5	3687	1716,3	0,68	0,73
K13	5	2181	3222,3	0,40	0,43
K14	5	2161	3242,3	0,40	0,43
K15	6	1048	2041,4	0,34	0,36
K16	6	1306	1783,4	0,42	0,45
K17	6	1055	2034,4	0,34	0,36
K18	6	1055	2034,4	0,34	0,36
K19	6	874	2215,4	0,28	0,30
K20	7	807	2004,6	0,29	0,30
P1	6	1341	1748,1	0,43	0,47
P2	6	1246	1843,6	0,40	0,43
P3	8	1287	2400,8	0,35	0,37
P4	6	1206	1883,8	0,39	0,42
P5	6	1341	1748,1	0,43	0,47
P6	6	1334	1754,9	0,43	0,46
P7	6	1025	2064,3	0,33	0,35

	P8	6	823	2266,8	0,27	0,28
	P9	6	328	2761,5	0,11	0,11
	P10	6	1058	2031,3	0,34	0,37
	P11	6	987	2102,4	0,32	0,34
	P12	6	987	2102,4	0,32	0,34

Table 6-8 Unity check columns 1st floor - Current state – NEN 1990

	Column code	Column type	Applied axial load (kN)	Residual axial load capacity (kN)	Unity check
1^{st} floor	K1	11	1213	1424,0	0,46
	K2	14	1192	3735,3	0,24
	K3	11	531	2105,8	0,20

Table 6-9 Unity check columns 1st floor - Current state – NEN 8700

	Column code	Column type	Applied axial load (kN)	Residual axial load capacity (kN)	Unity check NEN 8700	Unity check NEN 1990
1^{st} floor	K1	11	1136	1501,1	0,43	**0,35**
	K2	14	1119	3808,3	0,23	0,21
	K3	11	497	2140,1	0,19	0,14

It is observed that the critical column for the basement, as well as for the ground floor, is column K5, followed by columns K11 and K12 (Figure 6-10).

Moreover, remarkable is the very small unity check of the columns along the façades. A deeper research and study of the existing documentation reveals that the façade columns with the code 'P' (see Figure 6-10) were meant to be prefabricated columns that would be put on position after the construction of the structural skeleton, in order to carry only the variable loads. However, in real practice these columns have been constructed as in-situ columns, with the same reinforcement as 'K' columns, that had been designed to carry the permanent loads above double permanent loads. At the same time the 'K' columns are loaded with smaller loads than originally designed.

Stability

The stability of the building is provided per direction by a reinforced concrete wall and the reinforced concrete core. In order to compute the percentage of the wind load that is taken by each element two different schematizations have been used:

- Statically determined beam (Figure 6-11). This simple structural scheme is used when having only two stability elements per direction and separates totally the contribution of the stability elements in the perpendicular direction.

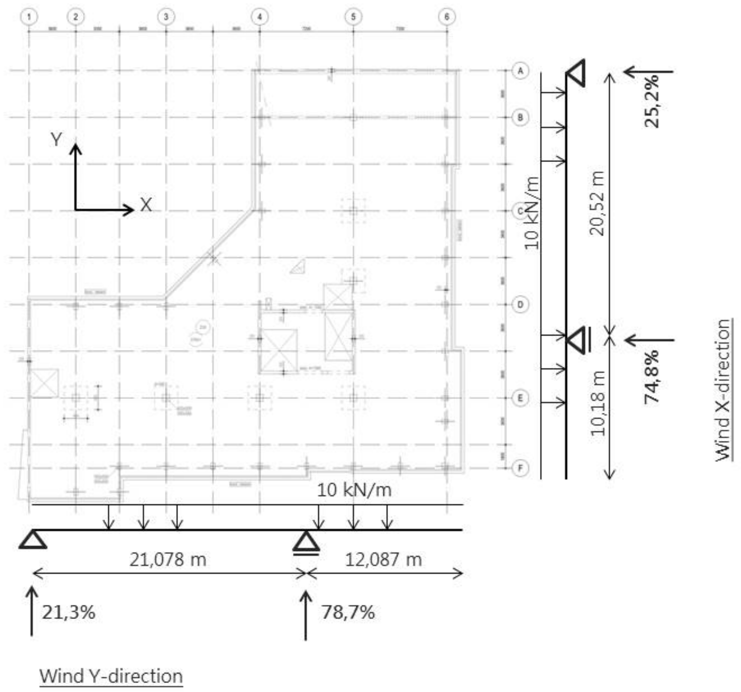


Figure 6-11 Distribution of the wind load - Statically determined beam

- Framework supported by springs (Figure 6-12). The detailed computation of the wind load distribution percentages according to this method can be found in the Appendix B.7.

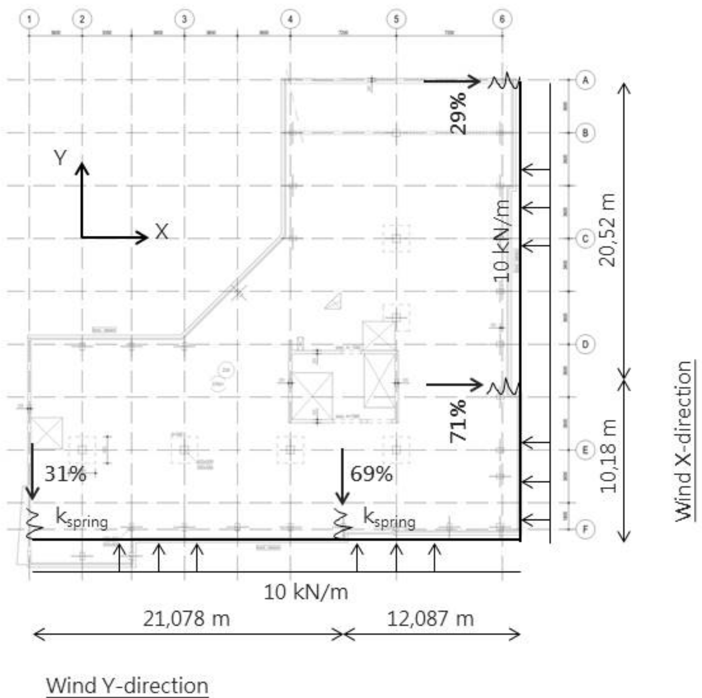


Figure 6-12 Distribution of wind load - Framework supported by springs

The value 10 kN/m is a random value used for the purposes of computing the percentages. The results of both methodologies can be found in Table 6-10. Considering that the scheme and methodology of the spring supported framework is representing more accurate the way the particular structure behaves and reacts to the wind loads, the percentages for the distribution of the wind loads will be taken from this method.

Table 6-10 Summary table wind load distribution percentages

	Statically determined beam		Framework on springs	
Wind direction	Stability wall	Shear core	Stability wall	Shear core
Y-Y	21,3%	78,7%	31%	69%
X-X	25,2%	74,8%	29%	71%

Stability wall axis 1

The stability walls will be checked for the compressive stresses, caused from the gravity loads and the wind, on the basement level (ULS_2= 1.2*Permanent+1.5*Variable+1.5*Wind), and for the tensile stresses, from the combination of the gravity and the wind loads, on the ground floor level (ULS_3= 0.9*Permanent+1.5*Wind). Given the fact that the y-y direction of the wind load is the dominant and that the stability wall on axis 1 is shorter than the one on axis A, the wall on axis 1 will be checked against the tension stresses. As it has already been calculated in the previous section, the stability wall on axis 1 takes the 31% of the wind load. In Table 6-11 and Table 6-12 the unity checks for tension and compression respectively are shown.

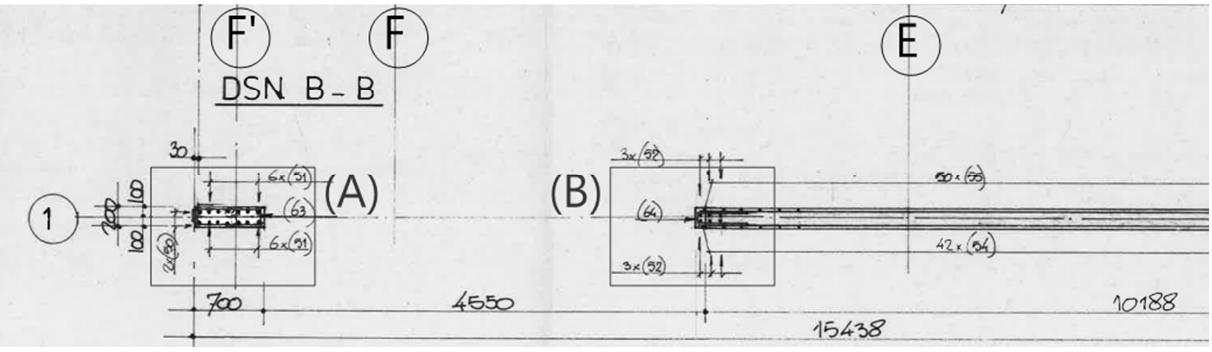


Figure 6-13 Stability wall axis 1 tensile reinforcement

Table 6-11 Stability wall axis 1 unity check tensile reinforcement - Current state – NEN 1990

	Thickness (mm)	Length (mm)	Rebars	Area (mm^2)	max $F_{tensile}$ (kN)	max $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity check
Wall axis 1 (A)	200	700	12ø20+2ø16	4172	1451	10,37	0,025	0,00
Wall axis 1 (B)	200	1000	6ø16+10ø10	1992	693	3,46	0,025	0,00

Table 6-12 Stability wall axis 1 unity check concrete C16/20 compression - Current state – NEN 1990

	Thickness (mm)	Length (mm)	Applied $F_{compression}$ (kN/m)	Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check
Wall axis 1	250	1000	423	1,692	10,67	0,16

Shear core

The stresses resulting from the gravity and the wind loads acting on the concrete core are computed and therefore compared to the ultimate capacities of the concrete core. The load combinations are the same as for the stability wall 1. The core is assumed to take the 70% of the wind load parallel to the y-y direction. For the calculation of the core the mechanical scheme used can be seen in Figure 6-8. The analytical calculation is to be found in the Appendix B.8. The main information and results are summarized in the Table 6-14 and Table 6-15. Figure 6-14, Figure 6-15, Figure 6-16 and Figure 6-17 can be used for better reading and understanding of tables.

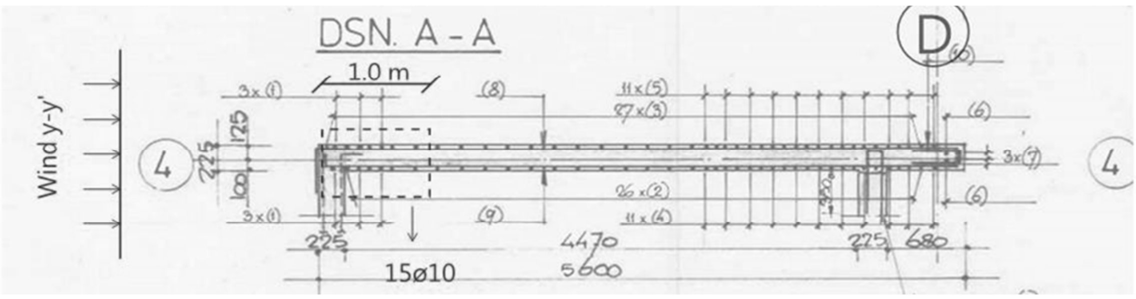


Figure 6-14 Position of tensile reinforcement core-wall 1

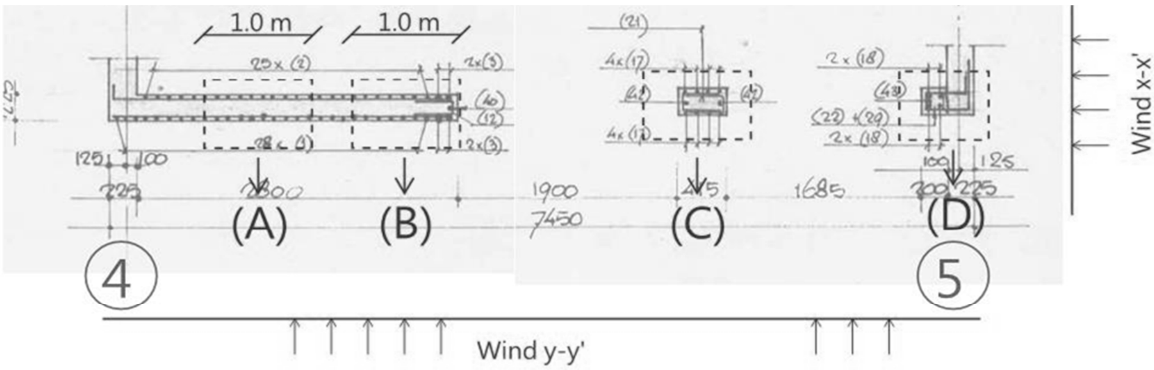


Figure 6-15 Position of tensile reinforcement core-wall 2

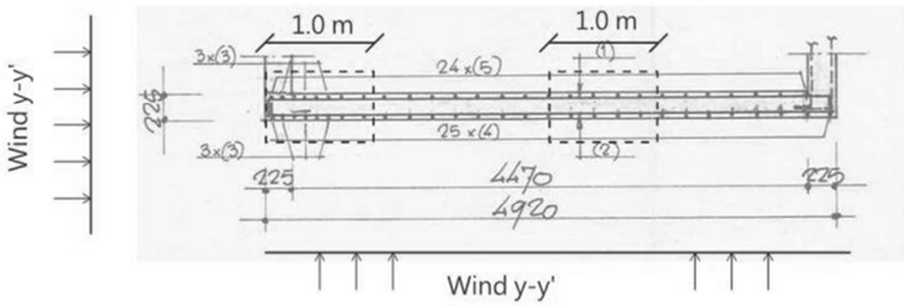


Figure 6-16 Position of tensile reinforcement core-wall 3

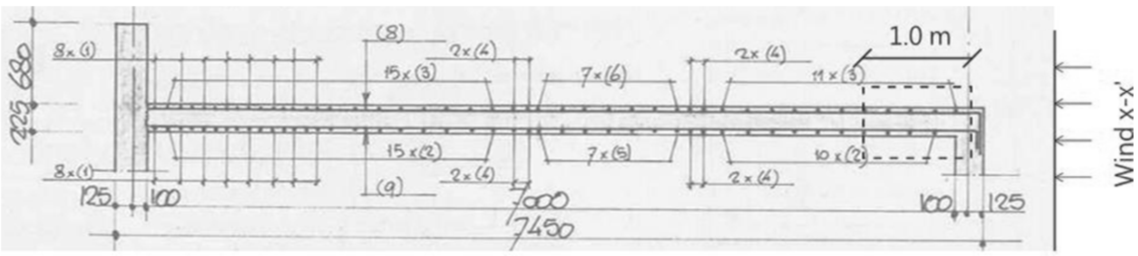


Figure 6-17 Position of tensile reinforcement core-wall 4

Table 6-13 Core-walls existing reinforcement and ultimate tensile capacity – NEN 1990

	Thickness (mm)	Length (mm)	Rebars	Area (mm^2)	max $F_{tensile}$ (kN)	max $\sigma_{tensile}$ (N/mm^2)
Wall 1 - y-y'	225	1000	15ø10	1178	410	1,82
Wall 2 - y-y' (A)	225	1000	20ø10	1571	546	2,43
Wall 2 - y-y' (B)	225	1000	15ø10+5ø12	1744	606	2,70
Wall 2 - y-y' (C)	225	415	10ø12	1131	393	4,21
Wall 2 - y-y' (D)	225	425	5ø20+4ø10	1885	656	6,86
Wall 3 - y-y'	225	1000	16ø10	1257	437	1,94
Wall 3 - x-x'	225	1000	10ø10	785	273	1,21
Wall 4 - x-x'	225	1000	10ø10	785	273	1,21
Wall 2 - x-x' (D)	225	425	5ø20+4ø10	1885	656	6,86

Table 6-14 Unity check tensile reinforcement ground floor level core walls - Current state – NEN 1990

	Ultimate $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity Check
Wall 1 - y-y'	1,821	No tension	-
Wall 2 - y-y' (A)	2,428	0,211	0,09
Wall 2 - y-y' (B)	2,695	0,211	0,08
Wall 2 - y-y' (C)	4,213	0,211	0,05
Wall 2 - y-y' (D)	6,856	0,211	0,03
Wall 3 - y-y'	1,943	No tension	-
Wall 3 - x-x'	1,214	No tension	-
Wall 4 - x-x'	1,214	No tension	-
Wall 2 - x-x' (D)	6,856	No tension	-

Table 6-15 Unity check concrete C16/20 compressive strength basement level - Current state – NEN 1990

	Applied $F_{compressive}$ (kN/m)		Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check
	Wind Y-Y'	**Wind X-X'**			
Wall 1	791	721	3,517	10,667	0,33
Wall 2	-	618	2,742	10,667	0,26
Wall 3	911	-	4,051	10,667	0,38
Wall 4	996	925	4,426	10,667	0,41

Basement walls

The basement walls are subject to the gravity loads of the upper structure (compression) and the forces generating from the ground layer and possible ground water table around the building pot. The overview of the design line loads applied on the existing basement walls is available in Appendix B.9 and the check of the existing reinforcement against the vertical load in Appendix B.10.

In the same direction as the core- and stability walls, the vertical loads are not normative for the capacity of the basement walls.

Foundation

Geomet Consult carried out an investigation of the existing condition of the underground and developed a methodology to calculate the bearing capacity of the foundation plate. This methodology is further explained in the report of *Geomet Consult* (Appendix B.2). The foundation plate will be checked at the critical locations. These are under the columns, the basement walls, and the concrete core for the bottom reinforcement and under the columns for punching shear. All calculations are to be found in Appendix B.11. The outcomes of the calculation for the estimation of the residual bearing capacity of the foundation plate are summarized in Table 6-16. The ultimate compressive forces result from a reverse calculation on the basis of the bottom reinforcement and the effective width of the foundation slab.

Table 6-16 Unity check bottom reinforcement foundation plate - Current state – NEN 1990

	Max. applied $F_{compression}$ (kN/m)		Ultimate $F_{compression}$ (kN/m)	Unity Check
	Concrete core walls			
	Wind Y-Y'	Wind X-X'		
Wall 1	769	721	3100	0,25
Wall 2	-	617	2100	0,30
Wall 3	889	-	1800	0,50
Wall 4	974	924	2100	0,46
	Columns (kN)			
Column K5	5126		7800	0,66
Column K11	4247		7800	0,54
Column K12	4747		7800	0,61
	Basement walls			
Stability wall axis 1	473		1800	0,26
Basement wall axis 6	463		1800	0,26
Basement wall axis F	360		1800	0,20

Conclusions existing structure

Reviewing the unity checks of the structural elements one could draw some conclusions for the overcapacity of the structure, and the critical elements.

- ✓ Columns K5, K11 and K12 (Figure 6-10) are distinguished as the ones with the highest unity checks.
- ✓ Consequently, attention should be given to the foundation plate under the aforementioned critical columns. This is confirmed by the unity check of the bending reinforcement of the foundation plate (Table 6-16).
- ✓ The unity checks of the basement and core walls, give the impression of having a great overcapacity under compression. However, tensile forces could be critical, since they have higher unity checks.

The last bullet is enhanced, by the indisputable impact of the wind load, during the increase of the height of a building, on the wind moment at the base of the building, and thus on the tensile stresses there.

It should be mentioned that the calculation of the residual capacities of the existing structural elements, is, more precisely, an estimation of these overcapacities. The research during the literature review of the current thesis, revealed that in the normal practice a number of on-site and laboratory tests is performed, so that to define the actual state, quality, material properties and strength of the concrete and steel.

6.4 VERTICAL EXTENSION

6.4.1 Structural design new block

The structural design of the new block is based on the strategy described in section 6.1.2. In this way, some general conclusions can be fixed from the beginning of the design, forming the base in order to proceed to more specific choices. These are:

- ✓ The use of steel as the main structural material. The advantages of steel upon concrete in vertical extension have been emphasized in both the literature research and the example case studies.
- ✓ The function of the new block as well as the existing part of the building is residential, except for the commercial ground floor. This decision is supported by the lower variable loads that have a positive effect on the total loads of the structure.

An analysis is performed for the features of the design that could not be fixed straight away, so as to result to the optimal use of the existing structure. More precisely, two parametric analyses are introduced, one for the floor system and one for the structural configuration of a typical floor.

Floor system

At a first sight, the more desirable feature of the floor system is the light self-weight. However, looking more carefully at a vertical extension project and at the design parameters that emerged out of the literature review, one can say undeniably that there are more attributes which play an important role. An in depth research of such attributes leads to a list of the most relevant ones to the specific design study case. The aforementioned attributes are investigated for a number of different floor systems with the aim of concluding to the one that fulfills in the best way the requirements of the project. A summary of this research is presented in Table 6-17 and for the detailed calculations see Appendix C.1.

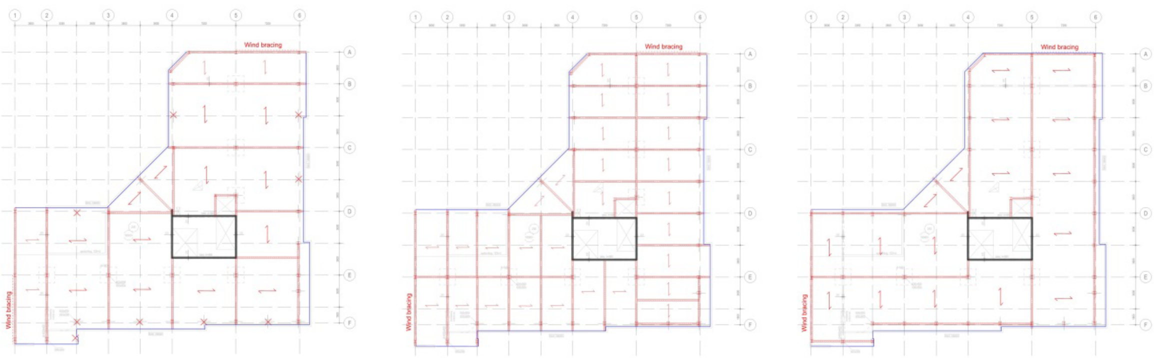
Table 6-17 Parametric study floor systems

Floor systems	Span (mm)	Height (mm)	Weight * (kN/m²)	Shadow price (€/m²)	Construction costs (€/m²)
Timber joists with Lewis Zwaluwstaartplaat	3600	530	2,53	€ 3,38	€ 115,00
Quantum Deck	7200	345	2,56	€ 3,32	€ 130,00
SlimLine	7200	373	4,29	€ 4,77	€ 115,00
Hollow core slabs	7200	442,5	6,26	€ 4,33	€ 80,00
Bubble Deck	7200	392,5	7,07	€ 8,59	€ 100,00

The chosen floor system that meets better the requirements of the project and has relatively favorable values for the several parameters of Table 6-17 is Quantum Deck floor. The design tables and more information about Quantum Deck floor are presented in Appendix C.2.

Structural configuration

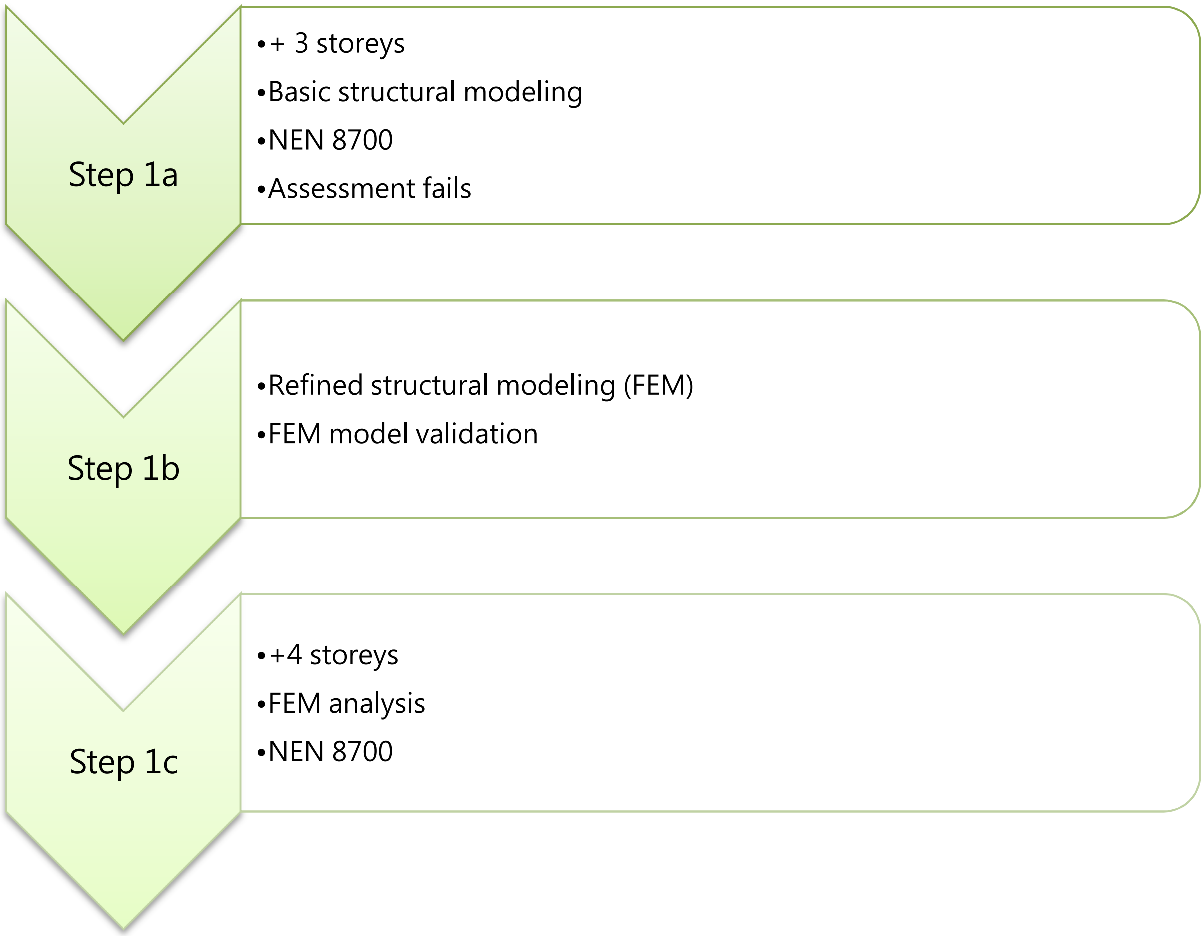
Three different structural configurations are designed and considered in combination with the optional floor systems studied in Table 6-17. These configurations are depicted in Figure 6-18. The main principle of the structural design of this project is to take advantage of the overcapacity of the existing structure to its ultimate limits. By this principle, option N^{o}1 is rejected since a number of the existing façade columns is not included in the vertical transfer of the loads of the new block. The second option could be chosen in case the floor system with the timber joists would be applied. A small floor span of 3,6 m is not configured for the Quantum Deck floor. However, option N^{o}3 is combining the two characteristics the other two options are missing; the floor span is 7,2 m and the structural configuration makes use of all the existing columns. This design will be applied eventually at the new block.



Option N°1 (l=7,2 m) Option N°2 (l=3,6 m) Option N°3 (l=7,2 m)

Figure 6-18 Three optional structural configurations of the new block

6.5 SCENARIO 1



6.5.1 Basic structural assessment

The first scenario includes the addition of such an amount of floors so that no strengthening of the structural elements is necessary. Attention is paid firstly to the critical elements and the stability of the structure. Checks are carried out for the ultimate as well as the serviceability limit state. It is really interesting to look at the impact of the extra load on the various unity checks. For the calculation of the vertical and lateral loads the safety factors of NEN 8700 is applied, provided the fact that the existing structure is left totally untouched. The structure is checked for the load combinations:

- ULS_2 =1.20*Per.Loads +1.50*Var.Loads +1.50*Wind (compression)
- ULS_3 =0.90*Per.Loads +1.50*Wind (tension)
- SLS =1.00*Per.Loads +1.00*Var.Loads +1,00*Wind (displacements)

Determining the limits of an existing structure with regard to vertical extension is an iterative process. The methodology followed in Scenario 1 is described in the following and the main conclusions are presented. For the detailed calculation look in Appendix D.2.

As a start, three floors are added on top of the existing building. The total height of the building is then 31,60 m. The structure is checked for strength and stability. The basic structural models that are used to carry out the calculations are again bars that have the stiffness of the respective structural element (wall, core, steel frame). An interesting point is the modeling of the steel frames above the 6th floor with fictive walls of the same thickness (Figure 6-19).

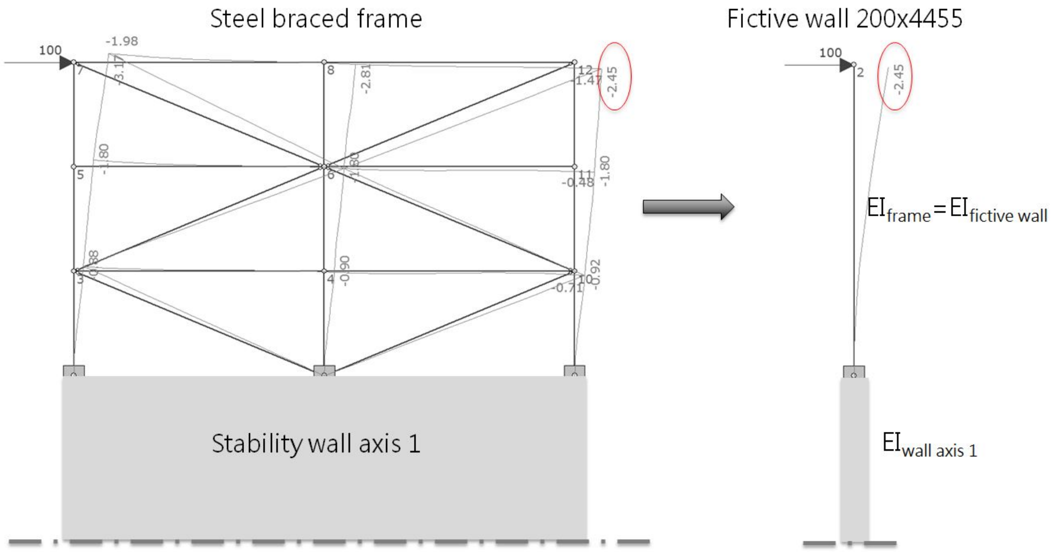


Figure 6-19 Displacement diagram for steel braced frame and fictive wall under an horizontal force 100 kN

The unity checks of the critical elements, both for ULS and SLS, fulfill the requirements. How the unity checks are formed for the structural elements and the comparison to the unity checks of the current situation can be seen in the following tables.

Table 6-18 Stability wall axis 1 unity check tensile reinforcement - Vertical extension +3 levels

	max $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity check	Unity check current state (Table 6-11)
Wall axis 1 (A)	10,37	0,937	0,09	0,04
Wall axis 1 (B)	3,46	0,937	0,27	0,12

Table 6-19 Stability wall axis 1 unity check concrete C16/20 compression - Vertical extension +3 levels

	Applied $F_{compression}$ (kN/m)	Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check	Unity check current state (Table 6-12)
Wall axis 1	680	3,400	10,67	0,32	0,16

Table 6-20 Unity check tensile reinforcement ground floor level core walls - Vertical extension +3 levels

	Ultimate $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity Check	Unity check current state (Table 6-14)
Wall 1 - y-y'	1,821	0,913	0,50	0,18
Wall 2 - y-y' (A)	2,428	1,108	0,46	0,11
Wall 2 - y-y' (B)	2,695	1,108	0,41	0,16
Wall 2 - y-y' (C)	4,213	1,108	0,26	0,10
Wall 2 - y-y' (D)	6,856	1,108	0,16	0,06

Wall 3 - y-y'	1,943	0,643	0,33	-
Wall 3 - x-x'	1,214	No tension	-	-
Wall 4 - x-x'	1,214	No tension	-	-
Wall 2 - x-x' (D)	6,856	No tension	-	-

Table 6-21 Unity check concrete C16/20 compressive strength basement level core walls- Vertical extension +3 levels

	Applied $F_{compression}$ (kN/m)		Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check	Unity check current state (Table 6-15)
	Wind Y-Y'	Wind X-X'				
Wall 1	990	888	4,399	10,667	0,41	0,32
Wall 2	-	795	3,532	10,667	0,39	0,26
Wall 3	1109	-	4,927	10,667	0,46	0,37
Wall 4	1220	1118	5,421	10,667	0,51	0,41

Table 6-22 Unity check bottom reinforcement foundation plate - Vertical extension +3 levels

	Max. applied $F_{compression}$ (kN/m)		Ultimate $F_{compression}$ (kN/m)	Unity Check	Unity check current state (Table 6-16)
		Concrete core walls			
	Wind Y-Y'	Wind X-X'			
Wall 1	990	888	3100	0,32	0,27
Wall 2	-	795	2100	0,38	0,30
Wall 3	1109	-	1800	0,62	0,53
Wall 4	1220	1118	2100	0,58	0,49
		Columns (kN)			
Column K5	5273		7800	0,68	0,65
Column K11	4815		7800	0,62	0,45
Column K12	4959		7800	0,64	0,51
		Basement walls			
Stability wall axis 1	680		1800	0,38	0,26
Basement wall axis 6	625,1		1800	0,35	0,26
Basement wall axis F	488,9		1800	0,27	0,20

It is noticeable that the structural elements meet the ULS requirements and withstand the extra loads resulting from the vertical extension. Emphasis is put on interpreting the above tables; it is obvious that the impact of the vertical extension is greater on the unity check of the tensile strength of the stabilizing elements than on the compressive strength. This can be explained, if one considers the light weight structure of the new part, that results to a small increase of compressive stresses, and at the same time the unbiased increase of the wind loads.

However, the type of foundation that Astoria building has, does not allow for tensile stresses at the contact surface with the subsoil. From the hand calculations it is concluded that for the normative y-y wind direction, at the position of the concrete core (Figure 6-20) tensile contact stresses are present (Table 6-23).

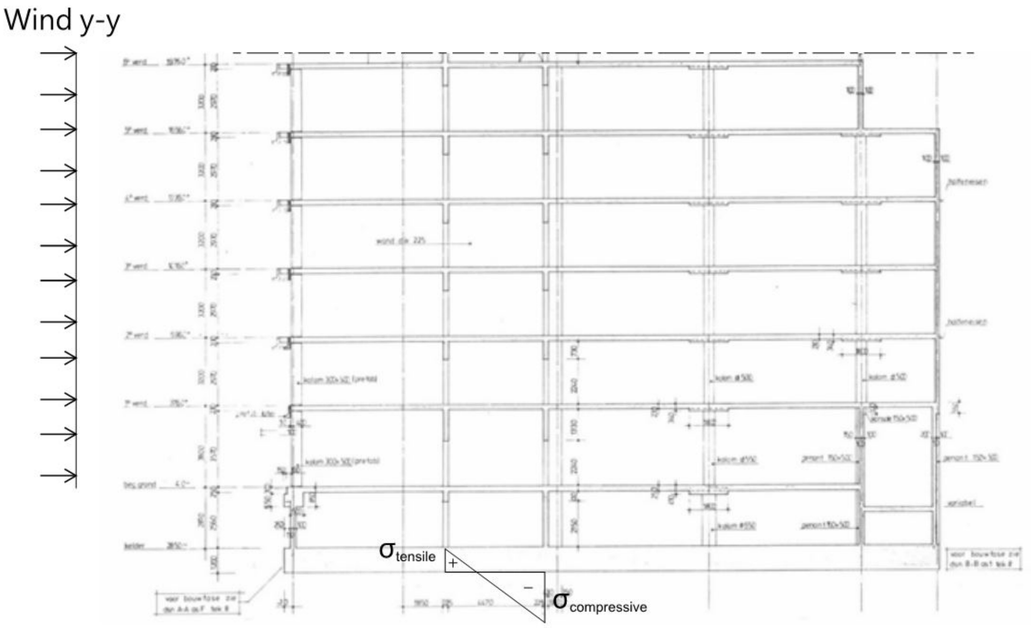


Figure 6-20 Contact tensile stresses under concrete core

Table 6-23 Contact stresses - Scenario 1 - +3 storeys - NEN 8700

	Tensile contact stresses (y-y wind direction)
Core wall 1	0.676 N/mm^2
Core wall 2	0.908 N/mm^2
Core wall 3	0.362 N/mm^2
Core wall 4	0.047 N/mm^2

Analyzing these tensile stresses, and considering the geometrical and physical characteristics of the foundation concrete slab, which make it actually a very stiff structural element, one can conclude that these tensile stresses cannot cause in reality failure of the structure. The tensile stresses are related to the modeling of the structural skeleton, and more precisely, to the methodology that was followed for the calculation of the concrete slab. This methodology is described in Figure 6-8 and in the report of GEOMET (Appendix B.2), and actually assumes that only a part of the foundation plate, with dimensions 'a' and 'b' (a=9,8m and b=7,6m, in particular), works together with the concrete core (Figure 6-21). This simplification leads to the existence of tensile forces under the foundation plate and thus to 'failure' of the structure, since a foundation plate cannot carry any tensile forces. In other words, this methodology neglects the continuity of the foundation slab. Therefore, an advanced structural assessment is necessary and a refined model is introduced.

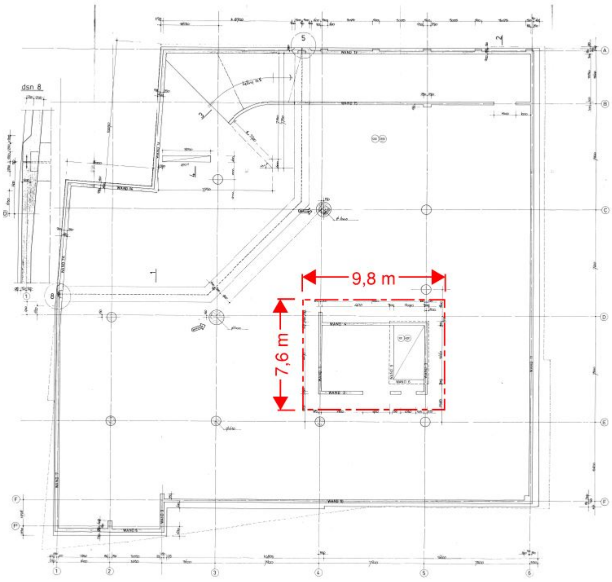


Figure 6-21 Collaborating part of the foundation plate under the core based on GEOMET methodology

Along with the aforementioned rationale, one more attention point arises and needs to be analyzed more precisely. This is related to the connection of steel braced frames to the existing stability walls (Figure 6-19). The basic structural schematization of two bars connected in one node is not representative of the real situation since the forces are in fact transferred via three connection points, and not one. Hence, the transition to a more sophisticated analysis is deemed necessary to proceed with more accurate structural calculations.

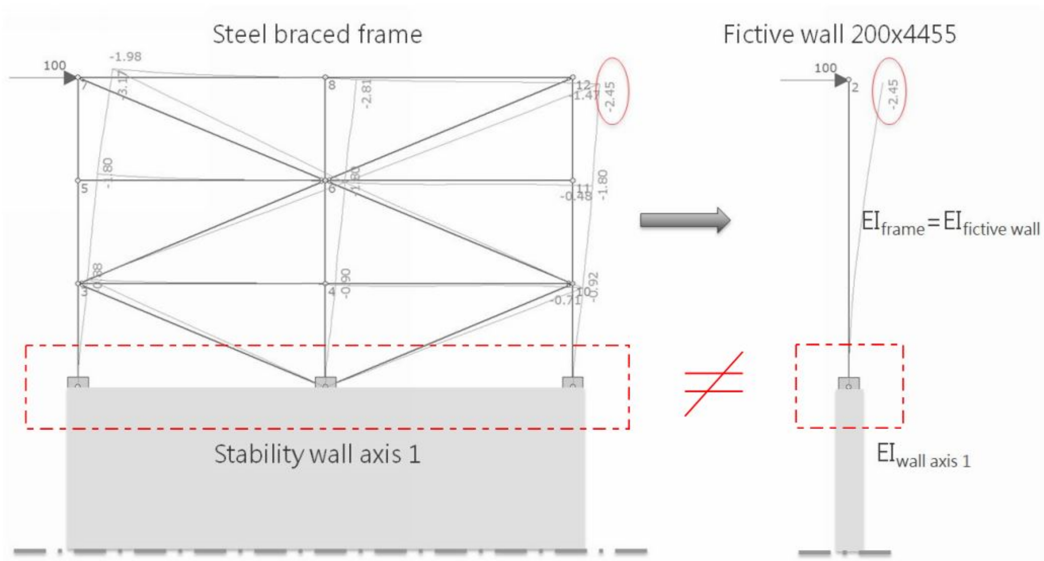


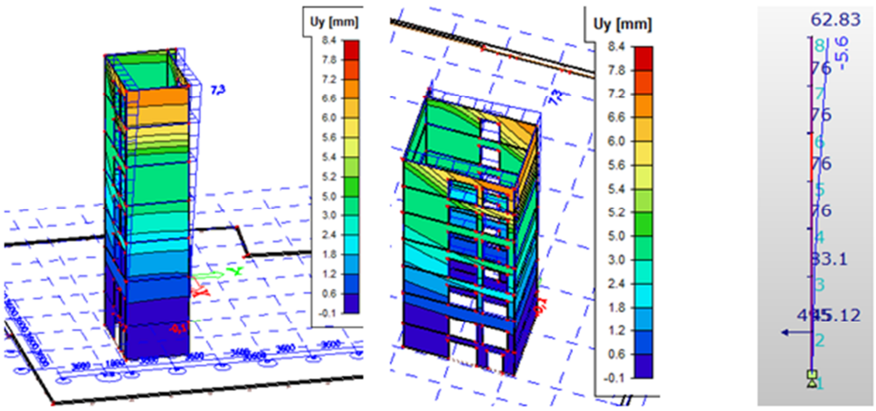
Figure 6-22 Connection existing stability wall and braced frame

6.5.2 FEM model validation

The Finite Element Method (FEM) is introduced as an advanced structural assessment of the existing structure. To that end, the Scia engineering software is used. This advanced analysis is intended to be as efficient as possible. The elements to be modeled are these where mismatches and inaccuracies have been observed, i.e. the stability elements in the normative wind direction (stability wall axis 1 and core) and the foundation plate.

One of the most important aspects of FEM is to input the correct data, in order for the model to behave as the real structure. To achieve this, some basic steps are followed with the purpose of validating the stiffness and the behavior of the concrete core, the foundation slab and the modeling of the supporting conditions in the Scia environment. The loads imposed are random and serve only the purposes of validation of the data in between the two assessment levels.

Step 1 : The Scia model of the concrete core fixed at the base gives a total deformation at the highest point of the core of 7,3 mm at the edge of the core and 5,4 mm in the middle. The higher value at the right edge can be justified by taking into consideration the openings in the core, that make it is less stiff at this side. Looking at the mechanical scheme of the core in the Technosoft calculation, where the core is modeled as a bar fixed at the base that has the same stiffness as the core, one can see that the maximum displacement of the core is 5,6 mm. The conclusion is that the 3D core is representative of the real situation.



Step 2 : Next step is to model the elastic foundation. The spring stiffness of the soil can be found in the geotechnical report of GEOMET (k=30MN/m^3). The walls are assumed to be elastically supporting by the soil of spring stiffness 4 MN/m^3. This stiffness is a result of an iterative process between the 2D model and the 3D model of the core in combination with the box structure of the basement. This box structure is modeled by the H_{wind} force in the 2D model (Figure 6-23).

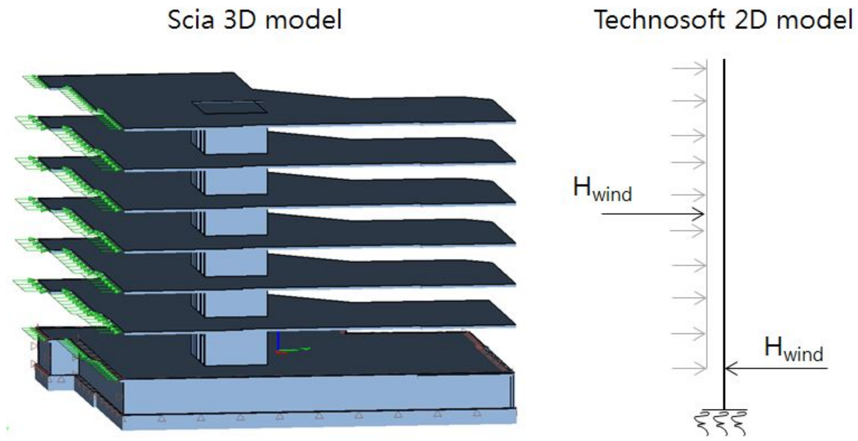


Figure 6-23 Comparison of support conditions modeling between basic and FEM structural assessment

In Technosoft 2D model, the support is modeled as a spring with a stiffness calculated according to the formula depicted in Figure 6-8. The dimensions 'a' and 'b' are chosen based on the vertical loads according to the graphs of the geotechnical report, see Appendix B.2. However the methodology that GEOMET suggests is useful to proceed with the calculations, it is only taking into consideration a part of the foundation plate neglecting the influence of the plate in distances larger than 'a' and 'b'. On the other hand, this influence is considered in the Scia analysis and this is the reason why the displacements of the core differ (Figure 6-24). By using the higher influence area out of the Scia model, the spring stiffness value increases and then the Technosoft model approaches the results of Scia (Figure 6-25).

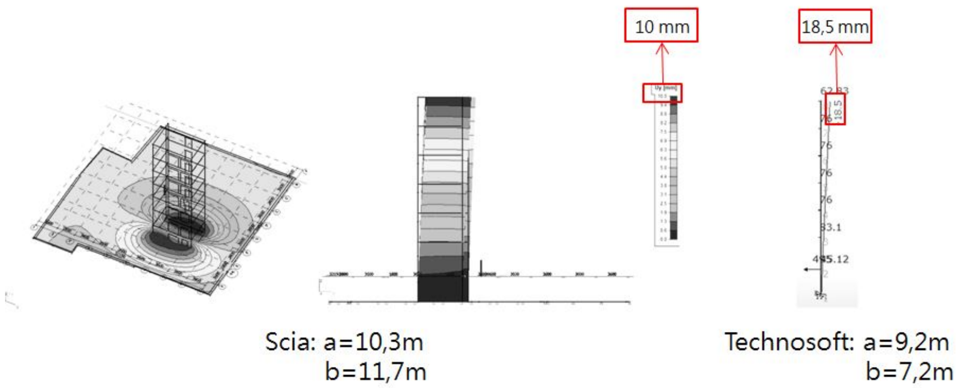


Figure 6-24 Effect of the different collaborating areas of the foundation plate on the core displacement

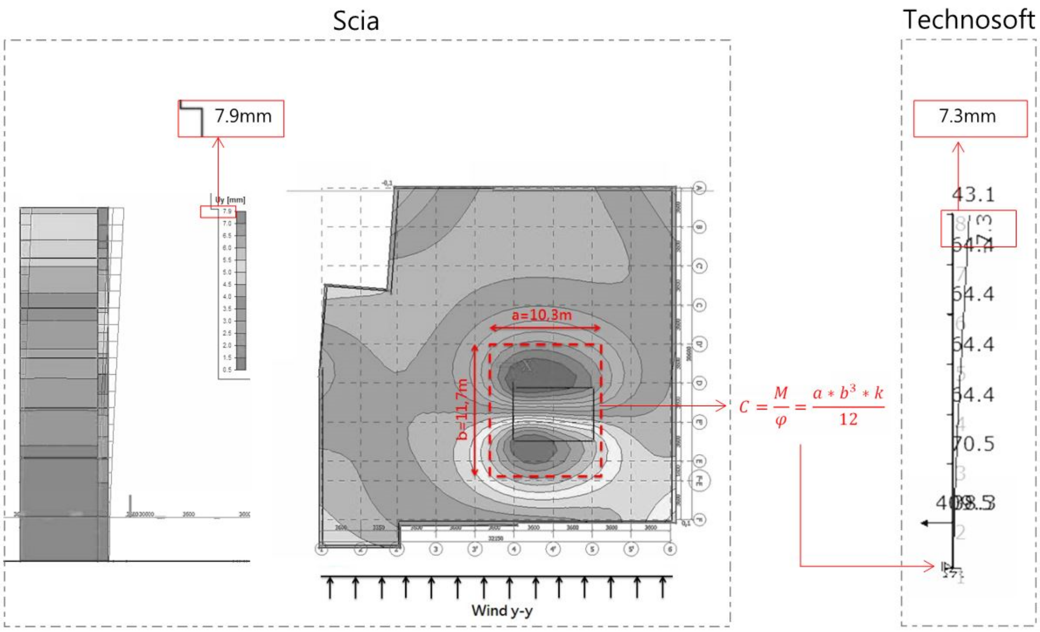


Figure 6-25 Effect of the same collaborating areas of the foundation plate on the core displacement

6.5.3 Advanced structural assessment

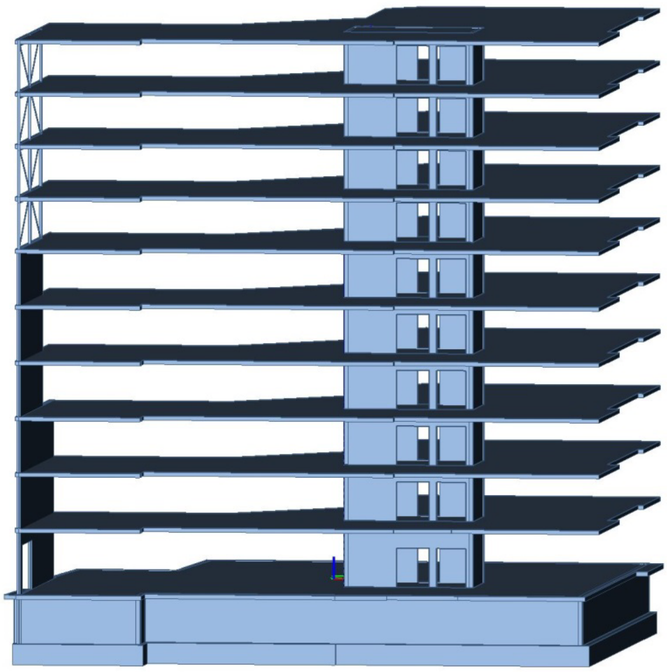


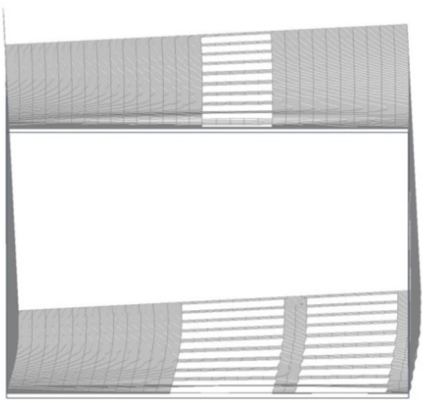
Figure 6-26 FEM model stability elements Scenario 1

The Scia model consists of the elements that provide the stability of the structure in y-y direction, which is the normative direction of the lateral loads. To have a better understanding of the model, an overview of the basic parameters that have been imported is presented in the following.

- A crack analysis is assumed for the concrete of the existing structure. Therefore, the modulus of elasticity is reduced to the value of 8000 N/mm^2.

- Non-linear analysis is performed.
- All vertical loads are manually inserted based on the representative values of the load calculation (Appendix D.1). Thus, the self-weight of the structural elements in the Scia environment is set to zero, both for concrete and steel elements.
- The vertical loads of the structural elements modelled, i.e. the stability wall on axis 1, the core, are imported as line loads on the edges of the elements.
- The wind load is inserted per floor, as a lateral line loads along the edges of the floor slabs. As follows, these loads are transferred via the diaphragm action of slabs to the stability elements and thereafter to the foundation plate.
- For the purpose of force equilibrium of the model, and considering that all lateral loads are inserted, the total weight of the building is entered as a surface load (permanent and variable load) on the foundation plate. This is a simplification that is chosen to be adapted for the sake of time efficiency.
- The non-linear analysis is carried out based on the load combinations:
  - ULS_1 =1.15*Perm. Loads +1.30*Var. Loads +1.30*Wind
  - ULS_2 =0.90*Perm. Loads +1.30*Wind
  - SLS =1.00*Perm. Loads +1.00*Var. Loads +1,00*Wind

Step by step individual models are created so as to check what the influence of the addition of storeys is on the unity checks of the critical elements. At the same time a comparison is made to the hand calculations and it is checked whether the model reacts and behaves as expected under the combination of the vertical and lateral loads. The models that comprise the addition of 3 and 4 storeys, respectively, on top of the existing building do not lead to excess of the unity checks and failure of any structural elements. Thus, one more level is introduced, and having in total 5 extra levels the first element fail. The model confirms what was first discussed in section 6.3.4, that the building rotates anti-clockwise (Figure 6-27) and this rotation is actually the reason of the failure. The rotation results from the stiffness difference between the two stability elements, i.e. the wall on axis 1 is 5 times stiffer than the concrete sore in this direction. The critical element is the core wall 2 and the tensile reinforcement close to axis 5 on the ground floor level. Hence, one step back is taken and the model with the 4 extra storeys is the one that defines scenario 1.



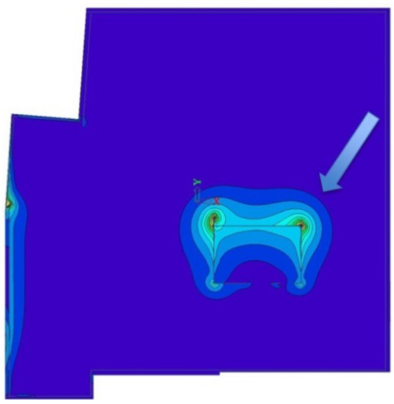


Figure 6-27 (Left) Rotation core (Right) Location punching shear check under core - Scenario 1

The influence of the addition of the extra floors on the unity checks of the critical elements are presented in the following tables. It should be clarified that punching shear for the core is checked at the intersection of walls 3 and 4 (Figure 6-27). The input and output of the FE model in Scia Engineer (v.15.2) are included in the engineering report that is presented in Appendix D.3.

Table 6-24 Critical unity check bottom reinforcement foundation plate - Vertical extension +4 levels – NEN8700 (FEM)

	Applied	Ultimate	Unity Check	Unity check +3 levels (Table 6-22)
Core wall 3	510 kNm	726 kNm	0,70	0,72
Core wall 4	661 kNm	865 kNm	0,76	0,67
Column K15	5697 kN	7800 kN	0,73	0,73

Table 6-25 Critical unity checks tensile reinforcement ground floor level core walls - Vertical extension +4 levels (FEM)

	Ultimate $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity Check	Unity check +3 levels (Table 6-20)
Wall 1 - y-y'	1,821	0,850	0,47	0,18
Wall 2 - y-y' (A)	2,428	0,915	0,38	0,11
Wall 2 - y-y' (C)	4,213	0,610	0,14	0,10
Wall 2 - y-y' (D)	6,856	6,170	0,90	0,06
Wall 3 - y-y'	1,943	1,670	0,86	-

Table 6-26 Critical unity checks concrete C16/20 compressive strength basement level core walls and stability wall axis 1 - Vertical extension +4 levels

	Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check	Unity check +3 levels (Table 6-21)
Core wall 1		10,667		0,48
Core wall 3		10,667		0,54
Core wall 4	8,485	10,667	0,80	0,59
Wall axis 1	5,034	10,667	0,47	0,26

Table 6-27 Unity check punching shear core and column K5 – Vertical extension +4 levels

	V_{Ed} (kN)	$V_{Rd,c}$ (kN)	Unity Check
Core	2218	4000	0,55
Column K5	5679	8100	0,70

Table 6-28 Critical unity checks columns ground floor - Vertical extension +4 levels

	$F_{applied}$ (kN)	$F_{ultimate}$ (kN)	Unity Check	Unity check current state (Table 6-5)
Column K5	4889	5403	0,90	0,70
Column K11	3894	5403	0,72	0,59
Column K12	4460	5403	0,83	0,66

6.5.4 Structural design new block

The structural design of the new part is the one chosen after a parametric analysis in section 6.4.1. In Figure 6-28 the floor plan of the 8th floor is presented, as an example, with the profiles of the steel structure as resulted from the structural calculation. The dimensioning of the steel columns is done according to the design loads of some normative columns for each floor. These design loads are retrieved from the analysis of the Scia model and are presented together with the corresponding sections and the extensive version of the checks in Appendix D.4.

Proceeding to the calculation of the steel beams, these carry the loads applied on the floors and transfer them to the columns. Provided that the structural configuration does not vary for the different levels, the computation of the steel beams is performed for a typical floor. The principal beams that are checked can be seen in Figure 6-28. For the extended computations see Appendix D.4.

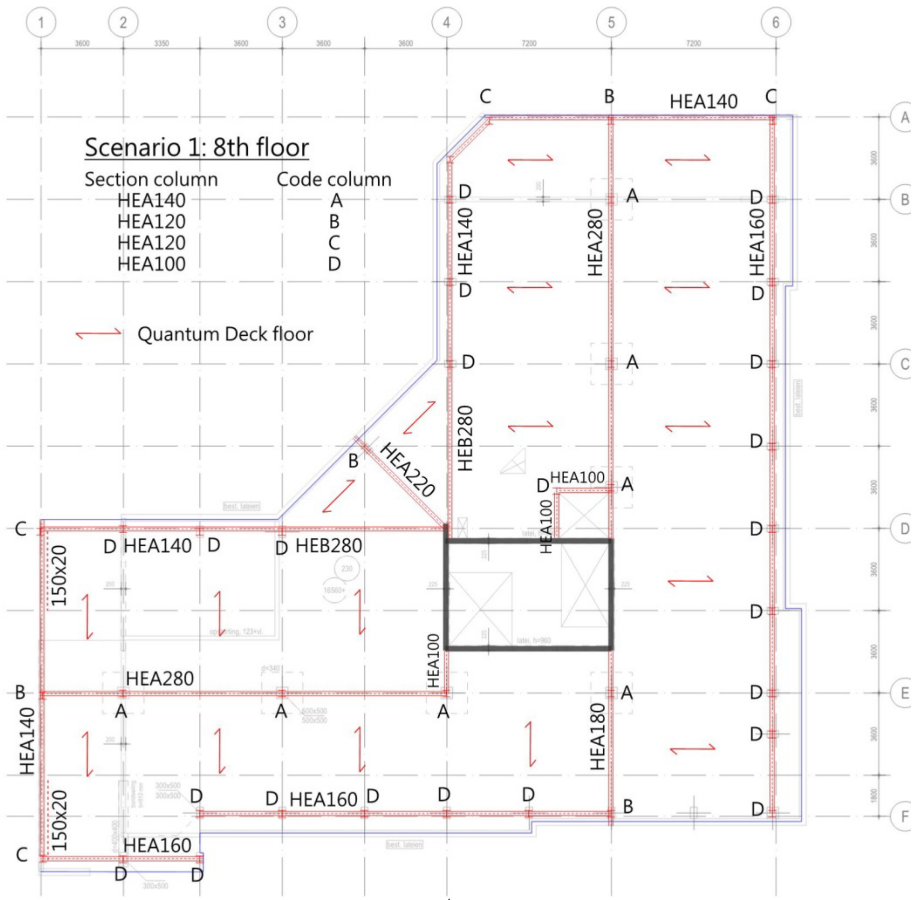
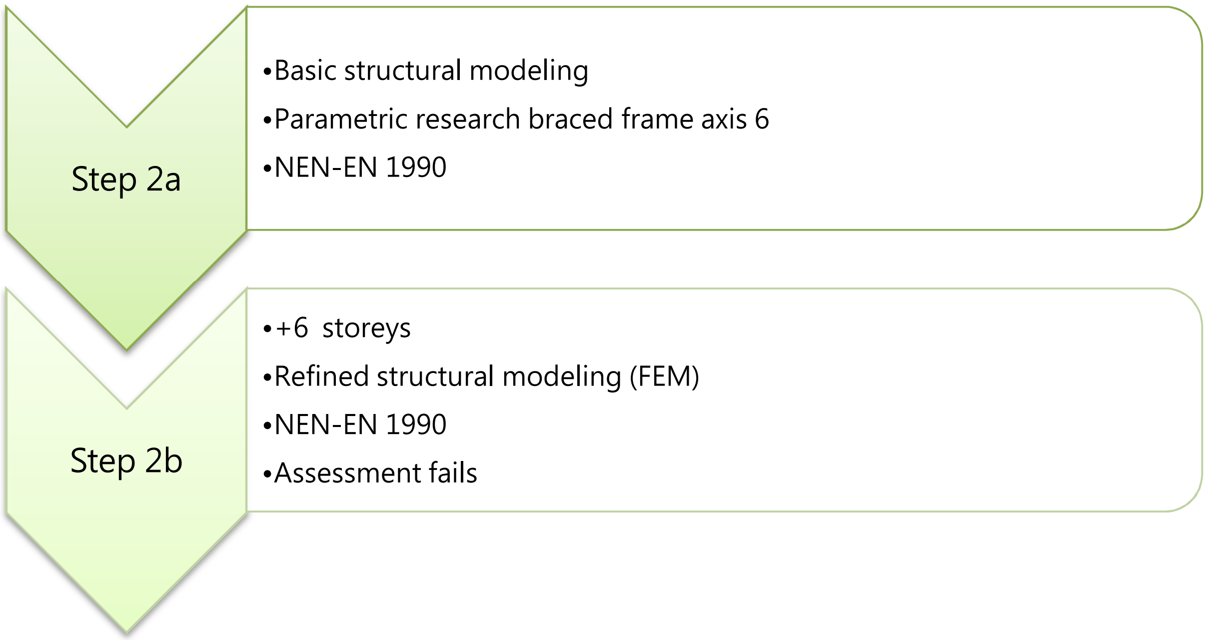


Figure 6-28 Structural floor plan 8th floor - Scenario 1 - Steel structure

6.6 SCENARIO 2



6.6.1 Basic structural assessment

Scenario 2 includes the introduction of a third stability element per direction with the intention to alleviate the concrete core and distribute the lateral loads more equally over the existing foundation. For the normative direction of the wind, that is also studied more in depth with the use of a FEM model, wind bracings are introduced at the position of axis 6 and together with the existing concrete columns a composite framework is formed. This extensive intervention in the existing structure makes, for this scenario, the application of NEN 8700 not possible. The design has already different features, and is not anymore a simple renovation. Thus, it is approached as a new building and the partial factors according to NEN-EN 1990 are applied.

Again, a parametric analysis is carried out to conclude to the configuration of the new framework. This analysis takes one step back, leaving for a while the FEM model and employing 2D basic structural models. The different configurations that are studied are presented in Figure 6-29. The selection is based on the stiffness, the displacement and the optimal distribution of the lateral loads over the existing columns. Variant 4 seems to match more to the desired features and is therefore applied in the design of scenario 2. In Appendix E.2 the variant analysis is further developed.

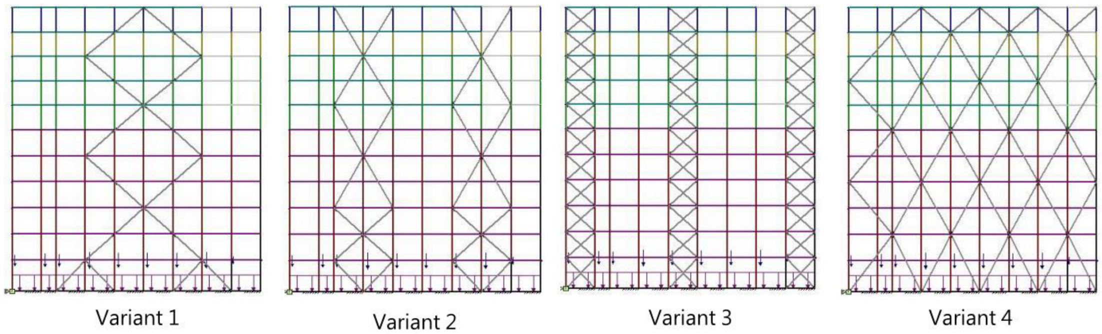


Figure 6-29 Variant study framework axis 6 - Scenario 2

6.6.2 Advanced structural assessment

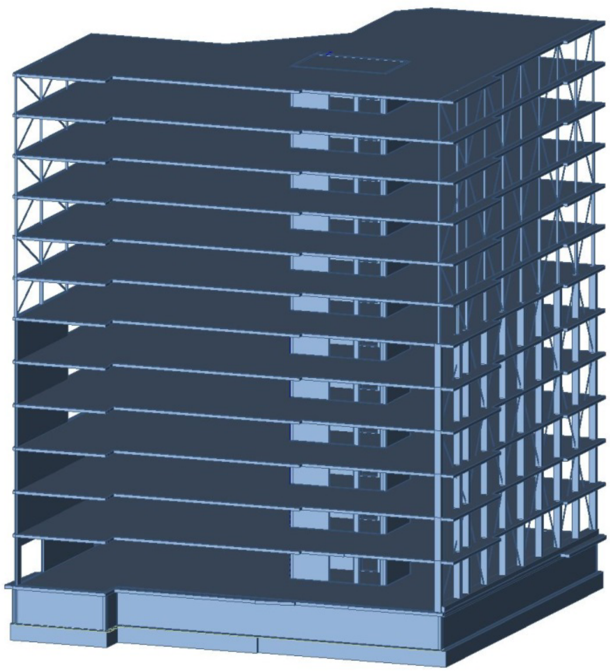


Figure 6-30 FEM model stability elements Scenario 2

Provided the complexity of the structural system after the addition of the third stability element, mostly regarding the distribution of the wind loads, the analysis for scenario 2 is carried out directly in Scia Engineer. Defining the stiffness of the new framework is an iterative process and hand calculations can be very time consuming. The aim of this scenario is again to find the critical elements that fail first. These elements will be the ones to be strengthened in scenario 3.

The addition of 2 more storeys (+6 for the total vertical extension) sets the limit for scenario 2. The critical elements for this scenario are the foundation plate and the concrete core. For the former, the bottom reinforcement is just sufficient for the moment that is present under the intersection of walls 3 and 4, and for the latter, the compressive stresses, that almost exceed the maximum compressive strength of concrete C16/20. In the following tables the unity checks of the critical elements are presented, the way they are configured for Scenario 2. The engineering report created during Scia structural analysis is to be found in Appendix E.3.

Table 6-29 Critical unity check bottom reinforcement foundation plate - Vertical extension +6 levels – (FEM)

	Applied	Ultimate	Unity Check	Unity check +4 levels (Table 6-24)
Core wall 3	572 kNm	726 kNm	0,79	0,70
Core wall 4	828 kNm	865 kNm	0,96	0,76
Column K15	6350 kN	7800 kN	0,81	0,73

Table 6-30 Critical unity checks tensile reinforcement ground floor level core walls - Vertical extension +6 levels (FEM)

	Ultimate $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity Check	Unity check +4 levels (Table 6-25)
Wall 1 - y-y'	1,821	0,502	0,28	0,47
Wall 2 - y-y' (A)	2,428	0,449	0,18	0,38
Wall 2 - y-y' (C)	4,213	-	-	0,14
Wall 2 - y-y' (D)	6,856	2,3	0,34	0,90
Wall 3 - y-y'	1,943	0,382	0,20	0,86

In the previous table it is noticeable the positive influence of the addition of the extra stability element on axis 6 on the tensile forces that were present in scenario 1 in the core walls. The addition of the framework in combination with the increase of the vertical loads decreased the tensile stresses.

Table 6-31 Critical unity checks concrete C16/20 compressive strength basement level core walls and stability wall axis 1 - Vertical extension +6 levels

	Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check	Unity check +4 levels (Table 6-26)
Core wall 1	3,98	10,667	0,37	
Core wall 3	10,12	10,667	0,95	
Core wall 4	11,77	10,667	1,1	0,80
Wall axis 1	6,315	10,667	0,60	0,47

In Table 6-31 it can be seen that the compressive stresses in core wall 4 is just over the limit of the maximum allowable stress for concrete C16/20. However, this small excess is acceptable with regard to the assumption that the concrete is still of quality C16/20, which is conservative considering that concrete gets stronger over the years. In the same direction, the unity checks of columns K5 and K12 are accepted (

Table 6-33).

Table 6-32 Unity check punching shear core and column K5 – Vertical extension +6 levels

	V_{Ed} (kN)	$V_{Rd,c}$ (kN)	Unity Check	Unity Check +4 levels (Table 6-27)
Core	2940	4000	0,74	0,55
Column K5	6350	8100	0,78	0,70

Table 6-33 Critical unity checks columns ground floor - Vertical extension +6 levels

	$F_{applied}$ (kN)	$F_{ultimate}$ (kN)	Unity Check	Unity check current state (Table 6-28)
Column K5	5400	5403	1,0	0,90
Column K11	4806	5403	0,89	0,72
Column K12	5246	5403	0,97	0,83

The previous tables indicate which are the critical elements that have to be strengthened in scenario 3 in order for the structure to be able to carry the extra loads resulting from the addition of extra storeys.

6.6.3 Structural design new block

The main principle of the structural configuration for the vertical extension part of scenario 2 does not differ from the one of scenario 1. The main difference is the addition of the wind bracings in axes 6 an F. As an example, the floor plan of the 8th floor is presented in Figure 6-31. All the floor plans of this scenario can be found in Appendix E.4.

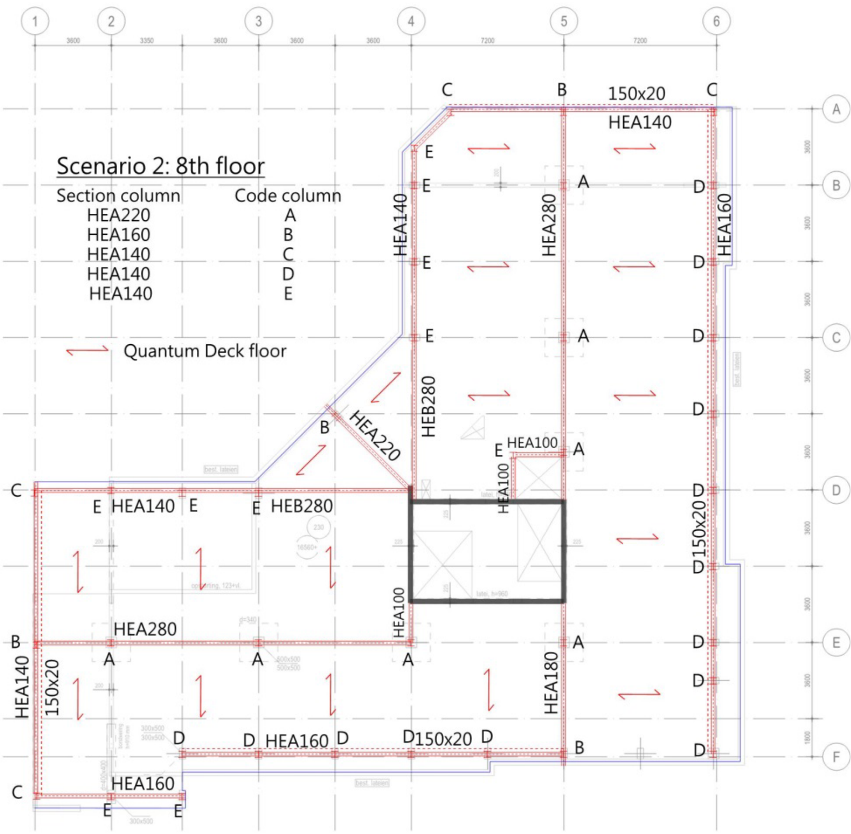
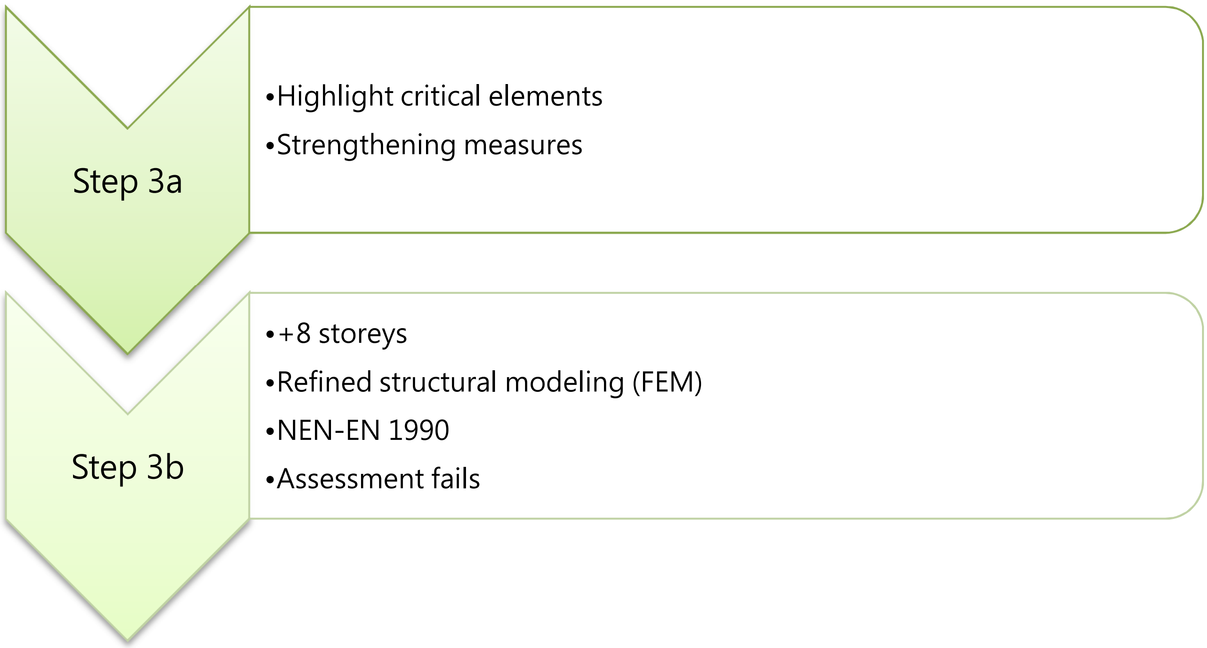


Figure 6-31 Structural floor plan 8th floor - Scenario 2 - Steel structure

6.7 SCENARIO 3



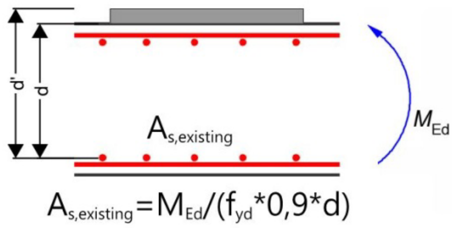
6.7.1 Strengthening existing structure

The main principle of scenario 3 is to have the maximum extra floor levels with the minimum interventions in the existing structure. Considering that construction costs is one of the parameters that will be studied in the next chapters in order to define the optimal vertical extension, the rationale is to keep a balance between the strengthening measures and the number of extra floors. Undoubtedly, there are advanced methods, as highlighted in the literature review, that can be applied in order to improve the performance of the existing structure in a great extent. However, this does not follow the principles of costs minimization and minimal intervention, according to which an engineer shall try to make the best out of an existing structure.

In the same direction, as prescribed in ISO 13822 (2010) regarding the difference between the economic considerations of the assessment of existing structures and the design of new structures "the cost between acceptance and upgrading the existing structure can be very large, whereas the cost of increasing the safety of a structural design is generally very small, consequently conservative generic criteria are used in design but should not be used in assessment". As a result, it is decided to strengthen only the elements that fail or were critical in scenario 2. These elements can be found looking back from Table 6-29 up to Table 6-33.

For the selection of the strengthening techniques it is only needed to go back to section 3.10 of the literature review. The measures are summarized in the following and the rough calculations are presented in Appendix F.2.

- Steel jacketing is used for the strengthening of the concrete columns K5 and K12 on the ground floor level. Four steel angle profiles 70x70x7 are applied on the 4 corners of the rectangular columns. The composite columns are calculated according to the design load that resulted from the addition of 2 more floors.
- Reinforced concrete jacketing is used for the concrete core. The idea is to lower the stresses or at least keep them under the 10,67 N/mm2. Regarding the increasing internal forces caused by the higher vertical and horizontal loads, one solution is to also increase the section of the walls ($\sigma = F/A$). Therefore, the concrete walls of the basement and the ground floor are thickened by 10cm with a reinforced concrete layer.
- The formula used to calculate the bending reinforcement under a certain bending moment reveals the solution for the insufficient bottom reinforcement of the foundation plate. Increasing the thickness of the plate, "d" increases and higher design moments can be carried by the same reinforcement. The thickness of the foundation slab is increased therefore by 20 cm reinforced concrete at the location of the concrete core.



- As a last measure, the core wall 3 is lengthened by 1 m, in order to alleviate the concentrated forces at the corner of core walls 3 and 4. This way the sectional area of the wall on its basis is increased and the stresses are staying low. This is also verified by the FEM model since the exact location of this extra wall is specified after some iterations.

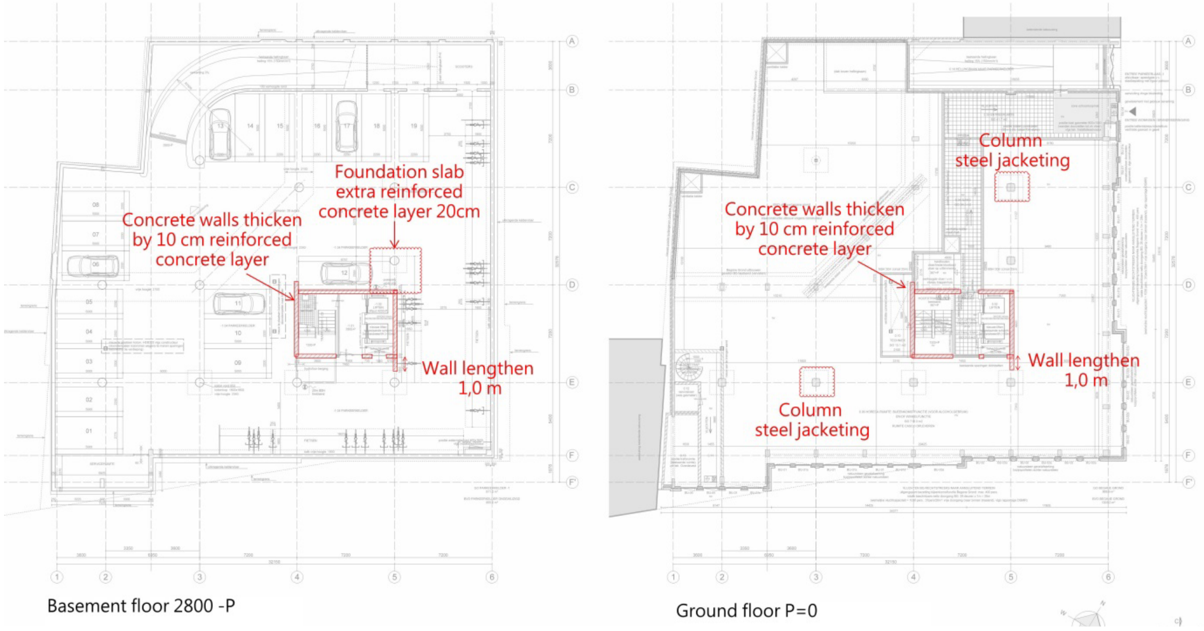


Figure 6-32 Strengthening measures on basement and ground floor - Scenario 3

6.7.2 Advanced structural assessment

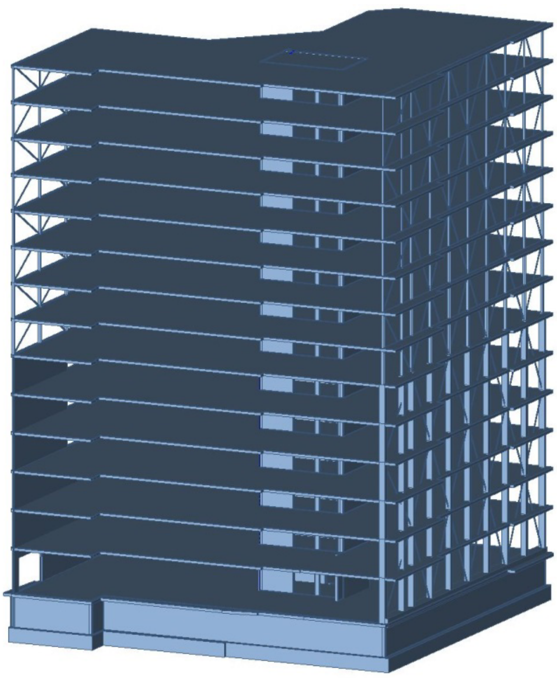


Figure 6-33 FEM model stability elements Scenario 3

Some of the strengthening measures are introduced in the FEM model, such as the increase of the thickness of the core walls and the lengthening of the core wall 3 by 1 m. Two more levels are added on top of the existing building, adjusting the vertical and wind loads and the analysis of the model shows that the concrete core reaches again the limit of the compressive strength. At this point it is decided to stop the addition of extra floors. Scenario 3 is already an extreme case considering that the total height of the building more than doubles. However, the concrete structure is proven to be stiff and strong enough to carry the extra loads. The unity checks of the critical elements are presented in the following and the engineering report from the Scia analysis is to be found in Appendix F.3.

Table 6-34 Critical unity check bottom reinforcement foundation plate - Vertical extension +8 levels (FEM)

	Applied	Ultimate	Unity Check
Core wall 3	679 kNm	726 kNm	0,94
Core wall 4	892 kNm	1015 kNm	0,88
Column K15	6816 kN	7800 kN	0,87

Table 6-35 Critical unity checks tensile reinforcement ground floor level core walls - Vertical extension +8 levels (FEM)

	Ultimate $\sigma_{tensile}$ (N/mm^2)	Applied $\sigma_{tensile}$ (N/mm^2)	Unity Check
Wall 1 - y-y'	1,821	0,947	0,52
Wall 2 - y-y' (A)	2,428	0,689	0,28
Wall 2 - y-y' (D)	6,856	0,335	0,05

Table 6-36 Critical unity checks concrete C16/20 compressive strength basement level core walls and stability wall axis 1 - Vertical extension +8 levels

	Applied $\sigma_{compressive}$ (N/mm^2)	Concrete $\sigma_{compressive}$ (N/mm2)	Unity Check
Core wall 1	3,715	10,667	0,35
Core wall 3	9,67	10,667	0,91
Core wall 4	10,9	10,667	1,02
Wall axis 1	7,2	10,667	0,67

Table 6-37 Unity check punching shear core and column K5 – Vertical extension +8 levels

	V_{Ed} (kN)	$V_{Rd,c}$ (kN)	Unity Check
Core	3647	4000	0,91
Column K5	6816	8100	0,84

6.7.3 Structural design new block

The calculations of the new sections for the columns of the vertical extension part are included in Appendix F.3. The design forces result from the FEM analysis for the columns on axes 1,6,A and F and from the calculation of the gravity loads for the rest.

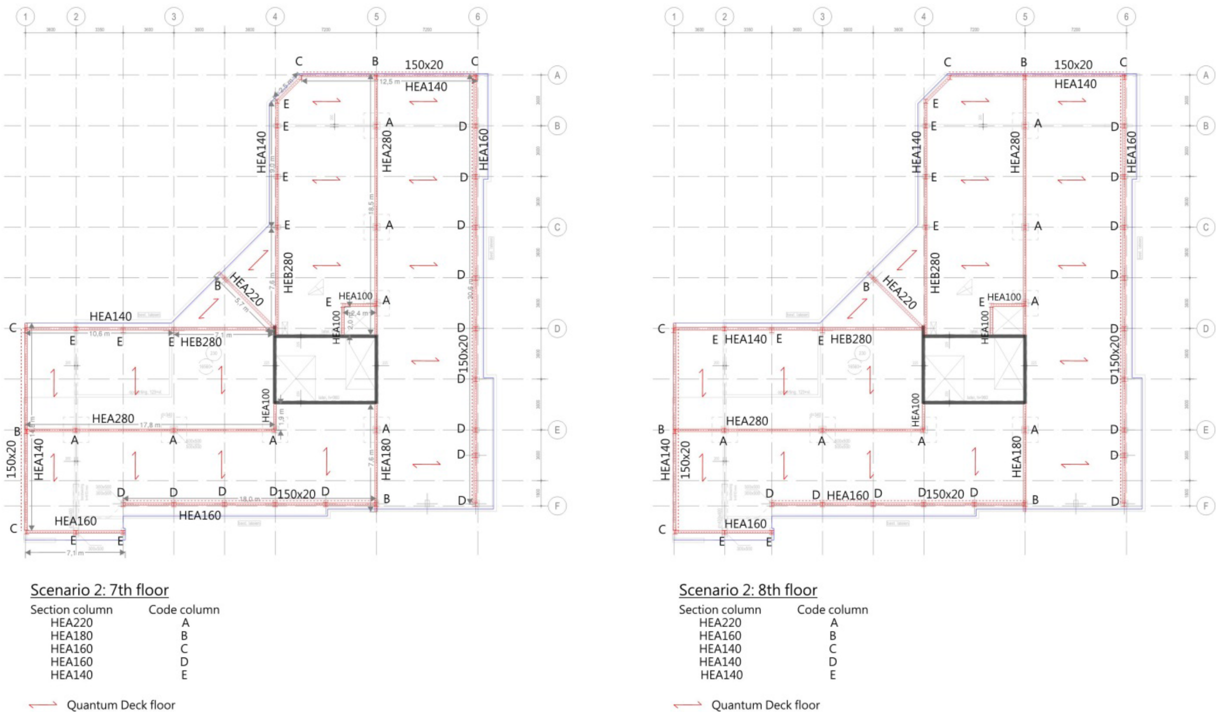


Figure 6-34 Structural floor plans 7th and 8th floor - Scenario 3 - Steel structure

6.8 DEMOLITION & NEW STRUCTURE

6.8.1 Introduction

One of the main goals of the current study is to research and compare the opportunities between the option of reusing and vertically extend an existing structure of a vacant building, and another alternative, that of demolishing the structure and building a new one instead. For the former design option, a preliminary structural design has been carried out in the previous sections of this chapter, considering that the possibilities for vertical extension are absolutely relative to the existing structure and its current state. Nevertheless, the preliminary structural design of a new structure can be performed on the basis of experience, normal practice and rules of thumb. The objective is to compare two realistic ways of approaching an existing structure, and in this sense, such assumptions for the design of a new structure are accepted. Once more four different scenarios are designed as depicted in Figure 6-35 and are to be compared on the basis of Environmental Cost Indicator (ECI) and construction costs with the scenarios of Figure 6-1.

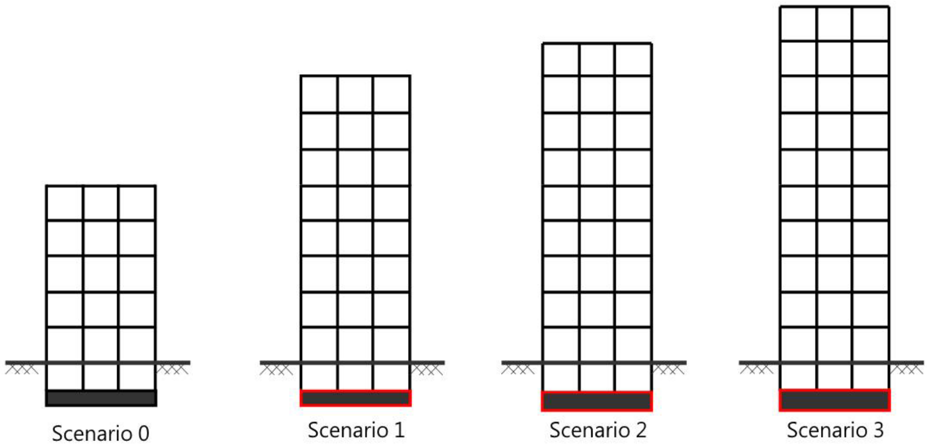


Figure 6-35 Schematic representation of the four design scenarios "Demolition & New structure"

6.8.2 Boundary conditions

Before introducing the four scenarios for the new structure and the new structural configurations, the boundary conditions are set down. The ECI and construction costs calculations are to be carried out on this basis of these prescribed conditions.

- ✓ Two alternative structural configurations are researched for the new structure. These are discussed in the following. As a short introduction, one structure comprises concrete slabs and concrete walls, and the other concrete slabs, beams and columns. From now on, these structure are referred as "new wall structure" and "new column structure" respectively.
- ✓ The preliminary structural design of a new structures is based on experience, normal practice and rules of thumb. Choices are made under the guidance of IMd Raadgevende Ingenieurs and *BAM Advies & Engineering*.
- ✓ Regarding the demolition of the existing structure, it is decided to maintain, and strengthen when needed, the foundation slab. Moreover, for the option of a new wall structure, the basement walls are maintained, and for the option of a new column structure all the structural elements of the basement are used except for the concrete core walls. The rationale behind these choices, is to make optimal use of the existing elements that were estimated to have relatively low unity checks.
- ✓ The main structural material of the new structures is reinforced concrete, as the common practice stipulates.

6.8.3 New structural designs

Two alternative structural designs are proposed for the new structure, both on the basis of the architectural design that has been suggested for the transformation of the existing building into student accommodation.

New "wall" structure

The first alternative follows literally the standard principles that are applied during the structural design of a new residential building, as indicated also by the engineers of IMd Raadgevende Ingenieurs. It is about a structure that consists of composite plank floor slabs, 290 mm thickness, and concrete load bearing walls of thickness 250 mm. The stability in both directions is provided by the load bearing walls and the concrete core, which hosts the stairs and elevators. The existing foundation is maintained, and for scenarios 1, 2 and 3 it is strengthened. A scheme with the principal structural configuration is shown in Figure 6-36. For a more in depth information about the structural elements, their physical properties and the strengthening of the existing foundation see Appendix H.1, in the global cost estimation, or Appendix G, in the ECI calculation.

Figure 6-36 Schematic structural configuration new wall structure

New "column" structure

The structure of the second alternative comprises the combination of beams and columns to transfer the vertical loads to the foundation. The stability is provided per direction by a stability wall and the concrete core. This structural system is the same as the one of the existing structure, however larger sections are provided in the new design. A typical floor plan is illustrated in Figure 6-37. For a deeper insight in the individual scenarios see Appendix H.1, in the global cost estimation, or Appendix G.1, in the ECI calculation.

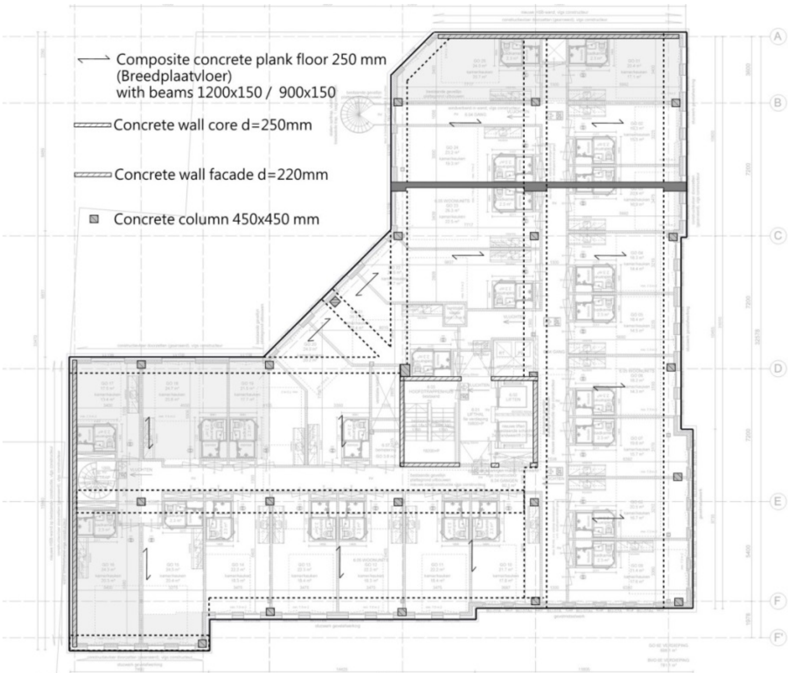
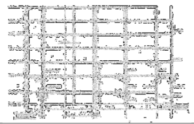
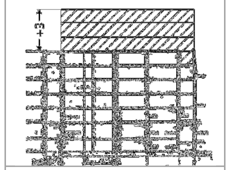
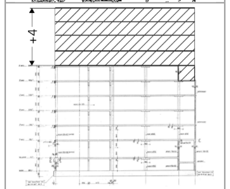
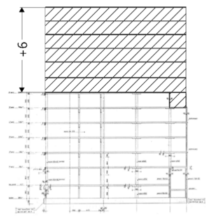
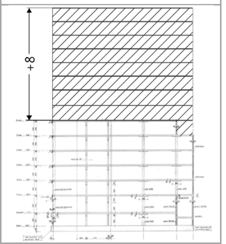


Figure 6-37 Schematic structural configuration new concrete column structure

6.9 CONCLUSIONS

In this chapter, the existing structure of Astoria building is studied, an estimation is done for the residual load bearing capacity of the existing structural elements and a preliminary structural design is carried out for the 3 vertical extension scenarios. This analysis is performed on the basis of two levels of structural assessment, basic and advanced. The critical structural elements are identified and studied more in depth regarding eventual strengthening methods. The key points of the preliminary structural analysis and the methodology that was followed to define the amount of extra storeys per scenario were:

- the introduction of the FE model, that revealed the possibility for one extra storey for scenario 1, and modelled in a more representative way, at first, the interaction between the main stabilizing elements and the foundation plate, and secondly, the distribution of the wind loads over the stabilizing elements, and
  the overcapacity of the existing structural elements, that was estimated based on the existing documentation.

Description	Level of structural assessment	Key factors	Conclusions	Scheme
<u>Scenario 0</u>: Analysis existing structure	Basic	– Calculations based on existing documentation – No testing methods used	– Existing structure over-dimensioned – NEN8700 creates extra margins – Critical elements: columns K5 and K12	
<u>Scenario 1</u>: Vertical extension, no interventions	Basic	– No interventions in the existing structure – Introductions of Finite Element model – Light-weight floor system of new block and smart structural configuration – Application of NEN8700	– Advanced structural assessment allowed for one more extra storey – Significant influence of the increased wind loads on the unity checks – Limited influence of the increased vertical loads on the unity checks – Critical elements: columns K5 and K12, concrete core and foundation at the location of the core	
	Advanced (FEM)			
<u>Scenario 2</u>: Vertical extension, limited interventions	Advanced (FEM)	– Addition of an extra stabilizing element – No strengthening of structural elements – Application of NEN1990 – Finite Element analysis	– Limited interventions result to the addition of 2 more storeys – When intervening in the existing structure apply NEN-EN1990 – Critical elements: columns K5 and K12, concrete core and foundation at the location of the core	
<u>Scenario 3</u>: Vertical extension, extended interventions	Advanced (FEM)	– Strengthening critical elements of scenario 2 – Principle of minimal intervention regarding economic considerations	– Strengthening of critical elements allows for more than doubled height	

The design parameters that influenced the vertical extension of Astoria and played an important role in configuring the scenarios and the corresponding number of extra storeys, are summarized in the following.

- The municipal policy, which is left out of the scope of the research. However, in the specific location there are restrictions regarding both the height of the building and the number of parking spaces provided, that limit the possibilities for vertical extension.
- The type of foundation limits the possibilities for strengthening and adjustments. In the first place, because of the small height of the basement level, and secondly, due to the increased construction costs that such an intervention includes. Therefore, the estimated ultimate load bearing capacity of the foundation is one of the critical factors.
- The transition from the basic level of structural assessment to the advanced. With the basic assessment the model failed already when having 3 extra storeys on top of the existing structure, whereas the advanced FE model it was possible to bring 4 extra storeys without any strengthening.
- The structural configuration of the new block was designed to make optimal use of the overcapacity of the existing structure.
- The floor system of the new block was chosen based on a parametric study. The weight, the height and the construction cost/m^2 are only some of the studied parameters that have a great influence on a vertical extension project.

Table 6-38 Design parameters of Astoria case study

Building name	Design parameters
Astoria	▪ Municipal policy ▪ Type of existing foundation (1,2 m thick plate) ▪ Advanced structural assessment (FEM) ▪ Structural configuration new block ▪ Floor system new block

7 ENVIRONMENTAL IMPACT

7.1 PURPOSE OF THE ASSESSMENT

The purpose of the assessment is defined by the goal and the intended use of the assessment. According to NEN-EN 15978:2011, the goal of the assessment is to quantify the environmental performance of the object of assessment, in particular of the structures of the 4 scenarios, by means of the compilation of environmental information.

The intended use of the assessment is to be used as assistance in order to identify the potential of reusing the structure of the existing building and vertically extent it. A comparison will be done of the environmental impact of the two different design options for the Astoria building:

- reuse and vertical extension, and
- demolition and new structure.

7.2 SPECIFICATION OF THE OBJECT OF ASSESSMENT

7.2.1 Functional equivalent

As NEN-EN 15978:2011 clearly prescribes, comparisons between the results of assessments of buildings or assembled systems (part of works) - at the design stage or whenever the results are used - shall be made only on the basis of their functional equivalency. This requires that the major functional requirements shall be described together with intended use and the relevant specific technical requirements. This description allows the functional equivalency of different options and building types to be determined and forms the basis for transparent and unbiased comparison.

In particular, the environmental assessments of two design options are to be compared. Both design options are prescribed by the same functional and technical boundary conditions, that are described in the following:

- design service life = 50 years (Class 3, Eurocode),
- intended function = residential (student accommodation),
- assessment's reference study period = 50 years,
- same building plot and dimensions,
- same acoustic and fire safety requirements for both floor systems, regarding the intended use of the building, and

– the column structure and the vertical extension block, include also separating walls, at the locations were the concrete walls of the new wall structure are designed, for the purpose of keeping the same functional unit between the design options.

The common reference unit to be derived from the functional equivalent and to be used to present the results of the indicators of the environmental assessment relative to the functional equivalent will be the Environmental Cost Indicator (ECI) for the total gross floor area (GFA). The ECI is also called shadow price (in Dutch: schaduwprijs).

7.3 SYSTEM BOUNDARY

The system boundary is set on the material level and concerns only the assessment of the materials that consist the structural skeleton of the two design options, i.e. reuse and vertical extension, and demolition and new building. For a new building, the system boundary shall include the building life cycle as shown in Figure 5-6. For an existing building (or part thereof), the system boundary shall include all stages representing the remaining service life, and the end of life stage of the building. In this context, the environmental impact indicator of the existing structure, that exists already for 33 years, shall be calculated only for the remaining service life, and this value shall be taken into account in the ECI computation of the design option of reuse and vertical extension of the building. Additionally, in case of demolishing and constructing a new structural skeleton, the remaining shadow price of the existing building shall be added in the total shadow price of the new design. The aforementioned rationale is depicted in Figure 7-1.

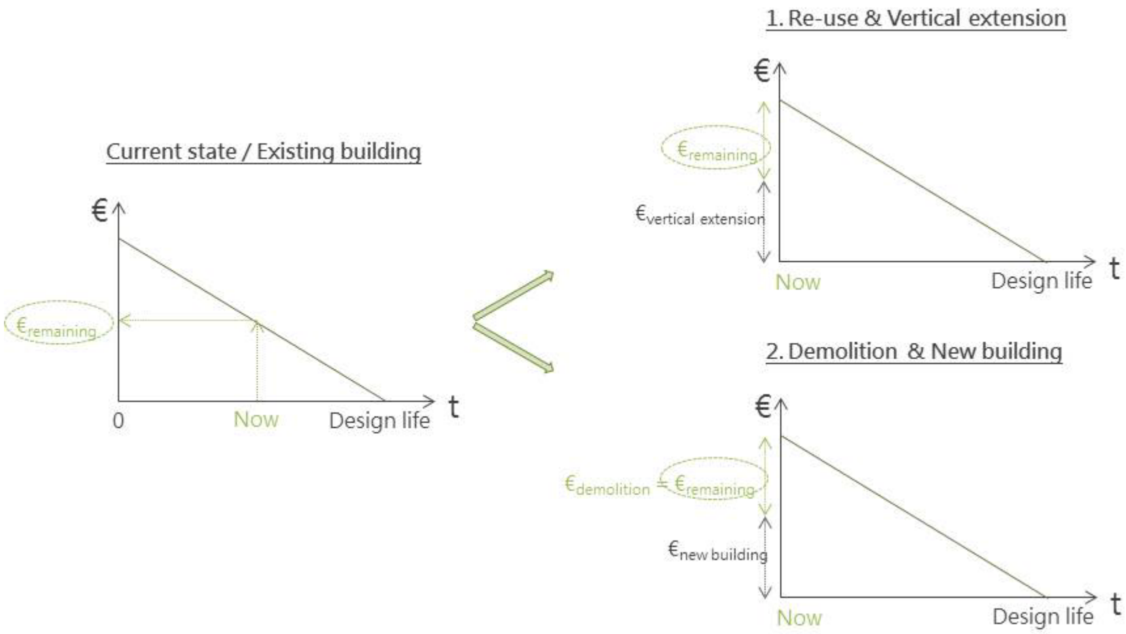


Figure 7-1 Environmental Impact Indicator existing structure

Provided this, it is concluded that the impact of the remaining shadow price of the existing structure is not of influence for the difference that might emerge in between the shadow prices of the two design options. The purpose of this analysis is primarily to eventually highlight the predominance of one of the options against the other, and in a second place the exact calculation of the Environmental Impact Calculation of these design options.

With this in mind, the object of the assessment is:

a. For the reuse and vertical extension design option:
   - the materials of the structure of the vertical extension part,
   - the materials used for the strengthening of the existing structure,
   - the materials included in the floor systems (mineral wool, ceiling gypsum boards, EPS etc.) that contribute to the comparison on the different design options based on the a common functional equivalent,
   - the materials of the separating walls where necessary according to the new wall structure.

b. For the demolition and new building design option:
   - the materials of the new structure, and
   - the materials used for the strengthening of the foundation plate and the rest structural elements that are assumed to be maintained,
   - the materials included in the floor systems (mineral wool, ceiling gypsum boards, EPS etc.) that contribute to the comparison on the different design options based on the a common functional equivalent,
   - for the new column structure, the materials of the separating walls where necessary according to the new wall structure.

It should be mentioned that the fire protection boards needed for the structural steelwork as well as the fire protection for the composite plank floors are not included in the calculation due to absence of data for the specific materials.

Moreover, due to limited access to the Dutch Environmental Database some simplification are made regarding the type of materials included in the LCA. It is assumed that all concrete elements have the same concrete quality C30/37. The exact material categories, their corresponding environmental impact effect categories and relative shadow costs are extensively presented in Appendix G.1.

7.4 QUANTIFICATION OF THE BUILDING MODELS

7.4.1 The building model

The input in order to carry out the environmental assessment of the two design options, is related to the building itself and its physical characteristics:

- the number of storeys,
- the storey height, and overall dimensions of the building plot,
- the shape and size of the structural frame (beams, columns, walls),
- the ceiling components that contribute to the acoustic requirements, regarding the residential function of the building.

For the design option of re-using and vertically extend the existing building, the physical characteristics of the building, i.e. mostly the dimensions of the steel structural frame and the floor system, are based on a preliminary structural design that has been presented in chapter 6. For the design option of demolition and new construction, these physical characteristics are defined by using rules of thumb and normal practices that are usually applied in the phase of preliminary structural design by structural engineers.

7.5 Dutch Environmental Database

In chapter 5 an extended review is done concerning the quantification of the environmental impact in the Netherlands, the environmental impact effect categories that are taken into account and the tools used to carry out the LCA. The Dutch Environmental Database contains a large amount of information regarding the quantification of the environmental impact effect categories for a big variety of building materials and products available in the Dutch market. This database is transparent only for licensees, and just a small amount of information is available online for free. However, some online tools, such as *MRPI Freetool* or *DGBC Materialentool*, are updated according to the last version of the database (version 1.7) and by making use of them it is possible to have indirect access.

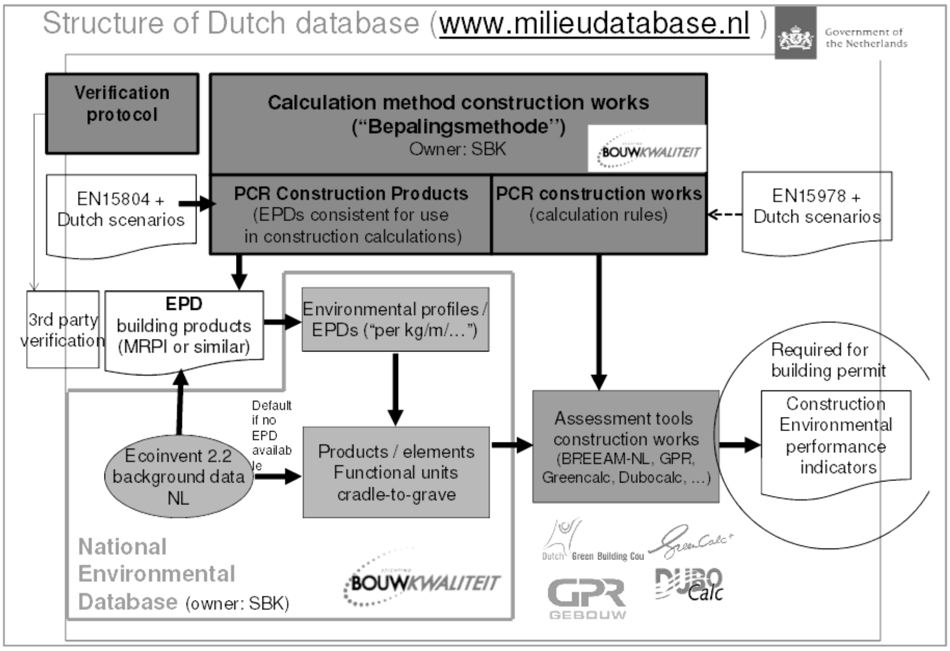


Figure 7-2 Structure of Dutch Environmental Database ((Agnes Schuurmans, 2014)

7.6 DuCo-tool

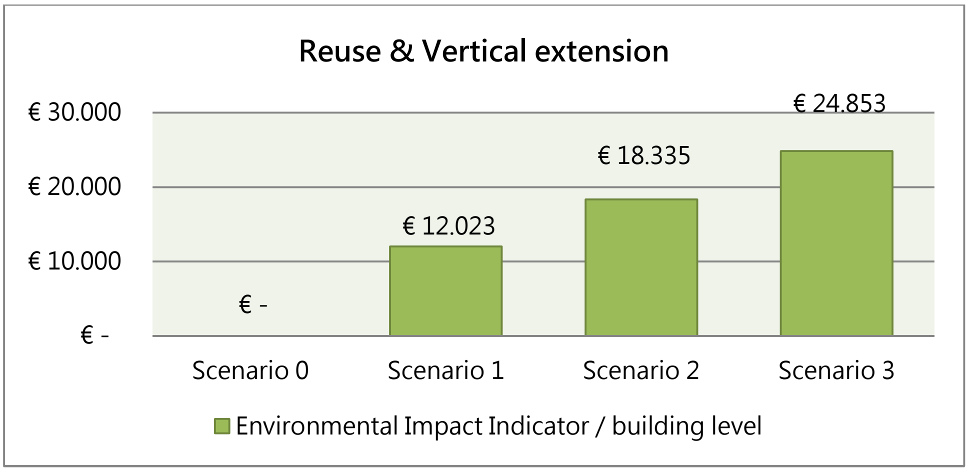
IMd Raadgevende Ingenieurs has developed a calculation tool that determines the shadow costs already in the preliminary design phase of a project. *DuCo-tool* is a calculation tool, material related, that provides the opportunity to compare the environmental impacts of various design options on the basis of numerical data. As it is previously mentioned, the Dutch Environmental Database contains the environmental impacts of various building materials and products that are used in the building industry, but only a small amount of information is available license-free. DuCo-tool is not directly linked to the last version of the licensed extensive database, however some of the missing information has been provided by the Dutch Institute for Building Biology and Ecology (NIBE). In this way, the LCA is based on pragmatic data and one can draw reliable conclusions.

The methodology followed for the computation of the environmental impacts of the scenarios and the design options is to be found in Appendix G.

7.7 Results & interpretation

7.7.1 Optimal vertical extension

The environmental impact assessment and the analysis of the results are expected to give an indication for the optimal vertical extension on the basis of the ECI/building level. Graph 7-1 summarizes the ECI for the 4 scenarios of reuse and vertical extension, with respect to the corresponding GFA, and Table 7-1 shows the ΔECI per extra level for the 3 steps. At first glance, the differences between the ΔECI per extra level for the 3 steps are not significant. There is indeed an increase, that for step 2 originates from the extra wind bracings added on axis 6 from the ground floor up to the 5th floor, and for step 3 originates from these wind bracings and the measures taken to strengthen the core, the foundation and the columns. This increase is also related to the larger steel profiles that are calculated due to the increase of loads. In terms of percentages, there is an increase of 5% between step 1 and step 2 and an increase of 3,3% between step 2 and step 3. These rates cannot be indicative for an optimal scenario. However, there is an explanation behind this results. The system boundary of the environmental impact assessment is set on material level, and the total quantities of material needed to strengthen the existing structure, steel or concrete, is very small compared to the material quantities of the main structure. As an example, the extra wind bracings that are introduced on axes 6 and F have a total length of 36,1m per floor, weight of 23,55kg/m, and therefore the total weight per floor is 850kg. The shadow cost of structural steel is 0,0294€/kg which gives an increase of the ECI for each floor of 25€. At the same time a whole new floor of the vertical extension part is calculated with an ECI of approx. 3.000€. Obviously, the values differ significantly and, no matter the added value of the extra framework on axis 6 for the structural behavior of the whole structure, from the ECI standpoint it is not possible to come to conclusions for an optimal solution.



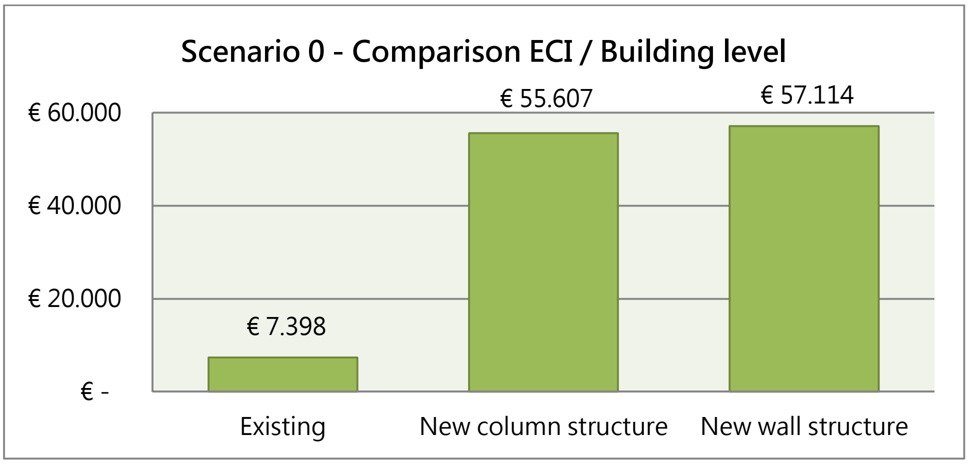
Graph 7-1 Environmental Cost Indicator / building level - Four scenarios "Reuse & Vertical extension"

Table 7-1 ECI per extra storey - Reuse and vertical extension

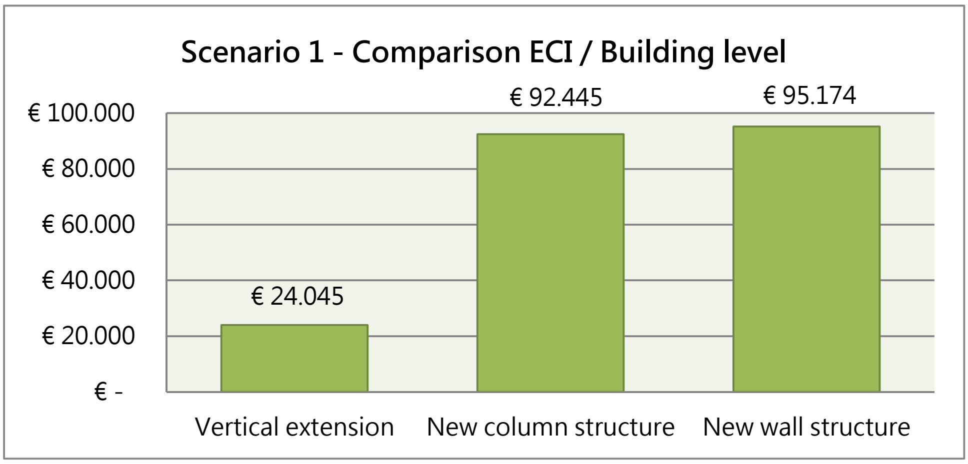
Step	Δ_{ECI}	Δ_{level}	Δ_{ECI}/extra level	
Step 1: Scenario 0 → Scenario 1	€ 12.023	4	€ 3.006	
				+ € 150
Step 2: Scenario 1 → Scenario 2	€ 6.312	2	€ 3.156	
				+ € 103
Step 3: Scenario 2 → Scenario 3	€ 6.517	2	€ 3.259	

7.7.2 Reuse & vertical extension vs Demolition & new structure

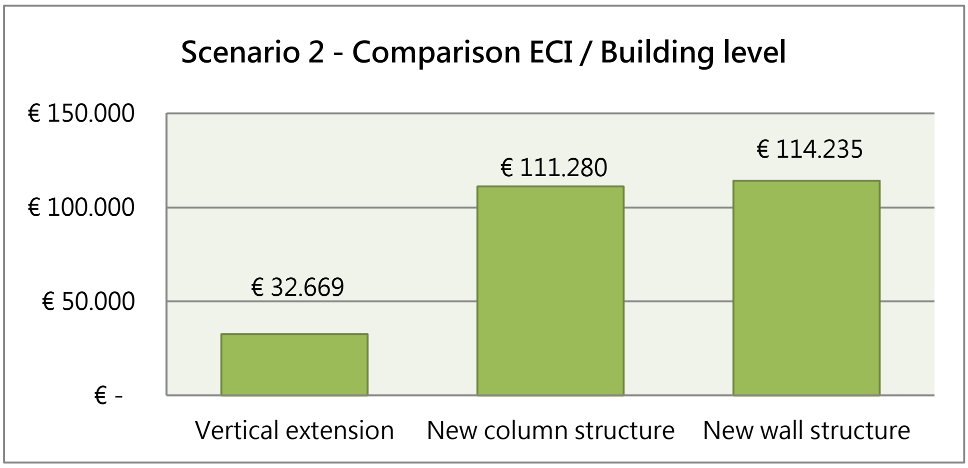
The next four graphs show the Environmental Cost Indicator (ECI) with respect to the gross floor area (GFA) of the building, comparing per scenario the ECI for the two options, i.e. reuse and vertical extension on the one hand, and demolition and new structure on the other. The new structures that are considered are the new column structure and the new wall structure. As an overall trend, it is clear that reusing the existing structure, with and without addition of extra storeys, results to much lower ECI values, compared to the option of demolition and new structure. In scenario 0, regarding the boundaries of the LCA that have been analyzed in section 7.3 and that do not include the material of the existing structure, one can justify the zero rate when reusing the existing structure. For scenarios 1, 2 and 3 reduced shadow costs are observed up to 75%, 71% and 69% respectively, when choosing reuse against demolition and new structure with walls. These rates vary ±5-10% when the ECI of the existing structure is taken into account. It is more than obvious, that the choice of reusing the existing structure of Astoria building, and eventually extending it with extra storeys, is environmentally the most conscious decision.



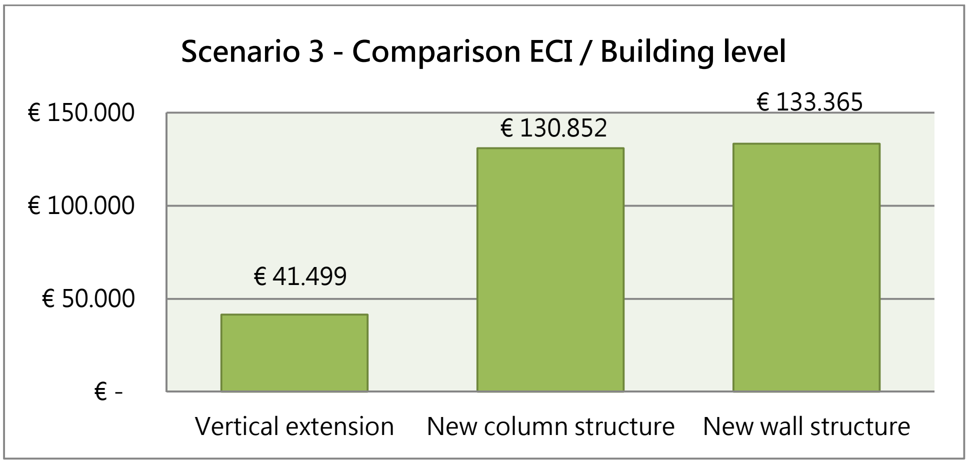
Graph 7-2 Environmental Cost Indicator / building level – Scenario 0 – Reuse vs Demolition & New structure"



Graph 7-3 Environmental Cost Indicator / building level – Scenario 1 – Reuse & Vertical extension vs Demolition & New structure"



Graph 7-4 Environmental Cost Indicator / building level – Scenario 2 – Reuse & Vertical extension vs Demolition & New structure"



Graph 7-5 Environmental Cost Indicator / building level – Scenario 3 – Reuse & Vertical extension vs Demolition & New structure"

8 CONSTRUCTION COSTS

8.1 INTRODUCTION

Reuse and vertical extension of an existing structure can become an interesting and attractive alternative for an investor as long as it is economically feasible. According to the report of the municipality of Amsterdam "The financial feasibility of transformation into student accommodation" the main factors that influence the financial feasibility are:

- ✓ the exploitation period,
- ✓ rental income,
- ✓ purchase income,
- ✓ building costs.

The building costs consist of the construction costs and the additional costs, such as purchase of the building plot, consultancy costs, interest costs and taxes. The construction costs constitute the majority of the building costs, and consequently, play an important role in the decision-making of a project.

Undoubtedly, the building industry has to be more environmentally conscious, however, no one can neglect the fact that the financial factor has a leading role. Therefrom motivated, an analysis is carried out for the construction costs of the two main design options and their individual scenarios. More precisely the design options to be studied are:

- reuse and vertical extension (four scenarios),
- demolition and new structure
  - new structure with load bearing walls (four scenarios)
  - new structure with beams and columns (four scenarios).

Moreover, the rental income is embraced in the cost analysis and based on that, a global estimation is done of the maximum allowable investment regarding the structure of every extra storey. This is done using the Gross Yield index (in Dutch: Bruto Aanvangsrendement) which is 5,25% for the Hague (Syntrus Achmea, 2015) and expresses the ratio between the rental income and the total investment. The comparison of the maximum investment to the construction costs is determinant for the optimal option regarding the vertical extension.

One of the priorities of the current thesis is to have a direct link with the market and the construction world, and to try to face the aspects related to vertical extension projects on a realistic basis. In this direction, the global estimation of the construction costs is carried out in collaboration with one of the leader companies in the construction sector, *BAM Advies & Engineering*.

8.2 BOUNDARY CONDITIONS

It should be clearly mentioned that the calculation of the costs and revenues is a global estimation that is based on the normal practices, techniques and average cost rates. The boundary conditions of this estimation are listed hereinafter.

- In order to be able to compare the different scenarios, a functional unit is defined. As an example, the concrete walls of the new wall structure are also functioning as separating walls. To compensate this function in the design of the new column structure and the design of the vertical extension, separating walls are incorporated.
- The estimation of the construction costs refers exclusively to the construction of the structural skeleton and the elements that are included in the design for the purpose of keeping the same functional unit. Hence, the labor, machinery, structural elements and engineering taken into consideration in the calculations are only related to the construction of these elements.
- All the information and cost rates for the estimation of the construction costs are provided by *BAM Advies & Engineering* and serve the purpose of a global estimation in the beginning of a project. More information about the rates can be found in Appendix H.1.
- The quantities of the materials and dimensioning of the structural elements of the "new-built" scenarios are based on experience and estimation according to the normal practices in the field of building engineering.
- The quantities of the materials and dimensioning of the structural elements of the "vertical extension" scenarios are based on the preliminary structural design that has been presented in Chapter 6.
- The monthly rent is calculated based on the program of requirements designed from the architect of the project for a typical floor.

The two main purposes of the cost analysis are mentioned and analyzed in the following.

1. To define the optimal vertical extension. For this, the total construction costs of the 4 scenarios of "reuse and vertical extension" are analyzed and expressed per added floor. The fluctuation of these costs and the comparison to the maximum allowable investment per added floor, as a result of the rental income and the Gross Yield index, define the optimal vertical extension. In these costs, the separating walls are not included, considering they do not contribute to the functional unit.

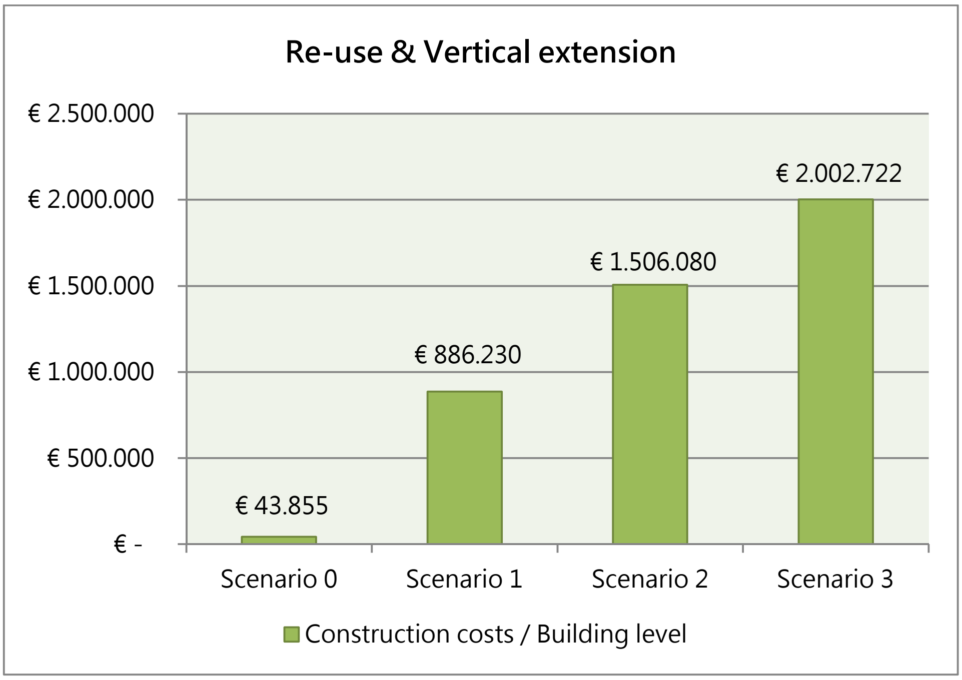
2. To compare the design option of reuse and vertical extension versus the option of demolition and new structure, when dealing with an existing building. Here the functional unit embraces the separating walls in both the new column structure and the vertical extension.

8.3 Results & Interpretation

The analytical calculation that has been carried out for the estimation of the construction costs for all four scenarios for both design options is to be found in Appendix H.1. The computation of the maximum allowable investment per extra storey is also included in Appendix H.2 In this section the results are presented in graphs and tables to facilitate their interpretation.

8.3.1 Optimal vertical extension

Graph 8-1 is about the construction costs of the four scenarios of reuse and vertical extension. Overall, it can be seen that the costs increase along the increase of the total storeys of the building, as certainly expected. The step between scenario 0 and scenario 1 is the one with the greatest increase of costs, fact that can easily be explained if one considers that in scenario 0 the structure is just reused the way it stands at the moment. Moreover, it is very interesting to look how the costs range for the rest of the scenarios. In the transition from scenario 1 to scenario 2 an increase of 70% is observed in the costs, whereas the respective increase between scenarios 2 and 3 is not larger than 33%. The former sharp increase is mostly related to the interventions in the existing part of the structure and more specifically to the wind bracings that are introduced in axis 6.



Graph 8-1 Construction costs / building level - Four scenarios "Reuse & Vertical extension"

To define the optimal vertical extension, it is useful to look how the construction costs per extra floor fluctuate while adding storeys, and compare them to the maximum allowable investment per floor. Looking at Table 8-1, it is seen that during first step of the vertical extension, when the first 4 storeys are added on top of the existing structure, the construction costs per extra floor are 210.594€, whereas the respective costs for the second step are increased namely by 47%. The construction costs per extra floor of step 3 are increased by 18% compared to these of step 1, and are 20% less than step 2. This sharp increase of costs at step 2 results from the introduction of the wind bracings in the existing structure, and more specifically, from fixing, anchoring and mounting works. Comparing now the construction costs to the maximum allowable investment, per extra floor, one can see that scenario 1 is financially feasible, whereas stepping to scenario 2 is prohibitive, regarding the costs exceed the maximum allowable investment. However, if a bigger step is taken, from scenario 1 to scenario 3 (Step 2b, Table 8-1), the venture of vertical extension seems to be interesting again. This analysis actually shows that, intervening in the existing structure is such an expensive process that is only worth to be introduced in case 4 extra storeys are added, and not for the addition of just 2 storeys.

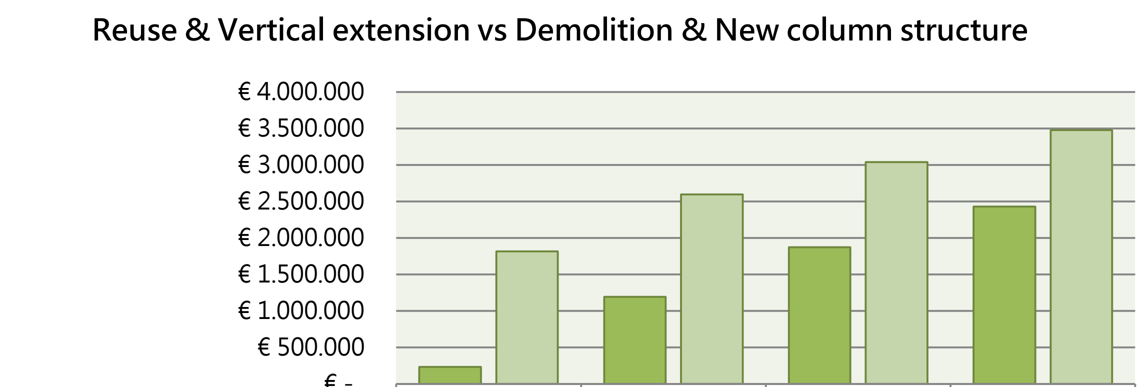
Consequently, for the Astoria building, and from the construction costs point of view, it is advised to stop the vertical extension at the addition of 4 extra storeys. Taking into consideration the margin between the construction costs and the maximum allowable investment for the various steps, and the uncertainty hidden behind a global estimation, scenario 1 is considered as the optimal option, and scenario 3 is deliberated as an interesting option. For all scenarios it is recommended to include also a more sophisticated financial feasibility and risk analysis in the process, so as to conclude for the viability of the project.

Table 8-1 Construction costs and maximum allowable investment per extra storey - Reuse and vertical extension

Step	Δ_{costs}	Δ_{level}	Construction costs	Investment
Step 1 : Scenario 0 → Scenario 1	€ 842.375	4	€ 210.594	€ 299.200
Step 2a: Scenario 1 → Scenario 2	€ 619.850	2	€ 309.925	€ 292.000
Step 3 : Scenario 2 → Scenario 3	€ 496.642	2	€ 248.321	€ 292.000
Step 2b: Scenario 1 → Scenario 3	€ 1.116.492	4	€ 279.123	€ 292.000

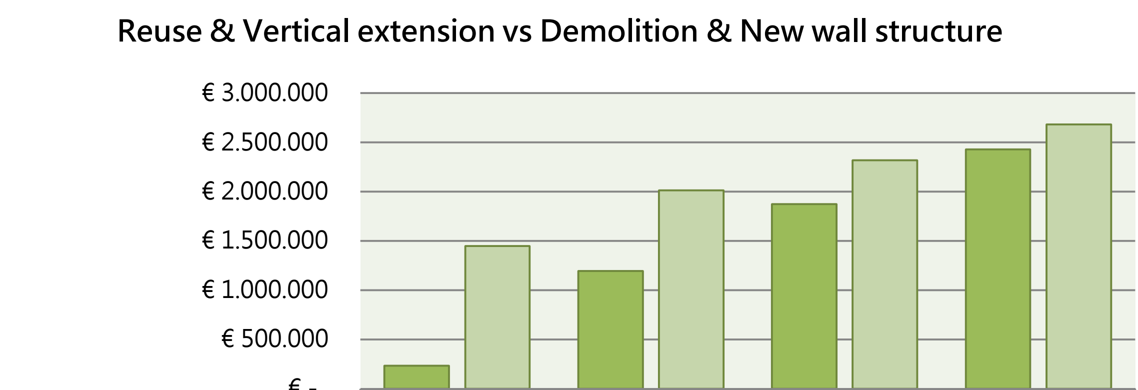
8.3.2 Reuse & vertical extension vs Demolition & new structure

The following two graphs, Graph 8-2 and Graph 8-3, compare the two main design options for the existing structure of Astoria building, namely reuse and vertical extension versus demolition and new structure. Both graphs clearly show that reusing the existing structure, with or without vertical extension, is more profitable in terms of construction costs, regardless the type of new structure. More specifically for the vertical extension options, scenario 1 seems to be the most advantageous, with 54% less construction costs for reuse against a new column structure, and 41% less costs against a new wall structure. These rates reduce to 38% and 19% respectively for scenario 2, and 30% and 9% for scenario 3. These reductions arise from the structural interventions and strengthening at the existing part of the structure, which are necessary for the realization of scenarios 2 and 3, and increase considerably the construction costs. Especially from the comparison between vertical extension and new wall structure (Graph 8-2), it is obvious that the higher the vertical extension the less interesting and advantageous reuse becomes, compared to demolition and new structure. Consequently, for scenarios 0 and 1, reusing the existing structure of Astoria building is certainly advised. Scenarios 2 and 3, also reveals some economic benefits for reuse and vertical extension, however, it is suggested to research further these options, regarding the risk of such a project.



	Scenario 0	Scenario 1	Scenario 2	Scenario 3
Re-use & Vertical extension	€ 233.696	€ 1.194.722	€ 1.873.897	€ 2.429.864
Demolition & New column structure	€ 1.814.201	€ 2.598.658	€ 3.039.997	€ 3.477.178
Save of reuse %	87%	54%	38%	30%

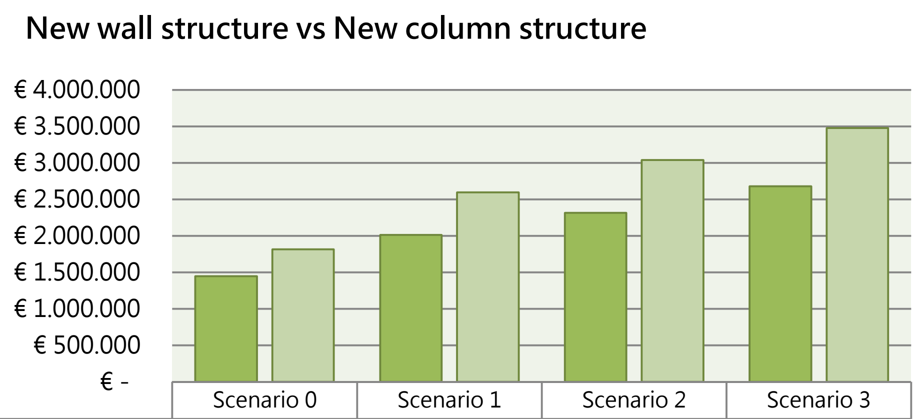
Graph 8-2 Comparison construction costs / building level – Four scenarios – Reuse & vertical extension vs Demolition & New column structure



	Scenario 0	Scenario 1	Scenario 2	Scenario 3
Re-use & Vertical extension	€ 233.696	€ 1.194.722	€ 1.873.897	€ 2.429.864
Demolition & New wall structure	€ 1.449.053	€ 2.012.821	€ 2.317.443	€ 2.681.011
Save of reuse %	84%	41%	19%	9%

Graph 8-3 Comparison construction costs / building level – Four scenarios – Reuse & vertical extension vs Demolition & New wall structure

At last, a comparison is made between the construction costs of the two design options for the new structure. As it is observed in Graph 8-4, the design of a new wall structure saves approximately 22% of the construction costs compared to a new column structure. This difference arises from the higher construction costs of the slab with the reinforced strips (hidden beams) in the case of a column structure, as well as the extra separating walls, that are taken into account for the sake of comparing the options on the basis of the same functional unit.



	Scenario 0	Scenario 1	Scenario 2	Scenario 3
Demolition & New wall structure	€ 1.449.053	€ 2.012.821	€ 2.317.443	€ 2.681.011
Demolition & New column structure	€ 1.814.201	€ 2.598.658	€ 3.039.997	€ 3.477.178
Save of wall structure %	20%	23%	24%	23%

Graph 8-4 Comparison construction costs / building level – Four scenarios – New wall structure vs New column structure

Conclusions

9 CONCLUSIONS & RECOMMENDATIONS

9.1 CONCLUSIONS

9.1.1 Process summary

The aim of this thesis project was to research the design option of vertical extension in building renovation from the structural engineer's standpoint. An integrated approach was adopted to answer the main research question and the sub-questions, and different subject areas were crossed. Focus was put, not only on the structural design, but also on the environmental impact and the construction costs of vertical extension projects. A cross-research is carried out in order to answer and verify the main research question on the basis of different sources of information; i.e. literature review, interviews, existing case studies, and at last the verification via a new case study.

Step	Objective
1. Literature research 2. Study of existing case studies + interviews	- *Vertical extension "state of the art" understanding* - *Parameters definition*
<u>Astoria case study</u> 3. Structural analysis 4. Environmental impact analysis 5. Costs calculation	- *Optimal vertical extension for Astoria building* - *Parameters definition* - *Useful directives for developers & decision-makers*

Figure 9-1 Process summary and objectives

The main research question and research objectives of the case study are presented hereinafter, along with the conclusions drawn as an outcome of the current thesis project.

9.1.2 Existing case studies conclusions

The design parameters that dominated during the design process of the five existing case studies are briefly summarized in Table 4-6. Reflecting on these parameters, one can notice that, in three out of five cases, municipal policies determined the decision about the vertical extension. It is in the opinion of the author, that this is a factor open to manipulation as well as negotiation. Vertical extension could be used as a solution for the big cities where only little, if not zero, unbuilt plots are available. The scientific research can help in this direction, by arguing about the benefits of this strategy. In any case, a different way of thinking and designing shall be adopted ahead the man-made climate change that has been unprecedented. Furthermore, the building's foundation played a decisive role in two out of five cases, which means that it is probably a factor to be considered for future projects. At last, in two out of the five case studies a feasibility study was carried out at the beginning of the project and revealed the most profitable option for the vertical extension regarding the economic considerations of the venture.

Building name	Design parameters
Karel Doorman	▪ Load bearing capacity of the foundation ▪ Risks taken; time and money invested for the light-weight solution ▪ Testing revealed higher concrete quality ▪ Alteration of structural system
Groot Willemsplein	▪ Timeframe restrictions ▪ Municipal policy; restrictions from the urban planning regulations (m^2 office area)
Westerlaantoren	▪ Type of existing foundation (2 m thick plate) ▪ Municipal policy; restrictions from the urban planning regulations (height of the building) ▪ Testing revealed higher concrete quality ▪ Feasibility study
Zeemanshuis	▪ Municipal policy; restrictions from the urban planning regulations (height of the building) ▪ Absence of data for the existing structure
St. Jobsveem	▪ National listed monument ▪ Feasibility study ▪ Testing revealed critical elements

9.1.3 Astoria case study conclusions

Below, the objectives stated at the beginning of the research are quoted. Each of them is then discussed.

"To investigate which load bearing components are most critical and require the most drastic strengthening measures when the amount of square meters is increased."

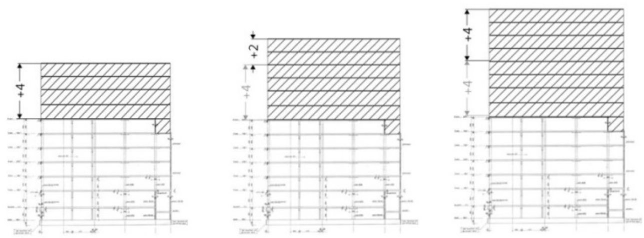
The structural analysis of the existing structure of Astoria focuses on the critical structural elements and on the influence of the addition of extra floors on the unity checks of these elements. The initial structural analysis of the existing structure revealed column K5 as the critical element with a unity check under compression close to 0,70. However, during the process of adding storeys on the existing structure the correlations changed. Both basic and advanced levels of structural assessment concluded that the concrete shear core and the foundation slab are the critical structural elements that are affected the most from the vertical extension. The former is critical due to the compressive strength of the concrete, and the latter fails because of insufficient bottom reinforcement. This is easily explained, considering the light-weight structure that is applied in the new part. The influence of the extra storeys on top of the existing structure on the unity check of the column is insignificant compared to the increase of stresses that result from the wind loads.

Hence, the first measures are taken to alleviate the concrete core and the foundation plate, at the location of the core, from the high stresses. This is achieved by introducing an extra stabilizing element per direction, distributing in such a way the increased wind loads more equally via three elements over the foundation plate. Nonetheless, the critical elements were still the same and failed again after the addition of 2 more storeys. More drastic measures are taken this time, such as enlarging the thickness of the concrete core walls and the thickness of the foundation slab as well. The key factor for all the interventions has been the principle of minimum intervention, regarding the disproportional economic burdens that the upgrading of an existing structure includes.

> *The shear concrete core and the foundation slab are the critical structural elements during the vertical extension of Astoria case study. The measures taken include introduction of an extra stability element and enlarging the thicknesses of the reinforced concrete elements.*

"To examine what is the influence of the addition of extra storeys on the environmental impact and the costs of the building and conclude to an optimal vertical extension for this specific building."

Regarding the material related environmental impact, the results from the study of the vertical extension scenarios are not helpful to draw conclusions about the optimal option. The impact of the extra materials that are added in the existing structure, either to change the structural system, or to strengthen existing structural elements, is very small regarding the overall GFA, and causes an insignificant increase of the ECI. On the other hand, the benefits that these interventions bring to the strength and stiffness of the structure are very important for the vertical extension.



	Scenario 1	Scenario 2	Scenario 3
Extra storeys	+4 storeys	(+4) +2 storeys	(+4) +4 storeys
ECI	€ 3.006	€ 3.156	€ 3.208
Construction costs	€ 210.594	€ 309.925	€ 279.123
Investment	€ 299.200	€ 292.000	€ 292.000

** All values are expressed per extra storey*

Figure 9-2 Optimal vertical extension for Astoria case study – Summary ECI & Costs

The cost analysis, on the other hand, reveals clear differences between the 3 vertical extension scenarios. Interpreting the results, scenario 1 is concluded to be the optimal option, regarding mostly the big margin between the construction costs and the maximum allowable investment per extra storey. Scenario 2 is reported as financially unviable, taken into account that the increased construction costs, resulted from the interventions in the existing structure, are not compensated by the revenues from the 2 extra storeys. At last, scenario 3 appears to be an interesting but very complex option, that embraces a lot of risk and uncertainly. Undoubtedly, a more sophisticated economic analysis is needed to conclude for the feasibility and the most optimal solution regarding the vertical extension of this building, considering that all scenarios are proved to have economic advantages compared to the respective scenarios of demolition and new structure.

Environmental impact and cost analyses reveal the addition of 4 storeys as the optimal vertical extension for Astoria building. Further vertical extension requires interventions in the existing structure, which makes the project economically unviable.

"To compare the two design options for the Astoria building, reuse and vertical extension on the one hand, and demolition and new structure on the other hand, in terms of environmental impact and construction costs, and indicate the best option."

The LCA and construction costs analysis for the two basic design options and their individual scenarios show evidently that reuse and vertical extension takes precedence over the option of demolition and new structure. From the Environmental Cost Indicator standpoint, the option of reuse can save more than 70% €/GFA, compared to the option of demolition and new structure. From the construction costs standpoint, the differences amongst the scenarios are more profound and differ also according to the design of the new structure, i.e. column or wall structure. Compared to a new column structure, reuse is calculated to save around 54% for scenario 1, while the same percentages for scenarios 2 and 3 are approx. 38% and 30% respectively. The construction costs of a new wall structure are estimated at lower numbers and therefore the corresponding aforementioned rates decrease to namely 41%, 19% and 9%. The last reveals, that for scenario 3 the construction costs of the reuse and vertical extension do not diverge much from the costs of demolition and new building, and regarding the risk of investing on an existing structure, a more in depth research is necessary. The higher the amount of interventions in the existing structure the higher the construction costs. From this it is concluded that when extensive interventions are required in the existing structure during vertical extension projects, the costs increase, and considering also the risk that is taken in such a project, further research is required before making decisions and critical choices.

Reuse and vertical extension results to more than 70% reduced environmental impact, compared to demolition and new structure. Regarding construction costs, the former comprises savings of 40%-50% when no interventions are required in the existing structure. Extensive interventions lead to construction costs closer to these of a new structure.

"Which design parameters defined the vertical extension in Astoria case study?"

The analysis of the existing building of Astoria and the research carried out for the vertical extension potentials, revealed a number of critical design parameters. These are reported in the following table.

Building name	Design parameters
Astoria	▪ Municipal policy ▪ Type of existing foundation (1,2 m thick plate) ▪ Advanced structural assessment (FEM) ▪ Structural configuration new block ▪ Floor system new block

9.1.4 General conclusions

"Which design parameters define the optimal vertical extension in building renovation with respect to costs and environmental impact?"

The process of defining a list of the design parameters, that play an important role in vertical extension of existing buildings, started by studying 5 already completed vertical extension projects. In order to get a better insight in the processes and every project, interviews are performed with the in charge structural engineers. The parameters distinguished from the example case studies were to be verified, or not, through the design of a case study. During the design of the vertical extension of Astoria case study, a second list of parameters emerged. In total 11 different parameters are distinguished from the example case studies and these are presented in the following table.

Table 9-1 Summary table of design parameters

Design parameter	Karel Doorman	Groot Willemsplein	Westerlaantoren	Zeemanshuis	St. Jobsveem	Astoria
Foundation	X		X			X
Floor system new block	X					X
Testing	X		X		X	
Alternative structural system	X					
Time restrictions		X				
Municipal policy		X	X	X		X
Feasibility study			X		X	
Absence of data				X		
National listed monument					X	
Advanced structural analysis						X
Structural configuration new block						X

As result of the current thesis project, a list of the five principal design parameters that define vertical extension in building renovation, studied from the structural engineer's standpoint, is shown in the following table, with a small reflection on each one.

Table 9-2 Principal design parameters of optimal vertical extension

Design parameter	Reflection
Municipal policy	The most common parameter that limits the possibilities for vertical extension. Undoubtedly, this parameter is subject to alteration. Local governments and municipal authorities should develop flexible policies that adjust to the changing needs of society.
Foundation	The type and load bearing capacity of foundation has been mainly a limiting factors for the case studies examined. Regarding shallow foundations with thick foundation plates, strengthening possibilities are very limited. A smart structural design can redistribute the loads more evenly over the foundation. That was a common practice in the studied case studies.
Testing	Testing methods are helpful tools during a vertical extension project. Especially when the optimum is researched. Absence of testing was a limiting factor for Astoria case study, whereas revealed hidden overcapacity for Karel Doorman and Westerlaantoren.
Floor system new block	A factor of great influence when researching the optimal vertical extension. A parametric analysis is recommended, considering the particularities of every project.
Feasibility study	The relationship between costs and value is, at the end, the critical factor in a vertical extension project. A feasibility study can give the optimum solution taken into account risk, construction costs, revenues, operational costs and other highly influencing factors.

Municipal policies, foundation, testing methods, floor system of the new block and feasibility study are concluded as the critical parameters that defined the optimal vertical extension in the researched case studies. For future projects, it is advised to introduce these parameters in the initial design stages, as bullet points to be studied, in order to investigate and optimize the possibilities for vertical extension.

"To draw some useful directives that could be used from developers and decision-makers in building renovation and adaptation projects when having to deal with structurally vacant office buildings, with regard to possibilities for vertical extension."

In the Netherlands, the large amount of structurally vacant offices is a challenge that the building market has to face and deal with in an efficient and optimal way. Sometimes, decisions are made on the basis of outdated practices, whereas, society and climate change demand for different approaches and innovative solutions. The current thesis, highlights the possibilities and perspectives of reusing and vertically extend structures of vacant office buildings that have not reached yet the end-of-life. The results of the Astoria case study are indicative for the possibilities and perspectives that may be in the field of renovation, and the main purpose is to highlight another option that might have multiple benefits for both developers and society. As an outcome of the current thesis, it is strongly advised to introduce vertical extension in the normal practices of approaching existing buildings. In the first place to carefully research the extra load bearing capacity of an existing structure and add the corresponding amount of floors, regarding the encouraging results of the Astoria case study for lower environmental impact and lower construction costs compared to a new structure. Afterwards, the analysis of different scenarios, as presented in the current thesis, can reveal the optimal solution for the particular project. The optimum depends undeniably on the special features of every structure and no rules can be defined to specify this. The path, however, to be followed is set and is open to further research and development.

It is strongly advised to introduce vertical extension as a normal practice in building renovation. Astoria case study revealed multiple benefits and perspectives for both developers and society on the basis of environmental impact and construction costs.

9.2 General remarks & Recommendations

9.2.1 Reliability of the process

The Astoria case study represents the most important step of this research. The aim of this step was to determine the relevant parameters that play a role in the process of designing optimal solutions for vertical extension. Given this specific goal, the design was performed at a *preliminary level*. The Finite Element models employed were checked and validated on the basis of the hand calculations. However certain assumptions needed to be made; in case of actual realization these assumptions should be checked by a number of tests (e.g. laboratory strength testing for the existing structure). For instance, the quality of the concrete was assumed to be equal to that reported on the existing documentation, whereas, the normal practices suggest laboratory tests to conclude to the actual strength of the concrete. Having performed a preliminary design, in case of actual realization of the project, a number of drawings and calculations should be performed at a more detailed level (e.g. connections design, typical of the final design phase).

Additionally, it should be emphasized that there are also assumptions which might have an influence in the final result regarding the optimal vertical extension of Astoria. Aspects such as the vertical transportation and compartmentalization, will definitely play an important role in a vertical extension project and were not taken into account in the current research.

At last, the initial part of the research explores the general practices during vertical extensions of existing buildings. Information gained with the interviews was cross checked with literature research and the analysis of 6 case studies. This sample is not sufficient from the statistics point of view, in order to generalize the results and conclusions. Though, the selection of the projects was random, and considering that the amount of vertical extension projects in the Netherlands is limited compared to new buildings, the conclusions regarding the design parameters could be deliberated as useful information for future vertical extension projects. This gives to the conclusions a general validity that can be useful to approach every project aiming at performing a vertical extension.

9.2.2 General remarks

At first, the analysis of the vertical extension is based on storey-level. The scenarios and the corresponding extra storeys are defined by removing a storey from the model that failed. For instance, during the design of scenario 1, the model that comprises the addition of 5 storeys fails and therefore, the model is adjusted to 4 extra storeys and the analysis that is performed again fulfils all requirements. These simplifications have been introduced as means to reach the initial research objectives of the thesis project.

Moreover, the list of the principal design parameters could be introduced, as bullet points to be firstly checked or points of attention, in the decision-making phase of a project as a supplementary tool for the design team.

At last, it is of great importance to clarify that all the simplifications and assumptions made during the process of the current thesis project, have been introduced as means to reach the initial research objectives of the thesis project. The conclusions regarding the design parameters are in their essence qualitative and intent to highlight possibilities in a yet unexplored scientific area.

9.2.3 Recommendations for further research

This thesis project approaches from the structural engineer's point of view the possibilities for vertical extension in building renovation, and the conclusions reached are limited by some specific boundary conditions. These boundary conditions have to be broaden and include more parameters related to all building components. *Vertical extension* as a scientific topic should be approached from the various standpoints of all stakeholders involved in building renovation. The recommendations for further research are listed below:

- A research that incorporates an LCA with broader boundary conditions, including the labor and the construction works, and different scenarios for the life-cycle stages.
- Research the financial feasibility of a vertical extension project looking all the influencing parameters and all the financial aspects that participate in such a cost analysis.

Incorporate a risk and sensitivity analysis to identify the factors that play the most important role.

- Study the building as a whole with regard to vertical extension. Include all building components except for just the structural skeleton. This refers to the different actors participating in a building project, i.e. architect, building services engineer, construction companies.
- For building and structural engineering students, it would be interesting to study the vertical extension in terms of extra square meters instead of storeys, researching techniques for smart structural design and more efficient structural systems
- The current research is not taking into account the possibilities for deconstruction of the existing structure and reuse of the existing elements in the new building design option. Recent research (Glias, 2013) has emphasized the possibilities and advantages of this technique. Therefore, it would be interesting for a future research to compare the design option of "reuse and vertical extension" to the design option of "deconstruction and new building from existing elements" in terms of environmental impact and costs.

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