TRANSFORMABLE
OFFICE TOWER

A flexible breeding place at the RDM campus
University: TU Delft
Faculty: Architecture
Studio: Architectural Engineering
Semester: MSc 4

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As the Maasvlakte allows the port to shift west into the North Sea, older parts become obsolete and are available to the city.
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Preface

This report aims to presents the research that was carried out as part of the graduation project during the second and third quarter of the Architectural Engineering Studio. The studio focuses on design as a collaborative process informed by research findings drawn from different disciplines. A particular emphasis is placed on the interaction, correlation between engineering sciences and the architectural design. While exploring aspects such as construction, materialisation and building physics, students are tutored by specialized teachers to set out the research topics and to help them with the implementation in the design.

The architectural engineering program (two semesters) consists of two separate design assignments; the design of a small pavilion during the first quarter and the design of a more complex building during the 2nd -4th quarters. During the second project, students work on the design of a building accommodating a self proposed programme at the RDM campus in Heijplaat, Rotterdam. The design process is driven by research in the field of a research discipline within the building technology spectrum. The topic of choice discussed in this report is the design of a load bearing structure of great capacity to facilitate a transformable office building. This research was supervised by tutors of the chair of Structural Design.

I would like to thank my tutors Jan Engels and Wim Kamerling for their inspiring and competent support during the entire project.
Introduction

This report presents the research and design process of the graduation project within the architectural engineering studio. The report is divided into five chapters, each elaborating on a specific aspect of the project. Chapter one involves an introduction to the project location and its context. The design brief and goals are set out in the second chapter. The third and fourth chapter will discuss the design and research process. Chapter three focuses on design and materialization whereas chapter four contains the structural design calculations. The last chapter will present the final design and a conclusion.

1. Context

The former shipyard of the Rotterdam Dry Dock Company (RDM), situated within the “stadshavens” area, is being developed as a campus for educational institutions and innovative companies. Based on the theme Research, Design and Manufacturing this terrain will be developed into a unique location and breeding ground for the creative and innovative industry.

The key area of the RDM Campus is formed by the former machinery hall, the RDM Innovation Dock. On February 9th 2009 the technical education of the Rotterdam University started at this location, followed by students from Albeda College one month later. Apart from this an area of 12.000 m2 will be available for companies who want to share their knowledge with these educational institutions.

The former head-office, the RDM Droogdok 17 (school of architecture), RDM Village (concept housing) and the RDM Dokhaven, where the waterbus started mooring in Spring 2008, are also part of the current RDM Campus.

master plan

The students graduating in the architectural engineering studio jointly created a master plan for the expansion of the RDM campus. The area involves the zone to the north of the Heijplaat garden village (see Figure 02). All individual graduation projects in the studio were to be embedded in this joint master plan. This report will briefly describe this master plan as far as the direct context of the proposed building design is concerned.

The site was analysed at a larger scale with students of the studio. What regional input is needed, and what input can be used coming from the needs and wishes of Rotterdam. From this data a master plan (Figure 02) was created for the Area. The master plan consists the expansion of the campus (educational fascilities, student accomodation etc.) throughout the blue area. This is the zone adjacent to the current Innovation Dock and school of architecture. The zone surrounding the campus, currently accommodating port related activity, is to lose its industrial function and is in need of a new purpose. In line with the master plan, the east part of the area will be used to accommodate starting businesses within the creative (manufacturing) industry to strengthen the envisaged character of the Research Design & Manufacturing campus at Heijplaat in the 21st century. Sports and leisure will be developed on the Quaraintaine Island (green).

accessibility

An important infra-structural intervention is included in the plans with the introduction of a light rail connection between Rotterdam centre and Heijplaat. A secondary transportation service will be used for public transport in the area itself. Together with the already existing ferry service this will provide excellent access to the area (see Figure 02 and Figure 03).

The site for the project presented in this report within this master plan is indicated by the dashed oval in the yellow zone in Figure 02. The choice for this particular spot will be explained later. The zone is destined to accommodate starting businesses within the creative and innovative industry. The zone surrounding the campus, currently accommodating port related activity, is to lose its industrial function and is in need of a new purpose. In line with the master plan for the new RDM campus, the east part of the area will be used to accommodate starting businesses within the creative (manufacturing) industry to strengthen the envisaged character of the Research Design & Manufacturing campus at Heijplaat in the 21st century.
Figure 01  Stadshavens (blue) and RDM campus (green) within the main infrastructural network of Rotterdam

Figure 02  Northern part of Heijplaat; The main aspects of the master plan are a recreational area on the quarantine island (green), an education campus (blue), and an area for businesses in the creative manufacturing industry (yellow) The oval on the top right is the site for the proposed project. The campus is reached by ferry, light rail (red) or road
Figure 03  East RDM campus (yellow zone in Figure 02): businesses in the creative manufacturing industry. The proposed project is the square building on the top right of the map. The blue and red lines represent the primary (light rail to Rotterdam) and secondary (people mover) public transport systems.

Figure 04  Top: View of the site when approaching over water
First thoughts
During a first visit to the north of Heijplaat it is the enormous scale of the worn-out industrial sheds that is imposing. After crossing a jungle of cranes, containers and alien objects before driving past an unexpected garden village the RDM terrain unfolds itself. A small number of colossal sheds dominates the area and dwarf the houses in the village right next to it. “Raw” that’s a word that characterizes the environment at the northern most tip of Heijplaat quite well. When haunting the Netherlands in search of scenes providing “rawnness”, the vast port of Rotterdam might be a good place to start. Where most countries would offer mountainous shorelines, deserts, or snowy peaks, in our country raw and rough may be easier found in industrial areas than nature.

Typical for the structure of a port, water and land are intertwined like two folded hands, creating a richness peninsulas. You are never far from the water and the banks of the Maas offer impressive panoramic views. Following the Maas to the east you’ll see the high rises at the “kop van zuid” the Erasmus Bridge and the high rises surrounding the central station. The contrast between the somewhat desolate feel at Heijplaat and the urban appearance of the skyline provides an enjoyable contrast. It works enthusing to revitalize the worn out area and prepare it for its future as part of the city instead of the port. The industrial past of the area seems to be the ideal fertilizer creative and innovative future…

This link between the desolate and the urban makes the north-eastern most tip of Heijplaat an interesting place. The site features continuous views across the Maas. As it is positioned on a mini peninsula, the site is surrounded by water, facing Schiedam and Rotterdam to the north and east while the south and west face Heijplaat and the Waalhaven. A straight line between the main buildings of the RDM campus and the city cut’s the site in halve (This became an important motive to embed the designed project in its surroundings). A place with potential as a beacon to notify the city of its existence, and a place where the ambitions of the RDM campus are encouraged by the tension between what was and what lays ahead.

How can architecture contribute to this ambition? In what way can the aims and ambitions of the RDM mission be accommodated in an efficient, arousing and meaningful manner? It is clear that one type of building is widely available in the area; the worn-out sheds indicating the industrial past. These colossi, the largest of which has a footprint of over 30,000m2 probably have greater potential for future production facilities than for the light, well serviced office spaces needed for the moment. Also, the RDM main building proofs a mix of education and workshops can be housed in these sheds in an attractive manner as well. The role of education is ensured by the polytechnic and school of architecture located on the campus. Some businesses linked to the educational institutions are also already accommodated. For small and starting businesses, which naturally play an important role within the concept but are not directly linked to the educational programs, no good accommodation is provided quite yet. Research also shows there is a great need for small scale offices in the Netherlands.
Shortage small scale offices

There is a considerable shortage of suitable office space for small and starting companies in the Netherlands. When searching for recent news coverage on the subject, articles indicating the problem are easily found (see Figure 06). While the market for large scale offices (500m2+) is at a historic low due to the recession, providers of small scale office space cannot meet the still growing demands within this segment. Only in 2009 the demand for temporary, small scale office spaces has risen a staggering 30% (Parool 30-10-09).

Rotterdam

The Rotterdam city council has recently investigated the demands for small scale offices in the city. Research showed that there exists a large need for small scale office spaces within the Rotterdam area. The creative industry alone is good for a demand of about 25.000m2 (Eskamp 2008). The council also investigated the specific needs of this group of potential tenants. De average size of the required space is about 100m2 while a large part of the group requires less than 60m2 of office space, more than half of the researched group would like to share a building with other entrepreneurs. Almost all of the participants indicated the importance of easy access to the building (public transport of car) as well as sufficient parking provisions (Eskamp 2008).

Starters

The fourth quarter of 2008 shows an increase in starting businesses of 8% (2,717) in Rotterdam compared to the same period a year earlier. A total of 11.527 starting companies was registered in 2008, a new record. The number of bankruptcies amongst starting companies increased with 11 percent compared to last year. The increase in both starting businesses and bankruptcies relates to the current economic downturn. Where a number of companies have let employees go only to hire them again on a free-lance basis. These people then require office space, sometimes on a temporary project base, which again explains the shortage of this type of spaces.

Figure 06 Some examples of recent news coverage on the shortage of small and temporary office space
A number of organisations has reacted to these developments. Companies such as Spaces in Amsterdam and Compotex with subsidiaries across the country offer small scale and/or temporary office spaces within multi tenant buildings. Apart from office space alone, these organizations offer a variety of additional services from the receiving of guests and making appointments to the booking of business trips. The concept has proven very successful. The interest in this form of office rental is great and waiting list of twenty companies for a place in the buildings are not exceptional.

This type of building usually accommodates 15-40 smaller companies. These individual companies rent average office spaces of 50m², with the lower limit around 15 and highs of up to 200m². The renting prices vary from about 150€/m² in the lower- up to 250€/m² in the higher segment. Spaces on the herengracht in Amsterdam for instance aims at the higher segment with prices up to 260€/m² and offers a wide range of extra services.

Flexibility in the size of the rented space is an important factor. As the tenants usually concern younger companies with fluctuating success, the space need may change rapidly as well. A returning element in these building are the zones for common use. These spaces are characterized by an informal atmosphere and are ideal for informal or spontaneous meetings.

Some figures
Prices:
Spaces (Amsterdam): 250€/m²
Compotex (Various): 170€/m²
Office2start (Utrecht): 155€/m²

Average gross Floor area: 5000m²
Average Floor area per tenant: 50m²
Average number of tenants per building: +- 40
Introduction
This chapter will discuss the design brief and resulting research and design questions. The first section introduces the building typology and proposed programme in relation to the context. The following sections will present an exploratory study of the area and some precedents.

Brief
So where do we stand so far? The RDM campus aims at the development of the creative manufacturing industry as part of the transformation of the stadshavens on the southern bank of the Maas. Educational facilities are already accommodated in the existing buildings in the area. Remaining is, amongst others, adequate office space for businesses starting up on the campus to realize the RDM ambitions. The general demand for office space for starting businesses was already discussed. The importance of solutions for transformable, sustainable buildings that will stand the test of time will be addressed later in this report, and plays an important role in the design. The research carried out by Durmisevic in this field introduces design for disassembly as an important factor for transformability. In short these will be the most important starting points for the design presented in the rest of this report.

The brief involves a multi tenant building to accommodate starting businesses and stimulate entrepreneurship at the RDM campus. Special attention should be paid to students finishing RDM campus related study programs who want to start up their own business. The building should also relate to the construction of “pioneer projects” in new developments in a phase of economic downturn, as this is context the project will take place in. Given the apparent contrast between the ambitious plans for the RDM campus on one side and the infant state of the project in this time of economic downturn on the other the challenge of the brief may very well be to find a way to bridge this gap. The engineering challenge of lies in the design of a building infrastructure and infill system that allows for the required flexibility and facilitates interaction and by means of large circulation areas/atria.

A transformable office tower that will function as a breeding place to help realize the ambition for the RDM campus to become a centre for the creative manufacturing industry.

Research questions
The design brief building poses great challenges on the load bearing structure. Where is the balance between flexible construction methods (disassembly options), great users comfort and appealing architectural expression? What kind of supporting infrastructure would be suitable to accommodate the ever expanding and contracting program? How can the load bearing structure be employed to emphasise this process? An important part of the answer to these questions might be in finding or developing suitable systems for independence between building components (site, structure, skin, services, and separation). The main focus will be on a suitable infrastructure that can accommodate change.

Figure 09 The proposed location; the project is located at the north-east tip of Heijplaat (see figure) with a terrific view accross the Maas to the north, east and south.
Francis Duffy on office design

As discussed in the last chapters, there are good opportunities for office buildings within the segment of multitenant buildings for small companies. Accommodation for this type of tenant is underprovided and the market for these spaces is under tension within a global office market with vacancies up to 20%! Taking this segment as the point of departure for an office project at Heijplaat is therefore very plausible. For an office building that will stand the test of time however, just focusing on the current situation will not suffice. When the economic conditions change or successful businesses at RDM require larger spaces these changing demands should also be met. One of the pioneers within the research on office spaces is Francis Duffy. Duffy's book on "the new office" argues for a great degree of flexibility within the arrangement of office buildings. For an assessment of a good layout of the designed building, Duffy's findings were an important lead in the design of the building.

Duffy warned that in the changing workplace, only those speculative office buildings designed with the benefit of systematic organisational thinking would survive commercially. “Only those corporate users who have the imagination to link the organisational development to design imagination are likely to procure buildings that will escape obsolescence.” (Duffy changing workplace 94) In addition, Duffy argues that long term thinking is a good way to start when designing an office building and then shift towards the specific. Of course it is important for users to be clear about their specific requirements at an early stage of the design. However, it is inevitable for architects and developers to design buildings which are suitable for a variety of functions. Building appraisal was started in the mid eighties to close the gap between the developer and end user. The method analyses various factors such as image and the capacity of the plans to facilitate various office concepts.

Different types of offices yield very different results when it comes to time utilisation studies (Figure 10). These surveys show whether an workplace is in use, empty which activity it is used for etc. There are great similarities between similar types of work places; individual work places are only used 30% of the time! On the other hand, there is often a shortage of space for small meetings and informal interaction. TUS helps to decide how much space is needed for which type of support etc.

Duffy stresses that the potential profit to be made as the result of increased productivity of the office worker is considerable. As only 10% on average of a companies turnover is spend on accommodation, finding ways to make a building that yields more productive workers is more interesting that only focusing on net office space. The architect can offer added value here by creating a high standard working environment. Sadly, the effectiveness of work places is much harder to proof than their efficiency (m2 net area). Research does show however that space for small-, informal-, and spontaneous meeting is often underprovided. The percentage of this type of spaces could easily be increased from 10% to 30%!

Figure 10  Time utilisation studies (The new Office, Francis Duffy)
The shortage of these spaces is usually caused by conflicting needs between the building user and the building owner/exploiter. As research proves that the usage intensity of individual work places is much lower commonly assumed, the focus should indeed be on informal meeting areas for common use. Business models where tenants can rent flexible work places and are offered extensive extra services make these service spaces much more valuable. Multi tenant buildings exploited by organisations such as spaces or compotex would therefore justify a drastically increased capacity support area.

The implementation of large circulation spaces/atria for informal meetings etc is further supported by duffy’s claim that the importance of interaction within office spaces is often underestimated.

The image of office buildings is another factor of importance. Especially within the creative industry, the appearance of the accommodation is of crucial importance. Contrast between the architectural expression and a company’s organisation works very contra productive. Off course, as the proposed design involves a multi tenant building it is impossible to reflect all identities in the design. However, as the building will mainly involve companies within the creative industry and port related these could provide guidance for the appearance of the design.

Figure 11 The fraction of the annual costs of employing each office worker that is taken up by occupancy, i.e. rent, taxes and services (the New Office, Francis Duffy)

Figure 12 Driving down occupancy costs vs. adding value (the New Office, Francis Duffy)

Figure 13 Typical floor plan of the office building exploited by Spaces at the Herengracht in Amsterdam. The organisation is characterised by various types of workplaces around a central atrium
Flexibility
The relevance of design for transformability is described very adequately in the introduction to the 2006 dissertation on transformable building structures by E. Durmisevic. In the dissertation, Durmisevic argues that an important progress regarding the sustainability problem within the building industry are to be sought in the capacity of building to be transformed and disassembled.

“Observers of current and future trends predict that the 21st century is the beginning of an era that will be defined by temporary, multi-functional, and virtual organisations. The nature of working and living will change drastically such that society will require completely new types of structures. Besides the dynamic changes within society, another factor that indicates the need for an alternative way of building is the pattern of use of natural resources within the construction industry — a pattern which has proven to be unsustainable. Recent estimates indicate that existing buildings account for 2/5 of the world’s annual energy use, one sixth of its water consumption, and one half of its waste stream. The World Resource Institute projects a 300% rise in energy and material use as world population and economic activity increases over the next fifty years. The physical impact of an increasing building mass in the industrialised and developing world becomes undeniable in the 21st century.” (Durmisevic, 2006)

In her thesis, Durmisevic refers to various studies that indicate that demolition processes contribute a lot to the negative environmental impact of buildings. She argues for the development of methods to reuse and recycle building materials. Design for transformability and disassembly would contribute greatly to solve this problem. One of the problems of the current building practice is the tendency for components with different life cycles to be intertwined. This disproportion tears building structures apart and is responsible for the negative environmental impact of building assemblies. In other words, the combination of current building methods and market activities, which result in shorter phases for use of buildings, systems, and components, is a bottleneck for the decrease of demolition waste.

Figure 14 Division of investment costs in office sector (Durmisevic 2006)

In other words, the combination of current building methods and market activities, which result in shorter phases for use of buildings, systems, and components, is a bottleneck for the decrease of demolition waste.

Figure 15 A building demolished the old fashioned way, this type of practise plays an important role in the environmental footprint of buildings.
Durmisevic poses that the main issue of sustainable building is to find a balance between the increasing dynamics of change and principles of sustainable engineering such as saving energy and reducing demolition waste. She questions whether this could be achieved by extending the life cycle of buildings and their materials, as many claim. Buildings made of more durable materials do not always stand longer because of a number of causes two of which are:

1. the frequent functional changes often cause the ‘use life cycle’ of materials to be shorter than the ‘technical life cycle’ of materials;
2. materials are often integrated into a fixed assembly; the replaceability of one element means the demolition of others

Bearing this in mind, one can say that a key element of extending the life cycle of buildings and their materials involves designing the ability to transform all levels of technical composition by means of disassembly and reconfiguration, regardless of the materials used. To achieve this, a new design approach is needed that focuses on the long-term performance of building structures and its match with a technical composition.” (Durmisevic 2006)

The hypothesis of Durmisevic’s research is that buildings with a greater capacity to transform will result in a lower environmental impact. High transformation capacity means high flexibility and low negative impact on the environment.

Three interdependent subsystems that form a transformational system are introduced by Durmisevic:
1. Spatial transformation (implies use requirements and functional decomposition);
2. Structural transformation (implies technical decomposition);

A transformable system has impact on the other major factors of sustainable development such as the social, environmental, and economic systems. This research focuses on the impact of the Transformational System on the Environmental System, although the other two are considered and discussed as well.

“The trend in the office market today is for office spaces that are often rented as independent units. Therefore, it is important that these building can be split into separate units. Another requirement on office buildings today is that they can easily mutate from one organizational concept to another. Building structures should be designed not as static but as dynamic structures based on disassembly (Francis Duffy 1999).”

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Figure 16  Flexibility determined by fixed and flexible levels (Leupen 2002)
**Flexibility within architectural theory**

The following is a short summary of the essay on flexibility from the book Words & Buildings by Adrian Forty. In his essay Forty provides a brief history of the notion of flexibility within architecture and the main architects and theorists involved.

“The notion of flexibility in architecture first gained importance after 1950 as it offered an alternative to modernist deterministic functionalism. Against the presumption that all parts of a building should be destined for specific uses, the recognition that not all uses could be foreseen at the moment of design made ‘flexibility’ a desirable architectural property. As Alan Colquhoun has put it,”

“The philosophy behind the notion of flexibility is that the requirements of modern life are so complex and changeable that any attempt on the part of the designer to anticipate them results in a building which is unsuited to its function and represents, as it were, a ‘false consciousness’ of the society in which he operates.”

Another early statement on flexibility is by Walter Gropius who, in 1954 noted: “(1) that the architect should conceive buildings not as monuments but as receptacles for the flow of life which they have to serve, and (2) that his conception should be flexible enough to create a background fit to absorb the dynamic features of our modern life”

The purpose of ‘flexibility’ within modernist architectural discourse was as a way of dealing with the contradiction that arose between the expectation, so well articulated by Gropius, that the architect’s ultimate concern in designing buildings was with their human use and occupation, and the reality that the architect’s involvement in a building ceased at the very moment that occupation began. The incorporation of ‘flexibility’ into the design allowed architects the illusion of projecting their control over the building into the future, beyond the period of their actual responsibility for it. There are different strategies of flexibility in architecture.

**Redundancy**

This is a form of flexibility provided for by the creation of a margin excess, as Rem Koolhaas has put it in S, M, L, XL (1995) Het uses the koepel in Arnhem as an example. This building owes its flexibility to its monumental, space consuming features, which allowed for various configurations at a later stage. Koolhaas states that in functionalist architecture this quality is absent because every space meets exactly the needs of the program it was originally designed for.

**Flexibility by technical means**

An early example of this type of flexibility is Rietveld’s Schreuder house in Utrecht (1924). The open upper floor was installed with moveable partitions. There have been many subsequent modernist buildings in which there have been attempts to attain flexibility through making elements of the building moveable. In the postwar period, this type of flexibility concentrated instead on the development of lightweight building structures and of mechanical services, which allowed climatic control of spaces without the need for traditional architectural elements at all. Particularly influential were the systems developed in the United States the 1950s by Anton Ehrenkrantz and Konrad Wachsmann for buildings in which all services were carried in the roof space. Intended so as to offer freedom in the layout.

European architects such as Yona Friedman in France, Constant Nieuwenhuys (known as Constant) in the Netherlands, and Cedric Price in Britain, regarded flexibility as holding the potential for something very much more, offering not merely flexibility within buildings, but releasing buildings from their traditional fixity, and making possible a city within which all buildings could be mobile. Friedmans demand that “New constructions serving for individual shelters must 1. touch a minimum surface on the ground; 2. Be demountable and moveable; 3. be transformable at will by the individual” (1957,294) envisaged a city carried in a service structure, within which everything would be mobile and flexible.

**Flexibility as a political strategy**

This form results from a critique against capitalism, which in the eyes Situationist International (1950) had the tendency to commodify everyday life. Part of the objective was to resist this process and recover all those aspects of life that had been brought under capitalist regulation. This is evident in, for example, Constant’s article ‘The Great Game to Come’ (1959): ‘We believe that all static, unchanged elements must be avoided and that the variable or changing character of architectural elements is the precondition for a flexible relationship with the events that will take place within them’ (63). Within this scheme, ‘flexibility’ is not a property of buildings but of spaces; and it is a property which they acquire through the uses to which they are put.” (Forty 2000)
Precedents
The following pages present a selection of projects related to this project. Various ways to address the concept of flexibility are presented. The presented projects are only part of the complete collection of projects that were analysed throughout the design process. The selection aims at presenting an overview of solutions and designs that were made through the years based on the concept of flexibility. The figures below are some recent examples of projects accommodating change to meet user demand over time.

Figure 21  A project for a vertical housing plot in Madrid by Lorena Franco (Arroyo 2007)

Figure 22  Project de zeven hemels, een ontwerp voor een appartementen van zeven lagen ontworpen door 7 architecten (Demonstpieprojecten IFD, 2002)

Figure 23  Casco-plus en drager, een concept voor een nader in te vullen draagstructuur (Demonstratieprojecten IFD, 2004)
Cedric Price: Fun Palace Interaction Centre

“Cedric Price (1934 – 2003) was one of the most influential and visionary architects of the late-twentieth century, focusing on time-based urban interventions and flexible or adaptable projects that invited the user’s participation. His implicit criticism of contemporary notions of architecture earned Price heroic status among fellow architects such as Will Alsop, Archigram, Arata Isozaki, Rem Koolhaas, and Bernard Tschumi. Fun Palace is Cedric Price’s most celebrated work. Whether characterised as a giant toy or as a building-sized transformable machine, the project’s interest resides in its radical reliance on structure and technology, its exemplification of notions of time-based and anticipatory architecture. With Fun Palace, Price addressed social and political issues that go far beyond the typical bounds of architecture. Fun Palace was conceived and commissioned in 1961 by renowned theatre director and producer Joan Littlewood. Price developed plans for the project through 1964, both for the main project and for a smaller, more mobile “pilot” project. Neither were realised. Attempts to get planning permission for a wide variety of sites within and around London continued through 1970, amidst opposition from church, citizen groups and confounded city councils.

The columns, or service towers, in addition to anchoring the project, also contained service and emergency stairs, elevators, plumbing, and electrical connections. In conjunction with the main Fun Palace project, Cedric Price developed a smaller scheme or pilot project that could be assembled more quickly and disassembled and re-erected on a new site as required. One site in particular, the London borough of Camden Town, was investigated through a series of drawings.” (Canadian Centre for Architecture, http://www.cca.qc.ca/en 28-11-09)

“So that is another rule for the whole nature of architecture: it must actually create new appetites, new hungers—not solve problems; architecture is too slow to solve problems.” (Cedric Price, Obrist pp.68)
Kisho Kurokawa (Metabolists): The Nakagin Capsule Tower (Tokyo, 1970)

“The Nakagin Capsule Tower is a mixed-use residential and office tower designed by architect Kisho Kurokawa and located in Shimbashi, Tokyo, Japan. Completed in 1972, the building is a rare built example of Japanese Metabolism, a movement that became emblematic of Japan’s postwar cultural resurgence. The building was the world's first example of capsule architecture built for actual use. The building is still in use as of 2009, but has fallen into disrepair.

It is actually composed of two interconnected concrete towers, respectively eleven and thirteen floors, which house 140 prefabricated modules (or "capsules") which are each self-contained units. Each capsule measures 2.3 m (8 ft) × 3.8 m (12 ft) × 2.1 m (7 ft) and functions as a small living or office space. Capsules can be connected and combined to create larger spaces. Each capsule is connected to one of the two main shafts only by four high-tension bolts and is designed to be replaceable. No units have been replaced since the original construction.

Construction occurred on site and off site. On site work included the two towers and their energy-supply systems and equipment, while the capsule parts were fabricated and the capsules were assembled at a factory. The capsules were fitted with utilities and interior fittings before being shipped to the building site, where they were attached to the concrete towers. Each capsule is attached independently and cantilevered from the shaft, so that any capsule may be removed easily without affecting the others. The capsules are all-welded lightweight steel-truss boxes clad in galvanized, rib-reinforced steel panels. The cores are rigid-frame, made of a steel frame and reinforced concrete. From the basement to the second floor, ordinary concrete was used; above those levels, lightweight concrete was used." (Wikipedia 28-11-09)
Richard Rogers and Renzo Piano: Pompidou Centre (Paris, 1977)

Cedric Price’s vision of constructing a dominant structural frame, against which a number of interchangeable building components could be placed from services, enclosures, and different partitioning units, can be clearly seen in the design of the George Pompidou Centre, Paris. This project enlarged the design vocabulary by treating all components equally, and by using services as decorative elements. The structural frame is more than 168 m long, and maps out the space. Vertical service elements are placed on the east façade. The glazing façade is placed behind the structural frame. Actually, services, circulation routes, and cladding materials have a secondary influence on the building’s final appearance. The structural frame provides the organisation, controls the relief, scale, and visual detail, and in the end empowers the whole design (Andrew Orton 91). The building was declared by its designers to be a ‘non-building’ or a neutral framework in which various activities can take place, creating a form of architecture based on the events themselves. The building is an icon in the history of architecture, plays courageously with functions and their corresponding elements, and exposes them to the observer’s eye. This celebration of building tectonics tells the story about how buildings work, from their functional and structural organisation, to their smallest detail. The structure literally decomposes itself in front of our eyes. Its frame contains the whole vocabulary of different types of elements and connections (from pinned to cast connections). It is this design of detail that gives refinement to the whole building, and accentuates the designer’s determination to nurture change on all scales. (Durmisevic 2006)
Like the Pompidou Centre, the building was innovative in having its services such as staircases, lifts, electrical power conduits and water pipes on the outside, leaving an uncluttered space inside. The buildings make a clear distinction between the different functional groups of the building. They bring a focus back to the assembly and combination of functions and their materials at connections. The twelve glass lifts were the first of their kind in the UK.

The building consists of three main towers and three service towers around a central, rectangular space. Its focal point is the large Underwriting Room on the ground floor, which houses the famous Lutine Bell. The Underwriting Room (often simply known as ‘the Room’) is overlooked by galleries, forming a 60 metres (197 ft) high atrium lit naturally through a huge barrel-vaulted glass roof. The first four galleries open onto the atrium space, and are connected by escalators through the middle of the structure. The higher floors are glassed-in, and can only be reached via the outside lifts.

Figure 30  Loyds of London façade

Figure 31  Loyds of London interior

Figure 32  Column-beam joint

Figure 33  Façade detail
Next 21: Gas Housing Projec (Osaka 1993)

“The congregation in modern cities poses new challenges on housing. NEXT 21 is an experimental housing project designed to face these challenges through continuous trial and error. The joint cooperation of specialists in the fields of construction and the environment for actually constructing a single, localized form of building has been an important factor in the Osaka project.

A variety of up-to-date technologies incorporated into this apartment house are now being verified by the actual living experience of the occupants while various experiments are being conducted to find a way landing to the optimum living environment compatible with energy-saving and ecology.

Osaka Gas will evaluate how housing should be by measuring the various statistics collected from the inhabitants of NEXT 21, a futuristic experimental collective housing project developed through the cooperation of many people in a variety of fields. Grasping a firm idea of which direction technology should be developed in the future is an important theme of NEXT 21.” (www.arch.hku)
Jean Nouvel: Technology & Research Center (Wismar 1999)
The Technology centre in Wismar accommodates fixed program inside the large volume with glazed facades. The flexible program is simply stacked on top of this volume to create a fully flexible 2nd site for all sorts of research related functions.
Lacaton & Vassal: Nantes school of Architecture (Nantes 2009)

In building a structure of great capacity, the project design comes up with a scheme capable of creating a set of rich and diverse situations of interest to the School of architecture, the city and the landscape.

Three decks at nine, sixteen and twenty-two meters above the natural ground level, served by a gentle sloping external ramp, progressively put the ground surface of the city in touch with the sky overhead. A lightweight steel structure redivides the height of these main levels. It enables the spaces devoted to the program to be generously installed and creates a system adapted to their extension and their future evolution.

Linked to the spaces of the program are ample, double-height volumes with non-attributed functions, the transparent facades of which harness the sun's rays and vouchsafe the indoor climate. On the initiative of the students, teachers or visitors, these spaces become the locus of possible appropriations, events and programming. At any one moment the adaptation of the school to new interventions and its reconversion are possible. Like a pedagogical tool, the project questions the program and the practices of the school as much as the norms, technologies and its own process of elaboration.
Discussion

The analysis of projects where flexibility was an important design parameter proves many inspiring precedents exits. In general it can be concluded that an important characteristic is the development of new building techniques that improve structural and material performance and offered a variety of products to answer different requirements. These techniques change many building functions from being fixed to having less dependent conditions. Consequently, independent building systems were developed as performance driven systems where in use of materials and their arrangement into components and systems (by means of industrialised processes) was optimised to answer specific requirements. This resulted often into more efficient use of materials, better quality of components and buildings and greater client’s satisfaction.

Apart from material systems, the search for flexible architecture also introduced systematical approaches to the design of floor plans etc. The “open bouwen” method introduced by John Habraken developed into a complete design strategy with standardized dimensions for all sorts of functions. This approach with modular units such as applied in the project of for instance the Nagasin capsule tower seems an interesting manner to solve the problem of varying demand for capacity.

Recent designs such as the Nantes school of architecture are an interesting contribution to transformation capacity aspects of buildings by departing from an excess margin in both load bearing capacity and space. An interesting feature that is absent in most of the discussed building is the potential of the latent functional space while it is still unused. When this space is utilised to form atria and other collective spaces this could contribute greatly to the attractiveness of the building to potential tenants. This is of course an important issue when building in times of economical down-turn. The building does not necessarily have to be cheaper, extra effort should instead be put in providing extra quality instead of extra functional space while leaving space for transformation and growth of the functional floor area in times of high demand.
All buildings are predictions.
All predictions are wrong
(Stewart Brand)
3. Design Process

Introduction
This chapter will discuss the design process during the graduation project. A lot of aspects that were introduced in earlier chapters of the report were adopted as guiding principles for the design of an office building on the RDM campus in Rotterdam. The main aspects that will be treated are the Flexible and Dismountable design approach and the infrastructure with a margin excess capacity that allows for the densification process that is envisaged through the lifetime of the building.

Design Development
Several approaches to deal with the design problem were analysed while working on the design problem. A “modular” approach was adopted in an early stage of the design process. After studying literature on “open building” and design for disassembly. Various carrier infill options were sketched to study the potential of a system which separated joined and individual functions within the building. In the early phases of design, an infrastructure was envisaged to provide for all vertical and horizontal transport. This infrastructure consisted of four vertical cores connected by horizontal walkways for horizontal transportation. The offices themselves, consisting of prefabricated components would be mounted to this infrastructure on the interior and exterior of the horizontal walkways. This approach was later abandoned as the proposed grand scale of the units and structural feasibility of the whole system required some reconsiderations. The answer was found in a structural framework in which floor, facades, interior walls etc. could be mounted at will. The second step was to use components which a high degree of independancy, allowing different building elements to be altered without having to damage other parts. An approach advocated by Durmisevic as discussed in the last chapter.

Figure 39   A study of various carrier-infill approaches made early in the design process. The scheme proved to be a helpful tool during the process.
Figure 40  The growth of functional space in an infrastructure of predetermined volume was an important metaphor in designing the appearance of the building.

Figure 41  Formal studies in foam to study the special qualities of a partially filled volume. The study was not focused on a functional lay-out but to demonstrate the spacial tension that can arise using this principle.
Figure 42  The north-eastern part of Heijplaat; the site specific conditions obviously played an important role in the design process. Although the technical solutions to the engineering aspects of the problem ask for generic solutions. The embedment of the design into its actual context required the system to be tuned to match these boundary conditions.

Figure 43  The north-eastern part of Heijplaat; the site specific conditions obviously played an important role in the design process. Although the technical solutions to the engineering aspects of the problem ask for generic solutions. The embedment of the design into its actual context required the system to be tuned to match these boundary conditions.
The site is right on the between the main RDM buildings and the centre of Rotterdam. As the site features great views towards the Rotterdam skyline and the close links between building and campus, this orientation was used when designing the configuration of the concrete cores which are placed asymmetrically. This configuration also allows more light to enter the central area of the building, which is important for the "social" function of this part of the building.
Figure 47  Ground floor plan (1/250) The approach direction and orientation on the water and Rotterdam skyline are leading factor in the spacial organisation of the entrance lobby and expo. The expo is adjacent to the car park, making sure anyone arriving at the building by car will get a glance of the work exposed by tenants.

Figure 48  Situation (1/1000)
**Structure**

The design consists of a rectangular volume with square floor plan. The building has a height of 15 storeys or 54 meters and a width of 28.8 meters. The volume is partially filled with modules measuring 3600*3600*3400mm with are also the basic grid dimensions. The volume is envisaged to be subject to densification over time through the addition of additional modules. These "modules" do not concern 100% prefabricated units but are the basic "pixels" on which the composition is based. The load bearing structure provides for the easy addition and removal of programme anywhere in the structure. The following chapter will discuss the way in which this principle is materialised. The figures below illustrate the physical appearance and organisational structure of the building further.

*Figure 49  Section (1/north-south) The parking garage and expo are located on basement level. A waterfront terrace at mezzanine level enables visitors of the expo to enjoy the Skyline of Rotterdam accross the Nieuwe Maas*
Figure 50  Impression of the main structural elements. The diagonal trusses in the centre later became obsolete with the introduction of two stiff cores to provide for stability.

Figure 51  The loadbearing structure consists of a steel frame. Stability and vertical transport are provided for by the two concrete cores.

Figure 52  Scheme of the south elevation of the building; a game of solids and voids
Floor Systems

As the building is envisaged to be subject to densification over time, the building will consist of elements that are integrated during construction and elements that will added at a later stage. This is an especially important matter when selecting suitable floor systems as a floor system will have to be found that can be installed into the main load bearing structure after the building is finished. The design therefore departs from two different flooring systems; one that is built during initial construction and functions as a rigid slab, and a second that can be added later and enables the same quality of office space.

Several Floor systems were compared in order to choose the best “initial” floor for the building. The table below shows a brief summary of a comparison from “bouwen met staal”. A very important factor in the selection process was the need for accessible service ducts to make the envisaged flexibility possible. Another important factor was the weight and the thickness of the floor system. The floor that came out of the comparison best was the slime-line floor system. This floor system scores high on amongst others: cost, weight, thickness, accessibility of ducts and fire proofing. The floor consists of IPE profiles carrying a 70mm concrete floor slab which is also a finished ceiling. A computer floor can be used as the top floor resulting in a thin, light, flexible floor system.

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*Figure 53*  Comparison of various floor systems available in the Dutch building industry (bouwen met staal)
The slim line floor elements (see Figure 54) measure 6800*2400mm and can be connected by poured in steel connectors that are welded together on site. This leads to a floor which can act as one slab for stability purposes. As one of the main challenges of the design is to allow for program to be added to the volume at a later stage, a light weight floor system was sought that would match the hollow floor layout of the slim line system. The solution was found in a floor package based on metal web trusses to allow for the required span of 6800mm. These trusses consist of wooden horizontals and metal diagonals (see Figure 56) and can be fireproofed with gypsum board or promatect. The required acoustic performance is obtained by applying an acoustic break between the trusses and the raised floor panels.

The concrete slabs of the slim line floor system are thermally activated and used as cooling ceilings. To meet this same standard with the metal web truss floor, light weight- high thermal mass- plug and play cooling units containing a phase change material (micronal by BASF) are used. These innovative panels are produced by Ilkatherm and offer ideal lightweight solutions for this purpose. The system is illustrated in Figure 55 to Figure 57.

Figure 54  Render of the proposed hollow floor construction using the slim line floor system
Figure 55  Render of the proposed floor construction based on metal web trusses
Figure 56  Uncovered metal web truss floor (mitek)
Figure 57  Ilkatherm ceiling cooling panels
Figure 58  Ilkatherm micronal ceiling panel detail
Facade
As the building is envisaged to be subject to densification over time, the building will consist of elements that are integrated during construction and elements that will added at a later stage. The facade will therefore consist of modular elements. To allow for independant replacement of the facade elements, there sizes match that of the main structural grid.

The facade is build up of two types of facade element: the “closed” elements and the “open” elements. The closed elements are used at those parts of the facade behind which office functions are located. The open panels are used for parts of the facade adjacent to the atria.

The open elements are based on a curtain walling system developed for the horticultural industry. The system includes standardised solutions for sun screening and ventilation. The use of horticultural curtain walling for atria is not novel; the glazed atria of the faculty of architecture make use of them as well as the Martini hospital in Groningen (see Figure 59). A second advantage of this type of glazed facade is their low cost resulting from the industrial background, and the possibility of reuse in horticultural greenhouses in for instance the vast “glazed city” of Westland across the river.

The closed facade elements contain prefabricated timber frame elements. This choice offers a low weight solutions and also offers the possibility to separate the protective function from the aesthetic one, allowing greater transformation capacity. When the original facade cladding may become out of fashion or worn out, it can be replaced without having to remove the insulation layer.

Figure 59  The new building of the Martini hospital in Groningen proves the use of horticultural curtain walling can create a good solution with regard to aesthetics, performance and cost
Climate zones and building services

As the building is a composition of intertwined solids and void, the question arises what this means for the conditioning of the climate in these zones. “Figure 60 Diagram illustrating various options for dividing the building into climate zones: fully conditioned throughout, unconditioned atria and conditioned workplaces or conditioned workplace within an atria with intermediate climate” on page 47 shows different solutions to this problem. It was opted to divide the building into two different zones: fully conditioned workplaces (red) and an atrium with intermediate climate. With this approach, the atria will act as climate buffer zones for the fully climatised offices and can be used for the extraction of preheated air in wintertime. In summertime, the air heated up in the atrium will be transported directly out of the building through large opening sections in the curtainwalling. The system applied is based on those developed for the horticultural industry which also contains integrated sunscreen systems.

The use of horticultural curtain walling for atria isn’t new; the Martini hospital in Groningen features an enormous facade of type.

Figure 60  Diagram illustrating various options for dividing the building into climate zones: fully conditioned throughout, unconditioned atria and conditioned workplaces or conditioned workplace within an atria with intermediate climate

Figure 61  Scheme of the conditioning of the working spaces: A hollow floor system (slim line) contains all ducting for heating and cooling requirements. The concrete slab of the floor package is used as a cooling ceiling.
Figure 62  3d impression of the proposed facade and flooring system

Figure 63  Detail of the connection between an open and closed facade element and the slim line floor system
Figure 64  Facade fragment containing both “open” and “closed” facade panels. The applied system allows for great flexibility due to a high degree of independance between the separate building components.
Figure 65  Detail of a vertical section through the facade where two closed facade elements meet

**Legend:**
1. Promatect
2. Raised access floor
3. Mass customised facade cladding panel
4. Bracket for fixing timber frame element
5. IPE 300
6. UNP 220
7. 70mm concrete
8. Insulated aluminium window frame
9. Insulation glass (t=25)
10. Timber frame facade element (t=150)

Figure 66  Detail of a horizontal section through the facade where two closed facade elements meet
Figure 67  Detail of a vertical section through the facade where a closed facade element meets an open element

Figure 68  Detail of a horizontal section through the facade where a closed facade element meets an open element
4 Structural design

This chapter will present the research on the load bearing structure of the designed building. The first part will discuss the structure as a whole with regard to aspects such as stability etc. The second part will discuss the dimensioning and shape of the individual structural members. The last part of the report will deal with aspects such as sound and fire proofing concluded by a general conclusion on the building structure. A list of used symbols can be found at the bottom of this page.

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**List of Symbols**

- \(A\) = surface area [mm\(^2\)]
- \(E\) = modus of elasticity [N=mm\(^2\)]
- \(f_y\) = yield strength [N=mm\(^2\)]
- \(f_t\) = vloegrens [N=mm\(^2\)]
- \(I\) = moment of inertia [mm\(^4\)]
- \(i\) = traagheidsstraal [mm]
- \(l_{buc}\) = buckling length [mm]
- \(N\) = normal force [N]
- \(N_{kr}\) = critical normal force with buckling [N]
- \(N_p\) = vloeinormaalkracht[N]
- \(u\) = deflection
- \(W\) = moment of resistance [mm\(^3\)]
- \(\sigma\) = tension
- \(\lambda\) = slenderness [-]
- \(\lambda_{rel}\) = relative slenderness [-]
- \(\omega_{buc}\) = buckling factor [-]
- \(\varphi\) = rotation [rad]
Stability principle

As the design considers a high rise building, stability of the load bearing structure under wind loads will be one of the main issues in designing the load bearing structure. Prior to choosing a suitable method to provide for the buildings’ stability, various strategies were studied. As one of the design premises was to keep the central part of the building open to for joint functions such as conferences, workshops etc, a central stability core was not desirable. Another reason for the desire to keep the central area free from solid structural elements is to suggest densification of program within the building by creating a game of solid and open within a grid. Finally a load bearing structure was required to accommodate change. This means it should not be integrated with any other building components and it should enable disassembly. The latter was an important reason to choose for a steel frame structure.

A preliminary study of some basic stability principles was carried out and some precedents with decentralized stability cores were analyzed. The most interesting of these being the AG tower in Hannover by Thomas Herzog (Figure 69) and the RWE tower by Ingenhoven Overdick und Partner in Essen (Figure 70).

AG tower Hannover
The stability of the AG tower in Hannover is provided by two cores outside the main building volume (see floor plan). One of the cores is located at the north east, the other in the south west. The torsion forces created in this asymmetrical system is absorbed through fixed connections of all (in situ concrete) floors and columns. The placement of the cores outside the building volume and the fixed connection between concrete floors and columns would not be suitable for the proposed design as the cores would obviously obstruct the clear volume in which the densification takes place. The rigid connections of the in-situ concrete are in contrast with the ambition to build a dismountable building. Although rigid connections are of course feasible in steel as well, it seems odd to have two cores to provide for stability and still have the need for rigid connections between all other structural members.

RWE tower Essen
The RWE tower in Essen (Figure 70) is another example of a tower with decentralised stability cores. The cores in this building are within the main volume; however there is an additional lift shaft outside the cylindrical volume (see figure). In this building the cores are placed symmetrically and therefore the problem of torsion forces is absent as well. A disadvantage is that the cores are placed at the perimeter of the building and therefore subtracting a lot of “window space” from the floor plans. Apart from this, the central area of the building is very open creating a very spacious feel. The central zone is used for various functions on various floors such as: routing, conference rooms, toilets etc. This flexibility of the central zone is obviously enabled by the decentralised cores.
Figure 69  The AG tower in Hannover by Thomas Herzog (1999) The central area of the floor plans can house various functions as the cores are placed outside the main building volume.

Figure 70  The RWE tower in Essen by Ingenhoven Overdick und Partner (2000). Another example of a building where the central area of the floor plan can be used in a flexible manner through decentralised cores.
To develop an appropriate load bearing structure for the proposed building, various ways to approach high rise construction were studied. An overview of common strategies to provide for the stability of high rise structures is given in the figures below. As a central core was not desirable, strategies that allow for decentralization of the cores were sought. One obvious solution off course is to design a façade (e.g. perforated, vierendeel or trussed) to provide for stability. This however results in a much heavier facade columns while the façade should be as transparent as possible at those locations where no infill is present. It was then decided to opt for a system consisting of two smaller decentralised cores (similar to the RWE tower discussed in the last section). As these cores were too small to limit deflection and prevent tensional forces to occur at the bottom of the core, an outrigger was introduced to eliminate tensional forces and reduce deflection of the structure as a whole.

Figure 71  Various methods to provide for the stability of high rise structures.
Figure 72 (right) The basic functioning of the outrigger principle: a fixed connection between the core and the perimeter columns is introduced at a certain height to prevent rotation of the core by introducing a moment. The result is a drastically reduced horizontal deflection and a reduction of the normal forces due to horizontal loading in the core.

Figure 73 (bottom): The relation between the location of the outrigger and the reduction in displacement. It is typically at around 2/3 of the building height where the outrigger is most efficient.
As discussed in the last section, the stability of the building will be provided for by two decentralized cores which are rigidly fixed to the central columns to enable an outrigger effect. The structural scheme of this situation is given in Figure 74. To determine the required dimensions of all members, the wind load was determined (see appendix I).

Figure 74  Stability principle; the central columns and beams act as a truss. The figure on the right shows the geometry as imported into the infinite element analysis program. (central figure on the right taken from Bechtold & Schodeck 2007).

Figure 75  A floor plan showing the main structural members within the building. The dark U-shaped members are the main stability cores. The cores are provided with an outrigger construction on the 10th floor (lighter shade). The perimeter columns only carry vertical loads and can therefore be dimensioned very slim.
The total wind load acting on the building is 2.82 mN (see appendix II for the wind load calculation). This force is divided over two I-shaped concrete cores. This means that each truss needs to withstand a load of 1.41 mN.

A number of variants of the described layout were calculated on displacement and reaction forces. The resulting geometry was modelled and analysed in DIANA, the results are plotted on the next pages. In these models, the following input was entered (Location of loads and supports are plotted in figures on the next page):

- The wind load was applied per storey intersections (15*100kN).
- A Young’s modulus of 1.0*10^4 for reinforced concrete was assumed (which is very conservative)
- The vertical and horizontal members are steel HEM/HD profiles.
- All joints are hinges.

For the given situation the maximum deflection is 127mm. Which is just above the allowed value of 54000/500=108mm. The Young’s modulus of the concrete however is estimated very low, and may be more than two times bigger (assuming no tensional forces will occur in the concrete cores).

The load bearing principle was modelled in Rhino (see figure). The geometry was then exported and analysed in DIANA, a finite element analysis program by TNO. The results of the analysis are shown on the next pages. First the basic core elements are described and analysed, then three variants of the load bearing principle are given and analysed.

To determine the most suitable situation, the deflection of the structure without an outrigger was analysed first. The horizontal deflection in this situation indeed appeared to be far to high. To determine what exact outrigger design would be most suitable, three variants were tested. The first variant consists of one, one storey high outrigger at 2/3 of the building height. The second variant uses two one storey high outriggers at 1/3 and 2/3 of the building height. The third variant uses one 2 storey high outrigger construction. In the analysed model, the second and third variants remain below maximum deflection (see figures). The first variant is slightly to high. The Young’s modulus of the structure as modelled in Diana is very low (1*10^4). A slightly higher value would make the first variant comply with the (height/500) requirements.

For a double check, and to get a better understanding of the structural properties of the designed stability principle, a manual calculation was made. The results of this calculation are presented on page 54-58. These calculations were not completed entirely at the time of writing and will be updated in the next few weeks.
Figure 76  Smaller section: without an outrigger there is a deflection due to wind forces of 1360mm and a 6.9mN vertical reaction force near the core.

Figure 77  Wider section: without an outrigger there is a deflection due to wind forces of 390mm and a 5.8mN vertical reaction force near the core.

Figure 78  As the project should allow for densification of the program over time the load bearing structure may be subject to asymmetric loading. An asymmetric load case was modeled in Midas and calculated with DIANA. The results show no major problems with regard to deflection or tension caused by asymmetrical loads.
Figure 79  The structure in 3d with two, one storey high outriggers. There is a total deflection of 92mm due to wind forces and a maximum vertical tension at the bottom of the cores of 2.3mN.

The structure in 3d with two, one storey high outriggers. There is a total deflection of 99mm due to wind forces and a maximum vertical tension at the bottom of the cores of 2.87mN.
Figure 80  The structure in 3d with a single, one storey high outrigger. There is a total deflection of 141mm due to wind forces and a maximum vertical tension at the bottom of the cores of 3mN. The outrigger effect is very visible on the deflection plot in this version. This variant seems very attractive as it takes the least material and only interrupts the buildings transparency in one (be it two storeys high) location. To make sure this tactic will work in the other direction as well a second wind load was added in the other direction (see next page).

Figure 81  The structure in 3d with one, two storey high outrigger. There is a total deflection of 48mm due to wind forces in the direction of the wider core sections and a maximum vertical tension at the bottom of the cores of 2.80mN.
Manual analysis of the outrigger construction

The two cores and the outrigger truss together form a statically indeterminate portal frame. This section presents two manual calculation methods used to determine the moments, rotations, displacements and reactions of the structure. The first method is a combination of two “old school” methods for this type of structure: Luetkens, and Cross. Both methods had to be used as there act both a uniformly distributed load on the core-outrigger (Cross) joint as a concentrated load on the cores (Luetkens). The second method is the more analytical one and is based on equating displacements and rotations in the joints. The Young’s modulus of the concrete as used in these first calculations is $2 \times 10^4$. This value will change following the findings on whether there occurs tension in the cores or not.

![Schematic representation of the outrigger construction](image1)

**Figure 82** Section moment of inertia of the concrete core and the outrigger truss

**Figure 83** Schematic representation of the outrigger construction. The structure is split into segments which are then calculated to determine reaction forces, moments and displacements.
**Input:**

$I_{\text{outrigger}} = 3 \times 9.51304547 \times 10^{10} \times 0.9 = 8.56174092 \times 10^{10}$ (three outrigger trusses in structure!)

$E_{\text{steel}} = 210000$

$E_{I_{\text{outrigger}}} = 1.798 \times 10^{16}$ (*3 = 5.393$^\circ + 16$

$I_{\text{core}} = 8.0413 \times 10^{12}$

$E_{\text{concrete}} = 20000$

$E_{I_{\text{core}}} = 1.608 \times 10^{17}$

$n = E_{I_{\text{out}}} / E_{I_{\text{core}}} = 1 / 3$ (0.33533)

**Cross/Luetkens**

This first method treats the uniformly distributed load on the cores and the concentrated load and moment in B separately. The stiffness ratio ($n$) is determined with:

**Cross:**

First the ratio to determine the distributions of moments:

$$k_{BA} / k_{BC} = \frac{4EI}{h} \cdot \frac{3nEI}{l} = \frac{4}{34.2} \cdot \frac{3n}{8.4} = \frac{1}{8.4} = 0.496 / 0.504$$

The primary moments (due to q-load) acting in B are:

$$M_{\text{boven}} = \frac{1}{2} \cdot 27 \cdot 19.8^2 = 5292$$

$$M_{BA} = \frac{1}{12} \cdot 27 \cdot 34.2^2 = 2632$$

The “cross” moments are calculated in the table below:

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<th>Bc</th>
<th>Bd</th>
<th>C</th>
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<td>0.504</td>
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**Luetkens:**

The horizontal load to be used for the Luetkens method can now be determined:

$$H = 27 \left[ 19.8 + \frac{34.2}{2} \right] + \frac{3951 - 1972.5}{34.2} = 1054$$

$$M_{\text{primary}} = \frac{HH}{2} = \frac{1054 \cdot 34.2}{2} = 18027 \text{kNm}$$

$$k_{BA} / k_{BC} = \frac{EI}{h} \cdot \frac{3nEI}{0.5l} = \frac{1}{34.2} \cdot \frac{3n}{8.4} = \frac{8.4}{34.2(3n)} = 0.197 / 0.803$$

<table>
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<th>Bc (kNm)</th>
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</tr>
<tr>
<td>3551.3</td>
<td>-3551.3</td>
<td>-14475.5</td>
</tr>
<tr>
<td>21578.1</td>
<td>14475.5</td>
<td>-14475.5</td>
</tr>
</tbody>
</table>
The two methods combined yield the actual moments acting in A and B:

<table>
<thead>
<tr>
<th></th>
<th>ba</th>
<th>bc</th>
<th>bd</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1972.5</td>
<td>-3951</td>
<td>-1341</td>
<td>5292</td>
</tr>
<tr>
<td>21578.1</td>
<td>14475.5</td>
<td>-14475.5</td>
<td>5292</td>
<td>0</td>
</tr>
<tr>
<td>23550.6</td>
<td>10524.5</td>
<td>-15816.5</td>
<td>5292</td>
<td>0</td>
</tr>
</tbody>
</table>

**Mechanics**

The second (control) method to determine the moments in A and B first ignores the support in C. The vertical displacement in C is determined (statically determined situation) and the introduced a force in C to undo this displacement (causing a moment in B).

The upper part of the structure (above B and B') can be regarded as two clamped beams with a horizontal load (plus a certain rotation at the supports). The bottom part of the structure (including the horizontal member) can be regarded as a portal with clamped supports. There are thus three forces causing moments in the joints, de moment \((0.5*ql^2)\) and horizontal forces \(q*l2\) applied by the upper part, and the uniformly distributed load on the part below B and B'. The distribution of moment in the joints (A, Ba and Bc) depends on the ratio \((n)\) of the stiffness coefficients of the horizontal member \((EI)\) and the Vertical member \((n*EI)\).

\[ u_{cv} = \varphi \cdot l \]
\[ \varphi = \frac{Mh}{EI} + \frac{Fh^2}{EI} + \frac{qh^3}{EI} = \]
\[ \Rightarrow \frac{5292 \cdot 34200}{1.608 \cdot 10^{17}} + \frac{534.6 \cdot 34200^2}{1.608 \cdot 10^{17}} + \frac{27 \cdot 34200^3}{6 \cdot 1.608 \cdot 10^{17}} = 0.0042 \]

\[ u_{cv} = 0.0042 \cdot 8400 = 35.2 mm \]

This displacement is constrained by a vertical force in C:

\[ u = \frac{1}{3} \frac{Fl^3}{EI_{out}} + \frac{Fhl^2}{EI_{core}} = 35.2 \]
\[ \Rightarrow \frac{1}{3} \frac{l^3}{EI_{out}} + \frac{hl^2}{EI_{core}} = 35.2 \]
\[ \Rightarrow \frac{1}{3} \frac{8400^3}{5.36 \cdot 10^{16}} + \frac{34200 \cdot 8400^2}{1.608 \cdot 10^{17}} = 35.2 \]
\[ F = \frac{35.2}{1.87 \cdot 10^{-5}} = 1.882 \cdot 10^6 N \]
\[ M_{bc} = -1.882 \cdot 10^6 \cdot 8400 = 1.581 \cdot 10^{10} Nmm = -15813 kNm \]
\[
M_{BA} = 1.581 \cdot 10^{10} - 5.292 \cdot 10^9 = 1.052 \cdot 10^{10} \text{ Nmm} = 10521 \text{kNm}
\]
\[
M_{BD} = \frac{1}{2} \cdot 27 \cdot 19800^2 = 5.292 \cdot 10^9 \text{ Nmm} = 5292 \text{kNm}
\]
\[
M_A = M_{Aq} + M_{Bc} = (\frac{1}{2} \cdot 27 \cdot 54^2) - 15813 = 23553
\]

The calculated moments are compared in the table below (the bottom row gives the deviations):

<table>
<thead>
<tr>
<th>method</th>
<th>A</th>
<th>Ba</th>
<th>Bc</th>
<th>Bd</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kNm</td>
<td>kNm</td>
<td>kNm</td>
<td>kNm</td>
<td>kNm</td>
</tr>
<tr>
<td>Cross/Luetkens</td>
<td>23550.6</td>
<td>10525</td>
<td>-15817</td>
<td>5292</td>
<td>0</td>
</tr>
<tr>
<td>Mechanics</td>
<td>23553</td>
<td>10521</td>
<td>-15813</td>
<td>5292</td>
<td>0</td>
</tr>
<tr>
<td>Margin</td>
<td>0.01%</td>
<td>0.04%</td>
<td>0.03%</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The differences between the results are only 0.04%, a margin most likely to be caused by the rounding of values. Conclusion is that the calculations are accurate and will be used to refine the design of the cores.

**Tension**

With the moments known, it can be determined whether tension will occur in the concrete cores. Tension in the cores is countered by the dead loads of the cores and the floor area they carry:

The total weight of the concrete in each core is:
Volume of concrete in u-shaped cores: \((4.56 \times 54) + (20 \times 0.3 \times 16)\) \(342.2 \text{ m}^3\)
Load of this weight at the bottom of the core: \(342 \times 24000 = 8.21 \text{ mN}\)
This force works at 1.6 (or 2.8 in the other direction) meters from the edge.

Both cores carry 9 floors on the side of the façade; a total carried surface of 39\(\text{ m}^2\) is concerned here. The weight of these floors + drywalls etc. is 4\(\text{ kN/m}^2\) meaning an extra 39\( \times 4 \times 9 = 1.4 \text{ mN}\) is added. This force works at 4.3 meters from the edge of the cores.

It should be noted that, apart from a moment, a vertical force is applied to the cores by the outrigger. This force is -2.4\(\text{ mN}\) (calculation of reaction forces). This force works at 1.6 (centroid of the core including floors) meters from the edge of the cores and counters the other vertical forces.

The maximum moment without tension occurring in the cores is: \((8.21-2.4) \times 1.6+1.4 \times 4.3 = 15.3 \text{ mNm}\). This is below the actual moment working on the bottom of the cores. This means there is tension in the cores; the Young’s modulus of the concrete should therefore be taken at \(1.0 \times 10^4\). This means the stiffness ratio between the cores and the outrigger \(\frac{E_{\text{outrigger}}}{E_{\text{core}}}\) is now 2/3 instead of 1/3. This yields a new distribution of moments; the moment in A will decrease while the moment in B increases. The calculation for the cores for this situation is executed below (second method):

**New Input:**
\[
I_{\text{core}} = 8.04 \times 10^{12}
\]
\[
E_{\text{concrete}} = 1 \times 10^4
\]
\[
E_{I_{\text{core}}} = 8.04 \times 10^{16}
\]
Rotation in B:
\[
\begin{align*}
\Rightarrow & \quad \frac{5292 \cdot 34200}{8.04 \cdot 10^{16}} + \frac{1}{2} \left( \frac{534.6 \cdot 34200^2}{8.04 \cdot 10^{16}} \right) + \frac{1}{6} \left( \frac{27 \cdot 34200^3}{8.04 \cdot 10^{16}} \right) = 0.0084 \\
\end{align*}
\]
\[
\begin{align*}
u_{ev} = 0.0084 \cdot 8400 = 70.4 \text{mm}
\end{align*}
\]
This displacement is constrained by a vertical force (F) in C:
\[
\begin{align*}
u &= \frac{1}{3} \frac{Fl^3}{EI_{out}} + \frac{Fhl^2}{EI_{core}} = 70.4 \\
\Rightarrow & \quad \frac{1}{3} \frac{l^3}{EI_{out}} + \frac{hl^2}{EI_{core}} = \frac{70.4}{F} \\
\Rightarrow & \quad \frac{1}{3} \frac{8400^3}{5.36 \cdot 10^{16}} + \frac{34200 \cdot 8400^2}{8.04 \cdot 10^{16}} = \frac{70.4}{F} \\
F &= \frac{70.4}{3.0 \cdot 10^{-5}} = 2.09 \cdot 10^6 \text{N}
\end{align*}
\]
\[
\begin{align*}
M_{BC} &= -2.09 \cdot 10^6 \cdot 8400 = 1.7555 \cdot 10^{10} \text{Nm} = -17555 \text{kNm} \\
M_{BA} &= 1.7555 \cdot 10^{10} - 5.292 \cdot 10^9 = 1.2262 \cdot 10^{10} \text{Nm} = 12262 \text{kNm} \\
M_{BD} &= \frac{1}{2} \cdot 27 \cdot 19800^2 = 5.292 \cdot 10^9 \text{Nm} = 5292 \text{kNm} \\
M_A &= M_{Ag} + M_{Bc} = (\frac{1}{2} \cdot 27 \cdot 54^2) - 17555 = 21810 \text{kNm}
\end{align*}
\]
The moment at the bottom of the cores is then: 21.8-15.3=6.5mNm. This moment needs to be transferred by two foundation beams, each taking care of 3.2mNm. These beams therefore need a moment of resistance of at least:
\[
W = \frac{M}{\sigma_{max}} = \frac{3.2 \cdot 10^9}{45} = 7.1 \cdot 10^7 \text{ which implies a 400*1050mm beam would suffice.}
\]

\textbf{Reactions}

With all moments known, the reaction forces can be determined:

\textbf{Ah} and \textbf{A’}.h:
\[
A_h = A’_h = ql = 1458\text{kN}
\]
\[
A_v = -A_v’ \text{ (symmetry) }
\]
\[
\sum M_A = 21810 \Rightarrow q_{tot} h \cdot 0.5h - M_A' - A_v' \cdot 14.64 = 21810 \Rightarrow \frac{2 \cdot 27 \cdot 54 \cdot 0.5 \cdot 54 - 2 \cdot 21810}{14.64} = A_v'
\]
\[
A_v = 2398\text{kN}
\]
Displacement

With all moments and reactions known, the rotations and displacements can be determined. (All results are plotted in the figure below). The displacements \( w \) and rotations \( \phi \) in B and B' are determined using the scheme as shown in the figure below.

\[
\begin{align*}
\frac{w_B}{EI} &= \frac{Fl^3}{3EI} + \frac{ql^4}{8EI} - \frac{MI^2}{2EI} = \frac{5.35 \cdot 10^3 \cdot (34.2 \cdot 10^3)^3}{3 \cdot 8.04 \cdot 10^{16}} + \frac{27 \cdot (34.2 \cdot 10^3)^4}{8 \cdot 8.04 \cdot 10^{16}} - \frac{1.75 \cdot 10^{10} \cdot (34.2 \cdot 10^3)^2}{2 \cdot 8.04 \cdot 10^{16}} = 56.9 \text{mm} \\
\frac{\phi_B}{2EI} &= \frac{Fl^2}{2EI} + \frac{ql^3}{6EI} - \frac{ML}{EI} = \frac{5.35 \cdot 10^3 \cdot (34.2 \cdot 10^3)^2}{2 \cdot 8.04 \cdot 10^{16}} + \frac{27 \cdot (34.2 \cdot 10^3)^3}{6 \cdot 8.04 \cdot 10^{16}} - \frac{1.75 \cdot 10^{10} \cdot 34.2 \cdot 10^3}{8.04 \cdot 10^{16}} = 0.0009
\end{align*}
\]

With these figures known, the maximum displacement in D and D' can be determined (see next page for structural scheme):

\[
w_D = w_B + \phi \cdot l + \frac{ql^4}{8EI} = 56.9 + 0.0009 \cdot 19.8 \cdot 10^3 + \frac{27 \cdot (19.8 \cdot 10^3)^4}{8 \cdot 8.04 \cdot 10^{16}} = 81.4 \text{mm}
\]
The total displacement then includes the displacement due to rotation in the foundation (calculated in the next section): \( U_{tot} = 81.4 + 12 = 93.4 \)

The maximum allowed displacement is 54000/500=108mm which means the structure complies with the requirements.
To determine the displacement of the building due to the foundation, the rotational stiffness (C) of the foundation was determined. Figure 84 shows the scheme of foundation poles. This layout corresponds with the required load bearing capacity required at various locations. The groups of 9 poles are obviously located in line with the central columns.

The rotational stiffness of the foundation is determined in Table xx. The poles measure 400*400mm (A load bearing capacity per pole of 5*400*400=800kN). A translational stiffness of 1,0*10^6 was assumed for the poles. As the façade columns do not contribute to the buildings’ stability these were ignored in the calculation. The rotational stiffness of the foundation was calculated at 3,4*10^8N/mm². The maximum horizontal displacement at the top of the building due to wind loads is given as

\[ U = \frac{q \cdot h^3}{2C} = \frac{52 \cdot 54^3}{2 \cdot 3.4 \cdot 10^6} = 0.012 m = 12 \text{mm} \]

The maximum horizontal displacement due to rotation in the foundation is limited to 12mm which is acceptable.

<table>
<thead>
<tr>
<th>Grid size</th>
<th>Grid</th>
<th>E, F, and G</th>
<th>K</th>
<th>∑ a_i^2</th>
<th>3411.72 m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>q; dead load</td>
<td>=</td>
<td>120 mN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Capacity of pole</td>
<td>=</td>
<td>800 kN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Req nr of poles</td>
<td>=</td>
<td>150 piece (incl. façade)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grid size</td>
<td>=</td>
<td>3.6 m²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grid</td>
<td>E =</td>
<td>2</td>
<td>7 *</td>
<td>0.9</td>
<td>=</td>
</tr>
<tr>
<td></td>
<td>E =</td>
<td>2 *</td>
<td>3</td>
<td>3.6</td>
<td>=</td>
</tr>
<tr>
<td></td>
<td>C and G =</td>
<td>2</td>
<td>10</td>
<td>6.3</td>
<td>=</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>13</td>
<td>7.2</td>
<td>=</td>
<td>1347.84 m²</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>9</td>
<td>8.1</td>
<td>=</td>
<td>1180.98 m²</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>∑ a_i^2</td>
<td>3411.72 m²</td>
</tr>
</tbody>
</table>

**Figure 84** Scheme of foundation poles

**Figure 85** Calculation of the rotational stiffness of the foundations
Columns

The vertical loads on the proposed building are carried by columns on a 7200*7200 grid. The “central” columns, which carry the largest loads, carry an area of $7.2^2=52\text{m}^2$ (Figure 86). The building is 15 storeys high. The closely spaced columns in the façade carry an area of $13\text{m}^2$. As pointed out in the last section the façade columns do not contribute to the stability of the building which allows for thinner columns. To determine the total load carried by the columns, the loads per $\text{m}^2$ floor area are calculated in Table 2.

![Figure 86](image)

Figure 86  Floor area carried by various columns, the central area is carried by a beam between the two cores. This results in less loading for the steel columns and more weight on the cores which is desirable to fight tensional forces in the cores when strong winds occur.

<table>
<thead>
<tr>
<th>Live load</th>
<th>Dead loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offices</td>
<td>Floor (Slim-line)</td>
</tr>
<tr>
<td>Restaurant</td>
<td>Raised floor</td>
</tr>
<tr>
<td>Average per floor</td>
<td>Installations</td>
</tr>
<tr>
<td></td>
<td>Dry lining</td>
</tr>
<tr>
<td></td>
<td>Major beams</td>
</tr>
<tr>
<td></td>
<td>(concrete?)</td>
</tr>
<tr>
<td></td>
<td>Columns (Concrete?)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>3kN/m$^2$ extreme, 1 kN/m$^2$ momentarily</th>
</tr>
</thead>
<tbody>
<tr>
<td>Restaurant</td>
<td>5kN/m$^2$ extreme, 2 kN/m$^2$ momentarily</td>
</tr>
<tr>
<td>Average</td>
<td>1.4</td>
</tr>
<tr>
<td>Floor</td>
<td>2.5 kN/m$^2$</td>
</tr>
<tr>
<td>Raised</td>
<td>0.3 kN/m$^2$</td>
</tr>
<tr>
<td>Installations</td>
<td>0.5 kN/m$^2$</td>
</tr>
<tr>
<td>Dry lining</td>
<td>1.1 kN/m</td>
</tr>
<tr>
<td>Major beams</td>
<td>1.5 kN/m$^2$</td>
</tr>
<tr>
<td></td>
<td>(0.6<em>0.4</em>7.6<em>24</em>2/57.8=1.51)</td>
</tr>
<tr>
<td>(concrete?)</td>
<td></td>
</tr>
<tr>
<td>Columns</td>
<td>0.6 kN/m$^2$</td>
</tr>
<tr>
<td>(Concrete?)</td>
<td>(0.6<em>0.6</em>3.8*24/57.8=0.57)</td>
</tr>
<tr>
<td>Total +/-</td>
<td>8 kN/m$^2$</td>
</tr>
</tbody>
</table>

Figure 87  Loads on the floors. The calculated average load per $\text{m}^2$ assumes that the entire building will be filled with the slim-line floor system. As the actual floors will probably be of a lighter type this gives some margin for other unforeseen loads that may occur through the building’s lifetime.
Central columns

The maximum load on a central column of the building (assuming the building is occupied for 100%) is: 8.0 \times 65 \times 16 = 8.3 \times 10^3 \text{ kN per column. An extra load is put on the columns because they are part of the outrigger system. The structural analysis carried out using Diana show this extra load is approximately 1mN per column.}

The steel profile for the column needs a sectional surface area of at least 8.3+1=9.3 \times 10^6/235 (S235) = 3.96 \times 10^4 \text{ mm}^2. A steel HD320*368 will be sufficient to carry this load. Some properties of the profile are plotted in Figure 89. To ensure these columns will withstand all loads, the risk for buckling was analysed.

**Buckling Force of 3400mm long HD320*368 column:**

To determine the risk of buckling, the calculation model as described in the NEN6770 standard was used to establish the buckling force of the selected profile.

The (theoretical) Euler Buckling force of the selected column is (General rule of thumb is that the Euler value is 4-5 times the actual present load, a criterion that is met for the given situation):

$$F_{k} = \frac{\pi^2 EI}{l^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 31497 \cdot 10^4}{3400^2} = 56 mN$$

The calculation method as described in the NEN 6770 poses the following rule to test the structure on pressure:

$$\frac{N_{c,rd}}{\omega_{buc} N_{c,ud}} \leq 1$$

In which:

- $N_{c,rd}$ is the value for the stress due to the occurring loads
- $N_{c,ud}$ is the maximum stress which can be carried by the given section
- $\omega_{buc}$ is the buckling factor (depends on Euler buckling force and the maximum material pressure)

$$\omega_{buc} = \frac{\sigma_k}{f_y} = \frac{1}{\lambda^2_{rel}}$$

In which:

- $\sigma_k = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 82^2}{3400^2} = 1205.6 N/mm^2$ ($> 235!!$)
- $f_y = 235 N/mm^2$
- $\lambda_{rel} = \frac{\lambda}{\lambda_e} = \frac{41.4}{93.9} = 0.4412868416$ (Relative Slenderness factor)

With:

- $\lambda = \frac{l}{i} = \frac{3400}{82} = 41.44255921$ (Slenderness factor)
- $i = \sqrt{\frac{I}{A}} = \sqrt{\frac{3.15 \cdot 10^8}{4.68 \cdot 10^7}} = 82.04126541$ (Surface moment arm)
- $\lambda_e = \pi \frac{E}{f_y} = \pi \sqrt{\frac{2.1 \cdot 10^4}{235}} = 93.91297294$
In practice, one should consider multiple deviations that might occur due to “size effects”. This is why the determination of $\omega_{buc}$ in the NEN standard is based on the more empirical formula below:

$$\omega_{buc} = \frac{1 + \alpha_k(\lambda_{rel} - \lambda_0) + \lambda_{rel}^2}{2\lambda_{rel}^2} - \frac{1}{2\lambda_{rel}^2} \sqrt{(1 + \alpha_k(\lambda_{rel} - \lambda_0) + \lambda_{rel}^2)^2 - 4\lambda_{rel}^2}$$

$\alpha_k$ and $\lambda_0$ are to be taken from the table below. A, b, c & d are the possible buckling curves (Figure 88) of which one is to be selected depending on the profile configuration (Figure 88, left). Figure 88, right can be used to determine $\omega_{buc}$, the formula is used to determine the exact value.

<table>
<thead>
<tr>
<th>$\alpha_k$</th>
<th>a</th>
<th>0.21</th>
<th>b</th>
<th>0.34</th>
<th>c</th>
<th>0.49</th>
<th>d</th>
<th>0.76</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda_0$</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Figure 88](image)

Figure 88  Profile configurations and buckling curves as per NEN 6770

With all variables entered this yields:

$$\omega_{buc} = \frac{1 + 0.49(0.44 - 0.2) + 0.44^2}{2 \cdot 0.44^2} - \frac{1}{2 \cdot 0.44^2} \sqrt{(1 + 0.49(0.44 - 0.2) + 0.44^2)^2 - 4 \cdot 0.44^2} = 0.88$$

With the buckling factor known, the maximum force on the columns can now be determined.

$$N_{cxd} \leq 1 \cdot \omega_{buc} \cdot N_{cxd} = 1 \cdot 0.88 \cdot 1.1 \cdot 10^7 = 9.65 \cdot 10^7 \text{ kN} (> 9.3 \text{ mN})$$

Buckling will not be a problem in the given situation situation; the HD profiles are sufficient.
Figure 89  Specification of the steel HD profiles used for the construction of the central columns of the load bearing structure

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_1$</td>
<td>395</td>
<td>mm</td>
<td>$h_2$</td>
<td></td>
<td>mm</td>
</tr>
<tr>
<td>$b_1$</td>
<td>319</td>
<td>mm</td>
<td>$b_2$</td>
<td></td>
<td>mm</td>
</tr>
<tr>
<td>$t_1$</td>
<td>33</td>
<td>mm</td>
<td>$t_2$</td>
<td>58</td>
<td>mm</td>
</tr>
<tr>
<td>$r_1$</td>
<td>27</td>
<td>mm</td>
<td>$r_2$</td>
<td></td>
<td>mm</td>
</tr>
<tr>
<td>$G$</td>
<td>367.7</td>
<td>kg/m</td>
<td>$G_s$</td>
<td>374.7</td>
<td>kg/m</td>
</tr>
<tr>
<td>$A$</td>
<td>468.3 x $10^2$</td>
<td>mm$^2$</td>
<td>$A_L$</td>
<td>1.954</td>
<td>m$^2$/m</td>
</tr>
<tr>
<td>$I_y$</td>
<td>113189 x $10^4$</td>
<td>mm$^4$</td>
<td>$I_z$</td>
<td>31497 x $10^4$</td>
<td>mm$^4$</td>
</tr>
<tr>
<td>$W_{y1:el}$</td>
<td>5731 x $10^3$</td>
<td>mm$^3$</td>
<td>$W_{z1:el}$</td>
<td>1975 x $10^3$</td>
<td>mm$^3$</td>
</tr>
<tr>
<td>$W_{y2:el}$</td>
<td>x $10^3$</td>
<td>mm$^3$</td>
<td>$W_{z2:el}$</td>
<td>x $10^3$</td>
<td>mm$^3$</td>
</tr>
</tbody>
</table>
Facade columns
The maximum load on a facade column of the building (assuming the building is occupied for 100%) is: 
8.05*13*15=1.67*103kN per column.

The steel profile for the column needs a sectional surface area of at least 1.67*106/235 (S235)=7.1*103 mm2

A HEB 200 or HEM 140 is sufficient to support this load. An alternative would be to use square tubular sections180*180*12.5. With regard to the more favourable surface/mass index of the profile and with the ability to fill tube profiles with e.g. sand, the tubular sections are opted as the facade column profiles of choice.

For a rough indication of the risk for buckling, the buckle for the column is determined (the columns have a buckling length of 3400mm).

Buckle Force of 3400mm long CFRHS180*180 column:
As with the other columns, the calculation model as described in the NEN6770 standard was used to establish the buckling force of the selected profile to determine the buckling risk. The (theoretical) Euler Buckling force of the selected column is (General rule of thumb is that the Euler value is 4-5 times the actual present load, a criterion that is met for the given situation):

\[ F_k = \frac{\pi^2 EI}{l^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 336 \cdot 10^7}{3400^2} = 6 mN \]

The actual buckling force is determined as per NEN6770 (see calculation of central columns for more elaborate description).

\[ i = \sqrt{\frac{I}{A}} = \sqrt{\frac{3.36 \cdot 10^7}{7.70 \cdot 10^3}} = 66.05782591 \] (Surface moment arm)

\[ \lambda = \frac{i}{\xi} = \frac{3400}{66} = 51.47005602 \] (Slenderness factor)

\[ \lambda_{rel} = \frac{\lambda}{\lambda_e} = \frac{51.5}{93.9} = 0.5480611933 \] (Relative Slenderness factor)

\[ \omega_{buc} = \frac{1 + \alpha_k (\lambda_{rel} - \lambda_0) + \lambda_{rel}^2}{4 \lambda_{rel}^2} \frac{1}{2 \lambda_{rel}^2} \sqrt{(1 + \alpha_k (\lambda_{rel} - \lambda_0) + \lambda_{rel}^2)^2 - 4 \lambda_{rel}^2} \]

\[ \omega_{buc} = \frac{1 + 0.21(0.55 - 0.2) + 0.55^2}{2 \cdot 0.55^2} \frac{1}{2 \cdot 0.55^2} \sqrt{(1 + 0.21(0.55 - 0.2) + 0.55^2)^2 - 4 \cdot 0.55^2} = 0.91 \]

\[ N_{c,i,a} \leq 1 \cdot \omega_{buc} \cdot N_{c,a} = 1 \cdot 0.91 \cdot 1.81 \cdot 10^6 = 1.64 \cdot 10^3 kN < 1.67 mN \]

Buckling will be a problem will be a problem; a CFRHS 200*200 however will suffice.
Figure 90  Profile configurations and buckling curves as per NEN 6770

<table>
<thead>
<tr>
<th></th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.21</td>
<td>0.34</td>
<td>0.49</td>
<td>0.76</td>
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<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Figure 91  Specifications of the steel profile applied for the columns in the facade
Floor Systems

As the building is envisaged to be subject to densification over time, the building will consist of elements that are integrated during construction and elements that will added at a later stage. This is an especially important matter when selecting suitable floor systems as a floor system will have to be found that can be installed into the main load bearing structure after the building is finished. The design therefore departs from two different flooring systems; one that is built during initial construction and functions as a rigid slab, and a second that can be added later and enables the same quality of office space.

Several Floor systems were compared in order to choose the best “initial” floor for the building. The table below shows a brief summary of a comparison from “bouwen met staal”. A very important factor in the selection process was the need for accessible service ducts to make the envisaged flexibility possible. Another important factor was the weight and the thickness of the floor system. The floor that came out of the comparison best was the slime-line floor system. This floor system scores high on amongst others: cost, weight, thickness, accessibility of ducts and fire proofing. The floor consists of IPE profiles carrying a 70mm concrete floor slab which is also a finished ceiling. A computer floor can be used as the top floor resulting in a thin, light, flexible floor system.
The selected floor system consists of steel IPE profiles spaced at 1200mm centre to centre with a 70mm layer of concrete slab which is poured around the bottom flange of the beam. The bodies of the steel profiles are punched to allow for services to be integrated within the floor system. Another advantage is the fact that the floor is not simply supported on the top the main structure but is in the same level (see detail). The floor span is 7000mm, the load bearing scheme is given below.

![Figure 93](image)

The IPE profiles are spaced at 1200mm. If assuming a total load on the floor of 8kN/m2, the load on the beams is $8 \times 1.2 = 9.6$ kN/m. The most suitable IPE with regard to services purposes would be at least an IPE 300. Rules of thumb ($h=l/25$) suggest a height of at least 280. The calculation below is to assess an IPE 300 for the given situation (stiffness and strength).

\[
\begin{align*}
8.5 \text{ kN/m}^2 \\
\text{umax} & \quad 7000/250=28\text{mm} \\
\text{ly} & \quad 8.36\times107\text{mm}^4 \\
\text{Wy} & \quad 5.57\times105\text{mm}^3 \\
\text{deflection:} & \quad \frac{5}{384} \cdot \frac{gL^4}{EI} = \frac{5}{384} \cdot \frac{7000^4 \cdot 9.6}{2.1 \times 10^3 \cdot 8.36 \times 10^7} = 17\text{mm} < 28\text{mm} - \text{complies} \\
\text{stress:} & \quad \frac{5}{8 \cdot \sigma_{\text{max}}} \cdot \frac{4 \cdot 6800^2}{8.0 \cdot 5.57 \times 10^5} = 99.6\text{N/mm}^2 < 235\text{N/mm}^2 - \text{complies}
\end{align*}
\]

An IPE 270 would suffice, for some extra height to allow for services and with regard to the slightly weaker IPE condition due to the perforations, it was opted to use an IPE 300 for the perimeter floors in the building. Below the buckling risk in the given configuration is assessed.
Lateral torsion buckling
To check the chosen profile on lateral buckling, the calculation method as described in NEN6770 was used. NEN 6770 requires the following rule to be met to comply with the requirements for lateral buckling stability:

\[
\frac{M_{y:z;d}}{\omega_{buc}M_{y;u;d}} \leq 1
\]

The buckling factor \( \omega_{buc} \) follows from the relative slenderness of the beam and the type of profile that is used and is determined using the buckling curves for lateral buckling (Figure 96). For rolled profiles the a-curve is used, for welded profiles the b-curve is used.

\[
\lambda_{rel} = \zeta \sqrt{\frac{l_{kip} h f_{y;d}}{b t_f E_d}}
\]

In which:
- \( \zeta \) Factor depending on profile section (1.32 or 1.23, for an IPE300 1.32)
- \( l_{kip} \) Buckling length (=7000 mm)
- \( h \) Height of the profile (300 mm)
- \( f_{y;d} \) Yield stress (235 N/mm\(^2\) for S235)
- \( b \) Width of the profile (150 mm)
- \( t_f \) Thickness of the profile flange (10.7 mm)
- \( W_y;pl \) Moment of resistance (5.57\(^*\)105mm\(^3\))
- \( q \) Load (8.2kN/m)
- \( E_d \) Modus of elasticity (2.1\(^*\)105 for S235)

\[
\lambda_{rel,1.32} = \frac{7000 \cdot 300 \cdot 235}{150 \cdot 10.7 \cdot 2.1 \cdot 10^5} = 1.597
\]

The buckling factor is determined using the buckling curve: \( \omega_{kip} = 0.34 \). With all variables now known the floor beams shall be tested with the buckling equation presented above:

\[
\omega_{kip} = 0.34
\]

\[
M_{y;z;d} = \frac{1}{8} q l^2 = \frac{1}{8} \cdot 8.2 \cdot 7^2 = 50.2 \text{kNm}
\]

\[
M_{y;u;d} = f_{y;d} \cdot W_y = 235 \cdot 5.57 \cdot 10^5 = 1.31 \cdot 10^8
\]

\[
\frac{M_{y:z;d}}{\omega_{buc}M_{y;u;d}} = \frac{50.2 \cdot 10^6}{0.34 \cdot 1.31 \cdot 10^8} = 1.13 > 1 \text{doesnotcomply!}
\]

As the bottom flange of the IPE profiles is embedded in a 70mm layer of concrete, the lateral buckling stability will be increased. Furthermore, the calculation method used in the NEN 6770 standard is of a conservative (=very safe) nature. It is therefore expected that further buckling supports are not required. \( \omega \)

\[
q_{max:\text{NEN6770}} = \frac{8 \omega_{kip} M_{y;u;d}}{l^2} = \frac{8 \cdot 0.34 \cdot 235 \cdot 5.57 \cdot 10^5}{7000^2} = 7.266
\]
Figure 96  Lateral torsion buckling curves as per NEN 6770
Horizontal beams in façade
The horizontal beams in the perimeter of the building carry the IPE profiles described above. As the columns in the façade are spaced at 3600mm distances, the span of the members is only small. As the members are loaded asymmetrically it was decided to use a profile that best suites this situation. A UNP profile seemed a good candidate as the torsion centre of this profile is very close to the located that the vertical loads are absorbed.

A UPN 220 is expected to be sufficient in the given situation. The profile is assessed on stiffness and strength below (a maximum allowed deflection of span/250 is assumed):

\[
\begin{align*}
I \text{ required: } & \frac{5}{384} \cdot \frac{gl^4}{EI} = \frac{5}{384} \cdot \frac{3600^4 \cdot 28.8}{2.1 \cdot 10^7 \cdot 2.69 \cdot 10^3} = 11.1 \text{mm} < 14.4 \text{ – complies} \\
W \text{ required: } & \frac{gl^2}{8 \cdot W_y} = \frac{28.8 \cdot 3600^3}{8 \cdot 2.45 \cdot 10^5} = 190 \text{N/mm}^2 < 235 \text{N/mm}^2 \text{ – complies}
\end{align*}
\]

Torsion due to asymmetrical loading
The torsion centre of a UNP220 profile can be determined with:

\[
e = \frac{3 \cdot b^2 \cdot tf}{6 \cdot b \cdot tf + h \cdot tw} = \frac{3 \cdot 80^2 \cdot 11.5}{6 \cdot 80 \cdot 11.5 + 200 \cdot 9} = 31 \text{mm}
\]

The type of support of the slim-line floor system (see detail) in combination with a UNP profile is very favourable with regard to the torsion centre (The L-shaped support is welded or to the UNP profile).
Central Floor Beams
The main central beams of the load bearing structure carry a load of 7.2(width)*7(kN/m²)=50.4 kN/m

Ideally, the profile used for the central floor beams would fit inside the floor package of the building (height=340mm). Given the span and the load field width of 7200mm, the selected profile is a HEA 340 as this profile would be ideal in combination with the slim-line floor system.

\[
I \text{ required: } \frac{5}{384} \cdot \frac{ql^4}{EI} = \frac{5}{384} \cdot \frac{6800^4 \cdot 50.4}{2.1 \cdot 10^5 \cdot 2.77 \cdot 10^5} = 24mm < 27mm - \text{complies}
\]

\[
W \text{ required: } \frac{ql^2}{8 \cdot \sigma_{\max}} = \frac{50.4 \cdot 6800^2}{8 \cdot 1.68 \cdot 10^6} = 173N/mm^2 < 235N/mm^2 - \text{complies}
\]

Lateral torsion buckling
To check the chosen profile on lateral buckling, the calculation method as described in NEN6770 was used. NEN 6770 requires the following rule to be met to comply with the requirements for lateral buckling stability:

\[
\frac{M_{y,\max;d}}{\omega_{buc} \cdot M_{y;U;d}} \leq 1
\]

The buckling factor \(\omega_{buc}\) follows from the relative slenderness of the beam and the type of profile that is used and is determined using the buckling curves for lateral buckling (Figure 100). For rolled profiles the a-curve is used, for welded profiles the b-curve is used.

\[
\lambda_{rel} = \zeta \frac{l_{kp} \cdot h \cdot f_{y,d}}{b \cdot t_f \cdot E_d}
\]

In which:
- \(\zeta\) Factor depending on profile section (1.32 or 1.23, for an IPE300 1.32)
- \(l_{kp}\) Buckling length (=6800 mm)
- \(h\) Height of the profile (330 mm)
- \(f_{y,d}\) Yield stress (235 N/mm² for S235)
- \(b\) Width of the profile (300 mm)
- \(t_f\) Thickness of the profile flange (9.5 mm)
- \(W_y;\) pl moment of resistance (1.68*106mm3)
- \(q\) Load (50.4kN/m)
- \(E_d\) Modulus of elasticity (2.1*105 for S235)
The buckling factor is determined using the buckling curve: $ω_{kip}=0.34$. With all variables now known the floor beams shall be tested with the buckling equation presented above:

$$\lambda_{rel} = 1.32 \sqrt{\frac{6800 \cdot 330 \cdot 235}{300 \cdot 9.5 \cdot 2.1 \cdot 10^5}} = 1.24$$

As the top flange of the HE profiles is fixed to the IPE beams of the slim-line floor, the lateral buckling stability will be greatly increased. This provision should be sufficient as buckling support. Furthermore, the calculation method used in the NEN 6770 standard is of a conservative (=very safe) nature. It is therefore expected that further buckling supports are not required.

$$ω_{kip} = 0.5$$

$$M_{y:z;\text{d}} = \frac{1}{8} q l^2 = \frac{1}{8} \cdot 50.4 \cdot 6.8^2 = 291.3 \text{kNm}$$

$$M_{y:x;\text{d}} = f_{y;\text{d}} \cdot W_{y} = 235 \cdot 1.68 \cdot 10^6 = 3.95 \cdot 10^8$$

$$\frac{M_{y:\text{max};\text{d}}}{ω_{o,\text{rc}} M_{y:x;\text{d}}} = \frac{291.3 \cdot 10^6}{0.5 \cdot 3.95 \cdot 10^8} = 1.47 > 1 - \text{does not comply}!$$

Figure 100  Lateral tension buckling curves as per NEN 6770
Metal Web Truss

The floor system to be used for the floors that are added to the building at a later stage was selected on compatibility with the slim line system. The floor also has to be installable without the help of cranes as the building they are installed in already exists. The two best candidates are a steel system floor (e.g. SADEF) or a flooring system based on metal web trusses. The last system was selected due to its ideal compatibility with the original floor. Metal web trusses, which are commonly used in the U.S. and Australia etc, are a wood-metal hybrid truss. The diagonal members are made of metal; the horizontal members are made of wood. The result is an easy to use, light truss which is suitable for large spans (see figure).

Figure 101  Floor supported by metal web trusses

In the 7200*7200 grid of the designed building, the trusses span 7200 mm. As they are only 600 mm apart, the area for which they take the load is relatively small. The following calculation is to determine the required dimensions of the truss members (see figure):

Width of load area: 600 mm
Total load on floor: 6 kN/m2
Load on truss: 0.6*6=3.6kN/m
Spacing of joints: 1080 mm
Force on mid joint: 3.6*1.08=3.9kN
Force on side joint: 3.6*0.54=1.94kN

Figure 102  : Metal web truss structural scheme
**Strength**

The maximum element forces will be in the central (top) horizontal member and the outer diagonal members.

The reaction forces will be $3.9 \times 7 / 2 = 13.7\text{kN}$ in both supports.

**Wooden members:**
Next the normal force in the central horizontal member will be determined using external equilibrium in section S.

\[
M_s = 13.7 \times 3.24 - 1.95 \times 3.24 - 3.9 \times 2.16 - 3.9 \times 1.08 = 25.4\text{kNm}
\]

Members force is $25.4 / \text{truss height} = 25.4 / 0.54 = 47\text{kN}$

The required section surface of the softwood members is ($\sigma_{\text{max}} = 14\text{N/mm}^2$): $47.000 / 14 = 3360\text{mm}^2$

A softwood member with a section of $40 \times 90\text{ mm}$ would be sufficient.

**Metal web members:**
The maximum force in the diagonals will be in the most left and most right diagonals. The forces in these diagonals can be determined with the joint equilibrium of the outer left (or right) top joint:

\[
\sum F_v = 13.7 \times 1.95 - \text{member v} = 0
\]

Vertical force in diagonal is $11.76\text{kN} \gg$ Normal force is $\sqrt{2 \times 11.76} = 16.6\text{kN}$

The required steel section surface is $16.6 \times 103 / 235 = 71\text{mm}^2$

**Deflection**
To roughly determine the deflection of the metal web truss the truss can be approached as an I beam with given I. The moment of inertia of the two wooden members is $52970 \times 104\text{mm}^4$.

\[
I = 52970 \times 104\text{mm}^4
\]

E of softwood is $11000$.

The maximum deflection can be determined with:

\[
U_{\text{max}} = \frac{5}{384} \frac{ql^4}{EI} = \frac{5}{384} \frac{3.6 \times 7600^4}{1.1 \times 10^3 \times 5.3 \times 10^4} = 26.8
\]

The maximum allowed deflection is $7600 \times 0.004 = 30.4\text{mm}$. The truss thus complies with the requirements for deflection.
Discussion

The load bearing structure of the building was designed to answer to a number of different requirements set out in the design brief. The most influential of these are:

- Functional flexibility in the central area of the floor plan
- Largely dismountable building structure
- Intelligent system for expanding services network in densification process

It was opted to design a load bearing system based on a 7200*7200mm grid of columns as this is a widely applied grid size. Stability is provided for by two decentralised cores in which vertical transportation, service shafts, sanitary and emergency staircases are accommodated. The central area of the building is kept free from vertical shafts, therefore allowing a variety of functions to be housed in this zone.

The cores consist of prefabricated concrete elements that provide for stability. To reduce deflection and prevent tensional forces at the bottom of the cores, an outrigger is introduced at the tenth floor. This outrigger joins the cores to the heavy columns that support the central area of the building. The system of concrete cores and steel columns and outrigger trusses was modeled and analysed in DIANA, a finite element analysis program by TNO. The calculations proof that in the given wind load conditions the designed structure complies with requirements for load bearing structures.

The façade does not have to provide for stability, allowing very slim dimensioning of the columns. Apart from the cores, all structural members are steel profiles (mainly HEB, HD and UNP). The required dimensions of these members were calculated as presented in this report.

There are two flooring systems to be used in the building; one that is installed during construction and functions as a rigid slab for stability purposes and the other that is added to the structure during the densification process. The floors are based on the hollow floor principle, which means that services are integrated in the structural height of the floor slabs. The service ducts are accessible at all times through the application of a raised surface floor with dismountable panels.

Fireproofing of the structure will happen through the use of fire-proofing coating. An alternative that was studied during the design process was the use of water filled tube profiles. The complexity and cost of this system however motivated the choice for a coating.
Appendix I: Wind load as per NEN6702

The wind load on a building is calculated using the following formula:

\[ P_{rep} = C_{dim} \cdot C_{index} \cdot C_{eg} \cdot \varphi_1 \cdot P_w \]

In which:
\- \( P_{rep} \) = wind load caused by wind pressure and suction kN/m\(^2\)
\- \( C_{dim} \) = factor for building dimensions
\- \( C_{index} \) = wind shape factor
\- \( C_{eg} \) = pressure equalisation factor
\- \( \varphi_1 \) = factor dynamical influence
\- \( P_w \) = extreme wind load dependant on location and height

\[ C_{dim} \]: for a building between height=60 and width=30 is 0.89
\[ P_w \]: wind load area II, h=60 yields a pw of 1.50
\[ C_{index} \]: 0.8+0.4+0.08+0.02=1.3

<table>
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<tr>
<th>Height in meters</th>
<th>Pw (kN/m(^2))</th>
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<th>Region II</th>
<th>Region III</th>
</tr>
</thead>
<tbody>
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<td>Country</td>
<td>Town</td>
<td>Country</td>
</tr>
<tr>
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<td>1.69</td>
<td>1.50</td>
<td>1.46</td>
</tr>
</tbody>
</table>

Table 8: Wind loads as per table A.1 of NEN 6702

The wind load to be used for the stability is: \( p_{rep} = 1.50 \times 0.89 \times 1.3 = 1.74 \text{kN/m}^2 \)

Load per meter height of the building is: 1.74\( \times 29 \) (width)\( = 50.5 \text{kN/m} \)
5. Conclusion

With the project presented in this report, it was attempted to design a sustainable office tower able to accommodate change. The infant phase of the “RDM campus” means the future context is uncertain, which is why the capacity to adapt to a range of future situations may prove crucial for its success on the long run. This ability was embedded in the design by creating a high level of functional autonomy between components within the building structure.

The project involves a multi-tenant building for starting businesses. This program relates to the ambitions set out in the master plan for the extension of the RDM campus as well as the need for small scale and temporary office demand in both Rotterdam and throughout the Netherlands. Recent ideas concerning office design and exploitation were studied and integrated in the design. The most important of these being the need for transformability and the rejection of the focus on net surface area, into a building that takes account for the changing needs of office workers in the digital age. The result is an emphasis on flexibility and spacious circulation areas and atria; enabling informal meeting and interaction. The need for transformability asked for research driven design providing a high degree of independence between different building components.

The brief was translated into rational volume with square floor plan measuring 28.8 meters and a height of 51 meters (15 storeys). An additional subterranean floor connects an underground car park to the building via an expo. The volume is build up of modules measuring 3600*3600*3400mm, the basic grid dimensions which can be filled with program with a great degree of independence. The load bearing structure provides for the easy addition and removal of program anywhere in the structure. The volume is envisaged to be subject to densification over time through the addition of additional modules. These “modules” do not concern 100% prefabricated units but are the basic “pixels” on which the composition is based. The load bearing structure provides for the easy addition and removal of program anywhere in the structure.

A number of large atria ensure light to penetrate well into the building. The main atria, which also contain the vacant space for densification, are shaped by external conditions. The axis from the main campus building and school of architecture across the Maas towards the city is an important feature in both the entrance lobby. This axis returns at the 7th level where the restaurant and informal meeting area accommodate social contact within the building. Before anything else, the atria are an essential feature to enable informal meetings and spontaneous interaction within the building. The work places are located behind the closed parts of the façade.

The glazed atria and closed workplaces are intertwined in a simple principle form. This composition offers a rational but nonetheless exciting spatial quality of which the exact shape and arrangement can be easily transformed to user demands changing over time.

The building shows itself as an open structure that seems to be filled with program at random; a three-dimensional grid of solids and voids. This game of closed and open suggests gradual densification of the program. The grid is omnipresent in this composition; along with the cores, which obey to its rules, they form the boundary conditions within which time is allowed to pose new configurations for new conditions. Infill is the variable, structure the constant. The result is a structure with a rational but nonetheless exciting spatial quality of which the interior shape and arrangement can be easily transformed to user demands changing over time.

The infrastructure of the design, the carrier of ever changing program, was of major concern throughout the design process. The potential of the symbiosis between the efficient land use of typical high-rise buildings and the flexibility offered by horizontal housing plots where every property can be built and modified independent of adjacent ones was researched.

The load bearing structure consists of a steel frame. Two decentralised concrete cores in which vertical transportation, service shafts, sanitary and emergency staircases are accommodated provide for stability. The central area of the building is kept free from vertical shafts, therefore allowing a variety of functions to be housed in this zone.
There are two flooring systems to be used in the building; one that is installed during construction and functions as a rigid slab for stability purposes and the other that is added to the structure during the densification process. The floors are based on the hollow floor principle, which means that services are integrated in the structural height of the floor slabs. The service ducts are accessible at all times through the application of a raised surface floor with dismountable panels.

To conclude; the project aims to address the brief through an integrated approach of architecture and engineering. The result is a transformable office tower which may just stand the test of time.
a small bay to the south
intimate boulevard
features a surprisingly
approaching the building from the Rotterdam skyline in the...
From the south-west with the background
decentralised cores upgrade building from a functional core place of informal meeting area
the heart of the circulation space to a
and interaction
a game of solid and void with offering delight during the day
within a rational volume...
...and at night
Literature

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