Application of UHPFRC in the building engineering

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Challenge the future

APPLICATION OF ULTRA HIGH PERFORMANCE FIBRE REINFORCED CONCRETE IN THE BUILDING ENGINEERING

A search for applications of a new material

by

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ABSTRACT

Material properties of concrete have developed rapidly in the past fifteen years. With the advent of Ultra High Performance Fibre Reinforced Concrete (UHPFRC) a new material has emerged which is; much stronger, stiffer and durable than conventional concrete. Nonetheless despite these promising developments and due the high material costs UHPFRC is only rarely applied in small niche markets.

The characteristic properties of UHPFRC differs to a large extent from conventional concrete. With a compression resistance higher than 150 MPa and a flexural strength of at least 7 MPa the material is roughly twice as strong as conventional concrete. Simultaneously as a result of a well composed fine mixture of particles the durability and formability of the material are improved significantly. Taken into account the characteristics properties the material should be considered to be a new material which requires structures to be design with the characteristic properties in mind. By exploiting the characteristics correctly the material can enable innovative and distinctive designs for constructions.

The properties of UHPFRC and the potential to enable distinctive structures make the material interesting for application in the building engineering. Either on behalf of a client or out an urge to push the boundaries architects and engineers have always searched for new construction methods and innovative designs. Due to the characteristic properties, UHPFRC can enable new constructions methods and innovative designs. Some examples of successful application of UHPFRC in bridge and buildings structures have already confirmed the potential of the material. Nonetheless, due the novelty of the material it has only be applied for very few applications and often on a small scale.

The objective of this thesis is to derive an innovative and distinctive design which makes best advantage of the characteristic properties of UHPFRC. By means of a literature study the material specific challenges and possibilities of UHPFRC are researched. Next to this, a list with building applications is drafted allowing a match between; application requirements and material properties to be identified. Subsequently by means of a case study an application of UHPFRC in a fictitious building is elaborated from overall design down to detail. The result of the research is a proposed contemporary design approach which enables an innovative and distinctive design which is made possible by the characteristic properties of UHPFRC. The proposed approach may work to derive designs which are worth spending the additional material costs that are bounded on UHPFRC.

CONTENTS

ACRONYMS

AFGC	Association Française de Génie Civil
CSH	Calcium Silicate Hydrate
fib	Fédération Internationale du Béton / Federation for Structural Concrete
FRC	Fibre Reinforced Concrete
GGBFS	Ground Granulated Blast Furnace Slag
GPA	Gigapascal
HRWRA	High Range Water Reducing Admixture
MPA	Megapascal
N/A	Not applicable
RILEM	International Union of Laboratories and Experts in Construction Materials, Systems and Structures
SETRA	Service d'études techniques des routes et autoroutes / French Road and Motorway Technical Studies Department
SLS	Service Limite State
UHPC	Utra High Performance Concrete
UHPFRC	Ultra High Performance Fiber Reinforced Concrete
ULS	Ultimate Limit State
unkn	Unknown
W/C	Water / Cement

Part I

RESEARCH TOPIC

1 INTRODUCTION

1.1 CONTEXT

The material properties of concrete has been developed rapidly in the past ten years. With the advent of UHPFRC a new type of concrete has been developed which is much stronger, stiffer and durable than conventional concrete. With UHPFRC being a promising material for the construction industry pioneering architects and engineers have sought for suitable applications in buildings and bridges. By now, the material has been used in several projects. Although, because of the novelty of the material it is only used very rarely and only for a very small variety of applications.

Ultra thin balconies are a striking example of an possible application of UHPFRC. After the ultra thin balconies which are applied at Huize het Oosten won the Concrete price 2013 in the category dwelling, everyone in the Netherlands active in the field of building engineering was made aware of the potential of this material. The winner of the concrete price 2013 shows that UHPFRC has already been applied for suitable applications in the building engineering which confirms UHPFRC is a promising material for innovation in the building engineering.

Figure 1: Ultra thin balconies at Huize het Oosten

1.2 THESIS OBJECTIVE

The objective of this research is to exploit the design possibilities of the promising construction material UHPFRC in order to derive an innovative and distinctive design. The design should push to the boundaries of the material possibilities in order to derive a design which is worth spending the additional construction costs that are bounded on UHPFRC. Subsequently, the derived design will be used to propose a general design approach which can be used to derive innovative and distinctive designs by the use of UHPFRC.

1.3 RESEARCH SCOPE AND APPROACH

The primary scope of this research is to design an element which distinguish itself by aesthetics and innovation which will justify the possible additional construction costs. The application of UHPFRC for any application can and should eventual be considered. Though, this research aims at load carrying constructive elements which are visible and have a direct contribution to the aesthetic of a building. Economical and sustainability assessments are not within the primary scope of the research. Considering there importance in the construction industry they are taken into account.

The research has been conducted within two parts. First a literature study has been carried out. Through a thorough understanding of the material the characteristic and distinctive properties have been identified. Besides, the literature study has led to a short-list of building elements whereby UHPFRC can come to its best advantage. Secondly a design study has been conducted whereby one of the selected applications has been elaborated down to detail. Based upon these two parts the research questions are answered, conclusions are drawn and recommendations are done.

1.4 RESEARCH QUESTIONS

The following main and sub research questions are formulated.

1.4.1 *Main research question*

"How can Ultra High Performance Fibre Reinforced Concrete be used to develop and push innovative and distinctive designs in the building engineering?"

1.4.2 *Sub research questions*

1. What are the characteristics and distinctive properties of ultra high performance fibre reinforced concrete?
2. What are specific challenges and possibilities of the production process of prefabricated elements in ultra high performance fibre reinforced concrete?
3. Is the lack of construction regulations for ultra high performance fibre reinforced concrete in the Eurocode a bottleneck for application of the material in Europe?
4. Which building elements has already been constructed in ultra high performance fibre reinforced concrete?
5. Which applications, in the building engineering, are suitable to make advantage of the characteristic properties of ultra high performance fibre reinforced concretes?
6. What is a good design approach to derive a distinctive and innovative design which make use of the characteristic properties of ultra high performance fibre reinforced concrete?
7. Which design principles can enchant the application of ultra high performance fibre reinforced concrete?
8. Are the properties of the current ultra high performance fibre reinforced concretes suitable for innovative and distinctive designs or need the properties to be be modified or further developed?

Part II

LITERATURE STUDY

2 ULTRA HIGH PERFORMANCE FIBRE REINFORCED CONCRETE

2.1 DEFINITION OF THE MATERIAL

Ultra High Performance Fibre Reinforced Concrete is a material with a very fine and optimised mixture. In international literature concrete with a compression capacity between the 150 MPa and 250 MPa are considered to be a concrete of ultra high performance. The fibres added, either metal or a mixture of metal and synthetic, increase the ductility and flexural strength making steel reinforcement bars in some cases unnecessary. The following characteristic properties describe the material UHPFRC [40, 31]:

- Compression resistance generally higher than 150 MPa
- Very fine and well composite mixture with a large amount of bonding agent
- Dense material with a low capillary porosity
- Flexural strength consistently above 7 MPa

New materials often pushes new forms of structures. While arches used to be made by stones; long span girders and trusses where made possible by steel. Although UHPFRC is still called a concrete, the material should be considered to be an entirely new material because of it characteristics properties.

2.2 HISTORY

2.2.1 *Concrete*

Through centuries mankind has come up with various methods and materials to construct there accommodations and structures. In the beginning of mankind mud, straw and wood was used as main construction material. Due to an urge to build better, taller and higher structures, new improved construction materials where developed. What has emerged from this urge can still be seen through ancient structures showing a glimpse of the development in construction methods and materials through the centuries.

First time a material comparable with what we nowaday call concrete was applied in Italy. Near the volcano Vesuvius the Romans mixed volcanic ash, lime, sand and gravel with water. After the reaction of the cementitious components this free formable mixture transformed in a solid, strong and durable material. This development took place a few decades before BC. It turned out to be a major development since all the spectacular structures build by the Romans are made by the use of it [48].

After a long period without much developments next step in the development of concrete was the development of Portland Cement. This cement was officially introduced in 1824 by the English cement manufacturer Josepf Aspdin. The cement consists mainly of lime, alumina, quartz and iron oxide. By heating of these crushed components new compounds arise. Today still, Portland cement is the most commonly used cement in the world [48, 38].

The next invention in the development of concrete was the discovery of reinforced concrete. This invention took place in France. It was Joseph Monier, a gardener, who found out that he could construct very strong baskets and tubes by combining iron braid with cement mortar. His invention dates from 1850. The first large scale application of this invention has been the building of the Reichstag in Berlin which has been constructed in 1894 [35].

2.2.2 *The development of high strength concretes*

Regarding the development of high strength concretes there are two different paths that can be distinguished. The state of the art of ultimate compression strengths realised in laboratories and the characteristic strengths commonly applied in practice.

For decades the characteristic capacity of commonly applied concrete hardly improved. In the beginning of the twentieth century the characteristic compression strength was about 15 Mpa. This only developed to a maximum strength of about 35 Mpa in the sixties [32].

Halfway the twentieth century the ultimate capacity of concrete obtained in laboratories was already much further developed than the commonly applied. Around 1950 concrete with a strength of 70 MPa was obtained by Otto Graf. This was already improved quickly by Kurt Walz who obtained concrete with a strength of 140 MPa in 1966.

The application of these improved mixtures was however severely hold back by the complexity of the production process. It took up to the late eighties that concrete with a strength of 100-115 MPa could be applied in building practice. The invention of powerful superplastifiers and the effects of silica fume played a major role in this development. The concrete however remained to expensive compared with commonly used concrete for a widespread application [26].

The first researcher who produced UHPC was the Danish Henrik Bache who published about it in 1981 [21]. After this invention the development in concrete speeded up in the early nineties. A large scale research in co-operation with universities and research institutes from several countries was initiated by some French companies. Initially the research was focussed upon developing a material with a minimum of defect so that a greater percentage of the potential of the components from the mixture could be achieved. In order to prevent brittle failure fibres where added improving the ductility. The outcome of the research led to several formulation of Ductal the product Lafarge and Bouyues [22]. Ductal is an UHPFRC with a characteristic compression capacity of 200 MPa. Due to its excellent properties the product is now applied all over the world.

As from the eighties the ultimate strength of in practice applied concrete was held up at around 105 MPa, a major development took place just before the turn to the twenty-first century. The extensive research done made it possible to apply concrete with an ultra high strength in practice. The entire range from 105 MPa to 200 MPa was passed over with the construction of a cyclist bridge in Sherbrooke, Canada. At this bridge prefab element with a characteristic strength of 200 MPa have been applied. Although the the range between 105 - 200 MPa could as well hold different and promising mixtures this application showed what strength can now be applied in practice [21]. The maximum compression capacity of concrete made in laboratory can now, by further optimisation of the packing density and by heat and pressure curing, reach a up to a strength of 800 MPa [31].

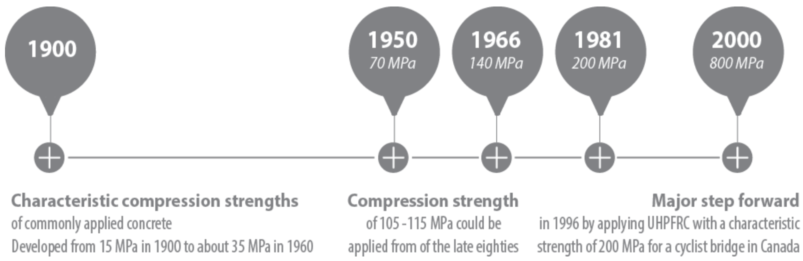


Figure 2: Timeline, laboratory (above) practice (bottom), see text above for sources

2.3 Components of the material

An UHPFRC consists mostly of similar components as conventional concrete. Though, for a mixture of high strength concrete there are several additional components added. Also the ratios between the various components are entirely different as depicted in Figure 3 . The following principles are used to increase the performance of UHPFRC [21]:

- To reduce internal stresses the maximum aggregate size is reduced.
- To obtain the maximum out of the potential of the mixture content the packing density has been increased as much as possible.
- Unreacted water after hydration causes voids which increase internal stresses, by decreasing the W/C ratio to a level that unreacted cement remains reduces these voids and pores.
- Increasing the strength of concrete also increases the brittle behaviour, to avoid brittle failure fibres are added.

The various components in a UHPFRC are introduced in this section.

2.3.1 *Past*

2.3.1.1 *Cement*

With cement being the binding agent in concrete, it has got a very decisive influence on both the curing process, the final properties and appearance of the concrete. The basic principle of concrete is the chemical reaction that occurs after the cement has come into touch with water. This reaction creates calcium silicate hydrate, also known as CSH gel and cement past. When this past hardens out it becomes of stone-like strength. [32]

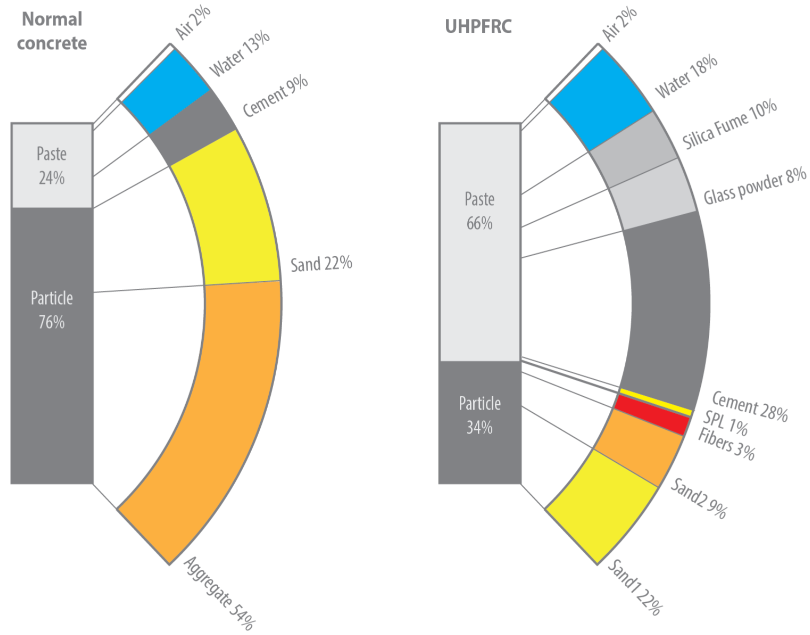


Figure 3: Concrete mixture, normal (left) UHPFRC (right) [31], own illustration

PORTLAND CEMENT The most common used cements are Portland cement and derivatives from it. As already mentioned in section 2.2 the basic components of Portland cement are lime ($CaCO_3$), alumina (Al_2O_3), quartz (SiO_2) and iron oxide (Fe_2O_3). After these components has been crushed and heated to temperatures of around 1500 °C new compounds arise. The hereby formed finely powdered crystalline minerals are called portland clinker, or just clinker. By altering the ratios between the basic components and the temperature during the production process specific properties can be obtained. A low level of iron oxide leads for instance to a more white cement and a lower level of CaO (which is formed from $CaCO_3$) leads to a lower heat production during curing [32, 38].

GRANULATED GRADED BLAST FURNACE SLAG AND FLY ASH Granulated graded blast furnace slag (GGBFS) and fly ash are pozzolanic particles that can replace a large amount of portland clinker and improve the density of the concrete. Pozzolans are materials that, if they are finely ground and if they are in the presence of moister, reacts with calcium hydroxide ($Ca(OH)_2$). After this reaction the pozzolas will form cementitious components. The ($Ca(OH)_2$) originates from the chemical reactions of the clinker. Replacing 35 % up to 70 % of the cement by GGBFS will influence the properties of the cement considerably. It will result in a slower hardening process and thereby also a lower heat production. [32, 38, 9].

2.3.1.2 *Silica fume*

Silica fume is just like GGBFS and fly ash a pozzolanic material. However in comparison the particle size of silia fume is two magnitudes smaller. As the average diameter of fly ash particles is around $10 - 12\ \mu m$ the diameter of silica fume is only $0.1 - 0.12\ \mu m$. Due to this small size the silica fume particles fills the voids between the larger cement particles and thereby it optimizes the transition zone. The amount of added silica fume generally varies from 5% to 20% of the weight of the cement. Disadvantage of the addition of silica fume is that the great particle surface increases the water demand significantly. To avoided this water reducers must be added in order to retain the workability of the mixture. [32, 8].

2.3.1.3 *Glass powder*

Glass powder, also known as silica powder or quartz powder, is frequently used to optimize the packing density of the mixture. With a particle size of $1.5 - 5\ \mu m$ the additions fit in between the relatively large particles of the GGBFS and/or fly ash and the relatively small particles of the silica fume. Glass powder can react with substances from the cement hydration to CSH gel. Which improves the cohesion of the structure. [9, 31] Large glass powder particles can as well be used as addition between the cement and sand particles.[9]

2.3.1.4 *Water*

As already mentioned water initiates the principle reaction of concrete, as well it lubricates the mixture to ensure workability. [32] Because cement is relatively an expensive component sufficient water is added in ordinary concrete to provide water for every cement particle to react. Disadvantage of this high w/c ratio is that water leads to pores and voids. Therefore the amount of water is significantly reduced in UHPFRC. Cement particles will stay unreated, but pores and voids are avoided. [21] To ensure the workability water reducers are added instead.

2.3.2 *Particle*

2.3.2.1 *Water reducers*

High-range water-reducing admixtures (HRWRAs) are often known as super plasticizers. The functioning is based upon giving the surfaces of the cement particles an electric charge causing them to repel each other [38]. The HRWRAs can be added in a concrete mixture for the following three reason: Firstly to increase the workability without changing the mixture composition. Secondly to reduce the amount of cement and water added in order to reduce the total creep, shrinkage and thermal strains. Thirdly to reduce the w/c ratio to improve the concrete strength and durability. [32] The addition of HRWRAs allows concrete mixtures to be self compacting as well it improves the final quality. [8]

2.3.2.2 *Fibres*

As already mentioned in the intro of this chapter fibers are added to avoid brittle failure of UHPCs. Fibres can improve the performance of concrete on two different levels. On a micro-level it can improve the strength of the matrix and on a macro-level it can improve the post fracture behaviour.

The first cracks in concrete originate at the interfacial transition zone between the cement matrix and the aggregates. As smaller diameter reinforcement bars with a lower spacing distance are applied these cracks are reduced. Similar to this small fibres can improve the strength of the concrete matrix and thereby increase the fracture toughness of the material.

To improve the ductility after cracking fibres are great addition. As the tensile capacity can drop suddenly after cracking fibres are a good addition to take over this tensile force and thereby improving the ductility. [4]

Different materials are used for fibres, fibres of these material all have there specific purpose. Orgass and Klug [36] enumerate the following materials with their effect on the concrete:

- Steel Fibres:
  - increase of fracture energy, subsequent improvement of ductility
  - increase of strength (compressive strength, tensile strength)
  - Reduction of tendency for cracking
- Polypropylene Fibres (PP fibres):
  - Decrease of microscopic crack growth with high loading
  - Gain in fire resistance
  - Decrease of early shrinkage
- Glass Fibre:
  - Reduction of internal stresses within young concrete

In general a mixture of different types fibres is added. Such a mixture includes fibres of various length and/or different material. The addition of a sufficient fibres, about 2% of the volume fraction, will result in ductile post fracture behaviour an increased flexural strength.

2.3.2.3 *Sand*

In traditional concrete more than two third of the total mixture is aggregate, in UHPFRC the share is only about one third. In order to reduce the peak stresses in UHPFRC the maximum particle size of aggregates is reduced significantly by adding only sand particles. These particles are selected by diameter, a mixture could for instance contain particles with a diameter of $0.1\ \mathrm{mm}$ and $0.5\ \mathrm{mm}$ [31]. Requirement of the aggregate particles is that they have at least the same strength of the cement matrix.

2.4 PRODUCTION PROCES

Producing UHPFRC put high demands to the productions process. In order to meet these requirements UHPFRC is practically only produced in factories for production of prefab elements.

2.4.1 *Workability*

In general the small size of particles in UHPFRC result in a mixture with excellent workability [24]. Through the addition of HRWRA the workability is maintained even at low w/c ratios. This property not only simplifies the production process it reduces as well the noise in factories. However the added fibres have a negative influence on the workability of a mixture. The high surface area and the long elongated shape of the fibres differs form the other aggregates making them affecting the workability. Depending on the composition of the mixture and the size and shape of the fibres the practical fibre content is limited [16].

2.4.2 *Shrinkage*

One of the major challenges during production is de excessive shrinkage that occurs during the first hours of hardening. There are three types of shrinkage that can be distinguished. Because of the high cement amount in UHPFRC especially the autogenous shrinkage will result in a significant volume reduction. After their research Eppers and Muller conclude that Autogenous shrinkage strain of UHPC can be very high and must be taken into account both for assessing the risk of cracking and for obtaining the correct dimensions of structural elements. [14]. During the production process the effects of this shrinkage must be taken in to account for instance by the use of formwork of semi-hard materials.

CHEMICAL SHRINKAGE As the volume fraction of the hydration product is smaller than the volume fraction of the original components the volume will reduce after the chemical reaction. This reduction is about 25 % of the original volume of the chemical bound water.

AUTOGENOUS SHRINKAGE During the hydration process the available water reacts. First the water from the larger pores will be used followed by water of the small pores. This could as well be explained as water that is being withdrawn from small pores. When no water is added to this process, this internal desiccation will result in a volume reduction. This shrinkage is especially influential by low w/c ratios.

PLASTIC SHRINKAGE As the heaviest particles tend to move downward the lighter water particles will move to the surface. Depending on the mould type and environmental conditions the water will disappear completely and plastic shrinkage can occur. This type of shrinkage can result in cracks over the entire width of a 400 mm thick slab. [53]

2.4.3 *Mixing procedures*

A mixture of UHPFR concrete contains many different particles. Because of the large proportion of very fine sized additions it is of importance that all the particles are evenly distributed. Small particles have the tendency to agglomerate. To prevent this it is recommended to first mix all the additions without the HRWRA and water which should be added as last. [31]

2.4.4 *Heat Treatment*

The performance of a concrete mixture can be improved significantly through hot curing. The most commonly used method is curing at 90 °C for 48 hours under a relatively high humidity. By applying such a heat treatment the pozzolanic reactions are accelerated leading to longer element chains and a more intertwined microstructure [19, 7]. By curing at a temperature above 250 °C additional reactions will occur and water will be extracted from the cement paste. This will make the concrete more a ceramic material and strengths up to 800 MPa can be obtained [25]. The optimum temperature for heat curing depends on the composition of the concrete [7]. After a heat treatment shrinkage and creep behaviour is drastically improved or these phenomena will hardly occur after treatment. Hot curing is however very expensive, a room has to be heated for several days and moulds can not be reused as long the treatment takes place.

2.4.5 *Compression curing*

By curing under compression even higher compression strengths of 800 MPa can be achieved [31]. However this treatment is very difficult to apply for large elements.

2.5 PROPERTIES

With the definition of the material at section 2.1 the characteristic properties of UHPFRC are already given. In this section other material aspects are described.

2.5.1 *Creep*

Compared with ordinary concrete the creep behaviour of UHPFRC is better, since the creep effect is significantly smaller. As ordinary concrete got a creep factor of about 2 the creep factor of UHPFRC will be around 0.8. In case heat treatment is applied this coefficient will decline further to 0.2 [2].

2.5.2 *Fire resistance*

A well known and dangerous phenomena attached to concrete is spalling. When concrete is exposed to fire, water particles inside the concrete will heat up and expand. This pushes off parts of the concrete [20]. In general UHPFRC is more sensitive for spalling than ordinary concrete because of the dense microstructure. As UHPFRC has a low w/c ratio there is not much water inside the material, however in the dense and brittle microstructure a high steam pressure can build up resulting in high internal stresses and spalling [6].

The addition of polypropylene fibres can improve the fire resistance. As these fibres melt above 150 °C and volatilize at temperatures above 200 °C they create additional capillary pores which give the water particles the additional required space [6]. Good fire resistance properties are not evident for UHPFRC. The properties strongly depend on the specific material composition. By conducting many tests the mechanical properties of a material exposed to fire can be determined [41]. According Rogier van Nalta in Appendix A section A.4 the long development process of the current UHPFRCs like CRC have resulted in desirable fire resistance.

2.5.3 *Fibre orientation*

At subsection 2.3.2 the function of fibres in concrete has been explained. Often a crack starts at the weakest point of a material. With concrete being a composite material a homogeneously distribution is vital for the performance properties. This applies especially for fibres which has to be distributed as homogeneously as possible [16].

The orientational factor K has been introduced to indicate the homogeneity of the fibre distribution. Material design strengths must be modified by divided through this orientational factor. When an isotropic fibre dividing is assumed the fibre orientational factor is 1. When fibres are distributed in a more favourable way an orientational factor of $K < 1.0$ can be applied and an orientational factor of $K > 1.0$ has to be applied in case of an unfavourable distribution. [10].

According the recommendations of the AFGC-SETRA the orientational factor must be determined by results of tests on a representative model of the structure. In case these values are not yet known a factor of $K = 1.75$ should be considered for local effects and a factor of $K = 1.25$ should be considered for any other effects. Although the way of casting can lead to favourable fibre distribution a minimum value greater than one is recommend for the K value [47].

2.6 Calculation codes

Because of the novelty of this material an approved Eurocode has not yet been published. However companies and research groups have already been developing code look alike calculation documents. In this section an overview of relevant documents will be given.

2.6.1 *AFGC/SETRA 2002*

A document with similar content as a building code is available. The document concerns an extensive recommendation published by the French Association for Civil Engineering AFGC [47]. Various parties involved in the construction industry have been brought together by an initiative of the French road an traffic governmental agency SETRA. The document is published under the name AFGC-SETRA recommendations. In various literature the document is acknowledged as the most code alike document. As the document provides a comprehensive set of calculation methods, which provide sufficient basis to prove structural safety of constructions made with UHPFRC.

2.6.2 *Beton kalender 2013*

The first document which will be international recognised as design code is likely to be developed by the International Concrete Federation *fib*. The *fib* Task Group "Ultra High Performance Fibre Concrete" lead by Prof. Walraven is developing a international approved document to be used as building code [55]. Unfortunately this document is not yet available neither is a draft version of this document.
However before a document is published by the fib Task Group the "Beton Kalendar 2013" has already been published this document developed by several professors of the University of Kassel in cooperation with Prof. Walraven [26]. The eventual document of the fib Task Group is likely to be rather similar with the Beton Kalendar 2013, as well the document will be in English instead of German.

2.6.3 *Japan Society of Civiel Engineers*

The document "Recommendation for Design and Construction of High Performance Fiber Reinforced Cement Composites with Multiple Fine Cracks (UHPFRCC)" is published in 2008 by the Japanese Society of Civil Engineers [46]. The documents is based upon the AFGC-SETRA recommendations though the document it is altered specific for the Japanese available material and building standards.

2.6.4 *CAE Nederland*

The Dutch independent consultancy firm CAE has developed calculation models for elementary calculations with UHPFRC [17, 18]. These models which are published in the Dutch journal Cement offer calculation methods for normal force, shear force and moment capacity calculations. Important noticing is that the published models are not meant to act as a building code, but purely to offer a method to perform elementary calculations. The models correspond with the Eurocode 2 as much as possible and are based upon published documents like the AFGC-SETRA recommendations and the Japanese equivalent.

2.7 CALCULATION METHODS

In this section the calculation methods used in the design phase are introduced. An elaborated version of these methods can be found in Appendix C.

2.7.1 *Column*

The column calculations in the design phase will be done with the use of the column graph developed by CAE. The use of column graphs is an conventional method for normal strengths concrete. The use of the graphs is a quick way to determine the size and the required reinforcement ratio's of columns. In a column purely loaded in compression moment forces will occur due to geometrical and material imperfections. These forces have to be considered for column calculations. For an elaborated explanation of the method to be used in the design phase see subsection C.1.1 in the appendix.

2.8 BRANDS AND TYPES

There are several types of UHPFRC commercially available. In this section an overview of brands, mixtures and characteristic properties is given.

2.8.1 *BSI/CERACEM*

The Béton Spécial Industriel (BSI) is an UHPFRC developed by the major french construction group Eiffage in the late nineties. The first patents were registered by Eiffage in 1998. The first time this material was applied on a large scale was for structural beams in the Cattenom nuclear power plant which was built in 1998. Two years later in 2000 Eiffage joined forces with the product developer Sika. In a partnership they developed a range of products under the name CERACEM. The CERACEM mixture does not make use of heat treatment which simplifies the production process. The BSI mixtures which was used for the cattenom nuclear power plant was primarily chosen because of it durability properties. After that first application the material has as well been used for application where the aesthetics played a major role. This was for instance the case for the canopy of the Millau toll station hoofdstuk 3 [52, 15, 56].

2.8.1.1 *Mechanical properties*

BSI-Ceracem has got two different product lines of BSI. The structural line and architectural line differ from each other mainly in the structural strength performance. It indicated as well that the properties of BSI-CERACEM can be tailored to specific needs. The properties of the BSI-Ceracem mixture as applied at the canopy of the Millau Viaduct toll gates and the properties as given in the AFGC-SETRA recommendations are provided in the table.

	BSI-CERACEM$_{(A)}$	BSI-CERACEM$_{(B)}$
Compressive strength [MPa]	165	180
Flexural strength [MPa]	-	45
Tensile strength [MPa]	8.8	9.1
Youngs modulus [GPa]	57	65
Density [Kg$\cdot 10^3$ / m^3]	2.75	-
Creep Coefficient	1	0.8
Total shrinkage [μm / m]	700	570
Fibre content [%]	3	-

(A) Properties of mixture used for Millau Viaduct [56]

(B) Properties as given at AFGC-SETRA recommendations [47]

(-) means property is unknown

2.8.2 *Ductal*

The French product Ductal dates from the late nineties and has been developed by a cooperation of several companies and universities. The contractor in civil and structural engineering Bouygues, the chemical materials manufacturer Rhodia and the construction materials manufacturer Lafarge co-operated with several universities in Europe. The result from this cooperation are several patented Ductal formulations. After the product was introduced in Europe it quickly spread out to Asia, Australia and America. The product is suitable for both structural as decorative applications [2, 40].

2.8.2.1 *Mechanical properties*

There are two ranges of ductal products the FM and the FO range.

THE DUCTAL-FM product line is developed for structural applications. Due to the use of short steel fibres the properties are characterized by high tensile and bending strengths. The flextural strength can even go up to an impressive 50 MPa. Without heat treatment the compression capacity strength of 150 MPa is guaranteed which can go up to a strength of at least 180 MPa with a 90 degree heat treatment for 48 hours.

THE DUCTAL-OM range is developed for applications where the mechanical properties are less demanding. In the mixtures organic fibres instead of steel fibres are added. These fibres greatly improves the surface finish. Adding pigments can create a wide range of colours, as well the material is very suitable to apply surface textures [12].

	FM(A)	FO(A)	DUCTAL(B)
Compressive strength [MPa]	150, 180(1)	110, 150(1)	200(1)
Flexural strength [MPa]	15 - 45	10 - 30	42
Tensile strength [MPa]	-	-	9
Youngs modulus [GPa]	50	40	58
Density [$Kg \cdot 10^3$ / m^3]	2.4	2.2	-
Creep Coefficient	< 0.8	< 0.8	1.0, 0.3(1)
Total shrinkage [µm / m]	500	500	550
Fibre content [%]	2.15	4.3	-

(1) In case heat treatment is applied.

(A) Properties as given at website supplier [12]

(B) Properties as given at AFGC-SETRA recommendations [47]

(-) means property is unknown

2.8.3 *Compact Reinforced Composite*

The Compact Reinforced Composite (CRC) is an UHPFRC that has been developed by the cement and concrete laboratory of Aalborg Portland A/S in 1986. The CRC mixture is a UHPFRC mixture that contains a high content of fibres combined with conventional reinforcement bars. Reinforcement ratios up to a ratio of 6 % is even possible. Due to the combination of steel with UHPFRC the material can withstand extremely high compression and flexural forces. Even compression forces up to 400 MPa are possible. Today this CRC technology is frequently applied in the building industry. This is also why by volume the CRC is the most used UHPFRC in the world. Originally CRC was designd for heavily loaded structures. However it is more often applied for structural less demanding applications like stairs and balcony slabs. [40]

2.8.3.1 *Mechanical properties*

The mixture contains a very high content of microsilia and a very low w/c ratio of typically 0.16. With curing at ambient temperatures the mixture results in a compression strength of 150 MPa. This strength can be increased, up to 400 MPa, by means of heat curing and by addition of different types of aggregates. In precast elements the mixture contains about 2 - 4 % fibres. When the mixture is used for joints insitu a fibres content of 6% can be used [1]. CRC is characterized by concrete in combination with a very dense structure of reinforcement bars. As a result the material can restrain extremely high flexural strengths up to 300 MPa. It should however be taken into account that the application of reinforcements bars goes on expense of the formability of the material.

	CRC$_{(A)}$
Compressive strength [MPa]	140 - 400
Flexural strength [MPa]	100 - 300
Tensile strength [MPa]	-
Youngs modulus [GPa]	40 - 100
Density [$Kg \cdot 10^3$ / m^3]	2.6 - 3.5
Creep Coefficient	-
Total shrinkage [μm / m]	-
Fibre content [%]	-

(A) Properties as given at website supplier [1]

(-) means property is unknown

2.8.4 *BCV*

The Beton Composite Vicat (BCV) is a product of the cement manufacturer Vicat and the French concessions and construction company VINCI. It is an alternative for the better-known BSI and Ductal. The first time this material was used on a large scale project was for the construction of a motorway bridge near Grenoble, in France [33].

2.8.4.1 *Mechanical properties*

There are three different types of BCV available. These types are, just as ductal and BSI, divided into a 'structural line' and 'color line'. Naturally the structural line is meant for load bearing applications and the color line is meant for architectural and thin elements. Main difference between the types is the fibre content. In the table the characteristics are given. As can be seen the compression strengths of this UHPFRC are relatively low. Only the BCV-2 can, with a characteristic strength of 150 MPa after heat treatment, be considered to be a UHPFRC according the definition as described at section 2.1. Because of the very dense matrix all the three types does however have all the durability advantages of a UHPFRC [33, 39, 37]. The relatively low amount, or the lack of, fibres does have a favourable effect upon the price of the material since the fibres are relatively an expense component.

	BCV-1	BVC-2	BCV-3
Compressive strength [MPa]	135	167	130
Flexural strength [MPa]	4.7	16.4	23.1
Tensile strength [MPa]	-	-	-
Youngs modulus [GPa]	40 - 100	45	55
Density [Kg·10^3 / m^3]	2.3	2.4	2.25
Creep Coefficient	-	-	-
Total shrinkage [μm / m]	-	-	-
Fibre content [kg/m^3]	non	79(1)	158(2)

(1) Fibres with a length of 13 mm

(2) Mix of fibres 1/3 with a length of 13 mm and 2/3 with a length of 20 mm

(-) means property is unknown

2.8.5 *CEMTEC*

The CEMTEC is an UHPFRC developed in the French LCPC laboratory. During a research project at the swiss Ecole Polytechnique de Lausanne the application of CEMTEC for repair work of bridge decks has been investigated. This research has led to effective method to strengthen bridges with the use of UHPFRC [40]. Although the mixture has already been used for application in Switzerland and Canada not much information can be found about it.

2.8.5.1 *Mechanical properties*

Generally this mixture contains a relatively high amount of fibres, up to a volume fraction of 11 % [40].

	CEMTEC(A)
Compressive strength [MPa]	205
Flexural strength [MPa]	20
Tensile strength [MPa]	-
Youngs modulus [GPa]	55
Density [$Kg \cdot 10^3$ / m^3]	-
Creep Coefficient	-
Total shrinkage [μm / m]	-
Fibre content [%]	> 11

(A) Properties given in an article [40]

(-) means property is unknown

3 APPLICATIONS

The material UHPFRC has been used for several application both for application in building engineering as for applications in different fields. Based on some striking examples an overview is given of what the diversity of applications the material has been used for until now. The first application given can as well be considered to be the first genuine application of UHPFRC in the construction industry.

3.1 FIRST APPLICATION

The first time UHPFRC has been used for an large scale application was in 1997 for a pedestrian bridge in Sherbrooke Canada. The Bridge spans 60 meters and the construction consists of six prefabricated slab segments which are supported by an open-web space truss. The slabs are made of ribbed slabs of 30 mm thick with transverse prestressing. The bottom chord and diagonals are made of stainless steel tubes which are filled with Ductal UHPFRC [2]. The diagonal tubes which measure a diameter of 150 mm are prestressed to prevent tensile forces and to avoid buckling. The six elements are connected by longitudinal prestress which is placed in each longitudinal flange, this prestress is externally anchored [42, 23]. Because prior this bridge, only concrete with a maximum compression capacity of only 105 MPa was used the application of UHPFRC with a compression strength of 200 MPa was a major step forward [21].

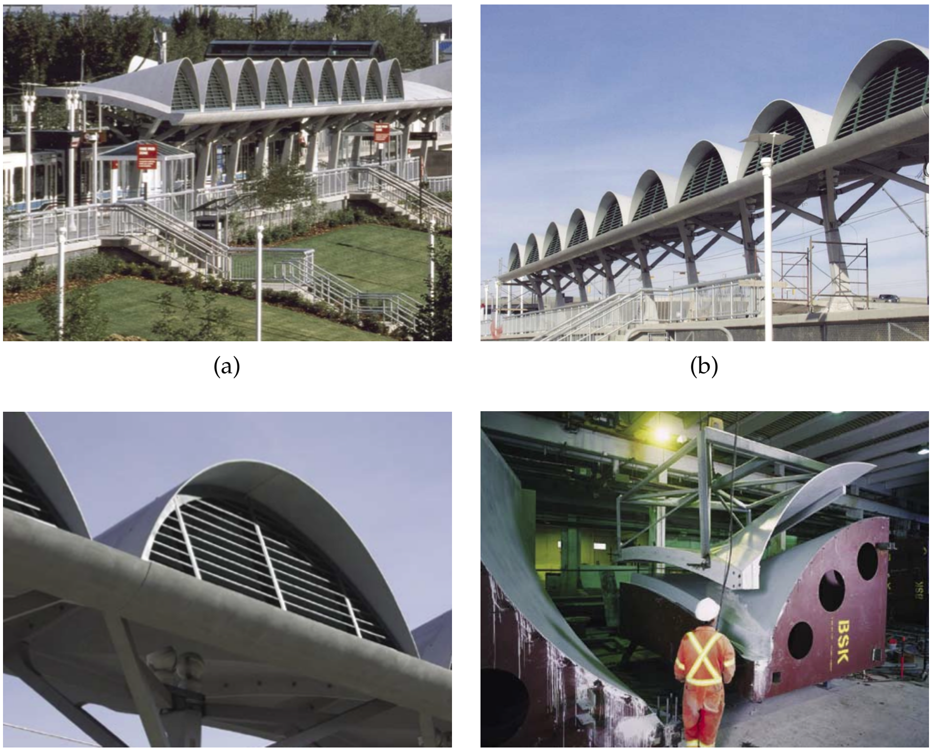
(a) (b)

Figure 4: Sherbrooke pedestrian bridge [2], UHPFRC used for filling of space-truss tube elements

3.2 APPLICATIONS IN BUILDING ENGINEERING

3.2.1 *Shawnessy Light Rail Transit Station*

The application of UHPFRC for the thin shell roof of the Shawnessy Light Rail Transit station in Calgary Canada has been one of the first applications of this material in a building structure. The architect first considered the roof shell to be made out of steel, however is has been executed in Ductal UHPFRC from Lafarge. Since the project has been one of the first application of UHPFRC the Shawnessy LRT Station has been a very educational project. As the production process has created a number of challenges especially with the pouring of the concrete in the injection mould. The unique result has demonstrated the potential of this material and showed that the UHPFRC can be used for thin-shelled roof systems with the same thinkness of a steel alternative. It also showed benefits such as superior durability, impermeability against corrosion, improved aesthetics and formability. After the application of UHPFRC for this roof shells it was concluded that the challenge for the implementing of UHPFRC as widely applicable material for building structures lies in finding optimal standardized shapes. For these shapes contractors and precasters can invest in formwork making mass production possible resulting in cost savings [5].



(a) (b) (c) (d)

Figure 5: Shawnessy Light Rail Transit Station [5], UHPFRC used for shell roof slabs, cast in injection moulds

3.2.2 *Huize het Oosten*

The ultra thin balconies applied at Huize het Oosten are a striking example of the application of UHPFRC in Netherlands. As winner of the Dutch Beton prijs 2013 in the category dwelling this building has received much publicity [13] . The advantage of the application of UHPFRC for balconies is versatile. Because of the strength and density of the material the balconies can be executed very thin namely only 65 mm at minimum. Not only this creates an aesthetic interesting appearance, it also reduces the weight for the structural bearing system and for the connection system. The reduced connection system reduces as well the thermal loss which is a recurrent problem for conventional balconies. For this application the various advantages of the UHPFRC make it possible to spend the additional costs bonded to this material. The applied balconies are a product of the Danish company Hi-Con. This company has got besides experience with the application of UHPFRC for balconies as well experience with the use of this material for Ultra thin stairs in buildings.

(a) Huize het Oosten

(b) Huizen het Oosten

(c) Project Amber, Delft

(d) Test load

Figure 6: Huize het Oosten [50], UHPFRC used for ultra thin balconies for residential buildings

3.2.3 *Peage du viaduc de Millau*

As the viaduct of Millau in France is well known because of the highest bridge in the world. However the aesthetically ambitious roof of the nearby toll station gets as well attention. The roof is a design of the architect Michel Herbert and is build by the French Eiffage-group who decided to use BSI concrete for the roof slab. When the roof structure was finished in 2004 it was one of the first large scale applications of the UHPFRC BSI. The curved roof covers a length of 98 at 28 meters and consists of 54 separate curved slabs. The slabs are connected with the help of prestressed beams. The bond between the various elements is improved with the use of epoxy resin based adhesive. One of the challenges to overcome during the designing phase has been the thermal loading which can result in significant tensile stresses [30, 41]

(a)

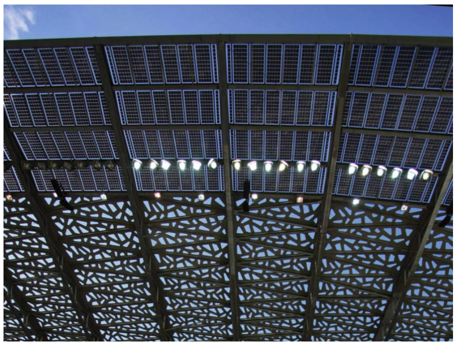
(b)

Figure 7: Peage du viaduc de Millau [30], UHPFRC used for large scale flat slab curved roof

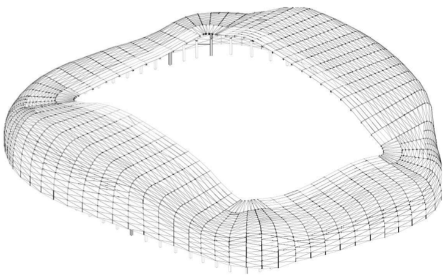
3.2.4 *Stade Jean Bouin*

In the last few years the French architect Rudy Ricciotti has set the benchmark regarding the application of UHPFRC in building engineering. With several buildings and bridges he has shown what kind of applications are possible with this material. One of his projects where the UHPFRC has been applied on a large scale is the new Jean Bouin rugby stadium in Paris. The material Ductal is used for the entire outer shell of this 20.000 places stadium. The facade elements are self-supporting and form a physical separation between the in- and outside of the building. Due to the properties of the material a very open structure was possible which has resulted in a very light and bright facade. In the roof elements glass has been placed to make the panels as well a waterproof occlusion. Between the roof and the facade transition panels with a different curvature has been applied. In total only 18 different type of moulds where required to produce the 23,000 m^2 outer shell entirely out of prefabricated panels [45]. With full-scale prototype tests a structural life load of 50 years has been conducted, which guarantee the durability [11].

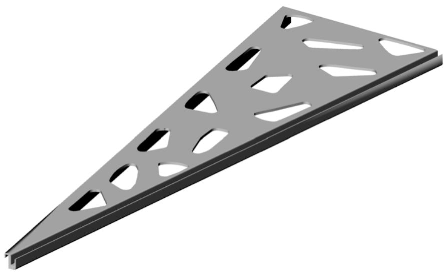
(a) [43]



(b) [49]



(c) [45]



(d) [45]

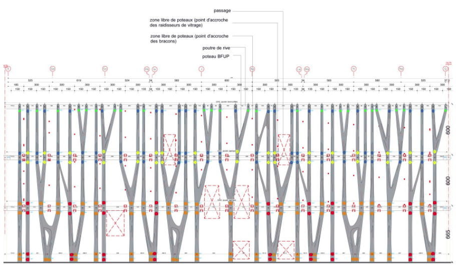
Figure 8: Stade Jean Bouin, UHPFRC used for self supporting facade and roof elements

3.2.5 *MuCEM*

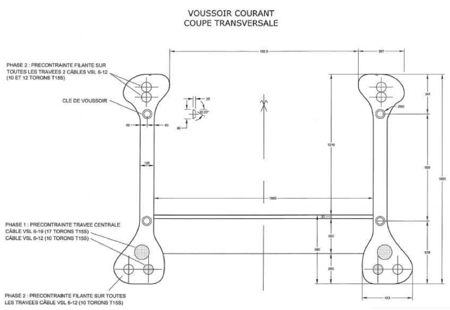
One of the latest pioneering projects of Rudy Ricciotti is the museum MuCEM in Marseille. The museum is about the civilisation of Europe and the Mediterranean and has been opened in 2013. The building has already become a iconic building due to the advanced implementation of UHPFRC. The concrete has been used for three different applications. Firstly, load bearing columns has been made with UHPFRC. These columns resemble trees and have smooth organic shapes. The random-looking effect of the columns is obtained by varying different parts at different manners. The parts are subsequently externally pretensioned to connect the various parts. Secondly, self-caring facade elements are made with UHPFRC. The elements are rather similar with the facade elements used for the Jean Bouin stadium although in this case they are square. Because of the density of the material they can easily withstand the tough conditions close to the salty sea. Just as at the Jean Bouin the facade element function as sunshade and as separation between the in- and outside of the building. Thirdly, UHPFRC has been used for a pedestrian bridge which spans a impressive 115 m. This bridge is only 1800 mm high and is build up out of several parts which are connected with prestress [44].

(a) [28]

(b) [28]



(c) [28]



(d) [44]

Figure 9: Museum MuCEM, UHPFRC used for self supporting facade/roof elements, load bearing columns and 115 m pedestrian bridge

4 BUILDING ELEMENTS

As a building consists of various elements it should first be decomposed before elements can be selected whereby the properties of UHPFRC deliver the most added value. In this chapter the elements of a building are listed and analysed.

4.1 DECOMPOSITION OF A BUILDING

In the construction industry several organisations developed tables and listings which cover the various elements of a building. The following two listings are used to inventory the elements of a building

STABU STANDARDISED SPECIFICATIONS SYSTEMS The Stabu standardised specifications systems is a system developed by a cooperation between various parties in the Dutch construction branch. The organisation intends to publish and administer standardised specifications systems for residential and non-residential buildings. Where their system is primarily meant to transfer building information between various parties, it provides an appropriate basis for an innovatory of building elements

SFB-SYSTEM The SfB-system is a system developed by the Dutch construction cost consulting firm BKS Schagen. The system is bases upon a Swedish variant and is meant to order layers and objects of buildings for cost calculations. The system is meant for residential, non-residential and industrial buildings. The SfB-system is used to supplement the derived list.

4.2 SELECTION CRITERIA

As mentioned in section 1.3 the emphasis of the research is on load carrying constructive elements which are visible and have a direct contribute to the aesthetic appearance of a building. Therefore elements which are *Load carrying* or *Influence the aesthetics of a building* are selected. The list with building elements hereby derived is presented in section B.1 in Appendix B

In order to reduce the list, the elements are subsequently assessed upon the degree to which they are *Load carrying* and to which they *Influence the aesthetics of a building*. This resulted in the following list with elements which all received the maximum grade for both considered aspects.

- Column
- Load carrying external wall / Façade
- Load carrying interior wall
- Structural roof element

- Main support structure

In addition to these elements, stairs and slopes are also considered and added to the list. With both stairs and slopes reviewed as a moderate load carrying element, because they only carry a low weight, they do not match the set selection criteria. Nonetheless, the structural design of these elements can be challenging and interesting. Therefore an exception has been made for these elements.

4.3 VARIANTS OF BUILDING ELEMENTS

The building elements derived at the previous section are elements which could be designed in UHPFRC and which are fully within the scope of the research. However the typology column, façade or roof element are rather unclear and vague definitions as numerous different type of these elements exist. In order to clarify the elements, possible variants of the them are identified. The listing of the variants can be found in section B.2.
In order to get an clear overview of the possibilities and disadvantages of the variants when they are made out of prefabricated UHPFRC, the variants are subjected to a pro-con analysis. This analysis can be found in section B.2.

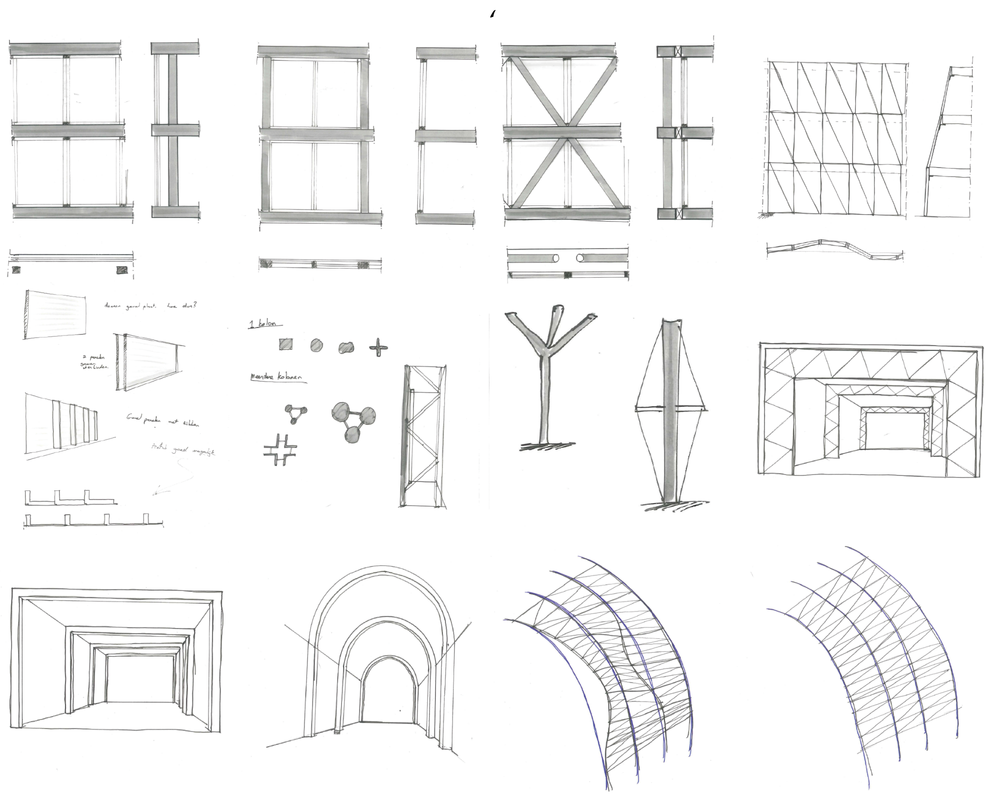


Figure 10: Different variants of the elements

Part III

DESIGN STUDY

5 DESIGN PRINCIPLES

In the literature study the characteristic properties of ultra high performance fibre reinforced concrete have been inventoried. In the conclusions of the literature study the innovative and distinctive properties of UHPFRC are identified. The design of the case study should exploit these properties. By doing so a design which is, just like the properties, innovative and distinctive should be derived. The properties which are listed in the conclusions of the literature study are given here below.

- The excellent structural properties such as
  - High compression capacity
  - Considerable flexural strength
- The durability of the material
- The formability of the material

One application is chosen to be elaborated in the design study. In the literature study several applications that can exploit the characteristic properties of UHPFRC are inventoried. In consultation with the graduation committee a *Load caring facçade that contribute to the global stability of the building* is chosen to be elaborate in the design study.

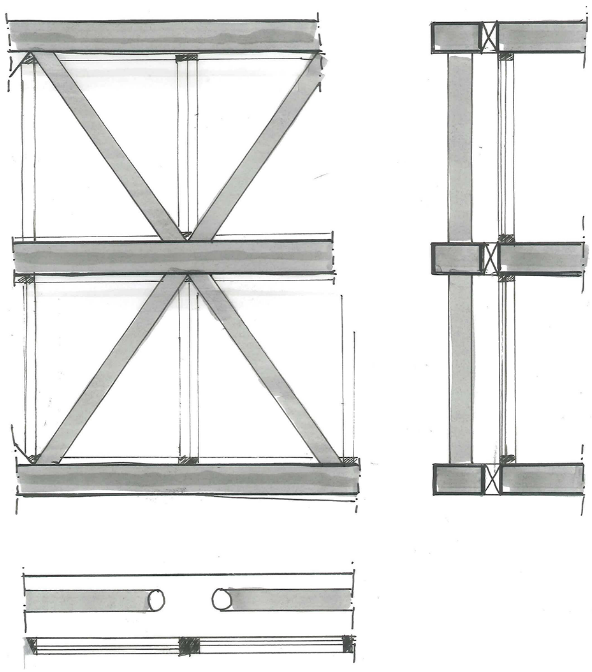


Figure 11: Typological presentation of the application

5.1 DESIGN PRINCIPLES

As just indicated UHPFRC will be used to design a load caring façade that contribute to the global stability of the building. Next to this, the general objective of the research is to exploit the design possibilities of UHPFRC in order to derive an innovative and distinctive design. The selected application and research objectives has been translated into the following design principles for the case study:

- The design will be a façade that is load carrying and contributes to the lateral stability of the building
- The load bearing elements of the façade are positioned on the exterior side of the building
- The design of the façade supports the quality of the floorspace within the building
- The façade is designed for a building with a height of at least 70 [m] and maximum 110 [m]
- The façcade contribute to a building energy consumption which complies with the contemporary energy standards

5.2 MATERIAL ASSUMPTIONS

5.2.1 *Material properties*

The material properties of UHPFRC are based upon Ductal C170/200 produced by Lafarge. The material properties of the prestress tendons are based upon a strands of the type Y1860S7.

MATERIAL PROPERTIES UHPFRC	
Characteristic cylinder compressive strength [f_{ck}]	180 MPa
Characteristic cube compressive strength [$f_{ck;cube}$]	200 MPa
Design value compressive strength [f_{cd}]	113 MPa
Characteristic tensile strength [$f_{ctk;0.05}$]	8 MPa
Design value tensile stress first crack [$f_{ctd;1}$]	5,3 MPa
Youngs modulus [E_{cm}]	50 GPa
Density [G]	2750 Kg/m^3
Thermal expansion coefficient [α_C]	0,011 mm/m/°C
MATERIAL PROPERTIES PRESSTRESS STEEL	
Diameter [d]	15.7 mm
Cross sectioned area [S_n]	150 MPa
Youngs modulus [E_p]	195 GPa
Characteristic tensile strength [f_{pk}]	1860 MPa
Design tensile strength [f_{pk}]	1691 MPa
Average tensile strength at midspan [$P_{avg,mid}$]	1520 MPa

Figure 12: Columns made with the considered UHPFRC, picture by Hurks delphi engineering

5.3 ADDITIONAL ASSUMPTIONS

The design study concerning the design of a fictitious building. Additional assumptions concerning locations and functionality are taken into account:

- The building is assumed to be a office building with retail space at the first two floors.
- The building is assumed to be located in a rural area in the Netherlands.
- The load on the building is assumed to be as described in the Eurocode, complemented with the accompanying Dutch Annex.
- For structural calculations concerning UHPFRC the AFGC-SETRA recommendations are assumed to be valid.

6 DESIGN APPROACH

Based upon the design principles a draft design of a building has been derived. The design approach will be presented in this chapter.

6.1 BUILDING CONFIGURATION

After the drafting of several preliminary designs a configuration of a diagrid structure was considered to be the most appropriate system to match the set design principles.

6.1.1 *Diagrid structure*

Due to the triangulated configuration a diagrid structure is structural supportive for vertical gravity load as well for lateral load. In Figure 13 a distinctive building with a clearly visible diagrid structure is presented. Besides this the additional advantages are versatile. According Boake advantages of a diagrid structure are [3]:

- Increased stability due to triangulation
- Combination of gravity and lateral load-bearing systems, potentially providing more efficiency
- Provision of alternate load path (redundancy) in the event of a structural failure
- Reduced use of structural materials translating into environmental savings
- Reduced weight of the superstructure can translate into a reduced load on the foundations
- Ability to provide structural support for a myriad of shapes

6.1.2 *Configuration parameters*

With the configuration of a diagrid system there are several parameters which determine the eventual configuration of the structure. The determining parameters are introduced.

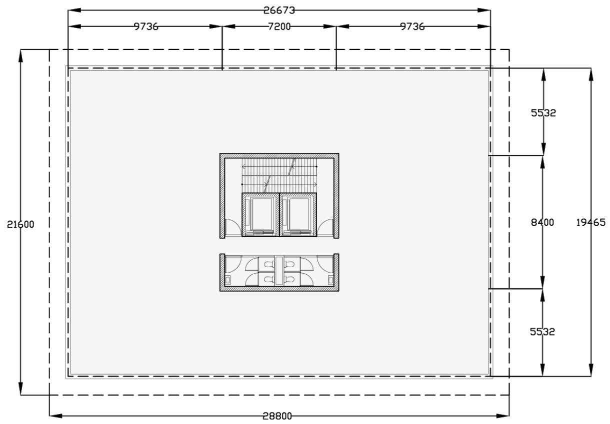
The angle of diagonals has got a decisive influence on the design of an effective diagrid system. Choosing an angle near 90° will results in a structure with too steep diagonals which will hardly support lateral forces. But choosing an angle near 45° results in too flat diagonals which develop high horizontal forces in order to handle the gravity load. Several studies had been carried out to determine the optimal angle for a diagrid structure. Often the optimal angle is coupled with a building height. This is for instance done by Moon who states that the optimal range of diagrids angle for 60 storey buildings is about 65° to 75°. This angle decreases along with the hight of a building as an optimum between about 55° to 65° is found for a 42 storey building [27]. In general an optimal angle will be approximately between 60° to 70° for tall diagrid structures with an aspect ratios ranging from about four to nine [29].

According the Dutch building code the clear spacing between ceiling and finished floor should be at least 2600 [mm] for office buildings [34]. Combined with a conventional floor thickness and floor- and ceiling finish thickness a floor to floor height of 3300 [mm] is derived.

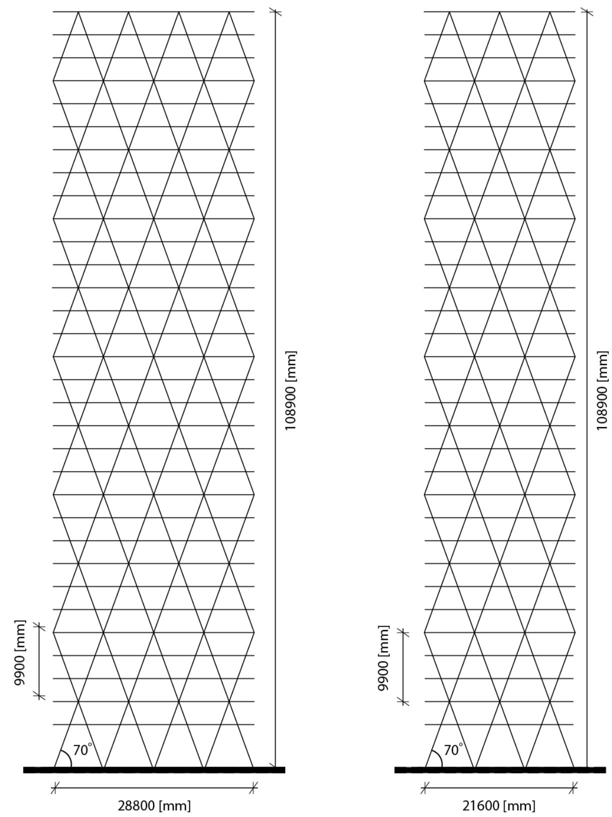
Figure 13: Hearts Tower in New York by Foster + Partners, [51]

6.1.3 *Floorplan configuration and façade grid*

By taking into account the various parameters elaborated above a floor configuration and a façade grid has been designed. The floor configuration and the façade grid are visualised in Figure 14.



(a) floor configuration



(b) Façade grid

Figure 14: Building grid design

6.2 GLOBAL LOAD DISTRIBUTION

With the configuration of the building a global load distribution could be determined in order to determine the normative loads and displacements. These calculations are done with the help of a computational model made with the software program Scia Engineer. The characteristic results of this model are presented in this section. The model, the applied load cases, the results and verifications with hand calculations are presented in Appendix E.

6.2.1 *Normative loads*

The normative loads at the ground floor elements of the façade have been calculated with the engineer model and are presented in Figure 15.

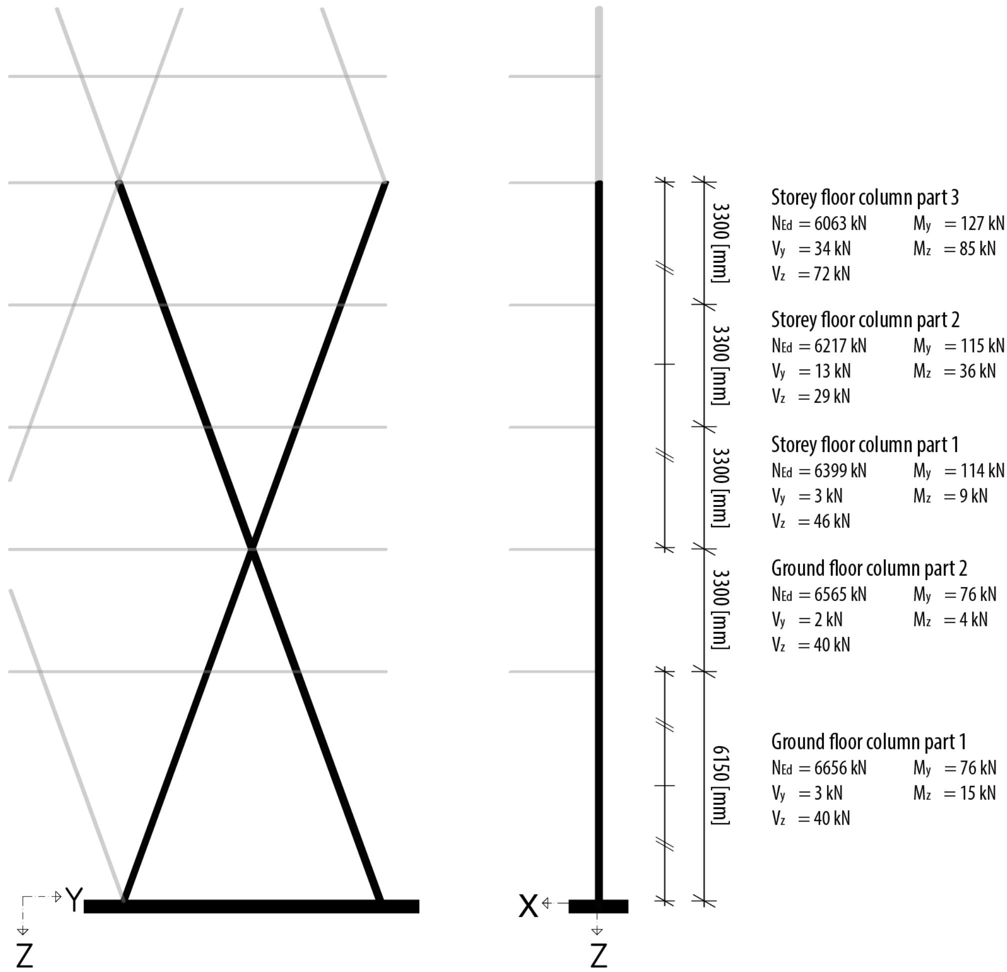
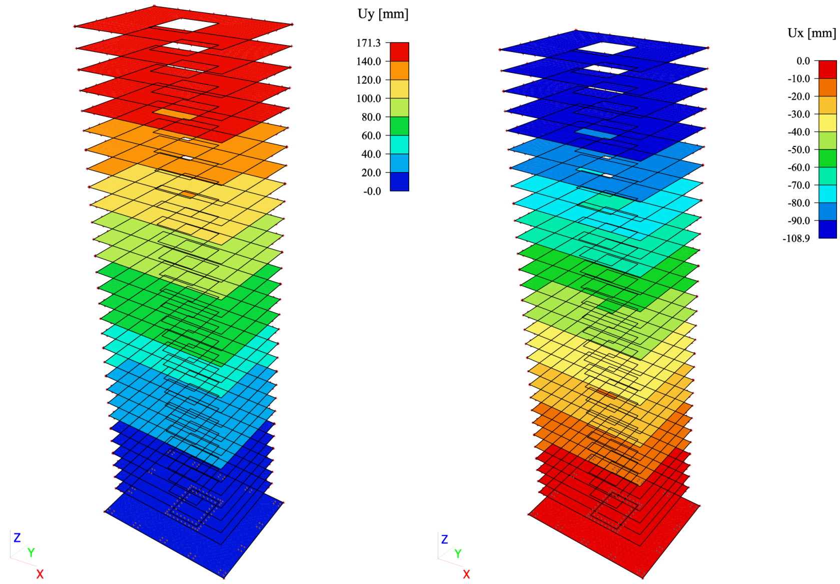


Figure 15: Maximum forces ground floor columns

6.2.2 *Displacements*

The maximum displacements of the structure for SLS load combinations are determined. The maximum horizontal displacement occurs in y-direction. A maximum displacement of 171 [mm] can occur. This displacement is within the Eurocode SLS limit of (height/500) 218 [mm]. The horizontal displacement of the building in y- and in x-direction is visualised in Figure 16.



(a) y-direction (b) x-direction

Figure 16: Displacement

6.2.2.1 *Buckling*

The normative failure mechanism of slender elements can be buckling. When buckling is the normative failure mechanism it is governing for the size of elements. One of the characteristic properties of UHPFRC is the high compression capacity. In order to fully make use of this property buckling should be avoided. By connecting the diagrid elements at every floor the buckling length of the elements stays limited to the floor-to-floor length. However this length is only valid in case the connection with the floor elements can be considered to be of, relatively, high enough stiffness. For buckling out plane of the façade the connection with the floors is considered to be stiff and a buckling length equal to the floor-to-floor length is considered to be realistic. However for buckling in plane of the façade the connection is not considered to be stiff therefor the buckling length for buckling in plane of the façade is assumed to be the floor-to-floor length multiplied by a factor of one and a half.

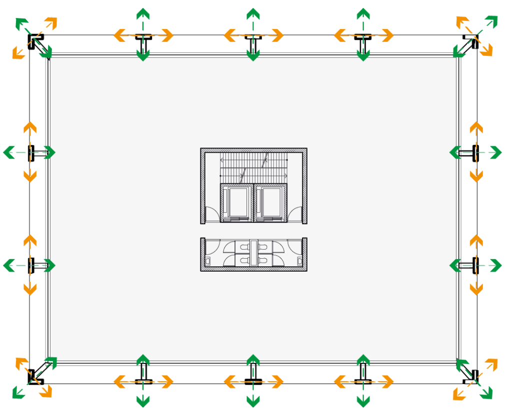


Figure 17: Connection with floor is considered to be of - high stiffness in direction out plane of the facade (green direction), - not high stiffnes in direction in plane of the facade (orange direction)

6.3 DESIGN OF STRUCTURAL MEMBERS

Diagrid structures are often made with hot rolled steel elements. Due to there high strength, slenderness and ductility they are suitable for this applications. However because of the production process hot rolled steel elements are not produced in a great variety of shapes. This results in diagrid elements which are always either rectangle or circular. Due to the great formability of UH-PFRC a great freedom in design of elements is enabled. Elements are not limited to rectangle or circular shapes but instead a whole lot of considerations can be influence the cross-section design of the diagrid elements. The considerations can roughly be classified into two aspects namely; architectural demands and structural demands. The following aspects can determine the cross-section design:

- Architectural demands
  - user experience
  - aesthetics
  - climate design
  - urban/location integration
- Structural requirements
  - strength
  - stiffness
  - buckling
  - production process
  - construction process

In the case study all structural requirements have influenced the cross-section design of the diagrid elements. From the architectural demands the user experience has influenced the design. Nonetheless the other demands can be used as well to design.

6.3.1 *Characteristic points*

The following approach is used to design of the diagid elements: Characteristic points along the length of the members are selected after which cross-sections, for these characteristic points, are shaped in an architectural and structural optimal figure. Along the length of the diagrid elements six characteristic points are selected. To derive an architectural and structural optimal figure at the characteristic points the structural requirement and architectural demands are further specified. The location of the selected points are given in Figure 18. The specified requirement and demands are presented here below.

- *P1.* Enable as much exposure as possible for the retail space located at the ground floor level of the building.
- *P2.* Enable a slender appearance which is adapted to the sensitivity for buckling of this point.

- *P3*. Enable a smooth transition to point P4 and resist the high imposed moments.
- *P4*. Enable a slender appearance which is adapted to the sensitivity for buckling of this point.
- *P5*. Enable a panoramic view from within the building and resist the stress concentration due the merging of members
- *P6*. Enable a panoramic view from within the building and resist the stress concentration due the merging of members out different planes.

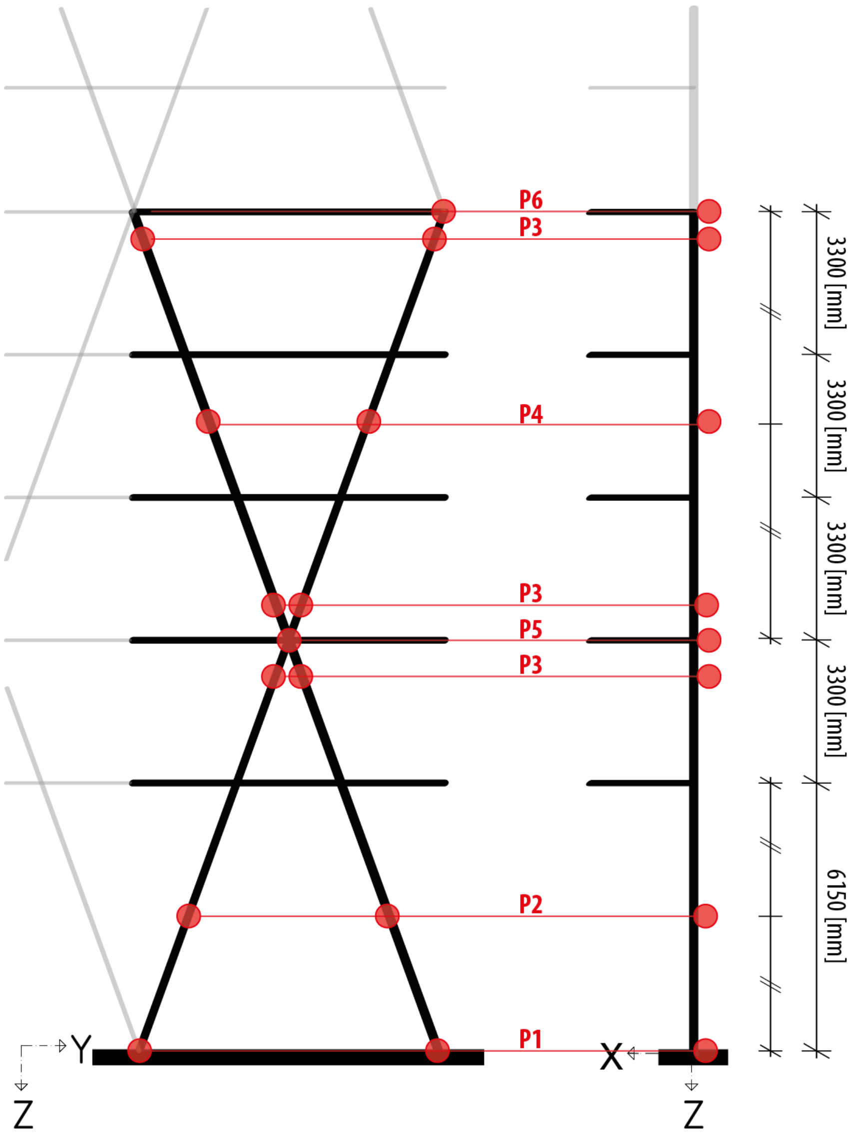


Figure 18: Characteristic points

6.3.1.1 *Design of the sections*

Based upon the structural requirements and architectural demands rectangle cross-sections have been designed. As changing the size of the cross-section affects the stress distribution and the second order effect this has been an iterative process. The second order effect and stress distribution of the eventual derived cross-sections are presented in Appendix F. After a rectangle cross-section design was derived which satisfied the set requirements the rectangle have been transformed into an organic shaped section with the exact same section properties. The rectangle and subsequent organic shaped sections are presented in Figure 19.

6.3.1.2 *Tensile stress*

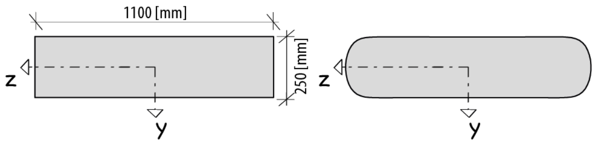
For the design of the cross-sections at the characteristic points the normative load for the elements at the bottom of the building are considered. From the structural model presented in Appendix E it can be concluded that the loads at these points are normative compression loads, besides it can be concluded that both the compression forces and moment forces will decreases towards the top of the building and that tension forces will be introduced at the top of the building. From the calculations of Appendix F it follows that tensile stress can only occur in section P2 and P3. Considered the previous tensile stress in the entire elements is expected at the top of the building.

Similar as it is for ordinary concrete the tensile capacity of UHPFRC is small compared with the compression capacity. The considered UHPFRC has a design tensile capacity of ($f_{ctd;1}$ =) $5,3$ $[MPa]$. When the tensile stress exceeds this design value the following two measures can be applied:

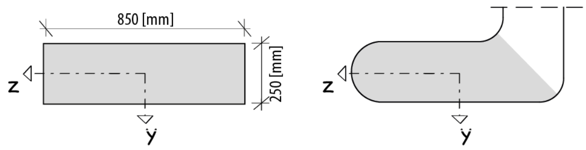
APPLY REINFORCEMENT Reinforcement can take over the tensile force after the concrete cracks. This is a relatively cheap solution, unfortunately this solution leads to cracks which reduces the stiffness of the concrete.

APPLY PRESTRESS The prestress will ensure that the concrete remains under compression. This solution is more expensive than the alternative but it is as well more effective as tensile stress, cracks and the subsequent stiffness reduction are prevented.

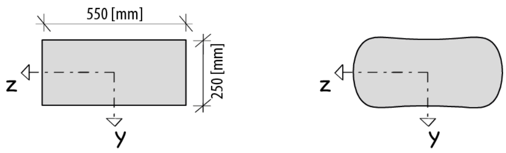
In order to prevent cracks prestress is applied. Both prestress without- and with-bond can be applied. In the case study design prestress with bond is applied. This is preferred over prestress without bond as this is in general a safer and more robust solution.



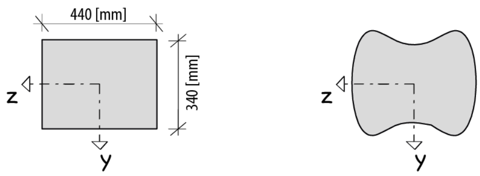
(a) Section at P5



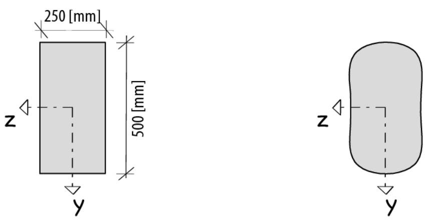
(b) Section at P6



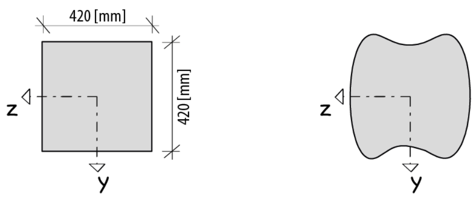
(c) Section at P3



(d) Section at P4



(e) Section at P1



(f) Section at P2

Figure 19: Designed sections at characteristic points, rectangle section [left], organic shaped sections [right]

7 BUILDING DESIGN

In continuation of the draft design the building has been further elaborated. The derived design is presented in this chapter.

7.1 FAÇADE AND FLOOR STRUCTURE

With the floor-, façade configuration and the cross-section derived in the previous chapter the following façade view and floor plans have been drafted. See: Figure 20 and Figure 21.

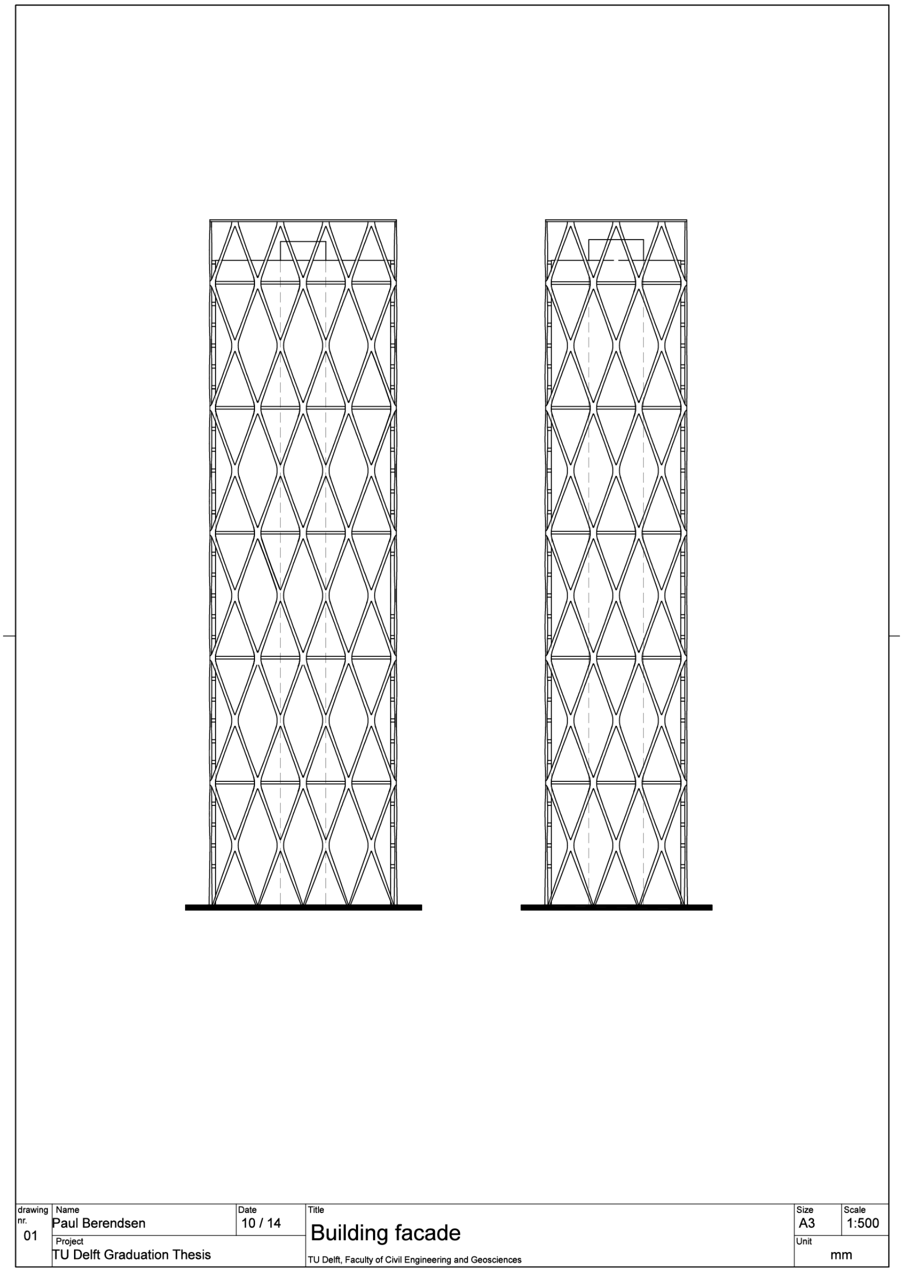
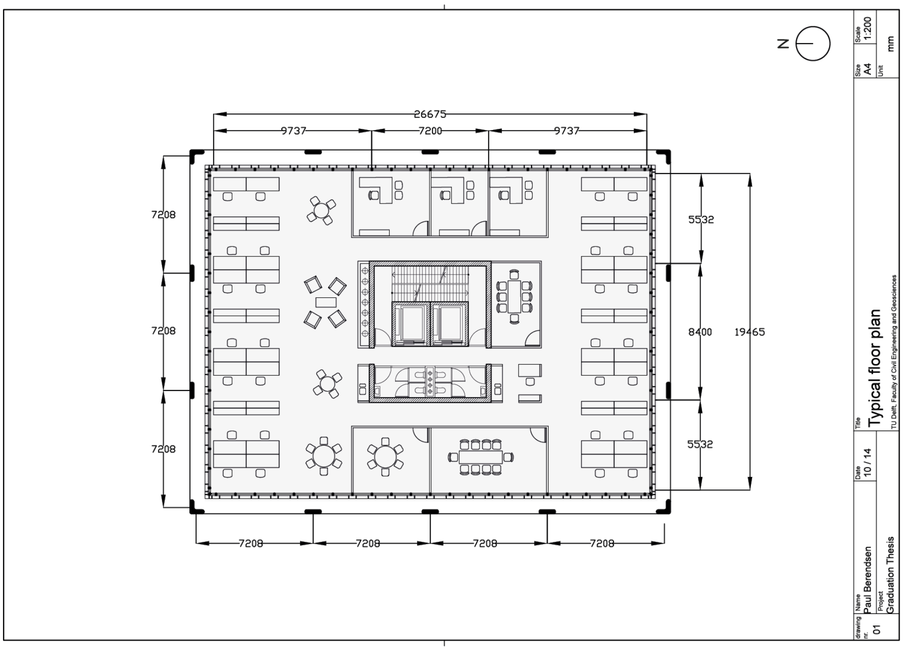
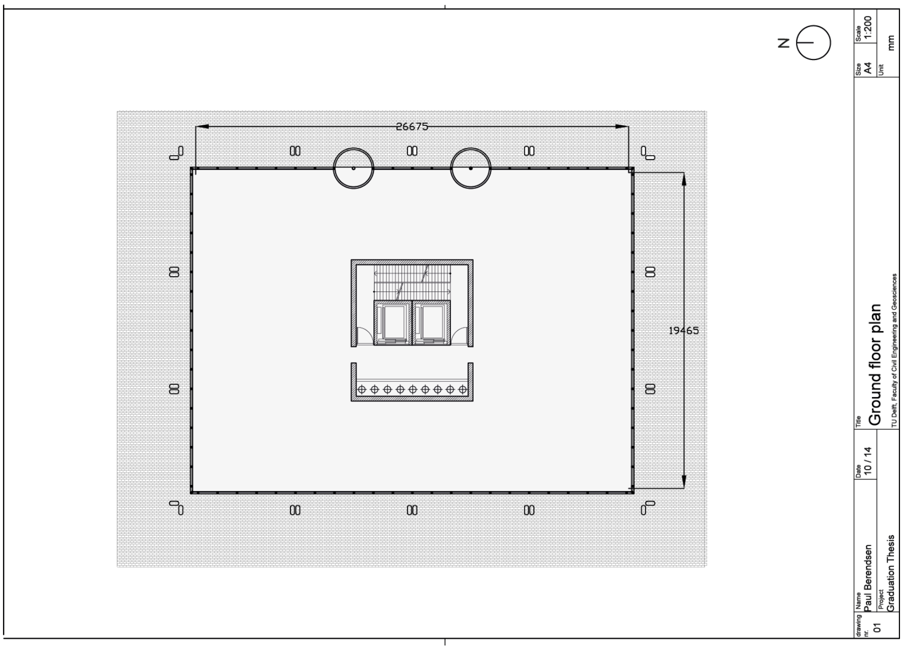


Figure 20: Façade design



(a) Typical floor plan



(b) Ground floor plan

Figure 21: Floor plans, see Appendix I for full size drawing

7.2 MODIFIED STRUCTURAL MODEL

After the building design was derived the structural model of section 6.2 has been modified in order to comply with final design. The modifications of the model and the resulting normative loads are presented in this section.

7.2.1 *Building design*

The top two floors have been removed in order to created a roof terrace with view upon the structural façade elements. This modification resulted in a significant load reduction for the ground floor elements.

7.2.2 *Tensile element*

In order to act like a prestressed beams the façade tensile members are modelled with a high stiffness. The elements in the model resemble fully prestressed concrete element with a cross-sectional area of $(350 \cdot 250 =)\ 8.75 \cdot 10^4\ [mm^2]$.

7.2.3 *Floor thickness*

The floor thickness of the 300 [mm] was considered to be too thin therefor the floor thickness is increased up to 400 [mm], this increased the self weight of the building. However due to the weight reduction from the removal of the top floors the maximum load at the ground floor was reduced.

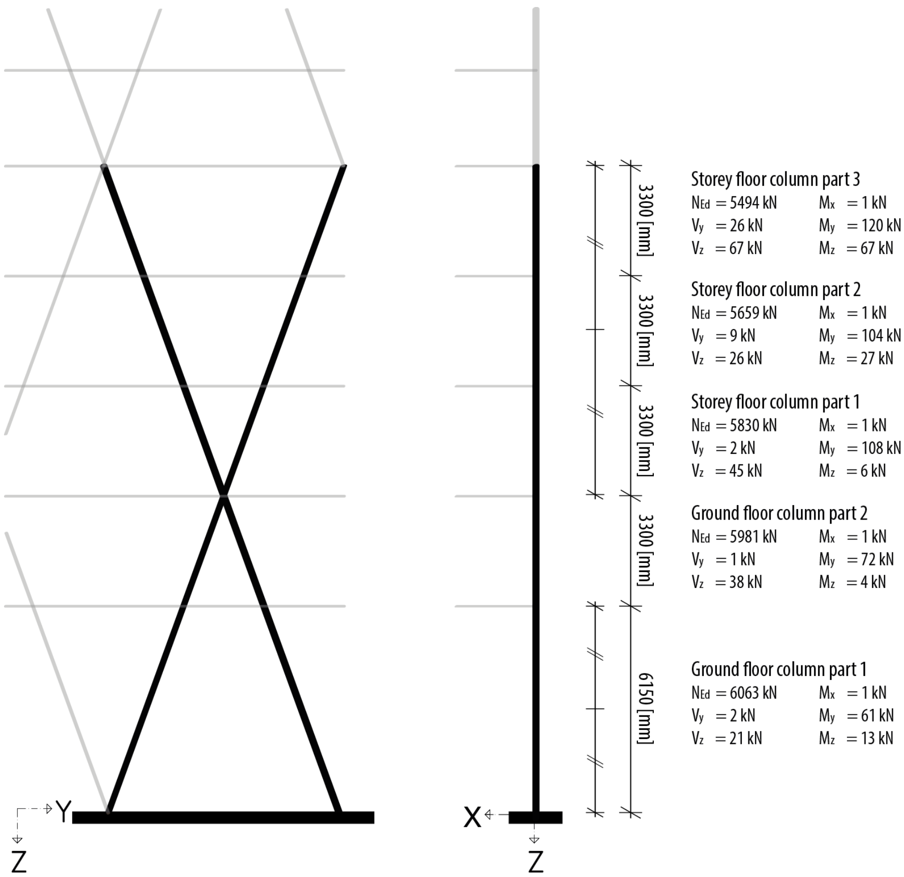


Figure 22: Maximum forces ground floor columns revised model

7.3 ultimate limit state

As stated in subsubsection 6.3.1.2 prestress has to be applied to prevent cracking in the outer fibres of the UHPFRC elements. Prestress leads to a significant higher tensile capacity and thus prevents cracks, however it also leads to a higher compression stress in the cross-section. For the characteristic points P1, P2, P3 and P4 the ULS load cases which result in the maximum and minimum stress are identified. With these stress limits the required amount of prestress force is determined. The calculations can be found in Appendix G. It is assumed that the prestress force is distributed equal over the entire cross-section. The maximum compression stress is allowed to be equal with the design value of the compressive capacity of $f_{cd} = -113$ [MPa] and the maximum tensile stress is allowed to be equal with the design value of the tensile capacity which is $f_{ctd;1} = +5,3$ [MPa].

7.3.1 *Characteristic point: P1*

The load case resulting in the maximum compression stress in section P1 is load cases 13 (Wind load1 plus thermal load south, west façade). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-86.4 \quad [MPa] \quad \& \quad -10.7 \quad [MPa] \tag{1}$$

The load case resulting in the minimum compression stress in section P1 is load cases 9 (Minimum floor load plus wind load2). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-27.2 \quad [MPa] \quad \& \quad -0.8 \quad [MPa] \tag{2}$$

With these upper- and lower- stress limit the difference with the design value of the material capacity can be determined. With this stress difference the required amount of prestress force is determined by a multiplication with the cross-sectional area. The upper- and lower- limit are then given by Equation 3. As tensile stress does not occur at this characteristic point the lower limit is zero.

$$0 \leqslant P_{p1} \leqslant 3337.5 \quad [kN] \tag{3}$$

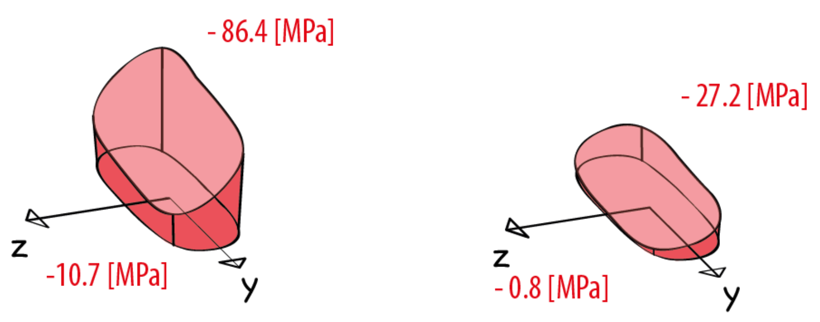


Figure 23: Stress distribution and maximum and minimum stress at the cross-section at P1 for, load case 13 [left], load case 9 [right]

7.3.2 *Characteristic point: P2*

The load case resulting in the maximum compression stress in section P2 is load cases 13 (Wind load1 plus thermal load south, west façade). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-91.8 \quad [MPa] \quad \& \quad +23 \quad [MPa] \tag{4}$$

The load case resulting in the minimum compression stress in section P1 is load cases 9 (Minimum floor load plus wind load2). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-30.5 \quad [MPa] \quad \& \quad +10.7 \quad [MPa] \tag{5}$$

With these upper- and lower- stress limit the difference with the design value of the material capacity can be determined. With this stress difference the required amount of prestress force is determined by a multiplication with the cross-sectional area. The upper- and lower- limit are then given by Equation 6.

$$3122.3 \leqslant P_{p2} \leqslant 3739.7 \quad [kN] \tag{6}$$

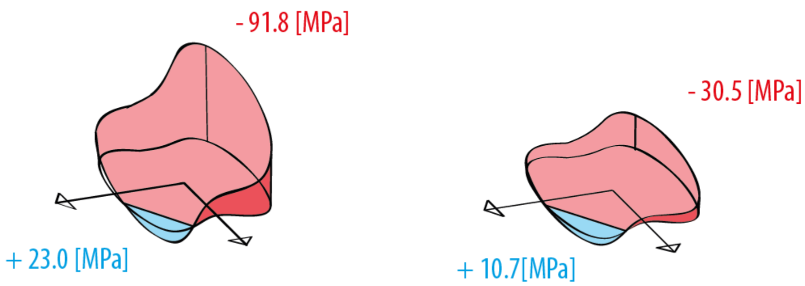


Figure 24: Stress distribution and maximum and minimum stress at the cross-section at P2 for, load case 13 [left], load case 9 [right]

7.3.3 *Characteristic point P3*

The load case resulting in the maximum compression stress in section P3 is load cases 13 (Wind load1 plus thermal load south, west façade). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-77.9 \quad [MPa] \quad \& \quad -6.9 \quad [MPa] \tag{7}$$

The load case resulting in the minimum compression stress in section P1 is load cases 9 (Minimum floor load plus wind load2). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-24.2 \quad [MPa] \quad \& \quad +0.2 \quad [MPa] \tag{8}$$

With these upper- and lower- stress limit the difference with the design value of the material capacity can be determined. With this stress difference the required amount of prestress force is determined by a multiplication with the cross-sectional area. The upper- and lower- limit are then given by Equation 9.

$$0 \leqslant P_{p3} \leqslant 4826.3 \quad [kN] \tag{9}$$

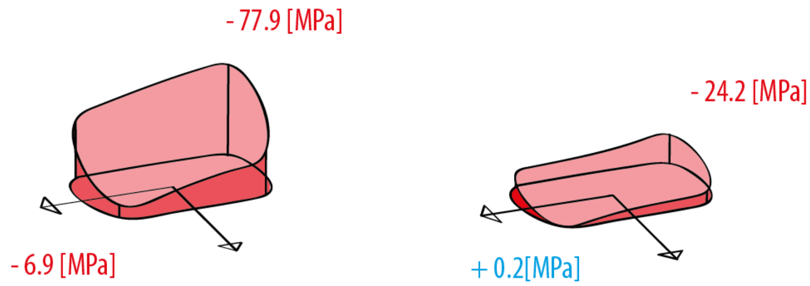


Figure 25: Stress distribution and maximum and minimum stress at the cross-section at P3 for, load case 13 [left], load case 9 [right]

7.3.4 *Characteristic point P4*

The load case resulting in the maximum compression stress in section P3 is load cases 13 (Wind load1 plus thermal load south, west façade). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-95.3 \quad [MPa] \quad \& \quad +19.7 \quad [MPa] \tag{10}$$

The load case resulting in the minimum compression stress in section P1 is load cases 11 (Minimum floor load plus thermal load south, west façade). This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$-30.5 \quad [MPa] \quad \& \quad +9.7 \quad [MPa] \tag{11}$$

With these upper- and lower- stress limit the difference with the design value of the material capacity can be determined. With this stress difference the required amount of prestress force is determined by a multiplication with the cross-sectional area. The upper- and lower- limit are then given by Equation 12.

$$2154.2 \leqslant P_{p4} \leqslant 2647.9 \quad [kN] \tag{12}$$

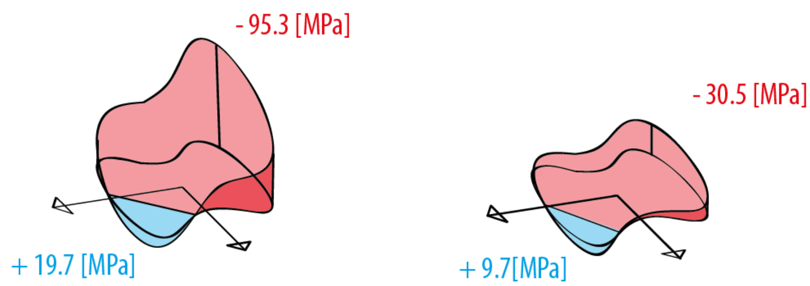


Figure 26: Stress distribution and maximum and minimum stress at the cross-section at P4 for, load case 13 [left], load case 11 [right]

7.3.5 *Maximum tension*

From the previous calculations if follows that the highest tensile stress can occur in characteristic section P4. In subsubsection 6.3.1.2 there was reasoned that high tensile forces can be expected at the top of the building. It follows that the high tensile stress can be expected at the middle of an element near the top of the building. To determine the maximum required amount of prestress the normative normal tensile force has been identified. This maximum normal tensile stress was found in case of load cases 11 (Minimum floor load plus thermal load south, west façade) in a element near the top of the building. This load case results a stress distribution with a upper- and lower- stress limit of respectively:

$$+1.8 \quad [MPa] \quad \& \quad +14.4 \quad [MPa] \tag{13}$$

With these upper- and lower- stress limit the difference with the design value of the material capacity can be determined. With this stress difference the required amount of prestress force is determined by a multiplication with the cross-sectional area. The upper- and lower- limit are then given by Equation 14.

$$1361.4 \leqslant P_{P.top} \leqslant N/A \quad [kN] \tag{14}$$

7.3.6 *ULS conclusion*

From the previous sections it is concluded that prestress is required at the characteristic points P2 and P4. While the normal tension force increases near the top of the building the highest tensile stress is found at the bottom of the building. This high tensile stress at the bottom is reached due to the significant impact of second order effects on slender elements, eventually leading to high second order moment forces.

With the calculated prestress force limits the required amount of prestress strands can be determined:

By applying 14 prestress strands at section P2 a prestress force of 3192 [kN] is imposed, as calculated with Equation 15. This prestress force is within the range to satisfy the ULS requirements at this section.

By applying 12 prestress strands at section P4 a prestress force of 2736 [kN] is imposed, as calculated with Equation 16. This prestress force is within the range to satisfy the ULS requirements at this section.

$$14 \cdot S_n \cdot P_{avg,mid} = 3192 \quad [kN] \tag{15}$$

$$12 \cdot S_n \cdot P_{avg,mid} = 2736 \quad [kN] \tag{16}$$

With the loads acting at section P4 being the normative load combination the required amount of prestress determined for P4 will satisfy for all floor columns. However this amount is not necessary for all storey floor elements. As the imposed normal force and moment forces decreases near the top of the building the required prestress force will likewise decrease.

7.4 service limit state

By positioning the load carrying members of the facade on the outside of the thermal isolation perimeter while keeping the load carrying core of the building inside the perimeter a thermal difference is introduced. A thermal difference will lead to differences in expansion of structural members. As a uniform thermal expansion over the height of a building will not cause difference in displacements and thus also will not introduce internal forces, this is likely to happen when a thermal difference is introduced. With the building fixed at a foundation the expansion difference will progress along the height of the building with the biggest difference in expansion to be found at the top floor of the building. The described effect is visualised in Figure 27 for a part of a section of the building.

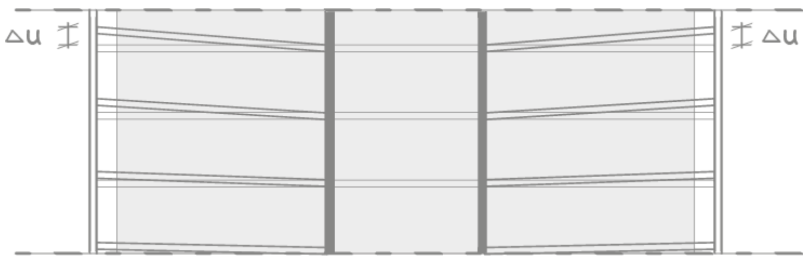


Figure 27: Different expansion of structural components

Due to the different expansion the floor beams will, in a 2D plane, rotate. By providing enough rotation capacity in the connections the difference should not cause ultimate limits state problems. However for service limit state requirements a too big rotation can cause problems. This for instance for interior elements like non-supportive partition walls. The Eurocode does not specify requirements concerning expansion differences between structural components, nor it specifies requirements concerning a maximum rotation of deflection of floor member. With no requirements specified in the Eurocode, a requirement specified in the former Dutch building code has been used which does specify a requirement concerning the maximum deflection of a floor beam. By translating the maximum allowed deflection into a maximum allowed angle rotation a requirement for the maximum allowed expansion difference has been composed to verify the SLS quality of the case study design.

7.4.1 *Maximum allowed expansion difference*

The maximum allowed deflection for a two side supported beam is given by Equation 17 [54]. The tangent of the deflection line at the supports can be determined by multiplying the deflection u_{max} at midspan. For a beam which is loaded by a even distributed load the multiplication value is around one and a half. With the hereby found rotation angle at the support a maximum allowed difference between the two supports can be drafted, see figure Figure 28. The maximum difference can than be determined as function of the beam length and is given by Equation 18

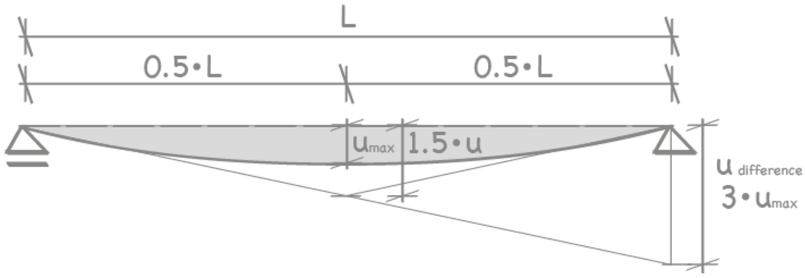


Figure 28: Extrapolation to determine maximum allowed rotation

$$u_{max} = 0.003 \cdot l \quad [mm] \tag{17}$$

$$u_{difference} = 0.009 \cdot l \quad [mm] \tag{18}$$

With the shortest length between the core of the building and the facade structure being 6600 [mm] an expansion difference limit of 59.4 [mm] is determined. Note that this limit may only be reached incase of an infinite stiff floor in order to stay under the ultimate allowed rotation angle.

7.4.2 *Occurring expansion*

With the use of the expansion coefficient of UHPFRC the magnitude of the expansion of a straight column up to the top floor of the building can easily be calculated. This is done in Equation 19, resulting in a maximum allowed difference of 36.6 [mm]. This difference is well within the determined limit. However as a diagrid structure is composed of elements under an angle and not of straight columns up to the top of the building, this calculation is not entirely valid. The exact effect of thermal expansion in a diagrid structure is however hard to predict as this is also depending of the stiffness thermal properties of the various elements.

$$u_{top} = L_{ttf} \cdot \alpha_C \cdot D_{temp} \quad [mm] \tag{19}$$

With:

L_{ttf} :	Length till top floor	=	95	[m]
α_C :	Thermal expansion coefficient	=	0,011	[mm/m/°C]
D_{temp} :	Considered temperature difference	=	35	[°C]

The occurring expansion differences can be extracted from the structural model elaborated in Appendix E. The expansion differences for various thermal load cases are presented in the following table. Hereby only the nodes with the minimal distance of 6600 [mm] to the core are considered.These nodes are considered to be normative. As can be seen the differences are lower than determined with the hand calculation and thus also well within the determined limit.

LOAD CASE	EXPANSION FAÇADE
Q_{t1} Thermal load south façade	22.4 [mm]
Q_{t2} Thermal load south, west façade	41.2 [mm]
Q_{t3} Thermal load all façades	35.1 [mm]

With the considered building height and floor configuration the expansions difference remains well within the self prescribed limit. However the expansion difference does impose a strict requirement upon the stiffness of the floors in order to say under the maximum allowed rotation angle.
The rotation angle which has been used, is determined by the building hight and the core-to-façade length. The height of a building with this configuration is restricted by this rotation angle.

7.5 FAÇADE SECTION

One of the design principles for the case study design concerns the energy consumption of the building. Due to a general drive to lower energy consumption of society, the energy consumption of buildings must decrease. This might eventual lead to energy neutral buildings. With the façcade being of major important for the energy consumption of a building a façcade design should enchant the energy consumption of a building. The energy consumption of cooling systems are often responsible for a major part of the energy consumption of a building. By locating the load carrying structure outside the thermal isolated perimeter of the building heat leaks are introduces. But, with the structure covering roughly 18 % of the façcade the structure will likewise act as a sunshade significantly reducing the cooling demand. In section 6.3 it was already stated that climate design demands can be used to design the shape of the structural element. By doing so in an effective way the structure might make additional cooling methods unnecessary. In this design the climate demands have not influenced the shape of the structural members, though additional research can be conducted to confirm this statement.

In order to fullfill isolating and separating requirements an so called unitised façade has been designed. The unitised façade elements will result in a double skin façade with excellent isolation and natural ventilation properties. Combined with the sun-shading effect of the load carrying structure active cooling is likely to be unnecessary. Nonetheless additional research is required to confirm this statement.

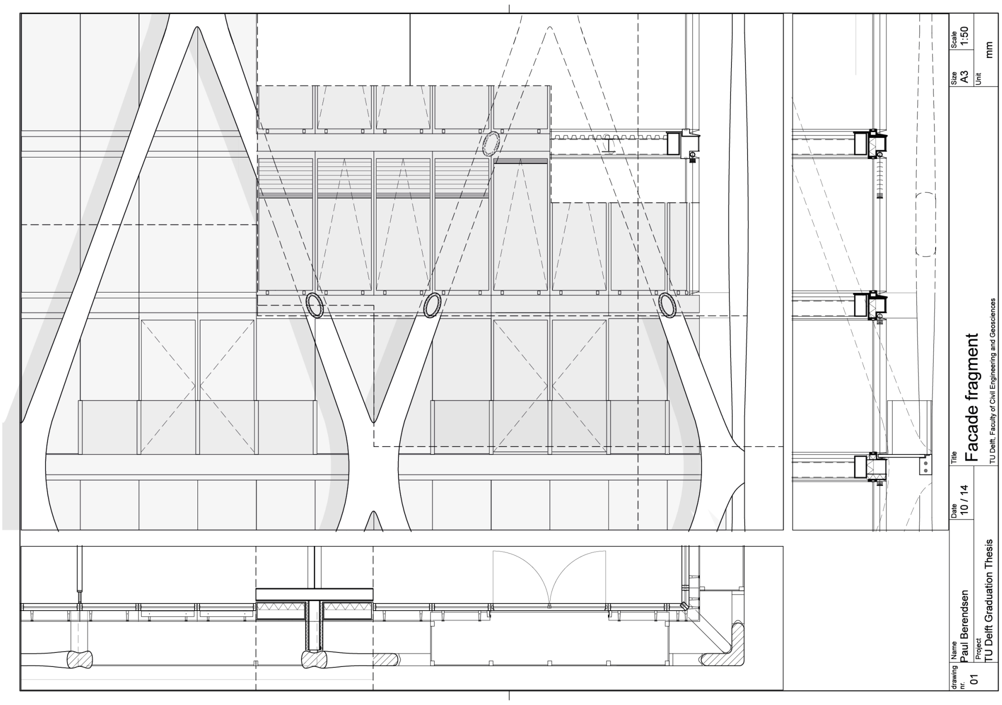


Figure 29: Façade fragment, full size drawing can be found in Appendix I

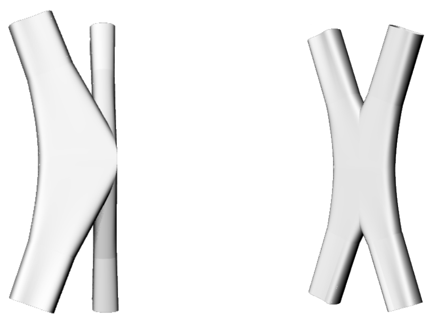
7.6 construction approach

The design approach for construction of the case study building is presented in the this section.

7.6.1 *Prefab elements*

The production requirements for UHPFRC imposes strict requirements upon the fabrication process. These requirements are certainly reached in specialised construction workshops. Construction with prefab elements is therefore considered. Prefabrication limits the possible member sizes due to transportation restrictions. A division whereby the structure is split into four elements different type of construction elements is chosen. The location of division is driven by the moment and shear forces lines, the connections are made where they find their minimum. The construction elements can be classified as members and nodes.

(a) Ground floor member (b) Storey floor member

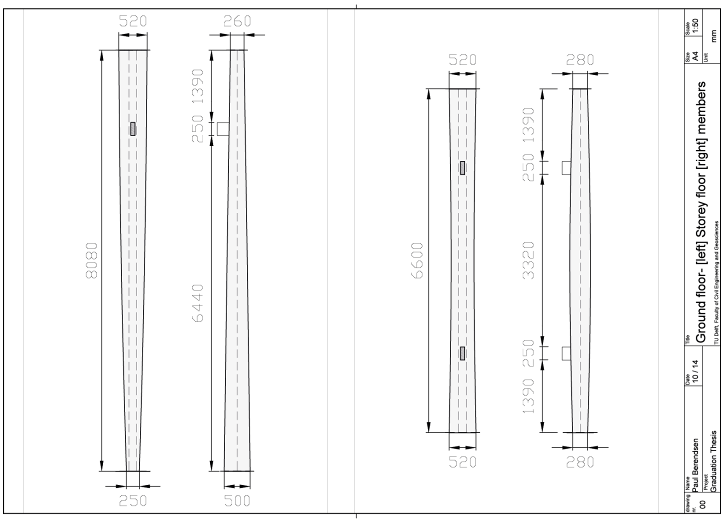


(c) In-plane node (d) Edge node

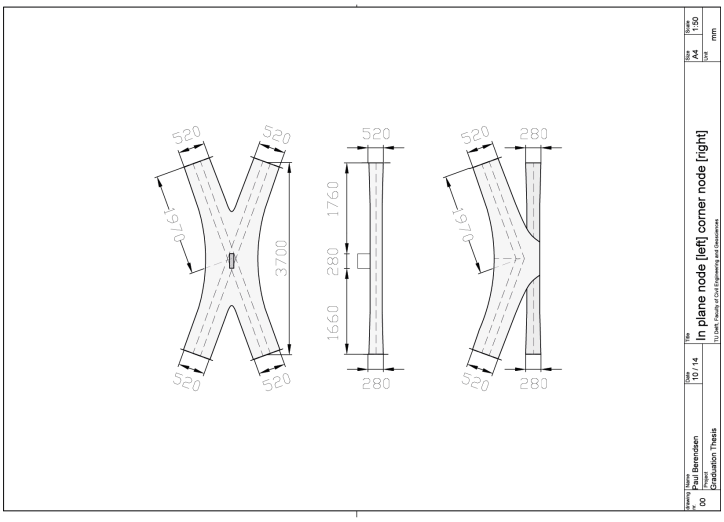
Figure 30: Prefabricated elements

7.6.2 *Prestress strands*

With the ULS calculations of section 7.3 showing tensile stresses above the material resistance, prestress strands to increase the tensile capacity appeared are required. The amount of prestress strands has already been determined in that same section. Prestress will not only increases the tensile capacity of the material, simultaneously it will significantly improve the shear and flexural capacity which all together improves the robustness of the structure. Respectively fourteen and twelve strands have been determined to be required for the ground floor- and storey floor members. This prestress will be provided by pretensioned prestress tendons with bond. Prestressing by this method relatively easy be applied with a construction process similar with hollow core slabs and prestressed beams. By the increases of tensile forces towards the top of the building high tensile stresses can as well occur in the nodes. For the majority of nodes this tensile stress will stay under the tensile capacity of the material. For the nodes where this tensile capacity is exceeded, sufficient conventional reinforcement is applied to provide sufficient tensile capacity.



(a) Ground floor member (left) storey floor member (right)



(b) In-plane node (left) edge node (right)

Figure 31: Technical drawing prefab elements

7.6.3 *Structural floor plans*

The storey floors are made out of a composite steel-concrete floor system which is supported by a steel beam structure. The floor system is suitable for quick erection on site and results in a floor with a low self weight. The technical floor plans of the steel beam structure is presented in Figure 32.

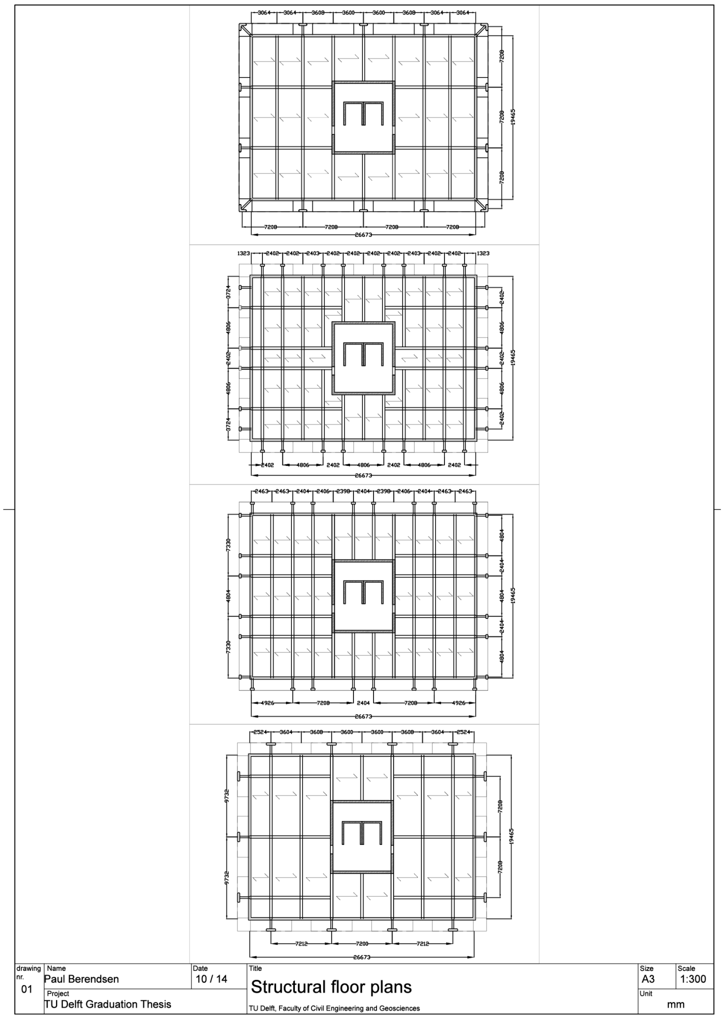
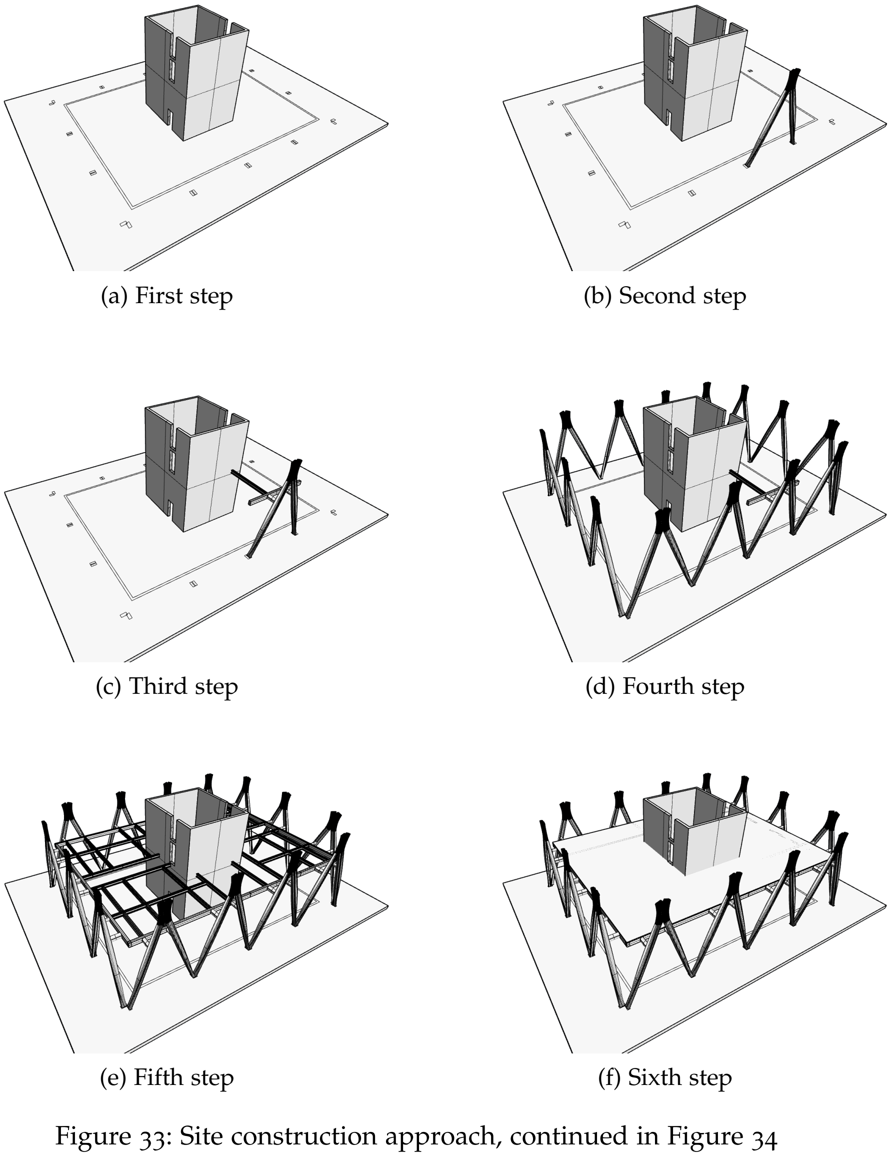


Figure 32: Technical floor plans steel structure, full size drawing can be found in Appendix I

7.6.4 *Erection on site*

The following steps have to be carried out for the erection on site, see Figure 33. The building erection begins with the construction of the structural stability core which is build up for the first few floors (first step). Thereafter the triangle diagrid elements can be hoisted in position and installed (second step). Next a connection with the stability core is established through the installation of floor beam, these prefabricated floor beam elements can hoisted in position (third step). After the diagrid triangles (fourth step) and floor elements (fifth step) for the first level are made the floor can be finished with a composite steel sheet floor (sixth step).



(a) First step

(b) Second step

(c) Third step

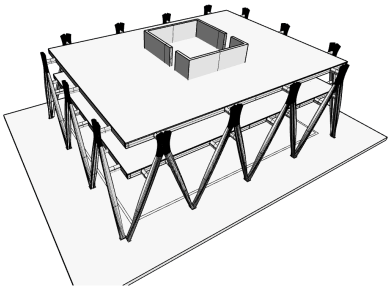
(d) Fourth step

(e) Fifth step

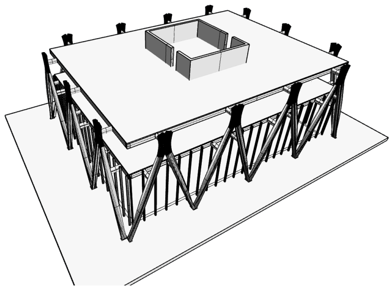
(f) Sixth step

Figure 33: Site construction approach, continued in Figure 34

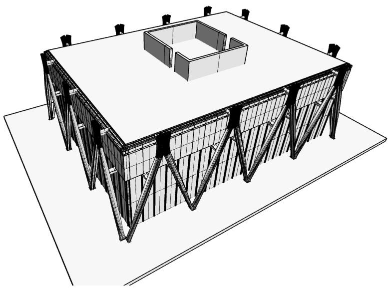
The fifth and sixth step are repeated for the next floor level (seventh step). The ground floor glass curtain can be installed at the ground floor level (eighth step). And the unitised façade elements can be hoisted in position and installed (ninth step). Subsequently a similar process is conducted for the next levels (tenth step) and continued until the building height is derived.



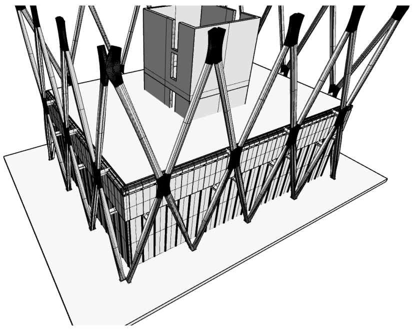
(a) Seventh step



(b) Eighth step



(c) Ninth step



(d) Tenth step

Figure 34: Site construction approach

7.7 DETAILING

The characteristic details of the case study building have been elaborated. The normative load cases have been considered for the design. Furthermore the building configuration, production process and erection approach imposed several requirements upon the design of the construction details.
The details are presented in this section. Structural calculations and considered loads are presented in Appendix H. Steel of a strength class S355 is considered. Bolts of a strength class 8.8 are considered if not otherwise indicated. Regarding bolt holes for bolts up to a diameter of 24 [mm] gap tolerances of 2 [mm] are considered, for bolts bigger a tolerance of 3 [mm].

7.7.1 *Detail1*

The connection of the UHPFRC diagrid members with the floor beam connection has been elaborated. The derived design is presented in Figure 35.

For architectural reasoning an elliptical section is used to connect the concrete diagrid with the steel floor system. At the end of the elliptical section an endplate is welded that makes a hinged connection with a thick steel plate that is casted into the concrete diagrid element. The connections is adjustable in high by means of levelling plates. Eccentricities are kept small, nonetheless an eccentricity of 230 [mm] is introduced which results in a moment of 55.2 [kNm]. The introduced moment is significantly lower than the moment forces introduced by second order effects, complications are therefor not immediately expected. The structural model of Appendix E can however be improved by the addition of this eccentricity moment forces.

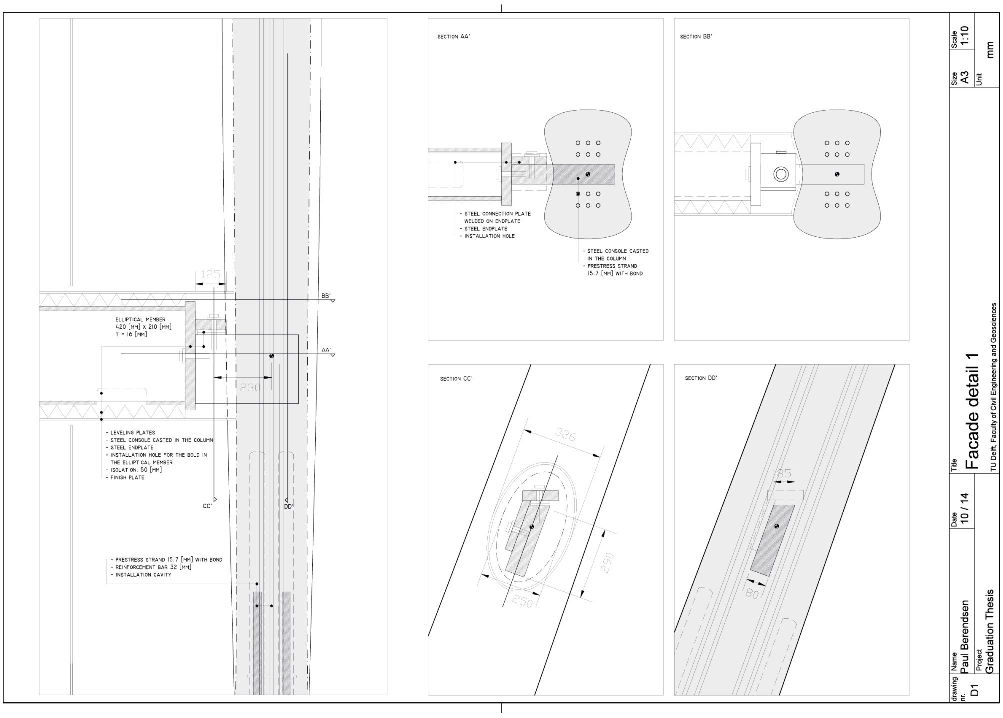


Figure 35: Detail 1, full size drawing can be found in Appendix I

7.7.2 *Detail2*

The connection of the UHPFRC diagrid nodes (in plane of the façade) with the floor beam connection has been elaborated. The derived design is presented in Figure 36.

The design of detail 2 is similar with the design of detail 1. Major difference with detail 1 is the magnitude of forces acting on the elliptical section. At the node elements these forces are significantly higher than the are at the members. A thicker elliptical section is therefor applied. As well the steel plate that is casted into the concrete element is bigger compared with the plated of detail 1.

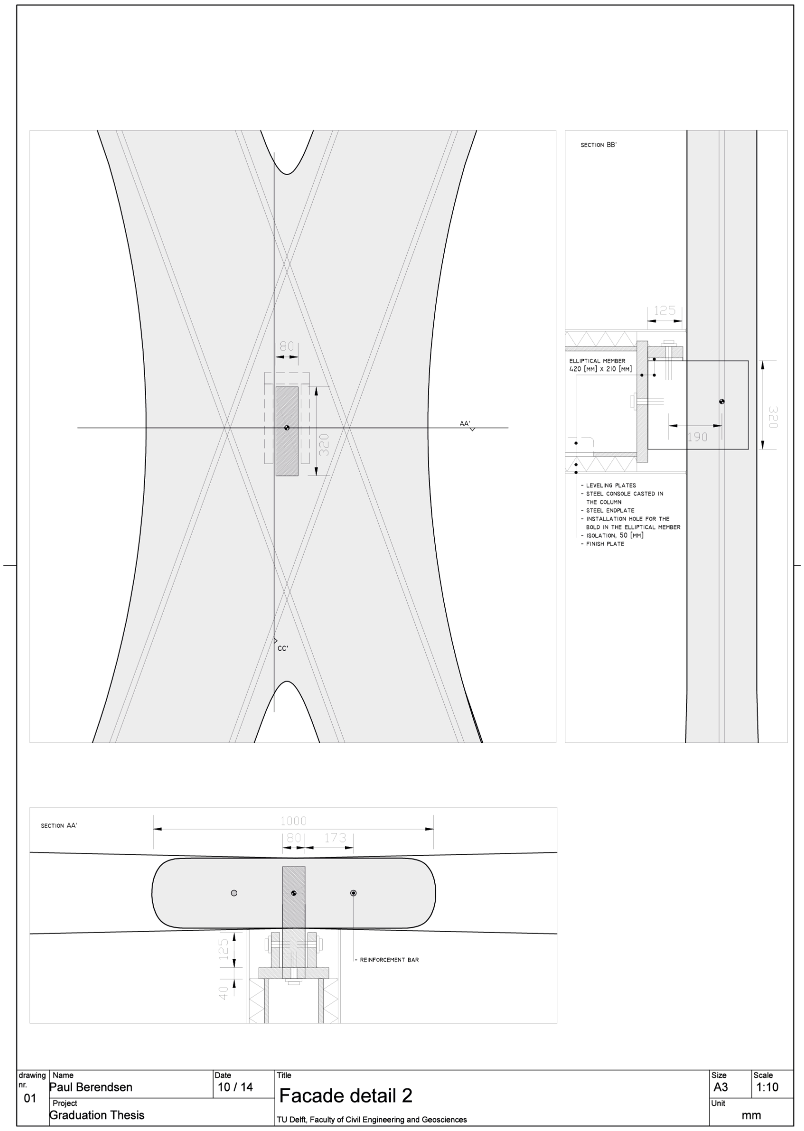


Figure 36: Detail 2, full size drawing can be found in Appendix I

7.7.3 *Detail3*

The connection of the UHPFRC diagrid node at the façade corner is designed. Several members merge at this location, including the horizontal members with transfer the outward force of the diagrid. The derived design is presented in Figure 37.

The horizontal tensile elements are connected by a thick steel plate in the concrete diagrid node element. This steel structure encloses the concrete in order to derive an isotropic pressure zone. Furthermore the connections with the various elements are kept as simple as possible for easy erection on site.

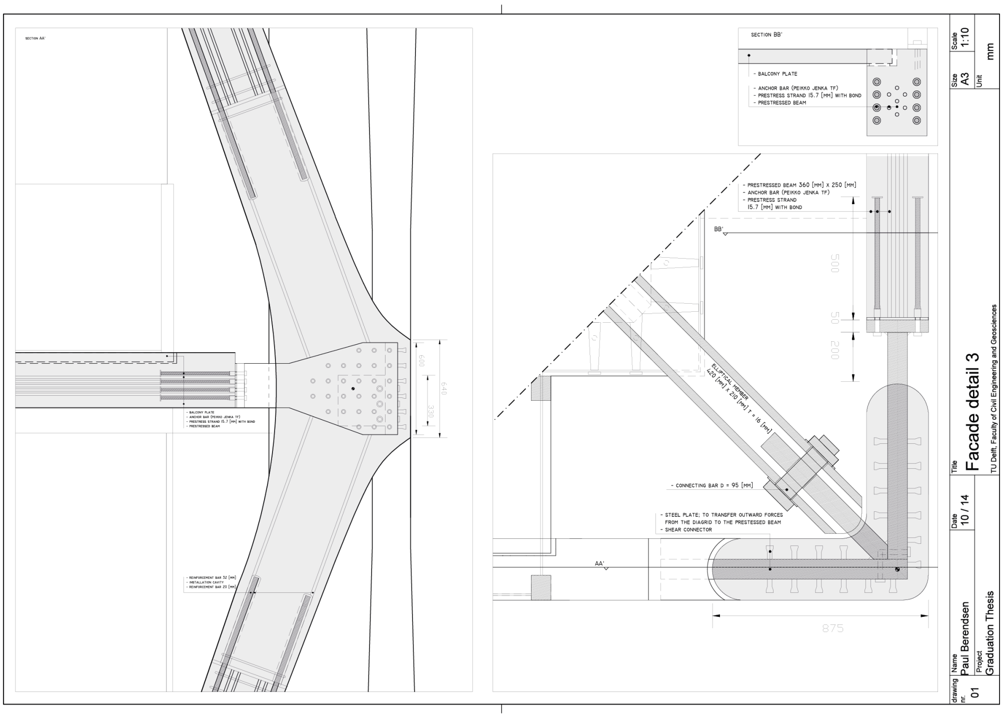


Figure 37: Detail 3, full size drawing can be found in Appendix I

7.7.4 *Detail4*

The connection of the prefab elements has been elaborated. The connection below the nodes is considered The derived design finds its origin in an conventional prefab concrete connection. The design is presented in Figure 38.

The connection between the two prefab elements is made with steel bars. These bars are casted into the diagrid member elements. By use of mortar the connection with the diagrid node elements can be made.

7.7.5 *Detail5*

The connection of the prefab elements has been elaborated. The connection above the nodes is considered The design is presented in Figure 38. The design of the detail is similar with detail 4. However the considered construction approached imposes additional requirements upon the design. For the erection on site the elements must be hoisted upon each other vertically. The reinforcement bars protrude out the elements under an angle, this will conflict with vertical hoisting installation. Therefor the bars are, for the erection, positioned in the diagrid member elements. After the element is hoisted at its location the bars can drop into the diagrid node elements whereafter a connection with mortar can be made.

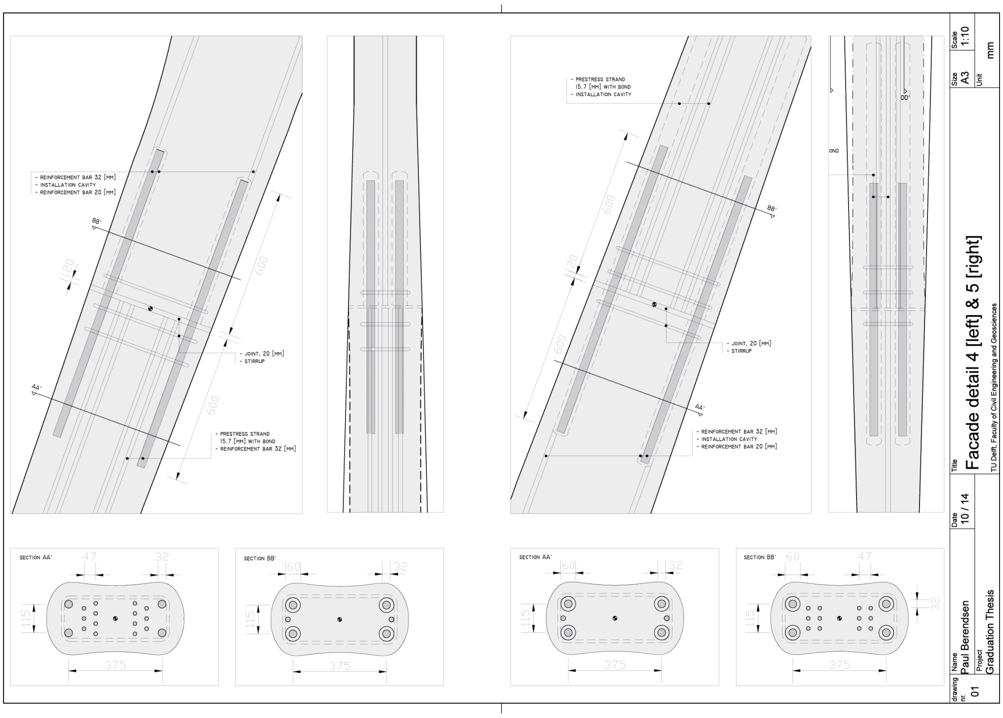
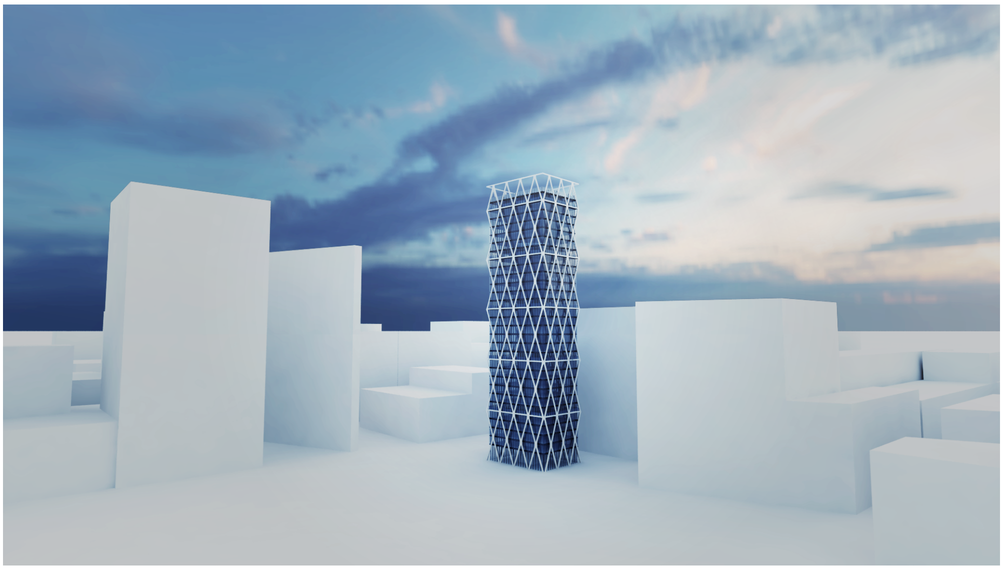


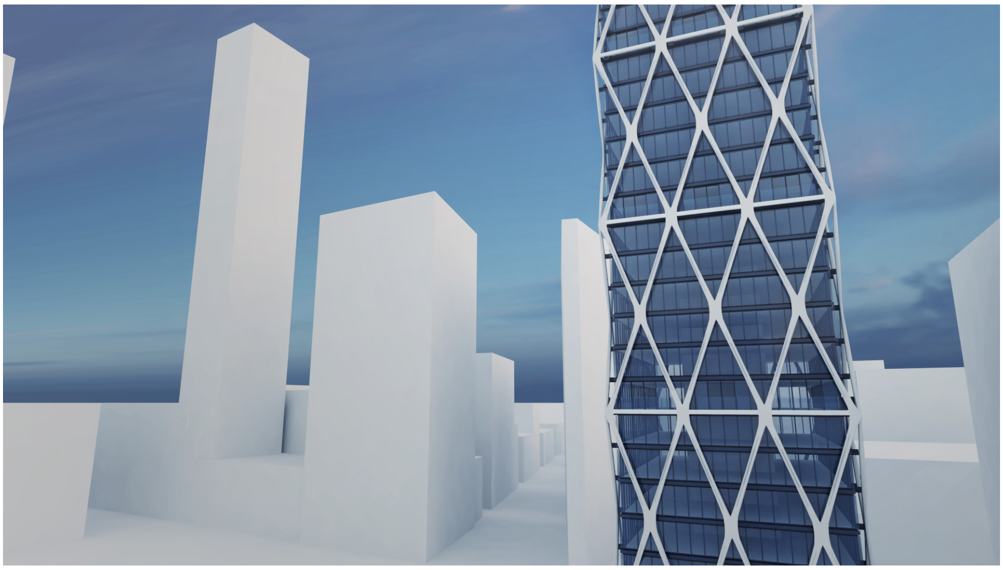
Figure 38: Detail 4 (left side) and 5 (right side), full size drawing can be found in Appendix I

7.8 VISUALISATIONS

A selection of visualisation is presented in this section.



(a)



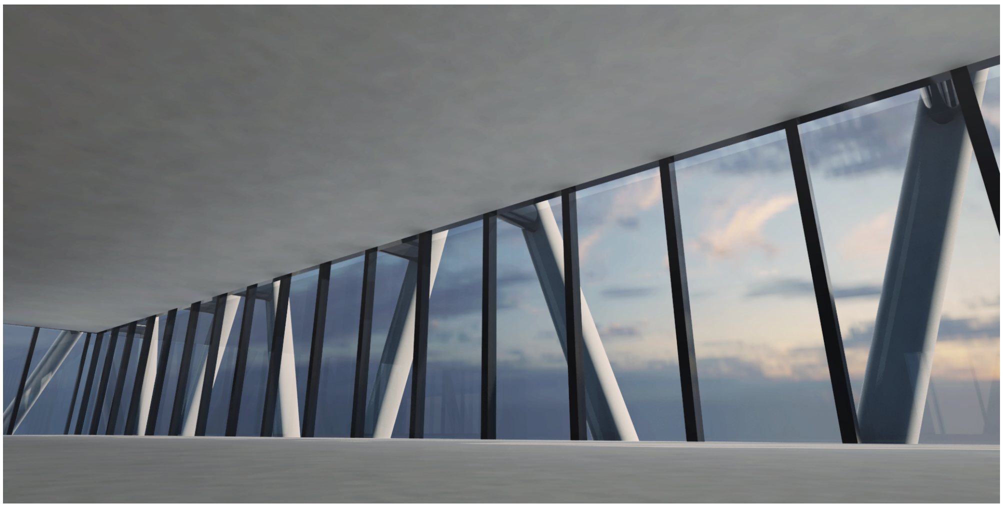
(b)

Figure 39: Birds eye view

Figure 40: Ground floor level of the building

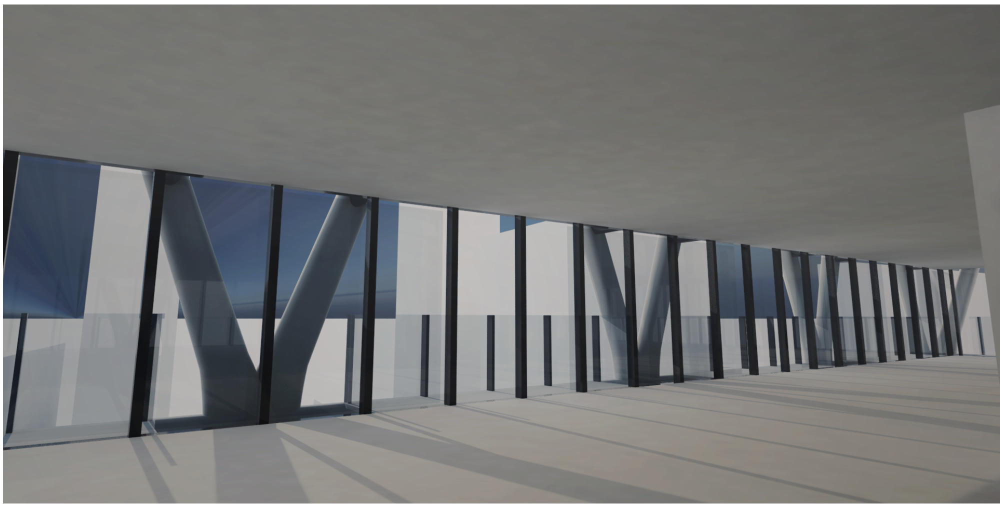
Figure 41: Top levels of the building

Figure 42: Detail of the Façade structure



(a)

(b)



(c)

Figure 43: Interior view

Part IV

FINAL REMARKS

8 CONCLUSIONS

The objective of this research is to exploit the design possibilities of the promising construction material UHPFRC in order to derive an innovative and distinctive design. In this chapter the conclusions derived by the conducted research are presented.

8.1 LITERATURE STUDY

8.1.1 *Characteristic and distinctive properties (Ch.2)*

The characteristic properties of UHPFRC are partly already given by the definition. Though, the combination of properties which distinguish the material from any other material as well from conventional concrete are given here below.

8.1.1.1 *The structural properties*

With a compression resistance generally higher than 150 MPa and a flexural strength consistently above the 7 MPa the material has excellent structural properties. The compression resistance is significant higher than the strength of conventional concrete of which the strength only goes up to 90 MPa. The maximum compression resistance that can be derived with UHPFRC is comparable with the resistance of high grade steel. The fibres added increase the tensile and flexural capacity of the material significantly compared with conventional concrete. Nonetheless, the tensile and flexural capacity remains low compared with the compression resistance. This makes prestress, where required, a valuable addition to the material.

8.1.1.2 *The durability*

Due to a very fine and well composite mixture with a large amount of bonding agent a very dense material with a low capillary porosity is created. This makes the material extreme durable even in tough and salty conditions. External influences will penetrate the material only superficially allowing steel reinforcement to be located only a few millimetres from the outer perimeter.

8.1.1.3 *The formability*

The small en refined particles in UHPFRC result in a mixture with excellent workability. This, combined with advanced mould types such as injection moulds enable a great variety of shapes. Though, complex shapes complicate the production process which put a limit on the formability.

8.1.2 *Construction calculations (Ch.2)*

The European standards and guidelines for the construction industry (Eurocode) provide calculations methods for concrete up to a strength of 90 MPa. Due to the significantly higher compression capacity of UHPFRC this document is not applicable. The French Association for Civil Engineering AFGC have published recommendations offering a comprehensive set of calculation methods for UHPFRC. Although, the document provides sufficient basis to prove the structural safety it has not yet been approved as valid document by the European Union. The major UHPFRC produces have sufficient know-how about their products to prove structural safety. By now, the applications of UHPFRC in various countries prove UHPFRC can be applied with the approval of the local building authorities. The lack of applicability of the Eurocode might be inconvenient and is likely to lead to additional engineering cost. Nonetheless, it does not exclude application of the material as safety can be demonstrated and can be approved by local building authorities

8.1.3 *State of applications (Ch.3)*

By now the material UHPFRC has been applied in diverse fields for different applications. The ultra thin balconies which won the Dutch Beton prijs in 2013 are a striking example of a successful application in the building engineering in the Netherlands. Globaly the French architect Ricciotti has been pushing the boundaries further than anyone. With unique applications of the material in the stadium Stade Jean Bouin and the recently finished museum MuCEM Ricciotti shows UHPFRC being used for distinctive building design. All the examples considered, a vivid tendency of innovative applications is demonstrated.

8.1.4 *Suitable elements (Ch.4)*

Building elements for which UHPFRC as construction material is most suited are application that can make use of all the characteristic properties of the material. In general this are load carrying elements which have a remarkable impact upon the aesthetics of a building. The following list with possible building elements is drafted:

- Column
- Load carrying external wall / Façade
- Load carrying interior wall
- Structural roof element
- Main support structure

The typology of elements remains vague as numerous different type of these elements exist, therefor the previous stated in this subsection is leading.

8.2 DESIGN STUDY

In the design study a the façade of a 110 m tall building is considered (Ch.5). With the use of simple hand calculations the imposed design principles could be used to derive a façade diagrid design and accompanying typical floor configuration. Subsequently the design could be model in engineer software to further analyse the structural behaviour. The normative loads which had been derived with the analyses combined with additional imposed architectural demands have, by means of an iterative process, been used to design the diagrid element cross-sections (Ch.6). The next part of the design study was focused upon a realistic elaboration of the design in order to consider all required aspects for a successful application of UHPFRC. The ULS and SLS have been verified, building sections have been elaborated, a construction method was considered and 1:10 construction details, according the structural en architectural demands, have been elaborated (Ch.7).

8.2.1 *Building configuration*

To exploit the formability of the material the load carrying structure was located at the outside of the façade. Where for the design of diagrid structures most often steel is selected as construction material, constructing in UHPFRC turns out to be a possible alternative which offers various technical and architectural advantages. By the locating the façade structure outside the thermal isolation perimeter while keeping the load carrying core of the building inside a thermal difference is introduced. As proven the complications as a result of this thermal difference can be overcome. A configuration with a load carrying structure outside the building which is highly visible is suitable for distinctive and prestige projects. Nonetheless, a tradeoff between a configuration with the structure outside the isolation perimeter or inside, whereby the structure remains visible from the outside of the building, should always be considered.

8.2.2 *Cross-section design*

The material UHPFRC is highly at its place for construction element which are mostly loaded in compression. Due to the formability of the material a great design freedom for the cross-section is introduced. This design freedom is applicable for every construction element predominantly loaded in compression. The freedom in design of cross-section shape makes the material stand out and to distinguish itself other materials.

8.2.3 *Construction details*

Construction detail should preferably be located at locations where the imposed loads are relatively low, connections therefor can be design. In the design study both a construction detail similar to; a conventional concrete connection and a conventional steel connection are elaborated. According the conducted strength calculations both connections prove to be applicable. The detail similar to the conventional concrete connection appears to be stiffer.

8.2.4 *Buckling*

The high compression capacity of UHPFRC leads to the possibility to design slender elements. For these elements buckling will become the normative failure mechanism. By designing a structure in UHPFRC it should be more considered to be a material similar with steel than a material similar with conventional concrete. By result there should be sought for configurations whereby the buckling length stays limited in order to avoid buckling and to remain the slenderness of the elements. Sensitivity for buckling leads to high imposed flexural forces at the material. The tensile and flexural capacity of the material should increase significantly to avoid the need for prestress steel.

8.3 RESEARCH QUESTION

The conclusions formulated at the literature- and design study have given answers to the sub research questions as posed in the introduction of this report, see section 1.4. The information gathered with these studies have given the required information to give answer to the following main research question:

"How can Ultra High Performance Fibre Reinforced Concrete be used to develop and push innovative and distinctive designs in the building engineering?"

The successful application of UHPFRC in the building engineering had already been demonstrated by a number of projects. However, a profound research in the wide range of possibilities in the building engineering still had to be addressed. Several applications whereby the characteristic properties of UHPFRC can be fully exploited are identified. One of these application is elaborated up from overall building design down to construction details showing the technical possibility to derive a design which is innovative and distinctive. Hereby a design approach is demonstrated

With the material-, construction- and design properties studied. A design approach is developed which can be applied to derive distinctive and innovative designs for construction elements that are mostly loaded in compression. With the feasibility of an actual application depending on the ambitions of a client for innovation and distinction in return for additional construction costs. The applied design approach is suitable for application in the niche market of prestige projects

It is concluded that UHPFRC is a very suitable material to push innovation and to develop distinctive designs in the building engineering. The developed design approach demonstrated in the case study, can be applied to derive designs which are likely to be worth spending the additional construction costs that are bound on the application of UHPFRC.

9 RECOMMENDATIONS

9.1 THE DESIGN

9.1.1 *Building configuration*

The configuration of a diagrid structure appeared to offer various advantages. Nonetheless challenging structural details can be avoided by though the design of a circular floor plan. With the rectangle configuration the outward forces of the diagrid structure concentrate at the end of the façade plane. This results a heigh imposed loads at one connection. The concentration of forces can be avoided by the implementation of a circular floor plan.
Building configurations of less height but without a structural core can as well be considered.

9.1.2 *Design of structural members*

The applied, proposed, design approach if suitable for UHPFRC and can be used to exploit the material characteristic and to derive a distinctive structure. In the cases study only architectural demands and structural requirements have been used for the cross-section design. Nonetheless, by considering climate design of the cross-sections, a façade structure which might make other climate design measures unnecessary can be derived.

9.1.3 *Construction method*

The prefabricated elements are divided into segments. A different configuration of element, for example, by constructing a column attached to a node. Can reduce the number of construction connections.

9.1.4 *Detailing*

The construction details strength has been verified with hand calculations. Modelling of the connections with finite element software can be used for further optimisation of the construction details.

9.2 FURTHER RESEARCH

9.2.1 *Economical feasibility*

Due to the possibility to design structures that are of a significantly lower weight, construction cost of the building façade can be reduced. simultaneously, the reduced structure size will lead to more useful floorspace. An economical feasibility study for the application of UHPFRC conventional buildings can be performed.

9.2.2 *Material*

The material characteristic as given by the manufactures has been considered in the case study. To compensate for the low tensile flexural resistance of the material prestress is applied. A material with higher tensile and flexural resistance, even on expense of the compression resistance, can avoid the use of prestress in many cases. Further research to improve the tensile and flexural capacity of UHPFRC can be performed leading to a more convenient construction material.

Part V

APPENDIX

A INTERVIEWS

A.1 Hans Köhne

After being educated in Building Engineering (Bouwkunde) and Marketing Strategy Hans Köhne works at Cement&BetonCentrum as marketing manager. His discipline is mainly focussed upon: market and product development, product development and market communication.

A.1.0.1 *Q. Which of the developments in the material properties of concrete do you find the most important?*

A. In order to design real life applications the Cement&BetonCentrum has done case studies with the material UHPFRC. With choosing the applications the following starting points where considered: firstly the material is free of maintenance and secondly it is not sensitive for external influences. Because of these principles applications in infrastructure where considered. Considering infrastructure structures have not to be closed for maintenance should exploit the advantage of this material. This will make application feasible in the long run.
The material is extremely free formable this was however not a starting point for selecting applications. Usually in the Netherlands little extra money is being paid for beautiful applications.

A.1.0.2 *Q. For which applications do you think UHPFRC is a suitable material?*

A. The material could be very suitable for columns at the first floor of a building; this will namely lead to several advantages. Less material is required. Weight of the building is reduced so a lighter foundation can be applied. The ground floor level of the building will get way more open. This is exactly inline with the needs of the users of tall buildings. These advantages could lead to a feasible application of UHPFRC.
The material seems most suitable for vertical elements. This includes columns and bearing façades.

Considering applications in buildings: Select an application that has big influence on the appearance of the building. Select an application that can benefit from the durability of the material. I think of smooth forms like UNStudio and Zaha Hadid and integrated sunshades. I think smooth lines are UHPFRC. Look for the extremes!

A.1.0.3 *Q. Why is UHPFRC of UHPC only very rarely applied in the Netherlands? Does this have something to do with the designs?*

A. This material will mostly be appropriate for special structures and extraordinary buildings. This will not be a material appropriate the bulk of the buildings. In the Netherlands we do not very often build those extraordinary

buildings.
It could however be very suitable for applications in renovation of viaducts and tunnels. It would be very profitable if the service live can be elongated by 30 years with the application of UHPFRC.

A.1.0.4 *Q. What do you think are the needs of the clients?*

A. The principals attach values to: The aesthetics & the functional quality.

A.1.0.5 *Q. Do you see drawbacks attached to this material? Can the design make up these drawbacks?*

A. Crucial are the production technics. As the material is extremely free formable it is being restrained by the possible production techniques.
Because of the high compression capacity of the material it is good to be used in combination with pre-tension.

A.1.0.6 *Q. Often façades of buildings are being shaped by the fashion of today and will be outdated after 30 years. Façades of UHPFRC will last at least 100 years. Will this conflict?*

A. For exceptional building this will not be a problem. These will be respected.

A.2 PIERRE VAN BOXTEL

Pierre van Boxtel is Commercial technical advisor at the concrete prefab factory Hurks. Due to his profession he is well acquainted with the demands from clients and the market.

A.2.0.7 *Q. Which of the developments in the material properties of concrete do you find the most important?*

A. The improved compression capacity is an improvement; because of this thinner designs can be made. As well the improved durability is an valuable improvement.

A.2.0.8 *Q. For which applications do you think UHPFRC is a suitable material?*

A. Very thin floor slab, comparable with the infra+ floor, can be made out of it. Applications in decorative elements where in reinforcement is hard to apply. And applications in the renovation seem interesting to me.

A.2.0.9 *Q. What do you think are the possibilities of UHPFRC?*

A. It certainly has the benefits of conventional concrete. So it is a very solid material, the surface can be modified/edited, it has good fire-resistant properties and it requires hardly any maintenance.

A.2.0.10 *Q. Which constrains do you see for UHPFRC?*

A. While this material will last for more than a 100 year, the fashion will not last for a 100 years. Is it as fire resistant as conventional concrete? Can it have the same finishing as conventional concrete?

A.2.0.11 *Q. Why is UHPFRC or UHPC only very rarely applied in the Netherlands? Does this have something to do with the designs?*

A. In the Netherlands parties are usually not willing to pay the required additional money. And unique structures are not very often being built in the Netherlands. In the Netherlands we are rather conservative, especially when something is expressive.

A.2.0.12 *Q. Which constrains do you see for UHPFRC?*

A. It is 20 times as expensive as conventional concrete.

The connections can be very problematic, steel can be welded together concrete required plastic connections. Because of the reduced size of elements it will as well result in very high forces at connections.

A.2.0.13 *Q. For which precast element do you experience a high demand from the market?*

A. Façade elements.

A.2.0.14 *Q. What do you think are the needs of the clients?*

A. The client wants something that is unique.

A.3 ROB VERGOOSSEN

Rob Vergoossen is a Structural Engineer at Royal HaskoningDHV. Rob has been involvement in several UHPFRC design-projects, like for instance the helix cycle bridge.

A.3.0.15 *Q. Which of the developments in the material properties of concrete do you find the most important?*

A. In general I do not consider the high compression capacity the advantage of high strength concrete. I think that the durability and the potential that the fibres can replace reinforcement bars are the two major advantages bound to this material. A high performance concrete with a compression capacity of about 100 MPa and a flexural strength of 40 MPa will be an ultimate mixture which will allow you to make every possible form.

A.3.0.16 *Q. For which applications do you think UHPFRC is a suitable material?*

A. The material will be useful for a structure that will have to last a long time and suffers under though conditions.
It can be applied for bridges where the vertical clearance should be increased. If a viaduct can span an entire road and supports in between roads are prevented due to the use of it; it will be very feasible application.
It can be used to improve the strength of columns during renovation. The cover of columns can be taken of and be replaced by UHPFRC. This will significantly improve the compression capacity of the columns.
This material will not improve the efficiency of a building. By translating a conventional building into UHPFRC it won't be made a much more efficient building (meaning lighter, more useful floor space and cheaper).

A.3.0.17 *Q. What do you think are the possibilities of UHPFRC?*

A. Two things of major importance for the possibilities of the material: repetition and connections. The mould should be reused as many times as possible and connections can be normative regarding the size of elements.
Regarding the UHPFRC material properties, you should use the flexural strength of the material. If you don't it is a waste of the fibres and you should not use more than 80 kg/m^3 of fibres.
Apply pre-stress in UHPFRC elements has many advantages and it goes well with the high compression strength of the material. Another possibility is to make very fine imprints in this material.

A.3.0.18 *Q. Which constrains do you see for UHPFRC?*

A. At reinforcement bars at least 10 mm of concrete cover should be applied considering the practical execution. The calculation method in France is based upon many material tests. So far the calculation methods are very conservative. For structures not the compression strength but buckling will be normative.

A.3.0.19 *Q. Do you see drawbacks attached to this material? Can the design make up these drawbacks?*

A. An arch can be made out of UHPFRC if the load is ideal constant. An asymmetric load will result in a changing load. If at a part of a structure both tension and compression can occurs steel is often a better solution. This can be solved by searching for a statically determined structure or make the variable load small with regards to the dead weight.

A.3.0.20 *Q. Which applications that are usually made out of steel should do you think be made out of UHPFRC?*

A. You should use this material for round and smooth shapes. These shapes are hard to make with steel but very good with concrete. Ricciotti also designs round and smooth shapes which are hard to make with steel.

A.3.0.21 *Q. I want to design a new application in UHPFRC which responds to the characteristic properties of UHPFRC. Which advice can you give me?*

A. You need to start with nothing and think how you can make that what you want. Think of the possibilities of this material and do not use it as an expensive cover for your prestress tendons. By making in you finally really learn really how to make it.

A.4 ROGIER VAN NALTA

Rogier van Nalta is a Engineer at Pieters Bouwtechniek. Rogier has played a major role in the development and the first application of the ultra thin balconies in UHPFRC in the Netherlands. Beside this Rogier has got experience with several other applications in UHPFRC.

A.4.0.22 *Q. Which of the developments in the material properties of concrete do you find the most important?*

A. It is the combination of all the developments. You shouldn't see these apart. The very dense structure increases several properties that are all important.

The ancestor of UHPFRC is Henrik Bache, from Aalborg Portland. His first breakthrough triggered an extensive research program and resulted in the UHPFRC Compact Reinforced Composite (CRC). The combination of the various properties make the high performance of this material.

A.4.0.23 *Q. For which applications do you think UHPFRC is a suitable material?*

A. Currently we use CRC for many applications. Our main application is currently balconies and stairs. We also use it for instance for stables; the material is very suitable to withstand the impact of a horse that kicks. It is also used for foundations of machinery and for foundations of offshore windmills. It can also be used for military applications like bunkers.

In Denmark student housings have been developed with sandwich walls made from UHPFRC.

A.4.0.24 *Q. What do you think are the possibilities of UHPFRC?*

A. The freedom in formability is a great property of the material. It behaves well in extreme environments. The material we use, CRC, has very good fire resistance properties. A square column of 120mm can withstand a fire of 60 minutes. However good fire resistance is certainly not standard for UHPFRC. There are examples of UHPFRC mixtures that dissolve under heat. It is after a long development process that the material mixture of CRC now has such a good fire resistance property.

The Ultra high strength of the material is used in de connections and details

A.4.0.25 *Q. Which constrains do you see for UHPFRC?*

A. Often the stiffness of the material becomes determinative. The balconies that we have designed can withstand three times the design load.

A.4.0.26 *Q. Do you see drawbacks attached to this material? Can the design make up these drawbacks?*

A. In general the price of the material is a drawback. Ductal is made with only the best of the best materials and production techniques, this makes the material very good but extremely expensive. CRC is a much cheaper alternative, the material is slightly less 'good' but it costs half the price.

When using the best material properties we are able to design and construct very extreme and special structures. Unfortunately than the price is determinative and a cheaper alternative is constructed. By using slightly lower properties the material cost is significantly reduced. This makes the structure a little bit less extreme, but the project becomes feasible.

The material must be prefabricated, so there is a limited element size. Prefab elements can be cast together with "Joint Cast" this is an UHPFRC which is even stronger than standard CRC.

A.4.0.27 *Q. Which applications that are usually made out of steel should do you think be made out of UHPFRC?*

A. The material is a very good alternative for cast steel. It is even cheaper than that.

A.4.0.28 *Q. For which precast element do you experience a high demand from the market?*

A. The past year there were over a 100 projects whereby UHPFRC was considered or applied. There is demand from many niche markets.

A.4.0.29 *Q. Do you think the design of the FDN Engineering bridge shows how a design in UHPFRC should be?*

A. FDN Engineering started with developing a bridge in UHPFRC, while they didn't had any experience with:

- Constructing in UHPFRC
- Designing in UHPFRC
- And Calculate with UHPFRC

This made the entire process very difficult. The design of the bridge could be better.

At Pieters bouwtechniek we have designed several bridges which in my opinion better show the possibilities of UHPFRC.

A.4.0.30 *Q. I want to design a new application in UHPFRC which responds to the characteristic properties of UHPFRC. Which advice can you give me?*

A. With this material you should design a flagship store. A building like the Apple Cube for instance. For these kinds of applications the client asks to get the maximum out of the material.

Choose material properties as a starting point.

B BUILDING ELEMENTS

B.1 INVENTORY

Inventory of building elements based upon the systems presented in chapter 4. The selected building elements are assessed upon the degree to which they are *Load carrying* and to which they *Influence the aesthetics of a building*.

BUILDING ELEMENT	LOAD CARRYING	VISABLE
Floor element	+	+/-
Column	+	+
Load carrying external wall / Façade	+	+
Certain façade	+/-	+
Load carrying interior wall	+	+
Structural roof element	+	+
Façade fences	-	+
Skylight domes and covers	+/-	+
Stairs	+/-	+
Movable staircases	-	+
Slopes	+/-	+
Ladders	+/-	+/-
Railings	-	+
Canopy elements	+/-	+
Lintels	-	+
Windows sills	-	+
(Foundation) Consoles	+	-
Retaining walls	+	-
Main support structure	+	+
Floor finishingś	-	+

(+) Load carrying element | Clearly visible element

(+/-) Self weight carrying element | Moderate visible element

(-) Supported element | Badly visible element

B.2 VARIANTS AND ANALYSIS

Of the following elements different variants are identified. In order to determine which variant is most suitable to be designed and made out of prefabricated UHPFRC, the variants are subjected to a pro-con analysis are: a column, external wall / Façade, Interior wall, roof element, main support structure and stairs and slopes.

B.2.1 *Column*

B.2.1.1 *Variant 1*

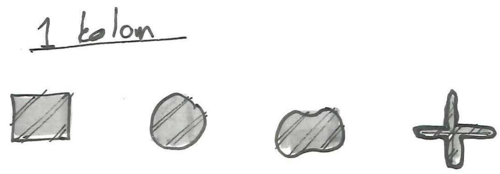


Figure 44: One load carrying column

- Any cross-section shape is possible (Pro)
- Columns can be build up out of multiple elements (Pro)
- Pre-stress can be applied to reduce buckling effect (Pro)
- Buckling of thin element (Con)
- No space for pre-stress anchorage system (Con)

B.2.1.2 *Variant 2*

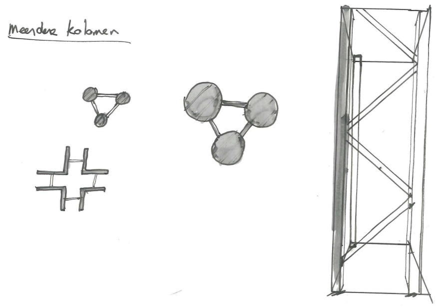


Figure 45: Multiple columns connected

- Reduction of buckling length, which lead to a more effective use of material (Pro)
- Column can contributes to aesthetic of the building (Pro)
- Additional functions can be integrated inside the column (Pro)
- Entire column requires much floor space (Con)

B.2.1.3 *Variant 3*

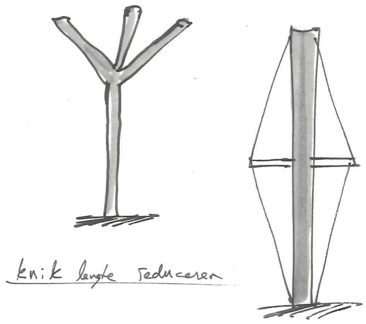


Figure 46: One column; reduced buckling length by either external prestressing or the addition of elements

- Reduction of buckling length, which lead to a more effective use of material (Pro)
- Column can contributes to aesthetic of the building (Pro)
- Addition of elements might be difficult to produce (Con)

B.2.2 *External wall / Façade*

B.2.2.1 *Variant 1*

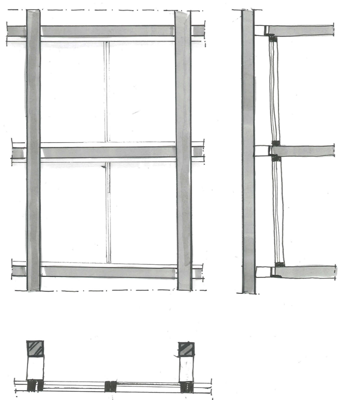


Figure 47: Load caring structure - Separating functions

- Structural system contributes to aesthetic of the building (Pro)
- material is purely loaded with compression (Pro)
- Cold bridges arise at the connections with the floor (Con)
- Sensitive for column buckling (Con)

B.2.2.2 *Variant 2*

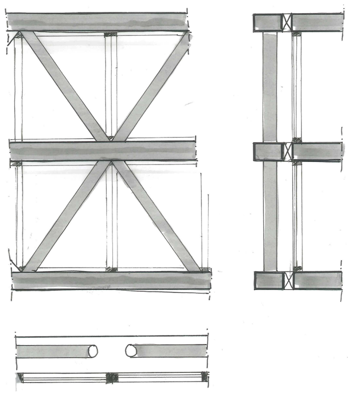


Figure 48: Load caring structure and Stability system - Separating functions

- Structural system contributes to aesthetic of the building (Pro)
- material is purely loaded with compression (Pro)
- Can as well be a stability system (Pro)
- Sunshade can be integrated into the structural system (Pro)
- Cold bridges arise at the connections with the floor (Con)
- Sensitive for column buckling (Con)

B.2.2.3 *Variant 3*

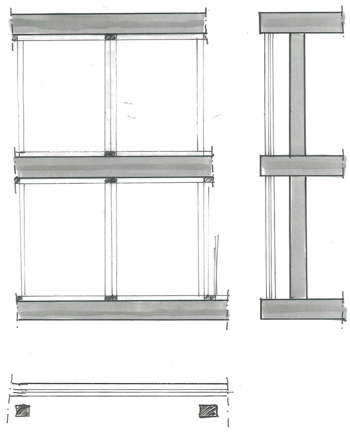


Figure 49: Separating functions - Load caring structure

- Structural system contributes to the aesthetic of the interior (Pro)
- No thermal leakage at connection between columns and floors (Pro)
- Structural system contributes less to exterior of the building (Con)
- Can cause problems for interior (Con)
- Sensitive for column buckling (Con)

B.2.2.4 *Variant 4*

Figure 50: Separating functions and Load caring structure

- Very thin façade element is possible (Pro)
- Very light façade elements is possible (Pro)
- Stability system can not be integrated (Con)
- Difficult to isolate (Con)

B.2.2.5 *Variant 5*

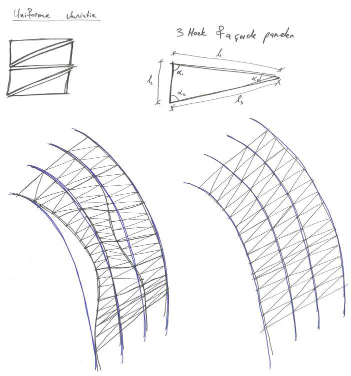


Figure 51: Separating functions, horizontal orientation

- Every possible shape can be made with triangles (Pro)
- Stability system can be integrated (Pro)
- A load carrying structure is still required (Con)

B.2.2.6 *Variant 6*

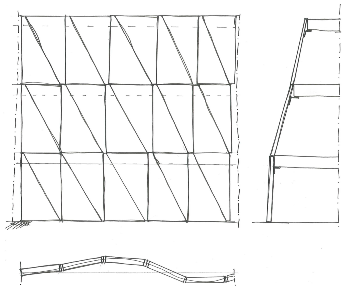


Figure 52: Separating functions, vertical orientation

- Can be a load barring system (Pro)
- Every possible shape can be made with triangles (Pro)
- Windows over entire hight of a floor are not possible (Con)
- Difficult connections are needed (Con)

B.2.3 *Interior wall*

B.2.3.1 *Variant 1*

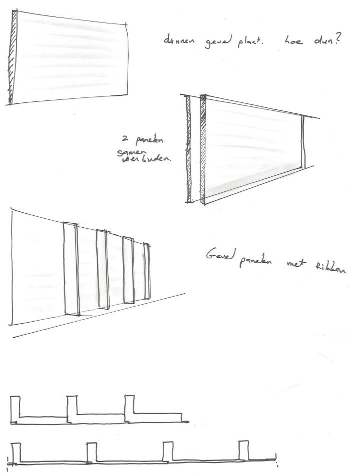


Figure 53: Separating functions and load caring structure (evolution of External wall / Façade variant 4)

- Very thin and light façade elements is possible (Pro)
- Inner and outer panel can work together to improve stability (Pro)
- Buckling of the plates (Con)
- Thin plates are difficult to produce (Con)

B.2.4 *Shell structures*

B.2.4.1 *Variant 1*

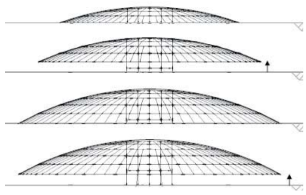


Figure 54: Structural system variant 5

- System contributes to aesthetic of the building (Pro)
- Extreme structures are possible (Pro)
- Already investigated by R. ter Maten in 2013 (Con)
- Moment forces will be introduced, incase of asymmetric load (Con)

B.2.5 *Main support structure*

B.2.5.1 *Variant 1*

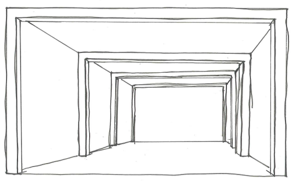


Figure 55: Column and beam portal

- Floorspace free of columns (Pro)
- Moment forces will be introduced into both column and beam (Con)
- Is this the kind of structure which belongs to UHPFRC? (Con)

B.2.5.2 *Variant 2*

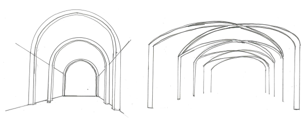


Figure 56: Column and arched beam portal

- Material mainly loaded in compression (Pro)
- Various layouts possible (Pro)
- Moment forces will be introduced, incase of asymmetric load (Con)

B.2.5.3 *Variant 3*

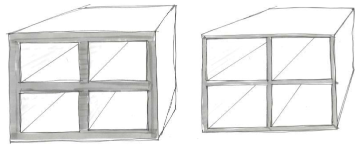


Figure 57: Wall and floors system

- Very thin walls and floors possible (Pro)
- Sound isolation will be critical (Con)
- Added value to the design of the building is small (Con)

B.2.5.4 *Variant 4*

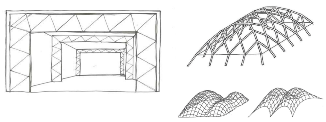


Figure 58: Space unit system, as a load carrying façade transformed into an arch.

- System contributes to aesthetic of the building (Pro)
- Extreme structures are possible (Pro)
- Difficult connections (Con)
- Moment forces will be introduced, incase of asymmetric load (Con)

B.2.6 *Stairs and slopes*

B.2.6.1 *Variant 1*

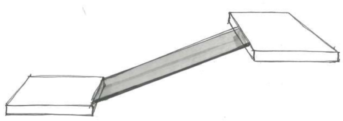


Figure 59: Stairs and slopes variant 1

- Very thin plats are aesthetically interesting (Pro)
- Very light slopes and stairs are possible (Pro)
- Element will can be self-supporting but it will not be a part of the structural system of the building (Con)

C

CALCULATION METHODS

C.1 COLUMN

C.1.1 *Required reinforcement ratio*

In order to perform a quick column calculation the consultancy firm CAE Nederland have developed a graphic method to determine the required reinforcement ratio, see subsection 2.6.4. This method is similar as the conventional graphical method to determine the required reinforcement ratio for normal strength concrete. With the graph developed for UHPFRC the method for conventional can be used for UHPFRC as well.

C.1.2 *Column graph*

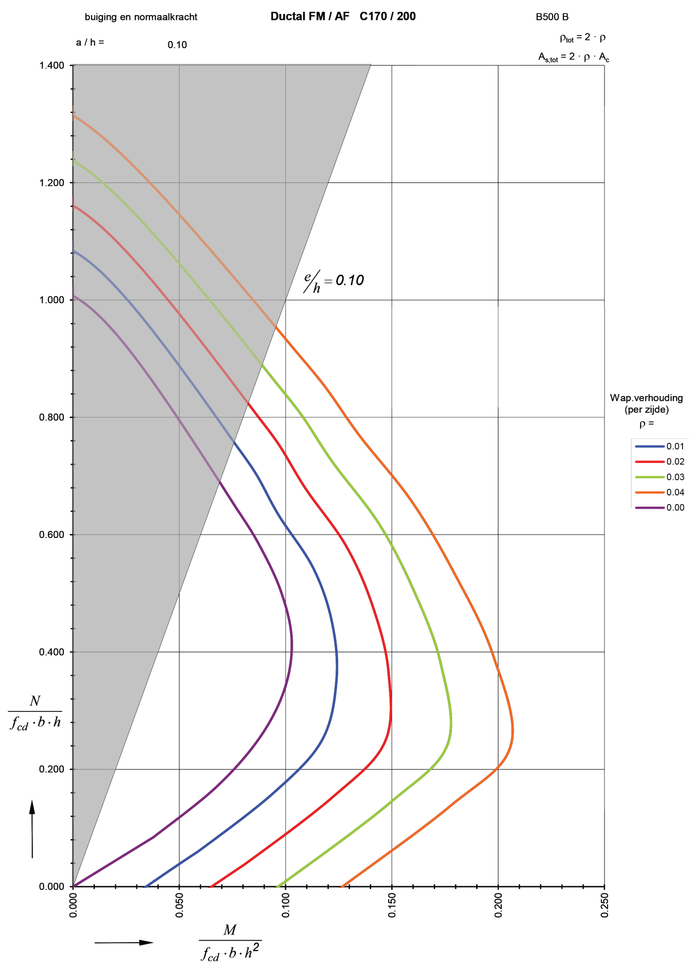


Figure 60: Column graph

The two input variables for this graphical method are:

$$N_{factor} = \frac{N_{Ed}}{f_{cd} \cdot b \cdot h} \quad [-] \tag{20}$$

$$M_{factor} = \frac{M_{Ed}}{f_{cd} \cdot b \cdot h^2} \quad [-] \tag{21}$$

Where in:

N_{Ed}	=	Design value of normal force (excluding prestress)	[N]
M_{Ed}	=	Design value of imposed moment force	[Nmm]
f_{cd}	=	Design value of concrete strength	[N/mm^2]

C.1.3 *Design Moment*

Besides an possible imposed moment force, an additional moment will develop due to geometrical imperfections and second order effects. The Eurocode 2: 'Design of Concrete Structures' includes a method to determine the magnitude of this moment forces. As the Eurocode document holds for concrete up to a strength class of C90, this method can not be used for UHPFRC. In order to still make use of this calculation method in the design study the concrete elements are, for this second order calculation, regarded to be concrete elements of a strength class C90. As the second order effects depend for a large extent upon creep, this approach is considered to be a safe approach, because the creep effect for UHPFRC is considerable lower than the creep effect for concrete of a lower strength class.

The design moment can be determined with Equation 22; this is as described in Eurocode 2 section 5.8.8.2.

$$M_{Ed} = max\,\{M_{02};\ M_{0e}\ +\ M_2;\ M_{01}\ +\ 0.5\ M_2\}\ +\ e_1 N_{Ed} \tag{22}$$

Where in:

M_{01}	=	$min\,\{\ \lvert M_{head}\rvert;\ \lvert M_{foot}\rvert\}$
M_{02}	=	$max\,\{\ \lvert M_{head}\rvert;\ \lvert M_{foot}\rvert\}$
e_i	=	$l_0/600$ = 1st order moment, see Equation 27
M_{head}	=	moment form framework calculation at head column
M_{foot}	=	moment form framework calculation at foot column
M_{0e}	=	$0.6\ M_{02}\ +\ 0.4\ M_{01}\ >\ 0.4\ M_{02}$
M_{02}	=	$N_{Ed}\ e_2$
N_{Ed}	=	Design value of normal force
e_2	=	Eccentricity halfway the column, see Equation 29

C.1.3.1 *Check to neglect*

According Eurocode 2 section 5.8.3 the second order effects may be neglected if the slenderness defined by Equation 24 is below the limit value of Equation 23. If this is not the case the second order effects should be considered, for this calculation see Equation 29.

$$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n} \tag{23}$$

$$\lambda = l_0 / i \tag{24}$$

Where in:

A	=	$1/(1 + 0.2 \cdot \varphi_{eff})$	0.7 may be used
B	=	$\sqrt{1 + 2 \cdot \omega}$	1.1 may be used
C	=	$1.7 - r_m$	0.7 may be used
n	=	$N_{Ed}/(A_c \cdot f_{cd})$	
l_0	=	Effective buckling length	
i	=	$\sqrt{I/A_c}$	
φ_{ef}	=	Effective creep ratio	See Equation 25
ω	=	Mechanical reinforcement ratio	
r_m	=	M_{01}/M_{02} = Moment ratio	1 may be used
N_{Ed}	=	Design value of axial load	
f_{Ed}	=	Design compressive strength concrete	[MPa]
A_c	=	cross-sectional area	

C.1.3.2 *Creep coefficient*

The effect of creep must be taken into account for a second order calculation. The effect is taken into account by means of an effective creep ratio. This ratio can be determined with Equation 25 as described in Eurocode 2 section 5.8.4 . Elaboration of this equation results in Equation 26.

$$\varphi_{ef} = \varphi_{\infty,t0} \cdot \frac{M_{0Eqp}}{M_{0Ed}} \quad [\text{-}] \tag{25}$$

$$\varphi_{ef} = \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) \cdot \beta_c(t, t_0) \cdot \frac{M_{0Eqp}}{M_{0Ed}} \quad [\text{-}] \tag{26}$$

Where in:

φ_{RH}	=	$\left(1 + \frac{1 - RH/100}{0.1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1\right) \cdot \alpha_2$	for $f_{cm} > 35MPA$
$\beta(f_{cm})$	=	$\frac{16.8}{\sqrt{f_{cm}}}$	[-]
$\beta(t_0)$	=	$\frac{1}{0.1 + t_0^{0.2}}$	[-]
$\beta_c(t, t_0)$	=	$\left(\frac{t - t_0}{\beta_H + t - t_0}\right)^{0.3}$	[-]
M_{0Eqp}	=	First order bending moment in SLS	
M_{0Eqp}	=	First order bending moment in ULS	
RH	=	Relative humidity of the environment	[%]
h_0	=	$\frac{2 \cdot A_c}{u}$	
α_1	=	$\left(\frac{35}{f_{cm}}\right)^{0.7}$	
α_2	=	$\left(\frac{35}{f_{cm}}\right)^{0.2}$	
f_{cm}	=	Mean compressive strength of concrete	[MPa]
t_0	=	age of concrete at day of loading	
t	=	age of concrete at moment considered	
β_H	=	$1.5 \cdot \left(1 + (0.012 \cdot RH)^{18}\right) \cdot h_0 + 250 \cdot \alpha_3$	$f_{cm} > 35MPA$
u	=	perimeter in contact with atmosphere	
α_3	=	$\left(\frac{35}{f_{cm}}\right)^{0.5}$	

C.1.3.3 *Geometric imperfections*

The magnitude of a geometric imperfections of a column in a braced system can be determined with Equation 27 (Eurocode 2 section 5.2 (9) complemented with the accompanying Dutch Annex). For ULS calculations a minimum eccentricity should be taken into account. This minimum eccentricity is given by by Equation 28 with h being the depth of the section (Eurocode 2 section 6.1 (4)).

$$e_i = l_0/600 \quad [mm] \tag{27}$$

$$e_0 = h/30 \geqslant 20 \quad [mm] \tag{28}$$

C.1.3.4 *Second order eccentricity*

The second order eccentricity can be determined with Equation 29 as described in Eurocode 2 section 5.8.8.2 (3) . Elaboration of this equation results in Equation 30.

$$e_2 = \frac{l_0^2}{r \cdot c} \tag{29}$$

$$e_2 = K_r \cdot K_\varphi \cdot \frac{l_0^2}{\pi^2} \cdot \frac{\epsilon_{yd}}{0.45 \cdot d} \tag{30}$$

Where in:

K_r	=	$\frac{1+\omega-n}{0.6+\omega} \leqslant 1$	
K_φ	=	$1+\beta \cdot \varphi_{ef} \geqslant 1$	
ϵ_{yd}	=	f_{yd}/E_s	
d	=	Effective depth of reinforcement	
β	=	$0.35+\frac{f_{ck}}{200}-\frac{\lambda}{150}$	
λ	=	Slenderness ratio	See Equation 24

D DESIGN LOAD

D.1 Building weight load

In order to determine the design load on the bearing system of the building, with in particular the load on the ground floor columns. The various loads are added up. The following load per floor are considered:

- Variable load
  - Roof: $2\,\mathrm{kN/m^2}$
  - Floor: $3\,\mathrm{kN/m^2}$
- Dead load
  - Roof: $3\,\mathrm{kN/m^2}$
  - Floor: $4\,\mathrm{kN/m^2}$
  - Facade: $1.8\,\mathrm{kN/m^2}$

Considering the building is public accessible the safety factors belonging to Eurocode consequence class III are used.

- Variable load: 1.65
- Variable load advantageous 0
- Dead load: 1.3
- Dead load advantageous 0.9

D.2 Wind load

To determine the design load on the load bearing system of the building the Eurocode approach has been used. As described in Eurocode 1 section 5.3 the governing equation is given by Equation 31.

$$F_w = c_s c_d \cdot \sum_{elements} c_f \cdot q_p(z_e) \cdot A_{ref} \quad [\mathrm{kN}] \tag{31}$$

Where in:

F_w	=	Wind force on structure	*to be determined*
$c_s c_d$	=	Structural factor	Assume 1
c_f	=	Force coefficient	$(+0.8) - (-0.7) = 1.5$
$q_p(z_e)$	=	Peak velocity pressure	subsection D.2.1
A_{ref}	=	Reference area on structure	subsection D.2.2

D.2.1 *Peak velocity pressure*

The building is located in location area II in a rural area. For this location the following peak velocity's have to be considered.

- Reference height equals width building; $q_p(27) = 1.16\,[Kn/m^2]$
- Reference height equals hight building; $q_p(105) = 1.66\,[Kn/m^2]$

D.2.2 *Reference area*

The reference area of the vertical facade can be divided in the area's as described in Figure 61 . This division is as described in Eurocode 1 section 7.2.2.

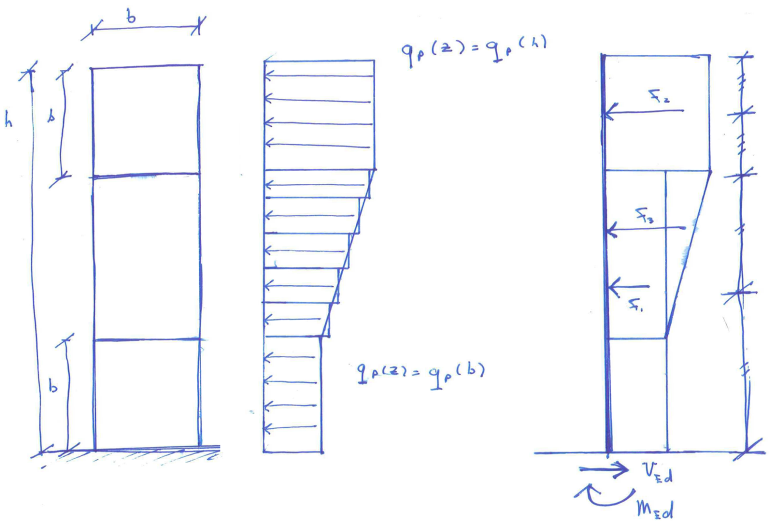


Figure 61: Reference area

E STRUCTURAL MODEL

In this appendix the Scia engineer model and validations of this model with hand calculations will be presented.

E.1 THE MODEL

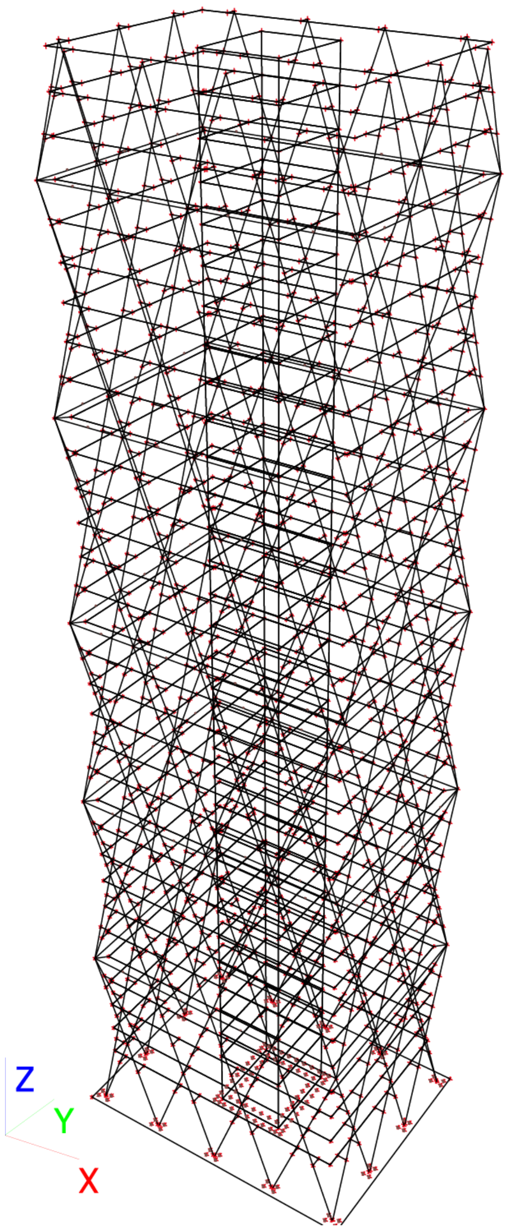


Figure 62: Overview model

E.1.1 *Materials*

Young's modulus of concrete has been reduced in order to represent cracked concrete.

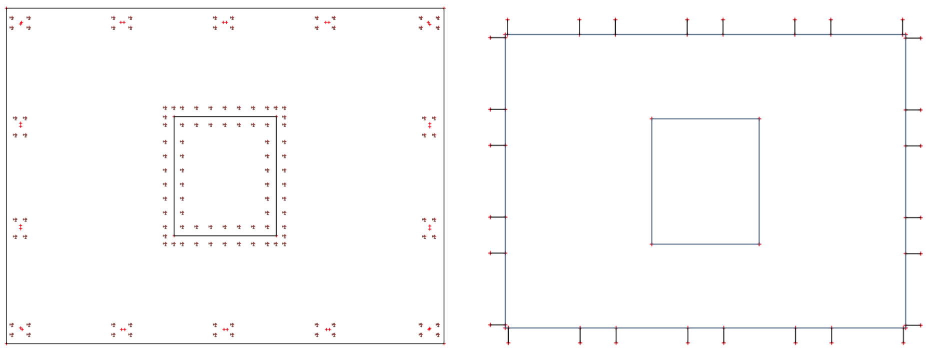
ELEMENT	STRENGTH CLASS	YOUNG'S MODULUS	[MPA]
Façade columns	C90/105	$4.4 \cdot 10^4$	$3.0 \cdot 10^4$
Core	C40/50	$3.5 \cdot 10^4$	$2.0 \cdot 10^4$
Floor slabs	C40/50	$3.5 \cdot 10^4$	$2.0 \cdot 10^4$
Foundation slab	C30/37	$3.3 \cdot 10^4$	–
Tension bar	S355	$2.1 \cdot 10^5$	–

E.1.2 *Foundation slab*

The foundations slab is made up of a 2000 [mm] thick slab which is founded upon springs which represent foundation piles. The springs have a stiffens in z-direction of $2.0 \cdot 10^2$ [mN/m]. The core is founded upon 76 piles with a minimum spacing of 1000 [mm]. The façade columns are founded upon two piles each, resulting in a total of 54 piles in the façade perimete. See Figure 63 for foundation plan.

E.1.3 *Storey floors*

The storey floors are build up out of plate elements with a thickness of 300 [mm]. The plates are connected with the façade columns through steel beam elements (IPE360). In order to represent floor beams the steel beam elements have moment resistance connections with the floor plate and hinged with the façade columns.



(a) Foundation slab (b) Storey floor slab

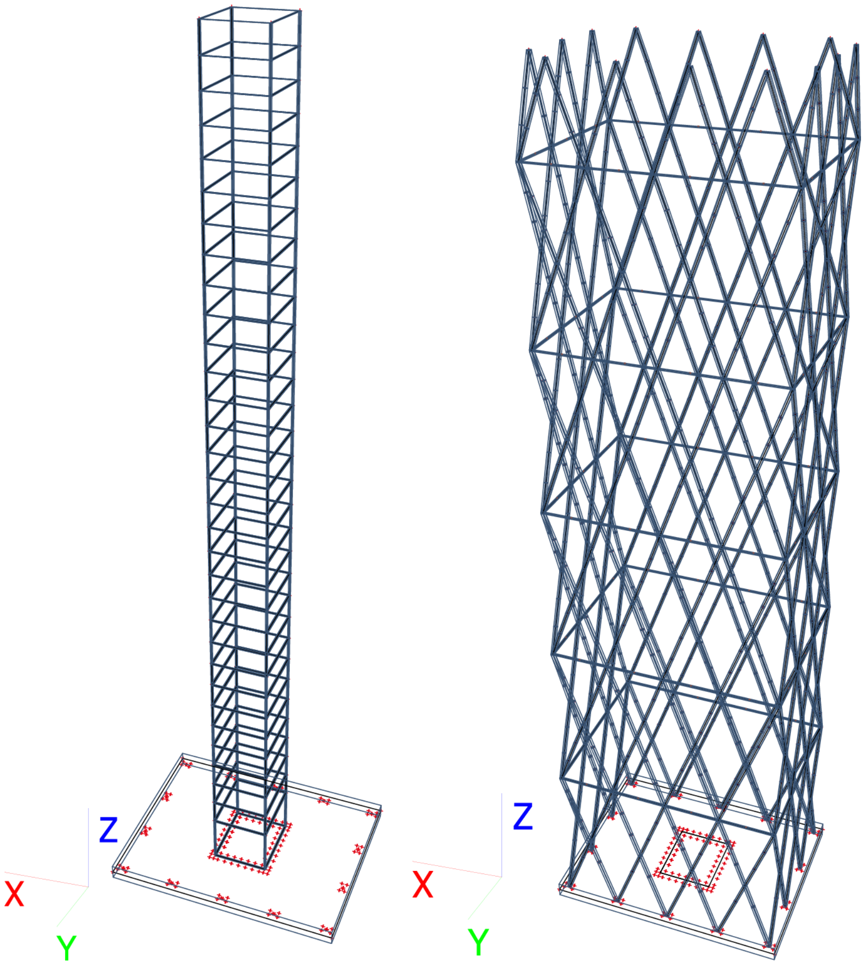
Figure 63: Structural slabs

E.1.4 *Stability core*

The core of the building is build up out of walls with a thickness of 300 [mm]. This differs from the design drawings where a wall thickness of 400 [mm] for the core walls is drawn. The reduced width of these walls in this model represent holes and openings which are not modelled.

E.1.5 *Façade*

The façade columns are made of square elements of 360 x 360 [mm]. The tension bar is made out of a rectangular element of 280 x 100 [mm].



(a) Stability core (b) Façade

Figure 64: Structural model

E.2 LOAD AT THE STRUCTURE

E.2.1 *Load cases*

	LOAD	MAGNITUDE	AXIS	LOCATION
G_{sw}	Self weight	a	Z	all elements
G_{w1}	Permanent weight	$-0.8[kN/m^2]$	Z	storey floors
G_{w2}	Permanent weight	$-5.94[kN/m]$	Z	all floor edges
Q_{fl}	Variable floor load	$-3.[kN/m^2]$	Z	storey floors
Q_{wl}	Wind load 1	b	Y	E, W floor edge
Q_{w2}	Wind load 2	b	Y	N, S floor edge
Q_{t1}	Thermal load	d 35[K]	-	S façade
Q_{t2}	Thermal load	d 40[K]	-	S, W façade
Q_{t3}	Thermal load	d -36[K]	-	all façades

note: N, E, S & W stands for respectively north, east, south and west.
(A): Self weight is produced by Scia Engineer according material and element properties.
(B): Wind load is determined according Eurocode see Figure 61.

E.2.2 *Load combinations*

The following sixteen load combinations are considered. The combinations are considered for both a ULS and SLS calculations.

1. Maximum floor load.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot Q_{f1}$

2. Wind load1.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot Q_{f1} + \gamma_Q \cdot Q_{w1}$

3. Wind load2.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot Q_{f1} + \gamma_Q \cdot Q_{w2}$

4. Thermal load south façade.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot Q_{f1} + \gamma_Q \cdot Q_{t1}$

5. Thermal load south, west façade.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot Q_{f1} + \gamma_Q \cdot Q_{t2}$

6. Thermal load all façades.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot Q_{f1} + \gamma_Q \cdot Q_{t3}$

7. Minimum floor load.
$\gamma_{G,min} \cdot (G_{sw} + G_{w1} + G_{w2})$

8. Minimum floor load plus wind load1.
$\gamma_{G,min} \cdot (G_{sw} + G_{w1} + G_{w2})\gamma_Q \cdot Q_{w1}$

9. Minimum floor load plus wind load2.
$\gamma_{G,min} \cdot (G_{sw} + G_{w1} + G_{w2})\gamma_Q \cdot Q_{w2}$

10. Minimum floor load plus thermal load south façade.
$\gamma_{G,min} \cdot (G_{sw} + G_{w1} + G_{w2})\gamma_Q \cdot Q_{t1}$

11. Minimum floor load plus thermal load south, west façade.
$\gamma_{G,min} \cdot (G_{sw} + G_{w1} + G_{w2})\gamma_Q \cdot Q_{t2}$

12. Minimum floor load plus thermal load all façades.
$\gamma_{G,min} \cdot (G_{sw} + G_{w1} + G_{w2})\gamma_Q \cdot Q_{t3}$

13. Wind load1 plus thermal load south, west façade.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot (Q_{f1} + Qt2) + \gamma_Q \cdot Q_{w1}$

14. Wind load1 plus thermal load all façades.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot (Q_{f1} + Qt3) + \gamma_Q \cdot Q_{w1}$

15. Wind load2 plus thermal load south, west façade.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot (Q_{f1} + Qt2) + \gamma_Q \cdot Q_{w2}$

16. Wind load2 plus thermal load all façades.
$\gamma_G \cdot (G_{sw} + G_{w1} + G_{w2}) + \gamma_Q \cdot \psi_0 \cdot (Q_{f1} + Qt3) + \gamma_Q \cdot Q_{w2}$

For the ULS calculation the following safety factores and combination factor has been considered:

Negative permanent loading	γ_G	1.32
Positive permanent loading	$\gamma_{G,min}$	0.9
Negative variable loading	γ_Q	1.65
Load combination factor	ψ_0	0.5

E.3 RESULTS

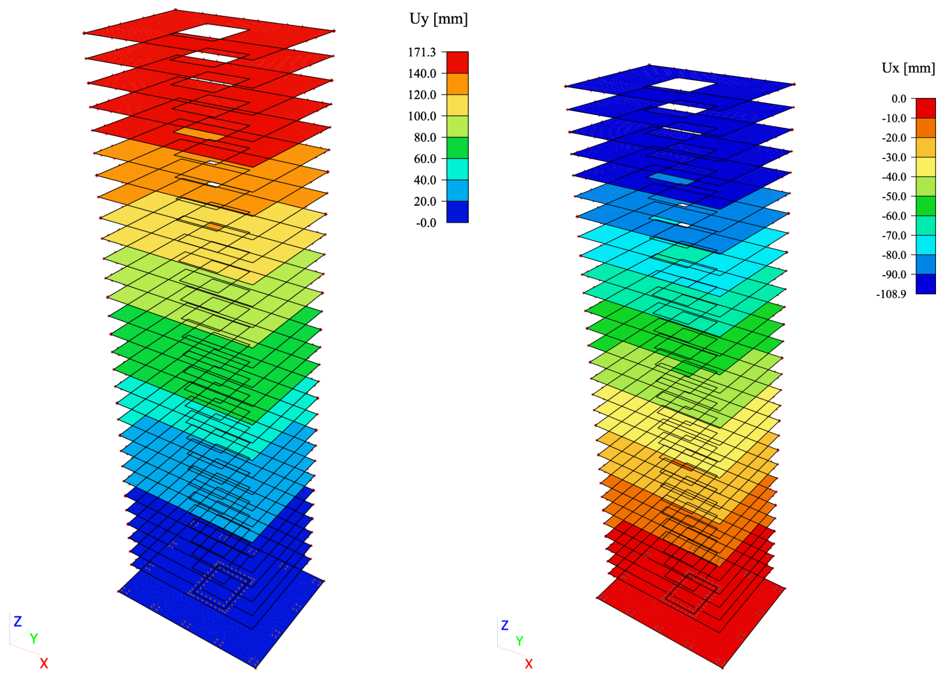
E.3.1 *Displacements*

E.3.1.1 *Displacement in y-direction*

The maximum displacement for a SLS combination in y-direction is found at load combination 13. Wind load1 plus thermal load south, west façade. The maximum displacement in this SLS combination is 218 [mm] this is value is below the EC2 SLS limit of (height/500) 183 [mm].

E.3.1.2 *Displacement in x-direction*

The maximum displacement for a SLS combination in x-direction is found at load combination 15. Wind load2 plus thermal load south, west façade. The maximum displacement of 108.9 [mm] is aswel below the EC2 SLS limit of (height/500) 218 [mm].



(a) y-direction (b) x-direction

Figure 65: Displacement

E.3.2 *Maximum loads*

E.3.2.1 *Ground floor columns*

The maximum forces at the ground floor columns are presented in Figure 66.

E.3.2.2 *Ground floor columns*

The maximum forces at the tension member are presented in Figure 67.

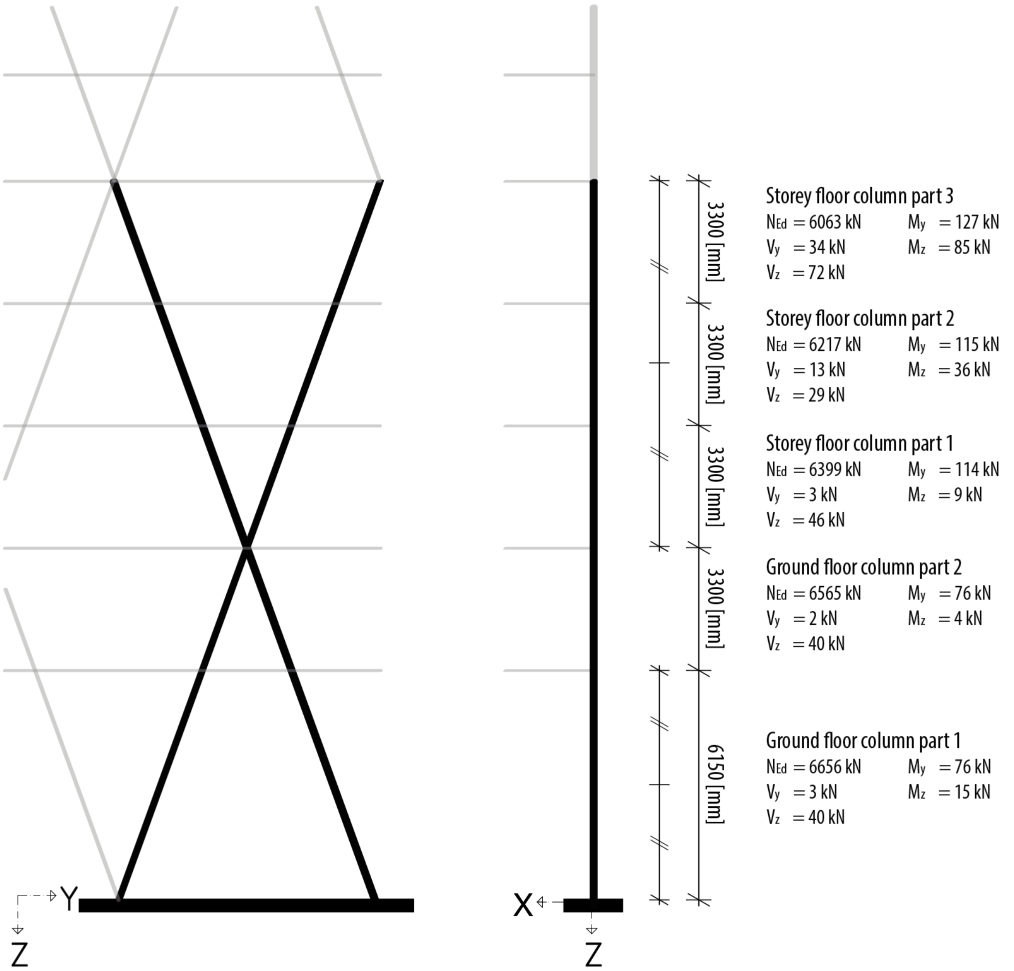


Figure 66: Maximum forces ground floor columns

E.3.3 *Force distributions*

The typical load distribution of normal forces through the diagrid structure are presented in Figure 69.

E.3.3.1 *Normal forces*

A typical normal-, shear, and moment force distribution at the ground columns are presented in Figure 70.

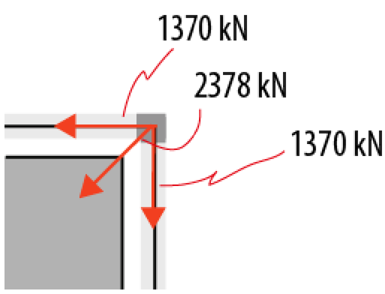


Figure 67: Maximum forces tensile member

E.4 VERIFICATIONS WITH HAND CALCULATIONS

E.4.1 *Total weight*

To check the total weight of the building the vertical reaction forces of the foundations springs, in cases of load combination 1, are added up. This value is than compared with the total weight of the building, under the same load conditions, derived with a hand calculation. As the results of the table below show, there is no significant difference between the model and the hand calculation.

	TOTAL WEIGHT	PERCENTAGE
Hand calculation	$3.1 \cdot 10^5$ [kN]	100 [%]
Scia Engineer	$3.26 \cdot 10^5$ [kN]	105.2 [%]

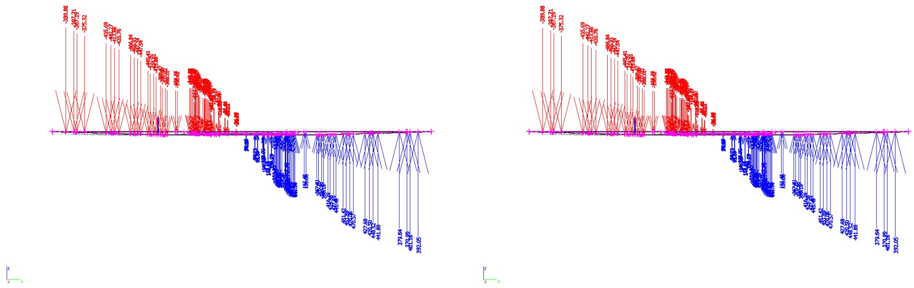
E.4.2 *Wind load*

To check the wind load the vertical reaction forces of the foundations springs, under the load case 'wind load 1' and 'wind load 2', are used to determine the developing moment at the foundation slab. This value is compared with the developing moment at the foundation slab as result of the corresponding wind load. As the results of subsection E.4.2 show, there is no significant difference between the model and the hand calculation.

	LOADCASE	MOMENT	PERCENTAGE
Hand calculation	WL1	$3.28 \cdot 10^5$ [kNm]	100 [%]
	WL2	$2.41 \cdot 10^5$ [kNm]	100 [%]
Scia Engineer	WL1	$3.16 \cdot 10^5$ [kNm]	96.3 [%]
	WL2	$2.33 \cdot 10^5$ [kNm]	96.7 [%]

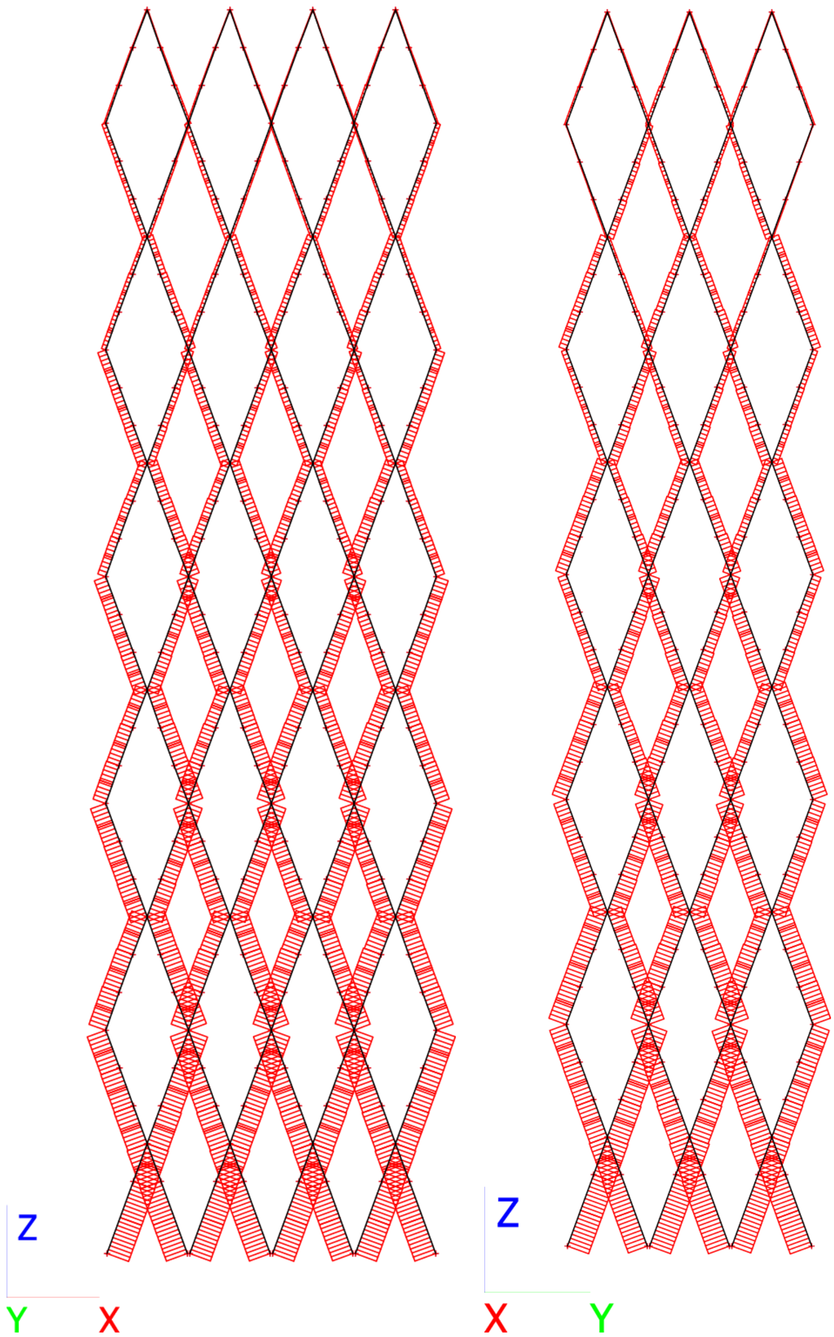
E.4.2.1 *Distribution between core and façade*

With the total developed moment and the accompanying normal forces in the façade columns the distribution of the moment between the core and the façade can be calculated. At load case 'wind load 1' the façade takes on 30 [%] of the developed moment. At load case 'wind load 2' the façade takes on 42 [%]. Regarding the dimensions of the floorplan and the difference in stiffness between the core and the façade this difference seams reasonable.



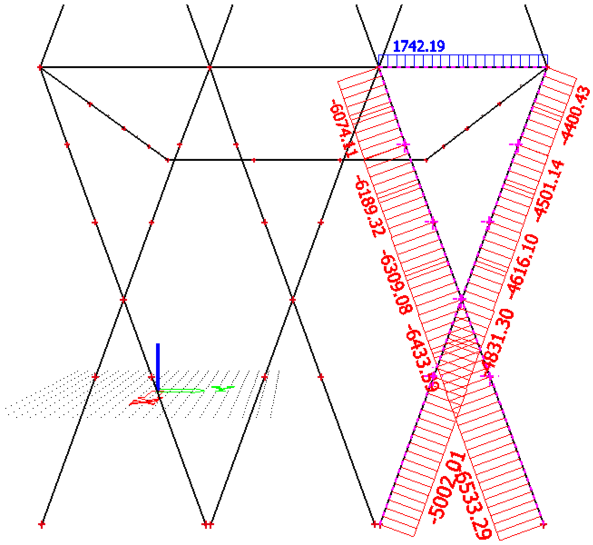
(a) load case 'wind load 1'
(b) load case 'wind load 1

Figure 68: Foundation reaction under wind load

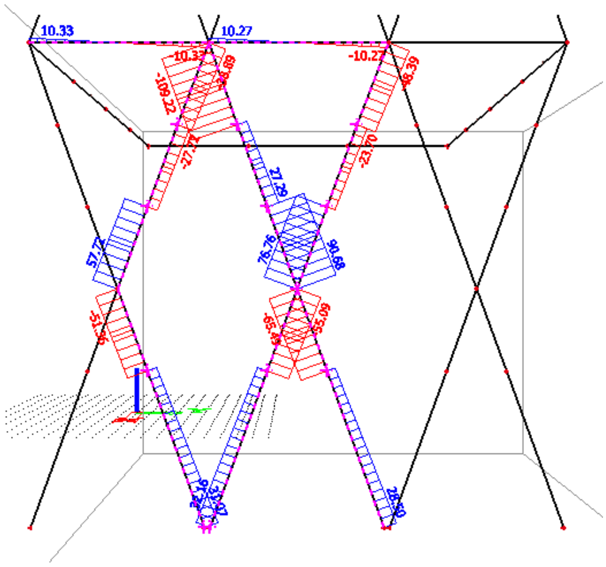


(a) West façade
(b) North façade

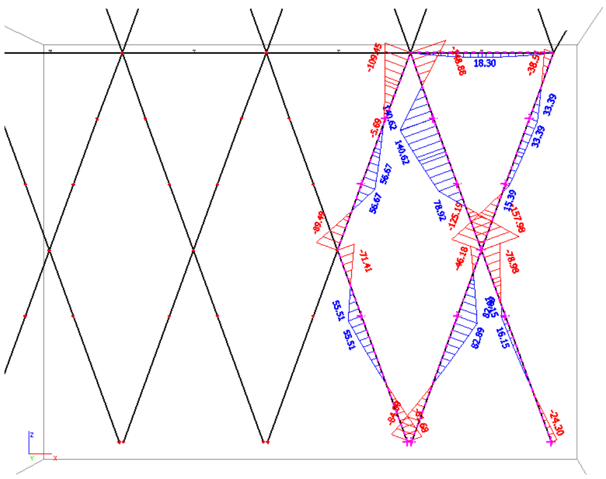
Figure 69: Force distribution under load combination 1



(a) Normal force



(b) Shear force



(c) Moment force

Figure 70: Force distribution ground floor columns

F DESIGN OF STRUCTURAL MEMBERS

F.1 CHARACTERISTIC POINT: P1

Architectural demand to enable as much exposure as possible for the retail space located at the ground floor level of the building.

The following rectangle section has been designed considering both architectural and structural requirements: $h = 500\ [mm]$, $b = 250\ [mm]$. Resulting in the following section properties: $A = 12.5 \cdot 10^4\ [mm]$, $I_y = 6.5 \cdot 10^8\ [mm^4]$ & $I_z = 26.5 \cdot 10^8\ [mm^4]$.

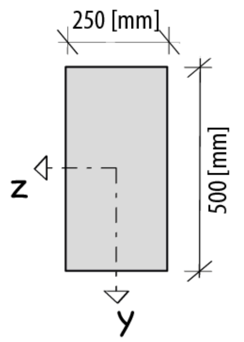


Figure 71: Design of the section at P1 (local coordinate system)

F.1.1 *Normative loads*

The following normative loads act at the considered section. These values are extracted from the structural model, see Appendix E.

Normal force	N_{Ed}	=	6063 [kN]
Shear force y-direction	V_y	=	2 [kN]
Shear force z-direction	V_y	=	21 [kN]
Imposed moment around y-axis	M_y	=	61 [kNm]
Imposed moment around z-axis	M_z	=	21 [kNm]

F.1.2 *Second order effects*

With the use of the calculation method described in Appendix C, the second order effect and required reinforcement has been determined.

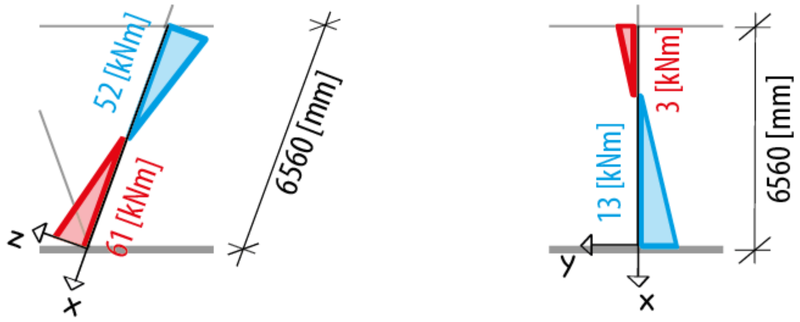


Figure 72: Moment line ground floor column (local coordinate system)

Z-DIRECTION				
Buckling length	$l_{buc,z}$	=	N/A	[mm]
First order eccentricity	e_i	=	16.4	[mm]
Second order eccentricity	e_2	=	N/A	[mm]
Design value moment force	$M_{Ed,y}$	=	160	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.04	[–]
Moment force stress	M_{Ed}/W_y	=	31	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,y}$	=	N/A	[mm]
First order eccentricity	e_i	=	10.9	[mm]
Second order eccentricity	e_2	=	N/A	[mm]
Design value moment force	$M_{Ed,z}$	=	79	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.01	[–]
Moment force stress	M_{Ed}/W_z	=	8	[MPa]
Normal force factor$_{(A)}$	N_{factor}	=	0.42	[–]
Normal force stress	N_{Ed}/A	=	48.5	[MPa]
Required reinforcement x-direction	R_x	=	0	[%]
Required reinforcement y-direction	R_y	=	0	[%]

(A) See Equation 20

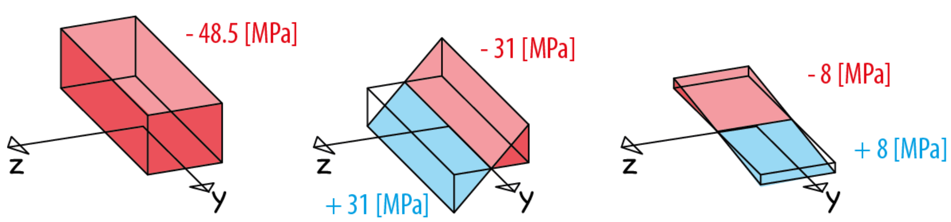


Figure 73: Visualisation stress distribution in section P1, normal stress [left], M_y stress [centre], M_z stress [right]

F.2 CHARACTERISTIC POINT: P2

Architectural and structural demand to enable a slender appearance which is adapted to the sensitivity for buckling of this point.

The following rectangle section has been designed considering both architectural and structural requirements: $h = 420\,mm$, $b = 420\,mm$. Resulting in the following section properties: $A = 17.6 \cdot 10^4\ mm$, $I_y = 25.9 \cdot 10^8\ mm^4$, $I_z = 25.9 \cdot 10^8\ mm^4$.

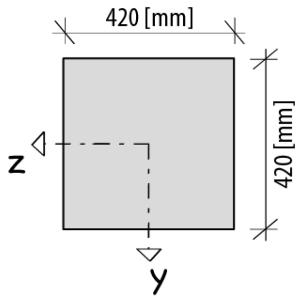


Figure 74: Design of the section at P2 (local coordinate system)

F.2.1 *Normative loads*

The following normative loads act at the considered section. These values are extracted from the structural model, see Appendix E.

Normal force	N_{Ed}	=	6063 [kN]
Shear force y-direction	V_y	=	2 [kN]
Shear force z-direction	V_y	=	21 [kN]
Imposed moment around y-axis	M_y	=	61 [kNm]
Imposed moment around z-axis	M_z	=	21 [kNm]

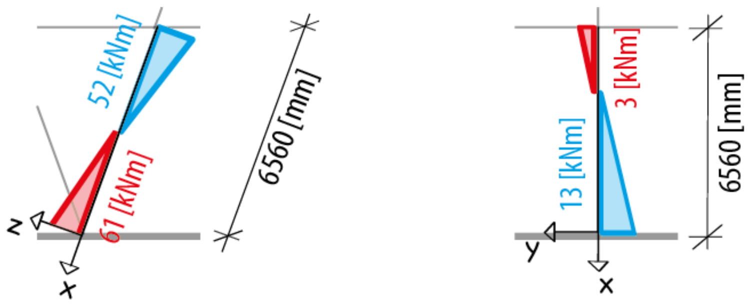


Figure 75: Moment line at ground floor column part 2 (local coordinate system)

F.2.2 *Second order effects*

With the use of the calculation method described in Appendix C, the second order effect and required reinforcement ratio has been determined.

Z-DIRECTION				
Buckling length $(0.95 \cdot l_{syst})$	$l_{buc,y}$	=	6232	[mm]
First order eccentricity	e_i	=	10.4	[mm]
Second order eccentricity	e_2	=	48.8	[mm]
Design value moment force	$M_{Ed,y}$	=	395	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.05	[–]
Moment force stress	M_{Ed}/W_y	=	32.0	[MPa]
Y-DIRECTION				
Buckling length $(0.7 \cdot l_{syst})$	$l_{buc,x}$	=	4592	[mm]
First order eccentricity	e_i	=	7.7	[mm]
Second order eccentricity	e_2	=	27.7	[mm]
Design value moment force	$M_{Ed,x}$	=	222	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.03	[–]
Moment force stress	M_{Ed}/W_z	=	18	[MPa]

Normal force factor$_{(A)}$	N_{factor}	=	0.30	[–]
Normal force stress	N_{Ed}/A	=	34.4	[MPa]
Required reinforcement x-direction	R_x	=	0	[%]
Required reinforcement y-direction	R_y	=	0	[%]

(A) See Equation 20

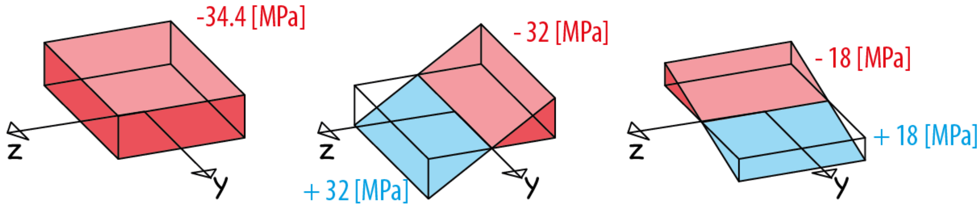


Figure 76: Visualisation stress distribution in section P2, normal stress [left], M_y stress [centre], M_z stress [right]

F.3 CHARACTERISTIC POINT: P3

Architectural and structural demand to enable a smooth transition to point P4 and resist the high imposed moments.

The following rectangle section has been designed considering both architectural and structural requirements: $h = 250\,mm$, $b = 550\,mm$. Resulting in the following section properties: $A = 13.8 \cdot 10^4\ mm$, $I_y = 347 \cdot 10^7 mm^4$ & $I_z = 72 \cdot 10^7\ mm^4$.

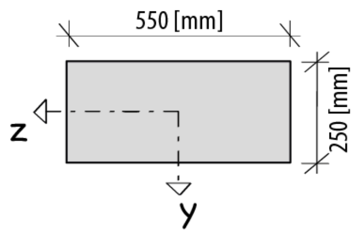


Figure 77: Design of the section at P3 (local coordinate system)

F.3.1 *Normative loads*

The following normative loads act at the considered section. These values are extracted from the structural model, see Appendix E.

Normal force	N_{Ed}	=	5981 [kN]
Shear force y-direction	V_y	=	1 [kN]
Shear force z-direction	V_y	=	38 [kN]
Imposed moment around y-axis	M_y	=	120 [kNm]
Imposed moment around z-axis	M_z	=	67 [kNm]

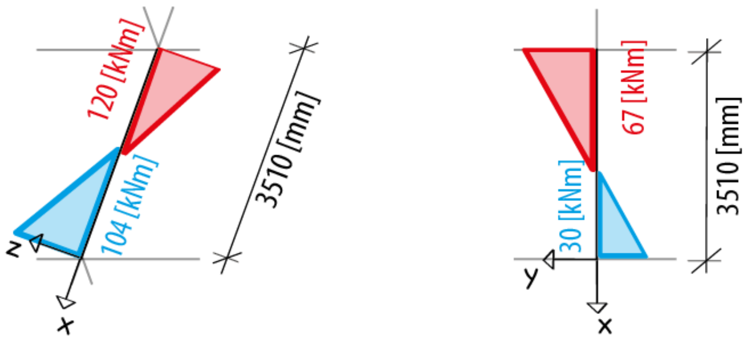


Figure 78: Moment line at ground floor column part 2 (local coordinate system)

F.3.2 *Second order effects*

With the use of the calculation method described in Appendix C, the second order effect and required reinforcement ratio has been determined.

Z-DIRECTION				
Buckling length	$l_{buc,y}$	=	N/A	[mm]
First order eccentricity	e_i	=	8.8	[mm]
Second order eccentricity	e_2	=	N/A	[mm]
Design value moment force	$M_{Ed,y}$	=	173	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.02	[–]
Moment force stress	M_{Ed}/W_y	=	14.0	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,x}$	=	N/A	[mm]
First order eccentricity	e_i	=	5.9	[mm]
Second order eccentricity	e_2	=	N/A	[mm]
Design value moment force	$M_{Ed,x}$	=	102	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.01	[–]
Moment force stress	M_{Ed}/W_z	=	8.3	[MPa]

Normal force factor$_{(A)}$	N_{factor}	=	0.30	[–]
Normal force stress	N_{Ed}/A	=	34.4	[MPa]
Required reinforcement x-direction	R_x	=	0	[%]
Required reinforcement y-direction	R_y	=	0	[%]

(A) See Equation 20

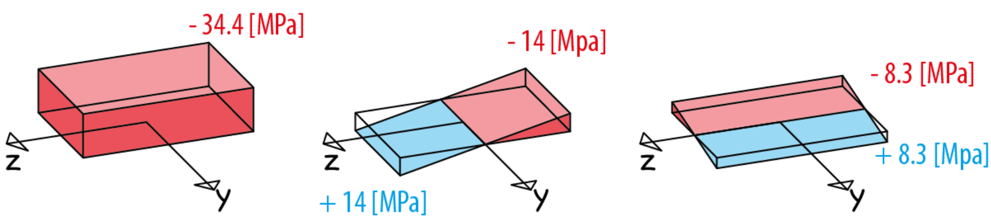


Figure 79: Visualisation stress distribution in section P3, normal stress [left], M_y stress [centre], M_z stress [right]

F.4 CHARACTERISTIC POINT: P4

Enable a slender appearance which is adapted to the sensitivity for buckling of this point.

The following rectangle section has been designed considering both architectural and structural requirements: $h = 340$ [mm], $b = 440$ [mm]. Resulting in the following section properties: $A = 15 \cdot 10^4$ [mm], $I_y = 24.1 \cdot 10^8 [mm^4]$ & $I_z = 14.4 \cdot 10^8$ [mm^4].

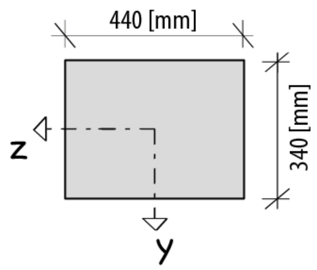


Figure 80: Design of the section at P4 (local coordinate system)

F.4.1 *Normative loads*

The following normative loads act at the considered section. These values are extracted from the structural model, see Appendix E.

Normal force	N_{Ed}	=	5659 [kN]
Shear force y-direction	V_y	=	9 [kN]
Shear force z-direction	V_y	=	26 [kN]
Imposed moment around y-axis	M_y	=	104 [kNm]
Imposed moment around z-axis	M_z	=	27 [kNm]

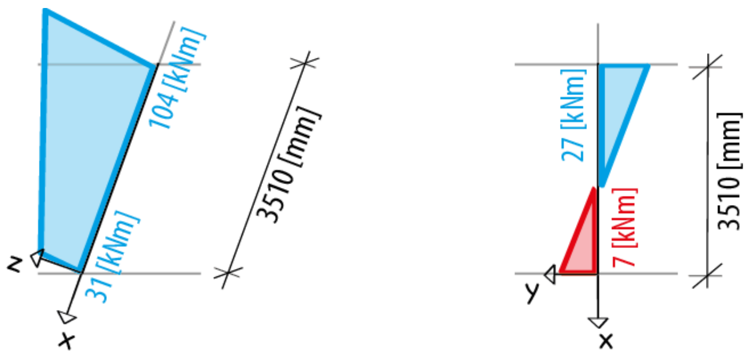


Figure 81: Moment line at ground floor column part 2 (local coordinate system)

F.4.2 *Second order effects*

With the use of the calculation method described in Appendix C, the second order effect and required reinforcement ratio has been determined.

Z-DIRECTION				
Buckling length $(1.5 \cdot l_{syst})$	$l_{buc,y}$	=	5265	[mm]
First order eccentricity	e_i	=	8.8	[mm]
Second order eccentricity	e_2	=	30.7.	[mm]
Design value moment force	$M_{Ed,y}$	=	298	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.04	[–]
Moment force stress	M_{Ed}/W_y	=	27.0	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,x}$	=	3510	[mm]
First order eccentricity	e_i	=	5.9	[mm]
Second order eccentricity	e_2	=	18.4	[mm]
Design value moment force	$M_{Ed,x}$	=	154	[kNm]
Moment force factor$_{(A)}$	M_{factor}	=	0.03	[–]
Moment force stress	M_{Ed}/W_z	=	18.1	[MPa]

Normal force factor$_{(A)}$	N_{factor}	=	0.33	[–]
Normal force stress	N_{Ed}/A	=	37.8	[MPa]
Required reinforcement x-direction	R_x	=	0	[%]
Required reinforcement y-direction	R_y	=	0	[%]

(A) See Equation 20

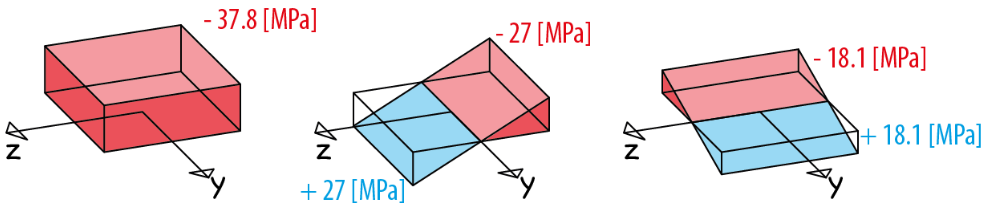


Figure 82: Visualisation stress distribution in section P4, normal stress [left], M_y stress [centre], M_z stress [right]

F.5 CHARACTERISTIC POINT: P5

Architectural and structural demand to enable a panoramic view from within the building and resist the stress concentration due the merging of members.

The following rectangle section has been designed considering both architectural and structural requirements: $h = 250$ [mm], $b = 1100$ [mm]. Resulting in the following section properties: $A = 27.5 \cdot 10^4$ [mm], $I_y = 277.3 \cdot 10^8$[mm^4] & $I_z = 14.3 \cdot 10^8$ mm^4.

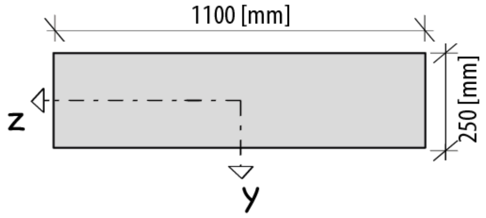


Figure 83: Design of the section at P5 (local coordinate system)

F.5.0.1 *Normative loads*

The following normative loads act at the considered section. These values are extracted from the structural model, see Appendix E.

Normal force	N_{Ed}	=	11660 [kN]
Shear force y-direction	V_y	=	4 [kN]
Shear force z-direction	V_y	=	90 [kN]
Imposed moment around y-axis	M_y	=	203 [kNm]
Imposed moment around z-axis	M_z	=	12 [kNm]

The shape of the cross-sections at the nodes is guided by the normal force factor. As the normal force at the nodes is twice as high as the force at the members the section area doubles. Second order effects are not expected at the nodes and the imposed moments at the nodes are considerable lower than the moments imposed at the members. Therefor the effects of moment forces are not considered for the design of the cross-sections shapes.

Normal force factor$_{(A)}$	N_{factor}	=	0.30	[–]
Normal force stress	N_{Ed}/A	=	42.4	[MPa]

(A) See Equation 20

F.6 CHARACTERISTIC POINT: P6

Architectural and structural demand to enable a panoramic view from within the building and resist the stress concentration due the merging of members out different planes.

The following rectangle section has been designed considering both architectural and structural requirements: $h = 250\,[mm]$, $b = 850\,mm$. Resulting in the following section properties: $A = 21.25 \cdot 10^4\,[mm]$, $I_y = 127.9 \cdot 10^8 [mm^4]$ & $I_z = 11.1 \cdot 10^8\,[mm^4]$.

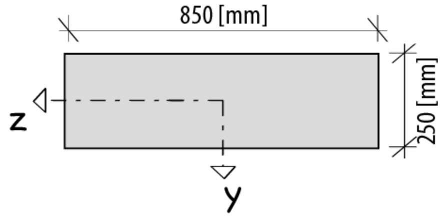


Figure 84: Design of the section at P6 (local coordinate system)

F.6.0.2 *Normative loads*

The following normative loads act at the considered section. These values are extracted from the structural model, see Appendix E.

Normal force	N_{Ed}	=	5086 [kN]
Shear force y-direction	V_y	=	25 [kN]
Shear force z-direction	V_y	=	25 [kN]
Imposed moment around y-axis	M_y	=	108 [kNm]
Imposed moment around z-axis	M_z	=	108 [kNm]

The shape of the cross-sections at the nodes is guided by the normal force factor. As the normal force at the nodes is twice as high as the force at the members the section area doubles. Second order effects are not expected at the nodes and the imposed moments at the nodes are considerable lower than the moments imposed at the members. Therefor the effects of moment forces are not considered for the design of the cross-sections shapes.

Normal force factor$_{(A)}$	N_{factor}	=	0.21	[–]
Normal force stress	N_{Ed}/A	=	23.9	[MPa]

(A) See Equation 20

G

STRUCTURAL CALCULATIONS DIAGRID ELEMENTS

G.1 ULS VERIFICATIONS SECTION P1

With the use of the calculation method described in Appendix C, the second order effect and the ULS stress distribution at the cross-section plane have been determined.

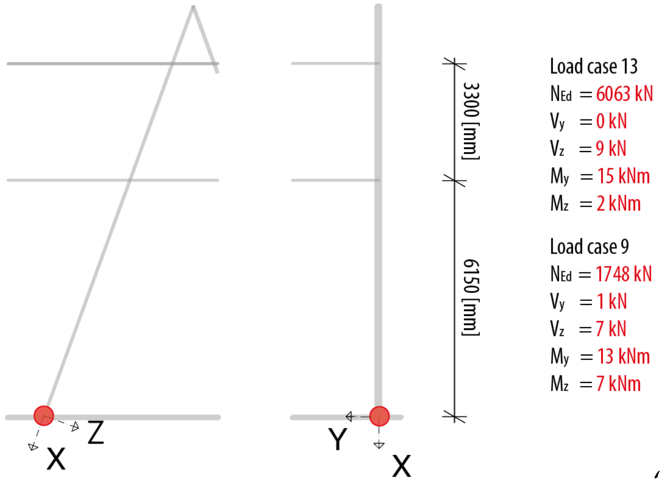


Figure 85: Normative load cases at section P1 (Local coordinate system)

Z-DIRECTION				
Buckling length	$l_{buc,y}$	=	N/A.	[mm]
First order eccentricity	e_i	=	20	[mm]
Design value moment force, load case 13	$M_{Ed,y}$	=	136	[kNm]
Design value moment force, load case 9	$M_{Ed,y}$	=	48	[kNm]
Moment force stress, load case 13	M_{Ed}/W_y	=	26	[MPa]
Moment force stress, load case 9	M_{Ed}/W_y	=	9.2	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,x}$	=	N/A	[mm]
First order eccentricity	e_i	=	20	[mm]
Design value moment force, load case 13	$M_{Ed,x}$	=	123	[kNm]
Design value moment force, load case 9	$M_{Ed,x}$	=	42	[kNm]
Moment force stress, load case 13	M_{Ed}/W_z	=	11.8	[MPa]
Moment force stress, load case 9	M_{Ed}/W_z	=	4	[MPa]

Normal force stress, load case 13	N_{Ed}/A	=	48.5	[MPa]
Normal force stress, load case 9	N_{Ed}/A	=	14	[MPa]

G.2 ULS VERIFICATIONS SECTION P2

With the use of the calculation method described in Appendix C, the second order effect and occurring stress have been determined.

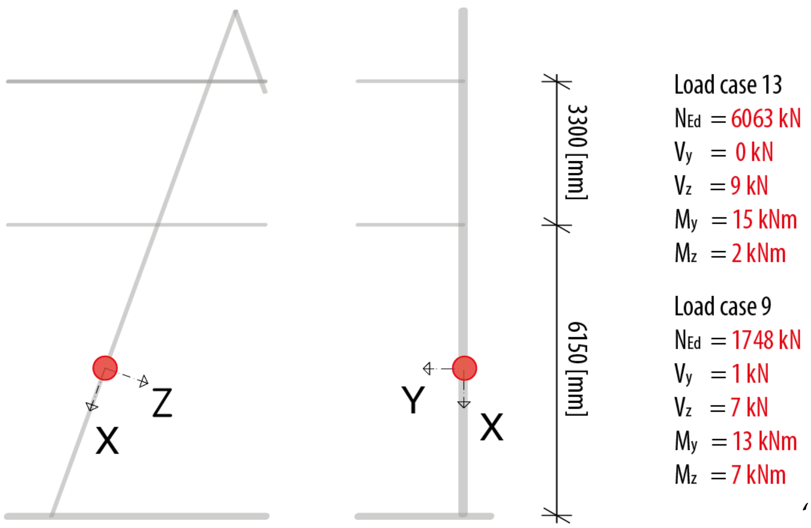


Figure 86: Normative load cases at section P2 (Local coordinate system)

Z-DIRECTION				
Buckling length $(0.95 \cdot l_{syst})$	$l_{buc,y}$	=	6216	[mm]
First order eccentricity	e_i	=	20	[mm]
Second order eccentricity, load case 13	e_2	=	48.5	[mm]
Second order eccentricity,load case 9	e_2	=	64.2	[mm]
Design value moment force, load case 13	$M_{Ed,y}$	=	424.7	[kNm]
Design value moment force, load case 9	$M_{Ed,y}$	=	151	[kNm]
Moment force stress, load case 13	M_{Ed}/W_y	=	34.4	[MPa]
Moment force stress, load case 9	M_{Ed}/W_y	=	12.3	[MPa]

Y-DIRECTION				
Buckling length $(0.7 \cdot l_{syst})$	$l_{buc,x}$	=	4580	[mm]
First order eccentricity	e_i	=	20	[mm]
Second order eccentricity, load case 13	e_2	=	27.6	[mm]
Second order eccentricity, load case 9	e_2	=	36.5	[mm]
Design value moment force, load case 13	$M_{Ed,x}$	=	290	[kNm]
Design value moment force, load case 9	$M_{Ed,x}$	=	103.0	[kNm]
Moment force stress, load case 13	M_{Ed}/W_z	=	23	[MPa]
Moment force stress, load case 9	M_{Ed}/W_z	=	8.3	[MPa]

Normal force stress, load case 13	N_{Ed}/A	=	34.4	[MPa]
Normal force stress, load case 9	N_{Ed}/A	=	9.9	[MPa]

G.3 ULS VERIFICATIONS SECTION P3

With the use of the calculation method described in Appendix C, the second order effect and occurring stress have been determined.

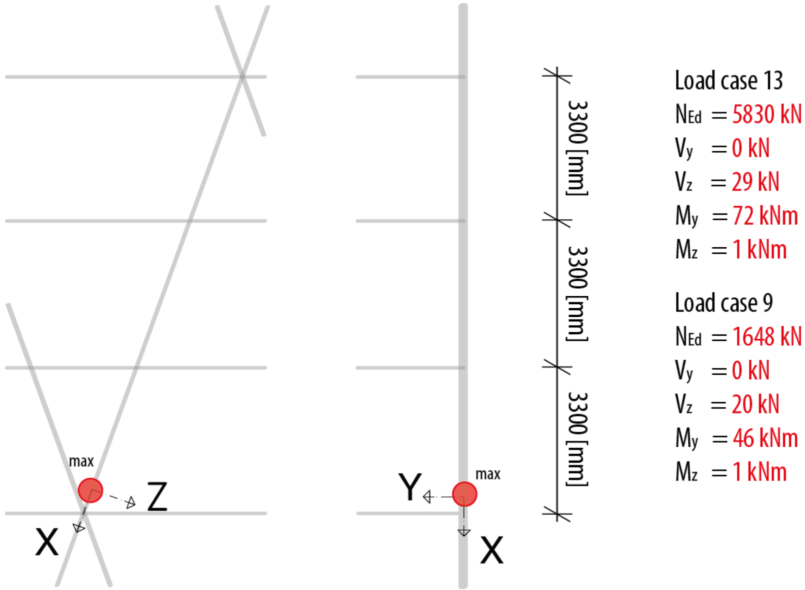


Figure 87: Normative load cases at section P3 (Local coordinate system)

Z-DIRECTION				
Buckling length	$l_{buc,y}$	=	N/A	[mm]
First order eccentricity	e_i	=	20	[mm]
Design value moment force, load case 13	$M_{Ed,y}$	=	188.6	[kNm]
Design value moment force, load case 9	$M_{Ed,y}$	=	79	[kNm]
Moment force stress, load case 13	M_{Ed}/W_y	=	15	[MPa]
Moment force stress, load case 9	M_{Ed}/W_y	=	6.3	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,x}$	=	N/A	[mm]
First order eccentricity	e_i	=	20	[mm]
Design value moment force, load case 13	$M_{Ed,x}$	=	117.6	[kNm]
Design value moment force, load case 9	$M_{Ed,x}$	=	34	[kNm]
Moment force stress, load case 13	M_{Ed}/W_z	=	20.5	[MPa]
Moment force stress, load case 9	M_{Ed}/W_z	=	5.9	[MPa]

Normal force stress, load case 13	N_{Ed}/A	=	42.4	[MPa]
Normal force stress, load case 9	N_{Ed}/A	=	12	[MPa]

G.4 ULS VERIFICATIONS SECTION P4

With the use of the calculation method described in Appendix C, the second order effect and occurring stress have been determined.

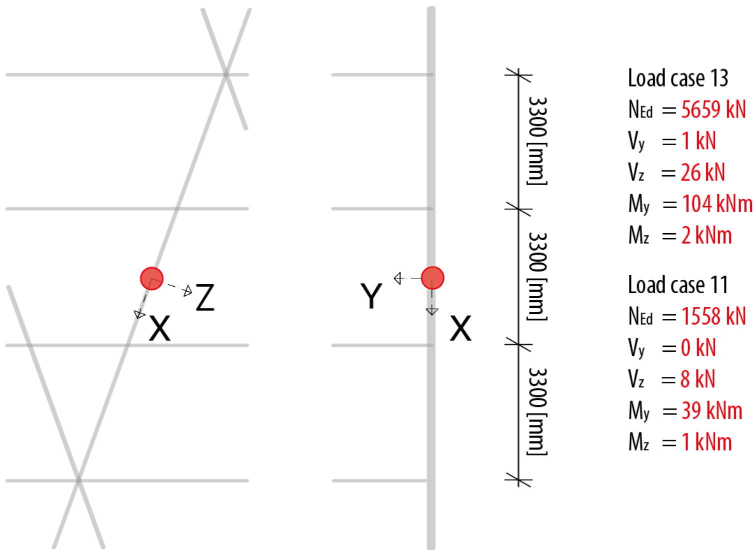


Figure 88: Normative load cases at section P4 (Local coordinate system)

Z-DIRECTION				
Buckling length	$l_{buc,y}$	=	5265	[mm]
First order eccentricity	e_i	=	20	[mm]
Second order eccentricity, load case 13	e_2	=	30.6	[mm]
Second order eccentricity, load case 9	e_2	=	45.4	[mm]
Design value moment force, load case 13	$M_{Ed,y}$	=	349	[kNm]
Design value moment force, load case 9	$M_{Ed,y}$	=	125	[kNm]
Moment force stress, load case 13	M_{Ed}/W_y	=	31.8	[MPa]
Moment force stress, load case 9	M_{Ed}/W_y	=	11.4	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,x}$	=	3510	[mm]
First order eccentricity	e_i	=	20	[mm]
Second order eccentricity, load case 13	e_2	=	18.4	[mm]
Second order eccentricity, load case 9	e_2	=	27.2	[mm]
Design value moment force, load case 13	$M_{Ed,x}$	=	218	[kNm]
Design value moment force, load case 9	$M_{Ed,x}$	=	74	[kNm]
Moment force stress, load case 13	M_{Ed}/W_z	=	25.7	[MPa]
Moment force stress, load case 9	M_{Ed}/W_z	=	8.7	[MPa]

Normal force stress, load case 13	N_{Ed}/A	=	37.8	[MPa]
Normal force stress, load case 9	N_{Ed}/A	=	10.4	[MPa]

G.5 ULS VERIFICATIONS MAXIMUM TENSION

With the use of the calculation method described in Appendix C, the second order effect and occurring stress have been determined.

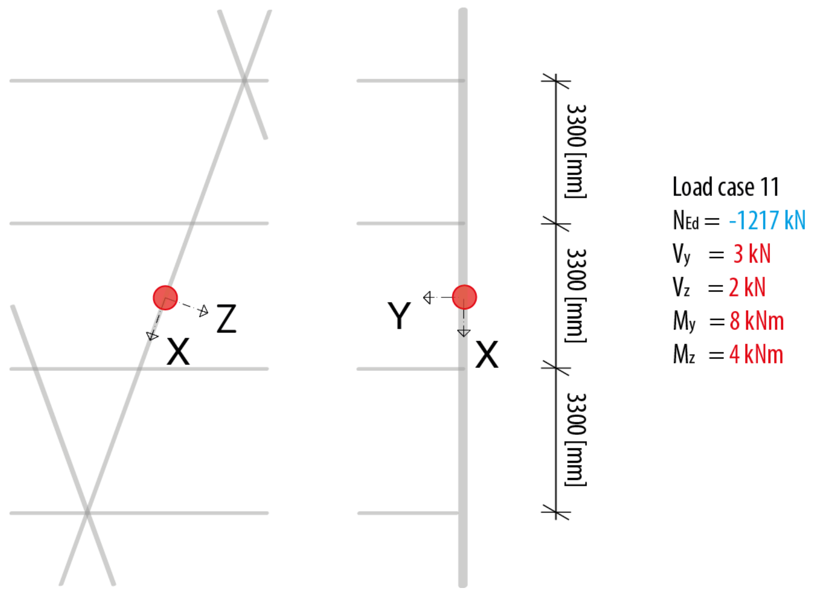


Figure 89: Normative load cases at section P4 (Local coordinate system)

Z-DIRECTION				
Buckling length	$l_{buc,y}$	=	5265	[mm]
First order eccentricity	e_i	=	20	[mm]
Design value moment force, load case 11	$M_{Ed,y}$	=	32.3	[kNm]
Moment force stress, load case 11	M_{Ed}/W_y	=	3	[MPa]
Y-DIRECTION				
Buckling length	$l_{buc,x}$	=	3510	[mm]
First order eccentricity	e_i	=	20	[mm]
Design value moment force, load case 11	$M_{Ed,x}$	=	28.3	[kNm]
Moment force stress, load case 11	M_{Ed}/W_z	=	3.3	[MPa]
Normal force stress, load case 11	N_{Ed}/A	=	-8.1	[MPa]

H

STRUCTURAL CALCULATIONS DETAILS

H.1 CALCULATIONS DETAIL 1

The loads of the normative load case for detail 1 are presented in the figure below. Note that the M_y and M_z moment forces do not occur at the connection with the concrete diagrid, but at the connection of member FM_{d1} with the steel floor structure.

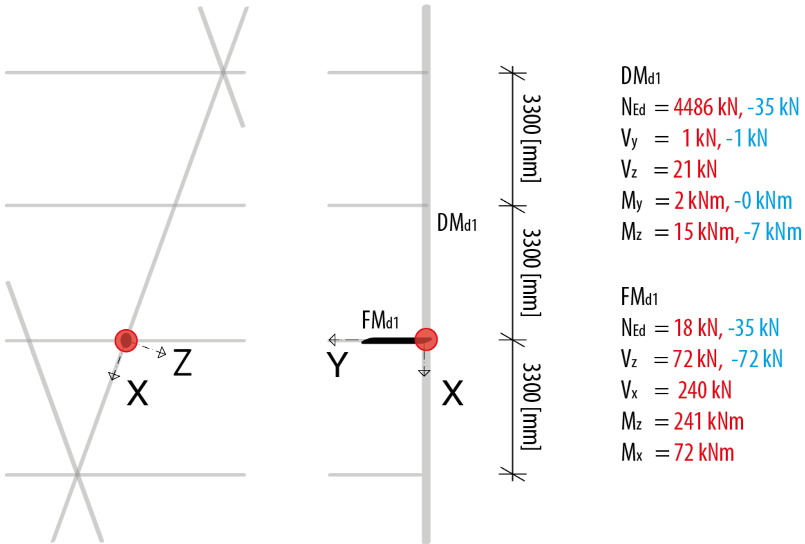


Figure 90: Detail 1, normative load attached floor beam (Local coordinate system).

H.1.1 *ULS calculation regarding the elliptical section*

An elliptical section of $h = 420$, $b = 210$, $t = 16$ is applied. A shear force of 240 [kN] is imposed at the member. Shear capacity of the section is checked:

$$UC_{shear} = \frac{V_{Ed}}{A_w \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}} = 0.29 \rightarrow Ok \tag{32}$$

A moment forces progresses towards the connection with the steel edge beam. Maximum moments of; 241 [kNm] in M_z direction and 72 [kNM] in M_x direction are imposed.

$$UC_{Moment,z} = \frac{\frac{M_z \cdot h}{2 \cdot I_z}}{f_y \cdot \gamma_{M,0}} = 0.56 \rightarrow Ok \tag{33}$$

$$UC_{Moment,x} = \frac{\frac{M_x \cdot b}{2 \cdot I_x}}{f_y \cdot \gamma_{M,0}} = 0.25 \rightarrow Ok \tag{34}$$

The interaction between the two moment forces is not considered.

H.2 CALCULATIONS DETAIL 2

The loads of the normative load case for detail 2 are presented in the figure below. Note that the M_y and M_z moment forces do not occur at the connection with the concrete diagrid, but at the connection of member FM_{d2} with the floor slab.

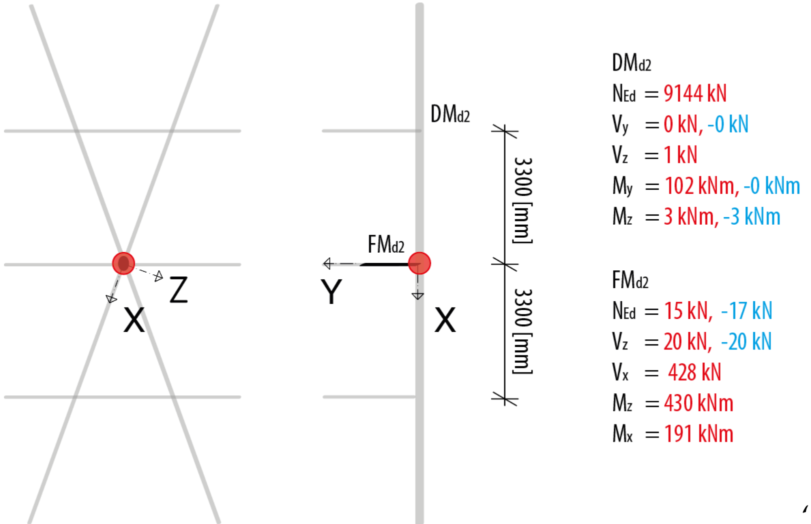


Figure 91: Detail 2, normative load attached floor beam (Local coordinate system)

H.2.1 *ULS calculation regarding the elliptical section*

An elliptical section of $h = 420$, $b = 210$, $t = 32$ is applied. A shear force of 428 [kN] is imposed at the member. Shear capacity of the section is checked:

$$UC_{shear} = \frac{V_{Ed}}{A_w \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}} = 0.31 \rightarrow \text{Ok} \tag{35}$$

A moment forces progresses towards the connection with the steel edge beam. Maximum moments of; 4281 [kNm] in M_z direction and 191 [kNM] in M_x direction are imposed.

$$UC_{Moment,z} = \frac{\frac{M_z \cdot h}{2 \cdot I_z}}{f_y \cdot \gamma_{M,0}} = 0.58 \rightarrow \text{Ok} \tag{36}$$

$$UC_{Moment,x} = \frac{\frac{M_x \cdot b}{2 \cdot I_x}}{f_y \cdot \gamma_{M,0}} = 0.41 \rightarrow \text{Ok} \tag{37}$$

The interaction between the two moment forces is not considered.

H.3 CALCULATIONS DETAIL 3

The loads of the normative load case for detail 3 are presented in the figure below. Note that the M_y and M_z moment forces do not occur at the connection with the concrete diagrid, but at the connection of member FM_{d3} with the floor slab.

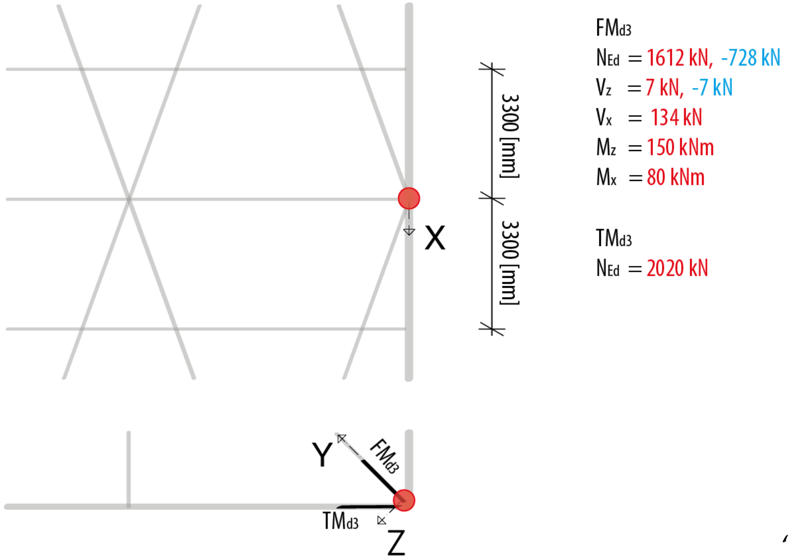


Figure 92: Detail 3, normative load attached floor beam (Local coordinate system)

H.4 CALCULATIONS DETAIL 4 AND 5

H.4.1 *Detail4*

The loads of the normative load case for detail 4 are presented in the figure below. The connection of prefab elements is located at the position where the imposed moment lines about find there minimum. Nonetheless due to construction tolerances moments can be introduced. The moments are considered to be normative for the design of the connection of prefab elements. Therefore moments generated by construction tolerances a considered for the design. The normative loads for detail 4 are presented in the figure below.

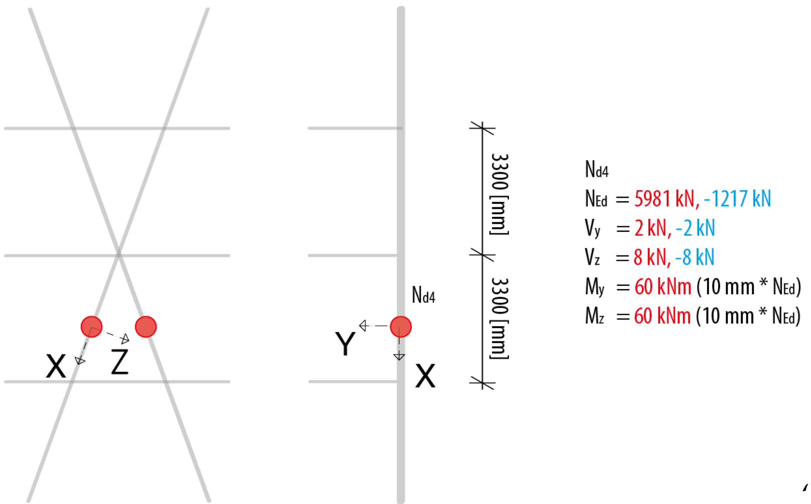


Figure 93: Detail 4, normative load prefab element connection (Local coordinate system). Moments are generated by construction tolerances

H.4.2 *Steel reinforcement*

The possible tensile forces will be transferred by reinforcement bars. The reinforcement bars are casted into the prefab floor members. At the construction side the floor members can be connect with the node elements. Through concrete mortar the reinforcement bars are fixed into the node elements.

The introduction length of the prestress force may reach up to 360 [mm] With a considered slip of 0.1 [mm] the bond length can reach up to 220 [mm]. The reinforcement bars are therefor embedded for 600 [mm] to ensure the tensile forces to be transferred.

H.4.3 *Calculations detail 5*

The connection of prefab elements is located at the position where the imposed moment lines about find there minimum. Nonetheless due to construction tolerances moments can be introduced. The moments are considered to be normative for the design of the connection of prefab elements. Therefore moments generated by construction tolerances are considered for the design.

The normative loads for detail 5 are presented in the figure below.

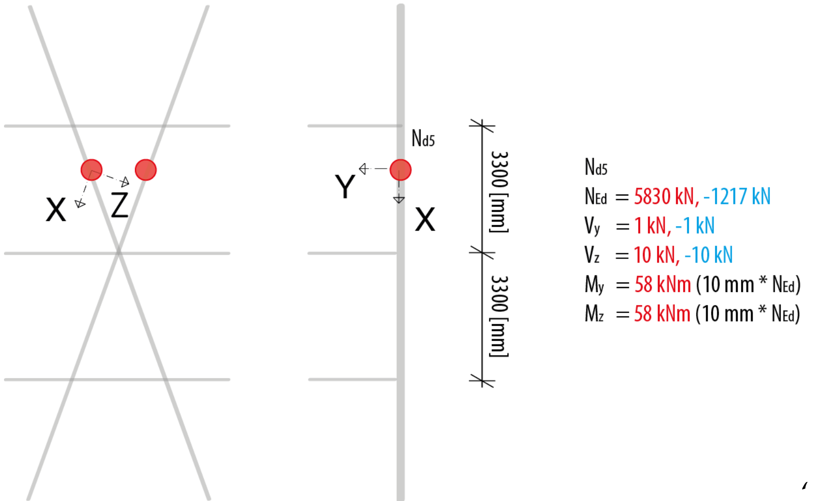
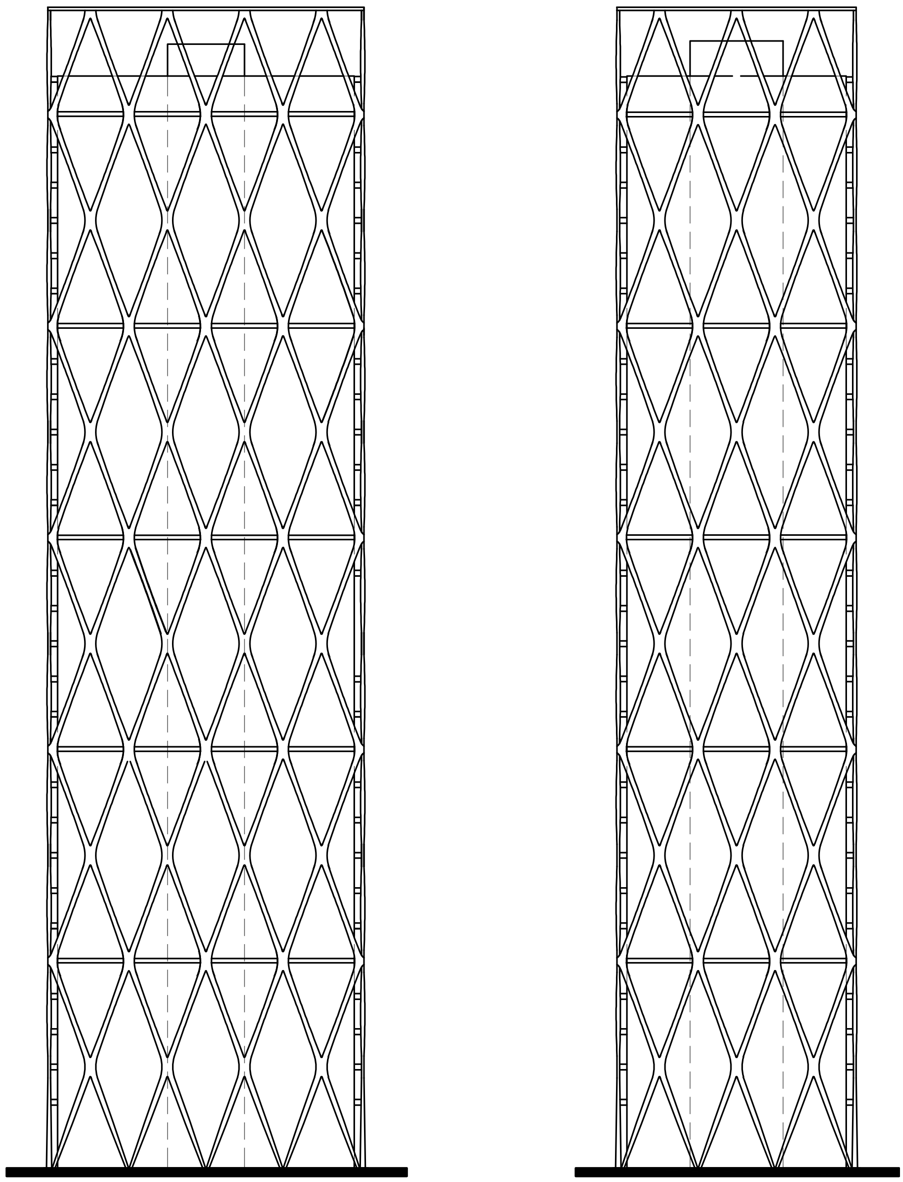


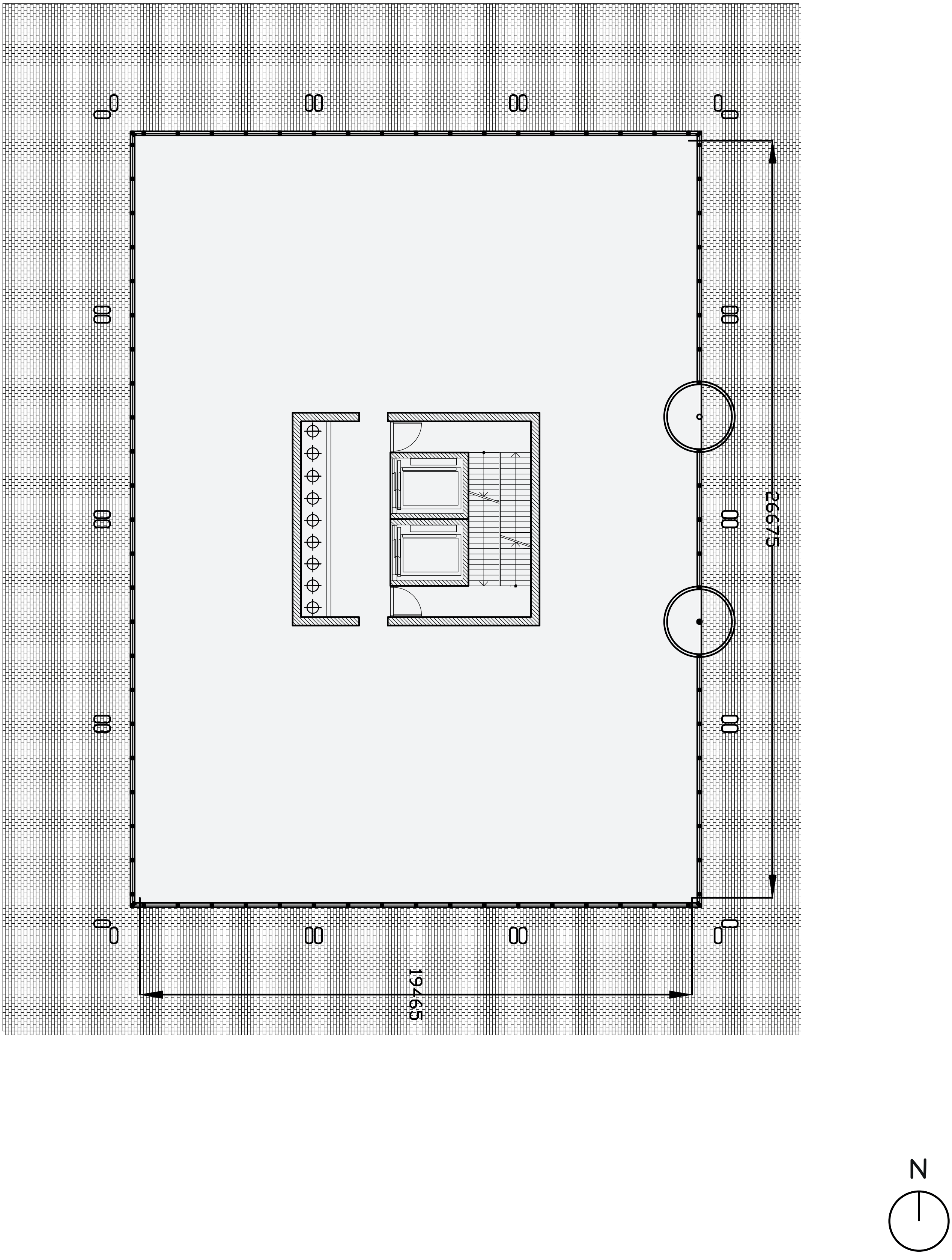
Figure 94: Detail 5, normative load prefab element connection (Local coordinate system)

I
TECHNICAL DRAWINGS

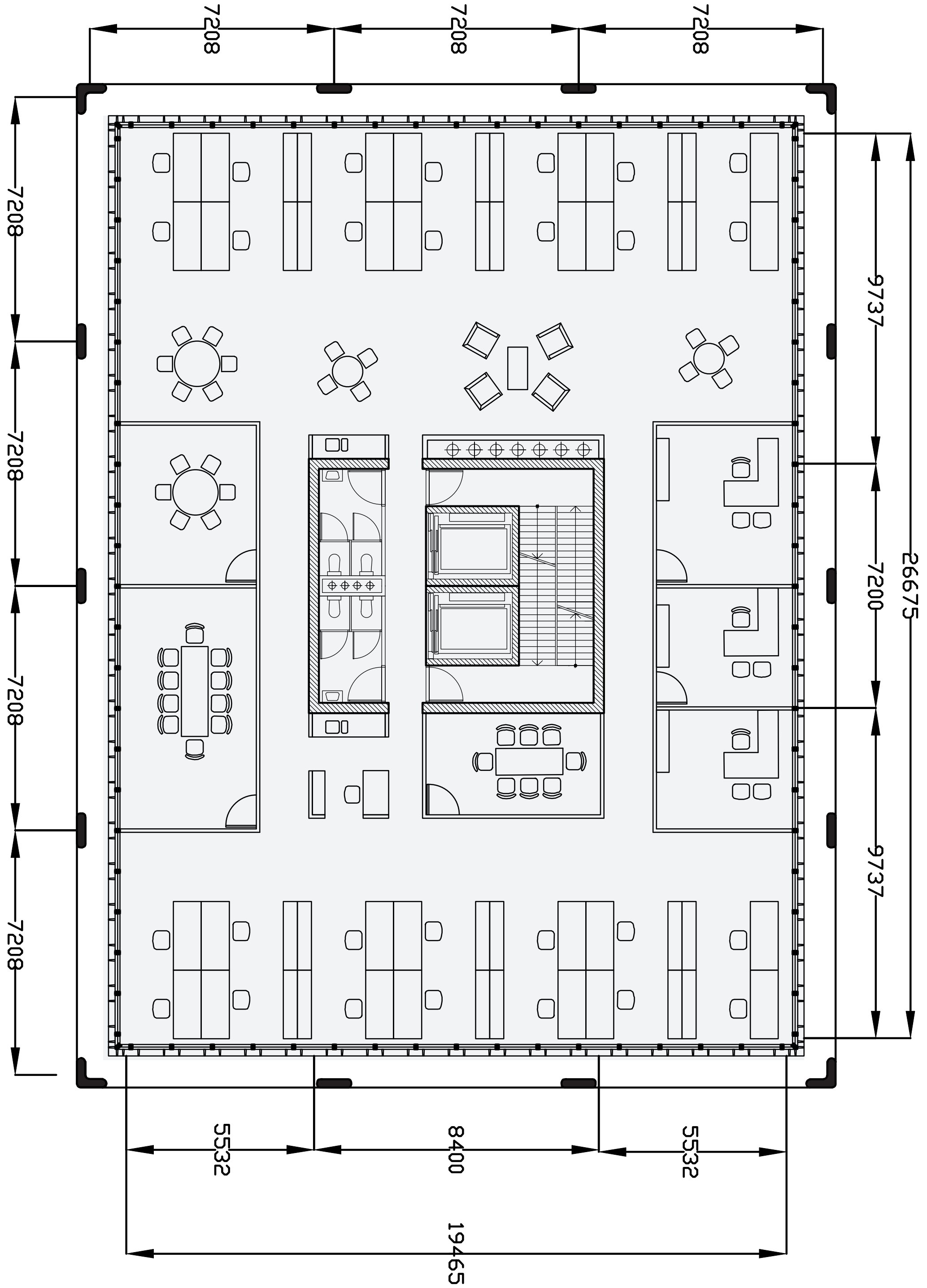
- Façade
- Ground floor plan
- Typical floor plan
- Façade fragment
- Structural floor plan
- Detail 1
- Detail 2
- Detail 3
- Detail 4
- Detail 5

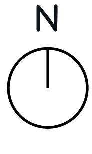


drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Building facade	A3	1:500
	Project			Unit	
	TU Delft Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	mm	

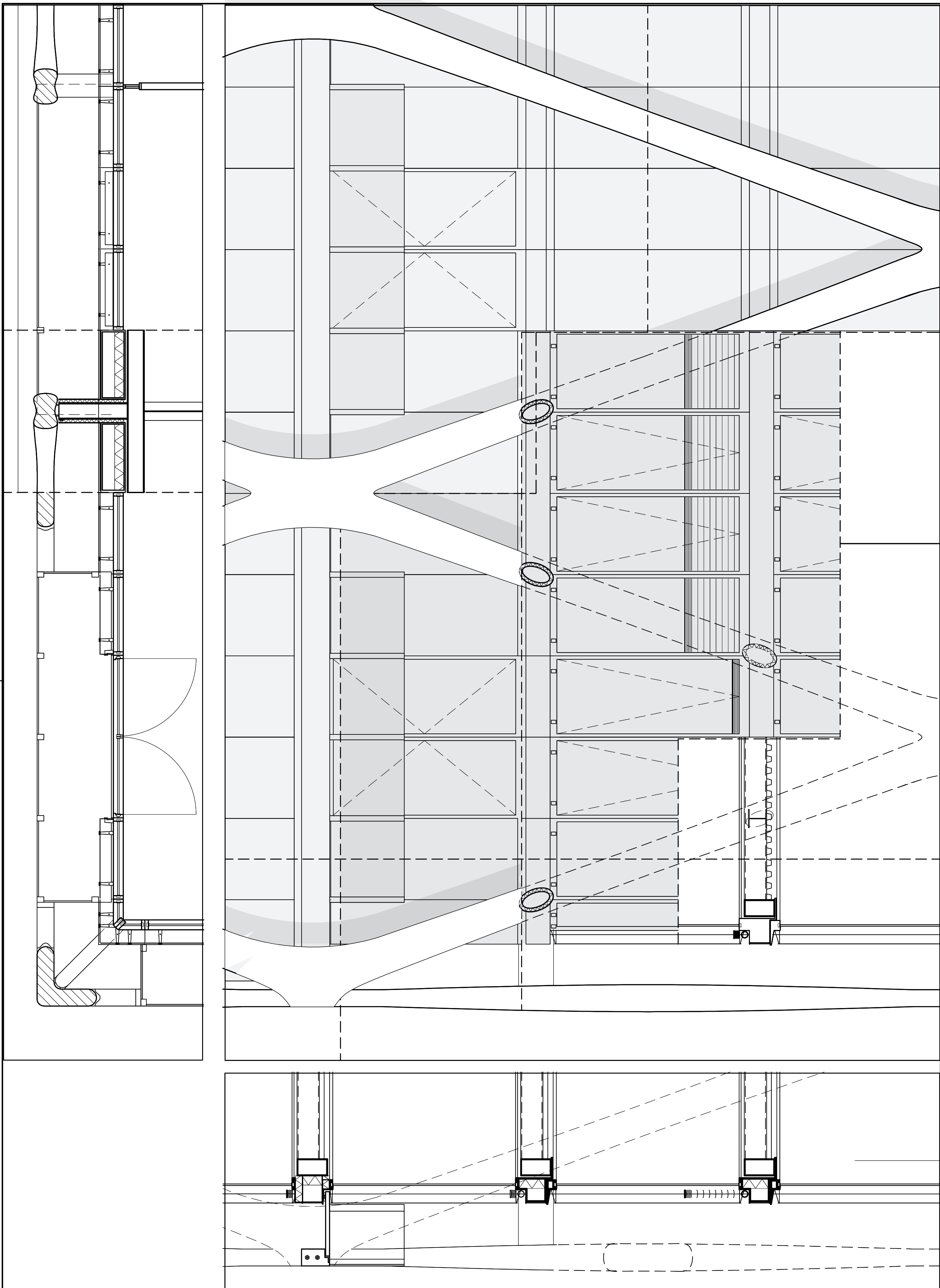


drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Ground floor plan	A4	1:200
	Project			Unit	
	Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	mm	

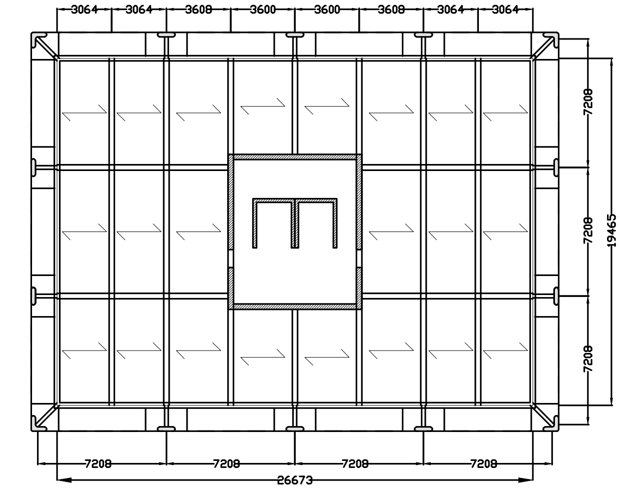


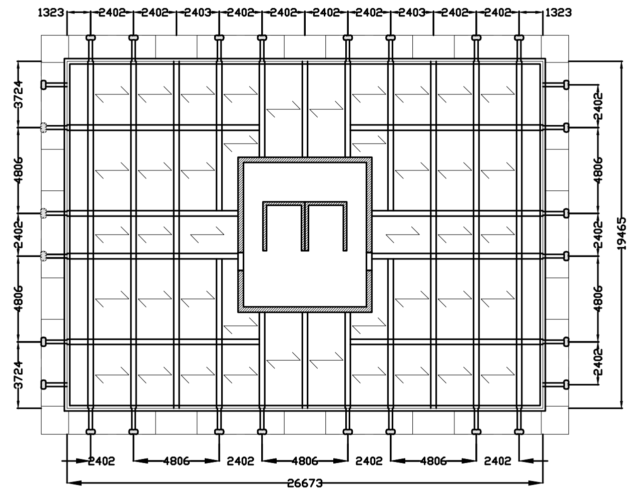


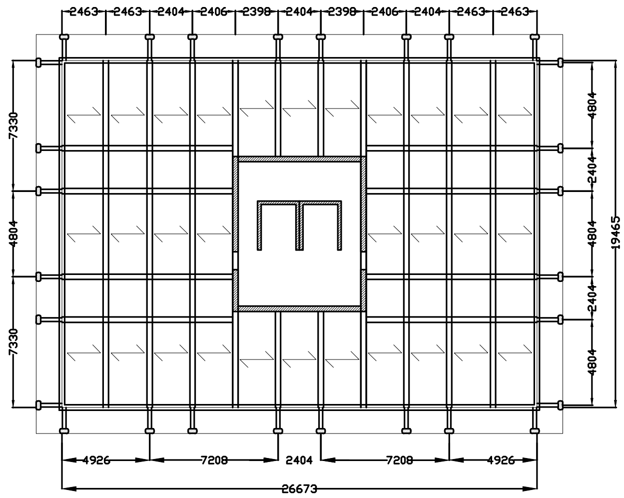
drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Typical floor plan	A4	1:200
	Project: Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	

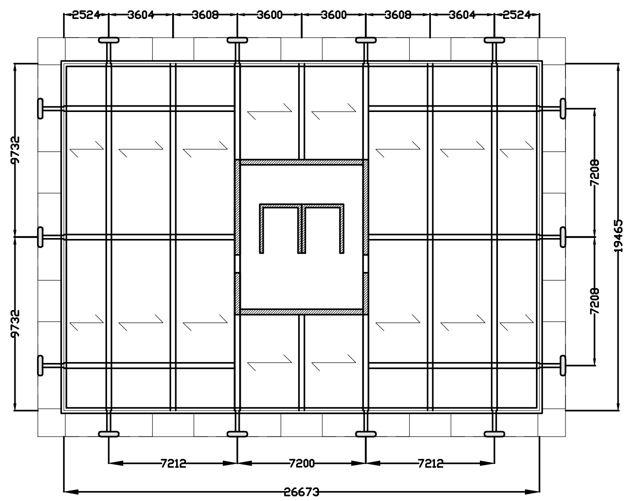


drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Facade fragment	A3	1:50
	Project: TU Delft Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	

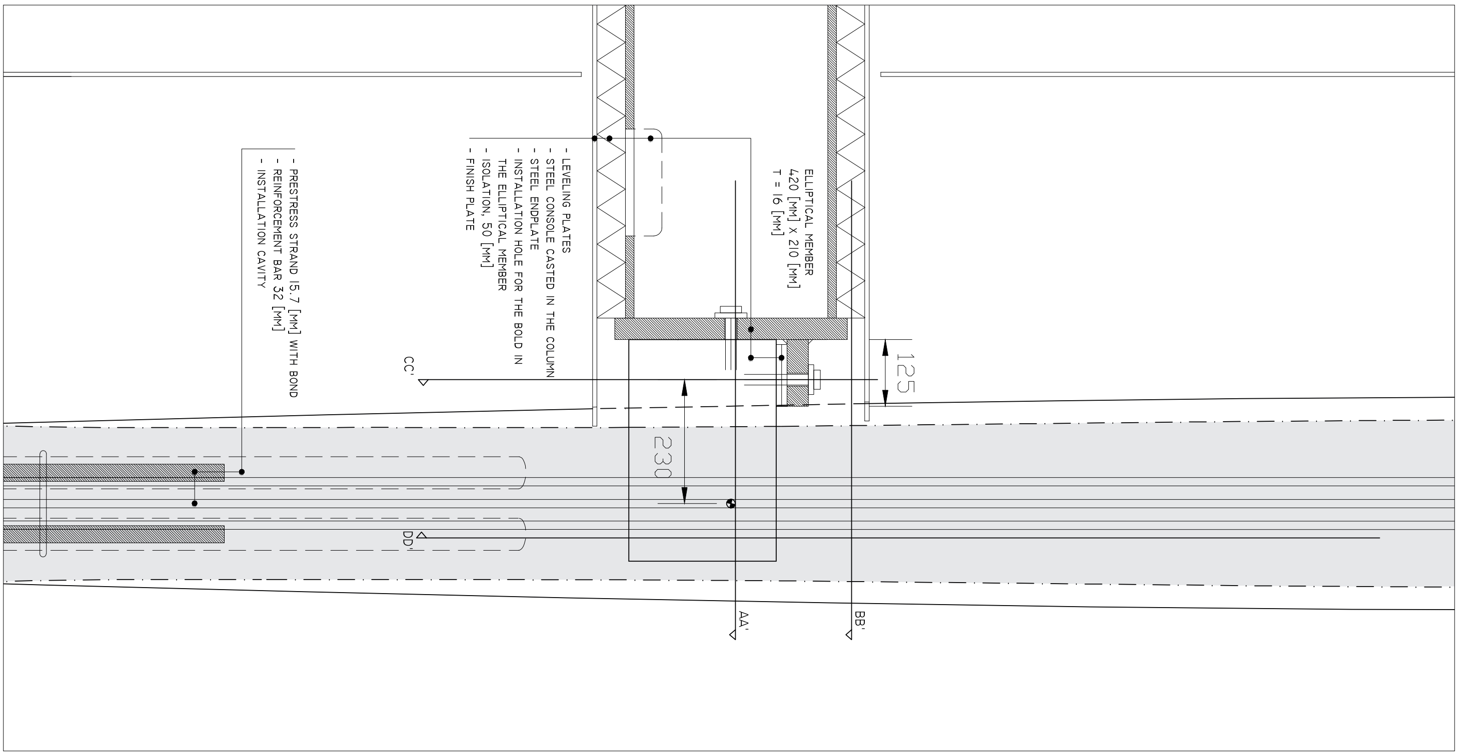


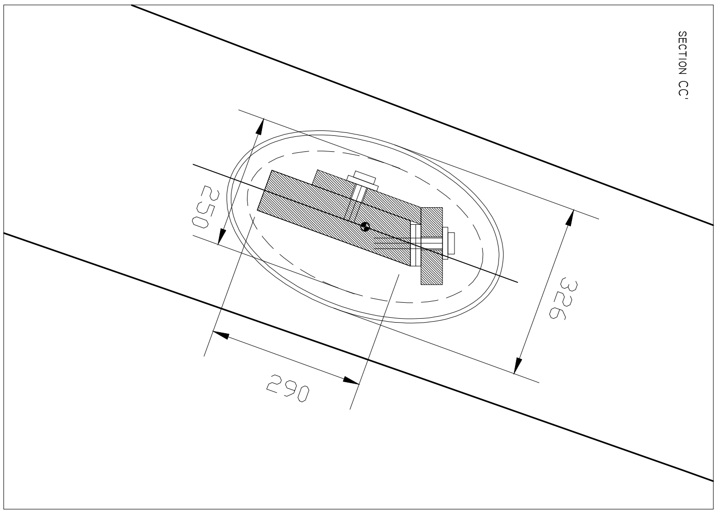


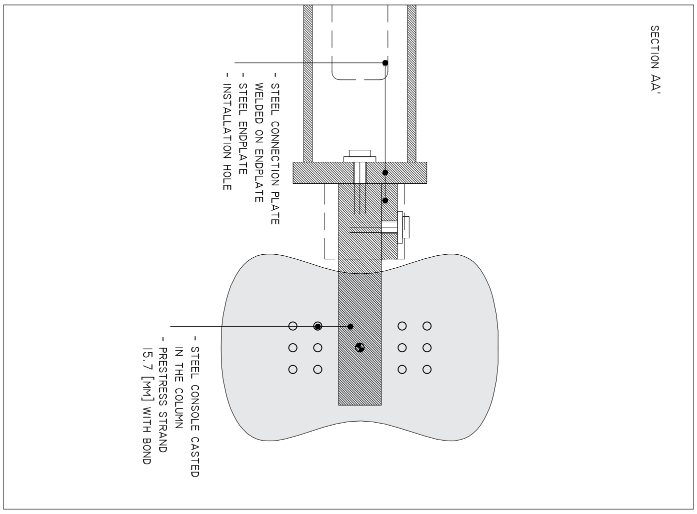


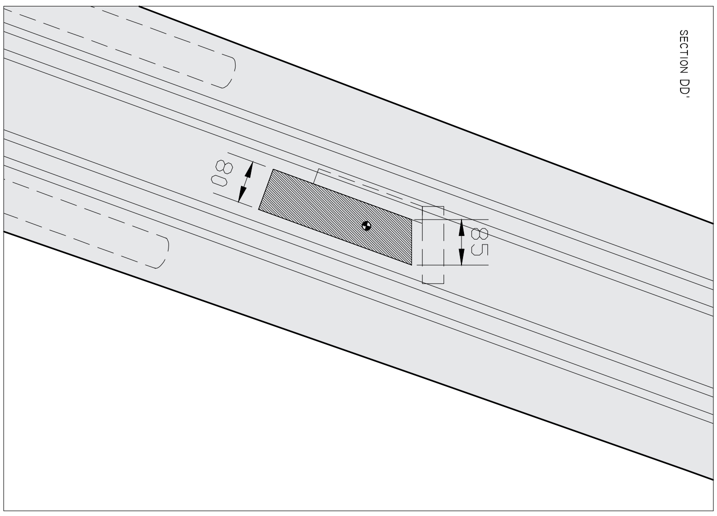


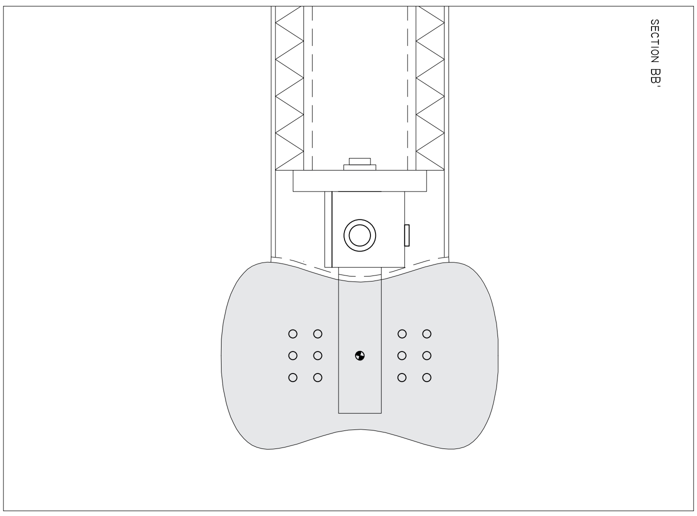
drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Structural floor plans	A3	1:300
	Project: TU Delft Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	



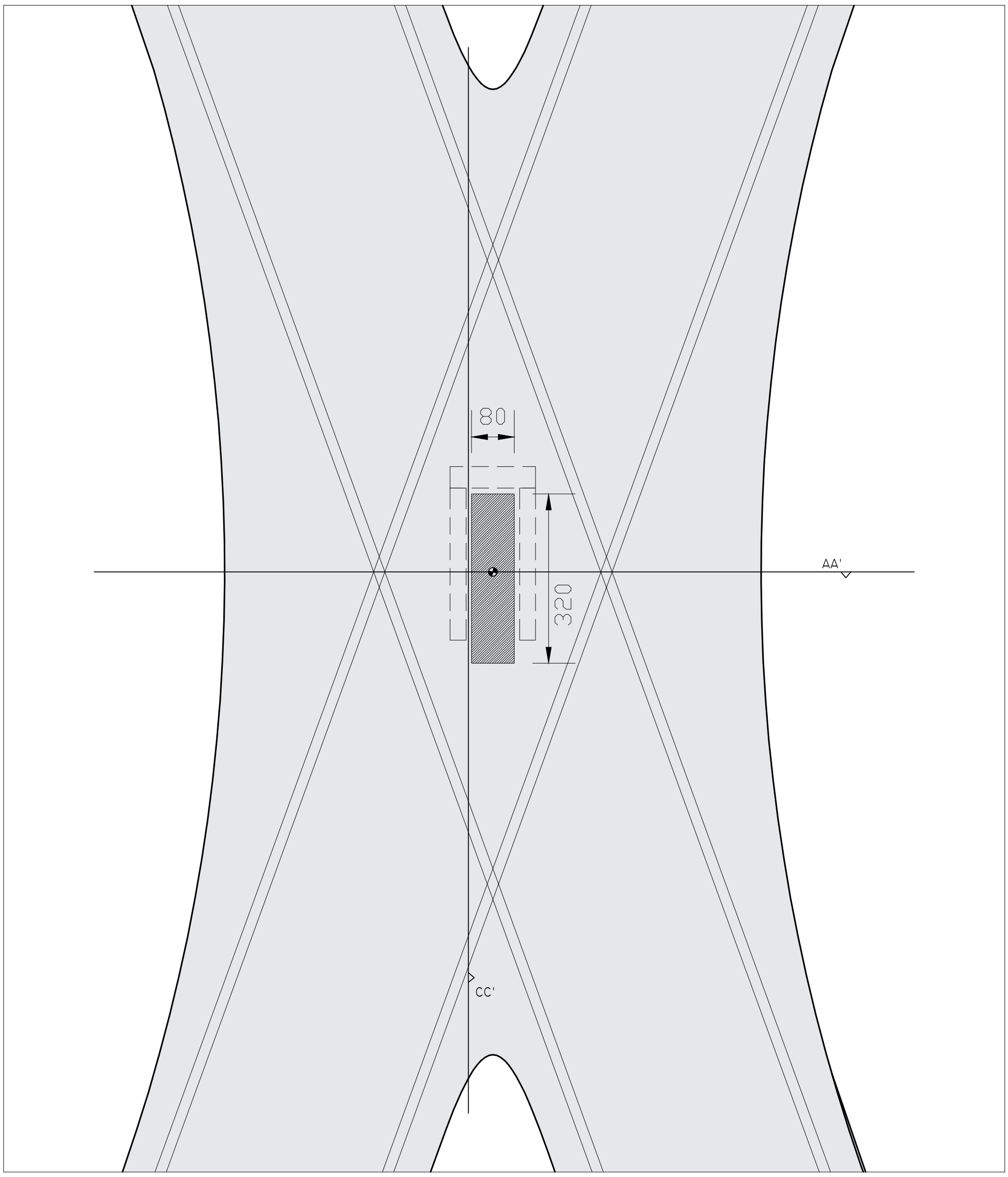


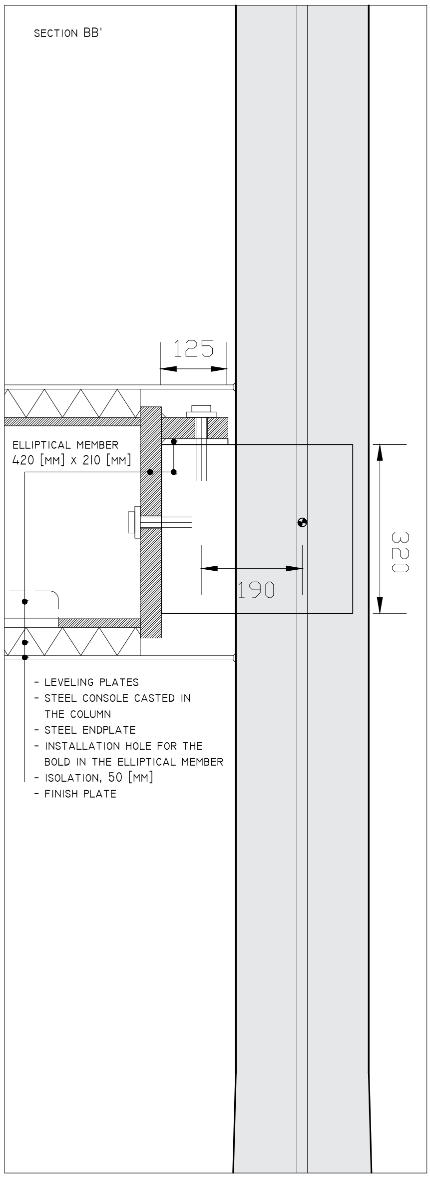


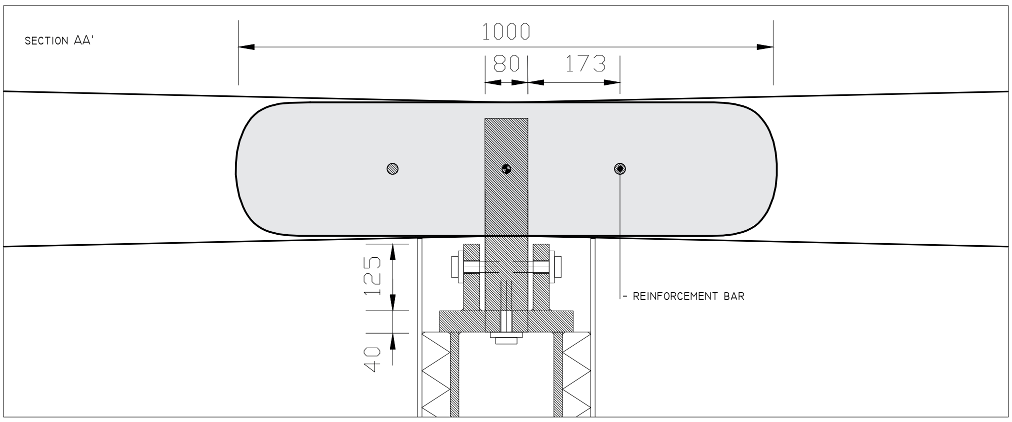




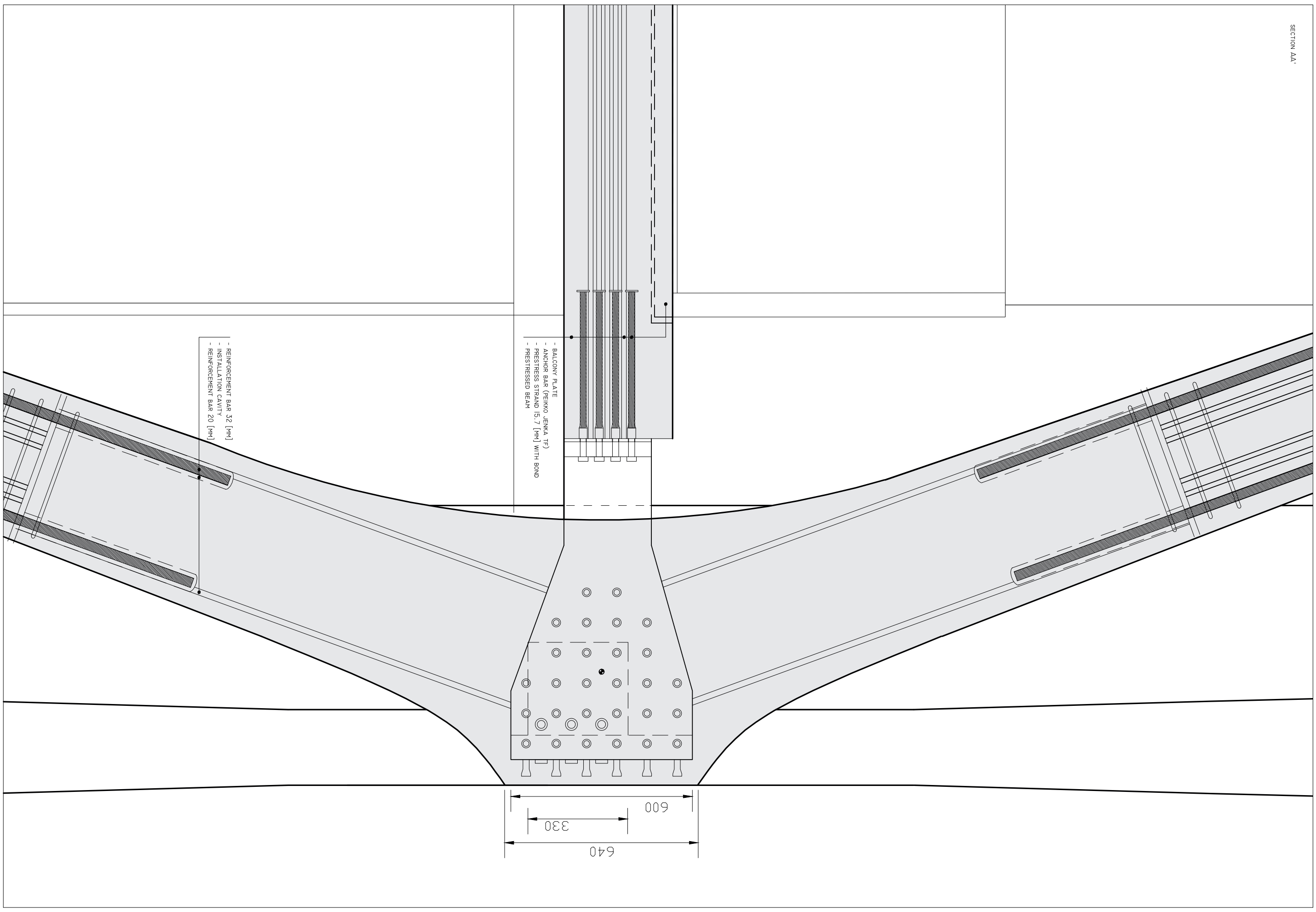
drawing nr.	Name	Date	Title	Size	Scale
D1	Paul Berendsen	10 / 14	Facade detail 1	A3	1:10
	Project: Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	

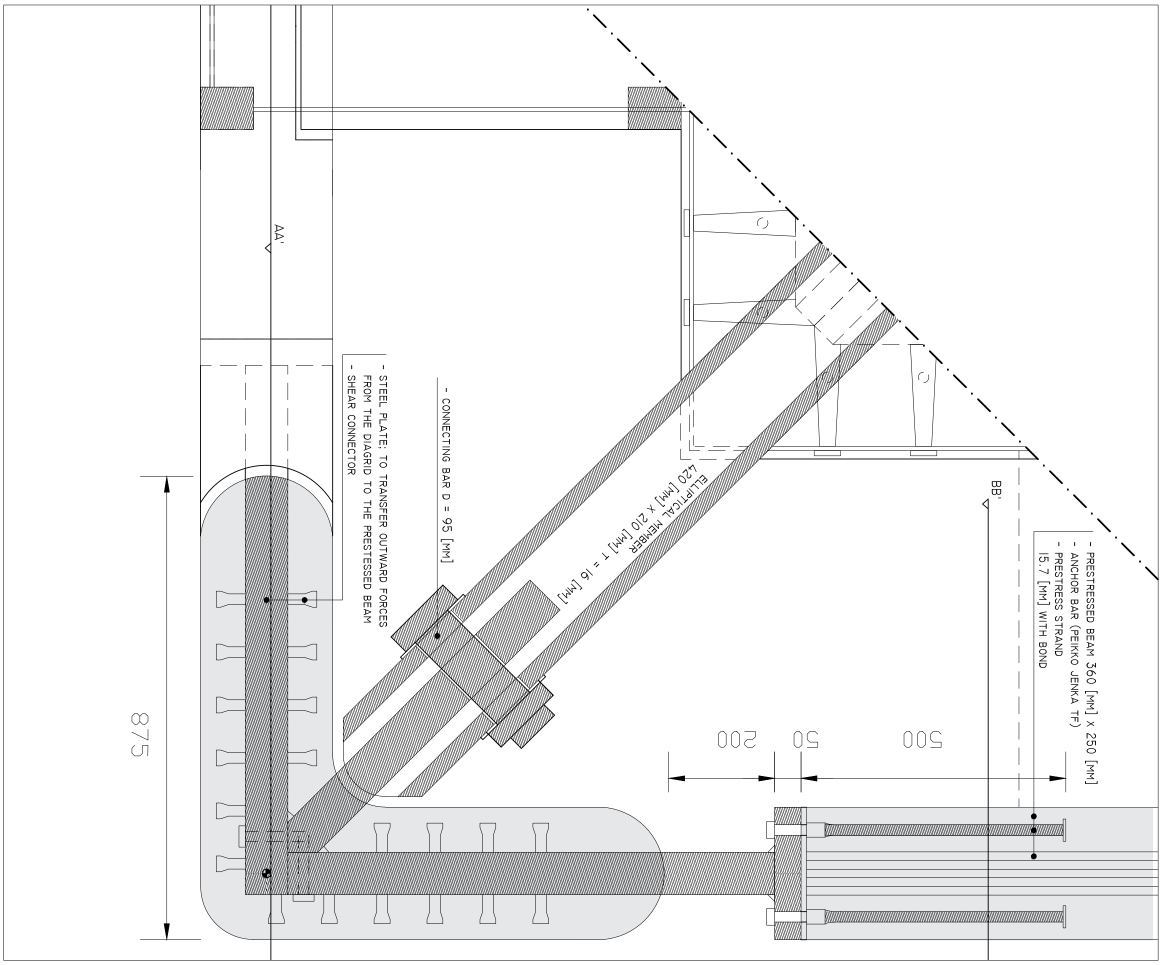


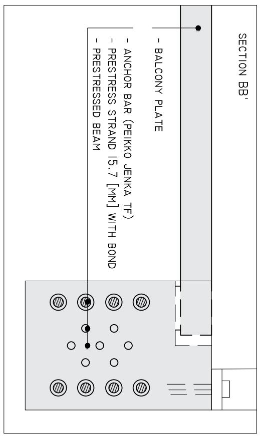




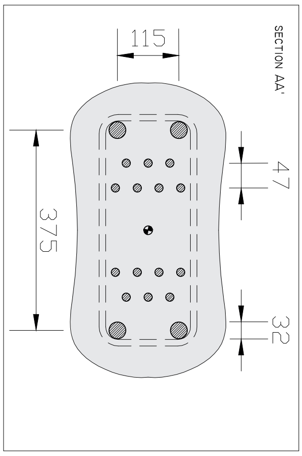
drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Facade detail 2	A3	1:10
	Project: Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	

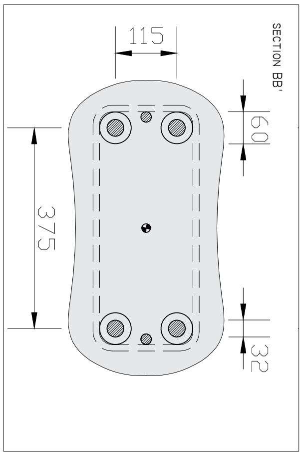


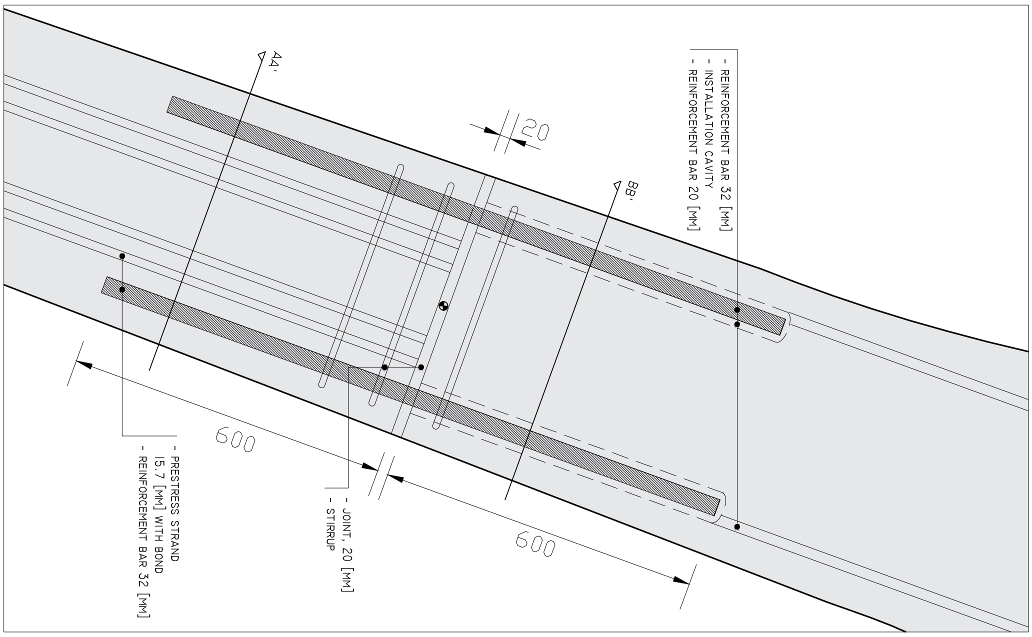


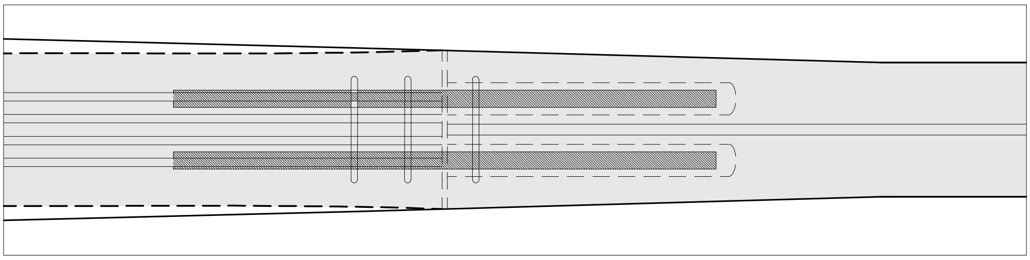


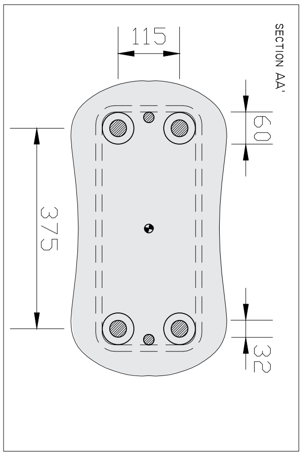
drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Facade detail 3	A3	1:10
	Project: Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	

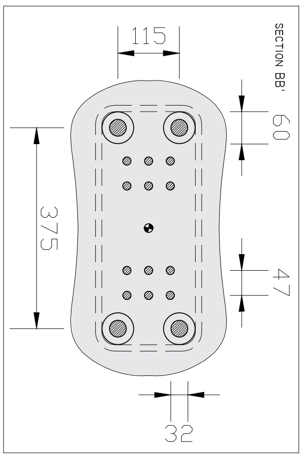


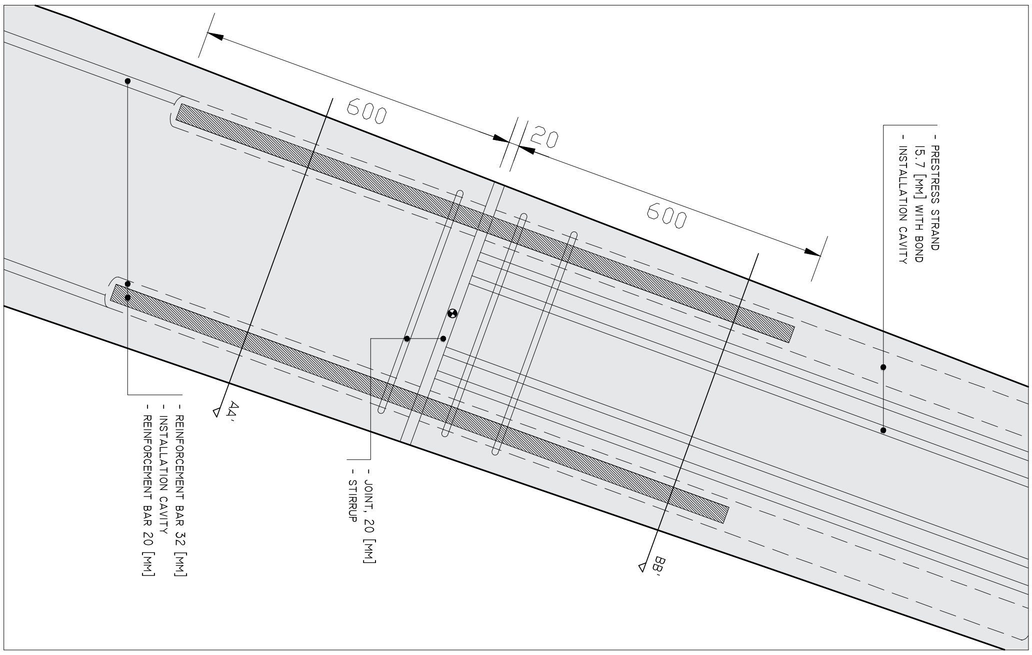


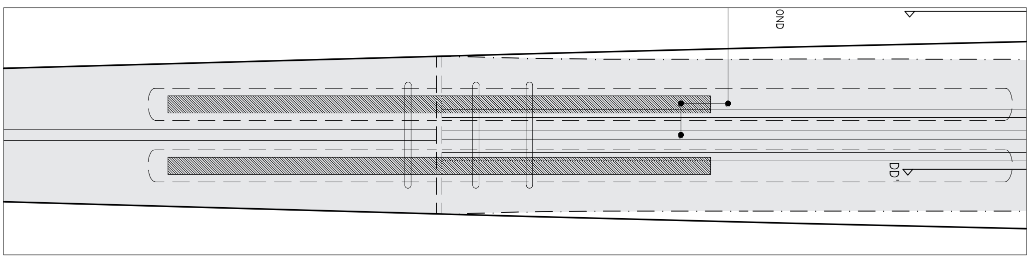












drawing nr.	Name	Date	Title	Size	Scale
01	Paul Berendsen	10 / 14	Facade detail 4 [left] & 5 [right]	A3	1:10
	Project: Graduation Thesis		TU Delft, Faculty of Civil Engineering and Geosciences	Unit: mm	

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