Alternative Reinforcement Approaches

Extended service life of exposed concrete structures

Master of Science Thesis in the Master's Programme Structural Engineering and Building Technology

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Division of Structural Engineering
Concrete Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover:
Rusted reinforcement bars (Gudmundsson 2008).

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ABSTRACT

Concrete is the most used construction material today, the vast majority of it reinforced with ordinary reinforcing steel. Much research has been put into improving the protection of steel reinforcement by increasing concrete cover and impenetrability. However, even though concrete quality has indeed improved much in the last decades, reinforcement corrosion is still a serious problem.

The aim of this project was to evaluate alternative reinforcement approaches. In other parts of the world, especially North America, the use of alternatives to ordinary reinforcing steel is much more adapted. Distribution channels, real-life examples and design codes are all available there and have been used as sources in this project. The material properties, availability, required reinforcement amounts, sustainability and cost were evaluated for each reinforcement approach with subsequent conclusions.

Initially, research on available reinforcement methods was studied. The experiences, material properties and cost of the alternatives were evaluated. Some of the reinforcement approaches were judged as promising and chosen for further study in the following phases. The chosen materials are: Ordinary reinforcing steel; ferritic and austenitic stainless steel; carbon, aramid, basalt and glass fibre reinforced plastics; plain and fibre reinforced concrete.

A case study project was chosen from the stock of projects designed at COWI, where the research project was carried out. A part of the building's basement was designed with the several reinforcement approaches. The reinforcement arrangements were presented in drawings and conclusions from the design processes were noted.

A life-cycle analysis of both cost and sustainability was performed based on the literature study and reinforcement designs. It was concluded that if the longer service life made possible by more durable reinforcement is utilized, all studied alternatives to ordinary reinforcing steel are more economic over the life-cycle of the building. The initial cost though, as expected, is higher for most alternatives to ordinary reinforcing steel. However as many large costs are independent of reinforcement approach; the impact on the total initial investment would be within just 6 % for all options.

Keywords: *Concrete, reinforcement, stainless steel, corrosion, durability, FRP, life-cycle cost, LCC, LCA.*

Alternativa armeringsmetoder

Förlängd livslängd hos utsatta betongstrukturer

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SAMMANFATTNING

Betong är det mest använda byggmaterialet idag, majoriteten av det armerat med ordinärt armeringsstål. Mycket forskning har utförts för att förbättra skyddet för armeringsstål genom att öka täckande betongskikt och betongens täthet. Trots att betongkvaliteten faktiskt har förbättrats mycket under de senaste decennierna, är armeringskorrosion fortfarande ett allvarligt problem.

Syftet med detta projekt är att utvärdera alternativa armeringsmetoder. I andra delar av världen, särskilt i Nordamerika, är användningen av alternativ till vanligt stål mycket mer utbredd. Distributionskanaler, verkliga exempel och normer är samtliga tillgängliga där och har använts som källor i detta projekt. Materialegenskaper, tillgänglighet, armeringsmängd, hållbarhet och kostnad har utvärderats för varje armeringsmetod med efterföljande slutsatser.

Inledningsvis studerades tillgänglig forskning rörande möjliga armeringsmetoder. Erfarenheter, materialegenskaper och kostnader för alternativen utvärderas. Några av armeringsmetoderna bedömdes vara lovande och valdes för fortsatt utvärdering i följande faser. De valda materialen är: Vanligt stål; ferritiskt och austenitiskt rostfritt stål; FRP med fibrer av kol, aramid, basalt och glas; oarmerad och fiberarmerad betong.

En fallstudie genomfördes på ett byggprojekt som valdes från beståndet av projekt hos COWI, där examensarbetet utfördes. En del av byggnadens källare dimensionerades med flera armeringssmetoder. Armeringsfördelningen presenterades i ritningar och slutsatser från designprocessen kommenterades.

En livscykelanalys av både kostnad och hållbarhet utfördes utifrån litteraturstudien och armeringsdimensioneringen. Slutsatsen är att om den längre livslängden som möjliggörs av de mer beständiga armeringsmaterialen utnyttjas, så är samtliga studerade alternativ till vanligt stål billigare mätt över hela byggnadens livscykel. Den initiala kostnaden är som väntat högre för de flesta alternativ till vanlig stålarmering. Många stora kostnader är emellertid oberoende av armeringstyp och slutgiltig inverkan på den totala initiala investeringen är inom 6 % för samtliga alternativ.

Nyckelord: *Betong, armering, rostfritt stål, korrosion, beständighet, FRP, livscykelkostnad, LCC, LCA.*

Contents

Preface and acknowledgements

This thesis is a state of the art report of alternative reinforcement approaches. It intends to give recommendations on possible replacements for carbon steel reinforcement bars for applications in aggressive environments.

The project has been carried out from June to October 2014 at COWI's office in Gothenburg, Sweden. The work is a standalone project based upon a personal initiative, i.e. not in direct contact with ongoing research at any university. The project details were developed in collaboration with COWI and Chalmers.

The thesis' examiner is Björn Engström, Professor/Vice Head of Department at the department of Civil and Environmental Engineering, Structural Engineering at Chalmers University, Gothenburg, Sweden. Supervisors have been Pontus Nilsson and Magnus Clase at the Building and Property division at COWI, Gothenburg, Sweden.

I direct my sincere gratitude to Björn, Pontus and Magnus for their help during this project. I would also like to thank my opponent Mattias Blomfors for his continuous support and much valued opinions. Furthermore I thank all the people at COWI whom I have interviewed, asked questions and who have given me of their time and knowledge. Finally I thank my dear wife Jessica for her constant love and support.

Gothenburg, October 2014

Johan Karlsson

1 Introduction

1.1 Background

1.1.1 State of the art

Concrete is the most used construction material today and has been so for some time. Simplified, concrete can be said to be artificial rock which is mouldable to practically any shape. In itself it is a composite material, typically with sand and gravel as aggregates and cement paste as matrix. There has been much development of concrete in the last decades; cement has partly been exchanged for other active substances such as fly ash and ground slag. The voids in the micro-structure can now be filled with micro-silica and improved grading of the aggregates. Compared to the typical 20 MPa compressive strength of a low grade concrete, the 800 MPa achieved in laboratory with active curing and steel grain aggregate is quite remarkable (Khan and Nazar 2011). These concretes also have tensile strengths to start counting with. The highest performing concretes used today in Sweden have strengths of around 160 MPa in compression and 10-20 MPa in tension (Olson 2014-06-12). This value give rise to questions whether the models and assumptions used for concrete design are still valid for the highest range of concretes and also if design codes should be revised, since Eurocode today does not directly include concrete strength classes above C100/115 (SIS 2008).

However, for the most widespread range of concretes, the low tensile capacity is a problem and often the limitation for the resistance of a member in plain concrete. Therefore, concrete is in most of its applications reinforced. The reinforcing material that totally dominates the market is carbon steel rods, normally in the range of 6-32 mm diameter (Al-Emrani et al. 2011a). Fibres dispersed directly in the concrete mix are also used in some applications. Typically they are not primarily a part of the load-bearing system; instead they are added to improve cracking behaviour and to increase toughness. To minimize cracking and deflections, concrete members can also be prestressed. To achieve the prestressing effect tendons are tensioned to induce a compressive force onto the concrete member. It can also be achieved by imposed deformations during construction (SBK 2014).

Steel is passive, i.e. has very low corrosion currents in high pH environments. This is thanks to the formation of a protective film on the steel surface, which almost completely stops corrosion taking place. Young concrete provides such an environment and is thus an effective protective environment for steel reinforcement. With time, the alkalinity becomes reduced in the concrete due to chemical reactions. At the same time aggressive compounds find their way into the concrete cover. The combined effect of these processes can cause quick corrosion development.

Other reinforcement materials have been used with varying results. In North America, fibre-reinforced polymer rebars have been used extensively in infrastructure projects. For buildings however, there are a number of issues to overcome with this material. Different steel alloys have also been used as well as carbon steel covered in different coatings.

Figure 1 Concrete spalling (Sarang 2012).

1.1.2 Problem

As with all other sciences, the problems with corrosion and the causes of it were not entirely known to the reinforced concrete pioneers around the beginning of the 20th century. Yet some remarkable structures, still standing, such as the Salginatobel Bridge in Switzerland were built during this time. After the world wars, with a combination of the effects of bomb raids and years of fast urbanization and an ongoing baby boom, the need for housing and infrastructure was extraordinarily high. Concrete had now become this material of choice in the construction industry and much of the new production used the material. In Sweden, a political endeavour called “Miljonprogrammet” set out to build a million new dwellings in ten years starting in 1965 (NE 2014). Most of these buildings were constructed with prefabricated concrete elements, swiftly mounted on site with large cranes. Compared with today’s concrete quality and awareness of the need for concrete cover, the housing project’s buildings were poorly built. The consequence is apparent, in most of the façade elements there are cracks and surface spalling. The same thing can be seen in infrastructure from the time. Today concrete covers are thicker and concrete denser. Perhaps these measures provide sufficient protection of the reinforcement to provide repair-free service during the design service life. The true response will, as always, reveal itself with time.

Concrete in composite action with steel, embedded in the concrete section where best needed, is a very good arrangement in many ways. The brittle and low tenacity nature of concrete is compensated by the ductility, stiffness and strength of steel bars. The problem of corrosion of the steel is also seemingly solved by protection and passivation from the surrounding concrete.

However, both a decrease in pH at the steel level and penetration of aggressive chemical substances, primarily chloride ions, may depassivate the steel and create a corrosion cell. Corrosion is driven by potential differences between different zones of a continuously connected metal system. When for instance pH is locally lowered, or the chloride front reaches a part of the steel network, that part will get a lower potential. As such it will become anodic and will thus be gradually reduced to iron oxides, i.e. rust.

Rust has practically no strength, wherefore the corroded steel must be seen as lost. Furthermore rust has 2-10 times greater volume than steel, depending on the variant of iron oxide that is formed (Yeomans 2004). This effect is usually more critical than the loss of section. This is due to the confined space the steel bars are limited to. As the rust forms, it must soon crack the surrounding concrete cover to make room for itself. These cracks then propagate and grow in width, which allows for faster ingress of water, ions and oxygen, further speeding up the corrosion. Thus a vicious circle is created which after time leads to spalling of large concrete chunks that leave the steel bare to the elements, further increasing access. At this stage, thorough repair work or demolition is imminent, either one leading to large costs. Moreover the structure may fail due to loss of concrete, steel or bond. Thus the structural resistance may be compromised, posing both a material and personal risk. An image showing typical spalling of the concrete cover can be seen in Figure 1.

For corrosion to take place depassivation of the steel must be combined with a certain degree of moisture in the concrete. The water contains ions and acts as electrolyte in the corrosion cell. Hence, dry concrete members such as for example indoor slabs and walls are rather unlikely to suffer from extensive corrosion, unless they are in regular contact with water as in e.g. shower rooms.

1.1.3 Alternative reinforcement approaches

Much can be achieved by improving protection of the reinforcing steel within the concrete. This has been a topic of extensive research in the last decades and today money and perhaps conservatism are the only real limitations that hold practically impermeable concretes back. With these, steel would be sufficiently protected and passivated during a foreseeable future. However, such high quality concretes are up to ten times more expensive than average ones (Olson 2014-06-12).

It may also be argued that even with extremely high-performing concretes, there are applications where corrosion can be initiated. No matter how well the concrete performs where it is intact, at cracks and joints problems may still occur. Here chlorides can reach the steel level locally in the cracks and cause pitting or crevice corrosion. Freeze-thaw attack can also cause the cracks to widen, thus exposing the steel further. An example is joints in concrete slabs under swimming pools where problems have been identified also in more recent buildings with high performing concretes (Rydholm 2014-06-16). There are also extraordinarily aggressive environments such as certain industries, water treatment plants and desalination plants that pose great challenges. The possibility of human errors during execution must also be taken into account and be designed for.

Given that the above stated problems nonetheless could be solved with ultra-high-performance concretes and that steel reinforcement could be successfully protected for long periods of time, there is still the aspect of cost. Even though such high performing concrete members could be made slimmer, the bulk of concrete used is massive compared to the reinforcing material. An increase in price of up to ten times of that mass is unlikely to be accepted widely and simpler concretes will continue to be used. These will, however slowly, allow aggressive compounds to permeate the concrete cover and ultimately reach the reinforcement. If long service lives are desired, the concrete covers must be made thick, wherefore the amount of concrete increases and as a consequence also the structural self-weight and thus the load. If the steel rebars could be replaced by some other reinforcing material that were inert, or at least less sensitive, to the harsh environment that causes steel corrosion, that could be a more passable way of limiting reparation costs and prolonging service life of concrete structures.

Such alternative reinforcement approaches have, if not as much as concrete itself, been under investigation during the last decades. Hence there are many different alternatives to conventional concrete reinforced with carbon steel, many already tested and commercially available. Some of them are: Unreinforced high performance concrete (HPC); form-active compressive structures; fibre reinforced concrete (FRC); galvanized carbon steel bars; epoxy-coated carbon steel bars; stainless steel bars; fibre reinforced polymer (FRP) in various shapes; bamboo rods and more.

1.2 Purpose

The purpose of this project was to investigate possible ways of avoiding future corrosion-related problems in concrete. This report should then hopefully be a part in spreading knowledge about available methods to industry and academia in Sweden, where these are relatively unknown among engineers to date. The report should deliver an unbiased and true picture of the available alternatives to steel reinforcement and perhaps also new angles or insights on some of them.

The topic of reinforced concrete in general and corrosion of reinforcement in particular, is very extensively studied worldwide. Many of the studies are specific and treating rather small sub-issues and others try to sum up current knowledge to make it more accessible. This project should be of the latter type and with the aim to spread knowledge in Sweden, where many of the matters here to be studied are still rather unknown to the wider construction guild. The work should be carried out with the goal of reaching recommendations and comparisons that could be used in real design problems. To reach that, a work order of four steps was planned as is further developed in Section 1.3.

1.2.1 Sustainability

Recently legislators and people in general have been made increasingly aware of the negative consequences that the industrialization, population growth, materialism and careless attitude towards ecosystems bring with them. Previous generations have generally had the material poverty and hardship of simpler life in closer memory and have naturally focused more on the gains of the modern civilization than possible downsides. It is only when the basic needs of people are fulfilled that other concerns may be expected. Since poverty is still so widespread in the world, billions of people still have the “rise” from a life struggling with basic needs ahead of them. At the same time most people in the rich part of the world choose to close their eyes and conduct business as usual. This is now starting to show in terms of waste islands in the oceans, cut down virgin forests, bee death and climate change. There are powerful economic structures with great interest in maintaining the system as it is and in the meantime the resources for sustainable human life on planet earth are gradually being destroyed.

In the construction industry, the possibilities to make a change are big and upon its actors falls a large responsibility. In Sweden, buildings in use account for about 40% of the total energy consumption (Byggnads 2012). Additionally enormous amounts of energy, raw materials and money are used during construction. To these means, the construction industry must take up the responsibility of producing energy efficient buildings and versatile structures with longer service life expectancy.

The main construction materials in Sweden are timber, steel and concrete. Each of the materials have their own pros and cons. Timber is renewable, readily available, cheap and easy to work with but has limited mechanical properties and durability. Steel has excellent mechanical properties and recyclability but is expensive, energy intensive and has durability issues. Concrete is cheap, mouldable and durable, but is energy intensive, non-recyclable and brittle; the latter often compensated by durability compromising steel reinforcement.

From a sustainability point of view, all three materials are best choices in certain applications. Timber is suit for structures of short to medium service lives, or in well protected conditions. Steel could be used for reusable structures that after its end-of-life can be melted down and reshaped. Concrete could be the best choice for monumental structures or important infrastructure with very long service lives, as well as exposed parts of buildings.

This project has focused on concrete and ways of making it possible to design structures of it with practically unlimited service life expectancy and without excessive rehabilitation during the structure’s service life. A great example of what can be achieved is set by the Romans that used a version of the material in accordance with its natural benefits already 2 millennia ago. The best example is the Pantheon in Rome that still today has the largest unreinforced concrete dome in the world (Moore 1995). This is an example of a form-active structure that through its shape maintains the thrust line within the perimeters of its structure, thus limiting tensile stresses and need for reinforcement. Finding ways of extending the design service life of structures is important if people in another 2000 years should have other traces of the industrial civilization than toxins and plastics in the world oceans and a decimated biological diversity.

1.2.2 Economy

A large part of the present stock of concrete buildings and infrastructure was constructed in the after war period with clear durability issues. These structures are now in need of exchange or thorough rehabilitation. This means a lot of activity and responsibility for present day engineers, architects and contractors. The obvious here has to be stated; there are naturally incitements for the building industry to every 20 - 50 years have the business of rehabilitating and exchanging the existing stock. This does not necessarily mean that most businesses will be unscrupulous enough to on purpose deliver inferior products, but the conflict of interests must be kept in mind. From a simplified resource conservation perspective, the short service life of public properties is an economic disaster for a society. Furthermore interruptions due to repair work restrict the usage of governmental services. However, politicians also want jobs, economic activity and GDP growth to keep their voters happy on a short term, why also they have double incentives. With regard to just economics, what is most beneficial is somewhat unclear. Including resource usage and sustainability however, it is better that an available economic scope for reforms is used in service-based fields such as health and education and that the built environment is made to last.

1.2.3 Hypotheses

- Reinforcing steel in highly exposed parts of concrete structures such as bridge decks, edge beams and parking house slabs could be exchanged with alternative reinforcement approaches with economical, practical and sustainability benefits.
- Alternative reinforcement methods could be combined with conventional where each method is used where best fit.
- Structural engineering codes for alternative reinforcement materials and approaches ought to be included in future editions of national and international calculation codes.
- If usage of alternative reinforcement materials increases, the prices could be largely decreased.
- The lower mechanical properties of some reinforcement materials can be overcome in design.
- The challenges and lack of experience in design execution can be overcome by planning and training.

1.2.4 Questions

- Can reinforcement bars be eliminated from exposed concrete structures altogether?
- Where reinforcement is unavoidable, are there reasonable alternatives to ordinary steel with regards to stiffness, strength, price, availability, execution, sustainability etc.?

- Could there be methods of replacing ordinary steel reinforcement so far unimagined?
- Could reinforced concrete be built with practically unlimited service life?

1.3 Method

First a literature review was to be performed, where the purpose was to gain a broad knowledge of the subject and gather material properties as well as advantages and disadvantages of each alternative. In this step most of the information should be acquired from books, reports and articles, but should also be complemented by interviews and field trips. Thereafter design calculations should be performed on an example project, with reinforcement arrangements for different approaches as the expected end result. Hopefully this step may also show strengths and weaknesses of the different alternatives in terms of execution etc. With the design finished a life-cycle cost analysis should be performed in order to compare the different options economically.

1.4 Limitations

The project should be limited to concrete structures only. The main focus should be on alternative reinforcement approaches and only secondary regard should be taken to improvements in the concrete itself or other ways of protecting the reinforcing material.

The thesis should sum up available studies and data as well as design calculations. No laboratory tests will be performed.

Among the things close to the subject that should not be covered in the paper are: Detailing (cover and joints), surface barriers, corrosion inhibitors, cathodic protection, chloride extraction and structural monitoring.

1.5 Outline

The order of the report follows the work plan described in Section 1.3. The nearest following part, Chapter 2, covers available reinforcement materials. Advantages and disadvantages as well as mechanical properties and approximate prices are listed. Chapter 3 treats reinforcement design of an example project with resulting reinforcement arrangements. Chapter 4 compares the results from the case study for price and sustainability from a life-cycle perspective. Chapter 5 concludes the project with a discussion, recommendations and suggestions for further research. At the end of the report there are literature references and appendices.

2 Material characteristics

2.1 Wish list of reinforcement material properties

Naturally, what is deemed desirable is situation dependent. In some cases it is crucial that a structural member is non-magnetic in order to avoid disturbance of sensitive equipment such as MR-scanners. In other cases corrosion resistance is the most important feature. In all situations price is an important factor that must be considered. Nevertheless some generally required properties are stated here.

- High strength and modulus of elasticity
- Similar thermal expansion as concrete
- High corrosion resistance
- Low weight
- Durable
- Homogeneity
- Simple to design
- Simple to work with on site
- Insensitive during transport and execution
- Chemically inert
- Cheap
- Available
- Low energy manufacture and transport
- From a renewable or abundant source
- Recyclable

2.2 Characteristics per material

All properties gathered in this section have to be seen as approximates. They are average values, meant to be used in a preliminary comparison of different materials and not meant for detailed design.

On today's free trade global market the price of materials, except those with limited shelf life, have the same price worldwide. Because of this many products are traded in the large global currencies. For this reason the prices in this report are stated in US dollars. Metal prices are spot prices gathered in June 2014 on the homepage of London Metal Exchange (LME 2014).

In Europe, North America and elsewhere there are structural engineering codes to aid structural designers. The codes are built upon years of research and are typically deterministic including substantial safety formats. In this paper, Eurocode is used as the first choice followed by ACI-codes from the USA and CSA-codes from Canada.

2.2.1 Concrete

Concrete is the most used structural material and has unique properties. In essence it is temporarily dissolved artificial limestone that can be moulded into any shape by man before it sets. It is a heavy construction material that is has good sound and fire resisting properties and that stores heat for evening out of temperature fluctuations.

2.2.1.1 Pozzolans

Concrete is highly alkaline, primarily due to the presence of calcium hydroxide $Ca(OH)_2$. This crystalline compound forms in the hydration of Portland cement. With time it is converted to calcium carbonate ($CaCO_3$) by reacting with the carbon dioxide (CO_2) in the air in a process called carbonation. The reaction product $CaCO_3$ is stronger and more compact than $Ca(OH)_2$. In fact it is the same compound that the process started with in the cement plant, namely limestone. In itself then the process is not adverse; the problem is that the passivating alkalinity also gradually disappears with pH dropping from around 13 to 8, which leaves steel activated and likely to corrode.

To a certain extent, cement can be replaced by other cementitious materials. One interesting group of such materials are pozzolans. These materials react with $Ca(OH)_2$ in the presence of water to form compounds with cementitious properties. Some examples of pozzolans are: Fly ash (FA), the top ash of combustion plants such as coal fired electricity plants and waste incineration plants; Ground-granulated blast-furnace slag (GGBS), which is produced by quenching and grounding the residual slag from iron production; Silica fume, or microsilica, which is a very fine powder that remains as a by-product from silicon production; Metakaolin, derived from a clay mineral; Rice husk ash (RHA), the ash from burning the rice hulls contains a lot of amorphous silica and is a potent pozzolan (Rajamane et al. 2009).

The initial alkalinity of concrete is dependent of the mix design and in particular the proportions of the binder materials. Since it is Portland cement that gives rise to the formation of $Ca(OH)_2$, any replacement or relative reduction of cement will cause lower initial pH. The $Ca(OH)_2$-reducing effect of pozzolans is twofold. The amount of $Ca(OH)_2$ is directly reduced by limiting the amount of cement, but also by the cementitious chemical reactions.

The pozzolans have slightly different effects in the concrete properties and workability, which were not studied deeper in this project. Generally speaking however, they can be said to have the following effects. They act as fillers improving the workability of fresh concrete by increasing the paste volume without increasing the cement content. They accelerate the strength development of the concrete, giving higher early strength. The final strength is often also increased. The voids in the concrete are more effectively filled both by the chemical reactions and by the more diverse grading of particles made possible by this very fine material. Finally the pH is lowered as discussed above.

If the cement is completely omitted from the concrete mix, replaced in whole by cement replacement materials such as those stated above, the final product is called

“Geopolymer concrete”. These concretes have no significant differences in final properties, but minor differences include slightly lower stiffness, improved tensile strength and lower permeability. Also better bond to reinforcement and faster development of strength can be noticed. The bond of the composite is not based on C-S-H gel as for ordinary cement concretes, but other binding materials following reactions of the pozzolans. To initiate the reaction, an alkaline activator substance must be added to the mix to compensate for the absence of $Ca(OH)_2$ (Rajamane et al. 2009).

2.2.1.2 Form-active structures

If possible, the best solution would arguably be to eliminate any reinforcement altogether. After all, for millennia human civilizations have built purely compressive structures, some of which still stand today. These structures use logs or blocks of stone to resist any tensile stresses in beam action, prevent tensile stress by massive self-weight, or are form-active generating pure compression through its shape. Often the construction methods were very labour intense and in the case of monumental buildings the construction time often spanned several generations.

During most of this time there have been authoritarian regimes and large social class inequality. Through this labour was cheap, or in slave driving societies, even practically free. Consequently labour intense constructions were no problem for those few that had power and resources. Today the situation is different with innumerable social benefits, at least in western countries. Nonetheless it is also clear that some historic buildings would be practically impossible to erect today.

With several decades of extremely fast technological development though, once again it is becoming viable to construct form-active structures, albeit different, slender structures. The construction industry is comparably slow to adapt, but pioneering engineers have for some time experimented with grid-shells, catenary arches and automated manufacturing methods of series of unique construction elements. Complex shapes can rather easily be generated with today’s advanced computer technology. With automation individual numbered mould elements could then be produced in a factory and mounted on site. This could be one way of achieving complex, form-active shape with a minimum of expensive man-hours.

2.2.1.3 High performance concrete (HPC)

There are no clear distinctions between a high performance concrete and a normal one. However so called high performance concretes (HPC) are concretes with one or more enhanced properties compared to normal concrete. It can be compressive strength, workability, early age strength, toughness or density among others. These features may be interesting to work with in combination with different reinforcement approaches and hopefully there are some synergetic effects.

2.2.1.4 Average properties

Although it is hard to generalize such a diverse material as concrete, this section will give an indication of its characteristics.

Mechanical properties

Concrete properties vary very much between different mix recipes and curing conditions. With the exception of laboratory tests, concretes vary from around 10 to 160 MPa in compressive strength. Generally tensile strength is only about 10 % of the compressive. This is why it is often insufficient to resist tension and needs reinforcement from another material. In Table 1 below a summary of some concrete properties are gathered.

Table 1 Mechanical properties of concrete.

Modulus of elasticity	25 - 70 GPa
Tensile strength	1 - 16 MPa
Compressive strength	10 - 160 MPa
Ultimate compressive strain	0.35 %
Density	2400 kg/m^3
Thermal expansion coefficient	12 e-6 ^{o}C^{-1}

Durability

Concrete is very durable with a few exceptions. The major such exception is if materials with expansive corrosion behaviour such as steel are embedded within the concrete body thus exploding the concrete from within. Then there are a number of chemical reactions that can take place under certain conditions; for instance alkali silicate reaction (ASR) and sulphate attacks. Furthermore repeated freezing-thawing cycles can give rise to damages if the solidifying water does not have room to expand within the element. Fire and heat is rather well handled by concrete, but long-lasting intense fires can almost completely destroy concrete. Finally there are mechanical damages such as surface abrasion, settlements and earthquakes (Monteiro 2012).

Price and availability

Concrete is the cheapest option in many applications and it is therefore no surprise that it is used extensively throughout the world. It also means that in most major cities there is a concrete station, so availability is very good. It is hard to put a price on concrete. A large part of the cost lies in handling and transportation. If those costs are excluded a guiding price range could be 100 – 2000 USD/m^3 with a normal construction concrete costing around 200 USD/m^3. The larger the casting is the lower will the overhead costs be per unit of concrete (Jönköpings betong 2007).

Design and execution

Due to the amount of concrete structures being built, most structural engineers are familiar with designing it. Likewise most contractors have concrete teams with high experience. The work on site can be loud and dirty and some concrete types are hard to spread well in the formwork. Both the noise and workability issues have largely been solved by self-compacting concrete that is getting more and more used.

Sustainability
The sustainability of concrete is a disputed issue. However most promoters of its sustainability are cement companies or interest organizations, why the credibility of those voices is questionable.

A benefit with some real substance is that the energy of producing one unit of concrete is several times lower than other materials such as metals and glass. Aluminium, which is renowned for its high energy need in virgin production, requires 200 times more energy per tonne than concrete. Almost all of the energy is consumed in the production of Portland cement. If energy consumption is to be limited, it is hence the cement content that should be limited, which could be done by using other binders. Of course the other materials such as chemical additives and rock derived products have their respective environmental impacts with leakage of toxins and disturbance of the nature respectively. Concrete is furthermore locally produced compared to some other building materials and is an effective heat storing material, thus levelling out thermal fluctuations. This may save energy from indoor climate installations, if some temperature fluctuations are accepted. Correctly used it could also be argued that concrete structures are more durable and could be used for longer time, thus lowering life-cycle resource usage (WBCD, 2014). This argument is not entirely true with concrete structures reaching its end-of-life today as these often fail due to corrosion of steel reinforcement. All structures of any material also have to fulfil the same service life demands and therefore have to be prepared to meet those criteria. Reinforced concrete is moreover seldom designed to exceed the specified design service life. How long an actual life length of today's concrete structures is naturally not entirely clear.

Apart from the energy and carbon footprint generated in the heat intensive production of cement, the chemical process emits a lot of carbon dioxide to the atmosphere. In total about one unit weight of CO_2 is emitted per unit weight of cement produced. Theoretically the part pertaining to chemical emission would be absorbed in the return of the carbon dioxide to the concrete by carbonation. In reality though and especially on short term, not all concrete gets carbonated; especially so if the concrete is dense with low permeability, thus not permitting CO_2 to enter beyond the surface zone of the element. If a concrete member is not fully carbonated at its end-of-life and is put on landfill or as filling material, CO_2 may no longer reach the concrete. Concrete production stands for a staggering 7% of the total CO_2 emissions worldwide, so the possible impact is huge (Rajamane et al. 2009).

Most pozzolanic materials are rest products, even considered waste in some cases. Fly ash for example is generated in enormous amounts; especially in countries with much coal fuelled electricity production. In India for instance, 65% of the total electricity originates from coal. The coal used contains in average 40% ash of which a significant part becomes activated fly ash. This ash is gathered in filters in the chimneys for a good reason. Fly ash contains traces of many harmful substances such as heavy metals. Fine particles released in the atmosphere also have several negative consequences, e.g. damages to the respiratory system and smog. Much of the fly ash is used as fillers in embankments and in other such applications, even though it could be used to replace cement with good results (Rajamane et al. 2009). In that way, the material would not only be taken care of effectively and in line with its merits, but the environmentally straining cement production could be limited.

2.2.2 Fibre reinforced concrete (FRC)

Brittle materials have been reinforced with fibres since ancient times. This practice improves toughness and ultimate elongation in flexure and tension. The naturally present mineral fibre asbestos was widely used until the 1970's before the now well-known health risks of the fibres were understood (ACI 2002). Fibres with diameter below 3 μm can interfere with the human respiratory system and cause severe health problems. Therefore today thicker, non-harmful fibres are used. Asbestos fibres are still used, but not in all countries and only for applications where their excellent heat resistance is required. It should be said however that in concrete, asbestos fibres are not unsafe as long as elements are not damaged or the building is demolished (Hertzberg 2005). Tests of the coarser fibres used today in different situations have shown that even under harsh circumstances the risk for fibrillation of the fibres into harmfully sized pieces is very small (Hertzberg 2005, Singha 2012).

Fibre reinforced concrete is not extensively used as a load-bearing construction material yet and, since researchers generally agree that fibre reinforcement does little for ultimate capacity, it may never be. Its most widespread uses are slabs on grade, façade elements and stay-in-place forms where the fibre's improvements in crack behaviour are in demand (ACI 2002).

A combination of very high performing concretes and fibre reinforcement gives so called Ultra high performance fibre reinforced concretes (UHPFRC). These are about 10 times more expensive than ordinary concretes, but have extraordinary properties (Olson 2014-06-12). The clearest differences are that UHPFRC lacks coarse aggregates and contain super plasticizers and silica fume to allow very low W/C ratios. The coarse aggregate is what normally provides ductility to concrete failure. Therefore fibre reinforcement is needed to provide some ductility to the material. UHPFRC is so dense that is can be considered impermeable and, since cracking is limited thanks to the fibres and the high tensile strength of the concrete, any embedded reinforcement will be very well protected (Khan and Nazar 2011).

The compressive strength is hardly affected by addition of fibres. Depending on the strength of the fibre, its effect may be marginally positive or negative. In direct tension the strength is increased markedly, up to 50%. In flexure the increase is even greater, up to 70%. Primarily though it is the ultimate elongation that is increased and in many cases the ultimate strength is no higher than the cracking load. The modulus of elasticity, abrasion resistance and Poisson's ratio are usually considered as unchanged by addition of small amounts of fibres (up to 2%) (ACI 2002).

Fracture energy is increased about 2 - 4 times with fibre reinforcement. Tests of impact loading response have shown improvements of 40% for steel-fibre reinforced concrete (SFRC). This can be exemplified by results indicating that airport pavements reinforced with steel fibres only needs to be half the thickness otherwise required (ACI 2002). It may be important here to clarify that the first crack strength of FRC is generally not higher than for plain concrete. As with other reinforcement, the concrete section is so much stiffer than the reinforcement, hence the reinforcement comes into play in very limited extension before cracking. Afterward, of course it increases in importance as the whole section now is totally dependent on the reinforcement (Ramakrishnan et al. 1998).

Fibres in concrete limit the workability somewhat by reducing the slump. Both the aspect ratio (relation between length and diameter) and the amount of fibres reduce workability and creates a risk of balling of fibres in clusters. Both these parameters are also determining for the mechanical behaviour. Another very important factor is the bond between fibre and concrete, which affects tensile capacity, toughness, impact resistance and fatigue endurance. Bond also controls ductility, since this is provided by the gradual pull-out of the fibres in the fracture surface.

Fibres are extensively used as controllers of bleeding and plastic shrinkage cracking in slabs. For these applications the fibres do not need to be high performing and the minimum fraction is low, from 0.1%. The fibres do not hinder the shrinkage to take place, but tightens the crack spacing, which in its turn reduces the crack widths. To obtain the mentioned mechanical benefits, the fibres should have high bond and mechanical properties as well as being added in high fractions, in the range of 0.4 – 0.7% of the concrete volume. Fibre to concrete bond can be improved by roughening the fibre surface or introducing hooks or deformations. These measures have to be made with precaution though as they increase the risk of balling (ACI 2002). It should be noted that there seems to be optimum fibre contents with local maxima or plateau values for different properties (Nordlund 2004).

No special equipment is needed for handling and production of concretes with fibres. It has even been shown that some equipment such as concrete pumps work better with some types of FRC (ACI 2002).

Concrete can be reinforced with a number of different fibres. The most prominent are: Steel, glass, basalt, carbon, polypropylene, polyvinyl alcohol (PVA), polyolefin and various natural fibres (Ramakrishnan et al. 1998). In the following, these fibres will be briefly evaluated.

2.2.2.1 Steel fibres

The shape of steel fibres depends on the application and the production method. Some shapes can be seen in Figure 2 below. The higher the bond strength of the fibre, the less amount of fibre is needed to attain the same increases in toughness, shear capacity etc. The aspect ratio of steel fibres is typically in the range of 20 to 100 with lengths of 6.4 to 76 mm (ACI 2002).

To reach higher fibre fractions than the around 2% without compromising workability, steel fibres can be placed in the form and then infiltrated such as is done with ordinary reinforcement. This method is called Slurry Infiltrated Fibre Concrete (SIFCON). The form is filled with steel fibres to the top without packing. Cement-based slurry is then infiltrated by pouring and light vibration. With this method volume fractions of up to 25% have been reached. With such high fractions of steel, the material obtained is quite different from ordinary concrete. Compressive strength can be up to 140 MPa, tensile around 40 MPa and flexural about 90 MPa (ACI 2002).

Steel fibres in the proximity of the concrete surface may become problematic. As they are unprotected there from the corrosive exterior environment they will rust, leaving unpleasant stains on the surface. Furthermore they may be sharp and harmful to animals or tyres. Steel fibres are seldom in contact with each other for normal volume fractions, why no galvanic circuits can form between different units. Any potential difference must be built up within each small fibre. For this reason, corrosion is typically of less importance in SFRC than in steel reinforced concrete.

Steel fibres have not been used much as the sole reinforcing material in flexural concrete members; the ACI even recommends the somewhat conservative design approach that reinforcement bars are necessary to resist tensile forces. This is primarily due to the uncertainty of whether the fibres have been evenly distributed throughout the member. It has however been shown that steel fibres effectively can substitute shear stirrups and secondary reinforcement (Nordlund 2004).

2.2.2.2 Mineral fibres

The same type of fibres used in mineral wool, i.e. rock and glass fibres can be used as reinforcing material. The two most promising fibre variants are alkali-resistant glass fibres (AR-glass) and basalt fibres.

Glass

Ordinary so called E-glass fibres are relatively rapidly destroyed in the alkaline environment of concrete. This can be delayed with varying success by either coating the surface of the fibres, or altering the chemical composition of the material. It was found about 50 years ago that adding zirconia in the glass melt improved alkali resistance and this is still the most important addition material in AR-glass. Another way of increasing the durability of alkali-sensitive fibres in concrete is, as previously discussed, to reduce the alkalinity of the concrete (ACI 2002).

Still, even the best glass fibres available will deteriorate and loose strength over time, especially if the concrete member is exposed to harsh environments. Typical remaining capacities of glass fibre reinforced concrete (GFRC) when fully aged are 40% of initial tensile and flexural strengths and 20% of the initial tensile strain capacity; higher for AR-glass and lower for E-glass. The loss of strain capacity can be seen as a type of embrittlement. AR-glass is presently under development to further improve its resistivity to the conditions in concrete (ACI 2002).

Glass fibre reinforced concrete has so far mostly been used for façade panels and complex geometry applications such as sinks and pipes (Ramakrishnan et al. 1998).

Basalt

Basalt fibres are produced from the common rock basalt, which is formed when lava solidifies. The first tests with basalt fibres were performed by the Soviet Union during the cold war (Prince 2014). Basalt as a reinforcement material for concrete; both directly dispersed and in epoxy rods, is a rather new application and therefore less researched than other fibres. Consequently some uncertainties remain regarding its mechanical properties and durability.

Most information sources place basalt fibres slightly above glass with regard to mechanical properties (Parnas et al. 2007). Dispersed in concrete, the workability and balling resistance is slightly better than with steel fibres (Ramakrishnan et al. 1998).

Some say basalt sustains high alkalinity very well and deteriorates far less than glass fibres (Singha 2012). In other studies it has been shown that it indeed loses less volume due to the concrete environment, but that its strength is more affected even than E-glass. A theory of the reason behind these losses is that there are alkali-sensitive pyroxenes in the chemical composition of basalt (Coricciati et al. 2009).

Much as for glass fibres, there have been attempts to alter the chemical composition of basalt in order to improve its durability properties. In one such experiment Al_2O_3, MgO and TiO_2 were added in different combinations to the basalt melt. A control batch of unchanged fibres was kept for comparison. All fibres were submerged in two liquids. One purely alkaline and another simulating concrete pore water. Both liquids gave similar results. The reduction in strength of the fibres is most dramatic initially and then loses pace gradually. After one week the remaining strength was in average 45% of the initial. As with all accelerated tests, it is hard to make certain predictions of the real behaviour. Some variations were demonstrated both between the unaltered fibres from different sources and between the modified variants. However any large land winnings such as zirconia for glass fibres were not found (Förster and Mäder 2011).

In another study the durability of basalt and E-glass in vinylester and epoxy matrices was analysed in water, salt solution and in freeze-thaw conditions. Here it was shown that the combination of E-glass and epoxy performed best. The stiffness of all other combinations was significantly decreased. Basalt fibres lost much tensile strength whereas E-glass was almost unaffected. The deterioration may happen in the fibre, in the matrix or in the interfacial zone. Regarding the interface, basalt showed less bond strength than glass fibres especially so in vinylester. In epoxy the difference was smaller (Parnas et al. 2007).

To prevent the large losses and embrittlement of mineral fibres in concrete the alkalinity can be lowered in a number of ways. The concrete mix design can be altered, fibres can be made more resistant by varying the chemical composition and surface coatings can be applied on the fibre surface. Such coatings are often epoxy or acrylic and are applied to bundles of fibres, thus making the smallest unit more manageable (ACI 2002, Olson 2014-06-12). The coatings do lower the tensile strength of the fibres a little, but more than compensates for it by decreasing the rate of deterioration. A special case is the AR-glass fibre where the initial application loss supersedes the end gain. This is primarily due to the already low rate of degradation of these fibres. The outside environment also has an effect where increased moisture and temperature speed up the degradation (Coricciati et al. 2009).

2.2.2.3 Synthetic fibres

Synthetic fibres are tailor-made human designed materials produced from organic polymers. Types of synthetic fibres that have been tried in FRC are: Acrylic, aramid, carbon, nylon, polyester, polyethylene, polyolefin, polypropylene and polyvinyl alcohol (PVA) (ACI 2002). In Table 2 their respective properties can be seen.

Table 2 Synthetic fibre properties (ACI 2002).

Fiber type	Equivalent diameter, in. x 10^{-3}	Specific gravity	Tensile strength, ksi	Elastic modulus, ksi	Ultimate elongation, percent	Ignition temperature, degrees F	Melt, oxidation, or decomposition temperature, degrees F	Water absorption per ASTM D 570, percent by weight
Acrylic	0.5-4.1	1.16-1.18	39-145	2000-2800	7.5-50.0	—	430-455	1.0-2.5
Aramid I	0.47	1.44	425	9000	4.4	high	900	4.3
Aramid II†	0.40	1.44	340	17,000	2.5	high	900	1.2
Carbon, PAN HM‡	0.30	1.6-1.7	360-440	55,100	0.5-0.7	high	752	nil
Carbon, PAN HT§	0.35	1.6-1.7	500-580	33,400	1.0-1.5	high	752	nil
Carbon, pitch GP**	0.39-0.51	1.6-1.7	70-115	4000-5000	2.0-2.4	high	752	3-7
Carbon, pitch HP††	0.35-0.70	1.80-2.15	220-450	22,000-70,000	0.5-1.1	high	932	nil
Nylon‡‡	0.90	1.14	140	750	20	—	392-430	2.8-5.0
Polyester	0.78	1.34-1.39	33-160	2500	12-150	1100	495	0.4
Polyethylene‡‡	1.0-40.0	0.92-0.96	11-85	725	3-80	—	273	nil
Polypropylene‡‡	—	0.90-0.91	20-100	500-700	15	1100	330	nil

Many of these fibres have been little used in practice. Several of them have low durability in concrete and are only functional in fresh concrete by reducing plastic shrinkage cracking. Some others are indeed good, but are so expensive that extensive usage of them dispersed in concrete is unlikely. These include aramid and carbon, which are only used in certain high strength applications given today's price levels. Quite small volume fractions of synthetic fibres, from as low as 0.1% can make large differences in plastic shrinkage cracking and this is the most common application of synthetic FRC today (ACI 2002).

Carbon fibres have extraordinary volumetric stability. It reacts very little in contact with water and heat. For this reason and thanks to its remarkable stiffness and strength, carbon fibres are a good limiting agent of shrinkage and creep deformations. As has been shown in many other applications, carbon fibres are almost inert to the environment within concrete. The few available examples with carbon FRC are often doubly-curved complex structures with high mechanical demands (ACI 2002).

Apart from the above treated aramid and carbon, most synthetic fibres are only moderately stiff and strong but are instead much cheaper. They still offer considerable increases in toughness, impact resistance and ductility compared to plain concrete. Additionally they distribute tensile strains to finer, more closely spaced cracks. Some synthetic fibres with low melting points are also added in concrete in order to create gas expansion channels for the event of fire. Of these fibres, polypropylene stands out as well researched and relatively much used in practice (ACI 2002).

2.2.2.4 Natural fibres

There are several plant fibres that have been used by humanity for ages, for example linen (flax), cotton, wood and jute. These fibres are often available in large quantities and are increasingly becoming waste products rather than an asset. The core of most of these fibres is cellulose fibres, which in its pure form has very high mechanical properties. Its tensile strength can be as high as 6400 MPa, although whole plant fibres never reach that level. One of the more promising natural fibres is linen with a Young's modulus of 100 GPa and ultimate tensile strength of 1000 MPa. Most other available sources including coconut, sisal, sugar cane, bamboo, jute, elephant grass and wood fibres have between 10 and 75% of flax's properties (ACI 2002). The high strength of flax fibres has to do with its high fraction of cellulose, which is around 70%. The high cellulose content also gives flax better durability than other fibres as it is primarily other compounds than cellulose that is broken down by alkaline (Islam, 2014). Nevertheless, even flax fibres deteriorate rather much when evaluated in accelerated tests. In one test 86% of the fibre weight was lost after 112 days in alkaline solution. When used in concrete though, the deterioration is much slower. It has been shown that when the natural fibres have lost their mechanical properties and remain useless in the matrix, they do not weaken the properties of the remaining concrete. This makes natural fibres an attractive choice for limitation of plastic shrinkage cracks. Attempts with protective coatings or chemical treatments on flax fibres have proven futile so far (Wegner 2011).

For significant result, the fibre volume fraction needs to be higher for natural fibres than for steel, mineral or synthetic ones, usually in the order of 3%. This affects workability negatively and problems with balling have been noticed. Cement replacement materials reduce alkalinity and may improve the longevity of natural fibres in concrete substantially. In Africa natural fibres have been used in concrete for some time and the durability in practice actually seems to be good (ACI 2002).

2.2.3 Steel

Steel is a remarkable reinforcement material as it combines high strength, modulus of elasticity, homogeneity and a relatively low price. Reinforcement steel is primarily produced in plain and ribbed bars and welded meshes (Al-Emrani et al. 2011a). The main downside to using ordinary steel is its expansive corrosion behaviour. Steel is highly recyclable and is therefore recycled to a large extent in practice. In concrete applications it is for natural reasons hard to retrieve the bars for recycling, but that goes for any material directly cast into concrete. Nevertheless, as can be seen in Figure 2 below, in China and other countries with cheap labour steel is sometimes retrieved from concrete rubble at demolition sites.

Steel corrodes in four modes, general, intergranular, pitting and crevice and stress corrosion. To ensure durability all of these modes must be prevented (Nürnberger 1996). In young concrete steel is protected by the high alkalinity; as previously mentioned though pH gets lower with time as the concrete reacts with CO_2. Ingress of chloride ions is the other main initiator of corrosion and appears primarily where chloride is abundant in the environment around the structure such as on highways with de-icing salts or in marine structures.

If the steel bars could be insulated from the environment in the concrete by some third material, it would not matter if the concrete at the steel level got carbonated or the chloride concentration too high. Three main coatings are used: Epoxy, zinc and stainless steel. These are evaluated in Sections 2.2.3.2, 2.2.3.3 and 2.2.3.5 below.

Figure 2 Retrieval of reinforcement steel from rubble (Frodesiak 2012).

2.2.3.1 Carbon steel

Ordinary steel, or carbon steel, is by far the most used reinforcing material in concrete. In essence it is an alloy of iron and carbon, often with small concentrations of other metals and substances. Due to its widespread use it is the natural reference point for the other alternatives discussed in this thesis. Typical material properties of steel can be seen in Table 3 below.

Mechanical properties

Table 3 Mechanical properties of carbon steel (Al-Emrani et al. 2011a).

Modulus of elasticity	200 GPa
Tensile strength (yielding)	500 MPa
Compressive strength (yielding)	500 MPa
Ultimate compressive strain	15 %
Density	7800 kg/m^3
Thermal expansion coefficient	12 e-6 $^{o}C^{-1}$

Durability

Steel is a refined product that according to natural laws is driven toward less noble but more stable states. Therefore naked steel oxidizes in an expansive reaction to form rust. In concrete steel is protected by the alkalinity of the pore solution that passivates the steel surface, greatly impeding the corrosion current for pH above 10 (BE Group 2012). With time, concrete loses its alkalinity at the same time as ions and other aggressors penetrate the concrete cover. When the steel responds by corroding and thus expands, it damages the surrounding concrete. In the worst case this results in spalling of large pieces of the concrete cover. This process is widely visible on structures all over and causes massive repair costs as well as service interruptions. If the embedding concrete could be made sufficiently protective, the requisites for corrosion could be hindered for reasonable service life periods (Olson 2014-06-12). Some suggest that for most applications, this quality level has been reached aldready and that concrete structures built today will not deteriorate prematurely (Svensson 2014-06-25).

Bond

Bond is excellent between steel and concrete and with the exception of plain bars and in prestressed concrete applications bond failure rarely governs failure.

Price and availability

Steel is a relatively cheap material that is produced in all parts of the world. As steel is an alloy and varies quite a lot in quality there is no conform spot price as for pure metals. For the purposes of this project, the price of hot-rolled reinforcement bars is what is most interesting. An average non-discounted list price from Stena stål (2014), a major steel company in Sweden, is about 2 USD/kg.

Design and execution

Since steel is the most used reinforcement material in concrete, there is a vast experience of using it in the construction industry. Everything from computer programs to cutting tools is readily available. Because of its high density, steel is heavy to handle and transport. Care has to be taken to ensure sufficient concrete covers. The design of ordinary steel reinforced concrete is very well developed and described in design codes such as Eurocode 2 and ACI 318.

Sustainability

Mining implies large environmental disturbance, both for ecosystems and people close to the mine. It is clearly exemplified by the necessity to move the whole town of Kiruna in the north of Sweden because of the mining activities that the town grew up around at one time.

2.2.3.2 Epoxy coated steel

Epoxy coating is a way of passively protecting the steel surface from a potentially harmful environment. The epoxy is applied on the hot rolled reinforcement bars by fusion bonding or electrostatic spraying. Typical thicknesses range from 150 to 300 μm (Sederholm 1996).

Mechanical properties

With regard to mechanical properties, except for bond to the concrete, epoxy coated steel bars can be designed in the same manner as carbon steel, as steel is the only load bearing component of the product.

Durability

If corrosion starts, it generally appears around a small imperfection or tear in the epoxy cover. The resulting process is similar to crevice corrosion and often spreads from the initiation zone to the still covered surroundings where the epoxy coating gets scaled off, which puts the bond to the surrounding concrete at risk. This type of corrosion is self-polarized and is therefore not dependent on electrical connection with other reinforcement bars. The point of initial damage may suffer large losses of cross-section due to the highly localized corrosion. These risks have been considered as so significant that epoxy coated bars are more unsafe than naked bars. (Kahhaleh et al. 1998, Sederholm 1996).

Completely undamaged epoxy coated steel rebars do provide long-lasting corrosion protection. If damaged though, the bars should be avoided. In practice it is of course very hard to guarantee that all placed reinforcement bars have no surface damages. For this reason the use of this type of reinforcement in questionable (Sederholm 1996).

It has been shown that smaller diameter bars have better corrosion resistance than larger diameters and also that the permeability of the surrounding concrete, as always, plays an important role in the corrosion resistance (Kahhaleh et al. 1998).

Bond

The surface of epoxy is glossier and smoother than that of steel. This results in a 20 – 50 % lower bond strength for epoxy coated bars compared to uncoated. This is in the same order of magnitude as the difference between plain and deformed steel bars. As with all reinforcement materials, weaker bond will lead to wider cracks at larger spacings (Yeomans 2004).

Price and availability

The cost of epoxy coating and hot-dip galvanization is about the same and the end product can simplified be said to cost 1.5 times that of ordinary reinforcing steel. With the above stated steel price that would mean 3 USD/kg. Epoxy coated bars have been used in Scandinavia since the 1980's, but its usage has never become widespread. In the US however, it is in use and is recommended by several states' transportation administrations (Federal Highway Administration 1999).

Design and execution

Epoxy coated bars have to be handled with more care than uncoated bars. They must be tied with plastic straps and should not be dropped or dragged on the ground. The reason is that the epoxy is rather sensitive to abrasion. Should the protective cover be damaged, the corrosion protection is vastly diminished. It has been shown in accelerated tests however that in total, bars with damaged epoxy coatings still corrode at a slower pace than completely naked bars (Kahhaleh et al. 1998).

In the beginning of the use of epoxy coated rebars in the 1970's, it was thought that minor damages to the cover were of little importance. Since then the maximum allowed extension of damages has been limited in recommendations. There have been several attempts to mend local damages, both in practice and in laboratory tests.

Several studies have shown that it does not work well though, in that corrosion will start at the mended damage relatively soon. Three reparation methods have been evaluated: Epoxy paint, taping or epoxy melting rods. None of them have proven to work satisfactorily. Thus there is no way of properly mending damaged epoxy coating to date (Kahhaleh et al. 1998, Sederholm 1996).

The ordinary structural engineering codes mentioned above can be used provided bond strength is reduced as there are no other major differences. Concrete cover should not be decreased if the goal is prolonged service life. Tight bends may damage the outside of the epoxy cover. Therefore bends after the coating is applied need to be performed with generous radii.

Sustainability
Handling epoxy can be harmful and, being an artificial polymer, its manufacture does include many potentially harmful chemicals. The underlying steel can be recycled as any steel, given that it can be extracted from the surrounding concrete. This is something that actually ought to be slightly easier than for ordinary steel bars.

2.2.3.3 Galvanized steel

Galvanization of steel is a much used corrosion protection for a number of applications. For use in concrete however, the method has not become widely embraced. It is natural to compare it with epoxy coating, since the price is about the same for the two. Some tests have put forward that epoxy coated bars last longer than galvanized steel in simulated marine environments (Nürnberger 1996). As discussed in Section 2.2.3.2 it seems that as long as the epoxy coating is intact, it provides excellent protection. However, outside of the laboratory it is practically impossible to avoid damages on the epoxy. Galvanization will never reach the high protection levels of epoxy in a laboratory, but the protection is predicTable and sturdy. Galvanization of reinforcement bars is usually performed by hot-dipping. This is performed by submerging the whole steel element in a bath of molten zinc. Zinc melts at around 420 ^{o}C and is thus relatively economic to apply. This temperature is also well below levels that could damage the steel. The layer thickness of zinc on structural members is usually >180 μm and it binds metallurgically to the underlying steel, forming an extremely strong bond. This in combination with the hardness of zinc renders excellent wear resistance, wherefore galvanized rebars can be handled as ordinary reinforcement bars. This also includes the possibility to bend rebars on site (Yeomans 2004).

When cast into concrete, the zinc surface reacts with chromates present in the cement paste. In this process about 10 μm of the coating is consumed. The reaction products form a protective passivating layer (AGA 2014).

Mechanical properties
As for epoxy coated bars, the mechanical properties of ordinary reinforcing steel can be assumed for galvanized reinforcement. In this case though, also bond can be assumed to be the same as for ribbed reinforcing steel bars (AGA, 2014).

Durability

After the passivating layer is consumed by corrosion, the zinc layer is used up by the corrosion circuit until the first bare steel appears. Until then the steel has been protected much in the same way as for epoxy coating. When the first naked steel gets exposed though, zinc continues to offer protection in that it is a metal less noble than steel. In this way it will act as an offer anode for the reinforcement bar. This protection can extend up to 10 mm from the nearest zinc, thus protecting a bare steel area of up to 20 mm in diameter (Yeomans 2004). This same action will protect surface scratches in the zinc surface; wherefore minor damages to the coating is no big issue.

It has been shown in several studies that galvanization of steel rebars postpones the initiation of steel corrosion in concrete. The endurance of the protection depends on concrete cover, concrete density and tightness, environmental conditions and zinc layer thickness. In environments where steel would otherwise corrode, so will galvanized reinforcement eventually. Galvanization can therefore not be seen as a permanent inhibitor of corrosion. Instead it should be seen as a way of prolonging the service life of a structure. In cost estimations, it has been assessed that one major overhaul of a concrete structure costs more than the added investment that galvanized reinforcement brings with it (Nürnberger 1996, Yeomans 2004).

Zinc is slightly more sensitive to alkalinity than steel, but much more tolerant to acidity. It is considered passive in the range of pH from 8 to 12.5, but tolerates acidity down to pH 6 and alkalinity up to pH 13 with just limited corrosion. For this reason it resists carbonation well. It may also be recommendable to limit the content of the alkaline $Ca(OH)_2$ in the concrete by e.g. adding a pozzolan. Chloride ions are also better tolerated by zinc than by steel. Approximately 2.5 to 5 times higher chloride concentrations can be withheld (AGA 2014, Yeomans 2004).

The corrosion products that are formed when zinc corrodes are less voluminous than iron oxides. In addition they have the ability to migrate away from the rebar up to 0.5 mm into the concrete pore structure. This reduces strain on the surrounding concrete and decreases risk for spalling. It has even been shown that zinc corrosion products reduce the permeability of the concrete closest to the steel, thus slowing down the corrosion current (Yeomans 2004).

Bond

Bond of galvanized steel is as good as for ordinary reinforcing steel or in some tests even better. It is therefore safe to use the standard design procedure in structural engineering codes (Yeomans 2004, AGA 2014).

Price and availability

Galvanized steel reinforcement bars are of about the same price as epoxy coated bars. As previously stated, they are both roughly 1.5 times the price of ordinary reinforcing steel, i.e. 3 USD/kg. Available to the wider market are reinforcement wire fabrics and special details.

Design and execution
Since the zinc coating is tough and durability is not dependent on a scratch-free surface galvanized steel can be handled like ordinary reinforcing steel, i.e. with no special care. Although rusting of reinforcement bars on site is no structural problem, it can still be seen as an advantage that galvanized steel will stay rust-free until casting.

Design can be performed without any alterations compared to uncovered steel. The concrete cover should not be reduced if the aim is longer service life since zinc is not inert, but provides protection for a limited period depending on the environment.

Sustainability
Zinc is not recycled to the same extent as other materials; about 75 % comes from virgin sources. It is however probable that recycling will increase. A zinc coating does not impede the recyclability of steel and there are processes with which the zinc coating also can be retrieved (IZA 2011). As with all materials cast into concrete, the first separation from the structural member is the biggest challenge.

2.2.3.4 Stainless steel

Stainless steel includes many different alloys. What defines them as a group is that they are all iron alloyed not only with carbon, but also with several other metals which provides corrosion resistance to the alloy. To be called stainless, a steel alloy must contain more than 12% chromium. Other common additions are nickel, molybdenum, titanium and nitrogen. Stainless steels are further broken down into sub-categories. The relevant groups for concrete reinforcement applications are ferritic, austenitic and ferritic-austenitic (duplex). Ferritic is the least alloyed and ferritic-austenitic the most. As all alloy compounds are more expensive than iron and carbon, the more alloyed a stainless steel is, the costlier it becomes. Normally, the more expensive types are also more durable. Durability and mechanical properties are not necessarily influenced in parallel, but there are ways to attain very strong and stiff steel grades that are also highly durable (Nürnberger 1996). Aside from the steel grades enclosed in the concept of stainless there are also less alloyed steels such as for example a product called MMFX. This is a low carbon steel with some added chromium. Thanks to production methodology, it also exhibits higher strength than ordinary reinforcing steel (MMFX Steel 2014). The corrosion resistance is however not as good as for fully stainless steel and the product is probably more suited as an alternative to galvanized or epoxy coated reinforcement. Another possible field of application is where better mechanical properties are required.

The most common corrosion mode of stainless steel in concrete is pitting and crevice corrosion. The other three modes of corrosion presented in Section 2.2.3 are generally effectively hindered by the alloy. In order to compare the corrosion resistance of stainless steel, the so-called pitting resistance equivalent (PRE) number is used. It is calculated as (Nürnberger 1996):

$$PRE = (1 * \%Cr + 3.3 * \%Mo) \tag{2.1}$$

Some available grades of stainless steels and their PRE-numbers are presented in Table 4. The two left-most columns present two different denotations for each

Table 4 Stainless steel grades (Nürnberger 1996).

1.4003	X2CrNi12	ferritic	:11
1.4016 (=430)	X6Cr17	ferritic	:17
1.4301 (=304)	X5CrNi 18–10	austenitic	:18
1.4541	X6CrNiTi 18–10	austenitic	:18
1.4401 (=316)	X5CrNiMo 17–12–2	austenitic	:24
1.4571	X6CrNiMoTi 17–12–2	austenitic	:24
1.4462	X2CrNiMoN 22–5–3	ferr.-austen.	:32

material type, the third column gives the category of the steel grade and the right-most column contains the PRE-number of the variety.

The way that corrosion is prevented in stainless steel is through a chemical process in which a film is created on the surface of the bar. The compound of the film consists of mixed oxides of iron and the addition metals. Damages to the surface such as scale and temper colouring due to welding may aggravate pitting corrosion and should be prevented by removing any such imperfections (Nürnberger 1996).

Mechanical properties

With up to 30% other materials than iron in stainless steel, of course the mechanical behaviour is affected. However, through research and development strength, ductility and other important properties has been developed alongside with durability. Because of this most stainless steels have properties as good as or better than ordinary carbon steel. There are variations and some grades are even significantly better than carbon steel. In this project though, for simplicity, it has been assumed that the properties in Table 3 are valid also for stainless steel (Nürnberger 1996). This assumption is on the safe side except for thermal expansion, which in highly alloyed stainless steels is higher than for carbon steel, thus putting more strain on the surrounding concrete under temperature fluctuations. Instead of 12 e-6 $^{o}C^{-1}$, 18 e-6 $^{o}C^{-1}$ should be used for austenitic steels (Nürnberger 1996).

Durability

The corrosion resistance of the different grades varies a lot, which can be seen in the PRE values in Table 4 above. The price is closely linked to this resistance and these conflicting parameters must be balanced individually for each project.

Welding is easily performed in stainless steel, but does have a negative impact on the corrosion resistance. The critical chloride content for welded zones can be reduced by up to 50 % compared with intact zones. Cold worked stainless steel is not recommended to be welded at all since the effect of the cold working can disappear when the material is heated. If the surface affected by the weld is pickled or otherwise treated, the decrease in performance can be limited (Nürnberger 1996). In Table 5 the effect of welds in combination with chlorides and carbonation can be seen. The results are from 2.5 years of open air exposure.

Table 5 The effect of welding with carbonation and chlorides (Nürnberger 1996).

Steel	Concrete	Normal-weight-concrete				None-dense-concrete	
		Alkaline			Carbon.	Carbonated	
	Cl^- M.–%[1]	0	0.12	0.3	0	0	0.3–0.7
Unalloyed	Unwelded		Moderate	Severe	Moderate	Severe	Very severe
	Welded		Moderate	Severe	Moderate	Severe	Very severe
Ferritic 12Cr	Unwelded						Moderate
	Welded		Moderate/Severe	Severe/Very severe			Very severe
Austenitic 17Cr–12Ni–2Mo	Unwelded						
	Welded						

[1] Chloride content in concrete

None Moderate Severe Very severe corrosion

Stainless steel is nobler than carbon steel and could therefore accelerate galvanic corrosion of ordinary reinforcing steel. It has been shown in studies that this effect is negligible in normal conditions, but some uncertainty remains for very humid and warm climates. While ordinary steel remains passive, the difference in potential is low. Once bare steel gets exposed though, the potential difference increases and corrosion may be accelerated by the stainless steel. In mixed reinforced members, care must therefore be taken to avoid this situation (Sederholm 2013).

In a study conducted by Swerea KIMAB, five different grades of stainless steels were cast into concrete with contents of 0-10% chloride per binder weight and additionally partly submerged in sea water for 2 years. When extracted, all five stainless grades were completely undamaged for chloride contents of up to 3%, whereas the ordinary reinforcing steel bars were heavily corroded. Even for the highest chloride contents of 10 %, the stainless bars looked undamaged on images in the report (Sederholm 2013).

The corrosion currents were recorded. Although no difference could be seen between the grades with the naked eye, small differences in corrosion current were observed. The grades performed in the following order 1.4301 < 1.4162 < 1.4436 < 1.4362 < 1.4462, with 1.4462 being the best. It is noteworthy that no ferritic steel grade was included in the test (Sederholm 2013).

The long term performance of austenitic stainless steel reinforcement has been tested in a study with 7 years of exposure to chloride contents of up to 4.8%. The test included hot-dip galvanized, epoxy coated, stainless steel coated and solid stainless steel rebars. Galvanization worked as a delayer of the corrosion whilst the zinc was consumed; thereafter it behaved as ordinary steel. Epoxy coating worked slightly longer, especially in lower chloride concentrations but when the corrosion starts, the consequences were graver as the resistance broke down systematically. Both the solid and covered stainless steel bars lasted the entire test even with the highest chloride concentration (Nürnberger 1996).

The less alloyed, cheaper ferritic grades of stainless steel reinforcement can be used successfully in moderately aggressive environments, with much better resistance than ordinary reinforcing steel. However, once the chloride content per binder weight exceeds 2%, pitting corrosion has been identified. This effect is further increased if the concrete at the steel level is carbonated (Nürnberger 1996).

In Figure 3, critical levels of chlorides for different stainless steel grades in concrete can be seen.

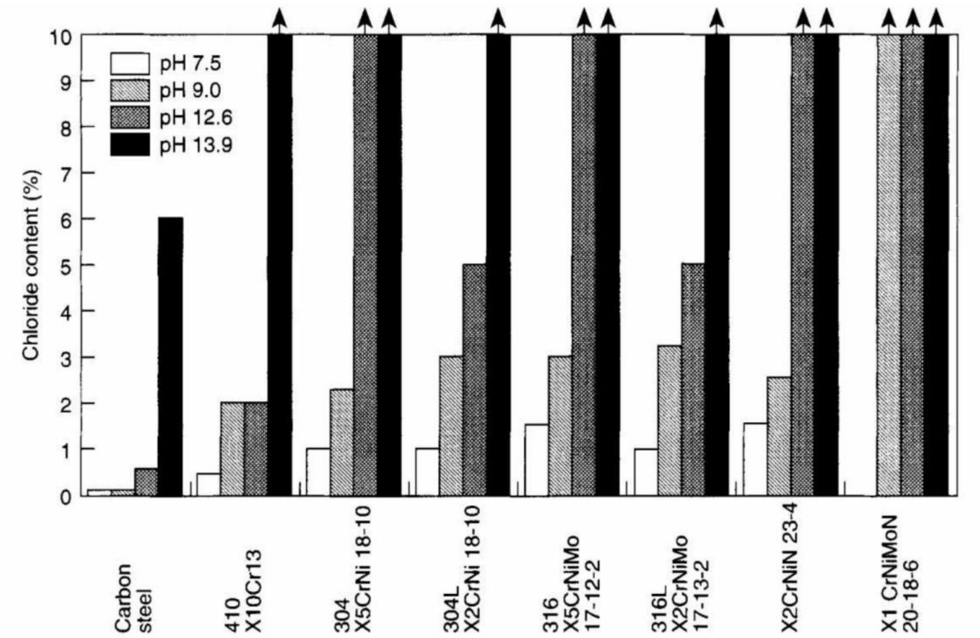


Figure 3 Critical chloride content for different stainless steel grades (Nürnberger 1996).

Bond

Stainless steel bars can be produced with the same ribs and deformations as ordinary steel reinforcement and the surface characteristics are not significantly different, wherefore any bond strength differences can be neglected. The small increase in bond strength that moderately rusted steel bars provide will however not appear. This increase in strength is however not considered in design in most applications anyway. In conclusion, bond issues can be treated as for ordinary steel reinforcement bars.

Price and availability

Prices of the included materials are volatile. However current spot prices can give an indication of their mutual price relations, see Table 6 below.

Table 6 Indicative metal prices (Stena Stål 2014, LME 2014).

Metal	Price USD/kg
Reinforcing steel	2.0
Chromium	9.5
Nickel	18.8
Molybdenum	32.0
Titanium	19.8
Zinc	2.1

Prices of stainless steels are in close relation to their constituent metals. As can be clearly concluded from Table 6, any addition to ordinary steel will increase the cost of the alloy. Final prices will also have an influence of the production process and stock-keeping of the different grades. In fact, different finishing methods of steel rods render prices with over 30% difference (Stena Stål, 2014). It can be seen in Table 7 that the real market price is about twice the theoretical, which is based solely on material cost.

Table 7 Stainless steel indicative prices.

Stainless steel grade	Price theoretically USD/kg	Pricelist, Stena USD/kg
Unalloyed	2	2
X2CrNi12 (1.4003)	2.9	-
X6Cr17 (1.4016)	3.3	-
X5CrNi 18-10 (1.4301)	4.5	8.0
X5CrNiMo 17-12-2 (1.4401)	5.9	-
X2CrNiMoN 22-5-3 (1.4462)	5.4	12.9

To sum up, stainless steel grades used as reinforcement are around 2 to 7 times as expensive as ordinary reinforcing steel. In numbers that would mean 6 USD/kg for ferritic, 8-14 USD/kg for austenitic and 13 USD/kg for ferritic-austenitic steel.

Stainless steel reinforcement bars are used in some applications, e.g. in masonry and concrete repair work in Sweden and elsewhere. In Figure 4, a connection detail for prefabricated sandwich elements can be seen. In North America, stainless steel bars have furthermore been used as structural reinforcement in new production of several civil engineering applications.

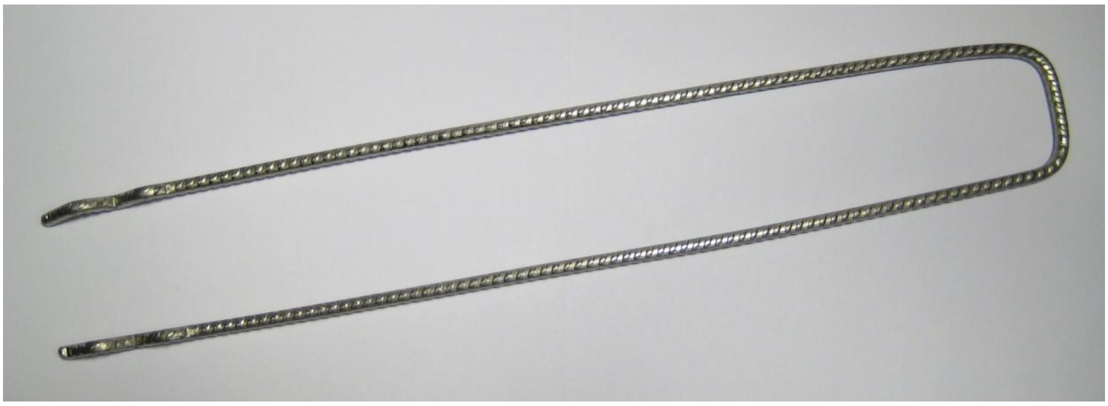


Figure 4 Stainless steel reinforcement for connection of the two concrete panels in prefabricated sandwich walls.

Design and execution
The procedure on site is mostly the same as for ordinary rebars, the difference being the higher sensitivity to and complexity of welding. Stainless steel bars are not especially sensitive to surface damage, as the passivating layer is then recreated without large material losses.

Most stainless steel grades have higher yield strength, ultimate strength and ultimate strain than carbon steel. This could be taken into account given specified data from the material supplier. As a time saving and in almost all aspects safe lower bound approach, carbon steel properties can be used.

Sustainability
Mining and processing of metals have a clear environmental impact; however the recycling rate of stainless steel is high. The problem as with all reinforcement is retrieving it from the concrete in an economic way.

2.2.3.5 Stainless steel coated steel

Stainless steel, being up to 7 times as expensive as ordinary reinforcing steel, makes material optimization interesting. As one measure, there have been attempts to cover carbon steel in a thin layer of stainless steel. However no economic way of applying the coating has been found and the material savings have so far been almost entirely consumed by the increased production costs (Nürnberger 1996).

Stainless steel covering is to compare with chromium or epoxy covers rather than zinc, since its protection is passive. This means that if there is a small damage to the covering layer, corrosion will soon be initiated and spread. In tests with intact covers, very good results have been retrieved (Nürnberger 1996). However if the bars are e.g. cut to shorter lengths than originally produced, the cut ends are put at risk and must be manually protected. This is a manual task which adds costs on site. The bars would also have to be handled with increased care to avoid damage of the surface, in order to keep the steel core protected. Surface imperfections can also be created in the factory, if the layer of stainless steel is unevenly applied (Magee and Schnell 2002).

2.2.4 Fibre reinforced polymer (FRP) reinforcement

Fibre reinforced polymers (FRP) are composite materials consisting of unidirectional fibres surrounded by a resin matrix. The materials vary largely within the group with different combinations of fibre and matrix. All in all, FRP represents a group of very differing properties and unlike most other materials it can be designed according to demands. Yet there are many similarities, especially when some of the more unsuitable FRPs for construction purposes are excluded. In the first part of this section, the material is therefore evaluated as one, after which possible variations in each variety are further examined.

2.2.4.1 General

Composite materials are not a new phenomenon. Textiles have been reinforced with metal threads and natural resins and bitumens have been used to glue together sheets of metal. In the construction industry mud and straw is a composite and masonry is actually a (macro-scale) composite as well. It was not until after the Second World War though that plastics became really commonplace and soon also started to be reinforced by fibres (ACI 2006, Zoghi 2013).

When fibre reinforced plastics had started to become used in many different applications including aerospace, marine and sports, it still took some time before it was seriously considered for use as concrete reinforcement (ACI 2006). During the same period the road network was vastly extended due to the growth of motor vehicle traffic. In regions with snowy and icy winters, de-icing salts started to be used. Soon enough it was discovered however that this practice caused reinforcement in the infrastructure to corrode. Since then very much research indeed has been put into understanding the behaviour of corrosion in reinforced concrete. It took yet a while, until around the late seventies, before FRP rebars were commercially developed as one contribution to the fight against corrosion. From this time and until about the mid-nineties, Japan lead both the development and usage of FRP reinforcement in concrete. In Europe, Germany leads the development with its first FRP reinforced highway bridge being built in 1986. Since the mid-nineties Canada has arguably been the leader of the development globally, with extensive research efforts connected to the ISIS institute and the Canadian highway administration (ACI 2006, ISIS 2003).

Considering the number of produced bridges and buildings using FRP rebars, North America is yet again the global leader. Over 200 highway bridges have been constructed to date with full or partial FRP reinforcement in the continent (Benmokrane 2013). The interest here for corrosion resisting alternatives to ordinary reinforcing steel has been high for a long time. There solutions such as epoxy coated, stainless and galvanized steel have been used to a higher extent than in Europe to date. Including FRP in this palette may therefore not have been as distant in North America as in Europe, where more effort has been put into hindering corrosion by improving the concrete and increasing concrete covers.

Materials

The typical FRP reinforcement rod is produced by pultrusion. This method is the most efficient for production of equal cross-section elements and can therefore be compared to hot-rolling of metals in some aspects. Fibres are sourced from spools in constant motion, dipped in resin and then clustered into the desired shape. The surface can then be modified with ribs and /or coatings before the bars are cured and cut. A schematic illustration of the process can be seen in Figure 5.

Typically a structural FRP for concrete has fibre fractions of about 50 - 80 %, why the behaviour is primarily governed by the fibre properties (ACI 2006). Higher fibre content is typically desirable as the matrix has inferior mechanical properties, hence decreasing the properties of the composite.

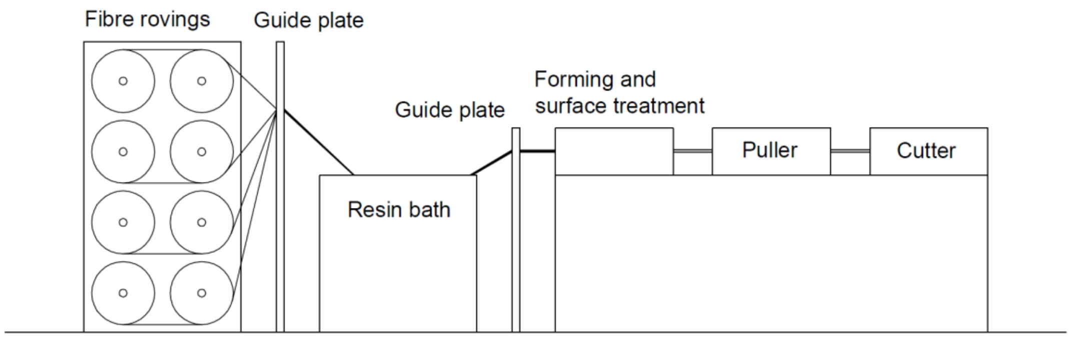


Figure 5 *Schematic illustration of the pultrusion process*

The polymer matrix is in almost all concrete reinforcing applications a thermoset resin, but thermoplastics are also used to some extent. A thermosetting material develops cross-linked chemical bonds during its curing in an irreversible process. Hence, once shaped, a thermoset based FRP cannot be remoulded. A thermoplastic material on the other hand is softened by heat and sets when cooled. This property is used in production of the composite and, with care, the end product can be reshaped with a simple application of heat. To achieve bends in thermoset based FRP reinforcement this has to be done in the factory before curing. This is readily offered by FRP manufacturers to an increased cost of around 20 % (Nanjing Fenghui 2014-06-25). The bent part of the bar is however weakened locally to as much as to 50 %, which must be accounted for in design (ACI 2006, ISIS 2003).

There are obvious resource conserving benefits to the repeatable nature of thermoplastics as the matrix and fibres can be separated and reused. A thermoset based FRP in contrast cannot be recycled effectively to date (ISIS 2007).

What provides strength to the composite are the fibres. The matrix is there to distribute and transfer stresses between the individual fibres and not least to protect the often sensitive fibres. FRP reinforcement is designed to resist tensile stresses and is not very suiTable in compressive applications. Therefore increased tensile strength and modulus of elasticity of the fibres are the most important parameters for the composite's behaviour, although the stiffness of the matrix is not unimportant (ACI 2006).

The most commonly used matrices are epoxy and vinylester, which are thermosetting and polyester that is thermoplastic. Epoxy is the stiffest whereas vinylester is generally considered to provide the best fibre protection (Tepfers 1993). Polyester is the least stiff and also offers the least protection for the fibres, but is in turn the cheapest and has the benefits of being a thermoplastic as previously mentioned. Some design codes have even banned the use of this matrix because of its low fibre protection (Zoghi 2013).

The most commonly used fibres and the ones considered in this project are: Aramid, basalt, carbon and glass. The composites based on these fibres are abbreviated AFRP, BFRP, CFRP and GFRP respectively.

Mechanical properties

The most remarkable differences between steel and FRP reinforcement is that FRP does not yield, but stays linearly elastic up to failure and also that its density is so much lower, namely about 20 - 30 % of that of steel. Tensile strength is, with a few exceptions, higher than for steel whereas modulus of elasticity is lower, also this with some exceptions. Deformations and cracking will therefore be the decisive limit state more often with FRP than with steel (ACI 2006). One way to compensate for the low modulus and at the same time make use of the typically very high strength of FRP is to prestress the reinforcement. Since the stiffness of most FRP prestressing-tendons is lower than those made from steel, another benefit of prestressing is that the effects of concrete shrinkage and creep has less relative effect on the prestressing force (Tepfers 1997).

Pultruded FRP bars have a distinct anisotropic behaviour. In the transverse direction of the bar, fibres are loaded perpendicular to their axle of extension, where they are close to useless in the composite. There are ways of introducing transversally oriented fibres but, where this has not been done, the matrix has to step in and support any stresses. As it is several times weaker and less stiff than the fibres, the shear strength of FRP reinforcement bars as a whole is quite low. Steel in contrast is isotropic with the same material properties in all directions and a large part of its tensile strength can be counted with in shear action.

Another difference in FRP compared to steel reinforcement is that the diameter of the bar influences strength and stiffness, with smaller diameters performing better. A doubling of bar diameter can decrease the tensile strength between 2 – 40 %. The effect behind this phenomenon is shear lag, i.e. that the centre of the bar is less stressed than the surface due to the way the force distributes within the bar. For this reason FRP bars over 25 mm diameter are not recommended without prestressing (ACI 2006, Zoghi 2013). One way of coming to terms with this problem and at the same time improve the relative bond strength is to use bars with hollow cores. Properly connected, the resulting channels can additionally be used for installations.

Due to less favourable internal failure modes than in tension, the compressive strength of FRP rebars is lower than the tensile. It varies between tests and constituent materials, but generally compressive strength lies in the region of 50 % of the tensile ditto. The modulus of elasticity differs less and is typically about 80 % (ACI 2006).

When subjected to constantly high stress levels, FRP materials can fail suddenly in a brittle manner, without prior load increase. This phenomenon is called creep rupture or stress corrosion. The time the material lasts until creep failure for a certain stress level is called the endurance time. Naturally as the sustained stress level increases, the endurance time decreases. As always, the environment also influences. In one investigation the ratios of the creep rupture failure stress to initial strength for 50 years endurance time was found to be 29, 47 and 93 % respectively for GFRP, AFRP and CFRP. Other tests have given slightly different results, although with the same pattern (ACI 2006). Because of this phenomenon, extra care must be taken in prestressed applications so that the service stress is not in the critical region (Tepfers 1993).

FRP displays very small levels of creep deformation under sustained loads. In a test during 3000 hours at 40 % of the ultimate load, creep was measured to about 0.002 – 0.004 % depending on material and environment (ACI 2006). Relaxation of prestressed FRP tendons is in about the same order of magnitude as for prestressing steel (Tepfers 1993).

Repeated cyclic loading decreases the maximum ultimate capacity gradually due to fatigue failure. The reduction in capacity depends both on number of cycles and the magnitude of the load applied in each cycle. For one million cycles on CFRP, which is considered as the most fatigue resistant composite, the remaining capacity is in the order of 60 % for a normal range of load magnitudes. Aramid gets about the same reduction whereas glass gets a bit more reduced (ACI 2006).

Thermal expansion of steel with its 12 e-6 $^{o}C^{-1}$ is very well suited for use in concrete, which has a thermal expansion coefficient of about 9 e-6 $^{o}C^{-1}$. FRP, as always, is a bit more diverse in its behaviour. Longitudinal expansion of GFRP and BFRP is close to concrete, whereas CFRP and AFRP have expansion coefficients close to zero. In the transverse direction the behaviour is governed primarily by the matrix. Therefore the difference in this direction between the various types of FRP is smaller, with all of them at around 2 – 3 times the expansion of concrete. Under increased temperature then, the rebars will compress the surrounding concrete in its transverse direction. If the temperature instead drops, bond may be at risk if the bar contracts too much (ACI 2006). With correct design these issues can be handled, but must be known to the engineer. Actually, since the transverse stiffness of FRP is so low, the result of temperature fluctuations may result rather in deformation of the reinforcement than in spalling (Zoghi 2013).

Except for CFRP, fibre composites have low thermal and electrical conductivities and are nonmagnetic. In fact these properties are the sole motive for using FRP in many applications. For example sensitive MR-equipment in hospitals can be disturbed by steel reinforcement in its surrounding walls. Here FRP reinforced concrete has been a popular solution. Another field where FRP has gained a large market share is in architectural concrete elements such as façade claddings. In these applications the most important feature is its lesser need for protective concrete cover so that the panels can be made thin and light (ACI 2006).

The mechanical properties of different FRPs are described in the fibre specific sections from Section 2.2.4.2 and onwards. The values there are rough averages, as the data for FRP properties sprawls considerably. Data is further based on information for thermosetting resin matrices and not the weaker thermoplastic alternatives, which were not treated in this project as a consequence of its low fibre protection properties. The difference between epoxy and vinylester is within the margin of variance wherefore no distinction has been done in the design calculations presented in Chapter 3.

Durability

There are still many uncertainties regarding FRP and its durability, especially its behaviour in the naturally harsh environment of concrete. Most available data originates from accelerated laboratory tests and only limited data from real life applications is obtainable. Laboratory tests are inevitably of shorter time span than the service life of buildings and infrastructure. Extrapolations are the only way to get any results, but they always contain a large uncertainty. There are indications that the accelerated tests overestimate degradation of FRP (Dejke 1998). Some durability issues are problematic and have not yet been fully resolved. Primarily low fire and alkalinity resistance are hard to come around. There is a lot of research going on at the moment to improve the materials with regard to these aspects.

All polymers are drastically weakened at a certain temperature at which the molecular structure starts to change. This temperature is called the glass transition temperature T_g. For structural applications the working temperature range must be kept well under this threshold. The transition for thermoset resins used for FRPs takes place at between 65 – 120 ^oC. Not all capacity is lost immediately, but the degradation of the mechanical properties is vastly accelerated and at a temperature twice that of T_g most of the properties will be practically gone (ACI 2006). Aramid, in contrast to the other fibres, is a polymer and in AFRPs the fibres are therefore only slightly more resistant to heat than the matrix. Basalt, carbon and glass fibres have very high temperature resistances of 1450, 1600 and 900 ^oC respectively (ACI 2006, Gencarelle 2014). Both carbon and basalt melt at temperatures higher than those typical for open flames; hence the fibres could be retrieved after the structure's end-of-life by burning away the matrix. Possible damages on the fibres can appear under heat that is close to its melting temperatures though, so care must be taken before reuse.

Since FRP materials loose most of their strength at such low temperatures, it is not recommended for usage in concrete structures that must maintain its load-bearing capacity in the case of fire. Perhaps this may be reconsidered in the future if better matrices or protection methods are found. One way of dealing with the problem could be to ensure protection locally over the anchorage zones. If full fibre anchorage could be provided there, ideally the matrix along the rest of the bar could be allowed to lose its mechanical properties without critically compromising the load-bearing capacity. The fibres would additionally be protected by the covering concrete.

The other major concern, as stated, is the natural environment in concrete. The highly alkaline pore solution of concrete, which so well suits and protects steel has the opposite effect on FRP. Apart from carbon fibres which are nearly inert to most aggressive compounds, all the other fibres degrade substantially in concrete pore solution. Both glass and basalt fibres have been chemically altered to better sustain alkalinity; basalt without very much success and glass with noTable improvements in AR and E-CR glasses (Förster and Mäder 2011, Coricciati et al. 2009). Matrices can also be improved to minimize penetration of alkali, water and other compounds. Successful attempts to reduce matrix permeability have been made by mixing nanoclay fillers into the polymer (Zoghi 2013).

Still, the fibres, with the possible exception of carbon are dependent on protection from the matrix to not degrade significantly. This topic is very thoroughly studied for glass fibres as they are the most used in FRP composites. Carbon, aramid and particularly basalt fibres are a bit less studied. Losses in mechanical properties in different studies show very different results. It can however be concluded without much uncertainty that they all are weakened when exposed directly to alkaline environments.

Bond
There are three mechanisms that together transfer stresses between the surrounding concrete to reinforcement bars. They are: Chemical bond (adhesion), friction and mechanical interlock thanks to the bar's surface deformations (ACI 2006).

The bond creating mechanisms change during the course of increased loading. First loads are transferred without any slip by chemical bond and then friction takes over with micro cracking and minor slips. A force cone with radial and tangential components starts to form in order to distribute the stress in the concrete. The radial component generated tensile stresses which, if the force is further increased, will lead to failure by either cover splitting, concrete shear failure along the bar, shear failure of bar lugs or shear failure of the bar surface (Tepfers 1997).

In early tests with FRP in concrete, the geometry of the reinforcement was not always bar shaped with surface deformations as today. Instead there were strips with rectangular cross section and smooth bars. Back then as a consequence, members often failed due to bond slip (Tepfers 1993, Ramakrishnan et al. 1998). Today the bars are typically either deformed with ribs or helically wound on the surface by an additional string of FRP. For extra bond capacity, the surface can be covered with sand or fine gravel before the material is hardened in the curing process. With these efforts bond strength approximately equal to that of deformed steel bars is possible to attain (ISIS 2007). In Figure 6 below, an example of a helically wound GFRP bar is presented.

Figure 6 Example of GFRP-rod with helical surface deformation

The ever elastic response of FRP materials eventually leads to brittle failure without any yielding phase with large deformations preceding it. Upcoming failure is therefore not forewarned as in the case of steel. To compensate for the lack of yielding, at least 3 % elongation on the tensed side should be provided before failure (Dejke et al. 1999).

Seen from a ductility perspective it may actually not be desirable to have as good bond as possible, since bond slip has been shown to reduce brittleness of FRP reinforced concrete elements (Ramakrishnan et al. 1998). Another way of providing ductility to FRP reinforced concrete members is to use mixed fibre composites. Carbon fibres are stiff but do not elongate much more than 1 % before rupture. Glass fibres on the other hand have lower stiffness but higher ultimate elongation of up to 3 %. The high stiffness of the carbon fibres comes in good use under service loads to limit deformations. In the unlikely event of loads great enough to cause failure, as the carbon fibres one by one reach their failure elongation, they will break and successively transfer load to the glass fibres which will stretch to 3 % before final failure occurs. This requires that the load application is not so violent that the transition from carbon to glass cannot take place (Dejke et al. 1999). Another method to provide ductility is to wind FRP around a weak hollow plastic tube. When heavily tensed, the helical fibres tend to straighten, thus collapsing the hollow core. This leads to increased bond slip and deformations, until final rupture (Tepfers 1993).

Price and availability

Comparing prices of reinforcement can be performed by different parameters such as weight, volume, stiffness, strength etc. One intuitive way is to compare the price of one metre of bar of the same diameter. For reference it can roughly be said that when comparing like this, FRP reinforcement is 0.5 – 12 times the price of steel. The cheapest is GFRP with practically no added cost and the most expensive CFRP (Nanjing Fenghui 2014-06-25, Stena Stål 2014).

Compared to different steel reinforcement products, FRP rebars are less available; especially so in Sweden where it has not yet been commonly embraced by the industry. In Sweden only one distribution channel could be found during the research of this project and in that case solely of GFRP rebars. Furthermore the bars are marketed as form retaining rods and not as structural reinforcement (Haucon 2014). There are however distributors in North America and Europe who market FRP reinforcement. In today's global marketplace it can also be found easily from online sources close to the factories, primarily in China.

Design and execution

The design procedure in available codes is in many ways based upon steel reinforcement design and refers to this continuously. To some extent this means that knowledge of ordinary reinforcement design is required. Founded on the extensive bulk of research that has been generated in the field of FRP reinforced concrete since

Table 8 FRP reinforced concrete codes and guidelines (Zoghi 2013).

Country	Publisher	Year	Codes and Guidelines
Japan	Japan Society of Civil Engineers (JSCE)	1997	*Recommendation for Design and Construction of Concrete Structures Using Continuous Fiber Reinforced Materials*
Norway	The Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology (SINTEF)	1998	*EUROCRETE Modifications to NS3473 When Using FRP Reinforcement*
United Kingdom	Institute of Structural Engineers (ISE)	1999	*Interim Guidance on the Design of Reinforced Concrete Structures Using Fibre Composite Reinforcement*
Canada	Canadian Standards Association (CSA-S6)	2000	*Section 16: Fiber-Reinforced Structures, Canadian Highway Bridge Design Code*
Canada	Intelligent Sensing for Innovative Structures (ISIS)	2001	*Reinforcing Concrete Structures with Fiber Reinforced Polymers (Design Manual No. 3)*
	Canadian Standards Association (CSA-S806)	2002	*Design and Construction of Building Components with Fiber-Reinforced Polymers*
United States of America	American Concrete Institute Committee 440 (ACI 440)	2003	*Guide for the Design and Construction of Concrete Reinforced with FRP Bars*

WW2, authorities in several countries have released design guides and codes in the last few decades. A list of them is shown in Table 8 below.
These code documents and appendices cover the most common design situations and are perfectly possible to use for an engineer with experience in steel reinforced concrete. Thus, the threshold is not very high for professionals in the construction industry. The typical design procedure is that the member is designed in the ultimate limit state and then checked for serviceability criteria as well as creep rupture and endurance. Lap splicing is still the best choice where bar lengths are insufficient, although it should be noted that there are FRP reinforcement coils. There are also some mechanical connectors available which fit helically threaded rebars, but welding is of course not possible (Nanjing Fenghui 2014-06-25, ACI 2006).

So far there is very limited real-life data on durability and long-term behaviour of FRP reinforced concrete. For this reason and also to compensate for lack of ductility and known weaknesses such as heat sensitivity and risk of creep rupture, partial factors reducing material strength and maximum loads are very severe. With increased knowledge and experience, they can be expected to become less harsh further ahead (Zoghi 2013).

On site there are some differences in the way the reinforcement should be handled before and during casting. Generally the lower stiffness of FRP bars will lead to larger amounts of reinforcement, with tighter reinforcement baskets. In this aspect FRP is therefore a bit harder to work with. To some extent the increasing effect of the low stiffness will however be limited by the possibility of longer internal lever arms thanks to the possibility of less concrete cover.

Many of the FRP varieties are more sensitive to the environment than steel. Ultra violet rays and exposure to water can be detrimental and should be avoided by e.g. covering the reinforcement pallets with plastic. When moving FRP rebars around they cannot, as steel, be dragged in the dust or carelessly thrown, but should be handled with more care so as to not damage the surface and ribs. This is however not as hard as it may seem as FRP weight a mere quarter of steel and can therefore easily be carried. FRP’s density is actually lower than that of concrete which also implies that the reinforcement baskets must be tied down in order not to float when the concrete is poured. The bars should be tied with a material no harder than themselves, for example plastic cable ties. Care must be taken not to get form oil on the bars and, if it happens, it should be wiped off with a special degreasing agent (ACI 2006).

Cutting and bending differs quite a lot from steel. FRP rebars should never be sheared off as steel bars usually are since this may damage the zones close to the breaking point. Instead they should be cut by e.g. a normal circle saw, often available on construction sites for timber cutting. Bending of thermoset based FRP bars is, as previously stated, not possible. All bent reinforcement must therefore be ordered to final shape. Naturally the space for erroneous orders and last minute changes gets smaller if no changes of the reinforcement setup can be made once ordered (Nanjing Fenghui 2014-06-25, ACI 2006). Also steel bars are usually ordered pre-bent as this is more efficient than doing it on site. In an informal enquiry with engineers at COWI, incorrect orders and last-minute changes of pre-bent reinforcement is not very frequent.

Since FRP does not yield, no plastic redistribution will take place which must be taken into account. Once failure is initiated it will be progressive and no further load will be supported (Tepfers 1993). In seismic zones this lack of plasticity may become problematic as energy cannot dissipate as effectively as in plastic deformations. Knowing this, there are however effective solutions such as dampers that can secure buildings in seismic zones (Zoghi 2013).

The required concrete cover for FRP-reinforcement is not dependent on durability and can therefore often be slimmer than for ordinary reinforcing steel. As a rule of thumb it is recommended to use twice the bar diameter plus usual margin of error. This subject is further developed in Section 3.7.

Sustainability

The issue of low alkalinity resistance of FRP materials can be resolved in at least three ways: Firstly through protecting the fibres sufficiently from the environment by the resin matrix, secondly by improving the resistance of the fibres and finally by reducing the alkalinity of the surrounding concrete. The third option is quite interesting as this can easily be done, even though the two former appear to have been more focussed upon. As discussed in Chapter 1, cement production is very resource demanding and releases large amounts of chemically bound CO_2 to the atmosphere. At the same time, cement replacers such as fly ash and ground slag etc. are produced as by-products in abundance and it is rather a problem how to dispose it. In numerous tests it has been shown that with the right mixing proportions, replacement of a large part of the Portland cement can result in concretes of equal or better properties. One thing most cement replacers have in common is to react with and consume the $Ca(OH)_2$ crystals that are the source of the alkalinity in concrete. In steel reinforced concrete this is a factor that limits the amount of replacer possible to use before putting the passivation of steel at risk. In FRP reinforced concrete quite the opposite is true, as the alkalinity if anything is damaging. Without the alkalinity, some limitations regarding aggregates could also be resolved. In normal concrete, the use of aggregates containing non-crystalline silicone dioxide are discouraged in order to prevent problems with so called alkali-aggregate reactions. A more neutral environment could therefore open up for a wider range of aggregates. Perhaps then without having to think about maintaining the high pH environment, science could come up with dense and high strength concretes with both aggregates and paste sourced largely from "waste" (Rajamane et al. 2009).

The production of FRP includes chemistry-industrial processes that may be harmful to both humans and ecosystems. The fumes from some resins during curing are very harmful. Some fibres are hazardous to inhale, as asbestos clearly has demonstrated. However the most common fibres used in FRP today are of larger diameter than the threshold value and are not considered as interfering with the human respiratory system (Mikheev 1997).

The difficulty, compared with metals, of recycling FRP materials is an issue that needs further research. However, as previously stated, in concrete the problem has to be met one level above the reinforcing material, since it is very difficult to separate the reinforcement from the concrete in the first place. In this sense then, it is yet of less importance how easily recycled the reinforcement is on its own. Nevertheless, since FRP materials are used in very many more applications aside from concrete, the subject is studied. One way is to use more heat resistant fibres such as carbon or basalt that can be retrieved by burning away the more easily combusted matrix (Gencarelle 2014).

In the rest of Section 2.2.4, the specific nature of the FRP's based on the different fibres is presented. Mechanical properties that were assumed in calculations are stated. These are approximate averages between different listings in several sources.

2.2.4.2 Glass fibre reinforced polymer (GFRP)

Glass fibres are the most commonly used for polymer reinforcement. They have very good mechanical properties as can be seen in Table 9, yet at a low price. There are several varieties of glass fibres, the difference in essence being the proportions of source materials.

E-glass is the most commonly used and cheapest kind. It was first developed for electrical applications, hence the name. E-glass is produced either with or without boron. Boron is feared to have negative environmental effects and is expensive, but simplifies manufacturing. Today more and more E-glass is produced without boron. It has been found that the resulting fibre is much more corrosion resistant than the boron containing version. Therefore it has been developed into so called E-CR-glass, where CR stands for corrosion resistance. Earlier a fibre called C-glass had been developed as an acid resistant variety. This is starting to get outcompeted by E-CR-glass which is about equal and also by AR-glass which is even better. AR-glass is designed for alkali resistance, but has a high acidity resistance as well. Fibres with higher tensile strength and modulus called S-glass have also been developed. Glass fibres have some degree of solubility in water and are also sensitive to chloride ions. The different kinds vary a lot in these aspects (Vectorply 2014).

Mechanical properties

Table 9 Mechanical properties of GFRP (ACI 2006, ISIS 2007, Zoghi 2013).

Modulus of elasticity	45 GPa
Tensile strength	700 MPa
Ultimate compressive strain	2 %
Density	1600 kg/m^3
Thermal expansion coef. long./trans.	8/22 e-6 $^{o}C^{-1}$

Durability

Some studies have shown that the type of matrix does practically no difference to the speed of degradation of glass fibres, whereas others show very varied results, often with vinylester as the best and polyester the poorest. Regardless, some degradation has always been recorded and without any protective coating or matrix at all the degradation is always faster.

Water, acidity, chloride ions and alkalinity speeds up corrosion of glass fibres. This leads both to volumetric losses, embrittlement and strength reductions. High temperature and stress levels can further increase the speed of the degradation.

AR-glass and boron-free E-CR-glass have levels of degradation which are arguably at acceptable levels for use in concrete, both in the form of freely dispersed fibres and in the form of bars. The AR-fibre with its superior durability is however much more expensive than E-glass or E-CR-glass and ought not to be used, if the matrix can be expected to sufficiently protect the fibres.

It is a bit hard to quantify the durability of glass fibres and GFRP exactly. Most data originates from extrapolation of accelerated laboratory tests and little is in fact known about the real-life or long-term behaviour. Early FRP reinforced concrete structures however typically used GFRP and there is therefore some data now coming along, primarily from monitored bridges. Several Canadian highway bridges reinforced with glass and carbon FRP are continuously monitored by ISIS. After ten years of service, none of them showed any signs of deterioration (ISIS, 2006b). For the time being partial factors of GFRP are strict to compensate for these uncertainties.

Price and availability

Among the FRP reinforcements available, GFRP is the most commonly found and used. It is also markedly cheaper than the others. The price per metre of a given diameter is now at approximately the same level as carbon steel reinforcement (Nanjing Fenghui 2014-06-25). It is available through European and American distributors, but also directly from factories in China.

2.2.4.3 Basalt fibre reinforced polymer (BFRP)

Basalt was developed for military use and has not been available for civil applications for more than about 20 years. Therefore it is not yet included in most books and design guidelines on FRP reinforced concrete. It is beginning to get more researched and is expected to become used more regularly in the construction industry soon (Prince 2014). Due to the novelty of this material, long term behaviour is unknown and many properties are still uncertain. Typical properties are stated in Table 10.

The fibre is extracted from pure basalt rock, which is created when lava cools rapidly. It is very abundant in the earth’s crust and is therefore highly available and with possibility to become cheap once production levels rise.

Mechanical properties

Table 10 Mechanical properties of BFRP (Ramakrishnan et al. 1998).

Modulus of elasticity	60 GPa
Tensile strength	800 MPa
Ultimate compressive strain	2.5 %
Density	2000 kg/m^3
Thermal expansion coef. long./trans.	11/22 e-6 $^{o}C^{-1}$

Durability

Many sources, especially those with commercial interest in the products, claim that basalt fibres have very good resistance to acids and alkali as well as other aggressive environments. There are not as many studies as for GFRP available, but none of the few cited in this thesis has demonstrated those claims in reality (Parnas et al. 2007). It has been shown that even though basalt loses less volume than glass in alkaline ageing, more importantly the material properties are reduced more. In one study basalt performs about as well as E-glass in alkaline environment, while E-CR-glass and AR-glass perform better (Coricciati et al. 2009). As stated above, the resistance of E-glass may be sufficient in real-life applications as reinforcement in polymer rods in concrete. Therefore the slightly better mechanical properties of basalt may be worth the added cost. Although if durability is the priority, AR-glass may still be a better way to spend project money judged by the scarce information available. In design calculations, where relevant partial factors are not given, a reasonable assumption could be to use the values for E-glass, an approach that was employed in the design calculations in Chapter 3.

Price and availability

Basalt fibre composites are starting to get readily distributed, but are still significantly more expensive than E-glass. When manufactured, the same production lines can be used for weaving and pultrusion as for glass fibres, which is a possible benefit for future expansion of basalt reinforcement production. The price of BFRP is presently about 2.5 times that of steel and GFRP per metre bar (Nanjing Fenghui 2014-06-25).

2.2.4.4 Aramid fibre reinforced polymer (AFRP)

Aramid fibres are also known under its most widespread commercial name Kevlar. Its use is widespread in apparel and sporting equipment to provide toughness and abrasion resistance. This is possible thanks to the high toughness, energy absorptive capacity and elasticity of the fibre. It is the fibre among the ones treated in this project that has the longest elongation at rupture and yet with almost twice as high modulus of elasticity as glass fibres. It is also the lightest fibre as can be seen in Table 11.

Mechanical properties

Table 11 Mechanical properties of AFRP (ACI 2006, ISIS 2007, Zoghi 2013).

Modulus of elasticity	80 GPa
Tensile strength	2000 MPa
Ultimate compressive strain	3 %
Density	1300 kg/m^3
Thermal expansion coef. long./trans.	-4/22 e-6 $^{o}C^{-1}$

Durability

There is less information about the durability of aramid fibres than of carbon or glass fibres. Contrary to the other three fibres treated in this project, aramid is a polymer, which means it absorbs water. It is also more sensitive to UV-light than the others.

The design codes reduce the characteristic material properties of AFRP to design values in magnitudes somewhere in between those for GFRP and CFRP, which says quite a lot about its relative durability (ISIS 2006b). Aramid fibres are almost as sensitive to heat as the matrices commonly used. This is a minor deficiency as the integrity of the composite is dependent on both components.

Price and availability

Aramid fibres have been used for a long time to reinforce plastics and it is often referred to in older studies. It seems however that it has never really got commercially popular, as it is almost as expensive as carbon, i.e. much dearer than glass, but with properties not quite matching the price for concrete applications in the way that carbon does. Surely, the mechanical properties are almost as good and in some applications the greater elongation could be deciding, but at the same time it is much less durable than carbon. It appears that the market share of AFRP has decreased and there are few examples of real-life applications; this despite the fact that some of the first FRP reinforcement products such as Arapree were aramid based. At the time of the writing of this thesis for instance, there is no AFRP reinforcement bars for sale on the international trading site Alibaba.

2.2.4.5 Carbon fibre reinforced polymer (CFRP)

The carbon fibre is the king of fibres. In almost all aspects it outperforms its competitors. The major exception is economy where carbon fibres are at least 10 times more expensive than glass fibres. This is much due to the complex production method and not so much the raw materials used. Carbon fibres are produced in a wide range of varieties, all with different intended use, mechanical properties and price (Tepfers 1993). The price of the most expensive types is several times that of the cheapest. This is true also for the mechanical properties. CFRP is the only composite reinforcement that is available with higher modulus of elasticity than steel; in fact it can be as high as 500 GPa. The common, slightly cheaper varieties though have a bit lower stiffness than steel. Carbon fibres do not creep and can therefore be used in combination with other FRP’s to limit this behaviour (Tepfers 2000). Because of their high cost, CFRP products are often used in combination with cheaper reinforcement. For instance in bridge decks CFRP rebars have been used in the longitudinal principle direction, with GFRP elsewhere (Benmokrane and El-Salakawy 2005).

Mechanical properties

Properties of a medium grade CFRP is to be seen in Table 12 below.

Table 12 Mechanical properties of CFRP (ACI 2006, ISIS 2007, Zoghi 2013).

Modulus of elasticity	145 GPa
Tensile strength	2000 MPa
Ultimate compressive strain	1.2 %
Density	1600 kg/m^3
Thermal expansion coef. long./trans.	0/22 e-6 $^{o}C^{-1}$

Durability

Carbon fibres are sensitive to oxidation and should therefore not be exposed to oxygen for long time periods, especially not in combination with high temperatures. Except for this, CFRP is very durable in most environments. The maximum allowable stress level in relation to ultimate strength is much higher for CFRP than for other FRPs (ISIS 2006b). Carbon fibres have higher conductivity than glass, aramid and basalt. If in direct contact with steel, it can form part in a galvanic cell where steel becomes anodic and starts to rust.

Price and availability

CFRP has been used in several real-life projects with good result (ISIS 2006b). With the above stated mechanical properties it is about 12 times more expensive than GFRP or steel. Reinforcement made from CFRP is relatively available, although less so than GFRP.

2.2.4.6 FRP Textile and geo-meshes

With dispersed-fibre reinforced concrete as the reference point, fibres arranged in textiles and meshes made up of thin FRP- or fibre-yarns is a way of optimizing material usage. Each yarn consists of thousands of individual fibres, either just bundled or joined by resin. The yarns are knitted or weaved into the final geometry which is typically two-dimensional, although some also have extension in the third dimension. These meshes can be used in the tensile zone of thin structural members or most commonly as sole reinforcement in architectural panels or other applications where FRC is normally used (Pettersson and Thorsson 2014).

2.2.5 Other reinforcement materials

Apart from the above treated reinforcement materials, attempts have been made with several others, of which some are briefly presented in this section.

2.2.5.2 Bamboo

In regions where abundant, bamboo has been used in many applications within the construction industry. Perhaps the most common is scaffolding. As bamboo stems are stiff, bar shaped and has a glossy, seemingly resistant surface it is not surprising that it has been placed also in concrete. In especially India, there is some research going on to quantify its properties and suitability as reinforcement material.

Bamboo, as other plants, absorbs water and responds by expanding and later contracting when the moisture is let go. Concrete is wet when cast and the liquid from the paste is highly alkaline. Thus before the concrete has set, the bamboo bars will have absorbed some of this liquid with subsequent expansion. Once the concrete has set, the moisture is gradually released with following contraction. Since the concrete has hardened around the larger saturated bamboo shape, when contracting a large part of its bond is lost. If the process is repeated, the losses may increase until finally no bond is left. The first alkaline dose of water additionally has the effect of breaking down the bamboo cells, which severely affects strength (Terai and Minami 2012).

Surface coatings could be a way of limiting the exposure to both fresh cement paste and concrete pore liquid. If this surface coating had a higher friction coefficient which provided better adhesion, bond strength could also be further improved. A test series of pull-out strength of bamboo rods covered with synthetic coatings, compared with plain bamboo, plain steel and deformed steel bars was performed by Terai and Minami (2012). In this test it was found that surface coatings increased bond strength about twofold. Compared with steel, the coated bamboo rods had about twice the bond of plain bars and half that of ribbed bars according to the tests.

2.2.5.2 Non-ferrous metals

In other applications than reinforcement for concrete, other metals than steel are often used when corrosion resistance is desired. Therefore it is natural to also evaluate the feasibility of reinforcement made out of non-ferrous metals. Compared to steel, most other metals in commercial use are less stiff and strong and at the same time more expensive. In addition the highly alkaline environment that concrete provides is very well suited for steel, but less so for many other materials. Steel, as a matter of fact benefits from the high alkalinity by creating a protective, passivating surface coating in concrete and corrodes at slower pace than in open air.

Aluminium

Aluminium often replaces steel thanks to its light weight and relatively high corrosion resistance in open air environments. It is in comparison with other light metals also relatively cheap, in the same order of magnitude as steel, i.e. 2 USD/kg (LME 2014). Both its density and modulus of elasticity are about one third steel's equivalent.

It is a less noble metal than steel in the galvanic series and is therefore often used as offer anodes to conserve the steel in ships and large structures. Not very much research is available on aluminium's behaviour in concrete. However in one study, Woods (1966) thoroughly treats the subject. Aluminium is more reactive in contact with alkali than steel. One of the by-products of the resulting reaction is hydrogen gas. This feature is used to entrain air into concrete or obtain expansive properties. For these applications, ground aluminium powder is used, because of its higher reactivity granted by its high relation between surface area and volume.

Where aluminium has been cast into concrete, either as reinforcement or installations, corrosion expansion has been enough to induce cracks. This was found for example in structures where electrical conduits had been cast unprotected into concrete. The corrosion gets worse both with increased chloride content and if the concrete is reinforced with another material such as steel with higher galvanic potential. Another problem with aluminium is that it has lower bond strength than steel for a given bar shape and that the bond is lost more suddenly (Woods 1966).

Titanium

Titanium is about ten times more expensive than steel and more complicated and energy demanding to produce. It is about equal to steel in strength, but with about half the density and modulus of elasticity. It has been proven to be very corrosion resistant in many media, including concrete. Titanium is also one of the few materials that are accepted by the human body and therefore extensively used in medical applications.

The durability of titanium in concrete is in the same order of magnitude as the highest grades of stainless steel, which are also in the same price range. Because of its higher stiffness though, stainless steel is still the better choice. This is most likely the reason why there are few, if any, examples from practice of titanium in concrete (Nürnberger 1996). Titanium in other fields is instead chosen for its aesthetics and low density.

2.3 Conclusions

The conclusion from this chapter is that the viable substitutes for carbon steel as primary reinforcement in highly exposed, load-bearing concrete are stainless steel and FRP rebars. Perhaps to some extent also unreinforced or only fibre reinforced high performance concrete for applications with small tensile forces. Non-ferrous metals are not options that seem feasible and neither does surface coated steel if long-term durability is the purpose. Dispersed fibres do not provide enough tensile strength as of yet to be used as sole reinforcement in structural concrete and neither do geomeshes and textiles.

Surely there will be much development of the existing reinforcement materials towards lower cost and higher mechanical properties. Entirely new materials may also come along, or perhaps concrete will not need reinforcement, but could be ductile and tough and tenacious in itself. For the time being and in this project, the materials in focus are FRP, stainless steel and high performance concrete.

3 Case study

3.1 Introduction

To concretize the differences, advantages and disadvantages of the different reinforcement approaches, some concrete members from a sharp example project were designed. Several parameters were compared such as design, workability, price and sustainability. The reinforcement approaches that were compared with ordinary reinforcing steel are the ones deemed suitable in the conclusion of Chapter 2, namely stainless steel, FRP and to some extent plain, or solely fibre reinforced concrete.

3.2 Studied building

Aurora is the name of an office building in Gothenburg that was inaugurated in 2013. It has six storeys above ground and one below. In total there is 9500 m² of rentable office space distributed over six floors of about 2000 m² gross area each. The basement floor lies under the whole building and is primarily used as parking. The project is certified as a green building thanks to its low energy consumption and environmental impact (Eklandia 2014). The foundation, including the basement was designed by COWI’s building department in Gothenburg where this project was carried out.

Figure 7 Aurora office building as seen from the river (Eklandia 2014).

3.2.1 Studied part

More corrosion resistant reinforcement materials are required primarily in highly aggressive environments, where problems are likely to occur in steel reinforced concrete. Since the basement of Aurora is primarily used as parking, there will be presence of de-icing salts and other impurities, especially in winter-time. Due to the proximity to Göta River as can be seen in Figure 7, about half the basement wall is below the mean water level. Only the lowest water level is completely below the bottom of the wall, at about the same level as the underside of the foundation slab. As a consequence the concrete will be constantly wet and must be “watertight”, i.e. have low permeability and small maximum crack widths. In addition, as it is a building which will incorporate a lot of people, fire safety issues play an important role in the comparison. All in all, the basement is well apt to broadly compare the different reinforcement approaches.

The basement was considered to be too large to perform a detailed study of several alternative reinforcement approaches. Therefore a representable part of it needed to be chosen. The basement walls and the foundation slab are all concrete cast on site, whereas the superstructure is a steel skeleton with prefabricated hollow core floors. To include at least two types of concrete structural members it was decided that a section through the wall and slab was a good choice. It was furthermore seen as advantageous, if these parts were highly loaded in order to try the worst case. Hence a place with maximum levels of external soil pressure and normal force in the wall and high moments in the slab was selected. Finally the part of the basement adjacent to the studied part should be used as parking since the environment should be as aggressive as possible.

According to the architectural drawing, which can be seen in Figure 8, the ground levels are highest around the entrances of the ground floor. Here the difference in outside and inside levels is just 20 mm. The steel structure rests on the basement wall with rather regularly spaced steel columns and the prefabricated concrete slabs are

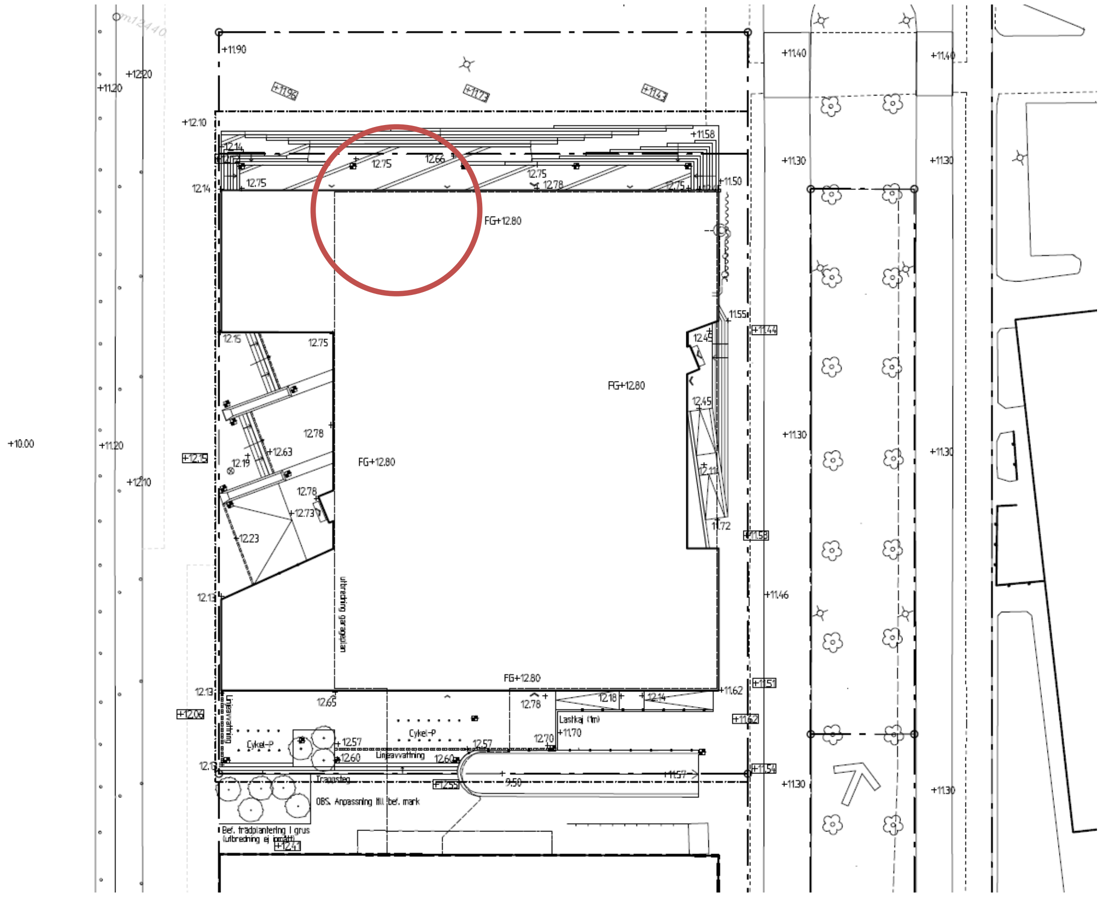


Figure 8 Site plan with studied part marked out (Liljewall Architects 2012).

oriented north-south, i.e. vertically in the plan in Figure 8. A section close to an entrance, along one of the walls supporting the slabs was therefore considered to be the best place. As one part fulfilling these criteria, the encircled area in the plan was chosen. The chosen part can be seen schematically in Figure 9.

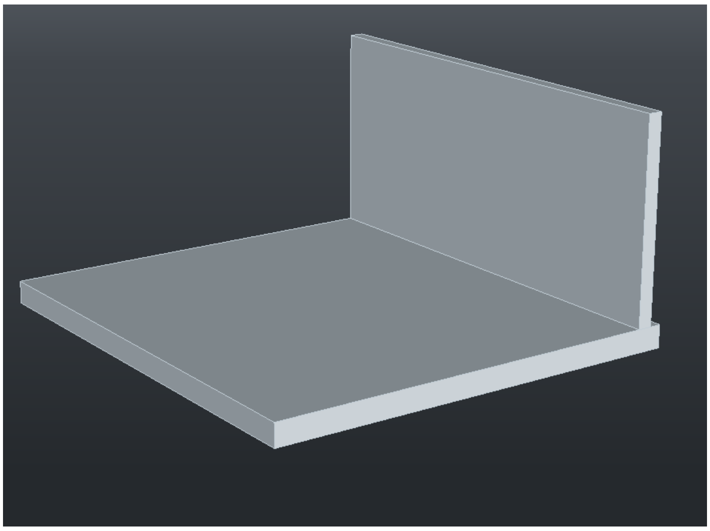


Figure 9 3D image of the studied slab and wall part

3.3 General conditions

General requirements and conditions were chosen following those of the real project and site. Here the so called “K0-drawing” containing the structural designer’s general prescriptions was an extensive source of information. A summary of the most important parameters for the studied part is presented in Table 13.

Table 13 General conditions for the foundation (COWI 2012).

Concrete strength class	C35/45
Concrete w/c-ratio	< 0.4
Creep coefficient	2.0
Shrinkage strain	0.25 ‰
Design service life of wall	L50
Design service life of slab	L100
Concrete cover in wall	45 mm
Concrete cover in slab inside/outside	55/45 mm
Fire protection class	BR1/R60
Maximum characteristic crack width	0.2 mm
Wall thickness	250 mm
Slab thickness	400 mm
Foundation slab level	+9.25 m
Ground floor level	+12.8 m
HHW	+11.6 m
MHW	+11.1 m
MW	+10.1 m
MLW	+9.4 m
LLW	+8.9 m

The slab and wall are cast in two stages but are connected with a moment resisting connection with continuous reinforcement across the joint. However, since the slab is almost twice as thick as the wall, a simplified model was used where the wall is affected by the transferred moment from the slab, but not vice versa.

The concrete strength class is a bit uncertain as it is determined by two parameters. If it should fulfil the w/c-ratio demand it will very probably be of a higher class than C35/45. For simplicity however, it was assumed that the class is indeed C35/45 without any variations to its tabulated properties.

3.4 Definition of loads

One benefit of choosing a real project is that much information, for instance concerning the loads, already defined and can be reused. Since the aim of this project was not to evaluate load identification and combinations, the possibility of reuse of predefined information has been used. Aurora is among the last buildings being completely designed according to the Swedish design standard BKR. Loads reused directly therefore follow the rules of this code. Additional load combinations performed uniquely in this case study were performed in accordance with the Swedish version of Eurocode. This slight inconsistency has not been viewed as a problem as the comparison will be fair as long as the loads are the same in all cases. In Figure 10, the loads, boundary conditions and possible deformation modes of the studied part can be seen. The load situations are further presented in the following two sections.

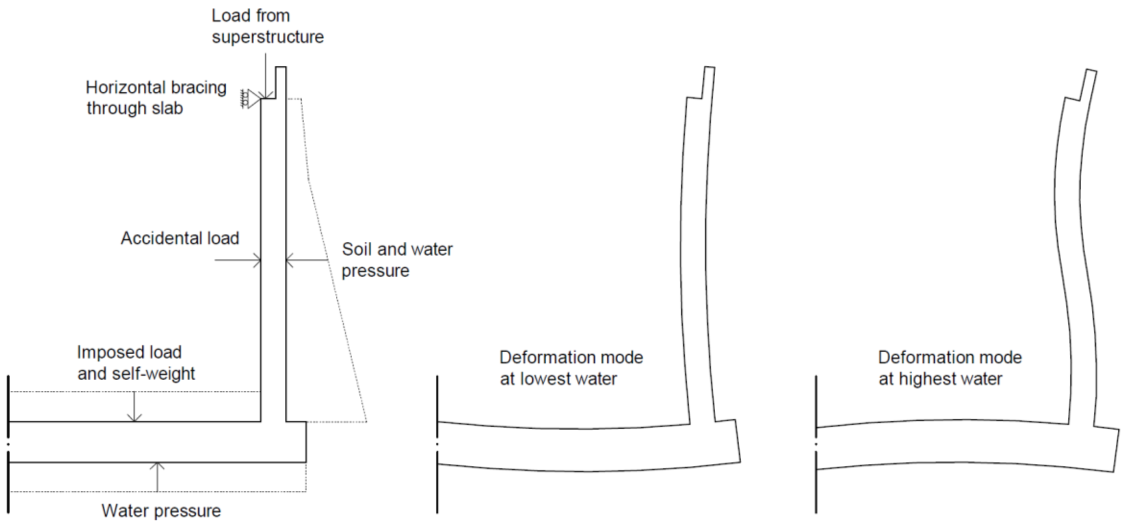


Figure 10 Images of loads, boundary conditions and possible deformations.

3.4.1 Slab

The slab was modelled as simply supported on the foundation piles and with a rigid connection to basement walls. The slab has two principal groups of load cases, one when the water pressure pushing it upwards predominates and another when the water is low and self-weight and imposed loads on the slab pushes it down. In fact the lowest low water is 50 mm above the bottom surface of the slab, but since this generates such a small upward push this is regarded as if the level were below the slab entirely. The design moments used for the case study were extracted from an FE-model made in Strusoft’s FEM-Design 8.0. The model was provided by COWI who designed the foundation in the real project. The model was loaded with characteristic load combinations in both ultimate limit state (ULS) verifying structural resistance and serviceability limit state (SLS), primarily checking cracking. Maximum positive and negative moments were extracted in both reinforcement directions and for both limit states. The moments were extracted so that the highest value of all load cases was registered in each point; in that way the moment diagrams may have parts from several load cases.

The moments are named m_x and m_y, and are unit moments measured in kNm/m. The index “x” means moment around the y-axis, i.e. moments generating reinforcement along the x-axis. The same principle applies for the index “y”. The coordinate system is defined such that the y-axis is perpendicular to the studied wall; that is up-down in the plans in Figures 8 and 11-18.

The m_y moments in the slab are greater than m_x, therefore this direction was chosen as the primary, i.e. the one where the reinforcement lies closest to the concrete surface in order to be as effective as possible.

For each reinforcement direction, moment distributions were extracted both perpendicular to and along its extension. The transverse sections show the width of peak moments, whereas the longitudinal give information on how the reinforcement can be curtailed.

In Figures 11-18, the moment distributions are presented for the chosen part of the slab, both in ULS and SLS. The analysed slab part is 8 by 7 metres and all specified moments are in kNm/m.

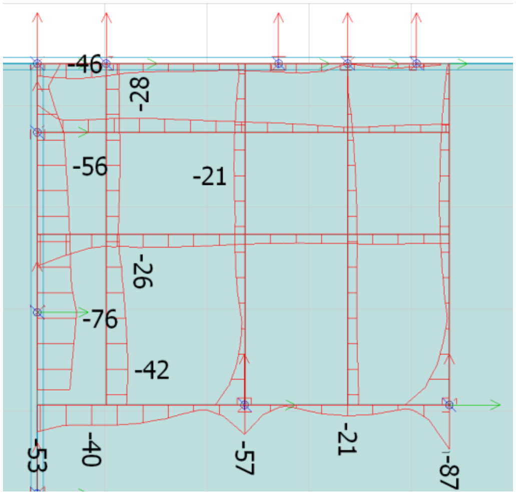


Figure 11 *Moment distribution* m$_x$ *top, SLS.*

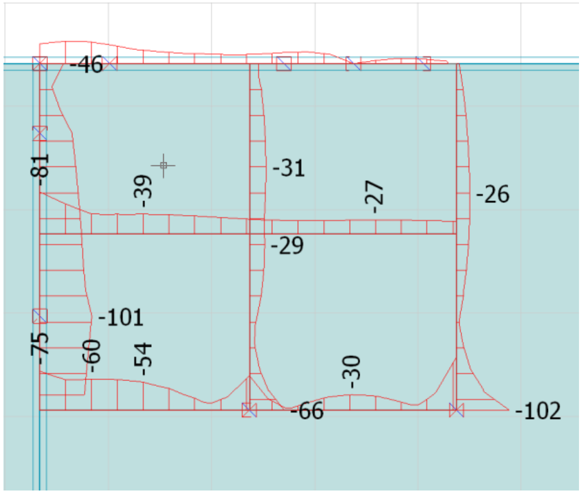


Figure 12 *Moment distribution* m$_x$ *top, ULS.*

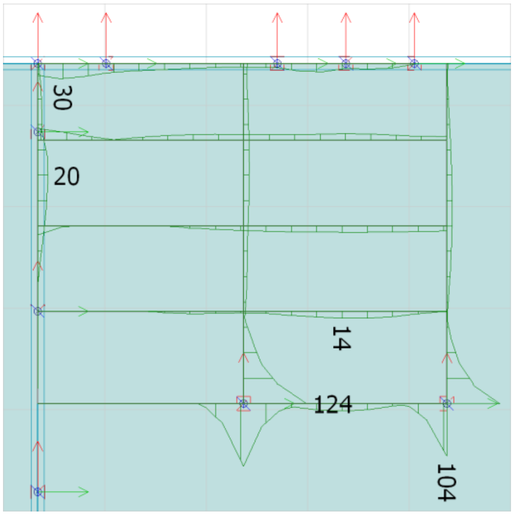


Figure 13 *Moment distribution* m$_x$ *bottom, SLS.*

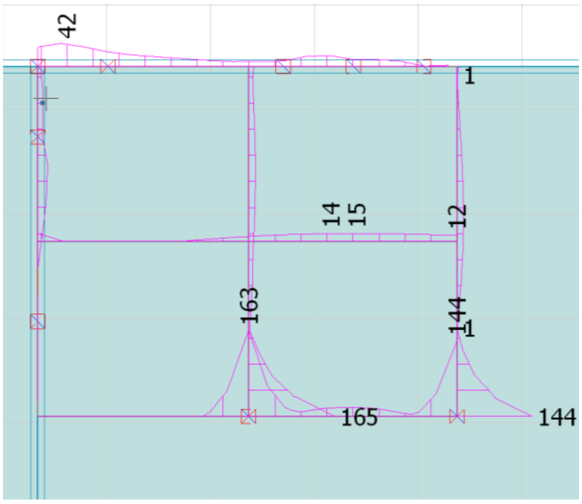


Figure 14 *Moment distribution* m$_x$ *bottom, ULS.*

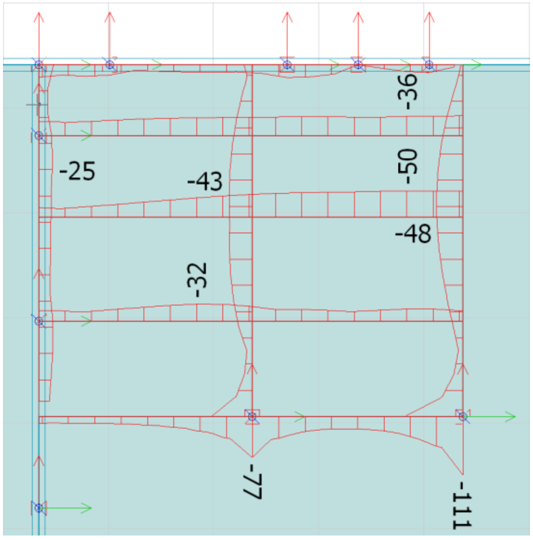


Figure 15 *Moment distribution* m$_y$ *top, SLS.*

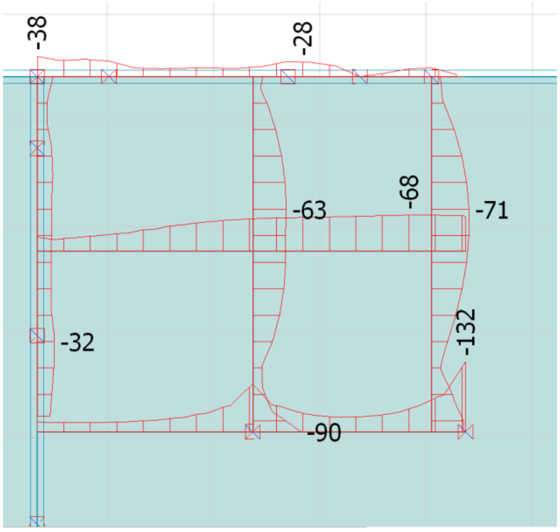


Figure 16 *Moment distribution* m$_y$ *top, ULS.*

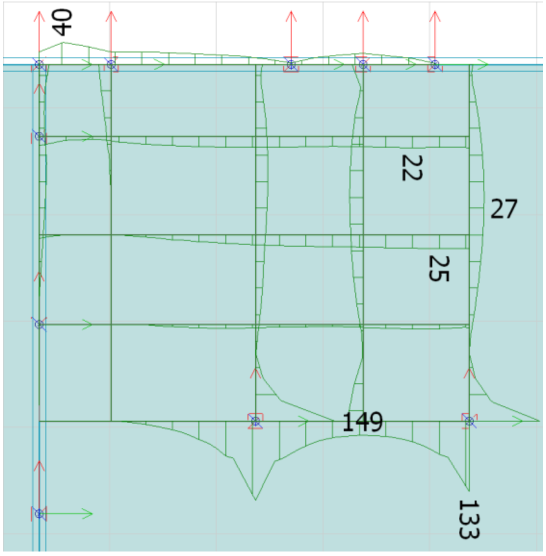


Figure 17 *Moment distribution* m$_y$ *bottom, SLS.*

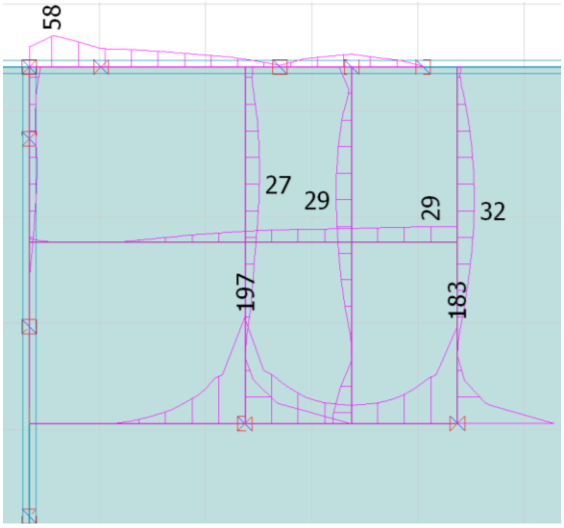
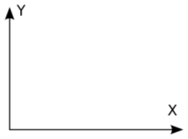


Figure 18 *Moment distribution* m$_y$ *bottom, ULS.*



3.4.2 Wall

The wall is loaded from all four sides as illustrated in the sections in Figure 10. From above it is loaded by the superstructure, including secondary effects such as load eccentricity and unintended curvature. However, unintended inclination is not considered as any effects resulting from it would be directly supported through the slab and bracing system. From the outside the wall is loaded by soil and water pressure. From below it is loaded by transferred moments from the slab and finally from the inside accidental loads should be considered, since there will be cars in traffic there.

All loads were either reused from the real project design or defined according to Eurocode. The loads were combined in all of the combinations that could theoretically occur (see Appendix F). It was noticed that the normal force was highly beneficial and that it is conceivable that the wall could get loaded before the superstructure is in place (Rydholm 2014-06-16). To be on the safe side it was therefore decided to include two load cases with no normal force. Naturally all disadvantageous effects of the normal force were also removed for consistency reasons.

With the load cases defined the software Frame Analysis, also by Strusoft, was used to attain moments and normal and shear forces in the wall. The wall was modelled as fully fixed in the bottom to get the highest support moment and hinged in the top. Diagrams of the resulting actions can be seen in Figures 19-21. From these results some of the load cases were picked out as critical for field moment (inside of the wall), support moment (outside of the joint with the slab) and shear force for each of the limit states.

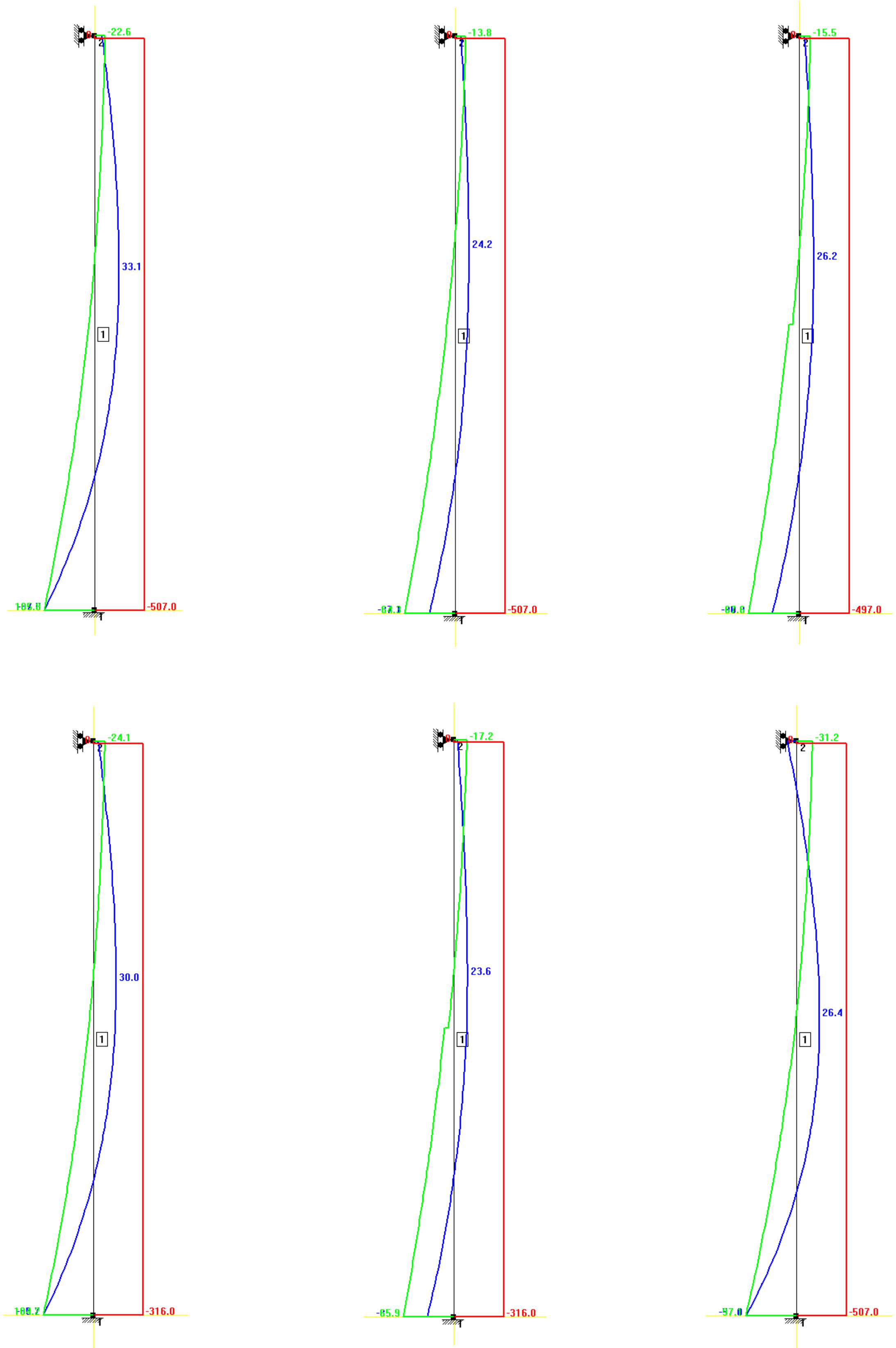


Figure 19 Moment (blue), shear force (green) and normal force (red) in kNm/kN of ULS load cases 1-6.

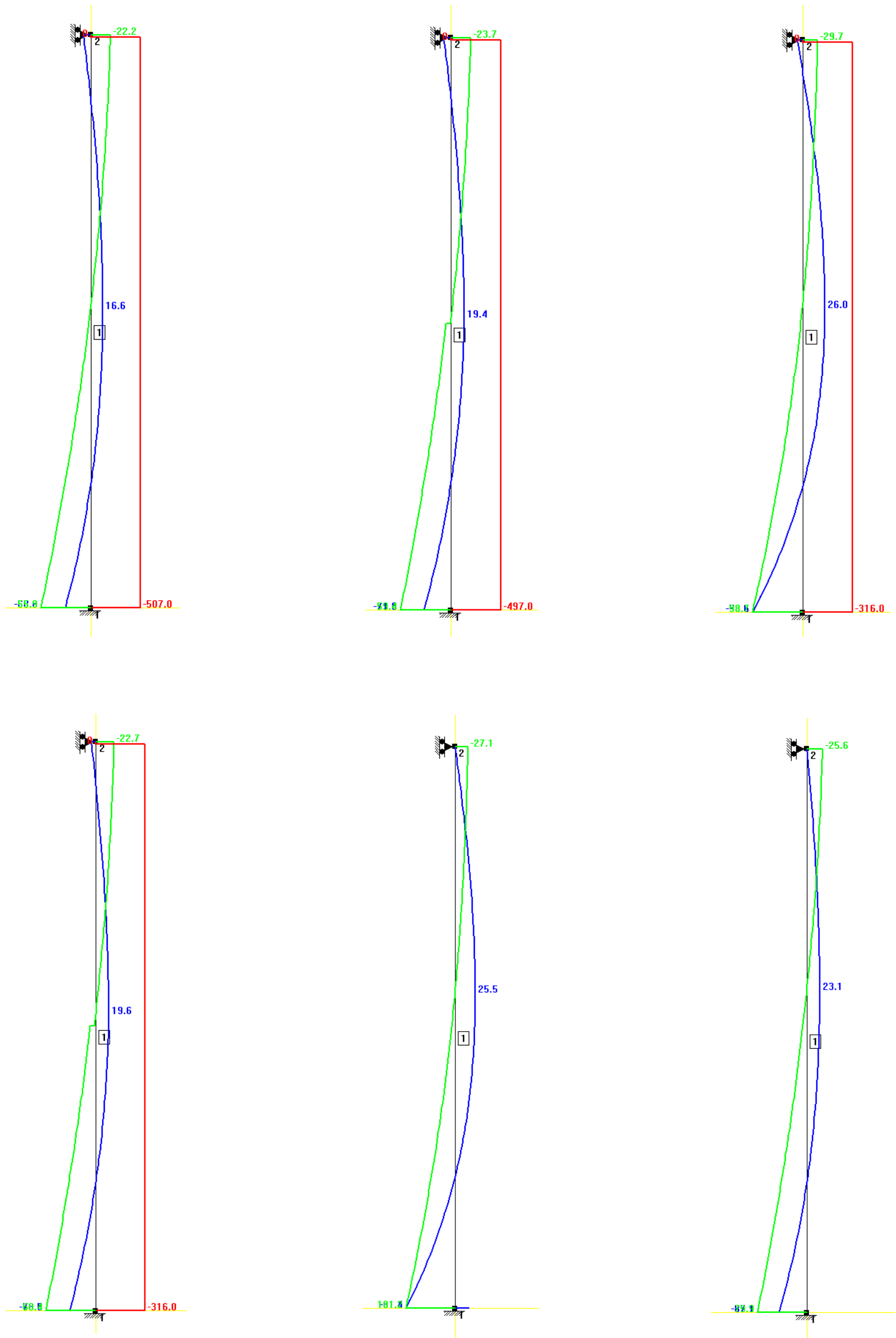


Figure 20 Moment (blue), shear force (green) and normal force (red) in kNm/kN of ULS load cases 7-12.

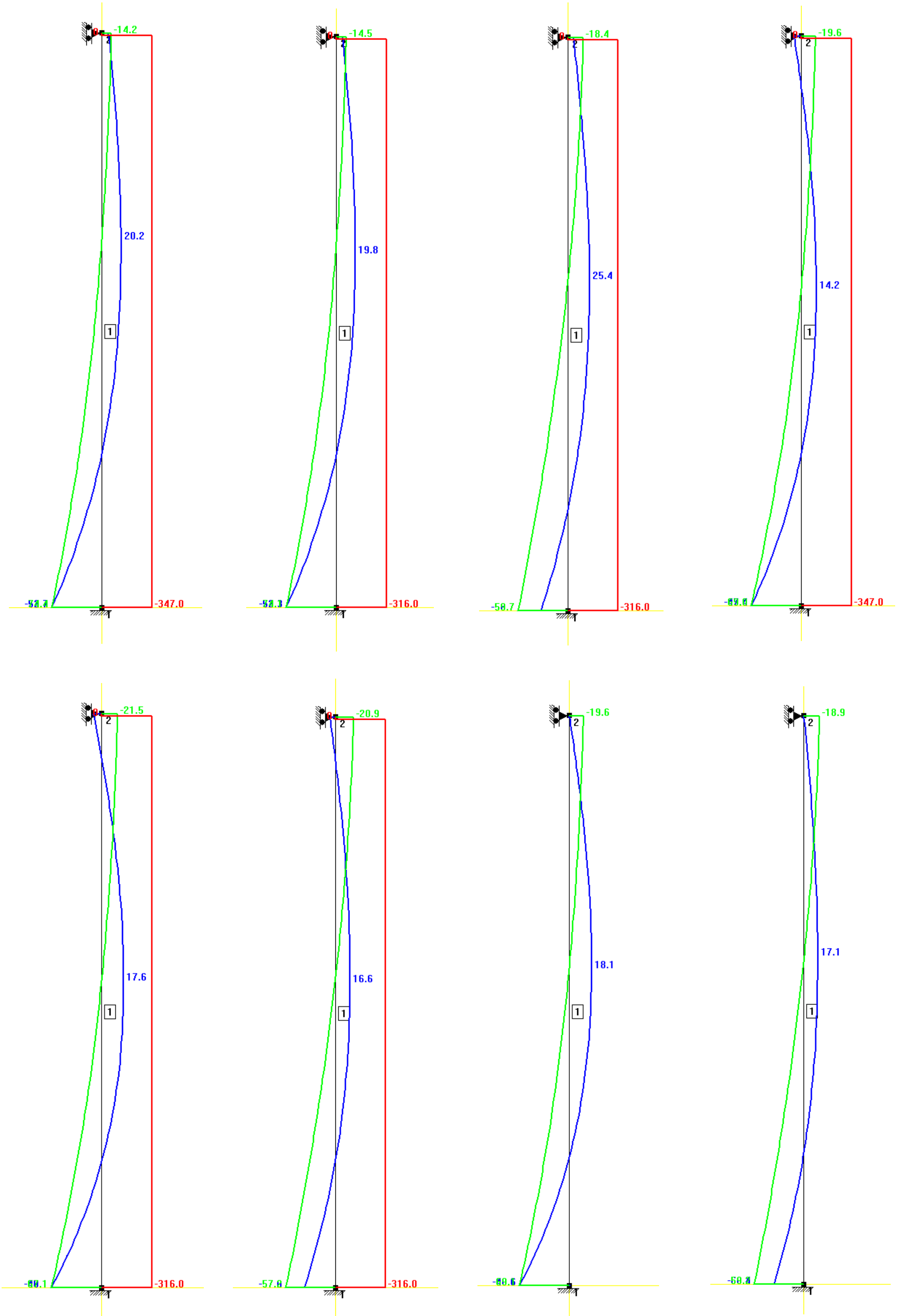


Figure 21 Moment (blue), shear force (green) and normal force (red) in kNm/kN of SLS load cases 1-8.

3.5 Steel reinforcement design

The first step in the comparison of the different reinforcement materials was to get a firm point of reference. The required amount of ordinary steel reinforcement is the obvious choice. Information regarding the reinforcement used in the project is naturally available, but in order to make a comparison as fair as possible, it is preferable that the designs of the different materials are all made by the same approach to minimize any misleading results. For this reason the steel reinforcement was redesigned, which is presented in this section.

High end stainless steel must be designed separately, since the concrete cover can be decreased quite a lot for the higher grades. Lower graded ferritic stainless steel however is more sensitive to low pH as can be seen in Figure 3 in Section 2.2.3. Therefore the best approach for low end stainless steel is arguably to keep the durability based concrete cover and obtain a longer life length and fewer reparations than would be the case if ordinary reinforcing steel were to be used.

3.5.1 Slab

The slab's reinforcement was designed in accordance with the Swedish version of Eurocode. First an initial assessment was made assuming only tensile reinforcement and an approximate internal lever arm to get a quick idea of the reinforcement requirement in a certain section depending on the applied moment. With this as a first guess, finer analyses were performed in the software Concrete Section to calculate the final reinforcement arrangement.

A level of reinforcement called base reinforcement, often corresponding to required minimum reinforcement amounts, is usually placed over the whole area of two-dimensional concrete members. The base reinforcement provides a certain moment capacity in both directions, which will provide resistance for the load in a substantial part of the slab's area. The studied slab has different concrete covers on its two sides and the two rebar orientations in each layer have different lever arms. Hence the moment capacity is different for all four combinations of moment and reinforcement, albeit slightly.

The regions with moments higher than the capacity given by the base reinforcement are provided with extra reinforcement corresponding to the excess over the base capacity. In the FE-model the supports were modelled as lines and points, whereas in reality they are walls and columns. Because of this the peak moments are exaggerated. To compensate for this the support moments were recorded not at the peak of the curve, but at the edges of the column or wall.

The width of each added reinforcement area over the base amount was defined by the points where the moment in the slab exceeds the capacity of the base reinforcement. These regions are identified using the transverse moment diagrams in Figures 11-18. In reinforcement design, moments can be rearranged through plastic redistribution as long as the total provided capacity is enough and ductile behaviour is ensured. To facilitate the work on site it was made sure that the final minimum spacing was no smaller than 100 mm in total. In practice this could mean e.g. base reinforcement and extra reinforcement each spaced 200 mm.

The curtailment of the extra reinforcement was the next step. First the same thing as for the determination of the width of the area was done, i.e. measuring where the longitudinal moment diagram recedes below the base reinforcement capacity on either side of the support. To this length a distance equal to *d* (distance from concrete surface to opposite reinforcement) is added on each side to account for the effect of inclined shear cracking. Finally the anchorage length simplified as 40Ø was added. It should further be checked that the moment diagram is not so steep that it exceeds the maximum anchorage force per unit length at any point.

The software Concrete Section, from which an example can be seen in Figure 22, was used to refine the rough results from the preliminary design and to get a visual representation of the section. Finally the required reinforcement arrangement was drawn in AutoCAD to get a clear picture of the distribution and reveal any logical mistakes. In this step the widths and lengths of the additional reinforcement areas were also illustrated.

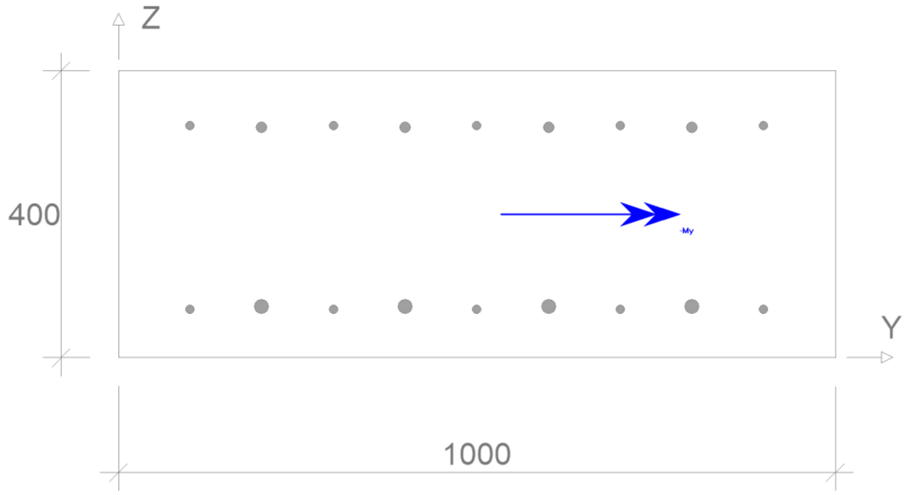


Figure 22 Slab section design in Concrete Section.

3.5.1.1 ULS

In the ultimate limit state the slab is designed for structural resistance. The design loads are larger than in the serviceability limit state, but no regard is taken to deflections and cracking. Therefore, the reinforcement need often increases when the performance in the serviceability limit state is checked. In Figures 23 and 24 both can be seen. Below each reinforcement bar symbol the needed capacity (N) is stated in kNm/m.

3.5.1.2 SLS

The function of the foundation slab is not sensitive to deflections, why check of these has been disregarded. However, crack width limitation is of utmost importance, since the slab is constantly under water. To assess the crack widths of the slab, once again

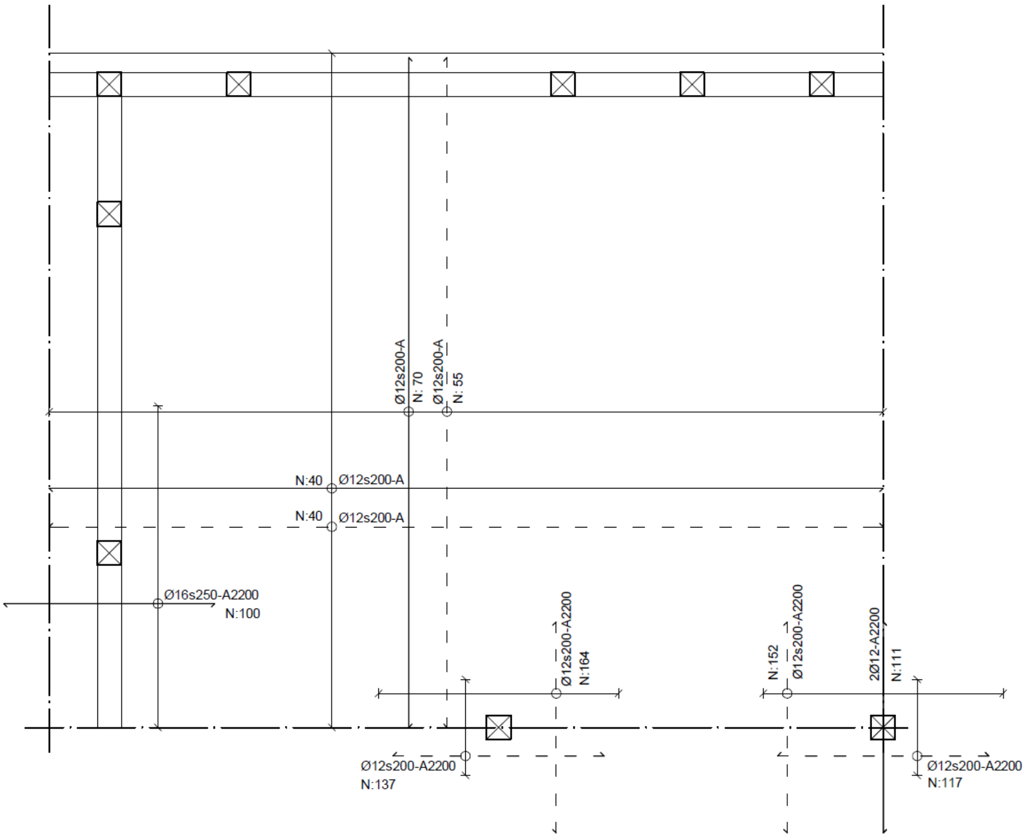


Figure 23 Needed arrangement of reinforcement with regard to the need in the ultimate limit state.

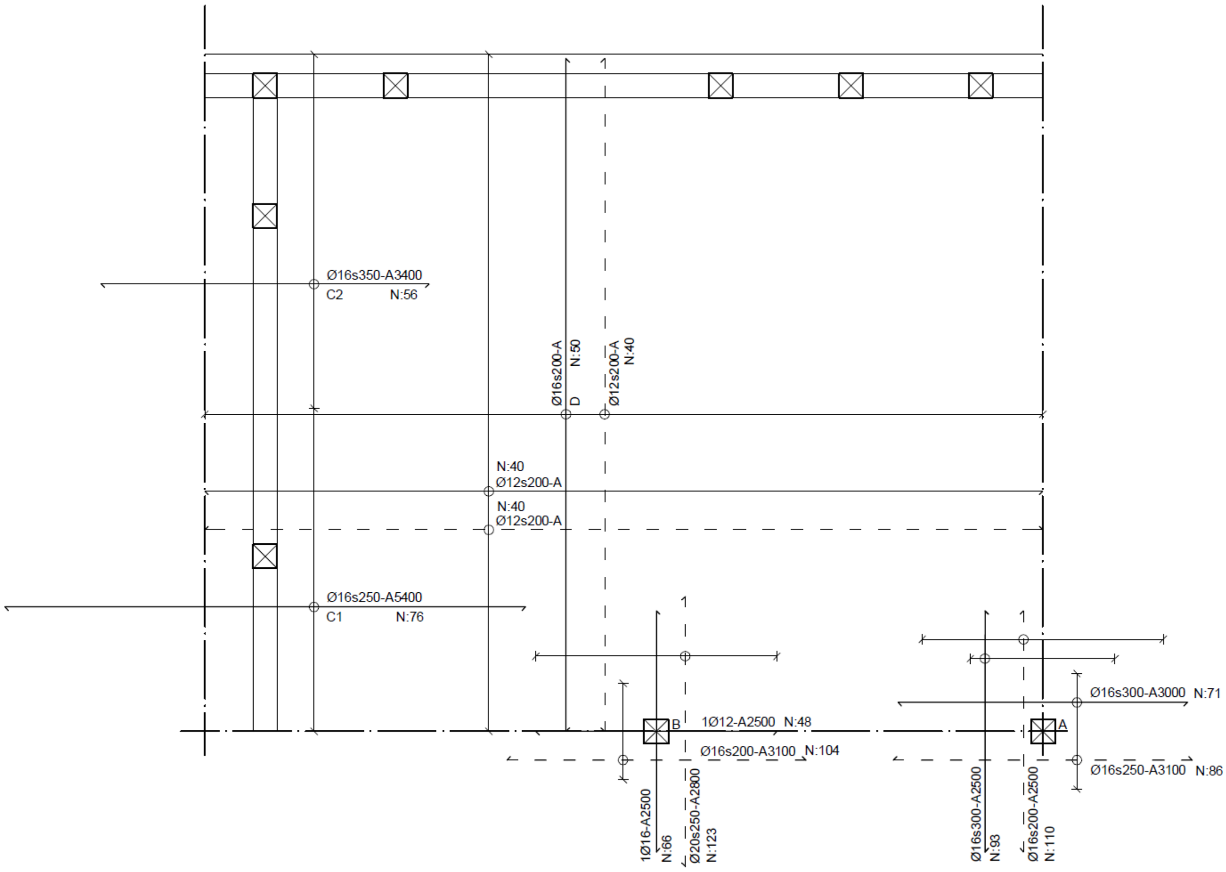


Figure 24 Needed arrangement of reinforcement with regard to the need in the serviceability limit state.

the program Concrete Section was utilized. The analysis includes the effect of shrinkage and creep. The ultimate limit state reinforcement arrangement was analysed for crack widths and was strengthened with more steel where needed until the crack width requirement was met. This meant increased steel amounts in the majority of the slab.

3.5.2 Wall

With the moments, shear and normal forces retrieved in Frame Analysis as described in Section 3.4.2, Concrete Section was used to design the reinforcement in the wall section, of which an example can be seen in figure 25. It soon became clear that the load cases, which exclude the normal force, were decisive. The field moment was below the capacity provided by the required minimum reinforcement, whereas the support moment demanded rather much extra reinforcement over the slab-wall joint. As in the case with the slab, crack width limitation was the governing design condition. The shear force capacity of the section reinforced for bending, also obtained from the program, is higher than the largest design shear force, wherefore no stirrups are needed.

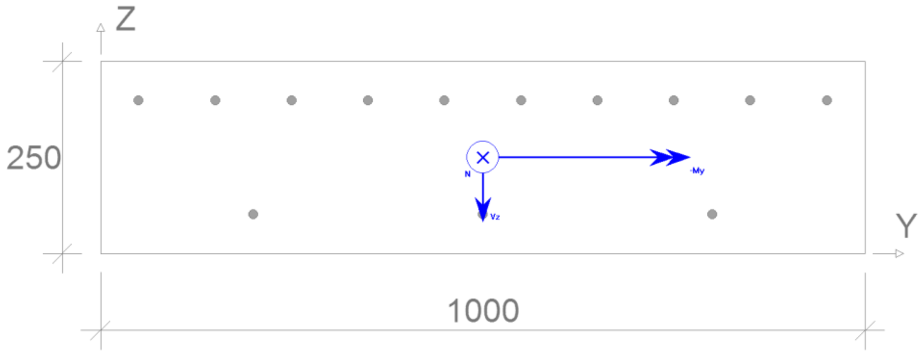


Figure 25   Reinforcement design of the wall in Concrete Section

3.5.2.1 ULS

The moment at the slab-wall connection was the most demanding part of the wall design. As the moment is greatest in the region between two concrete structural members the capacity must be provided by means of continuous reinforcement through the joint. On either side of the interface the moment then recedes and changes sign. These moments are suitably resisted by the same reinforcement. In the decision of the leg lengths, both the moment diagrams and anchorage lengths must be considered.

Neither of the load cases generates tensile stress on the outside of the wall above a section one metre from the joint. If the bar diameter is chosen as 12 mm, this leads to 480 mm of anchorage length using the prescribed 40Ø design rule of thumb recommended in the project information. To ensure sufficient capacity along the whole tension zone it must be checked that the maximum anchored force per unit length is sufficient with regard to the variation of the tensile force. Starting in the point where the moment changes sign in Figures 19-21 and looking down towards the slab it can clearly be seen that all the moment curves are concave. This means that a straight line of capacity increase will be sufficient, even if the whole tension zone would lie within the anchorage zone. This leads to a reasonable conclusion that 1 metre long legs would be sufficient on the interface-crossing B-bars (bars with one 90-degree bend). Above this length, it was shown in the design procedure that the minimum bending reinforcement is sufficient.

In the secondary, horizontal direction, the wall does not need any reinforcement in excess of the minimum requirements in the ultimate limit state, as this direction can be regarded practically unloaded. The only exception is effects of Poisson's ratio linked to the normal force in the wall which will render some tensile stresses. In a check that can be seen in appendix E it was concluded that these stresses are safely below the tensile strength of the concrete in the wall.

Nevertheless in local discontinuity regions, the tensile strength may be insufficient. Since the wall is loaded from above via steel columns, the forces are highly concentrated. This gives rise to transverse spread of compressive stresses in the concrete wall which in turn yields tensile stresses perpendicular to the compression force field. To consider this phenomenon a strut and tie model as can be seen in Figure 26 was used to design transverse reinforcement. In these places an additional 4Ø16 stirrups are needed at some distance below the load application point. The tensile zone will be spread over a relatively large area and therefore the reinforcement should be distributed vertically.

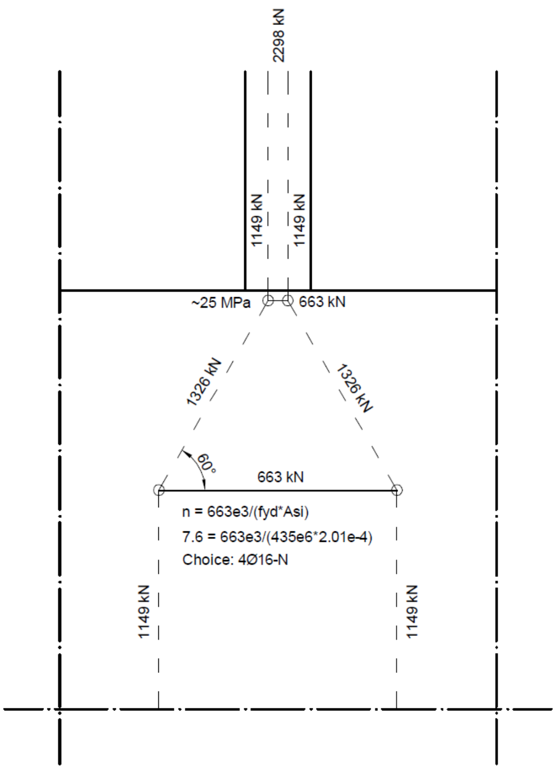


Figure 26 Strut and tie model for design of splitting reinforcement.

3.5.2.2 SLS

It was found that the ultimate limit state reinforcement in the wall is sufficient to maintain the cracks below the limiting value in the main direction. In the secondary, horizontal direction however, reinforcement had to be added with regard to crack width limitation. The need for reinforcement in this, supposedly unloaded direction is due to concrete shrinkage and constraint. Actually the combined effect of shrinkage and Poisson's effect is theoretically not enough to crack the concrete, even checked against the lowest 5-percentile of tensile strength. This conclusion is confirmed through a check in Concrete Section, where a cross section with the minimum reinforcement is loaded with a fictitious tensile force corresponding to the free shrinkage strain. This check can also be seen in Appendix E.

In the regions closest to the foundation slab though, strong constraints will appear. When the fresh concrete in the wall tends to contract, the previously cast slab will obstruct its movement, generating tensile stress in the wall. This is complicated to model but the resulting stresses are almost certainly going to supersede the tensile strength. According to practice at COWI, the bottom metre should be reinforced to a ratio of 0.7 % (steel area over concrete gross area) in the horizontal direction, given that this amount is greater than the already placed reinforcement. In the studied case it was slightly larger than the previously placed minimum amount and therefore the bar spacing was reduced the lowest metre. A drawing of the reinforcement arrangement can be seen in Figure 27.

Apart from the above mentioned reinforcement, some additional steel is needed according to the code. At a free end for example, a C-bar (bar with two 90-degree bends) should be placed together with two perpendicular straight bars as can be seen in the top of the walls in Figure 26. According to common practice at COWI, these

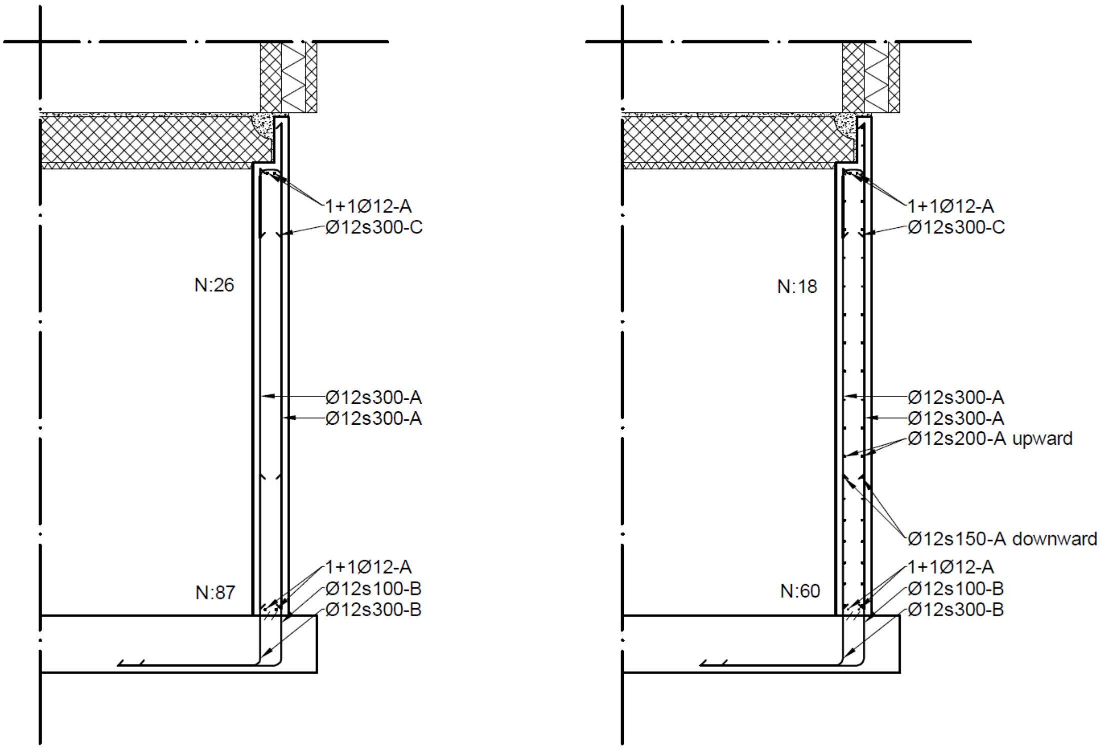


Figure 27 Wall reinforcement with regard to requirements in ULS (left) and SLS (right)

bars should be of the same diameter as the vertical reinforcing bars they connect parallel to. Also the spacing of the C-bars should be the same as the main reinforcement adjacent to it.

3.6 Stainless steel reinforcement design

Stainless steel has insignificant differences in its mechanical properties compared with ordinary reinforcing steel as presented in Section 2.2.3.4. Therefore the design of stainless steel reinforcement can be performed with the same tools that are normally used for ordinary reinforcement. The higher end grades of stainless steel could however be used with less concrete cover because they are not dependent of protection for durability reasons. In that way two possible savings appear, either lowering the amount of steel as the internal lever arm increases or maintaining the lever arm while decreasing the amount of concrete. Since stainless steel is more expensive than steel and much more than concrete, it is more economical to increase the lever arm and thus reduce the reinforcement amount than reducing the amount of concrete.

With respect to bond, the minimum concrete cover is equal to the bar diameter according to Eurocode (SIS, 2008). An allownace should always be added to take account of inaccuracy in production, with a recommended value of 10 mm. The minimum specified concrete cover would therefore be 22 mm for a 12 mm bar.

As mentioned in the previous section, the design for ordinary reinforcing steel is advisable to use also for lower grade stainless steel, thus using the better corrosion resistance for an extended service life.

3.6.1 Slab

The slab was designed in the same way as described for ordinary reinforcing steel in Section 3.5, with the only difference that the concrete cover was reduced. To go one step further in the reduction of the reinforcement amount, the slab thickness could be thickened, but this has a number of draw-backs. The excavation depth and water pressure increase and so do formwork demands. A middle-way of keeping the slab thickness at 400 mm was therefore chosen in this project.

3.6.1.1 ULS

As for ordinary reinforcing steel, the reinforcement need was first estimated roughly. In this preliminary design, the varying concrete cover depending on bar diameter etc. was not regarded. Thereafter the sections were studied more in detail in Concrete Section. The reinforcement solution of the discontinuity region under columns calculated for ordinary reinforcing steel could be directly used for stainless steel, since the concrete cover is of little importance there.

3.6.1.2 SLS

The load cases and model used for ordinary reinforcing steel were reused for stainless reinforcement. The concrete covers were adjusted and as a consequence of the longer lever arm, the amount of reinforcement slightly decreased. An ambition in the reduction of reinforcement was to minimize the largest used diameter, since the concrete cover of the bars and the position of the next layer is directly dependent of this. The largest bar size was indeed possible to decrease one step.

The total amount of reinforcement decreased noticeably, although not greatly compared with ordinary reinforcement. The minimum reinforcement requirements remain and is still the governing condition in the majority of the slab. Where the minimum requirements are decisive, there can naturally be no decrease in reinforcement. As for ordinary reinforcing steel the top base reinforcement in the y-direction differs from the others to generate higher base moment capacity.

Reinforcement arrangements for the slab reinforced with stainless steel are presented in Figures 28 and 29.

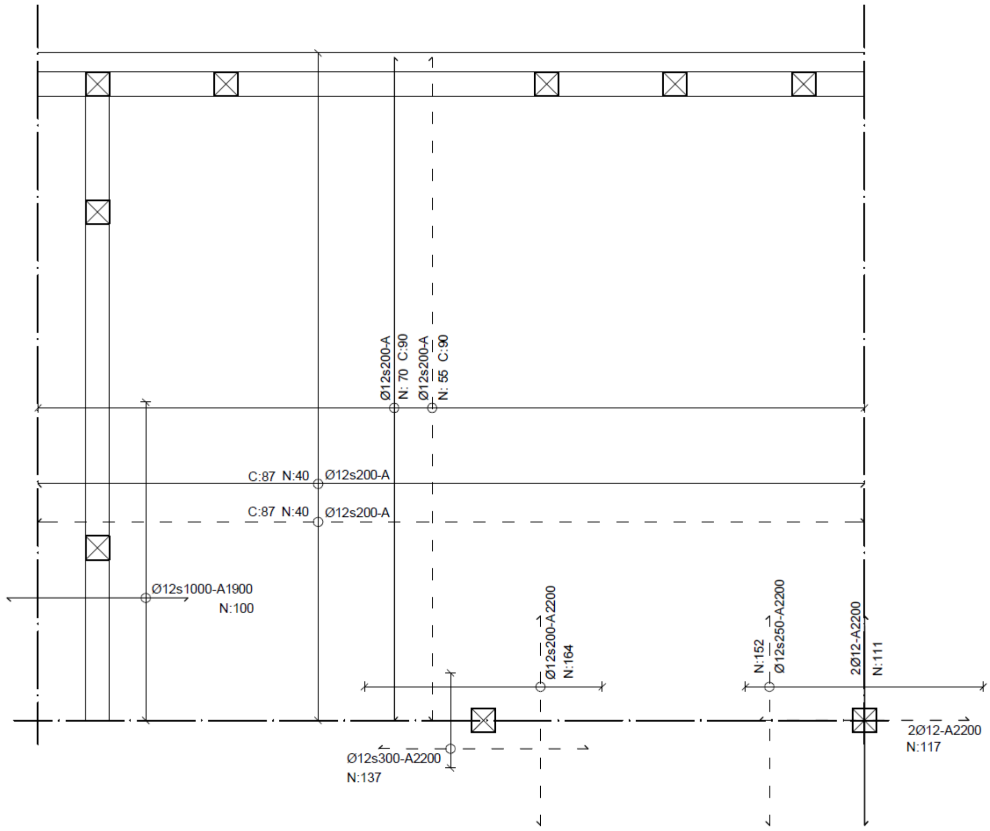


Figure 28 Needed arrangement of reinforcement with regard to the need in the ultimate limit state.

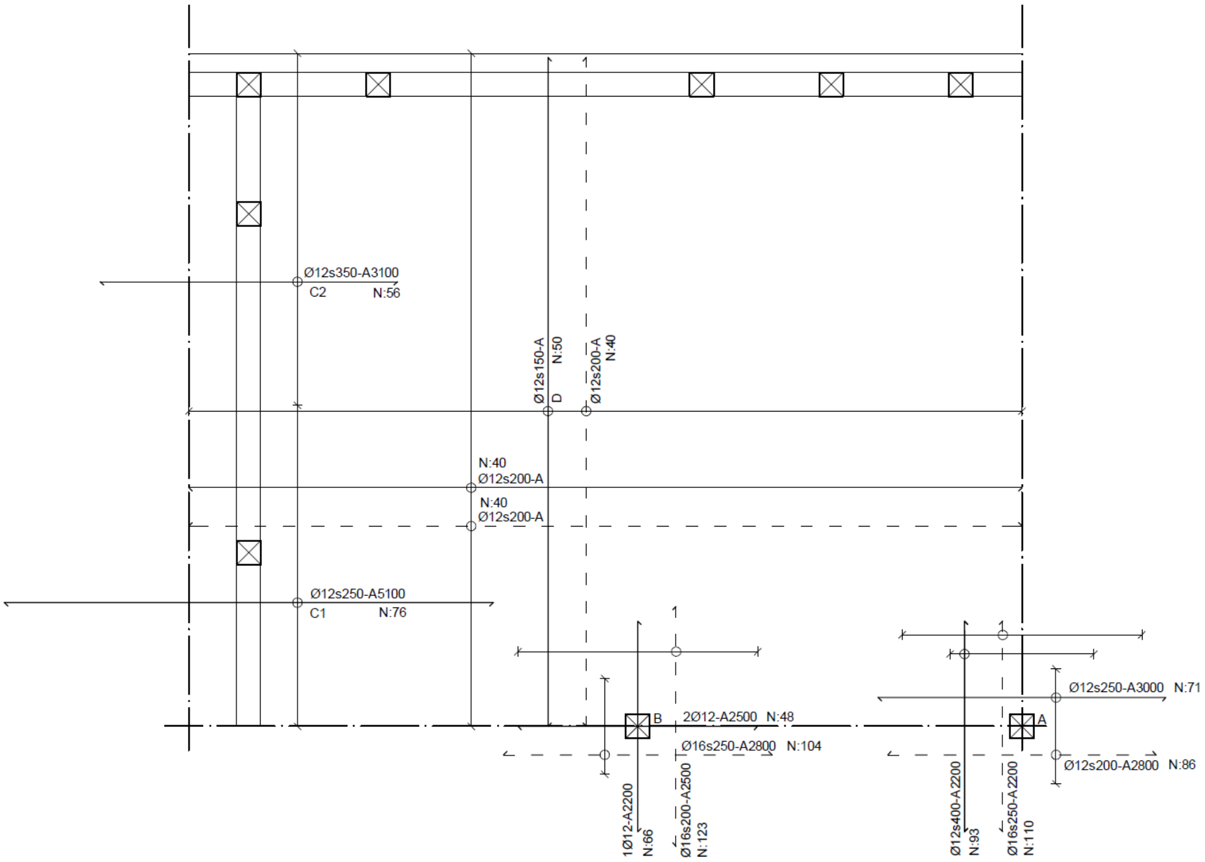


Figure 29 Needed arrangement of reinforcement with regard to the need in the serviceability limit state.

3.6.2 Wall

It was found that the wall requires no more than minimum reinforcement in most of its extension. Therefore, as argued in Section 3.6.1.2, an increase in internal lever arm will not make such a big impact on the total reinforcement amount. The only difference between ordinary reinforcing and stainless steel is that one less B-bar (bar with one 90-degree bend) per metre was found to be sufficient in the stainless case. Apart from this, since the mechanical properties can be approximated as equal to ordinary steel, the steps for the reinforcement design described in Section 3.5.2 apply also to this section. Figures 30 – 31 on the next page show the reinforcement arrangements for the stainless steel alternative.

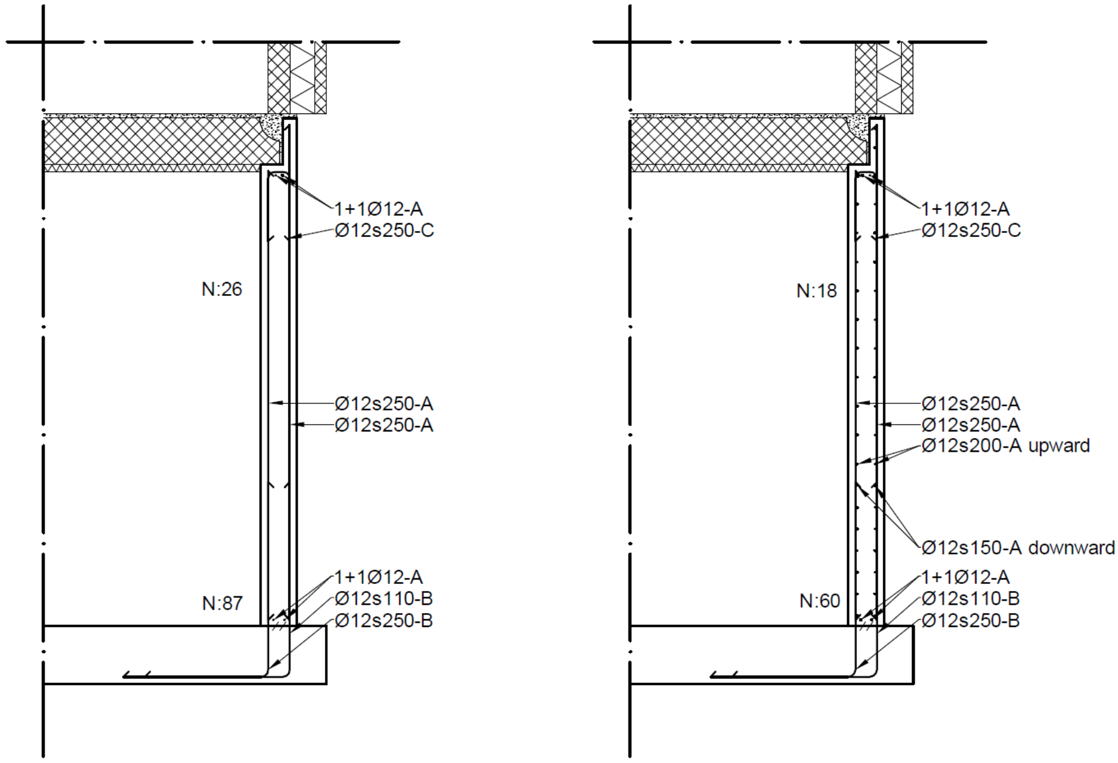


Figure 30 *Wall reinforcement with regard to requirements in ULS (left) and SLS (right).*

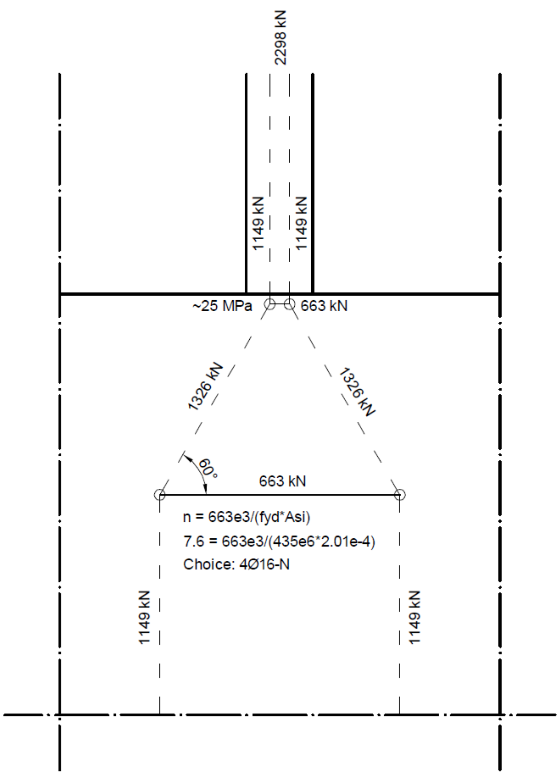


Figure 31 *Strut and tie model for design of splitting reinforcement.*

3.7 FRP reinforcement design

Reinforcing with FRP bars is similar to using steel in many aspects. Where there are major differences in the materials, the design formulae have been slightly adjusted. The code document followed in this project was ACI 440.1R-06, which is issued by the American Concrete Institute (ACI 2006). Eurocode has furthermore been used as a reference document. Since the programs by Strusoft, referred to in Section 3.4, are adapted to steel reinforcement they cannot be used. New reinforcement materials can be introduced in the programs with practically any properties, but the algorithm is unchangeably based upon Eurocode's steel reinforcement design. For this reason a Mathcad-file with the ACI's FRP algorithm was created. For details on differences in the calculation methods the reader is referred to the respective codes and calculation files in Appendices A and B. The Mathcad-file calculates the moment and shear capacities in ULS of a certain section with a defined reinforcement arrangement. An SLS moment for quasi-permanent load can then be input, with which a characteristic crack width is calculated.

Soon it became obvious that for the example project at hand, Aurora, FRP reinforcement would be a challenge to use. This is due to the high requirements of crack width limitation owing to the high ground water level. Because of the higher corrosion resistance, durability is not considered to determine concrete covers and maximum crack widths. Therefore, except for water tightness, crack width limitation is only a question of aesthetics and freeze damage prevention. The ACI refers to the Canadian Steel Institute when recommending 0.5 mm on outside surfaces and 0.7 mm on the inside. For reference, a typical limitation for steel reinforced concrete is 0.4 mm whereas in the case study project is should be no more than 0.2 mm for water tightness reasons (ACI 2006).

The modulus of elasticity of the most economically feasible FRP, namely those based upon glass fibres, is only 45 GPa, only slightly more than the concrete's 34 GPa. Therefore, when the section cracks, it is very hard to hold the cracks together and very much reinforcement is needed to do so. CFRP or AFRP though can be used with reasonable reinforcement amounts, equal to or just slightly above those of steel. However, as the price levels are today, they are unreasonable to recommend if the design service life is not extremely long, in which case FRP's are on unchartered territory in any case. Basalt FRP is an interesting middle-way with higher stiffness than GFRP at about 60 GPa. They are still relatively expensive, but the raw material is plentiful and the production method is conventional; therefore everything points at it becoming cheaper over the coming years (Gencarelle 2014).

A possible solution to the crack width problem could be to prestress the concrete member. In some sense, this is comparable to what happens when the studied wall is loaded from above. For fairness, the load-cases without normal force cannot simply be disregarded for this specific reinforcement material. However to see what is possible and with the skewed comparison in mind, it could be interesting to examine the crack behaviour if only the load-cases with normal force were evaluated. In reality this condition could be fulfilled if the construction plan required that the excavation around the basement would not be refilled before enough of the superstructure is mounted. Another option could be to prescribe external bracing of the wall from the inside.

The concrete cover cannot be quite as small for FRP as for stainless steel. This has to do with heat, both extreme heat from fire and moderate heat from ordinary service. As previously stated, the matrices of FRPs have temperature breaking points over which the mechanical properties quickly diminish. In fire situations, some layer must hold back the heat enough from the reinforcement so that the fire class demands can be fulfilled. The studied project has fire class R60. This means that the structure should withstand 60 minutes of a standard fire without endangering its users or inhabitants. In a study by Wang et al. (2009) the temperature distribution in FRP reinforced concrete columns under fire is examined. In the study a formula that calculates the fire resistance time was proposed and checked:

$$t = 0.38478c^2 - 0.00147b^2 + 3.71374b \tag{3.1}$$

where $c =$ cover thickness in mm
$b =$ column side in mm

Since the formula is customized for columns of quadratic cross-section with a surrounding fire and the studied section consists of a slab and a wall with a thought fire on one side, the formula cannot be directly adopted. If the column side length is chosen as the thickness of the wall or slab, the result would be too conservative since the wall and slab in fact are not subject to fire on both sides. A fairer choice could be to consider the wall as one half of a symmetrical column with its outside in the thought mid-section. Thus the equivalent thickness would be twice the wall thickness.

The goal of the “R” fire classes is to maintain the load-carrying capacity during the prescribed time. It could be argued that the slab is not dependent of its reinforcement to maintain the building from collapsing since its failure would not propagate into the rest of the structure. Therefore in this project, fire protection of the reinforcement in the slab was disregarded.

Since the fibres (with the exception of aramid) are much more heat resistant than the matrix, an interesting idea would be to seek more fire resistant substitutes or do away with the matrix altogether. The latter poses a couple of challenges. Firstly, the fibres with the possible exception of carbon need protection from the moist and alkaline environment in the concrete. Secondly the matrix is necessary to transfer load from the concrete to the reinforcement.

The first problem could possibly be solved by using the sufficiently resistant carbon fibres or developing the resistance of the other fibres. Alternatively the fibres could be coated with a material that was there only as an environmental protector. If protection did not have to be combined with mechanical properties, perhaps a capable material could be found more easily. The second issue of load transfer could be resolved in the way described in Section 2.2.4.1 under the subheading of Durability, namely by providing fire protection at the bar ends and allow the rest of the matrix to soften. A third option is of course to find a better matrix that could both transfer loads and offer protection, but without the today so limiting susceptibility to heat.

As can be seen among the material properties stated in Chapter 2, the thermal expansion coefficient of FRP is about twice that of concrete in the transverse direction where the behaviour is governed by the matrix. In the longitudinal direction where the fibre properties dominate, the difference is larger. Steel, for comparison has only slightly higher thermal expansion than concrete and is practically isotropic. To resist the stresses originating from uneven expansion when the temperature rises, the concrete cover must be thick enough. In a test of the effects of different cover thicknesses under thermal loading, Masmoudi et al. (2005) examined when the first cracks in the cover over the FRP bars appeared under a temperature range of -30 to +80 degrees Celsius. With covers of 1.5 times the bar diameter the first cracks became visible at around 55 degrees, whereas when 2 times the diameter was used, the whole spectrum was sustained without any cracks. To ensure that the cover thickness is not the limiting factor, the choice in this project was therefore be twice the bar diameter.

In design another major difference between FRP and steel reinforcement is that FRP does not yield. This leads to two conclusions. There will be limited ductility and there can be no plastic redistribution. The first conclusion implies that there is little difference between the failure modes of over- and under-reinforced sections. Crushing of the concrete is slightly more ductile than rupture of the reinforcement, but both modes can be accepted (ACI 2006). To represent this slight difference in ductility, a coefficient that reduces the allowed moment capacity is introduced. For under-reinforced sections it is 0.55 and for over-reinforced 0.65; this in addition to the coefficients reducing characteristic material properties to design values, which vary between different fibres (ACI 2006). The lack of plastic redistribution means that the full peak moment in the structural element has to be designed for and cannot like in steel reinforcement design be smeared out over a width where it is sufficient to support the total moment. Regarding curtailment, the same method as described in the steel reinforcement design in Section of 3.5.1 was applied.

FRP has a long-term failure mode called creep rupture which causes sudden unexpected failure of the concrete member. The failure is caused by high sustained stresses in the reinforcement. To ensure that this is averted, the ACI provides upper limits of sustained stress that are considered safe. The limits are dependent on the fibre type and are presented in Table 14 where f_{fud} is the characteristic tensile strength of the material..

Table 14 Limits of sustained stress levels regarding creep rupture (ACI 2006).

GFRP	$0.20 * f_{fud}$
AFRP	$0.30 * f_{fud}$
CFRP	$0.55 * f_{fud}$

As discussed in Section 2.2.4, basalt fibre based FRP does not have specified partial factors in any code to date. In this project as a solution the GFRP coefficients, that are the most limiting throughout, were used.

Since the crack limitation demands are so harsh in the studied project, with subsequent large reinforcement levels, sustained stress levels are way lower than the creep rupture limits. Compared with the reinforcement amount needed for the ultimate limit state however, it can be seen that creep rupture demands could be limiting there in some cases.

The effect of shear lag, depending on size of the bar as discussed in Section 2.2.4.1, where larger diameter FRP bars are less stiff and have less tensile strength, has been neglected in this project as a deliberate simplification. In a sharp project, this effect may have to be considered in the design procedure where each diameter of each material has its own properties. In such a context specific material sheets would also be requested from the supplier, instead of using approximated or average properties.

Highly compressed FRP reinforcement may suffer from instability problems; additionally both the strength and modulus of elasticity are lower in compression than in tension. For these reasons FRP is not recommended for use in columns by the ACI. In regions with high expected compression in bent members, transverse confining reinforcement should be considered (ACI 2006). This effect was not further examined in this project since compressive stress levels are rather low in the concrete walls due to their two-dimensional extension. Columns or beams with notable extension in one dimension designed purely for compression will however be much more prone to this phenomenon. In such members it is common to use confining reinforcement also for steel reinforcement.

3.7.1 Slab

The slab is relatively thick and, without serviceability considerations, would be able to sustain large loads with FRP reinforcement. As previously mentioned, no regard has been taken to fire resistance of the slab's top reinforcement.

3.7.1.1 ULS

In the ultimate limit state, FRP comes best to use. The low stiffness which leads to wide cracks and large deformations does not have to be taken into account, whereas its high strength can be used to its full potential. The same design moments in the slab were used as for the steel reinforcement. The Mathcad-file created based upon the ACI's FRP code was used to design the reinforcement. All four established fibres: Glass, basalt, aramid and carbon, were assumed and their corresponding reinforcement arrangements are shown in Figures 32 – 39. In these reinforcement plans the required moment capacity (N), capacity required beyond what the base reinforcement provides (O) and capacity that the reinforcement provides (C) are marked out by each reinforcement bar. All these values are given in kNm/m.

3.7.1.2 SLS

The concrete covers for FRP reinforcement need, as previously stated, to be two times the bar diameter to handle the expansions and contractions caused by thermal variations. For this reason, increasing the bar diameter also drastically decreases the internal lever arm. It was noted that in many cases, going up one bar size did not increase and in some cases even decreased the rotational capacity in ULS. The resulting reinforcement arrangements in SLS are not quite reasonable for glass and basalt fibre reinforcement. The spacing is too small in many parts of the slab and the large diameter bars would have significant losses due to shear lag. The simple conclusion is that for SLS design, in projects with high demands on crack width limitation, the lower grades of FRP reinforcement are not recommendable. Carbon and aramid based products however can be used with good result.

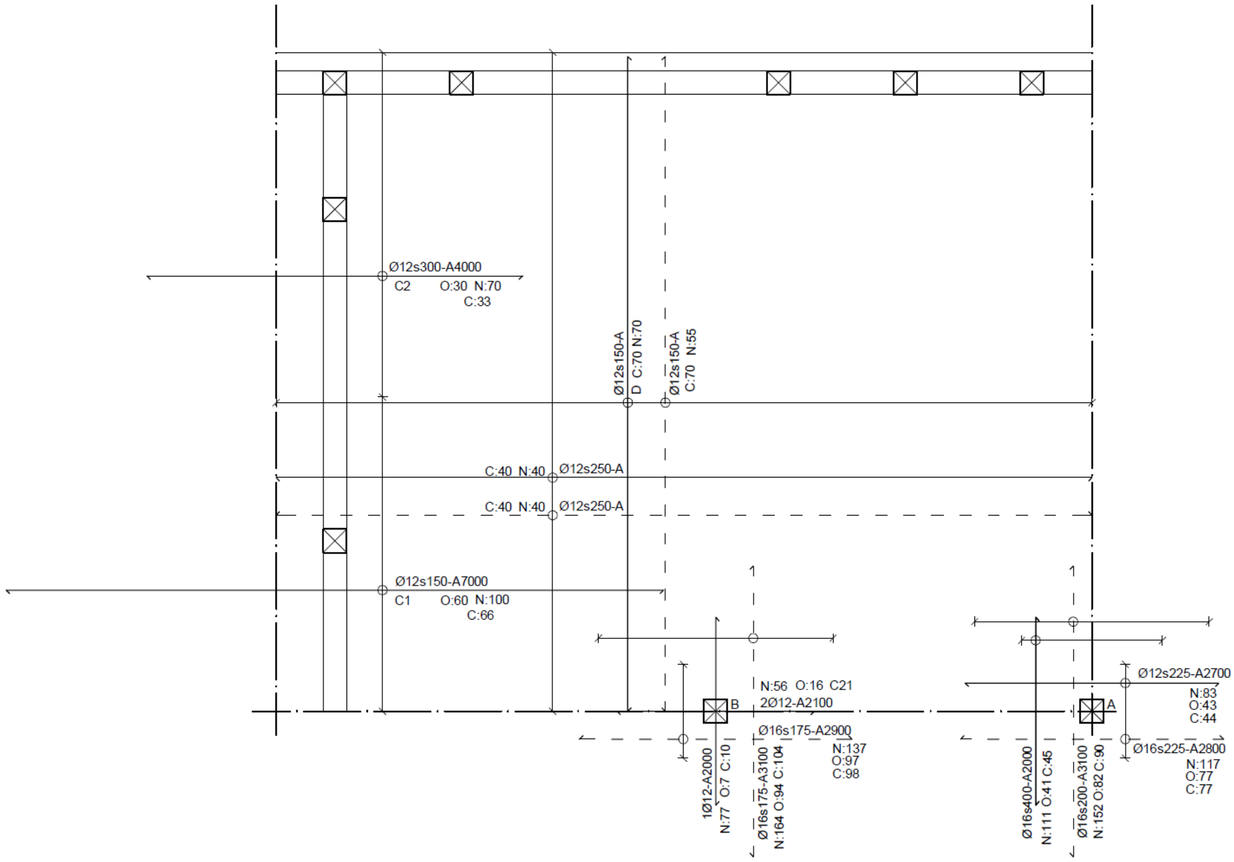


Figure 32 Needed arrangement of GFRP reinforcement with regard to the need in the ultimate limit state.

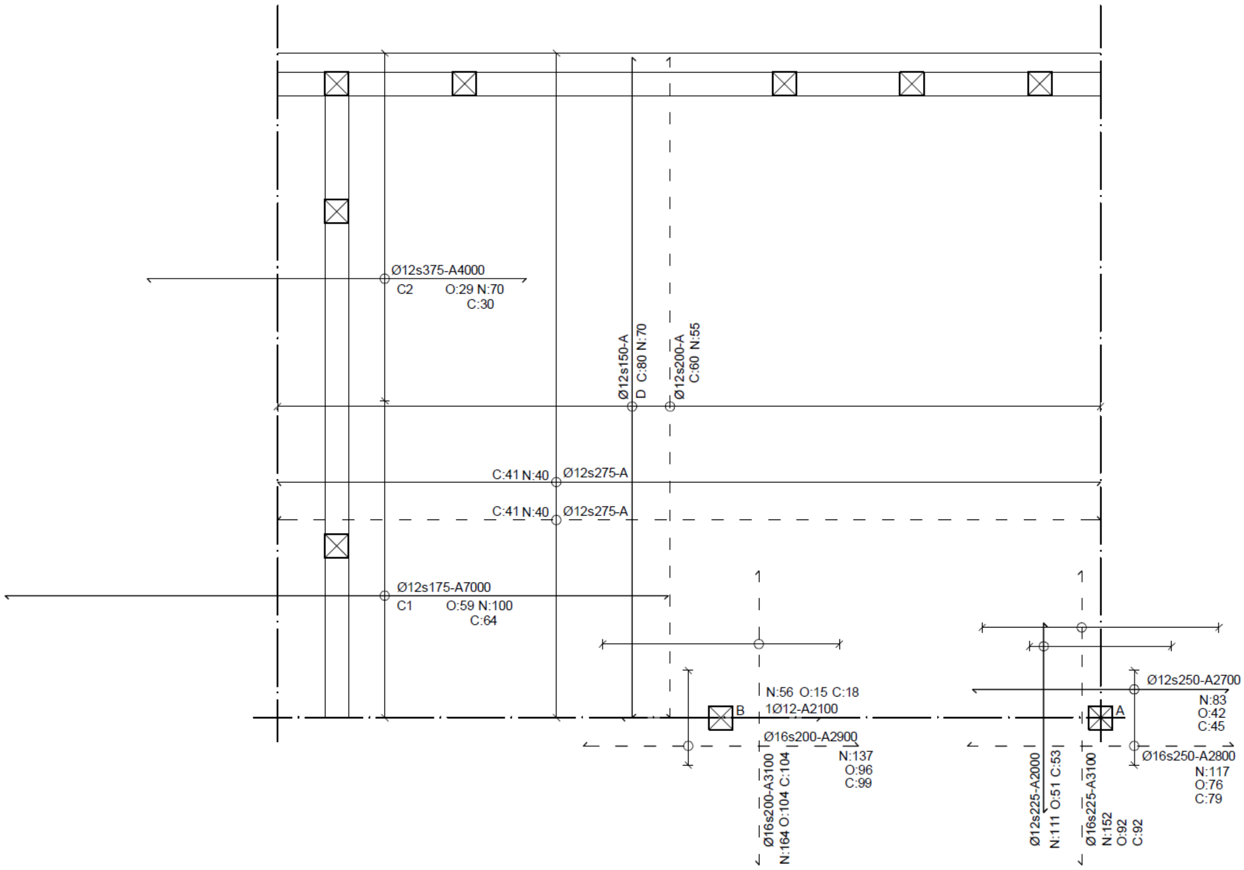


Figure 33 Needed arrangement of BFRP reinforcement with regard to the need in the ultimate limit state.

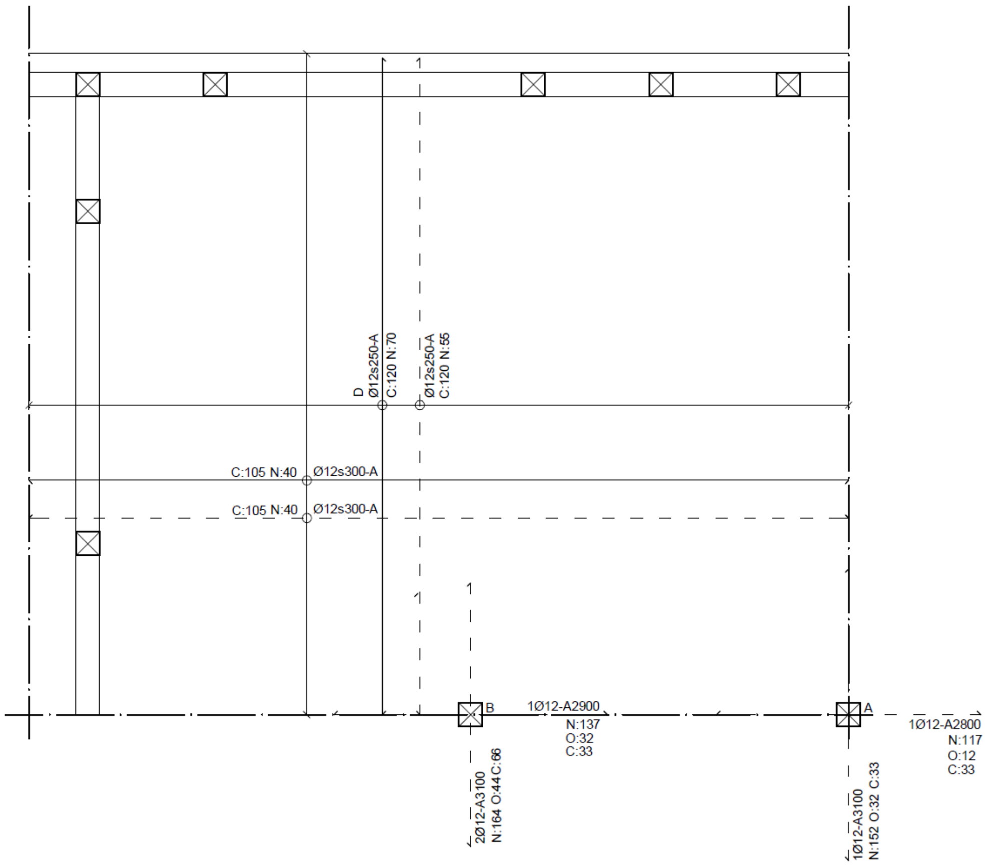


Figure 34 Needed arrangement of AFRP reinforcement with regard to the need in the ultimate limit state.

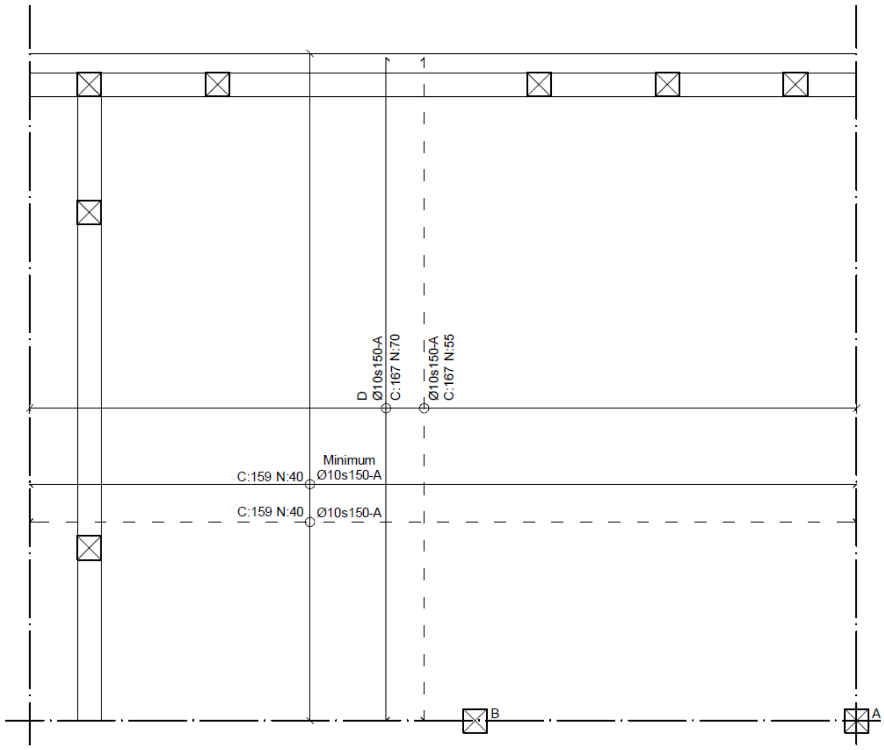


Figure 35 Needed arrangement of CFRP reinforcement with regard to the need in the ultimate limit state.

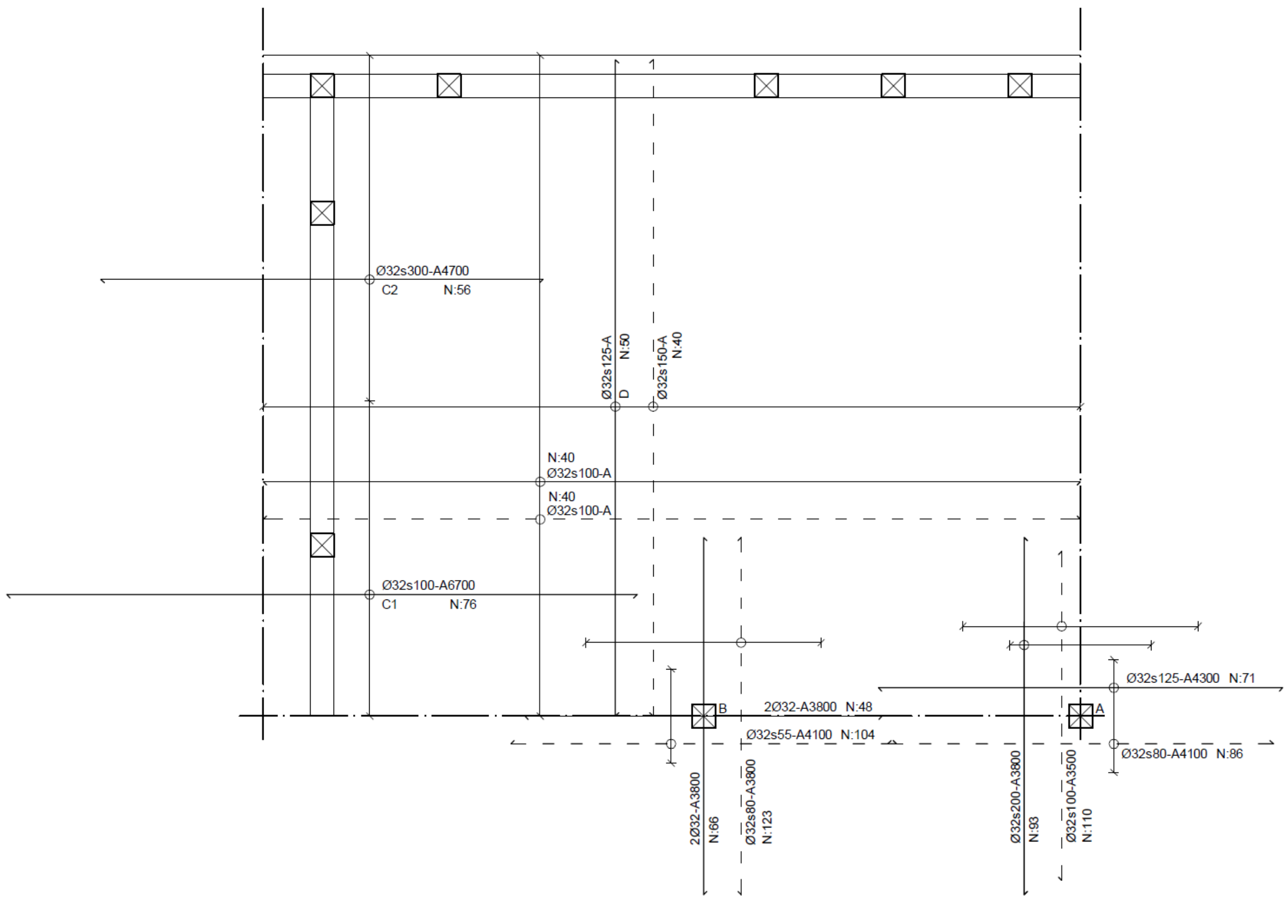


Figure 36 Needed arrangement of GFRP reinforcement with regard to the need in the serviceability limit state.

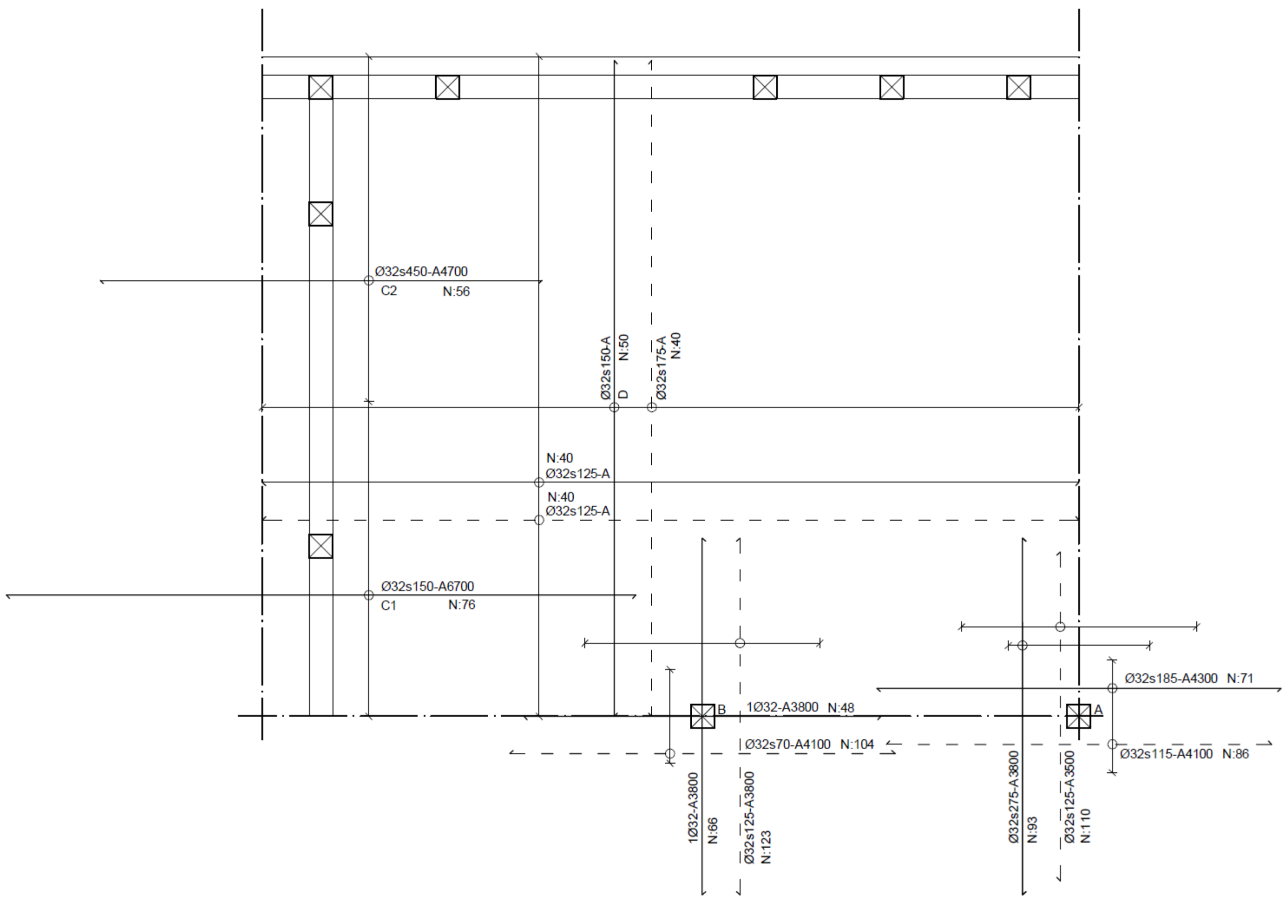


Figure 37 Needed arrangement of BFRP reinforcement with regard to the need in the serviceability limit state.

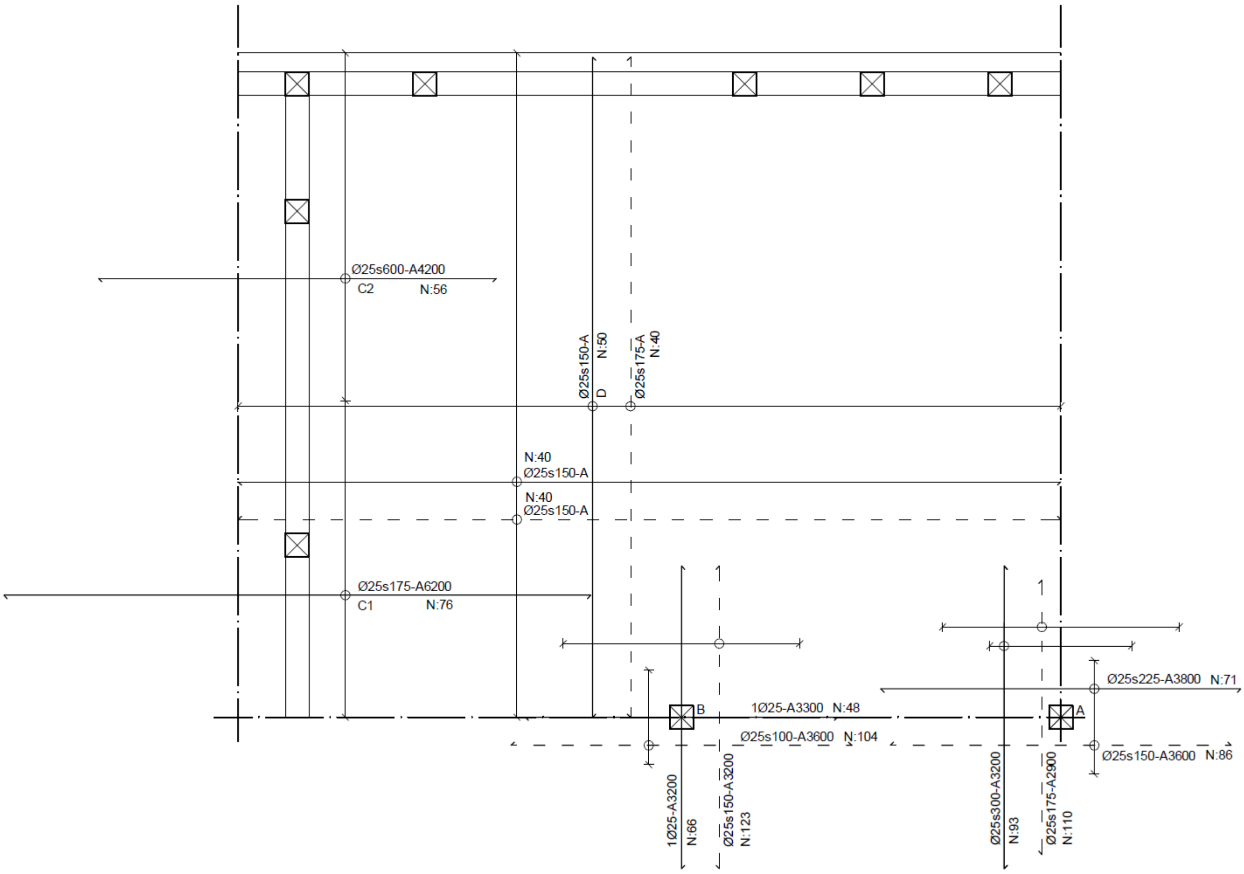


Figure 38 Needed arrangement of AFRP reinforcement with regard to the need in the serviceability limit state.

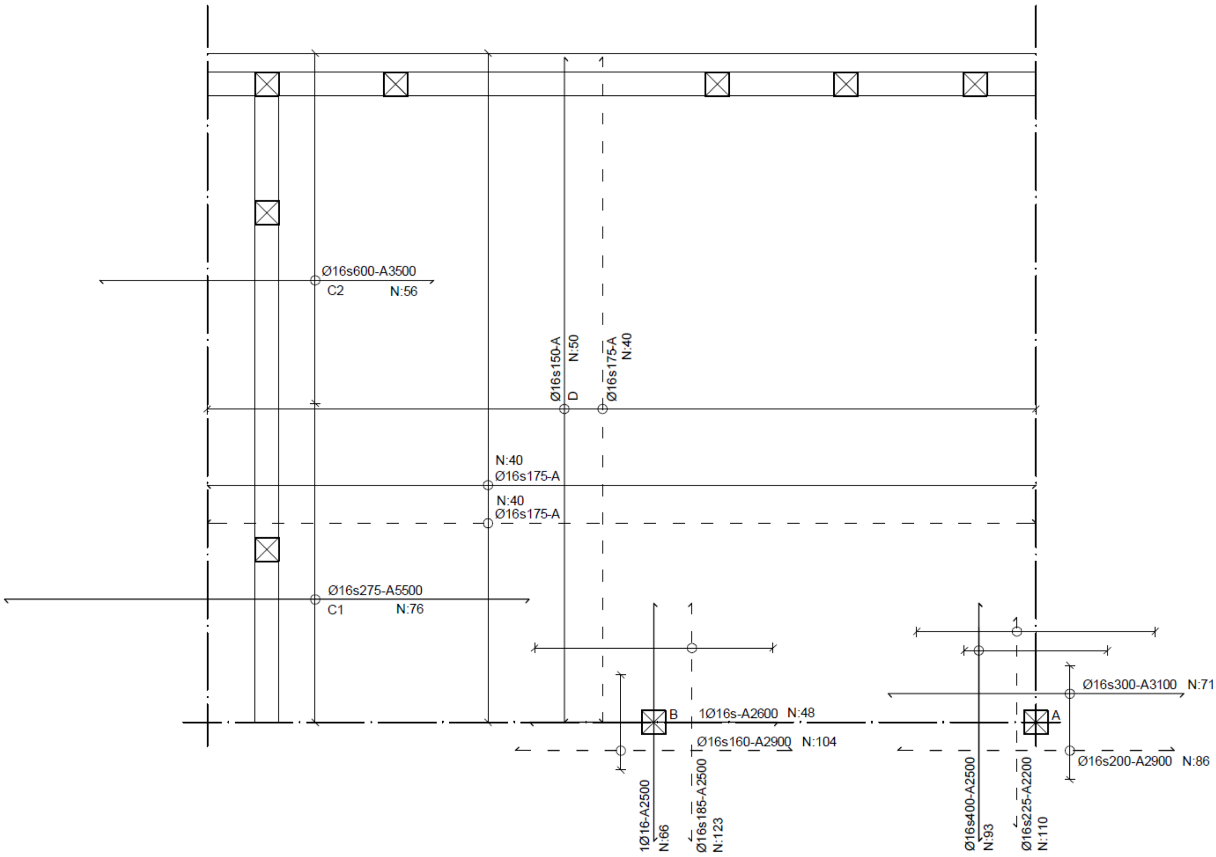


Figure 39 Needed arrangement of CFRP reinforcement with regard to the need in the serviceability limit state.

3.7.2 Wall

All FRP materials except CFRP were next to impossible to use in the design of the wall without altering the geometry. Therefore the wall had to be thickened to 300 mm instead of the 250 mm used in the real project for all fibres but carbon. Serviceability, just as for steel reinforcement, is almost without exception the decisive demand for both wall and slab. Even when disregarding cracking however, the wall reinforcement was undesignable without increasing the thickness.

If the concrete cover is chosen only with regard to the previously discussed rule of twice the bar diameter, the fire resistance for a part of the wall with Ø12-bars in the outermost layer the fire resistance time can be estimated by equation (3.1) as

$$t = 0.38478 * 24^2 - 0.00147 * 500^2 + 3.71374 * 500 = 1711 \text{ s}$$

This is less than half the required time of 3600 s, so not enough. The reinforcement in the wall is crucial for this member's load-bearing capacity and there can be no compromises in this aspect as could be argued for the slab.

This leaves two options. Either the reinforcement can reach the required level of protection by changing some material parameter, or the concrete cover or member thickness has to be increased until the formula returns a time in excess of 3600 s. The lowest round numbered concrete cover would be 75 mm giving

$$t = 0.38478 * 75^2 - 0.00147 * 500^2 + 3.71374 * 500 = 3654\ s$$

The other option in the formula, to increase the concrete thickness proved to be impossible, since the expression has a local maximum lower than the required value.

The required specified concrete cover including allowance for deviations would thus be at least 85 mm with regards to fire and without altered geometry. The other factors decisive for the concrete cover require a mere 24 mm plus margin, consequently 34 mm.

Instead of "passively" improving the performance as above, the matrix could be improved to remove the problem altogether or some kind of protective layer could be added. Improving the matrix does of course take time and does not solve anything presently. Different covering layers are however available and are used a lot for steel and also in some applications for concrete. Gypsum boards, expanding paints and mineral wool can all be used for this purpose.

In this project, expanding paint was chosen as the alternative to increasing the concrete cover thickness. In the study by Wang et al. (2009) the time an FRP reinforced concrete member will withstand a fire is calculated. The fire is a standard open fire with peak surface temperatures of 1100 degrees Celsius. The paints normally used for fire protection of steel are designed to reduce the surface temperature to about 500 degrees for the prescribed time. In diagrams seen in Wang et al. (2009) a rough estimation says that a reduction to 500 degrees would be about right to sustain 60 minutes with the a minimum concrete cover of 34 mm. Therefore the paints could be used with the layer thicknesses etc. prescribed for use on steel (Wang et al. 2009).

3.7.2.1 ULS

The same calculation sheet as used for designing the slab was also used for the wall. The detailing rules prescribed for steel reinforcement such as C-bars etc. were used also for FRP reinforcement.

The maximum shear force in the wall acts at the joint interface to the slab. Of the load cases, the highest value recorded is 106 kN and appears in load case ULS1 (see Appendix F). This load case however also has high normal force which helps in resisting the shear force. Therefore it is ULS11 which has the most critical combination of loads, even though its shear force 101 kN is less than in ULS1.

According to the ACI's FRP code (2006) the shear capacity should be verified in a seemingly very conservative way which only considers the compression zone as shear transferring. The formula is:

$$V_{c,ACI} = \frac{5}{2}\sqrt{f_{ck}}bx \qquad \text{[MPa]} \qquad (3.2)$$

where f_{ck} = Characteristic concrete strength in MPa
b = Width of the web or member in mm
x = Height of compressed zone in mm

The corresponding formula in Eurocode depends among others also on the reinforcement area in the tensile zone and is therefore not directly applicable because of the differences in the characteristics of the two reinforcement materials. Nevertheless, there is a minimum value of the shear capacity that is independent of the steel area, covered by the formula (SIS 2008):

$$V_{cminEC} = 0.035k^{3/2}f_{ck}bd \qquad (3.3)$$

where $k = 1 + \sqrt{\frac{200}{d}} \leq 2$
f_{ck} = Characteristic concrete compressive strength in MPa
b = Width of the web or member in mm
d = Effective depth in mm

In the calculation of shear capacity according to ACI, the reinforcement setup including serviceability requirements has been used, since the result depends on the final amount of reinforcement and that both limit states must be considered in the reinforcement design. For the four different FRP reinforcement materials and setups shown in Figures 40 – 43 and with the assumed concrete strength of C35/45, the resulting shear capacities are presented in Table 15.

Table 15 Shear capacities of the FRP reinforced wall.

Reinforcement	Shear capacity Eurocode	Shear capacity ACI
GFRP	132 kN	107 kN
BFRP	132 kN	110 kN
AFRP	132 kN	115 kN
CFRP	117 kN	108 kN

From Table 15 it is clearly seen that the Eurocode-based capacities are substantially larger than the ones based on the ACI rules, even though the reinforcement is disregarded in the Eurocode values and considered in the ACI values. All capacities however supersede maximum the load effect with some margin.

As in the case with steel reinforcement, the highly concentrated loads from the columns supported on the basement walls create splitting stresses below the support points. Corresponding splitting reinforcement was designed by a strut-and-tie model. The result can be seen in Figures 44 – 47.

3.7.2.2 SLS

There is no recommendation about horizontal reinforcement adjacent to a previously cast member in the ACI code. In lack of better guidelines no modifications were therefore made to the general prescribed horizontal reinforcement prescribed by the ACI code (2006). For steel there was in fact a rather small difference from the horizontal base reinforcement. It does seem that the prescribed minimum amounts in ACI 440.1R are relatively smaller than those for steel according to Eurocode 2 (SIS 2008), especially considering the lower stiffness of FRP reinforcement.

The moments indicated on the drawings in Figures 40-43 are the peak moments in field and support. Aside from these regions it has been assessed from the moment diagrams in Figures 19-21 that 10 kNm would be a safe level base reinforcement.

Just like for the slab, the reinforcement amounts became unreasonably high in the peak moment zones in order to maintain the characteristic crack widths below the limit of 0.2 mm. The resulting spacings are smaller than wished for and large diameter bars are needed. No pleasing solution was found to this problem, which is further discussed in Chapter 5. The final reinforcement arrangements can be seen in Figures 40 – 47. For reference the required moment capacities (N) in the field and the slab joint are marked out in kNm/m in the figures.

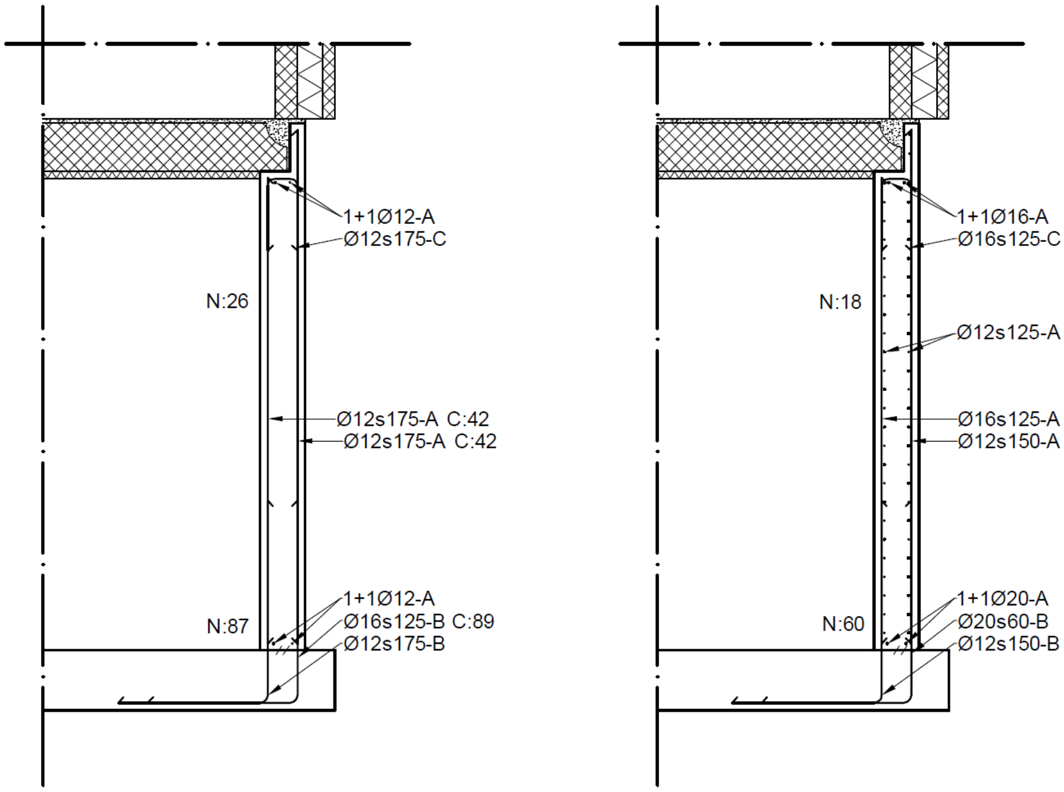


Figure 40 Wall reinforcement for GFRP with regard to requirements in ULS (left) and SLS (right).

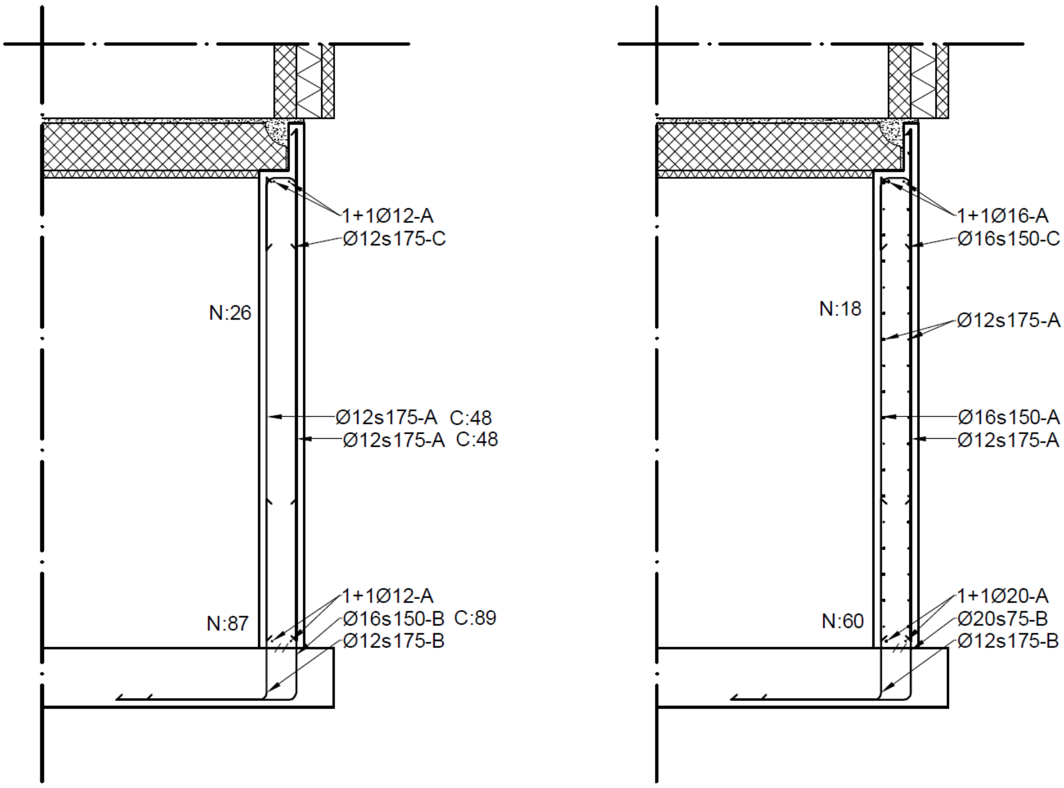


Figure 41 Wall reinforcement for BFRP with regard to requirements in ULS (left) and SLS (right).

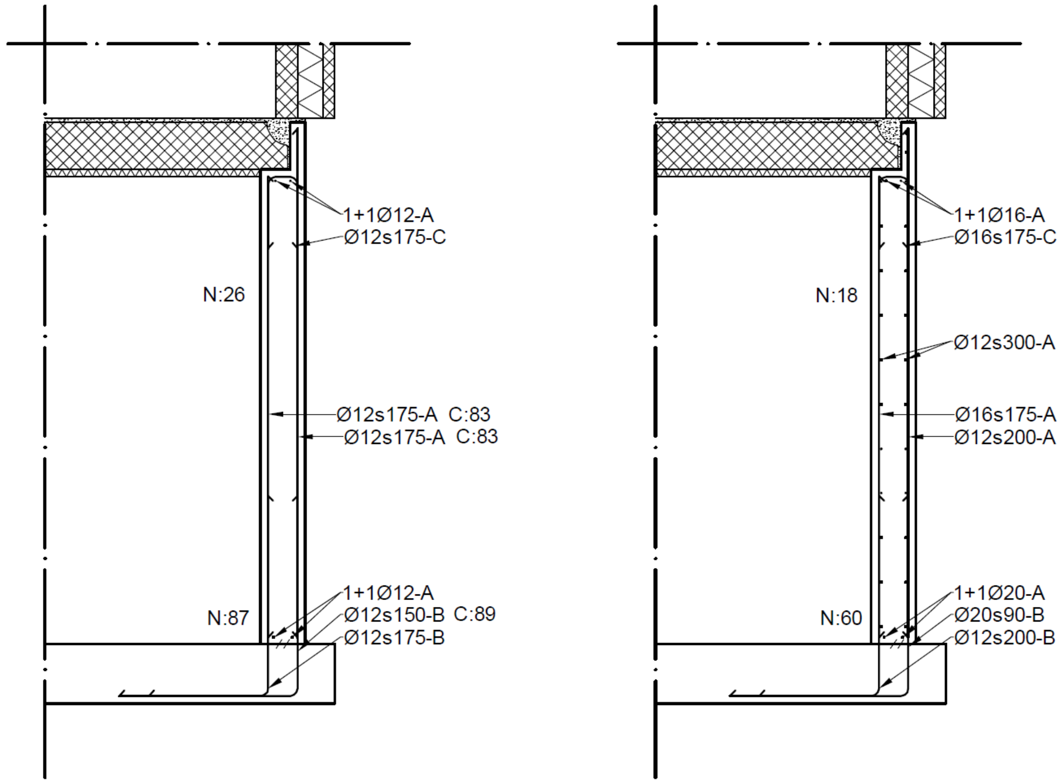


Figure 42 Wall reinforcement for AFRP with regard to requirements in ULS (left) and SLS (right)

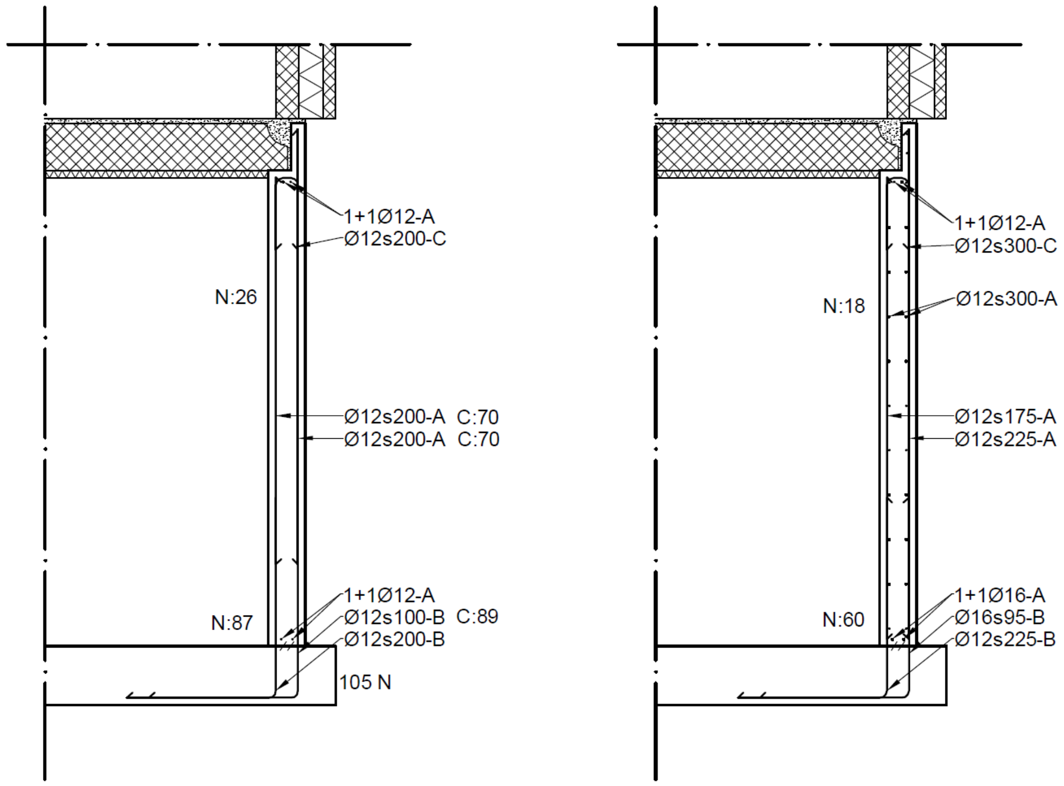


Figure 43 Wall reinforcement for CFRP with regard to requirements in ULS (left) and SLS (right)

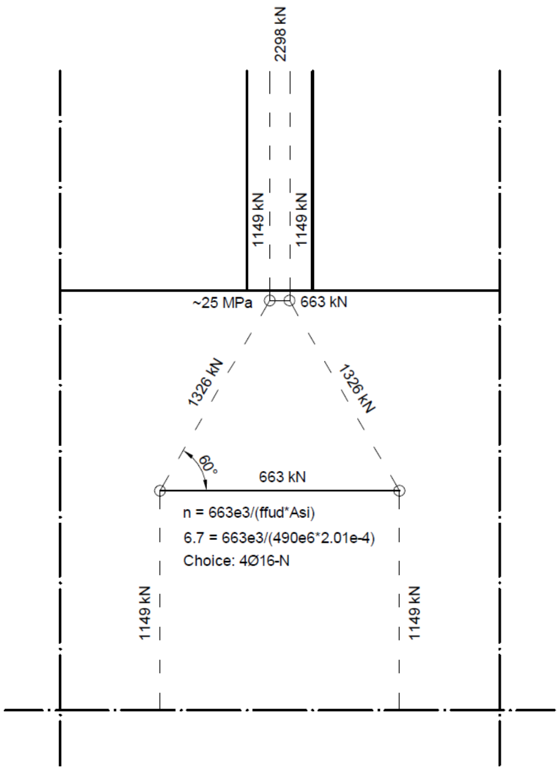


Figure 44 Strut and tie model for design of GFRP splitting reinforcement.

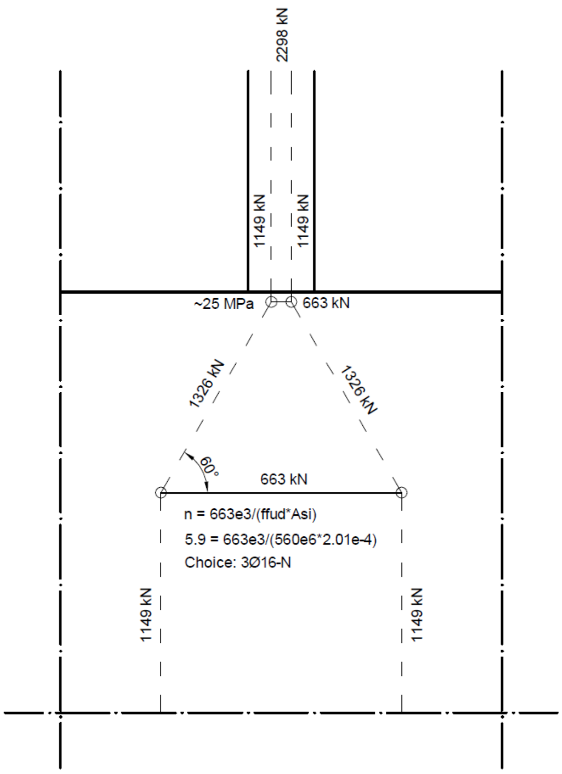


Figure 45 Strut and tie model for design of BFRP splitting reinforcement.

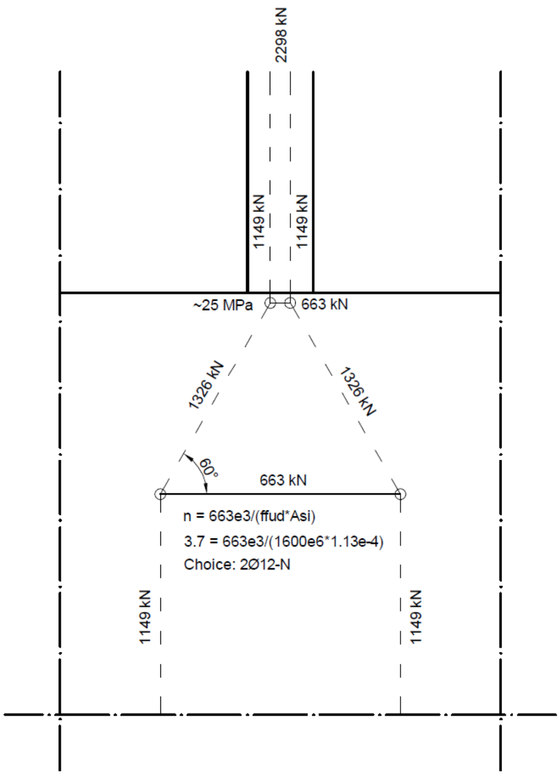


Figure 46 Strut and tie model for design of AFRP splitting reinforcement.

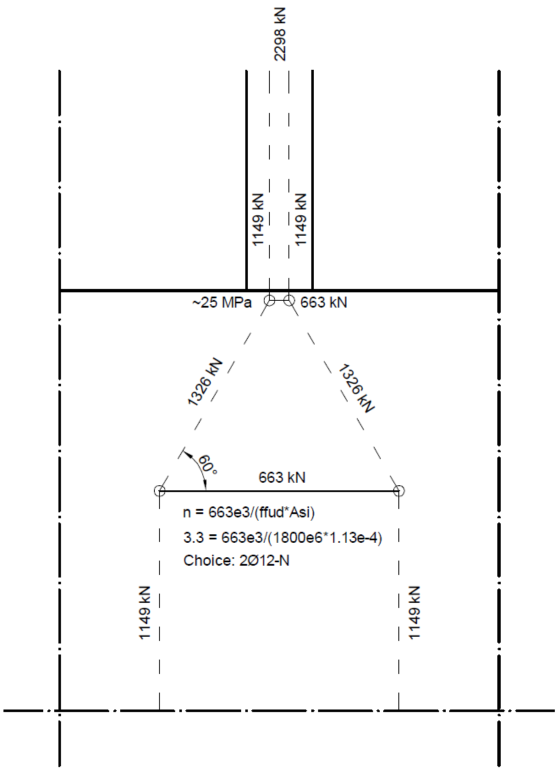


Figure 47 Strut and tie model for design of CFRP splitting reinforcement.

3.8 Plain and fibre reinforced concrete

If the section size would be slightly increased, high capacity concrete is used and the tensile capacity is utilized it might be possible to eliminate reinforcement from the design altogether, at least theoretically.

Tensile strength in concrete is a somewhat uncertain property with large statistical scatter. In calculations in the ultimate limit state, the lowest provided value of strength, i.e. the 5 % fraction, should be used and should additionally be reduced with the usual material partial factor of 1.5. Concretes with higher compressive strength also have higher tensile strength, but the increases are not linearly related. A C90/105 grade, for example, has 5 MPa mean tensile and 90 MPa characteristic compressive strength whereas a C12/15 has 1.6 MPa and 12 MPa correspondingly. In the weaker concrete the relation of tensile to compressive strength is 13 % and in the stronger only 5.6 %. The same phenomenon can be seen for the modulus of elasticity, which also increases nonlinearly with the compressive concrete strength.

If a structure should be built with brittle unreinforced materials such as plain concrete, the geometry of the elements must typically be altered compared to conventional designs. To do this properly, the designer must be conscious of the force pattern that the design loads create and shape the elements accordingly. For example the basement wall studied here must be shaped a bit more like the moment diagram caused by the principal load to make good use of the material. That is with a wide base and slimmer top, possibly with some arching along the span.

3.8.1 Wall

The wall in the studied case has a problem hard to solve. Since the ground water levels are so high and the basement must be waterproof, the interface between the slab and wall becomes tricky to solve without continuous reinforcement. This is unfortunately also the section where the highest moment appears.

3.8.1.1 Completely unreinforced

If there were no need for water-tightness, the bottom joint could be designed as hinged with no resulting support moment. Thus the greatest moment would appear in the field somewhere in the middle of the wall height and would be much easier to deal with than in the joint region. A quick check of this scenario in Frame Analysis revealed that the highest moment with the design loads except the slab induced moments and normal force, would be 57 kNm as can be seen in Figure 48. Navier's formula apply, which gives:

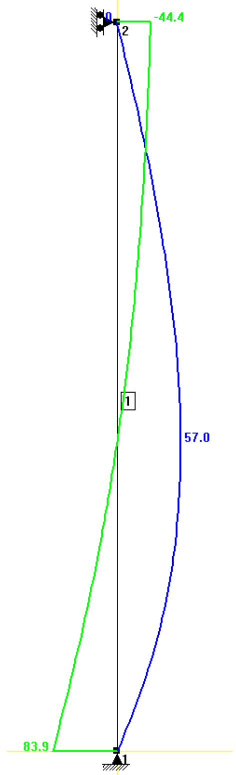


Figure 48 Load effect hinged wall.

$$\sigma = \frac{M}{I} z \tag{3.4}$$

$$where\ I = \frac{bh^3}{12};\ z = \frac{h}{2}$$

If for example C55/67 is chosen, the characteristic tensile strength is 3 MPa (SIS 2008). With the partial factor $\gamma_c = 1.5$ the design strength is 2 MPa. If this is put into the equation above, the minimum thickness to handle a moment of 57 kNm is:

$$h = \sqrt{\frac{6M}{\sigma}} = \sqrt{\frac{6*57e3}{2e6}} = 0.4135 \text{ m} \tag{3.5}$$

With the above stated conditions then, a similar wall as that in the case study project, but hinged and with 415 mm thickness could be built without reinforcement if the intersection with the slab would not have to be waterproof.

There are those who claim that the type of joint described above could in fact be made waterproof without continuous reinforcement with the help of external membranes that are glued over the joint interface. A membrane or sheet could also be cast into the slab so that it is projecting and could be fixed into the wall. In this way there would be a continuous membrane over the joint.

Another clear issue with a plain concrete basement wall is restrained shrinkage cracking. Certainly there would be no restraining reinforcement, but surrounding construction elements will most likely shrink at different times and at different paces thus creating stresses. In part this can be handled with fibre reinforcement, but e.g. in the connection to the foundation slab it is likely that the stresses will be too large also for such a setup.

In order to envision plain concrete in today's industrial construction production, unconventional solutions and development of the codes must be applied in most possible cases. According to the present code, it is allowed to design with plain concrete, but as soon as there is need for any reinforcement somewhere, the calculations must be changed and include minimum reinforcement etc.

3.8.1.2 Partly reinforced

One solution to the joint problem and to find a way forward for the studied project could of course be to add reinforcement only in the joint to enable transfer of forces between the two structural parts. Doing this however would mean either placing minimum reinforcement in the rest of the concrete, or breaking the idea of the code regulations as mentioned in the previous passage. There is a paragraph in Eurocode 2; 12.9.2 that is somewhat ambiguous regarding reinforcement in concrete construction joints. It states that if tensile stresses are likely to appear over construction joints, then reinforcement should be placed to reduce crack widths. The most reasonable interpretation though is that this paragraph is not intended for joints transferring large moments as is the case in the present construction elements (SIS 2008).

If it could be allowed, a 300 mm thick section of concrete strength class C55/67 would need 7Ø12 steel bars over the connection interface to handle the critical moments of 87.1 kNm in ULS and 60.4 in the serviceability limit state. If durability is the main concern, this reinforcement could advantageously be stainless. FRP bars are also thinkable but would have to be designed separately. As the bars are on the outside of the structure, temperature and fire aspects pose no problem. The bars placed in the wall concrete section are to be seen in Figures 49 – 51.

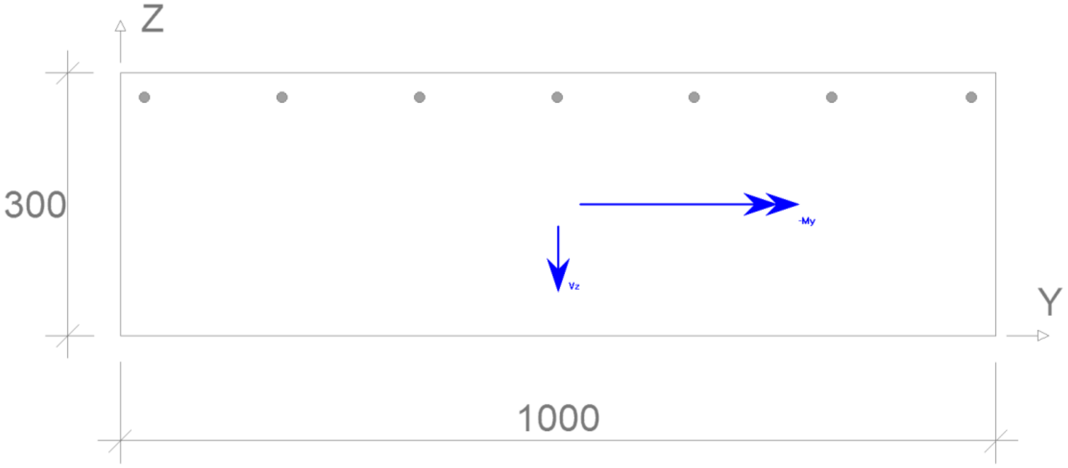


Figure 49 Concrete wall section partially reinforced

If this reinforcement is assumed to make the connection rigid, the previously defined load cases used for the reinforced sections can be used. The critical field moment is then 25.5 kNm in ULS and 18.1 in SLS. This would appear in the unreinforced part of the wall, wherefore the method based upon Navier's formula used above in Section 3.8.1.1 can be applied:

$$h = \sqrt{\frac{6M}{\sigma}} = \sqrt{\frac{6*25.5e3}{2e6}} = 0.2766 \text{ m}$$

Hence 300 mm as assumed in the joint reinforcement calculation above is enough and a theoretically sustainable solution is found. It has been concluded before that the accidental load is so small compared to the soil pressure that the moment will never switch sign, why designing for opposite moments never come into question for the fully developed load situations. In this case however with practically no moment capacity in the opposite direction at the joint, it should perhaps be prescribed that the soil must be refilled before the superstructure is built.

The discontinuity regions under the columns must of course be solved in some way also for these alternatives. The tensile force first calculated in the strut-and-tie model in Section 3.5.2 is 663 kN. If the tensile strength of concrete C55/67 is utilized, the required area to take up this force would be:

$$A_{min} = \frac{F}{f_{ctd}} = \frac{663e3}{2e6} = 0.3315 \text{ m}^2$$

The nature of this tensile force is that it is distributed. The above area means the force would have to distribute over about one metre height, which does not seem impossible. However, since plain concrete does not redistribute stresses in the same way as reinforced concrete a more detailed analysis would be required in a sharp design situation.

Apart from requirements concerning load-bearing capacity and crack width limits structural elements should also fail in a sufficiently ductile manner. A plain concrete member though fails in a very brittle way. To increase ductility and toughness, dispersed fibres can be used. These also help in taking care of another challenge, namely that of shrinkage, by generating finer cracks and crack spacing. The fibres do however not provide much resistance, nor in compression, tension or bending (Löfgren 2014-09-04).

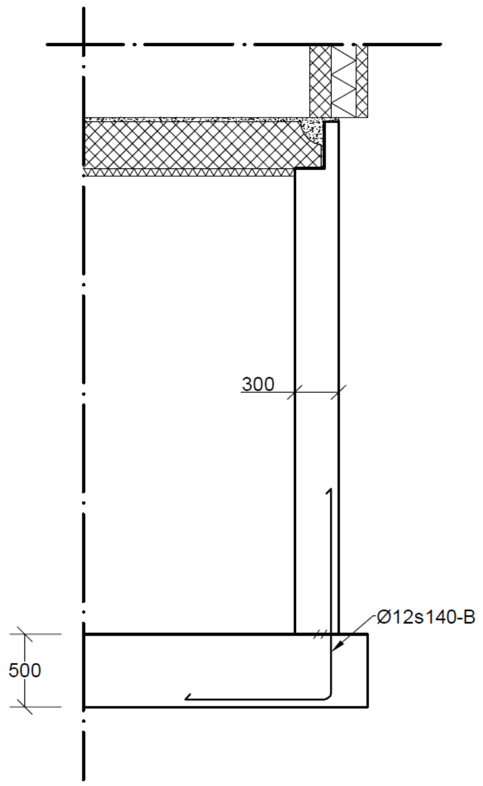


Figure 50 Partly stainless steel reinforced wall.

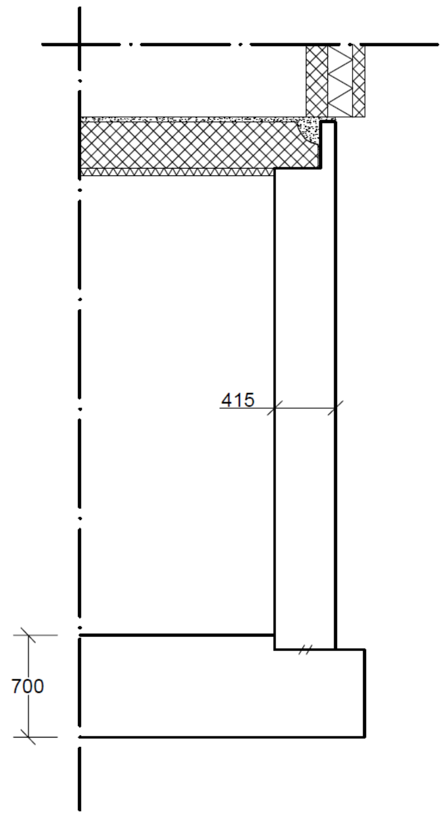


Figure 51 Completely unreinforced wall.

3.8.2 Slab

The cross-section, concrete strength class and boundary conditions are the major parameters to alter in order to achieve sufficient capacity in the slab without reinforcement.

If the boundary conditions are left as they are in the case study project and the added self-weight is neglected, the minimum concrete section could be calculated via the same method as was applied for the wall. The largest moment in ULS, after reduction of the unreasonable peak in the FE-model was found to be 165 kNm. This gives:

$$h = \sqrt{\frac{6M}{\sigma}} = \sqrt{\frac{6*165e3}{2e6}} = 0.7036 \text{ m}$$

Since the largest moments appear when the water pressure is high and the slab is bent upwards, the extra self-weight is advantageous. Because of this extra margin, the odd centimetres can safely be scaled off the thickness and hence the needed thickness is, rather coarsely determined to 700 mm.

This moment capacity is of course only needed in very small portions of the slab. In fact 80 kNm capacity covers the whole area except a few regions around supports. In the more demanding parts, the slab could either be geometrically strengthened by beams or as discussed for the wall, by adding reinforcement only where it is needed locally. A flexural capacity of 80 kNm is achieved by:

$$h = \sqrt{\frac{6M}{\sigma}} = \sqrt{\frac{6*80e3}{2e6}} = 0.4899 \text{ m}$$

This is suitably rounded up to 500 mm, only 100 mm more than what the reinforced slab needs to be. The added height also comes to good use in minimizing the reinforcement thanks to the increased lever arm. The results of partly reinforced designs in ultimate and serviceability limit states can be seen in Figures 52 and 53.

3.8.3 Concluding remarks

Brittle construction materials have been used for thousands of years without reinforcement. For many applications it may be a suitable solution, in some cases granted that the codes are further developed. In the studied project though, the demands of watertightness of the basement makes it hard to use here. A question raised by this conclusion is how the basement walls along Venice’s canals were built. Presumably the walls are masonry, most likely brick and lime mortar. Are they water-tight or is leakage accepted and the basements practically useless?

Another take on the problem of cracks in watertight structures is to add a second watertight layer to the building and allow the load-bearing part of the wall or slab to crack freely as long as it manages to support the loads. This would lift many demands from the design of the structure and could even be the most economical solution.

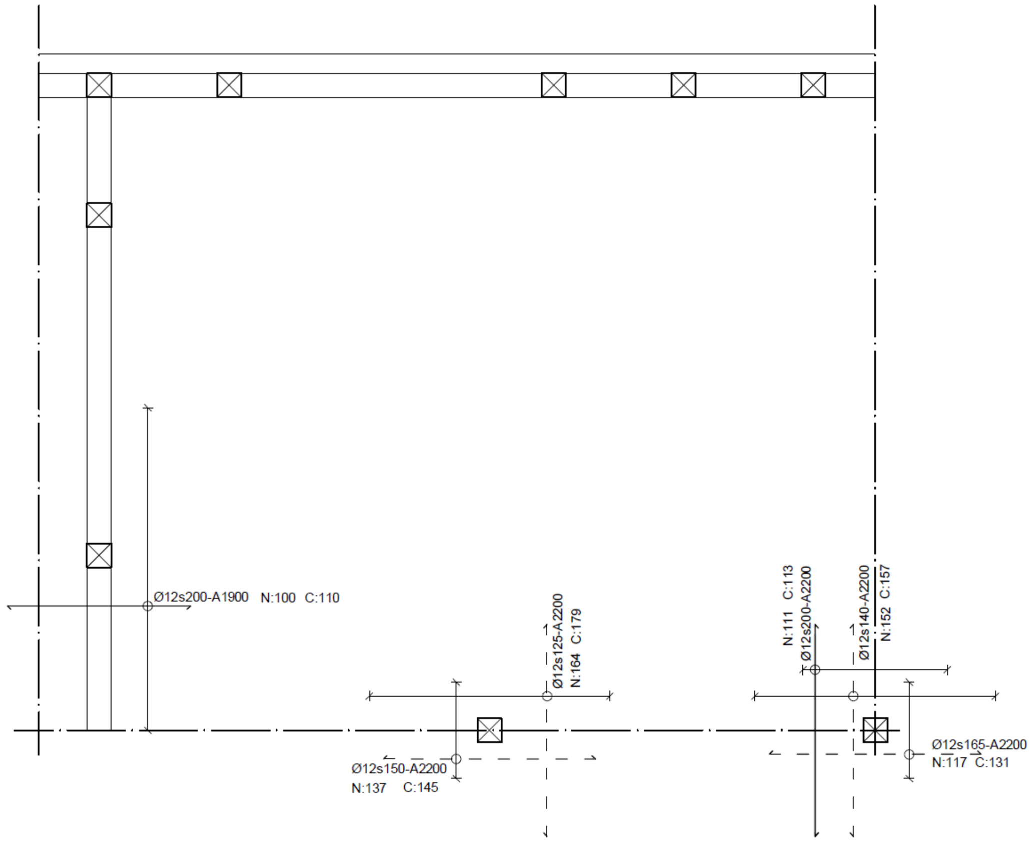


Figure 52 Needed arrangement of steel reinforcement with regard to the ultimate limit state.

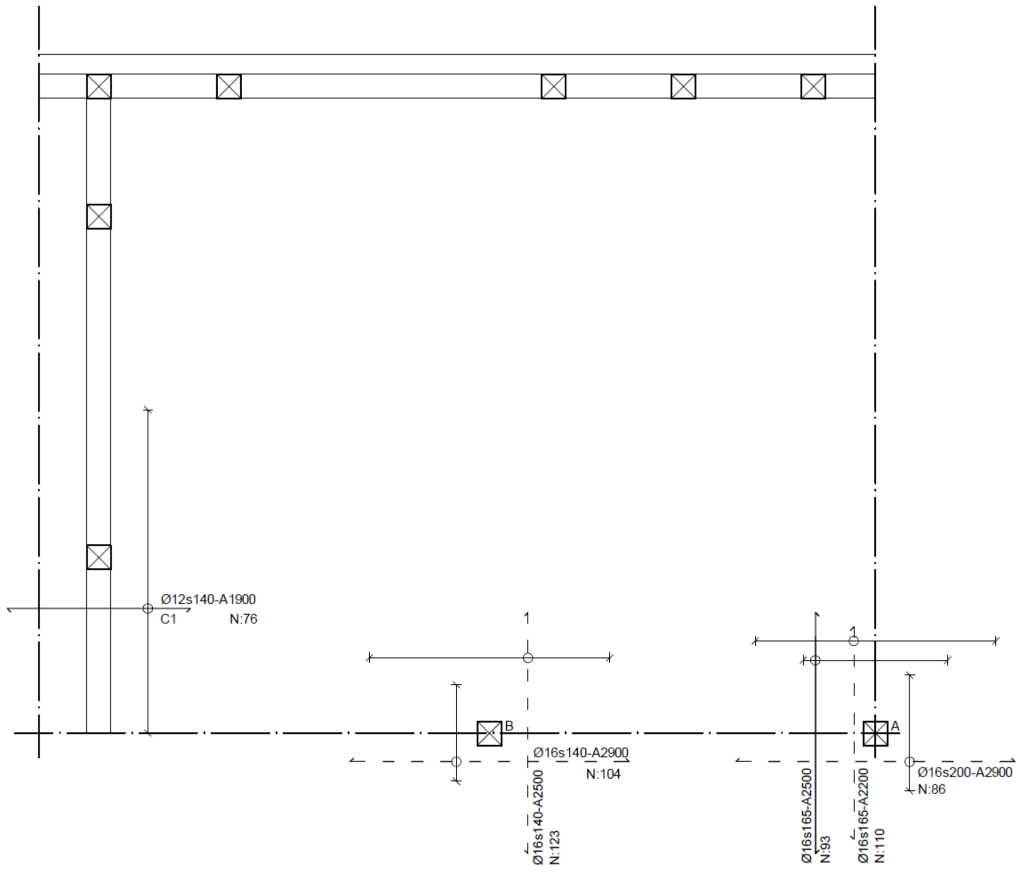


Figure 53 Needed arrangement of steel reinforcement with regard to the need in the serviceability limit state.

4 Life cycle analysis

When two or more products, services or reforms differ with regard to several aspects, an objective comparison of which is better becomes tricky. The parameters which are different have to be graded with regard to importance and an overall definition of what is desirable must be clearly stated.

In an attempt to make a comparison as unbiased and realistic as possible, a life-cycle approach was carried out. There is no doubt that life-cycle analysis (LCA) is somewhat arbitrary and that results must be considered critically. Yet important decisions have to be made every day on matters with high uncertainty and with such preconditions a life-cycle perspective may provide important input. Life-cycle analyses are usually focused on economy, often called life-cycle costing (LCC); sometimes though the analyses compare other factors such as environmental sustainability.

The life-cycle theory was born in the 1960s when the US military realized that they spent much more money on operation and maintenance through the life of their equipment than the cost of the initial purchase, whereas the purchase decision was almost exclusively based on the latter. This insight led to the development of life-cycle costing where the total cost of a product was calculated and compared (ISIS 2006a).

To perform a life-cycle analysis the cost or environmental impact of each cost group must be estimated. For buildings and other structures costs can typically be placed into three categories: Initial costs which are all costs until inauguration of the structure, operational costs which are all costs linked to the usage and maintenance of it and finally end-of-life costs which are the costs connected to the disposal. Some parts of the end-of-life category may actually generate incomes instead of costs, depending of the scrap value or reusability of the constituent materials of the product (ISIS 2006a).

These major groups of cost accommodate innumerable smaller components called cost items. The level of detail of the analysis should be chosen depending on the required refinement of the results. Once all cost items are defined they are ordered in a so called Cost Breakdown Structure (CBS). A similar tool, much used in project management is the Work Breakdown Structure (WBS) with the difference that project sub-tasks are studied instead of cost items.

Some of the cost items in the studied case are independent of the variable parameters in the study. These items are denoted fixed costs. An example of a fixed cost is the geotechnical survey of the lot, which will be the same regardless of the choice of reinforcement. The different items also have different levels of price certainties. Some costs are hard to assess, whereas others may be available in black and white from material suppliers. Apart from the obvious costs of labour and material, costs related to future overhauls and possible environmental hazards can also be included depending on the focus of the study.

For products with long service life periods the cost of capital and inflation become important factors in the comparison. For this reason all costs are appropriately normalized to their corresponding present value. Additionally the predicted service life periods of the compared alternatives may be different. In these cases the cost of the alternative with longer service life can be discounted with its residual value, or alternatively the cost of the one with shorter life can be increased corresponding to the remaining time. In this project the different life lengths have been accounted for by presenting the different costs divided by the number of service years. These circumstances are further developed in Section 4.1.3.

There are six steps to a life-cycle costing according to ISIS (2006a). They are:

1. Analysis planning:
   Definition of the purpose and scope of the analysis. Identification of the asset being studied. Definition of timeframe and planned operation. Identification of any limitations to the study.

2. Model development:
   Creation of a cost breakdown study (CBS). Differentiation of the importance, variability and uncertainty of cost items.

3. Model usage:
   Generate results for each possible alternative for subsequent comparison. Identify the most important cost drivers. Validation of model with a known historical project if available.

4. Sensitivity analysis:
   Varying of the important cost items to monitor the sensitivity of the model.

5. Result interpretation:
   Preparation of results and drawing conclusions from it.

6. Informed selection:
   Ranking of the different alternatives based on various parameters.

In this chapter as good as possible approximations of the economic and environmental implications of different reinforcement approaches are presented. The same concrete section that is defined in Chapter 3 was further evaluated using life cycle analysis based upon the reinforcement need for the different options.

4.1 Economics

Most agree that environmental issues have to be converted or interpreted as money to be fully appreciated. There have been attempts to put real prices on environmental damage in order to consider this effect in economic life-cycle analyses. The system of carbon credits is such an example. In this study though, the two fields were separated. In this section the pure economic aspects are treated, whereas in the next the environmental differences are treated a bit more freely.

The economic analysis that was carried out followed the 6 steps stated in the introduction to Chapter 4. The study focused on costs from the proprietor's point of view, therefore including also costs not directly linked to the construction of the building such as e.g. cleaning, maintenance, overhauls and demolition.

4.1.1 Analysis planning

In this project, the aim of the comparison was to assist in the selection of the most suitable reinforcement materials for various aggressive environments. The most promising materials were selected as described in Chapter 2 and have been used in the design of part of the basement of an office building in both the ultimate and serviceability limit states as presented in Chapter 3. Thereafter these designs were evaluated for cost. The service life of the building was set individually by the service life expectations of each alternative. The more durable options than carbon steel were rewarded with longer life lengths as is further discussed in Section 4.1.3.

4.1.2 Model development

In the model development all cost items were defined and categorized. Those deemed sufficiently important to be included in the CBS are gathered in Figure 54. Naturally the choice of cost items is up to the LCC analyst and the chosen items in Figure 54 are not to be seen as the unique solution. The relative cost of the alternatives is more interesting than the total cost of the whole project. With this in mind the cost items that differ between materials are the most interesting. The reason that the fixed costs were nevertheless included is because they will point out approximately how much the total investment changes due to the changes in an isolated part of it.

All costs were graded with regard to predictability and variability. Predictability aims to quantify to what certainty a certain cost can be estimated. Variability describes the extent to which the cost item varies between the different reinforcement options.

The results presented in Chapter 3 of the studied wall and slab parts were extrapolated to represent the whole basement. The measurements of this studied region can be seen in Figure 55. The whole floor is large and the studied part represents only about 3 % of the total area.

Just as the cost items can be chosen by the analyst, so can the combination and scaling of them. Some combinations were performed and are briefly described in the following sections. More detail can be seen in Appendix K.

In the rest of this section each of the cost items is explained shortly. At the end Table 16 sums up the predictability and variability of all the cost items.

Life cycle cost

Cost Breakdown Structure

Design	Material	Construction	Operation&Maintenance	Other
Planning, management, permits	Hollow core slabs	Excavation	Management	Rentable space
Architecture	Steel	Ground preparation	Cleaning	Environmental damage
Geotechnics	Roof	Rebar installation basement	Caretaking	Demolation
Structural design foundation	Facade	Piling	Overhauls	
Structural design superstructure	Interior			
	Installations			
	Basement concrete			
	Basement reinforcement			
	Fire protection			

Figure 54 Cost Breakdown Structure.

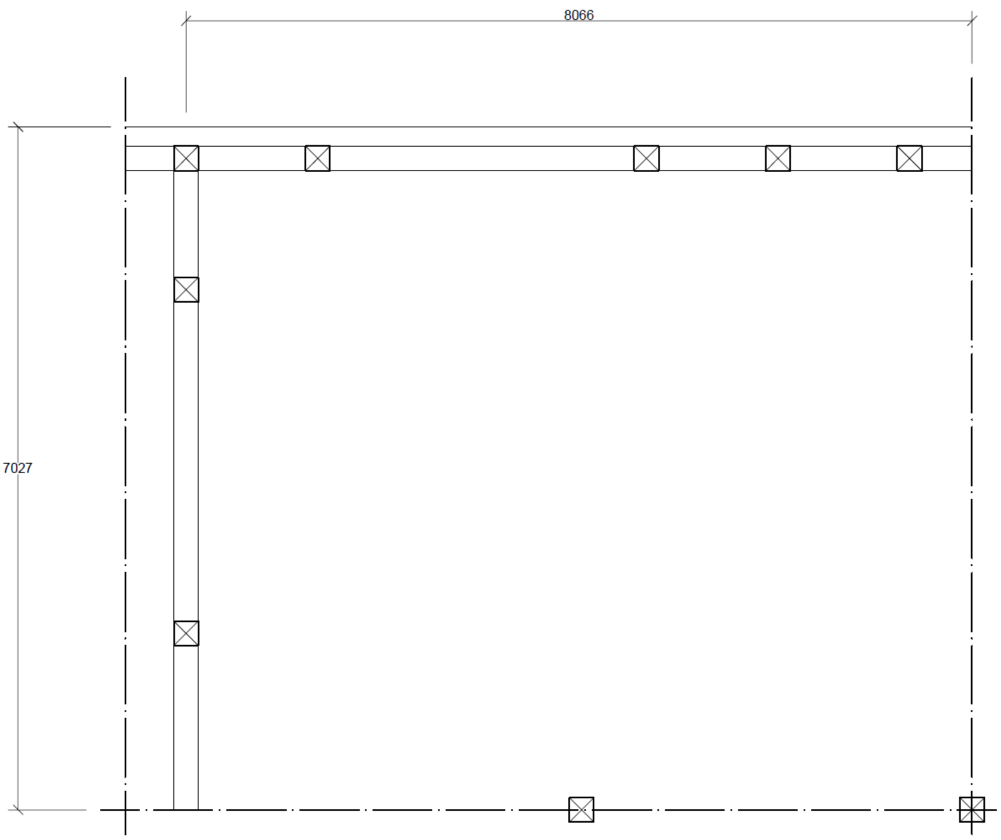


Figure 55 Measurements of the studied slab section.

4.1.2.1 Design

Design costs are those linked to planning, technical and economical design of the structure. This includes architectural, economical and engineering services in several instances from the client through authorities and consultants.

Planning, management, permits

The first step of a building project is to state needs and define a building program. This is a complex process with many variables. This cost item also includes application of permits and project management.

Architecture
The architectural design is unfortunately fairly independent of the structural design in many building projects. Engineers are consulted when the spaces are already largely defined and a structural system is proposed.

Geotechnics
Geotechnical survey of the lot is necessary for any larger project. The type of soil and its bearing capacity must be verified before foundation type is chosen.

Structural design foundation
The structural design of the building is where one of the clearest differences between the materials appears. This service was here further split into foundation and superstructure to clarify differences that may exist between the alternatives. Those with less common reinforcement approaches were considered to require more engineering resources.

Structural design superstructure
In this project the superstructure is not affected by the choice of reinforcement material in the basement and was therefore low in variability.

4.1.2.2 Material

The material category was one of the most important in this comparison. It includes the materials themselves, but also the purchasing, transport and in most cases installation of them. Parts of the construction process which were more labour-oriented are gathered in the next category.

Hollow core slabs
The floor slabs from entry level and upwards are made of prefabricated hollow core elements.

Steel
The structural system of the superstructure consists of prefabricated concrete and steel members. The steel is bought prepared and ready to mount, i.e. cut to length, with end details and painted.

Roof
The roof is a fixed cost that is rather highly predictable. Apart from its initial cost, it was assumed to need overhauls every 50 years.

Façade
The façade covers a large area and is therefore influential. Just like the roof it is assumed to need major overhauls every 50 years.

Interior
The interior of the building is the single most expensive part of the studied case and is also subjected to changes or renewals at close intervals. This time period was here chosen to 25 years.

Installations
In modern buildings there are many installations that handle the indoor climate and similar functions. These have a shorter service life than the building and will have to

be exchanged and/or overhauled several times during the building's service life. Like the interior the exchange frequency was set to 25 years.

Basement concrete
The concrete in the basement varies between the reinforcement alternatives, both in amount and strength class.

Basement reinforcement
The single most apparent variable factor in this comparison is the actual reinforcement material for the studied basement.

Fire protection
Some of the reinforcement materials are sensitive to high temperatures. Therefore they will need some kind of measure to meet the fire class of the building. Such costs were summed under this cost item.

4.1.2.3 Construction
The construction category includes costs of primarily non-material nature.

Excavation
Excavation is a large part of the construction cost and will therefore have an important diluting effect in an LCC analysis.

Ground preparation
In this cost item all activities between excavation and casting of the foundation slab was included.

Basement reinforcement installation
The amount of reinforcement, rather than its cost, is the most decisive factor for the labour cost of placing the reinforcement bars. Therefore it is separated from the material cost. In fact, FRPs, which are some of the lightest reinforcement materials and therefore probably the easiest to handle on site, are also among the most expensive. A labour cost per metre bar was derived from the steel alternative and used for all materials.

Piling
The building is founded on soft soil near Göta River. Piling is necessary to support its weight.

4.1.2.4 Operation and Maintenance
The LCC analysis was performed with the client or end user in focus. The operation and maintenance costs are highly influenced by the above cost categories. Apart from overhauls, these costs are not part of the decision material since they do not differ between the alternatives.

Management
The administration of the building and its maintenance induce a cost for the end user.

Cleaning
Cleaning is a large part of the ongoing maintenance cost. Most of it was assumed to be borne by the tenants, but some will belong to the public spaces of the building.

Caretaking
Things need to be mended and taken care of. In a large building this generates a substantial cost.

Overhauls
This is the major cost item in this category. It contains exchanges, reparations etc. It is highly dependent on the service life of the building.

4.1.2.5 Other

This category of costs gathers up those that are hard to categorise or that would otherwise comprise their own categories.

RenTable space
The total income from the building was not included in the evaluation. However, for fairness any differences in the amount of renTable space between alternatives were considered.

Environmental damage
There may be economical consequences of some environmental aspects of a building. In this analysis the cost was the result of a fixed estimated base cost which is multiplied by the environmental damage grading in Section 4.2.

Demolition
This cost item embodies all expenses connected to the future disassembly, recycling and deposition of the building. Because of discounting as discussed in Section 4.1.3, this expense in present value terms is highly dependent of the service life of the building.

Table 16 Predictability and variability of all cost items.

Cost item	Predictability	Variability
Planning, management, permits	Medium	Low
Architecture	Medium	Low
Geotechnics	Medium	Low
Structural design foundation	Medium	High
Structural design superstructure	Medium	Low
Hollowcore slabs	High	Low
Steel, painted and mounted	High	Low
Roof	High	Low
Façade	High	Low
Interior	Medium	Low
Installations	Medium	Low
Basement concrete	Medium	High
Basement reinforcement	Medium	High
Fire protection	Medium	High
Excavation	High	Low
Ground preparation	Medium	Low
Rebar installation basement	Medium	Medium
Piling	Medium	Low
Management	Medium	Low
Cleaning	Medium	Low
Caretaking	Medium	Low
Overhauls	Medium	Medium
RenTable space	Medium	Medium
Environmental damage	Low	Medium
Demolition	Medium	Low

4.1.3 Model usage

The calculations needed in the LCC analysis were performed on several linked Excel-sheets which are presented in Appendices G - K. The sheets were made as parametric as possible to facilitate changing of values and performing sensitivity analysis.

According to economic models, future costs should be discounted when compared with current costs. The idea is that an investor has several options for an investment and demands a certain percentage of revenue annually. In this project 5 % was assumed as this percentage. Thereupon the inflation reduces the pay-back somewhat by constantly decreasing the value of all assets. In Sweden there is an official policy of striving towards maintaining an inflation of 2 %. The effective investment pay-back is hence 3 % in this case. This principle was applied both for unique and returning cash-flows, positive and negative. The formulae to calculate the present value of a future cash-flow are (ISIS 2006a):

$$PV_s = \frac{F}{(1+i)^t} \tag{4.1}$$

$$PV_r = C\left(\frac{1-(1+i)^{-t}}{i}\right) \tag{4.2}$$

$where$ $F =$ Future cost
$PV_s =$ Present value of future single cashflow
$PV_r =$ Present value of future yearly cashflow
$i =$ Effective interest rate
$t =$ Number of years
$C =$ Periodic future cost

An important choice in the Life cycle costing (LCC) is how to treat the different service lives of the alternatives. The design service lives specified in the real project are 50 years for all but the foundation which has 100 years. These service lives are minimum values and represent a time period under which the structure is intended to withstand without major repairs. One way of making use of the better durability of some of the studied reinforcement alternatives would be to prescribe longer design service life for those. In this analysis the assumptions of the different service lives are inspired by ISIS' (2006a) educational module on LCA. In effect it is assumed that ferritic stainless steel provides a possibility of 25 %, FRP and austenitic stainless steel 50 % and plain concrete 100 % increased service life.

In order to fairly account for the differences in service life, the final cost for each alternative was divided by the number of service years. It is not fair to use the service life of the foundation, since most of the structure is designed for the shorter service life. An idea is of course to imagine a redesign of the whole building to meet the service life of the foundation. This action would imply overall increased costs and is more of a political or economic nature than a structural and was therefore not further pursued. To account for the increased costs of a longer service life, future overhaul expenses were included where applicable. It was assumed that installations and interior fittings would be exchanged every 25 and façade and roof every 50 years. Where the total service life of an alternative is not an even multiple of 25 or 50 years, the pertinent proportional part was used. Further it was assumed that some parts of the overhauled building components could be reused, for example in the case with the roof it may be sufficient to exchange the outer cladding and its bearing lath. To include this aspect, overhaul costs were estimated to half the respective initial cost.

The case project is very harsh on some of the reinforcement alternatives provided by due to the tough restrictions on crack widths and fire demands. Both the unreinforced alternatives and those with low modulus reinforcement are punished. Most of the present stock of FRP reinforced concrete is in the infrastructure sector with known good results. Therefore a project with the mentioned requirements is well fit to test the suitability of FRP reinforcement also in buildings. With added fire protection on exposed surfaces and very high reinforcement amounts it was proven possible. However in the case of glass and basalt FRP the wall thickness had to be increased and the reinforcement distribution is on the verge of impossibly tight. The economic result of these consequences can be seen in Appendix K and Table 17.

To demonstrate what the results could be in a different project with less severe serviceability demands, all alternatives were compared on ultimate limit state reinforcement amounts as a complement to the final amounts, governed by serviceability. Both reinforcement distributions are presented in drawings throughout Chapter 3. When comparing the ULS reinforcement, the amounts are much evener between the options and would be more representative for many applications.

4.1.4 Sensitivity analysis

It is ineviTable that there will be several parameters in a life-cycle cost analysis that are rather uncertain. In order to find the span of results in which it is probable that the real value will be the uncertain parameters can be altered within a reasonable range. The changes in the final result are noted for the variations in each of the parameters and conclusions can be drawn on how certain the result is and how important each of the factors is for the result.

The part of the total cost derived from the studied basement is small. Therefore changes in these parameters will not change the final sum dramatically. Instead the most impacting parameters are those that affect large areas of the building, i.e. the floor area, installation costs and so on. If just the group of directly variable costs are studied though, the span of difference is several times the price of the alternative with ordinary reinforcing steel.

The only cost item whose predictability is deemed as low in Section 4.1.2 and which is therefore most interesting to study the sensitivity of is “Environmental damage”. When the base cost was varied between 0 and 100000 USD the total project cost was changed on the third significant Figure number by one or two steps. For principal conclusions it can be determined that an inaccurate assumption will not make any crucial changes.

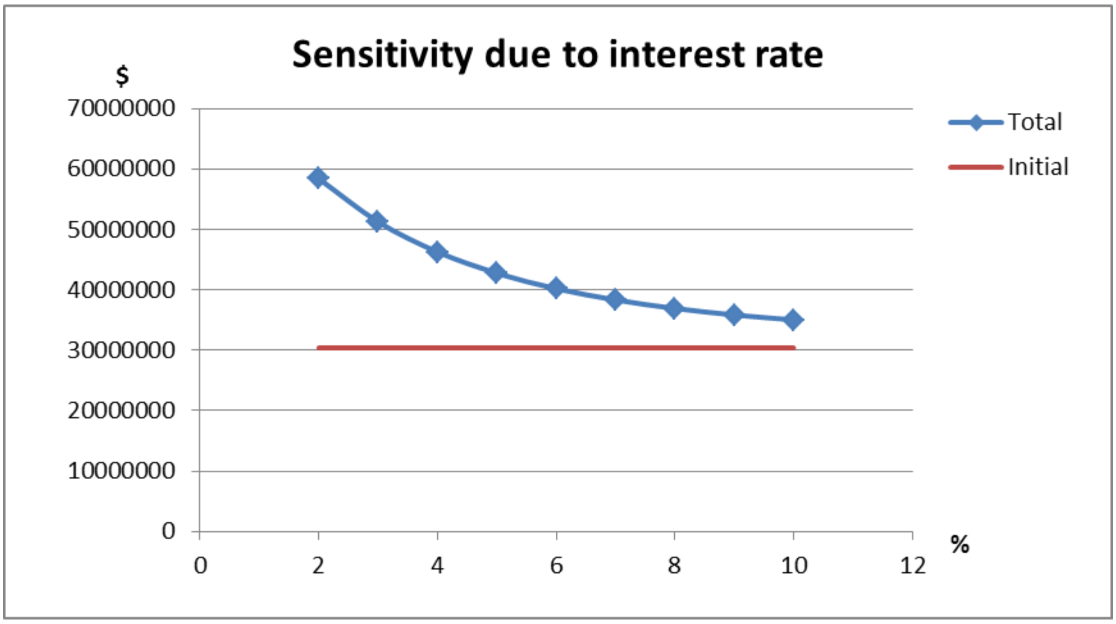


Figure 56 Plot of the total cost of the alternative with ordinary reinforcing steel for different interest rates.

Another uncertain parameter in the study is the effective interest rate demanded by investors. Changes to this parameter does change the total LCC of the building significantly. The initial costs do of course not change as they are unaffected by the discounting formulae in Section 4.1.3.

In Figure 56 the effect of changes to the interest rate can be observed. Since inflation is set to 2 %, no interest rates below this value are included. It can be seen that the effect of future costs diminishes exponentially with increased interest rate. Thus all future costs loose in importance as the interest rate increases. Alternatives with long expected service lives are thereby benefited as the increased cost for overhauls get increasingly unimportant. This effect gets even more impacting on the per-year costs where the costs are divided by the number of service years. Other effects such as a relatively larger part of the total cost originating from reinforcement material prices are so small that they do not affect the bigger picture.

A stabilizing factor is that there are many cost items. This means that the model is fairly sTable for inaccuracies in single factors one at a time, which is good. The single most important factor answering for about a third of the total cost is the interior of the building; i.e. everything except the structure, climate envelope and installations.

4.1.5 Results interpretation

Before interpretation of any Figures, the reasonability of the final result should be judged and compared with some reference value. In "Bygganalys little book of prices" (Bygganalys 2012) there are approximate costs per square metre for different types of buildings. The most suiTable standard type is "normal standard office, prefabricated, 4 floors including basement" which 2012 cost 17185 SEK/m² gross area. In the last two years there has been insignificant inflation in Sweden, wherefore the prices can still be used. The gross area of the building including the basement is about 14000 m². This would give a total price of 240 million SEK which converts to about 34 million USD. In the LCC in this project more cost items were included than in Bygganalys (2012). Most of the added costs take place in the future and therefore it is better to compare the initial rather than the total costs with the template in Bygganalys (2012). The alternative with ordinary reinforcing, specified in Chapter 3, steel turns out at just over 30 million USD, i.e. about 88 % of the guide's value. This is, regarding the complexity and uncertainty of the study, a fully acceptable discrepancy. As a second check the value was judged internally at the design office at COWI where it was confirmed to be in the right order of magnitude.

In a first rough interpretation it can be concluded that all the alternatives to ordinary steel reinforcement are less expensive on a life-cycle basis. The large fixed costs are spread over a longer service life while the increase in the variable costs needed to increase the service life influences to a much lower extent. Initially more costly alternatives with longer life expectations are further benefitted in the comparison by the increased discounting described by equations 4.1 and 4.2. Table 17 shows some of the results of the LCC analysis. For a more complete and complex version, the reader is referred to the calculation sheet in Appendix K.

Costs denoted “variable” in the Table are those that are affected by the choice of reinforcement and conversely “fixed” are independent. Another distinction is between “initial” and “future” costs. The prior includes all expenses from initial planning until the client takes possession of the building, whereas the latter stands for any costs thereafter. Costs denoted “total” include both initial and future items. To clarify the comparison, all costs are normalized with the ordinary steel alternative as index 100.

Table 17 Life-cycle costs for the different reinforcement approaches.

Alternatives All values in 1000 $	Initial variable	Initial variable per year	Initial with fixed	Total	Total per year
Ordinary reinforcing steel ULS	716.3 100 %	14.3 100 %	30379 100 %	42732 100 %	854.6 100 %
Ordinary reinforcing steel SLS	745.7 100 %	14.9 100 %	30409 100 %	42762 100 %	855.2 100 %
Ferritic stainless steel ULS	900.8 125.8 %	14.4 100.6 %	30564 100.6 %	44314 103.7 %	709.0 83.0 %
Ferritic stainless steel SLS	965.5 129.5 %	15.4 103.6 %	30629 100.7 %	44378 103.8 %	710.1 83.0 %
Austenitic stainless steel ULS	1143 159.7 %	15.2 106.4 %	30807 101.4 %	45881 107.4 %	611.7 71.6 %
Austenitic stainless steel SLS	1249 167.4 %	16.6 111.6 %	30912 101.7 %	45986 107.5 %	613.1 71.7 %
GFRP ULS	858.3 119.8 %	11.4 79.9 %	30521 100.5 %	45583 106.7 %	607.8 71.1 %
GFRP SLS	1202 161.1 %	16.0 107.4 %	30865 101.5 %	45927 107.4 %	612.3 71.6 %
BFRP ULS	935.1 130.5 %	12.5 87.0 %	30598 100.7 %	45646 106.8 %	608.6 71.2 %
BFRP SLS	1703 228.4 %	22.7 152.2 %	31366 103.1 %	46414 108.5 %	618.8 72.4 %
AFRP ULS	1105 154.3 %	14.7 102.9 %	30768 101.3 %	45790 107.2 %	610.5 71.4 %
AFRP SLS	2520 338.0 %	33.6 225.3 %	32183 105.8 %	47205 110.4 %	629.4 73.6 %
CFRP ULS	1282 179.1 %	17.1 119.4 %	30946 101.9 %	45906 107.4 %	612.1 71.6 %
CFRP SLS	1693 227.1 %	22.6 151.4 %	31356 103.1 %	46317 108.3 %	617.6 72.2 %
Mostly unreinforced austenitic ULS	636.4 88.9 %	8.49 59.2 %	30313 99.8 %	45306 106.0 %	604.1 70.7 %
Mostly unreinforced austenitic SLS	651.6 87.4 %	8.69 58.3 %	30315 99.7 %	45307 106.0 %	604.1 70.6 %
Completely unreinforced ULS	637.4 94.0 %	6.73 47.0 %	30378 100.0 %	47050 110.1 %	470.5 55.1 %
Completely unreinforced SLS	637.4 87.4 %	6.73 45.1 %	30378 99.9 %	47050 110.0 %	470.5 55.0 %

4.1.6 Informed selection

The purpose of this project was not to choose one alternative reinforcement approach over all others. They all have their pros and cons; each of them may be the most suitable choice is a certain situation. There are also considerations that cannot simply be boiled down to costs in a life-cycle analysis. Such considerations could be that even though it can be proved to work in a structural analysis, many engineers would not feel comfortable with completely plain concrete members in a load-bearing structure, even though it is allowed formally for certain types of structural members according to chapter 12 of Eurocode 2 (SIS 2008). This may have to do with the higher possibility of decreasing the probability of failure in reinforced concrete. Throughout design codes there are also recommendations to avoid brittle failures, something that plain concrete members are unable to comply with. This last point is valid also for FRP reinforced concrete.

Results from accelerated tests show much higher degradation in reinforcement of GFRP than in CFRP. However, when following the ACI's design code (ACI 2006) this is considered only by increasing the amount of GFRP compared to the unfactored need. According to ISIS both have the same expected service lives. It is not hard to imagine that a cautious engineer might recommend CFRP for a slightly increased cost with this in mind, even though it is not directly shown in an LCC study (ISIS 2006a).

4.2 Sustainability

In this section it is described how the environmental impact of the different reinforcement approaches was evaluated. The scope of environmental impact was chosen as wide as possible including carbon footprint, toxicity, energy consumption, transportation, mining impacts etc. The sustainability related to an option was not linked to money as is sometimes done; instead it will be rated. The result can then be the basis for different policies depending on ideology. The nature of this type of evaluation is that it is extremely uncertain. An economical LCA has high unpredictability, but is still much better than an environmental equivalent. With this in mind, this study is not as advanced as the one in Section 4.1; hopefully though it may give a valuable indication to the assessor of different reinforcement approaches.

To start the analysis, possible environmental damage factors were identified as can be seen in detail in Appendix J. The amplitude, probability and importance were then estimated for each of them. Amplitude in this context means how much the alternative fulfils the damage factor. Probability is included to take account of the variability in the risk of a certain damage of happening. The last parameter, importance, evaluates the consequence of the several damage factors. In order to compensate for the amount of reinforcement in the different alternatives a fourth parameter was introduced.

To obtain a damage grading for each damage factor the values of the four parameters were joined. Typically the four are simply multiplied together; however a few damage factors are of the nature that higher amplitude means improved sustainability. This applies to e.g. recyclability. In these factors ***(1 - amplitude)*** has been used in the multiplication. In yet other cases the reinforcement amount lacks importance and has been omitted.

Finally the ratings of all damage factors were added together into a final figure. This number can then be used in a coarse comparison of the different options.

The rating of the alternatives can be seen in Table 18. For more details on the grading the reader is referred to Appendix J.

Table 18 Environmental rating, low value means low risk of damage.

Alternative	Rating
Austenitic stainless steel	0.76
Plain concrete	1.00
Mostly unreinforced stainless	1.01
CFRP	1.14
Ferritic stainless steel	1.52
Ordinary reinforcing steel	1.78
AFRP	1.87
BFRP	2.39
GFRP	2.68

It can clearly be seen from the results in Table 18 that large amounts of reinforcement were punished in the comparison, but that the amount of concrete also has an important impact. All grades of steel were benefited by their high degree of recycling, even though it shows mostly in sourcing and left-overs on site and not at the end-of-life of the reinforced concrete; at least not yet since the two materials are hard to separate with present technology. The highly chemical processes of manufacturing FRP materials degraded those alternatives due to risks of leaching and emission of volatile gases. Metals on the other hand require very much energy in mining and smelting, whereas FRPs vary in this aspect depending on the fibre, where the mineral fibres have similar processing as metals and aramid fibres just as the matrix is primarily sourced from petroleum with certain environmental risks linked to it.

How the environmental rating would be used is beyond the scope of this project. Perhaps a company with high ambitions on sustainability could use this type of result in a decision-making process where more aspects than economy are evaluated.

5 Concluding remarks

In the beginning of the project, a number of hypotheses and questions were identified. These are listed in Section 1.2. To clearly gather conclusions in the light of the knowledge gained along the project, each of the hypotheses and questions are answered in this chapter.

5.1 Hypotheses

Each hypothesis is stated in italics and thereafter commented.

Reinforcing steel in highly exposed parts of concrete structures such as bridge decks, edge beams and parking house slabs could be exchanged with alternative reinforcement approaches with economical, practical and sustainability benefits.

Keeping in mind that the results presented in Chapters 3 and 4 are based one single example project, it does seem that this hypothesis holds, at least in parts. The basement is the only part which is altered and is also, as the hypothesis states, the part which can be considered as highly exposed due to its use as parking and its constant contact with water. Reinforcing the basement was mechanically possible for all the studied alternatives to ordinary reinforcing steel. The amounts of GFRP and BFRP reinforcement were on the verge of the practically performable though and are not recommended in a project with so harsh limitations of crack widths. CFRP and stainless steel reinforcement are excellent alternatives.

It is hard to draw any consistent conclusions on workability implications. On the one hand, once the initial understanding of it is established, FRP could be a more ergonomic and workable reinforcement material. On the other hand it does not allow bending on site and must be protected more than steel before casting. Regarding the different steel alternatives, they have no major practical or mechanical differences to today's common practice.

The lower grade FRP materials are punished in the sustainability comparison presented in Section 4.2 due to the high amounts of reinforcement. Since these are deemed unsuited for the studied project however, the remaining alternatives were actually considered better or equal to steel with regard to sustainability. Economically, if the longer service life made possible by the longevity of more durable reinforcement is fully utilized, all alternatives are cheaper than ordinary reinforcing steel on a life-cycle basis; on the other hand, ordinary reinforcing steel is still the cheapest fully reinforced alternative when measured on the initial investment only. The increase of the whole cost of the building, including maintenance and overhauls over its service life is in the order of 5 to 10 % for all alternatives. This is not very much, even though the difference in the initial costs directly linked to the choice of reinforcement varies by several hundred percent compared to ordinary reinforcing steel. The reason for the difference in these results is simply that there are so many large costs that do not depend on the reinforcement approach and that therefore dilute the differences.

Alternative reinforcement methods could be combined with conventional where each method is used where best fit.

It is advisable and much used in practice to combine different types of FRP reinforcement in the same concrete member. Carbon FRP can be used where high stiffness is required and glass or aramid FRP where large elongations are more important. In bridge decks CFRP is often used in the primary direction, with GFRP in the secondary. Different steel grades could also be used, for example with stainless steel in reinforcement across casting joints or in the layer closest to a source of aggression such as the side of a slab facing a parking house environment. Naturally the, in this project less studied, coated steel grades could also be used like this to provide some extension of the service life of a member.

Structural engineering codes for alternative reinforcement materials and approaches ought to be included in future editions of national and international calculation codes.

This hypothesis is already true in some sense. Several countries have issued codes and recommendations for FRP, plain concrete members are included in Eurocode and stainless and covered steel can effectively be designed according to the common codes for reinforced concrete. As a step forward, it would be desirable if Eurocode would include a part on FRP reinforcement and also if its committee would develop the part on unreinforced masonry and concrete with more focus on form-activity. Such an alteration would be highly useful in restoration of historic structures.

If an increase in usage of alternative reinforcement materials increase, the prices could be largely decreased.

This claim has not been developed that much in the course of this project. What can be said though is that some FRP materials, especially basalt and carbon based are both promising and freely judged, bound to get cheaper in the future. Both have common raw materials, but need further technical development and market shares to decrease significantly in price. Stainless, galvanized and ordinary reinforcing steel have a strong and well established connection between raw prices of its constituting materials and the final product. A general price drop can therefore hardly be expected. Instead, if the economy and population continue to grow, higher demand for metals is rather to be expected. As a consequence reinforcement materials may rise in the favour of FRP reinforcement and plain concrete solutions.

The lower mechanical properties of some reinforcement materials can be overcome in design.

In Chapter 3, the mechanical behaviour of the chosen reinforcement approaches was evaluated. It was concluded that the lower grades of FRP reinforcement, i.e. primarily glass and basalt based products, have such low stiffness that they are practically unsuiTable for the most demanding projects, such as the one studied here in terms of cracking. In the ultimate limit state though the reinforcement amounts were fully reasonable which indicates that for a project with higher crack width acceptance, also these could be successfully used. It should be remembered in this context that crack widths must not be limited with regard to durability for FRP reinforcement; instead water-tightness and aesthetics are the limiting factors.

The challenges and lack of experience in design and execution can be overcome by planning and training.

From examples in North America and Japan it can be concluded that large structures have been successfully erected with no reported labour-related problems. Naturally some training would be needed, both in design and execution. Based on the experience gathered from the design part described in Chapter 3 though, it can be concluded that at least the design part is no big issue for a structural engineer to adapt to.

5.2 Questions

Can reinforcement bars be eliminated from exposed concrete structures altogether?

Yes it could in many applications. In the case study the required water-tightness makes it harder than in other applications, but even here different solutions with membranes could theoretically make it possible. Mechanically though there exist much more challenging structures than the studied. Floor slabs for example would either need very short spans or be designed as purely compressed arches or domes. Furthermore, according to the codes, concrete members whose structural resistance depends on the tensile capacity of concrete are not allowed. In mainly compressed members with just minor bending however, it would work (and is working) just fine.

Where reinforcement is unavoidable, are there reasonable alternatives to carbon steel with regards to stiffness, strength, price, availability, execution, sustainability etc.?

Yes, there are reasonable alternatives to ordinary reinforcing steel. If the higher durability is used to extend the service life of the structure in the case study, all alternative reinforcement approaches studied here are actually cheaper than ordinary reinforcing steel. If this presumption is omitted, the initial cost of the actual project increases by 0 and 6 % for the more durable reinforcement options.

Could there be methods of replacing ordinary steel reinforcement so far unimagined?

One idea put forward in this project is to work with a combination of Geopolymer concrete and FRP reinforcement. This type of concrete uses other binders than cement, such as pozzolans. This may lead to better mechanical properties, but primarily it results in environmental benefits and a drop in concrete's pH. One of the remaining problems of FRP's durability is its behaviour in alkaline environments. Using less alkaline concrete could be more effective than improving FRP's resistance.

Another idea arose when facing the issue of crack limitation for the FRP alternatives presented in Chapter 3. Non-prestressed reinforcement was simply insufficient as a means of holding the cracks together and yielded unreasonably dense reinforcement arrangements. As FRP reinforcement has high strength but low stiffness, prestressing gives very suitable utilization of the material. In that way concrete members could be so compressed that tensile stresses under relevant load would never appear. As a way of prestressing plain concrete alternatives with the same objective, there is a method used by the contractor Spännbalkskonsult (SBK 2014). This method uses gravity to prestress slabs by deforming the formwork in the opposite direction from the expected deformation in service. When the form is removed and the slab is set free, arching will appear and the shaped deformation will counteract the deformation under the loads, resulting in plane slabs with less tensile stresses and much less cracked upper surfaces over supports (SBK 2014).

Could reinforced concrete be built with practically unlimited service life?

Yes, by using the most reliable reinforcement materials such as austenitic stainless steel and perhaps also CFRP, the reinforcement will not affect the longevity of the concrete member negatively. Concrete in right conditions can survive thousands of years as can be seen in Pantheon in Rome for example. In buildings that are meant to last, the added cost of stainless instead of ordinary reinforcing steel is quickly returned.

5.3 Reflection

Through the process of the project I have learned a lot about the present strivings in research of alternative reinforcement approaches. It was very clear that there is a lack of clear unanimity in the case of some solutions; especially so in the case of FRP reinforcement, which some promote with much engagement, whereas others deem it unfit for use in concrete. To make matters worse, both sides have scientific backing in their arguments. It is natural with this background that actors in the construction industry are cautious and do not embrace a solution until it can be proven to be safe. This is however very hard to do if no one takes the leap out into the uncertainty. Therefore we should be extra grateful to the pioneers in Japan, Canada, USA and Germany who are providing us with real-life examples that can be monitored and evaluated. So far results from these real structures are very promising, 20-30 years into their service lives. This, in my mind should be enough for the Eurocode committee to seriously consider a first official European design code covering the use of this reinforcing material.

For stainless steel the only obstacle for widespread usage is its cost. Apart from the uncertainties discussed in the previous paragraph, this is true also for all grades of FRP except the glass fibre based. For buildings and civil engineering structures with limited service life, ordinary reinforcing steel may very well be the best alternative, since it is the cheapest in initial terms and lasts 50 years or so without extreme measures. However, as part in taking responsibility for sustainable development, many structures could be planned to last much longer than 50 or even 100 years. Perhaps they must be more versatile in their design and facilitate changes in their function, but with that in mind it is fully possible. Seen in this perspective, the increase of initial cost is marginal.

5.4 Recommendations

In this section some tangible recommendations to structural engineers who deal with reinforced concrete in harsh environments are presented. The ideas and conclusions presented here are based on thoughts that have appeared during the project. The recommendations highlight some applications where ordinary reinforcing steel could advantageously be replaced.

Reinforcement across joints

In connections between structural parts where continuous reinforcement is required, alternatives to ordinary reinforcing steel are highly recommended. Today concrete has very low permeability and protects its reinforcement effectively in continuity regions. No matter the quality of the concrete however, the reinforcement can get exposed at casting joints. There are connection solutions that can be used to transfer loads and at the same time seal the joint, but it has often been shown through history that water-tight solutions seldom manage to maintain this capacity over time. Additionally such connections can be very expensive.

Balcony connections

One field where alternatives to ordinary reinforcing steel are used much already today is in balcony connection devices. Actually it is a special case of the previous point, although so common that it deserves its own comment. Alongside load transfer and durability, heat insulation is a very important factor for balcony connections. Stainless steel has lower thermal conductivity than ordinary reinforcing steel, but is still dramatically worse than FRP, that could therefore be an interesting development of these solutions.

Low pH concrete with FRP

As previously discussed, FRP is sensitive to the alkalinity of normal concrete whereas ordinary reinforcing steel needs it to stay passive. Portland cement is the ingredient in concrete that generates the high alkalinity. To achieve the cementitious bond in the concrete, there are several cement replacers that could be used instead of cement, often with environmental and economic advantages. These cannot be used as much as would be desirable today due to the need of alkalinity in reinforced concrete. At the same time the most prominent durability issues of FRP reinforcement are connected to low resistance to the alkaline pore water in ordinary Portland cement based concrete. The recommendation then is to use a combination of FRP reinforcement with concretes with full or partial replacement of cement in favour of pozzolans.

FRP where no SLS limitations apply

Where cracking and deflections are of little importance to the serviceability of a structure, GFRP and BFRP reinforcements are economic options with high durability deemed from available field results. Elsewhere higher grade alternatives such as CFRP and stainless steel must be used, with substantial increases in cost as a consequence.

Long service lives

More structures ought to be designed for longer design service lives as a means of improving the use of scarce resources. As it is today, demolished concrete structures are very low grade waste, whereas producing the material is highly resource demanding. Apart from these reasons, there are also cultural reasons to maintain buildings from different time periods for the future. The world without its historic buildings would be a substantially duller place. If the economic models governing construction projects shift towards including the total value and cost of owning a building, instead of comparing the initial investment costs; longer service life may become higher in demand. To achieve a long service life, the safest option would be plain or austenitic steel reinforced concrete. FRP reinforcement could be another option, but still has to prove its long-term durability in practice.

Compressed structural members

In mainly compressed members that could be designed without reinforcement, doing so should be seriously considered. Apart from the potential of economic savings, the more homogenous and isotropic plain concrete will not suffer constraints and uneven expansion between concrete and reinforcement. Constraints imposed at the boundary of the concrete member must however be considered and avoided. Without any reinforcement at all, durability is also likely to improve.

Reparations

If for example an edge beam on a bridge must be replaced due to corrosion before the end-of-life of the structure as a whole has been reached, it could be a good idea to use another material than ordinary reinforcing steel in the reconstruction of it. One such alternative could be FRP reinforcement. Also if a concrete member for some reason must be strengthened during its service life, FRP placed upon or near the surface is a very interesting option which is becoming more and more accepted among professionals in Sweden. The combination of high strength and no need for hefty concrete covers make FRP sheets the perfect strengthening material for some conditions.

Political instruments for sustainable construction

As discussed in Chapter 1, there is a lack of incitements for the different actors in construction projects to prioritize long service lives. Economic models do not reward future savings very much due to discounting models and short-sightedness. There are however clear environmental benefits with getting the maximum possible service out of the natural resources that are used for construction. If this is agreed upon but the free market does not seem to solve the problem, perhaps it has to be manipulated by some kind of subvention. One way could be to offer compensation to the construction industry if the design service life is increased. Another could be that certain demands are made mandatory or that government states demand for the buildings they procure. A more voluntary way would be that the service life is more heavily weighted in the several "sustainable building" labels that already exist.

5.5 Further research

Here the fields where improvements or better knowledge is needed are gathered.

Develop more fire resistant matrices for FRP

The most limiting feature of today's FRP materials is their low heat resistance. To reach further in the usage of the material, this is clearly an issue which needs more research.

Fire protection of FRP

Until more heat resistant matrices are developed, the issue must be met with today's tools. Different ways of protecting the sensitive matrix from heat therefore constitutes an interesting field of study. In that way FRP reinforcement could be further used in buildings and not only in civil engineering applications as is the case today. In this project three methods have been put forward: Increasing the concrete cover thickness, adding fire paint on the surface and finally protecting the anchorage zones and allow the rest of the matrix to soften under heat.

Long-term durability of FRP in concrete

FRP is still a rather new material for use in concrete; today though a reasonably wide range of structures have been constructed. To obtain more certainty and thus confidence for engineers to use the material in reinforced concrete, extensive monitoring and testing of the existing buildings ought to be performed.

Wider range of case studies

As a complement to the case study in this project, studies of other structures would be of interest to widen the basis for conclusions. The case study could also be taken forward by studying for instance separate waterproofing layers on the outside of the structure, which could then be allowed to crack freely.

Design code preparation for FRP reinforced concrete

If FRP reinforced concrete is to be included in Eurocode, a vast research work lies in preparing the design rules and recommendations. Surely, much of the material can be gathered from other countries' codes, work prepared in the fib model code and also from regular steel reinforcement design, but the work load is nevertheless extensive.

Development of FRC's load-bearing capacity

If fibre reinforced concrete could be developed to the extent that concrete could be reinforced for most load situations already in the mix design, that would be a huge break-through. Today the ultimate capacity is negligibly increased by fibre reinforcement, something that research however may be able to improve.

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