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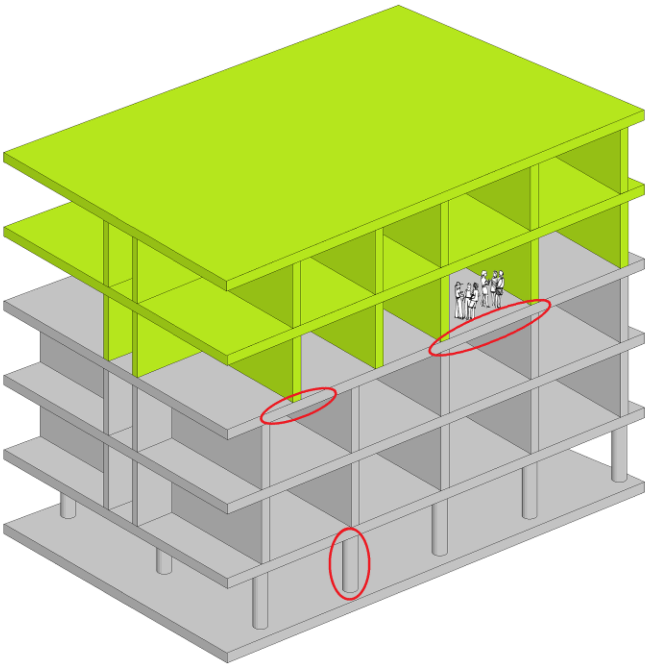

Strengthening of buildings for storey extension

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

BJÖRN JOHANSSON
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Department of Civil and Environmental Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover:
Illustration of members that may be critical in storey extension projects.

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ABSTRACT

Storey extensions are an increasingly popular way to densify cities. One problem may however be that designers sometimes lack experience and knowledge concerning the specific issues that arise during storey extension projects and an accompanying strengthening of the superstructure. The aim of this project was to ease the work for the designer by highlighting critical questions and possible solutions. The information was mainly gathered through interviews with persons actively involved in storey extension projects. The interviews gave, among other things, much input concerning experiences about how strengthening methods can be performed for varying boundary conditions. The knowledge collected from the interviews was thereafter complemented with fundamental information about strengthening methods through literature studies. Some focus was also put on older structures and the common considerations that follow building projects where existing buildings and their users are affected. Thereafter, the different strengthening methods were compared and some of them were also further evaluated through supplementary calculations. The results of the project show that there are many aspects to consider in storey extension projects, but also that many solutions are available. It is important to properly assess the building early to detect any critical members or unused capacities etc. It is also of importance to select the best suited strengthening method for the specific situation. Sometimes, the apparent solution may not be the most appropriate.

Key words: storey extension, strengthening of concrete structures, structural systems, early design phase.

Förstärkning av byggnader för våningspåbyggnad
Examensarbete inom Structural Engineering and Building Technology
BJÖRN JOHANSSON
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SAMMANFATTNING

Våningspåbyggnad blir allt vanligare i storstäder där en förtätning ofta eftersträvas, som till exempel i Göteborg. Ett problem kan dock vara att konstruktörer ibland saknar erfarenheter och kunskap om de speciella frågeställningar som kan uppstå vid påbyggnadsprojekt med eventuella stomförstärkningar. Detta projekt syftade till att underlätta konstruktörens arbete genom att belysa viktiga problem och möjliga lösningar. Informationen insamlades främst genom intervjuer med yrkesaktiva som varit inblandade i påbyggnadsprojekt. Intervjuerna gav bland annat många bra erfarenheter om hur förstärkningar etc. kan utföras vid olika förutsättningar. Kunskapen från intervjuerna kompletterades därefter via litteraturstudier med mer grundläggande information om olika förstärkningsmetoder. Viss fokus lades även på olika äldre stomsystem samt de särskilda frågeställningar som medföljer ett byggnadsprojekt där befintliga byggnader och användare berörs. Därefter jämfördes de olika förstärkningsmetoderna och några utvärderades även med kompletterande beräkningar. Projektets resultat visar att det finns många aspekter som måste beaktas i påbyggnadsprojekt, men även att det finns många bra lösningar. Det är viktigt att inventera byggnaden tidigt för att lokalisera kritiska element och outnyttjade kapaciteter etc. Därefter är det viktigt att välja rätt förstärkningsmetod till rätt situation. Möjligheten finns att någon annan förstärkningsmetod lämpar sig bättre i den specifika situationen än den för konstruktören mest uppenbara.

Nyckelord: våningspåbyggnad, förstärkning av betongkonstruktioner, tidig dimensionering.

Contents

Preface

This report is the product of a Master's thesis project conducted at Chalmers University of Technology in the spring semester of 2013. It was carried out to conclude our Master's degree in the field of Civil Engineering.

The work done in this project has been evenly divided between us and we feel that we have enjoyed the time spent on it. We have been working with close cooperation and have both contributed to all parts of the project. If some minor distinction should be made, Marcus Thyman has spent more time on literature studies while Björn Johansson has focused a bit more on calculations. However, all steps in the process have been taken based on discussions.

We could not have done this work without help and would like to thank everyone who has been involved. We want to direct special appreciation to our supervisor and mentor Lukas Jacobsson at VBK for his counselling and encouragement. We also like to thank our examiner, professor and supervisor at Chalmers, Björn Engström, for his assistance and useful advices on how to structure our work and proceed with our thesis.

The consultant office VBK has provided us with workspaces, counsel and breakfast every morning along with good company and interesting lunch discussions. We want to thank them for their support and patronage.

Emelie Eneland and Lina Mållberg, our opponents, who have shared workspace with us during the entire semester, also deserve many thanks. They have helped us develop ideas through feedback and given us many moments to cherish. This project would not have been the same without them.

We also want to thank all the other persons who have been involved during the development of this thesis, persons who have been available for interviews, helped us with queries and guided us towards the end result.

Last but not least, we want to thank our families and friends, for never-ending support and encouragement during all our years of education.

Göteborg, June 2013

Björn Johansson and Marcus Thyman

1 Introduction

The project presented in this report has treated different methods of strengthening existing buildings for storey extension. It is meant to help the designer in the early stages of a storey extension project.

1.1 Background

Göteborg is presently the second largest city of Sweden with more than 500 000 inhabitants in 2011, Statistiska Centralbyrån (A) (2012). However, the population density is quite low in Göteborg compared to other larger Swedish cities such as Stockholm and Malmö, Statistiska Centralbyrån (B) (2012). There might be several reasons for this, but for certain is that the city of Göteborg has great possibilities to become a more densely inhabited city.

This potential suits well with the intention of the City Council of Göteborg, who wishes to densify the central parts of the city, Stadsbyggnadskontoret (2009). Densification of the central parts enables the use of already established infrastructure, recreational facilities and similar. In this way existing neighbourhoods may also progress and evolve in new directions. Developing already attractive areas can therefore motivate a higher construction cost than a building on a less desired site. Densification can be performed in several manners, e.g. erecting new buildings on unused land, filling empty areas between existing buildings, replacing existing buildings with higher or denser ones, changing building functions or internal apartment arrangements to enable more people to live in already built structures, or vertically extend already existing buildings. The latter approach is the one that has treated in this project.

There are many issues that may prove problematic during the different stages of storey extension projects. When new floors are added, the building will be subjected to higher loads both vertically and horizontally. These must in some way safely be transferred downwards through the structure to the foundation. In many cases there is an excess capacity of the existing building and its foundation, but this can vary a lot depending on where, when and how the structure was built. If the capacity is too low, it might sometimes be necessary to strengthen the existing structure or its foundation.

Strengthening of existing structures has been performed many times before, but the experiences are neither very well documented nor treated thoroughly during the education in civil engineering. Therefore, it is relevant to research the field to create proper design handbooks or guidelines to aid the designer. When treating extensions and strengthening of existing buildings, each project may seem unique and case specific, but there are common aspects and considerations that make it possible to draw conclusions on when different approaches most often are suitable.

1.2 Purpose and objective

The purpose of this project was to develop strategies for how the designer should handle storey extension projects. The main focus was on how to identify a lack of capacity in the existing structural system and how to perform the needed strengthening in a good way.

To fulfil this purpose, guidelines that can be used in design of building extensions were created. These guidelines are meant to be used as an aid when determining if and how to strengthen an existing structure. The guidelines should take different situations and boundary conditions into consideration. The user should be enlightened about important steps in the design process and alerted on critical issues.

1.3 Scope and limitations

The main focus of this project was on strengthening of existing buildings in Göteborg. This choice was based upon the fact that the project was carried out with support from VBK, a structural design company located in Göteborg. The main part of the targeted audience is also active in the city. Conditions such as geology and building standards etc. are therefore influenced by the situations in Göteborg and Sweden. However, the results may also be applicable to buildings in other cities as long as the user is aware of the differences.

Even if the methods discussed here are meant to be applicable mainly to storey extension projects, it should also be possible to apply the results of this project to other types of situations where strengthening is needed. It should however be noted that the choice to focus on strengthening for storey extension may limit the number of investigated strengthening methods.

Since the soil conditions may have a large effect on the capacity of buildings, especially in Göteborg, evaluations and possible foundation improvements were also treated to some extent. Strengthening of the structure above ground was however treated more thoroughly.

The choice of structure for the extension itself was also treated, since it largely affects the need for strengthening of the existing structure. Issues other than the load-bearing system, such as accessibility and need of fireproofing etc., were handled in a simplified manner.

Furthermore, the type of buildings investigated was limited to concrete structures, mostly since this material is very common in Göteborg and Sweden. However, other building materials were treated when it comes to strengthening and the extension itself.

1.4 Method

The purpose of this project could have been reached in several ways. One possible way would have been to perform a case study where an existing structure is vertically extended. In this way, different strengthening methods could have been evaluated for the specific case. However, since the subject is very extensive and a vast variation of existing structures can come in question for storey extensions, another approach was chosen. The chosen approach is very dependent on information from previously executed projects, since these experiences are valuable for future projects.

The chosen approach consisted of two parts. The first part aimed to investigate methods for storey extension and strengthening. Apart from literature studies, where strengthening methods were investigated, emphasis was put on interviews. To cover a wider range of possible situations, persons involved in twelve different projects were interviewed. Some of the interviews were carried out during meetings, while others were conducted via telephone or email correspondence. Among the studied projects were examples of extensions on top of residential buildings, hotels, office buildings and garages. This approach was chosen to identify differences in the issues that can come in question for the various situations.

The main focus in the interviews was on the key aspects that the designer and/or site manager had to consider in the specific project, i.e. the main differences between the project at hand and a more regular design project. The questions asked in these interviews are presented in Appendix A. Emphasis was put on how the designer solved the problem with the increased load on the existing structure, but other important considerations such as new elevators and how to handle the current residents and tenants were also discussed.

The next step in the project was to evaluate and organise the information about strengthening and the studied projects. To supplement the information at hand, some experts in the fields of geotechnical engineering and fibre reinforced polymers were also contacted and interviewed. Thereafter, critical issues were connected to specific conditions and potential solutions. In other words, it was stated under which conditions a lack of capacity in a structural member often occurs and how this problem can be solved. These solutions were then investigated further and compared to each other with the ambition to find advantages and disadvantages. This comparison included calculations in which some of the most important structural members were strengthened according to different methods. It also included a discussion where the suitability of the methods in different situations was considered.

Based upon the gathered information, guidelines were created which should aid the designer to find a possible design. These guidelines should primarily pinpoint important steps and issues that can come in question during the process.

1.5 Thesis outline

The main result of this project, the guidelines, can be found in Chapter 8. This part of the thesis is therefore the one that will be of most use for the designer. However, the results presented here are based upon the information provided in the rest of the thesis. Chapter 8 is structured to follow the design process, from choice of building to design of structural members that should be strengthened. The strengthening methods are put into a context and their applicability in storey extension projects is discussed.

Chapter 2 contains background information about the conditions in the city, mostly concerning geology and common existing structures that may come in question for storey extension projects. Chapter 3 is another important part and contains the information that has been gathered from the interviews, i.e. collected experiences from executed projects. Considerations about the extension itself are treated in Chapter 4.

A big part of the report is located in Chapters 5, 6 and 7, where possible strengthening methods are explained and discussed. Chapter 6 contains the main facts about the methods and is organised after type of structural member so that the designer easily can find methods that are relevant. To get general information about how to use different materials to strengthen the members, the designer is instead referred to Chapter 5. This division is used to minimise the number of repetitions. In Chapter 7 some of the treated methods are evaluated further through calculations.

2 Conditions for storey extensions

In this chapter information is given about the conditions for storey extensions in Göteborg concerning the intent of the city council, the geological conditions and the typical existing structures that can come in question in a storey extension project.

2.1 Densification of the city by storey extension

Development of cities occurs continuously and whether this progress is in the right or wrong direction may differ from case to case and perspective of opinions. For a city to be able to advance and expand, it has to account for its current surroundings. The City Council of Göteborg has a desire to further utilise already existing infrastructure and public transportation systems, Fritiofsson et al. (2008). The fulfilment of this ambition can be achieved in various ways, but some sort of city densification is the common approach.

Göteborg also wishes to have an integrated society within local regions where people with different backgrounds and in different stages of life are living and working. This can be achieved through various types of leisure activities, but a range of various available apartments and offices etc. may also create a more diverse community. New and more modern apartments will for example attract different types of residents than older ones. The difference in price range may of course be a contributing factor to this. However, the layout and size of the apartments etc. may also be used as another tool to further attract a targeted tenant group.

Repairing and upgrading existing structures is in many cases less expensive than erecting new structures, Täljsten et al. (2011). Improving existing structures also consumes fewer resources than tearing down and rebuilding, making it more environmentally friendly. A more rapid construction process can be expected as well, while the building simultaneously remains usable.

2.1.1 Building in urban environment on top of an existing building

There are several benefits when building a new structure on top of an older, such as already disposable services and no need to build new connecting roads. Construction work in an urban environment can however also have its drawbacks. It is important to adapt the site to the current surrounding and its traffic flow, while also considering the people that are living and working within the area. Difficulties to find storage space for the building material close to the site may also put higher demands on logistics and planning of the construction process.

The size of the structural members and whether or not to use some kind of modules are to be decided from case to case. However, the vertical transport of structural members through a weather protection onto the existing building may also prove problematic and needs to be considered. Using for example large wall elements may give a quite fast result, but there can also be advantages to build with smaller parts which can be transported mainly through elevators and stairwells.

There can also be regulations for noise and vibration in certain regions that limit the use of specific equipments and working methods entirely or at certain hours. One way to reduce the number of disturbed persons might be to use the top floor of the existing building as offices for the contractor, Samuelsson, E. (2013-01-24). This enables the workers to be closer to the site, but the storey also acts as a barrier towards other parts of the building. To use the existing building as location for the office might however not always be feasible, since this require evacuation of an entire floor for quite some time.

2.1.2 Increased need for parking, storages etc.

Another issue that needs to be handled during densification is the increased need for parking spaces and facilities such as laundry rooms, storage rooms and waste disposal. In many cases the latter might be solved by implementing the facilities into the existing structure or by placing these in a detached shed. Parking spaces however require a large area and this issue may not be solved as easily. In some cases a new parking garage might even be necessary. However, in the central parts of Göteborg, the norm for available parking spaces per household has decreased quite drastically during the last decades. In some districts a decrease from two cars per household to only 0.5 or 0.6 might be possible, Östling (2013-02-06). The number of parking spaces per household in an area is dependent on its location and distance from the central areas, so such a reduction is not applicable everywhere.

2.2 Geological conditions in Göteborg

Göteborg is located by the mouth of Göta River and has therefore quite complicated geological properties with regard to structural engineering. The most common soil profile in Göteborg is topsoil above clay, followed by friction material and finally bedrock, Alén (2013-02-25). Some areas might be dominated with an almost 100 m deep layer of clay, while bedrock is visible directly at the surface in other areas. The intermediate situations may include different thicknesses of clay where the depth to bedrock may vary considerable under the very same building.

Constructing heavier buildings on this kind of soil may result in unwanted effects, such as uneven settlements, which ultimately may end in failure. There are however ways to manage and overcome this undesirable effect. In Göteborg piling is the most common solution. Early piling was limited to the length of available tree trunks, which also limited the possible weight and height of the buildings, Alén (2013-02-25). This is one of the reasons why Göteborg is a rather sparsely populated city. However, with improved knowledge of piling and soil improvement, an increasing amount of heavier and taller buildings have been erected during the last decades. Different kinds of foundations are treated further in Section 2.3.5.

The design process is not as straightforward when it comes to storey extensions as for new buildings, since a load increase must safely be transferred to the bedrock without causing damages on the original structure or its foundation. However, there might be cases where existing buildings have unutilised capacity, which enables the structure to carry additional loading without experiencing damages. Possible methods to strengthen the foundation beneath a structure are presented and discussed in Section 6.5.

2.3 Typical existing structures in Sweden and Göteborg

Since the scope of this project limits the types of investigated structures to those that are made of concrete, other kinds of structures were not treated at all. This excludes steel and timber structures as well as the many old buildings that were built with load-bearing masonry walls.

The choice to only consider concrete structures still allows a wide range of different structures to be studied, since the material has been frequently used during the last century. The ability to change the properties of concrete by altering the components in the mixture together with the ability to cast very free shapes has made the material popular. The desire to be able to design buildings for different kinds of activities and to shorten the erection time has in combination with increased knowledge about the material resulted in a variety of structural systems.

Even though the history of concrete dates back over two thousand years, the first Swedish building with a concrete structure was built in the 1910s, Carlsson (1965). Concrete slabs became increasingly popular during the ‘20s, while the use of load-bearing concrete walls developed during the ‘40s. However, the big breakthrough for the structural material came in the early ‘50s, when it quickly took over the market from structural masonry, Björk et al. (2003). The improved construction methods contributed to a reduced construction cost for the superstructure. In 1930 the superstructure represented 72 % of the total expenses in a building project, while the corresponding figure in 1960 had decreased to 38 %, Carlsson (1965).

2.3.1 Residential buildings

There are many ways to design a residential building with a load-carrying structure made of concrete and the methods have varied and developed throughout the years. In this section common existing residential buildings and their basic characteristics are described. The information is not primarily based on the situation in Göteborg due to the lack of statistics about the buildings in the city. Instead, the examples represent common residential buildings in Sweden.

One way to categorise apartment buildings is according to their primary shape. Figure 2.1 shows simplified sketches of the three basic appearances that symbolise the most common residential buildings in Sweden, which are long and narrow lower buildings, square-shaped tower blocks and long and narrow taller buildings.

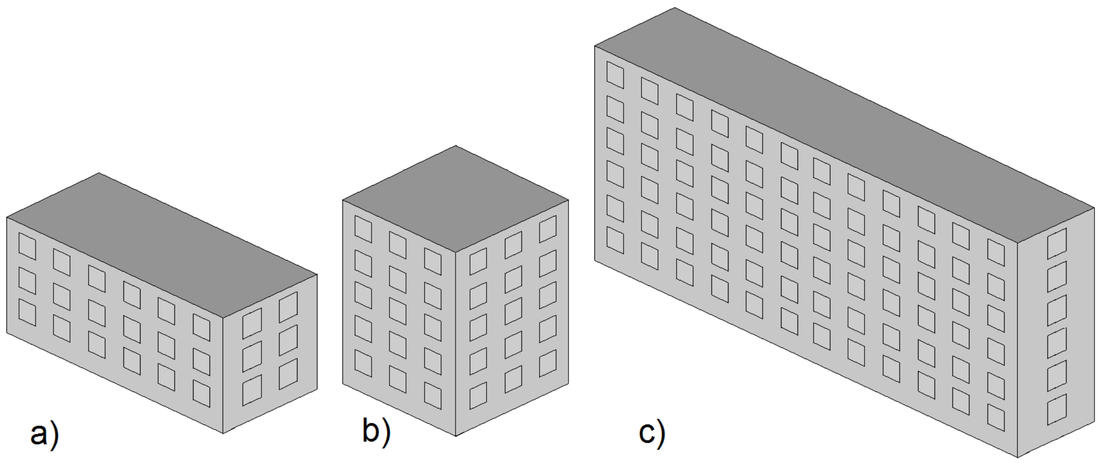

Figure 2.1 Different common shapes of existing residential building, a) long and narrow lower building, b) square-shaped tower block, c) long and narrow taller building.

2.3.1.1 Long and narrow lower buildings

The most common residential building in Sweden has a rectangular shape where the length is considerably longer than the depth, Björk et al. (2003). Many of these are about three to four storeys high since buildings of this height for a long time were permitted to be built without elevators. In southern Sweden four storeys without elevators were allowed to be built until 1960, while three storeys could be built in this way until 1977. Thereafter, residential buildings with more than two storeys needed elevators. The stairwells (and possible elevators) most often only serve the adjacent apartments without the use of corridors. This can induce problems in storey extension projects, since the extension requires either many elevators or the use of access balconies.

In the end of the ‘40s the use of regular masonry bricks in the load-bearing walls started to be replaced by use of blocks made of concrete, Björk et al. (2003). However, the old approach to use load-carrying façades together with load-bearing spine walls, the central wall illustrated in Figure 2.2a, still remained. Lightweight concrete blocks were sometimes placed in the façade, but the interior walls were often made up by regular concrete blocks due to sound demands. The slabs were often made of in-situ cast concrete.

Load-bearing walls of in-situ cast concrete became increasingly popular during the end of the ‘50s and the new method also brought a big change in the load-carrying structure, Björk et al. (2003). Load-carrying façades and spine walls were replaced by the cross-wall system with load-bearing transversal interior walls and gables. This cross-wall system is illustrated in Figure 2.2b. One big benefit in storey extension projects with the cross-wall system is that the transversal walls often have an excess capacity, since they have been designed with regard to sound demands. These buildings are also very stable in the transverse direction.

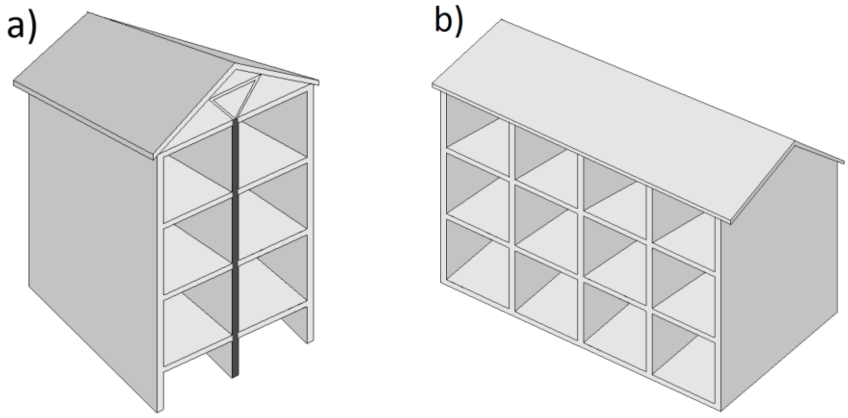

Figure 2.2 Different structural systems, a) load-bearing spine wall (dark) and façades, b) the cross-wall system that became popular during the '50s.

To be able to utilise the expensive elevators better, residential buildings with access balconies became quite popular during the '60s. The elevators were placed apart from the house itself and access to many apartments was gained without the need of interior corridors, see Figure 2.3. As before, the cross-wall system was most often used. An advantage with this type of building in storey extension projects is that it is easier to use a similar layout in the extension without needing to install many elevators.

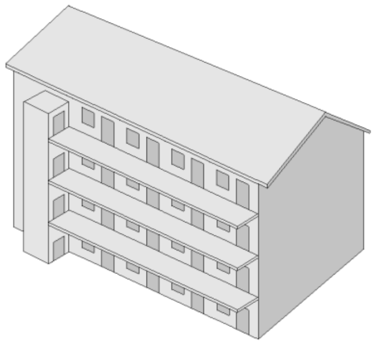

Figure 2.3 Building with access balconies.

The '70s brought a rapid increase for prefabricated concrete elements in the load-carrying structure, since the construction time then could be reduced. Slabs were often prestressed and the cross-wall system was often used. According to Stenberg (2012) the prefabricated residential buildings from the '70s are often very robust. One advantage for storey extensions with prefabricated buildings might be that many elements have the same size, which should simplify a possible use of prefabricated elements in the extension.

As mentioned earlier all residential buildings with more than two storeys that have been built after 1977 have elevators. The mass-production of large housing complexes subsided at this time and the buildings from the '80s and onward are more adapted for sites near the city centre rather than the suburbs. These buildings are often more unique, even if the knowledge from the ‘70s, e.g. the cross-wall system and prefabrication, in many cases was used.

2.3.1.2 Square-shaped tower blocks

The second category of residential buildings has a more square-shaped layout with stairwells located in the centre of the building. According to Willén (2013-02-06) this attribute can be advantageous for storey extensions, since only one elevator needs to be installed.

In a similar way as for the long and narrow buildings at that time, lightweight and ordinary concrete blocks were during the ‘40s used in the square-shaped buildings, Björk et al. (2003). Both the façades and the apartment-dividing walls are load-bearing and mainly arranged to meet the demands concerning the apartment layout, which means that their placing can be irregular.

Square-shaped buildings from the ‘50s and ‘60s are often higher, e.g. about eight to ten storeys, Björk et al. (2003). These buildings often have in-situ cast exterior load-bearing walls, sometimes with an outer insulating layer of lightweight concrete blocks. The walls on the highest storeys might however consist of only the lightweight blocks, since the load is lower in this part. This property can be unfavourable in a storey extension project.

2.3.1.3 Long and narrow taller buildings

Significantly taller, and often longer, versions of the long and narrow buildings also exist. An important difference that comes from the height is that they always have elevators. These tall and long buildings gained popularity during the ‘60s and were in the beginning often cast in-situ and built according to the cross-wall system. Non load-bearing façades could be made either by lightweight concrete blocks or prefabricated sandwich elements. In the end of the ‘60s and during the ‘70s, prefabricated elements were often used.

2.3.2 Office buildings

The functionality demands on office buildings have throughout the years resulted in a wide range of structural systems, Carlsson (1965). Some general considerations can however be noted, when it comes to the specifics about office buildings. One of the main differences when compared to residential buildings is that office buildings most often are designed to be adaptable to future changes. Since the activities in the building can alter many times during the service life of the building, structural systems that prevent changes in the layout are avoided. This desire has through the years often led to the use of structural systems with columns instead of load-bearing walls.

In the beginning of the 20th century, the office buildings were often built in steel, but during and after the Second World War, the steel price rose drastically, Carlsson (1965). Therefore, almost no structural steel was used in the ‘40s and ‘50s. The high steel price instead promoted the use of concrete. The concrete was in-situ cast in the beginning, but during the years, prefabricated elements were used more and more often. The structural systems often consist of columns with flat slabs or various combinations of columns and beams. An elevator shaft is also often used for stabilisation. In the end of the ‘60s, more and more structural steel was used again, according to Carlsson (1965). Steel columns, steel beams and concrete floors are today very common in office buildings, Skelander (2013-12-12).

When it comes to the general layout in office buildings, two main variants can be seen as the most important. The first one is referred to as *the European way* by Carlsson (1965). The main idea with this method is to use internal corridors that let the employees access their separated offices. When compared to dwellings, this kind of office building only permits windows in one direction, which is sufficient for offices. Figure 2.4 shows three different basic layout alternatives that have been used for office buildings with internal corridors.

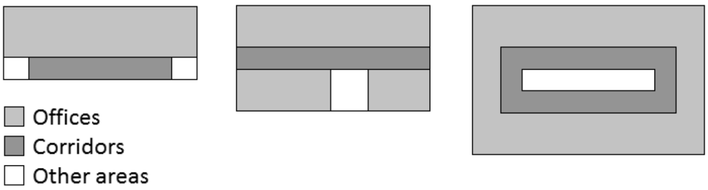

Figure 2.4 Different layouts of office buildings, after Carlsson (1965).

The second common layout contains big open plans in which the employees sit together. This kind of layout is by Carlsson (1965) referred to as *the American way*. It can easily be understood that this kind of layout demands another structural system than the alternative with separated offices. In many cases the slabs span the whole width of the building.

When considering the suitability for storey extension projects, office buildings show several important differences from residential buildings. The most prominent might be the lack of load-bearing walls designed with regard to sound restrictions. Since columns often have less excess capacity than load-bearing walls, see Section 3.4, this should mean that office buildings more often utilise a higher rate of their load-bearing capacity. Therefore, strengthening should be required more often for office buildings. Another disadvantage with structural systems that contain columns is the lack of extra stabilisation that comes with load-bearing walls. Even if the existing structure is braced by elevator shafts or steel trusses, it is not likely that it has any excess capacity.

However, one advantage with office buildings compared to housings is the internal layout. Regardless of whether internal corridors or open halls are used, it should be fairly easy to avoid many elevators or access balconies for the extension. Another advantage may be that the entire building can be rented by a few or even one single company. If these move to new offices, an opportunity to renovate the building can occur.

2.3.3 Hotel buildings

Hotel buildings can in many ways be seen as something in-between residential and office buildings. The storeys that contain the hotel rooms often have transversal load-bearing walls that are designed with regard to sound demands. This can, in the same way as for residential buildings, give a robust structure that has an excess capacity. The main difference from residential buildings is however the big open spaces at the entrance floor and possible restaurants etc. Both Scandic Opalen and Gothia Central Tower are examples of hotels where the structural system on the entrance floor was found to be critical during the storey extension, see Section 3.1.

One advantage with hotels, in the same way as for office buildings, is that the rooms often are one-sided with access via internal corridors. This property may reduce the need of new elevators.

2.3.4 Parking garages

Unlike for other types of buildings specific statistics about the parking garages in Göteborg are more available. According to Nilsson (1991) there were 92 parking garages with room for more than 30 cars in central Göteborg in 1990. Out of these, 80 garages were made of cast in-situ concrete, two consisted of prefabricated concrete and four were built with a combination of cast in-situ and prefabricated concrete. 43 of the garages were categorised as free standing by Nilsson, while two other garages were placed on the roof of existing buildings and two were built-together with adjacent buildings. The rest of the garages were placed beneath existing structures. Even if this information is relatively old, it gives a good indication about most of the existing parking garages in Göteborg.

When it comes to the structural system of garages, the most characteristic feature is the need of big open spaces. According to Jones and Stål (2007) most of the parking garages have structural systems that consist either of slabs and beams on columns or flat slabs directly on columns. The systems with beams can be designed either with the beams in the longitudinal or transversal direction. A combination can be used as well so that a system of crossing beams is created. A construction method that utilises interaction between prefabricated prestressed beams and in-situ cast slabs has also been used in the city during the last years.

Since parking garages in Sweden and Göteborg are subjected to relatively severe exposure conditions, mostly due to the wet climate and de-icing salts that the cars bring into the garage, many of the structures show signs of damage. One example is the parking garage at Tunnlandsgatan in Göteborg, which was vertically extended and renamed to Kaverösporten, see Section 3.1.9. The garage was built in 1965 and consists of columns, beams and slabs of in-situ cast concrete, Nilsson (1991). Before the extension was made, the garage showed several signs of damage. According to

Nilsson some of these were visible reinforcement bars in the façades, local damages at the top surface of the slabs (beneath tires), signs of reinforcement corrosion through thin concrete covers in columns and walls and water puddles on the floor close to columns. Even if these damages were found in one specific garage, they are examples of damages that often need to be handled when parking garages are extended.

The fact that parking garages often are in bad shape makes it even more important to inspect the existing structure carefully before the extension is decided and designed. Possible decay can result in a load-bearing capacity that seriously falls below the originally designed value. The extensive need of renovation in many parking garages may however imply that strengthening due to storey extension can be considered. If for example the columns need to be strengthened to take the already existing load, it can be motivated and cost effective to strengthen them a bit extra at the same time.

The simplicity of the structural system together with the low use of insulation and installations etc. in many self-standing parking garages are things that might facilitate a storey extension. When compared to residential buildings, it can be fairly easy to place installations etc. through the lower structure without major disturbances. It can even be reasonably simple to drill holes in the decks and place new columns down through the garage.

2.3.5 Different kinds of foundations

As mentioned in Section 2.2 the ground in Göteborg is dominated by bedrock and clay. This clay has complicated the construction process over the years and continues to do so even today. In this section it is described how the problem with the soil has been solved throughout the years and how this affects possible storey extensions. Methods to strengthen the foundations are instead found in Section 6.5.

2.3.5.1 Foundations on solid rock

In the beginning of the 20th century foundations on solid rock were simply realised by casting a concrete wall straight down to the rock, Björk et al. (2003). However, during the mid '50s a new method started to become popular. The bedrock was levelled into terraces and the blasted bits of rock were spread out to even out the surface. A reinforced slab was then cast on top of it, where thicker dimension were commonly used directly beneath the load-bearing walls. On the other hand, plinths have also been used in many cases throughout the years.

If the building is founded directly on bedrock, there are normally no problems with the foundation when increasing the load, Alén (2013-02-25). An inclined bedrock surface may require some extra attention, but Alén claimed that the increased frictional resistance that can be derived from the additional load most often is enough to avoid strengthening. Consequently, buildings founded on bedrock are very suitable for storey extensions.

2.3.5.2 Foundations on firm, semi-firm and soft soil

Until 1960 foundations on firm to semi-firm soil were made with concrete walls standing on narrow footings that were localised beneath the wall itself, Björk et al. (2003). As with the foundations on solid rock, the method could be replaced by a whole bottom slab with thickenings below the load-bearing walls.

In the beginning of the 20th century, foundations on fairly soft soil were designed as fascine works, similar to a raft made out of timber. At the time when the concrete buildings became increasingly popular, the method had however been replaced with the same type of foundation as presented in the previous sections, namely the cast slab with thickenings beneath load-carrying walls. The thickness of these slabs might however be greater than those on more solid ground. For deeper layers of soft frictional soil, end-bearing piles or friction piles have been used. As the names suggest, the end-bearing piles rest on more solid soil or bedrock while the forces from the friction piles are transferred between pile shaft and soil through friction.

2.3.5.3 Foundations on very soft soil (clay)

Clay is common in the Göteborg region and has over the years often required piling. As with frictional soil, end-bearing piles can be used if the distance to bedrock or firm soil is not too far. Otherwise, cohesion piles can be used where the forces are transferred through cohesion between the pile and the soil. Before 1930, timber piles were the only choice when buildings on clay were constructed. The use of concrete piles developed during the '30s, but the real popularity for the method came after the Second World War, Alén (2013-02-25). Timber piles are however still used in some situations and it is not uncommon with piles that combine timber and concrete. In these cases, the lower part (the part that is constantly beneath the ground water level) consists of a timber trunk, while the upper part is made of concrete. The surrounding groundwater helps to preserve the timber, while the overlying concrete is located within the transition area that can be quite severe for timber.

Driven concrete piles are very common in Sweden and Göteborg. In fact, the prefabricated pile elements that are spliced together were invented in western Sweden, Alén (2013-02-25). The elements are normally 13 m high, but due to the splicing, piling in Göteborg has reached about 80-90 m down into the soil.

According to Alén (2013-02-25) the design codes for the piles have changed throughout the years so that piles from e.g. the '50s today can take more load than they were originally designed for. This can be advantageous in a storey extension project. On the other hand, the geotechnical capacity is not treated in the same way. This means that there can be situations where the piles themselves can take the increased load, but at the same time, the ability to transfer the load to the soil is too low.

3 Experiences from previously executed projects

To be able to make use of existing knowledge concerning storey extensions, several reference projects have been studied. Information has been collected through research and interviews with persons involved in storey extension projects. A more thorough overview of the gathered information is available in Appendix B, while key aspects are described in this chapter. The main questions that were asked during the interviews can be found in Appendix A.

3.1 The studied projects

The majority of the studied projects are situated in the Göteborg region, but two of them are located in Stockholm. It was desired to find different types of projects that represent various types of structures. In this way, specific critical issues could be identified for each type.

3.1.1 Hotel – Gothia Central Tower

Gothia Central Tower, located in central Göteborg, was built in 1984 and initially reached 62 m above the ground with its 18 storeys. It consists mainly of in-situ cast concrete with a big core in the middle of the tower for stability, Samuelsson, E. (2013-01-24). Load-bearing walls between hotel rooms go downwards through the building except at the lower entrance and conference floors, where columns are used instead. This is illustrated in Figure 3.1. The building is mainly founded on footings on top of the bedrock, but short end-bearing piles have been used in some places.

Six new storeys are being added at the time of writing, giving the building a new height of 83 m. Even more storeys were sought, but the columns on the lower floors had too low capacity, Samuelsson, E. (2013-01-24). Unlike the original building, the structural system in the extension mainly consists of VKR-columns, HSQ-beams and hollow core slabs, see Figure 3.1c. Some slabs and beams in the upper storeys of the original building have been strengthened with carbon fibre reinforced polymers. The slabs were strengthened with regard to bending moment and the beams were strengthened to be able to spread the high concentrated loads from the new steel columns that were placed on top of the beams near the edge.

The anchorage of the new part was achieved by attaching post-tensioned steel plates to the upper core. These plates extend several storeys downwards where they are anchored into the existing core, see Figure 3.2. Careful surveying of the existing building showed that the building was vertically straighter than initially calculated, which meant that the design value of the horizontal load due to unintended inclination could be decreased. Another contributing factor to the decrease of the horizontal loads was that a more favourable terrain category with regard to wind load could be chosen than when the original building was designed.

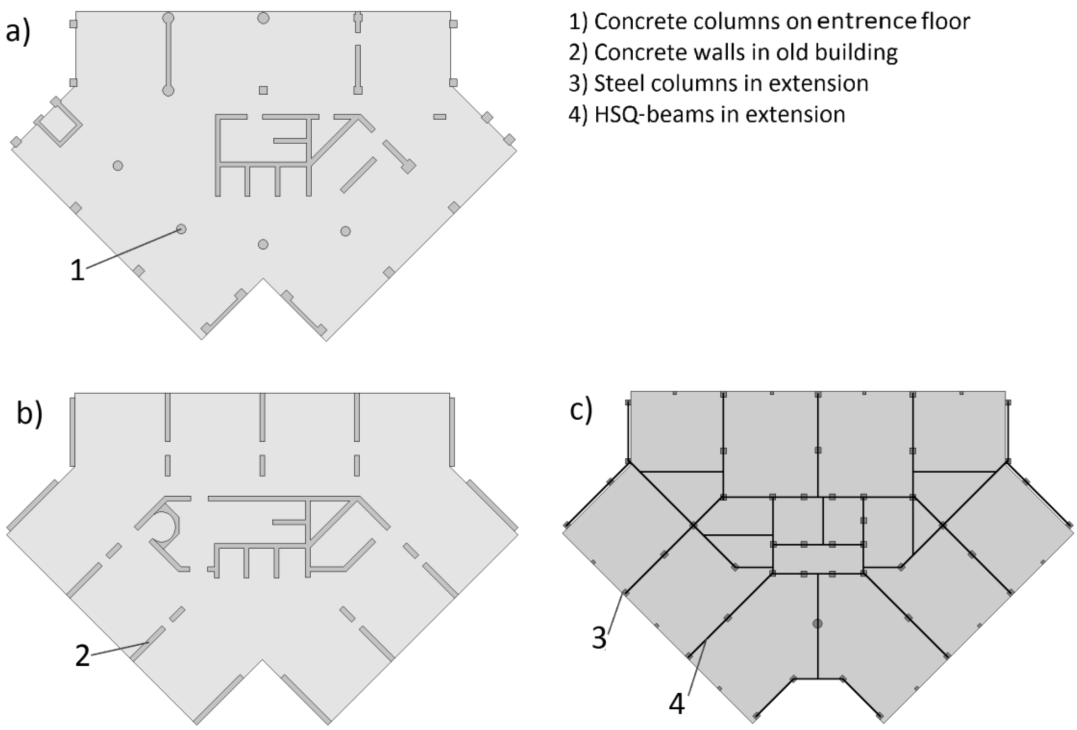

Figure 3.1 Structtural system of Gothia Central Tower, a) entrance floor, b) upper storey in old building, c) storey in the extension.

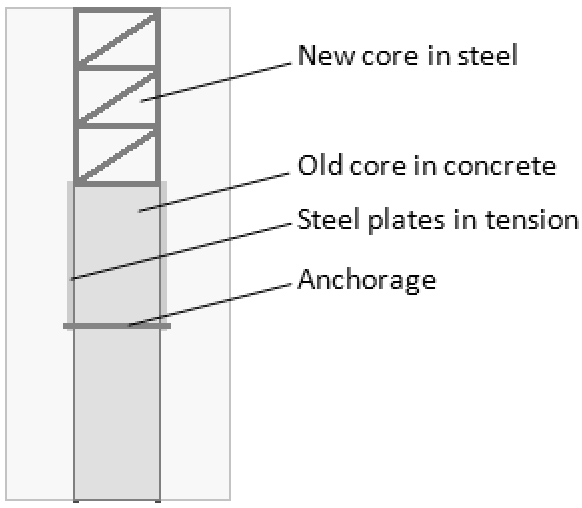

Figure 3.2 Principle for anchorage of the new core in Gothia Central Tower.

3.1.2 Hotel – Scandic Opalen

The hotel Scandic Opalen, located in central Göteborg, was built in the beginning of the ’60s. The original building has eleven floors and consists of in-situ cast concrete, Samuelsson, E. (2013-01-24). Transversal walls between the hotel rooms take the load in the upper part of the hotel, see Figure 3.3a. On the two lower storeys, the layout differs so that more open spaces are created. The horizontal loads are transported downward by the gable walls and the elevator shafts. The foundation consists of end-bearing piles.

Five extra storeys were added in 2009. As displayed in Figure 3.3b, the structural system consists of steel columns and beams. On top of the beams hollow core slabs are supported. To make the extension possible strengthening was performed in terms of increasing the bracing capacity of the gables, installing new columns through the old installation room and driving new piles into the clay along one of the gables, Samuelsson, E. (2013-01-24).

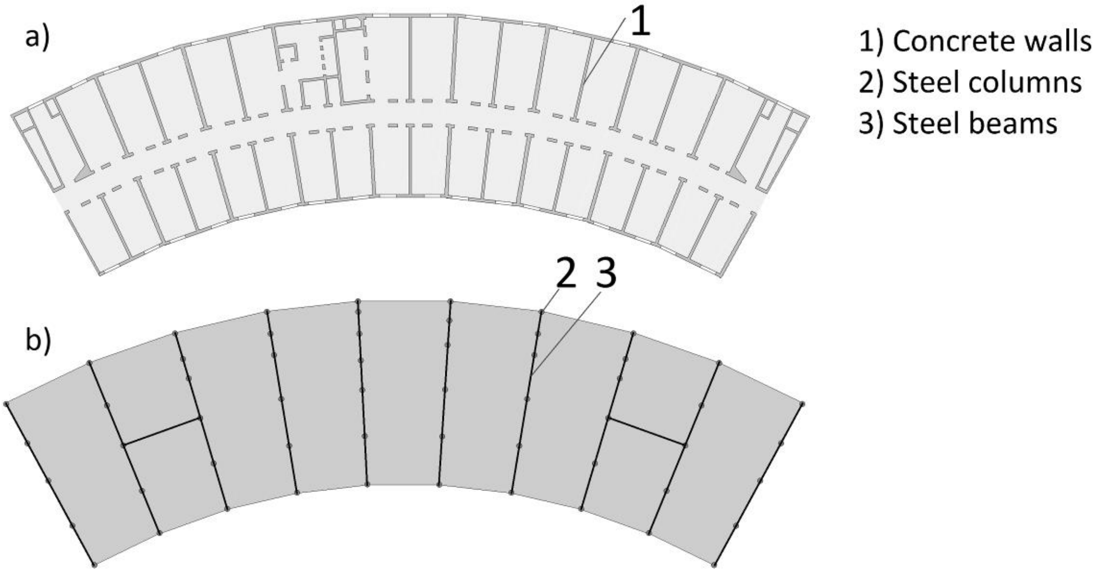

Figure 3.3 Plans in Scandic Opalen, a) storey in original building, b) storey in extension.

3.1.3 Hotel – Scandic Rubinen

Hotel Scandic Rubinen is located at Kungsportsavenyn in central Göteborg. The original building was built in the ‘60s and consists mainly of in-situ cast columns and beams, Jarlén (2013-03-13). On top of the beams prefabricated TT-slabs are supported. The height of the original building varies and the lower part contains three storeys above ground plus one basement.

At the time of writing a storey extension is being built on the lower part of the hotel. As can be seen in Figure 3.4 five new storeys are added so that the extended part will reach the same height as the left part in Figure 3.4. The new structure consists of steel columns and HSQ-beams with hollow core slabs, Jarlén (2013-03-13). To minimise the height of the beams, the steel columns stand with a spacing of 4 m, which can be compared with 12 m for the columns in the original structure. This difference in spacing is solved by storey-high trusses (number 3 in Figure 3.4) that shift the load to the concrete columns. Among other things, the project also includes strengthening of rectangular concrete columns by additional steel profiles on the sides of the columns.

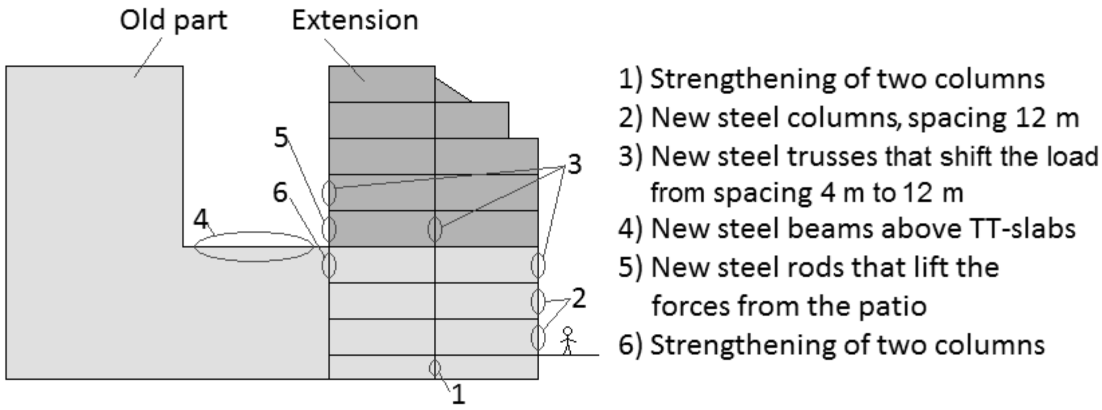

Figure 3.4 Section through Scandic Rubinen.

3.1.4 Office building etc. – Bonnier's Art Gallery

Bonnier's Art Gallery is located in central Stockholm and was built upon an existing three-storey building. The original superstructure consists of columns, walls and slabs of in-situ cast concrete founded on footings on bedrock, Skelander (2013-02-12). The old building lies in a steep slope which means that all three storeys are visible at one side of the building while the road on the other side of the building is in level with the roof of the old structure.

Five new storeys were added in 2006. The first two floors contain an art gallery, while the remaining levels hold offices. Many of the original columns were too weak for the extension and needed to be strengthened, see number 6 in Figure 3.5. This was achieved by section enlargement, Skelander (2013-02-12). Stability issues were solved by a new stabilising stairwell in prefabricated concrete (number 2 in Figure 3.5) and a new concrete wall that was installed at one gable, ELU (2013). The wall was prefabricated in the added part while the continuation of this wall in the old building was strengthened through section enlargement (number 3 and 4 in Figure 3.5). Drilled steel core piles were used to anchor the stabilising wall.

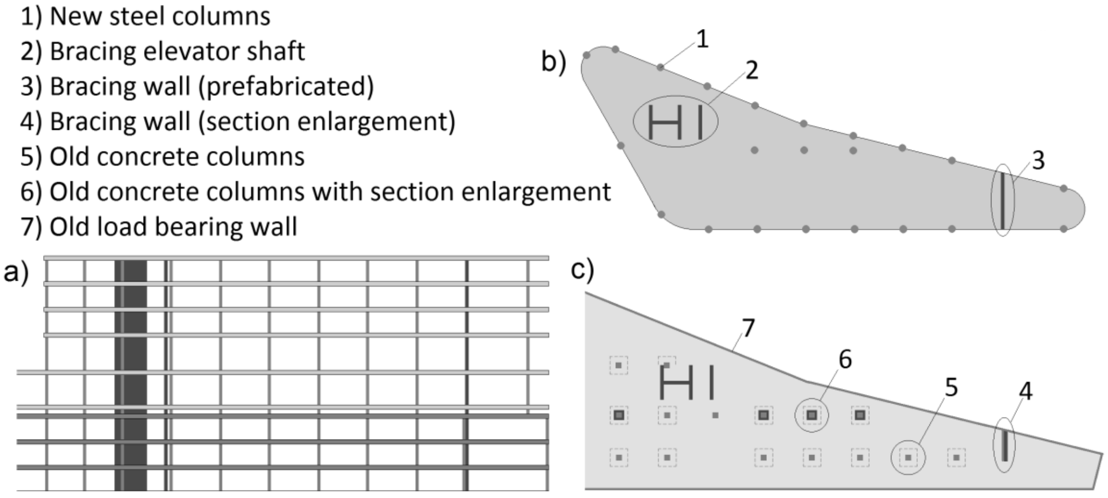

Figure 3.5 Structural system of Bonnier's Art Gallery, a) section, b) plan of new part and c) plan of old part.

3.1.5 Office building – HK60

HK60 is an office building in Sickla, Stockholm. The original building contained eight storeys and was constructed in 1962. The whole building was cast in-situ. The external walls in the longitudinal direction are load-bearing and inside the building there are two rows of columns with beams, see Figure 3.6.

The storey extension project, finished in 2013, included removal of the old roof and parts of the walls on the top floor, which earlier had been used for installations, Bågenvik (2013-03-14). Thereafter, four storeys with steel columns, HSQ-beams and hollow core slabs were added. The lower floors were renovated at the same time and, since a more open layout was desired, every second of the concrete columns were removed. To make up for this decrease in capacity, strengthening of the remaining columns was required. This was achieved by section enlargement, where 10-15 cm concrete was added on one side of the columns. According to Jonsson (2013-04-17), the choice to only strengthen one side of the columns was based on the fact that the added load was greater on that side. Self-compacting concrete was used and new stirrups were installed in the column to anchor the new layer. Interaction at the interface between the concrete layers was however neglected. Apart from the columns Bågenvik stated that the structure was strong enough to avoid strengthening.

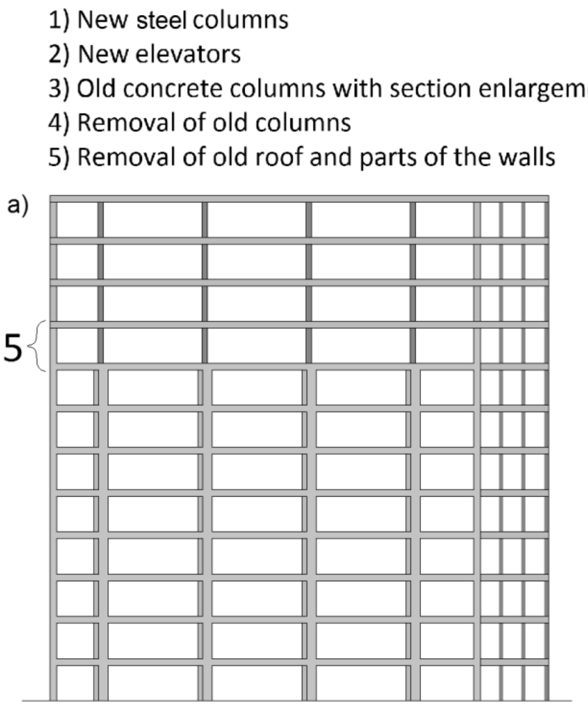

Figure 3.6 Structural system of HK60, a) section, b) plan of new part and c) plan of old part.

3.1.6 Residential building – Apelsinen

A storey extension is planned on a four-storey building located in Kungsbacka, 30 km south of Göteborg. The original structure was built in 1976 with a load-carrying system of concrete walls mainly oriented in the transverse direction of the building, Johansson (2013-01-31). The building is located on varying thickness of clay above bedrock and two thirds of the structure is founded on end-bearing piles, while the other end is founded on footings due to a shorter distance to the bedrock.

A two-storey extension is yet to be carried out along with renovation of the apartments in the building. The structural design for the extension and its accompanying strengthening was carried out according to the Eurocodes, since the magnitude of reconstruction was quite extensive, Kilersjö (2013-02-05). No real weaknesses were detected in the building, but as with the residential building at Glasmästaregatan (Section 3.1.8), beams must be placed upon the roof slab to shift the load to the walls. Strengthening of the foundation will also be required and extra piles are to be added and connected to the load-bearing walls through lintels in the same way as described in Section 6.5.1.

3.1.7 Residential buildings – Backa Röd

The five residential buildings in Backa (northern Göteborg), each with four storeys, were built in 1971 and are parts of a large residential complex. The buildings are low square-shaped tower blocks with a stairwell in the centre providing direct access to the apartments. The tower blocks are in need of renovation, which is to be carried out in association with a storey extension, Gerle (2013-02-12). Both internal and exterior walls are load-bearing and consist of prefabricated concrete elements, Carlsson (2013-03-28). Figure 3.7a shows the layout of the load-bearing walls in the original building. The buildings are founded on end-bearing piles due to a deep layer of clay.

In the future two storeys are to be added on each building. The load-bearing walls in the extension will instead consist of timber studs, Carlsson (2013-03-28). Figure 3.7b shows that, even if the extension contains six apartments per floor instead of four, the timber stud walls can be placed above the old walls. Additional glulam beams will however be needed above some openings in the original structure. The calculations for the foundation are not finished at the time of writing, but Carlsson estimated that the extension only will add about 5-10 % additional weight to the piles. However, additional piling will probably be needed beneath the new elevator.

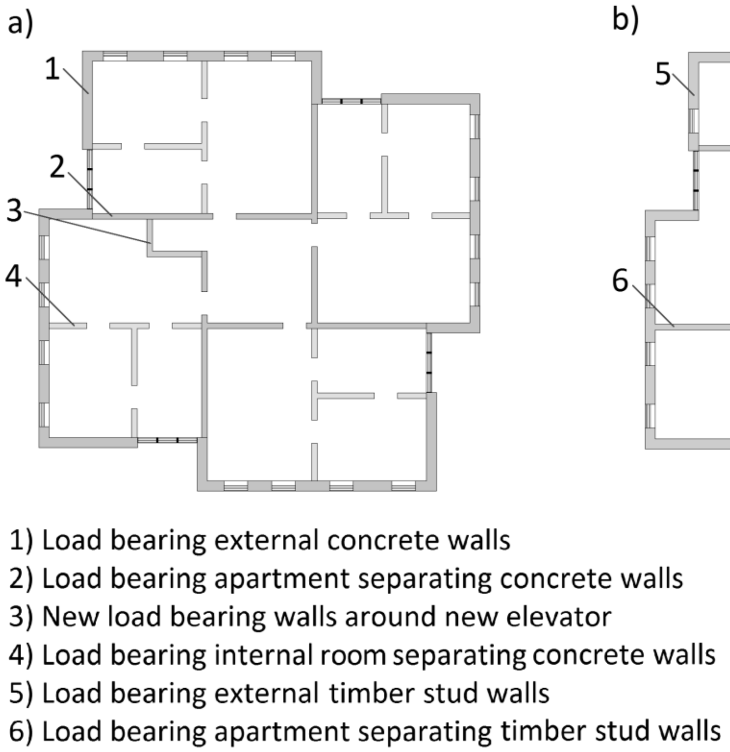

Figure 3.7 Load-bearing walls in the residential buildings in Backa Röd, a) storey in original building, b) storey in extension.

3.1.8 Residential buildings – Glasmästaregatan

This project includes two buildings built in 1965 and situated in Krokslätt in southern Göteborg. Both are residential buildings, mainly four storeys high. The structures are typical Swedish residential buildings where each stairwell only serves the adjacent apartments. The load-carrying internal concrete walls were cast in-situ along with the slabs, Carlsson (2013-02-06). A few prefabricated columns are located along the façade. The building is placed directly on bedrock.

Two new floors are being added at the time of writing. Most of the original structure is very robust and therefore not in need of any strengthening, Carlsson (2013-02-06). However, since the load-bearing walls of the new part do not coincide with the original walls, the roof slab needs to be strengthened with longitudinal steel beams that shift the loads to the walls. This is illustrated in a simplified way in Figure 3.8. It was decided to carry out renovations of the old apartments along with the storey extension, Östling (2013-02-06). To reduce the need for elevators and thereby the costs, internal corridors are being built to access the new apartments.

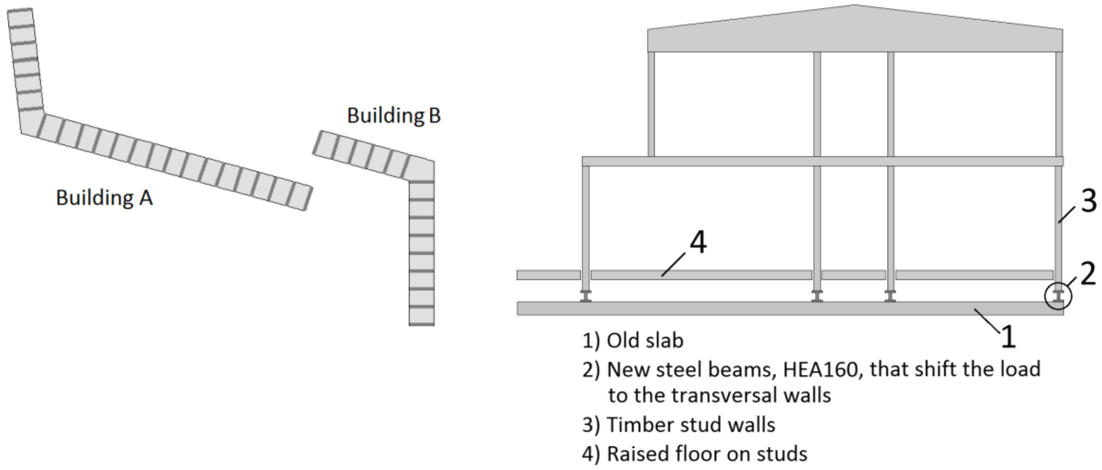

Figure 3.8 Buildings at Glasmästaregatan, a) the two buildings viewed from above with load-bearing transversal walls, b) load-bearing system for the extension.

3.1.9 Residential building on garage – KaverösPorten

KaverösPorten is situated in Kaverös in Göteborg and was originally a parking garage built in 1965. The garage, situated on bedrock, is three storeys high and consists of in-situ cast columns, beams and slabs, Nilsson (1991). The structure was in very bad state before the project started. This is described further in Section 2.3.4.

In 2009, the garage was renovated and an extension with two to three floors with apartments was added, Östling (2013-03-04). In addition to the renovation of the concrete members, a new system of beams was added on top of the old roof slab to be able to transfer the new loads to the columns. When compared to the project at Glasmästaregatan, Östling also said that it was a large benefit that the original building had no residents to consider.

KaverösPorten has not been investigated as thoroughly as the other projects and is therefore not treated in Appendix B.

3.1.10 Residential buildings on garage – Studio 57

Studio 57 is situated in Eriksberg on the north side of the river in Göteborg and consists of three residential buildings built on top of a parking garage. The garage was built during the '90s at which time a deeper knowledge about how to design with regard to resistance against de-icing salts etc. had developed. The building was therefore in a very good state. The structure consists of columns, beams and slabs that were all cast in-situ, Wibom (2013-04-05). Both the beams and the slabs were post-tensioned, which resulted in a tight structure that prevents cracking. The slabs were cast on top of a corrugated steel plate so that a composite slab was created. The foundation consists of end-bearing piles that go through an about 15 m deep clay layer to an inclined bedrock surface.

The extension was finished in 2009 and consists of three residential buildings with three to four storeys. Mostly due to a very tight schedule it was decided not to strengthen the original structure, but instead let the new residential buildings rest on big precast concrete beams that shift the load to new columns that go through the garage and down to new pile groups, Wibom (2013-04-05). This is illustrated in Figure 3.9. Wibom claimed that if a solution with strengthening of the old foundation had been chosen, it would have been hard to ensure that the added load would go to the new piles. For the case with end-bearing piles, the old piles must deform more before the new piles are loaded (if they are not prestressed). Figure 3.9 contains simplified sketches of the building.

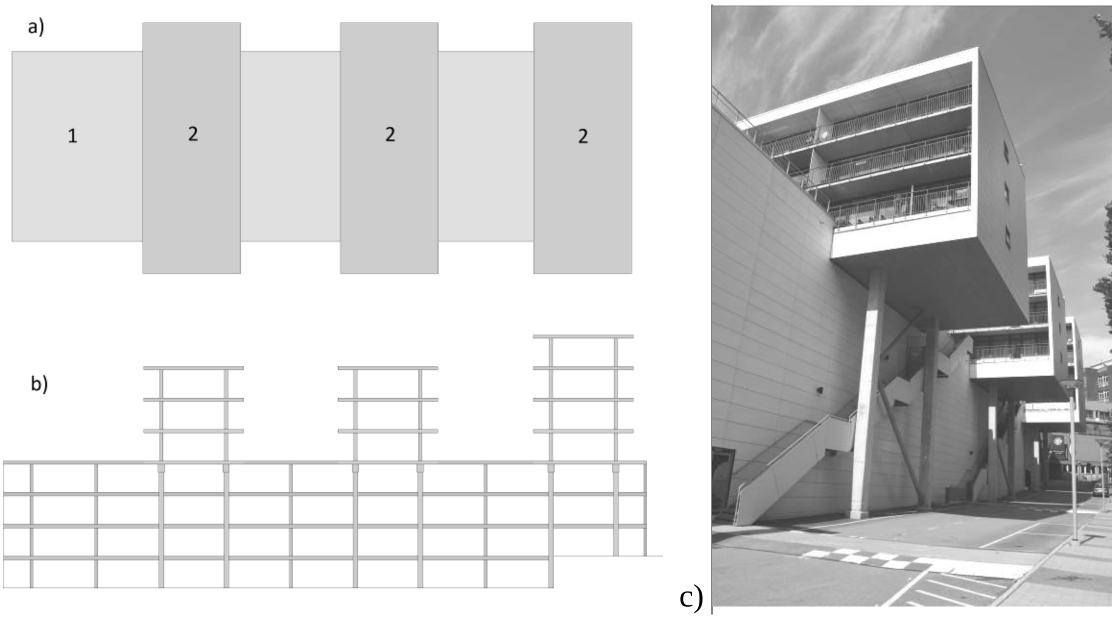

Figure 3.9 Structural system of Studio 57, a) overview from above where 1 shows the old garage and 2 shows the extensions, b) section in longitudinal direction, c) photo of the building with the new columns that support the extension.

3.1.11 Student housing – Emilsborg

The student housing built in the early '60s is a five to six storeys high building (excluding the basement) with a curved banana shape and internal corridors, Bergstrand (2013-03-01). This layout is rather similar to the one used in Scandic Opalen, see Section 3.1.2. The entire structure was cast in-situ on foundation walls, but since the underlying bedrock is inclined, concrete footings were also used in some places.

A two-storey extension was completed in 2012 in connection with a renovation of the existing apartments. The load-bearing walls of the original building had in general an excess capacity due to sound regulations, Bergstrand (2013-03-01). Strengthening of the roof slab was achieved by casting an additional layer of concrete. One of the most critical parts was the connection between the load-bearing walls and the foundation walls. Strengthening of the foundation was required beneath the new elevators. The building had very good stability in the transverse direction, but needed some extra attention in the longitudinal direction in form of cross bracings.

3.1.12 Student housing etc. – Odin

The building called Odin was erected in 1940 near the central station in Göteborg and a major reconstruction was performed in 2002, when six storeys were added. Today, the building contains student apartments, offices, a supermarket, a hotel and a restaurant. It also has a parking garage in the basement. The original structure has an in-situ cast column-beam system of rather poor quality concrete, with a strength class corresponding to around C15-C20, Wibom (2013-04-12). The building is located on deep thicknesses of clay and the original foundation was therefore performed with 18 m long timber trunks as cohesion piles. An illustration of a section through the building can be seen in Figure 3.10.

During the design of the extension the soil and foundation were analysed with a FEM-software. From this it was found out that the piles and soil could take the increased load, but the pile caps were too weak. To shift the new load from the pile cap, it was decided to strengthen the foundation with winged steel piles, Wibom (2013-04-12). This is illustrated in Figure 3.11. The VKR-profiles were prestressed to ensure that the winged steel piles were loaded immediately. Many of the columns also needed some extra attention and it was decided to increase their capacity by section enlargement with self-compacting concrete to ensure proper filling. The choice to use section enlargement instead of for example steel profiles or CFRP wrapping was primarily made to reduce the risk for punching shear. The increased area of the column reduces the local shear force per unit with on the pile cap. Figure 3.12 contains some illustrations of different section enlargements that were performed.

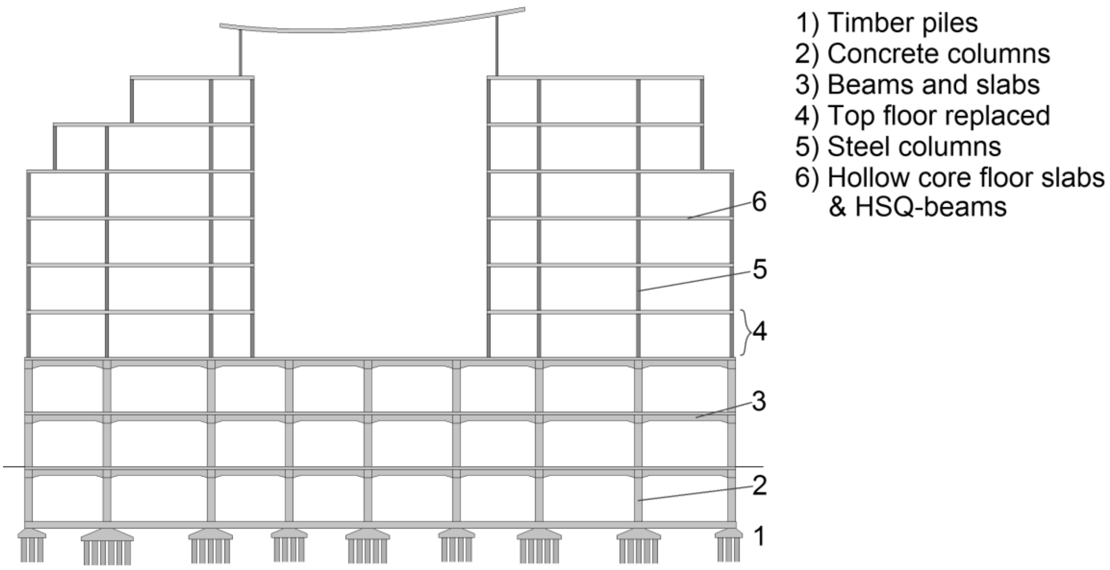

Figure 3.10 Structural system of Odin.

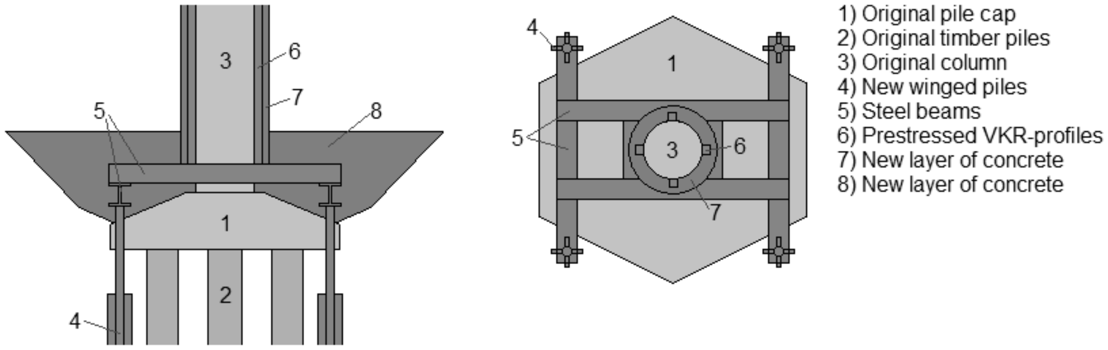

Figure 3.11 Illustration of how the foundation at Odin was strengthened.

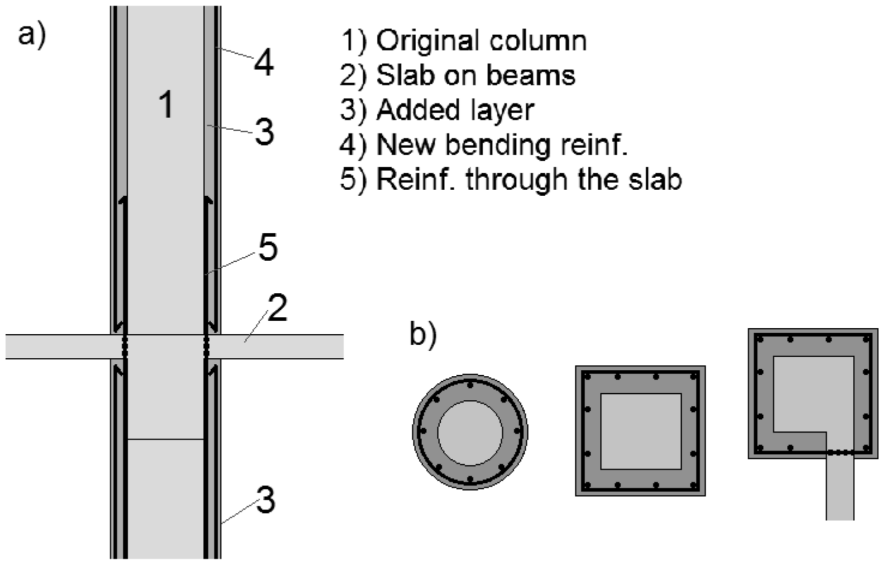

Figure 3.12 Strengthening of some of the columns at Odin.

3.2 Experiences about suitability of existing structures and extensions

To choose a structure with prospects for storey extension needs careful consideration. The same goes for the choice of superstructure for the extension, which can be very dependent on the existing structure. In this section the collected experiences concerning these decisions are discussed.

Buildings with load-bearing internal walls are very common in building structures in Göteborg. If buildings with such walls are used in residential buildings or hotels, sound demands can make the walls thicker than needed for the structural capacity for buildings with limited height. This often results in very robust structures, which are suitable for storey extensions.

Another type of buildings that, according to Östling (2013-03-04), generally are suitable for storey extensions are parking garages. Östling has experiences from storey extensions both on residential buildings and a parking garage, and argued that it was very beneficial to avoid having to consider people living in the original building during the construction. On the other hand, garages are often in a bad shape, which can result in extensive need of renovation. However, if columns etc. already are in need of strengthening, it can be advantageous to take the opportunity and strengthen them for storey extension as well.

3.2.1 Experiences about accessibility

Older buildings have often been built according to other accessibility demands than today, meaning that the inside measurements and lack of elevators differ from the current requirements. The rules of today must be fulfilled in the extension. However, an increase in accessibility can also be achieved for the existing apartments. Due to the geometrical properties of the existing building and layout of apartments etc., it may however not always be economically defendable to install elevators that are accessed from every apartment.

At Glasmästaregatan in Göteborg the accessibility was increased from 38% to 77% for the existing apartments. Even though the city planning office would prefer 100% accessibility, this was not possible to motivate economically, Östling (2013-02-06). It should however be noted that an installed elevator is considered as an increase in the standards of living for the residents and therefore motivates an increased rent. It is therefore important to consider the shape of the building when selecting a potential building for storey extension. A more favourable layout in terms of elevators is when the existing stairwells already serve a large amount of the apartments, e.g. in form of corridors. This was the case at Emilsborg, where only three new elevators gave full accessibility to both old and new apartments, Bergstrand (2013-03-01).

The placing of the elevators is decisive for how the new floors are to be designed. For square shaped tower blocks a good solution can sometimes be to incorporate the elevator shaft into the existing structure. This method can be possible since the apartments often are placed around one single stairwell, which gives accessibility to all flats. For long and narrow buildings, as with the project on Glasmästaregatan mentioned in the previous paragraph, several elevators are often needed to achieve accessibility to all new apartments. A way to reduce the number of elevators might however be to build internal or external passways to which a small number of elevators are connected.

Another aspect with the accessibility demands of today is how they affect the layout of apartments. The project in Backa Röd (Section 3.1.7) is one example where the demands on open spaces in bathrooms and kitchens prevented the use of similar layout in the extension as in the original building.

3.2.2 Experiences about economy

Bostads AB Poseidon, a housing company located in Göteborg, is in general very positive to storey extensions, Gerle (2013-02-12). However, according to Östling

(2013-02-06) it might sometimes be difficult to financially motivate a storey extension unless certain conditions are met. Poseidon intends to perform storey extensions of some tower blocks in Backa Röd, see Section 3.1.7, and has already completed a test project where they simultaneously renovated existing buildings and lowered the energy consumptions, Gerle (2013-02-12). According to Gerle, simultaneous renovation and storey extension makes the project more justifiable than just renovation.

One general consideration from several of the studied objects is that the choice of existing structure and extension often is made so that strengthening is limited or avoided, especially when it comes to residential buildings. The estimated rent is often what limits the price for the project, since a high construction cost ultimately leads to a higher rent for the residents. According to Östling (2013-02-06), some persons from the city council searched through Göteborg in the early 2000s to identify buildings that were suitable for storey extension projects. Apart from criteria concerning the surroundings and location of the building, they also searched for robust structures situated on bedrock. By choosing such structures the economic aspects are according to Östling optimised. However the more important and popular a location is, the more money might be motivated to spend on strengthening. As an example, when Gothia Central Tower was extended, rather large strengthening measures were taken.

3.2.3 Experiences about extensions

When it comes to the extension itself, the studied examples are very different and the interviewed persons have various opinions of what kind of structure is best suited. At Glasmästaregatan, Willén (2013-02-06) argued that a timber stud structure is good, since it is light-weight and reduces the number of times that the protecting tent needs to be opened. Kilersjö (2013-02-05) however mentioned that a timber alternative was rejected for the project at Apelsinen (Section 3.1.6) due to sound requirements. During the extension of Emilsborg (Section 3.1.10) a semi-prefabricated concrete solution was used instead, mostly due to the genuine and solid appearance that follows with the choice of a concrete structure. It was also desired to obtain a structure that corresponded with the rest of the building, Bergstrand (2013-03-01). Yet another solution that is frequently used is a column-beam system in steel with concrete hollow core slabs. This method is mainly chosen when open spaces or an adjustable layout is desired.

3.3 Experiences about the Eurocodes and older design codes

The Eurocodes were recently established as the governing design code, and since 2011 all new structures must be designed according to the Eurocodes. However, the scope of the Eurocodes is rather limited when it comes to redesign and strengthening of existing structures. Blanksvärd (2013-04-08) said that a code for treating existing structures was to be complemented and that Blanksvärd himself would contribute to it. He estimated that this part was to be finished sometimes between 2015 and 2020.

Until then the designer is forced to interpret the codes without certain guidelines. The Eurocodes must be applied for the extended part, but it is more unclear what rules are applicable for the old part. Wibom (2013-04-12) said that his consulting firm based their design on logical deductions and calculations, claiming that the upcoming design code would arrive to a similar result. He mentioned an example with a pile foundation being verified according to the old design code and said that it would be illogical to reverify it with new codes.

The partial safety factors are in the Eurocodes applied differently compared to the old Swedish codes. According to Skelander (2013-02-12) it is possible to apply the new standards to older existing buildings, but he pointed out that it is important to distinguish the loads from each other. It is also important to remember that the Eurocodes are harsher than their predecessors. The project at Gothia Central Tower was started earlier, just to be able to follow the old design code, Samuelsson, E. (2013-01-24). Samuelsson believed that it would not have been possible to continue with the project without reducing the number of added storeys if the Eurocodes were to be followed.

3.4 Experiences about critical members and strengthening

The structural member that came up to discussion the most times during the interviews was the roof slab. This slab is in general not designed to be loaded by a new structure with its imposed loads. If it is not possible to place the new members directly on the existing load-carrying members, some kind of strengthening of the slab is in general necessary. At Glasmästaregatan and KaverösPorten longitudinal steel beams were placed upon the roof slab to shift the load to the primary wall members as illustrated in Figure 3.8, Carlsson (2013-02-06) and Östling (2013-03-04). A similar method will be used at the residential building Apelsinen according to Johansson (2013-01-31). At Emilsborg the roof slab was instead strengthened by an additional layer of concrete that was applied after cleaning and wetting the already rough top surface, Bergstrand (2013-03-01). Yet another solution to cope with the new loads on the roof slab was used at Scandic Opalen, where new columns were placed in the installation room that is situated on the top floor of the old building, Samuelsson, E. (2013-01-24). In the projects Odin and HK60 the original roof slab was completely removed and replaced instead of strengthened.

Another part of the structure that was critical in some projects was the foundation. This was especially the case for buildings situated on clay. At Scandic Opalen, Odin and Apelsinen new piles were needed beneath some of the load-bearing walls. At Backa Röd additional piles will probably be needed to transfer the load from the new elevators, Carlsson (2013-03-28). When Bonnier’s Art Gallery was built, drilled steel core piles were installed and anchored to take the tension from the new bracing trusses, Skelander (2013-02-12). At Studio 57 end-bearing piles were used for the existing building, Wibom (2013-04-05). This type of piles complicates the strengthening of the foundation, since it might be difficult to increase the load without failure in the piles. This is due to the fact that the new piles need to deform before contributing to the global resistance. This effect can however be avoided if the added piles are prestressed. For this specific project new pile groups were instead used. New columns transfer the loads downwards to the new piles independently of the existing structure. Strengthening of the foundation can be expensive and Östling (2013-02-06) claimed that a foundation on bedrock is almost a prerequisite when performing storey extensions for residential buildings. This was however not the case for all the studied projects.

Load-bearing walls are seldom the most critical members for low buildings. The capacity of columns was on the other hand crucial in several of the treated projects. In both Gothia Central Tower and Scandic Opalen the capacity of the columns on the lower floors restricted the number of added floors, Samuelsson, E. (2013-01-24). In the project with Bonnier’s Art Gallery a large number of columns needed to be strengthened by an additional layer of concrete, Skelander (2013-02-12). However, the load increase in this project was very large, so strengthening of the load-bearing elements was quite expected. The residential building at Glasmästaregatan also had a few load-bearing columns that probably would have needed to be strengthened, if more storeys would have been added, Carlsson (2013-02-06). Every second of the columns in the office building HK60 were removed in order to achieve a freer internal layout, Bågenvik (2013-03-14). However, this led to that the remaining columns had to be strengthened. Here a one-sided section enlargement was chosen. For Hotel Scandic Rubinen rectangular columns were strengthened by applying steel HEB-profiles at the two opposing sides, which increased the capacity with regard to buckling and crushing, Jarlén (2013-03-13). Another project where the columns were critical was Odin, where they were strengthened on several storeys. At this site various shapes of section enlargements were applied, mainly due the fact that the columns needed an increased area to reduce the risk of punching shear failure.

Some of the interviewed persons mentioned problems with too high compressive forces at the connections between load-bearing members, e.g. when concentrated forces from columns should spread out into larger members or when narrow members are placed upon each other with a perpendicular orientation. Samuelsson, E. (2013-01-24) described that strengthening of several beams was required at Gothia Towers due to the small cross-sectional area of the new columns that were placed upon the beams. Here the strengthening of the beams was performed by use of carbon fibre reinforced polymers, CFRP. Another example is Emilsborg where one problem was that the connection between the foundation walls and the walls that rested on them in some places was rather small, Bergstrand (2013-03-01). The walls were not strengthened, but the capacity was limiting for the increased load.

How to take the horizontal forces from the wind and unintended inclination can also be a problem, especially for high rise buildings. For Scandic Opalen the ability to take the tilting moment was increased by the use of steel plates that were attached to the gables, Samuelsson, E. (2013-01-24). For Gothia Central Tower the global stability could instead be accounted for by utilising the fact that the existing building stands straighter than assumed in the initial design and that a better terrain category could be adopted. Bonnier’s Art Gallery also had critical stability issues, which were solved by a new shear wall and an additional staircase acting as a core.

The many transversal load-bearing walls in the studied residential buildings make them very stable in the direction that otherwise would seem to be the critical one. A lack of load-bearing walls in the longitudinal direction may however be a problem. Bergstrand (2013-03-01) explained that new bracing steel trusses were needed at Emilsborg to stabilise the building in the longitudinal direction.

Even if the building has enough capacity to transfer the tilting moment from the original structure down to the foundation, problems can occur when the tensile forces should be transferred from the extension to the original building. In several of the projects this was simply solved by overlapping steel ties. At Emilsborg, that has an extension with a rather high density compared to the other buildings, the main purpose of the added steel bars was however to fixate the semi-prefabricated wall elements before casting, Bergstrand (2013-03-01). However, at both Hotel Scandic Opalen and Hotel Gothia Central Tower, the two highest buildings that were investigated, it was critical to transfer the tensile forces from the new structure to the old building. A rather similar solution was chosen for both structures. In Gothia Central Tower steel plates were attached to the new core and stretched several storeys down where they were anchored. The steel plates were pretensioned to ensure that they are activated directly when elongated.

3.5 Experiences about the construction work at the building site

The interviews resulted in many important aspects about how a storey extension project differs from erection of new buildings. Both Kilersjö (2013-02-05) and Östling (2013-02-06) explained that the communication between commissioner, designer, contractor and other participants is even more important in storey extension projects than in normal building projects. Samuelsson, E. (2013-01-24), among others, also claimed that the designer must be engaged very early to ensure a superstructure for the extension that is adapted and fits properly to the existing load-bearing system. According to Samuelsson it is crucial to survey the building in an early state and compare it to the old documentation. He and several others of the interviewed persons explained that the measurements in the original drawings don’t always correspond exactly to reality. If this is not observed and trust is put into the drawings, it can give severe consequences, especially if prefabricated elements are used in the extension. Problems with unwanted load effects due to eccentricity can also occur, if the precise location of columns and load-bearing walls is unknown.

3.5.1 Experiences about logistics

The logistical problems of a storey extension project can be vast since the building often is situated in the middle of a built environment. It can therefore be difficult to find available space for building site offices and storages etc. Willén (2013-02-06) explained that this was a dilemma at Glasmästaregatan. One way to improve the situation might be to move the site offices inside the existing building, as was done at Gothia Central Tower, Samuelsson, E. (2013-01-24). However, for this to be possible an evacuation of the users is required. This may not always be appropriate and possible in residential buildings. It is also motivated to plan deliveries so that they occur just before the material is needed at the site to reduce the need for storage.

Another logistical issue that differs from when new buildings are erected is that all material must be lifted to the roof in some way. Kilersjö (2013-02-05) explained that this fact must be considered in an early state, when the economic aspects are treated.

3.5.2 Experiences about weather protection

When erecting a new structure on an already existing building, the old roof often needs to be removed. Since the roof is important for the weather protection, some temporary cover might become necessary. It is possible to erect the extension and make a tight building before the original slabs are torn down. Another approach is to erect a weather protective tent in which the extension then is built. If a timber solution is chosen, the use of a tent is often required to protect the timber anyway. A tent like this was for example used at Glastmästaregatan, where it also was desired to limit the number of times the tent was opened, Willén (2013-02-06). This intent even affected the choice of structural system so that an alternative with timber studs was selected.

Johnsson (2013-02-12) at Lindbäcks Bygg said that when their prefabricated timber modules are assembled, they start with the weather protecting roof. Each morning, if the weather is favourable, they start by lifting off the roof from the building to be able to mount the modules. By the end of the day the roof is put in place again to protect the building during the night. More information about these modules is provided in Section 4.2.

3.5.3 Experiences about residents and other affected persons

Throughout the different interviews the persons that live or work in the existing building came up for discussion many times. Several of the interviewed persons explained that it can be hard to satisfy the residents who often think that the project only causes them trouble without directly improving their situation.

One important issue is to encourage good cooperation with affected persons through the entire project, Östling (2013-02-06). At Glasmästaregatan this was achieved by forming a group with volunteers who discussed the project and provided suggestions for improvements. In that project the commissioner also chose to dedicate one person to keep the residents updated with information about the project and being available to answer question. At Backa Röd a pilot project was performed on one of the buildings, Gerle (2013-02-12). By doing so the commissioner could evaluate possibilities at the same time as residents in the upcoming buildings could view the results and benefits of the project. During this pilot project, several things that could be improved were discovered.

Another possible way to improve the popularity of the project is to make sure that the original residents benefit from it as well. This can for example be done by giving them access to new elevators or taking the opportunity to renovate the old building at the same time. It is however very important to consider the consequences of an increased rent carefully. Several tenants may choose to look for other alternatives, if an increased rent is forced upon them. It is of importance that these people are properly informed in advance and, if possible, helped towards new accommodations. If it is decided to increase the rent in the old part of the building, it is however very important that the raise only comes from the renovations and the access to new services, Östling (2013-02-06). The extension itself should never affect the economy for the original residents.

Concerning evacuation it was decided in many of the investigated projects to let the tenants stay in the building as long as possible, due to economical reasons. In several of the projects, the accompanying renovation of the original building however resulted in part-time evacuation. The extension of the office building HK60 was however performed after the old tenant had moved out. The purpose of this redesign was to improve the appearance of the building and attract new tenants, Bågenvik (2013-03-14). In this way, the construction work inside the old building was simplified. One way to remove the disturbance in the most affected part of the old building is to evacuate the top floor and possibly use it as building site office, Samuelsson, E. (2013-01-24). As discussed in Section 3.5.1 this can also simplify the logistical situation.

4 Consideration for the extension

In this chapter important considerations that should be regarded in an early stage of a storey extension project are presented. Focus is here put on the extension.

4.1 Height of extensions

There are many things that together limit the number of storeys that can or should be added to the building. Besides resistance and stability of the structure and its foundation, there are some other parameters that also should be considered. These limitations are presented in this section together with some of the effects that a higher building may generate.

4.1.1 Height allowed by zoning

The first and most significant limit is the regulations in the zoning documents. Sometimes, the allowed height has not been fully utilised by the existing building, which means that it is easier to get permission for a smaller extension. However, if changes in the zoning restrictions are needed, as is often the case, it is important to be prepared for a rather long processing time. The City Council of Göteborg will normally not make changes in the restrictions unless they are in alignment with their general plan of the area. In general, a processing time of 30 months is to be expected, Swan (2013-02-22). However, the processing time might be reduced if the plan is complementing a need in a certain area or is in agreement with the council’s aim to produce 3500 residencies each year.

4.1.2 Height nature of the surroundings

One thing that affects the height of the extension is how well it fits into the height nature of the surrounding built environment, Bergenudd (1981). This can according to Bergenudd be treated in several different ways and some of these are illustrated in Figure 4.1. Sometimes it is preferred to keep the buildings on an equal level by adding the same number of floors to all of the buildings. In other cases storeys can be added to lower buildings to create a more homogeneous height nature and in yet other situations it could be better to make an accentuation by adding a high extension to one building only.

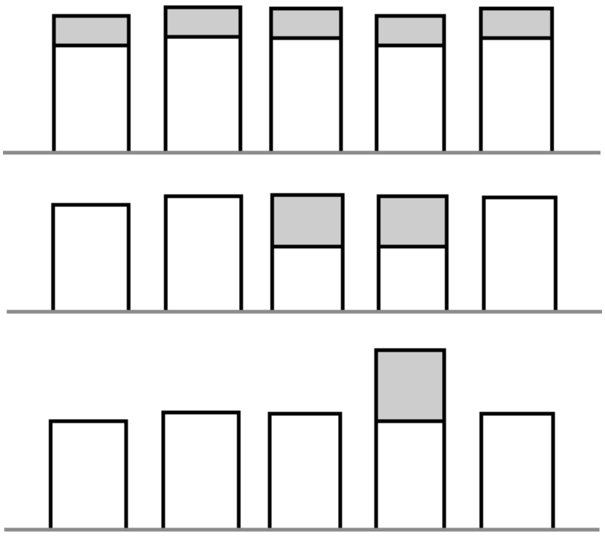

Figure 4.1 Different ways of treating the height nature and impression of the built environment, after Bergenudd (1981).

4.1.3 Consequences for fire regulations due to increased height

The fire regulations are highly dependent on the number of storeys in a building. If extra floors are added, the regulations for the entire structure may change. Therefore, this section contains an overview of parameters that change at the different heights.

The information is based on the Swedish building code *Boverkets byggregler (BBR 2012)*, Chapter 5 Brandskydd (fire protection) and has been arranged in a list presented in Appendix C to better illustrate the distinction between the changes that accompany the different heights. However, it is highly recommended to read the whole text at Boverket's homepage since the information in Appendix C does not cover the regulations that are independent of the number of storeys.

Figure 4.2 shows a summary of the list in Appendix C by indicating at which storeys the fire regulations are changed. Since some of the demands are based on the height of the building rather than the number of storeys, a second row in the figure has been added. Observe that some of the limits have a greater influence than others.

Figure 4.2 Critical heights concerning fire regulations.

Figure 4.2 indicates that, apart from the number of added storeys, the total number of storeys can have a great influence as well. As an example, it can be better to increase the number from six to eight than from seven to nine, thus avoiding harsher fire regulations.

4.2 Type of superstructure for the extension

Buildings of today can be built with a large variety of structural systems. In this section different structures and their applicability in a storey extension project are described.

Apart from the general pros and cons of the systems (which have not been deeply treated in this project), there are several aspects with storey extension projects that affect the choice of superstructure. The most prominent may be how the existing structure limits at which points or lines the added loads can be applied. According to Bergenudd (1981) it is advantageous to keep the same activities in the newly constructed part as in the already existing part. Since the two different parts then can have more similar layouts, an analogous placing of the load-bearing members can be used. However, the accessibility demands of today may make it difficult to keep the same internal layout, Gerle (2013-02-12). The general opinion of how apartments, offices etc. should be composed has also changed during the passing decades.

Another issue that can affect the choice of superstructure for the extension is how the original building influences the suitability of prefabrication. As Samuelsson, E. (2013-01-24) argued the dimensions of the old building might not be very regular. When new buildings are erected, it is desired that many prefabricated elements have the same size, but this may not always be possible in storey extension projects. This condition is in favour for the choice of a more adaptable superstructure.

Apart from the superstructures used in the studied projects, there is one additional method that is worth extra attention, namely to use prefabricated modules. This technique, which involves prefabrication of housing modules in a factory and transportation to the building site by truck, has become increasingly popular in parts of Sweden during the last years. One company that specialises on this kind of timber modules is Lindbäcks Bygg. According to Johnsson (2013-02-12) their housing modules are prefabricated to an extent of about 85 %. Johnsson thereby claimed that the erection itself only takes about one week after which some additional weeks are dedicated to the connections between modules and finishing the internal work etc. The modules that Lindbäcks Bygg provides are 4*8*3 m^3 and have load-bearing stud walls at all sides of the modules. The modules have both floor and inner ceiling which together create double layers so that sound class B is reached. One module weighs about eight to nine tons including the fittings and the number of added storeys is limited to six, mostly due to stability issues. This number is however based on the case when the building is erected directly on the ground. In storey extensions the wind load can be higher. When the modules are added to an existing structure, they must be anchored with steel ties.

The applicability of these modules to storey extension projects has, according to Johnsson (2013-02-12), proven to be good. The most significant advantage is the fast erection and low need of storage area at the building site. To better fit the modules to existing buildings, all modules are designed specifically to the current project in consultation with an architect, i.e. no standard modules are used. If it is difficult to place the load-bearing walls directly above the existing walls, an extra system of timber joists can be added on the roof slab of the existing structure. However, one disadvantage with these timber modules can, according to Johnsson, be a limitation of the architectural appearance. The restrictions on the size of the modules with regard to the transportation also make it complicated to create big open spaces.

4.2.1 Self-weight of the extension

The weight of the superstructure is of course very relevant in a storey extension project. In Section 3.1 the studied projects are treated. It was found that the same types of superstructures were used in many of the executed projects. To be able to overview the most common superstructures for extensions, rough estimations of the self-weight of the structures have been performed. The weights are based on drawings of studied buildings and may therefore vary greatly for other structures. The results are shown in Table 4.1. Observe that the calculations are approximate and that only the self-weight of the load-bearing members have been accounted for. For the vertical members the total weight on the whole storey was divided by the area of the floor.

Table 4.1 Approximate self-weights of common superstructures in extensions.

Type of superstructure	**Approximate weight per m^2 and storey**
Timber stud walls and SSP timber floors	110 kg (100 kg floor, 10 kg walls)
Steel columns, HSQ-beams and concrete hollow core slabs	350 kg (only about 15 kg from steel)
Concrete walls and slabs	600 kg (500 kg floor 100 kg walls)
Concrete columns and slabs	550 kg (500 kg floor 50 kg columns)

4.2.2 Fire protection

Another factor that needs to be considered when choosing superstructure is the need of fire proofing. Steel, timber, and concrete functions differently when subjected to fire and therefore need different attention.

Steel subjected to fire loses around half of its load-bearing capacity after 15 to 40 minutes, Paroc Firesafe (2008). Demands may however require the structure to remain safe much longer. For such cases some kind of external fire proofing is required. A variety of materials and configurations are available, such as cladding the member in stone wool or gypsum boards. It is also possible to use expandable paint as means to fire proof a member. Whichever method is chosen, the fire protection will add extra thickness to the member. This thickness does not only depend on the material chosen to strengthen, but is also dependent on how long the structure needs to remain functional, number of sides exposed to fire and the dimensions of the member. For example, a steel profile of type HEB180 subjected to fire from four sides needs three layers of Gyproc Normal gypsum boards (37.5 mm in total) to fulfil R 60, Gyproc (2010). If Paroc stone wool is used instead, 20 mm is required, Paroc Firesafe (2008). An example of how a column can be clad in gypsum is illustrated in Figure 4.3.

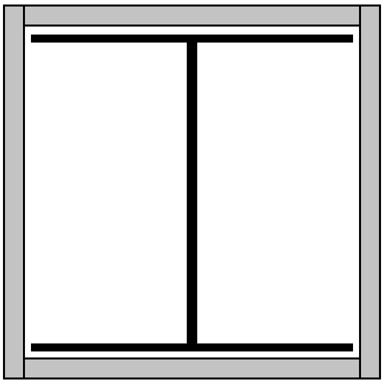

Figure 4.3 Steel H-profile clad in gypsum for fire protection, after Gyproc Firesafe (2008).

Larger timber members remain partly functional under fire loading. The outer layers burn and turns into char, losing their load-bearing capacity, while the inner layers remain intact. This means that timber members can be designed with extra thickness that allows for the outer parts to char. This charred timber also acts as thermal insulation, slowing the fire penetration, Carling (2001). Even under prolonged fire exposure the temperature of the uncharred parts remains below 100 °C, which means that the thermal expansion is limited as well. For a square glulam column with a side of 400 mm, an additional 50 mm of timber material can be anticipated to fulfil the fire demands. However, a column in a fire cell can be made smaller than the requirements, if there are alternative ways to safely carry the loads downwards, i.e. redundancy, TräGuiden (B) (2013).

Burning of timber can also be hampered chemically by various fire retardants, TräGuiden (A) (2013). These retardants mainly affect the early stages of fire propagation and prolong the time until flashover occurs. It is possible to clad the timber members or parts of them with different materials similarly as with the steel profiles. This might be of extra interest for details.

Concrete generally performs well with regard to fire loading and extra fire protection may sometimes be unnecessary, The Concrete Centre (2013). This can be derived from the non-combustible properties of the material and its slow rate of heat transfer. This slow rate of heat transfer helps to protect the concrete itself, but also things that may be located on the other side of the member. In general it is the reinforcement that is sensitive to fire and increased heat, but as long as the cover thickness is sufficient, it should be adequate, Engström (2013-04-25). O'Brien and Dixon (1995) recommend a preliminary design where sufficient cover thickness is handled both with regard to fire protection and corrosion. For example, a slab needs a cover thickness of around 35 mm to ensure two hour fire resistance.

When selecting high strength concrete with a tighter composition, it can however be problematic if the entrapped water expand too quickly, causing spalling of concrete, Engström (2013-04-25). This can be solved by casting in plastic granulates that melt in case of fire and make room for the water to expand.

5 General approaches for strengthening of structural members

In this chapter an introduction to the materials and approaches used to strengthen concrete structures is given. It contains information that mostly is independent of which structural member that is treated. Strengthening of specific members is instead addressed in Chapter 6. Extra weight is in this chapter put on fibre reinforced polymers, FRP, since they are used more and more frequently although many designers are quite unfamiliar with the methods and their applicability.

5.1 Sectional enlargement with additional reinforced or plain concrete

One simple way to strengthen concrete members is to just add a new layer of reinforced or plain concrete, increasing the thickness of the member. There are however some important aspects to consider, when fresh concrete is applied to old. There are mainly two different approaches when strengthening members with concrete; regular casting with moulds and with use of shotcrete.

5.1.1 Shear resistance at interfaces between old and new concrete

Good interaction between new and old layers of concrete is according to Statens råd för byggnadsforskning (1978) beneficial, since a homogeneous behaviour is desired. Figure 5.1 shows an example of how the bond strength at the interface might affect the behaviour of a composite member in bending. If interaction cannot be reassured, the whole member will deform more. To achieve good bond the old surface must be roughened and cleaned. Removing debris from the surface is crucial to achieve good bond. Sometimes epoxy glue might be useful to further strengthen the bond.

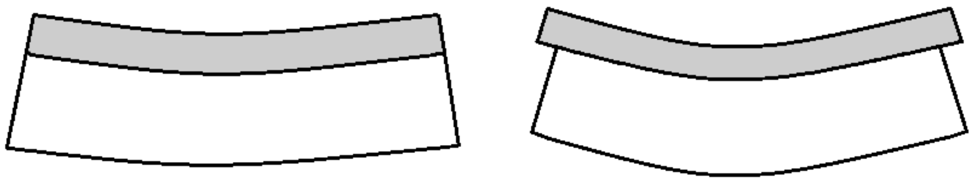

Figure 5.1 Bending of elements width a new layer of concrete, a) with full interaction and b) without interaction.

In Eurocode 2, the roughness of a surface is classified into four categories and assigned cohesion and frictional constants, see Table 5.1. Higher values for friction and cohesion lead to better interaction between the layers and reduce the need for mechanical anchors.

Table 5.1 Classification of surface roughness according to Eurocode 2, CEN (2004)

Category	Description	Friction, μ	Cohesion, c
Very smooth	A surface cast against steel, plastic or specially prepared wooden moulds.	0.5	0.25
Smooth	A slipformed or extruded surface, or a free surface left without further treatment after vibration.	0.6	0.35
Rough	A surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour.	0.7	0.45
Indented	A surface with indentations complying with Figure 6.9 in EN 1992-1-1:2003	0.9	0.50

If the shear forces at the joint interface are significant, additional anchors in the shape of bolts or stirrups might be needed as well. Such high shear situations may include casting additional layers on beams, while slabs usually generate low shear situations where stirrups often are superfluous, Statens råd för byggnadsforskning (1978).

It is also important to consider the shrinkage in the two layers of concrete and evaluate the risk of cracks due to uneven shrinkage. One way to reduce differential shrinkage is, according to Statens råd för byggnadsforskning (1978), to continuously wet the old surface during several days before application of the new concrete. This was for example done at the studied project at Emilsborg, Bergstrand (2013-03-01).

5.1.2 Strengthening with shotcrete

To minimise formwork and labour shotcrete can often be a good substitute to conventionally cast concrete, when it comes to strengthening of concrete members, Statens råd för byggnadsforskning (1978). Shotcrete is basically an approach where concrete is sprayed onto the surface instead of being cast in moulds. The process requires experienced workers if a good result should be achieved.

The two main properties of shotcrete are its rapid setting and early high strength, Häglund (2006). It is very suitable in tunnels and canal structures or in earth retention systems. It can also be used as fire proofing or protection of soft weak areas. The rather simple application also makes it possible to use to strengthen existing concrete members.

There are two different basic ways to apply the shotcrete. The first method implies that the almost dry components are sprayed through a hose with help of compressed air. The water is then added in the nozzle. This approach is known as the dry method.

Another way is to use the wet method, where the concrete is already mixed with water before it is pumped through the hose. The dry method is normally used for strengthening of concrete members, while the wet method is preferred for bigger projects, e.g. when tunnels are strengthened, Besab (B) (2013).

Any bad concrete should be removed and the surface should be cleaned before application of shotcrete, Weber (2011). The water from the cleaning should however dry out before the shotcrete is applied so that the surface better bonds to the new layer. For the best result no more than 5 cm of concrete should be added in each layer. When vertical surfaces are strengthened, to avoid collapse from self-weight, the application should be started from the bottom and continue upwards.

According to Statens råd för byggnadsforskning (1978) the water-cement ratio for the dry method can be rather limited due to the way the components are naturally mixed when they hit the surface. The strength of the shotcrete can thereby be quite high, about 70 MPa, Weber (2011). The values for the compressive strength and density are similar to high-strength concrete, but the hardened properties are more dependent on application, Häglund (2006). The use of small aggregates that only reach up to 8 mm is quite common in shotcrete. This gives rather large shrinkage, which is one of the drawbacks with shotcrete, Statens råd för byggnadsforskning (1978). The aggregates can be heated or cooled before mixing to influence the temperature development and setting properties, Häglund (2006).

To use fibres (either steel or glass) in shotcrete is also quite common. The fibres can be mixed in the concrete before the spraying and reduce the need of additional ordinary reinforcement, Weber (2011). Fibres are mainly applied to get a more ductile failure mode, Häglund (2006). However, too high fibre content leads to poor compaction and reduced strength and toughness. The length of the fibres is more important than their shape and the length/diameter ratio should therefore be kept fairly high.

The bond between concrete and a good quality shotcrete, with or without steel fibres, can in general become strong and durable, if the old concrete surface is prepared properly. Such methods may include hydroblasting or chipping with jackhammers followed by sandblasting, Talbot et al. (1995). Talbot et al. (1995) did not observe any considerable difference when testing the bond strength for the dry and wet methods in case of a hydroblasted surface.

One important consideration when members are strengthened with shotcrete is how to ensure that the added material is loaded. If parts of the added load should be directed through the new layer, measures must be taken to guarantee that the new layer is compressed and deformed directly when the load is increased. Otherwise, the enlargement can only contribute to a higher bending stiffness for the increased load (assuming that full interaction between the layers can be ensured). It is also important to check that the existing member has capacity to deform further and thereby resist its part of the load increase.

5.2 Strengthening with externally mounted steel

Regular reinforcement bars can of course be used in the added concrete, when performing a section enlargement on a member. There are however also other areas of application in which steel can be used to strengthen concrete members, for example by externally mounted sheets and profiles or as means to apply a prestressing force.

Steel plates attached by a surface adhesive have been used since the 1960s to retrofit concrete structures. However, the durability problem exhibited by steel in form of corrosion can affect the bond between the plate and the concrete, Norris et al. (1997). Damages of this sort might also be difficult to inspect and detect. The large and heavy steel plates can also be problematic to transport, handle and install.

Steel profiles can also be used to improve the resistance of structural members. These can be used for various kinds of members and be installed in different manners. However, the problem with how to transport and install the members remains. It is also important to be careful when design steel so that proper fire and corrosion protection is applied.

One of the main issues with strengthening with external steel is to attach them and achieve interaction with the concrete. If the steel is bolted to surface, it has to be accounted for that the steel and concrete won't have any strain compatibility. Interaction is especially important when the members are subjected to large bending moments.

Steel is quite an expensive material, and the price of steel products of today is around 12-20 SEK/kg depending on shape, size and steel quality, BE Group (2013). This means for example that a S355 HEA140 costs around 352 SEK/m and a S355 HEB180 around 719 SEK/m (both these profiles are used in the calculations in Appendix D and E).

5.2.1 Strengthening members with prestressing steel

Prestressing is primarily suited for strengthening members with regard to the flexural capacity. The most common application is to use it on beams or slabs. In this section it is described how prestressing is applied on existing members. Some of the problems that may follow are also treated.

Prestressing of existing members functions in a similar manner as internal post-tensioned tendons without bond, but with the tendons (or single strands) connected only to the exterior of an already existing structural member by end-anchors and deviators, Nordin (2005). Deviators are used to change the angle of the tendon to acquire a better utilisation of the prestressing, Ahmed Ghallab (2001). Since it is at the deviators and anchors that the forces are transferred to the concrete, it is not possible to follow the moment curve in the same way as with internal tendons. Figure 5.2 illustrates one possible way to apply external prestressing to a single span beam.

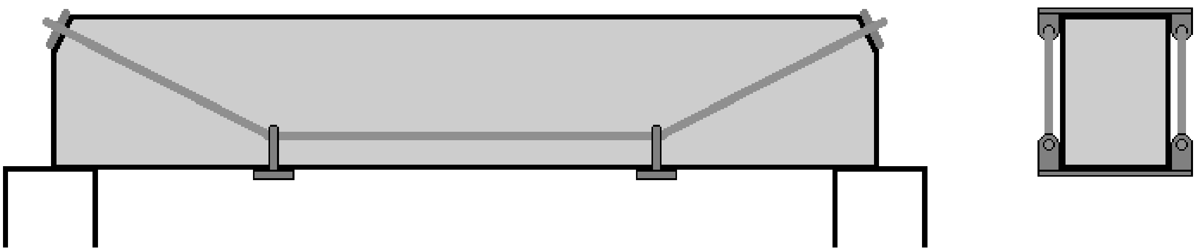

Figure 5.2 External prestressing of a simply supported beam, after Statens råd för byggnadsforskning (1978).

Utilising prestressing enhances the behaviour of structural members by means of constraint forces. The prestressing effect results in equilibrium of forces where the prestressing steel is in tension and the concrete is in compression. This creates a member with capacities to better resist tensile loading. It should be noted that it is not possible to get any strain interaction between the concrete and the external tendons since the steel are not bonded to the concrete. The effect can instead be calculated by introducing concentrated forces where the prestressing force is applied and also where the tendons change their direction, i.e. at the deviators, see Figure 5.3. It is important to keep in mind that the strain in the prestressing steel is not compatible with the strain at the same level in the concrete. The elongation of the tendon is instead spread out over the length between the anchors due to the lack of interaction.

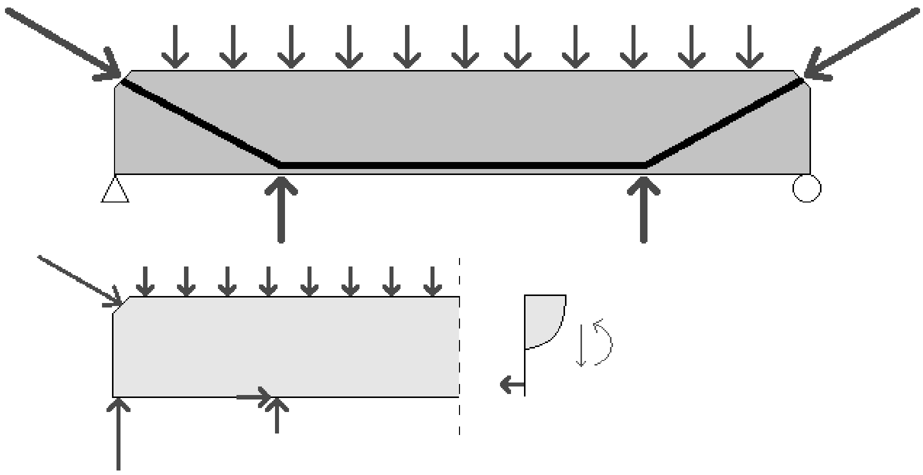

Figure 5.3 Effect of external prestressing.

The prestressing itself does not directly affect the ultimate load of the member. However, when the problem is viewed in the way that is illustrated in Figure 5.3, it can easily be seen that vertical force at the deviator acts like an intermediate support. In this way, the moment in the mid-section of the member can be reduced.

One advantage with external prestressing is a better serviceability state behaviour, since the deflection is reduced. The cracking will also be delayed and any existing cracks can be pulled together. Inspection, replacement and even re-tensioning of external tendons can be easily executed due to their accessibility. Low frictional losses can also be expected, since no interaction between steel and concrete is present.

There are also some disadvantages by using external prestressing. It may as mentioned above, for example give the member a more brittle behaviour in the ultimate state, Picard et al. (1995). Another issue is that the zones at which deviations and anchors are installed often are troublesome and it is therefore of importance that these discontinuity regions are designed to manage large concentrated horizontal or transversal forces. Anchor damage and failure at the anchor heads may completely disable the prestressing effect in the member. Fire protection must be fulfilled and can for example be achieved by covering the tendons by mineral wool or shotcrete, Statens råd för byggnadsforskning (1978).

In the same way as when designing new prestressed members, the prestressing steel will be subjected to relaxation. Therefore, high strength steel is most often used. The high strength steel also suffers relaxation, but since the strands can be tensioned to a higher level, the relaxation doesn't pose such a big problem as for regular steel. Another solution is to use carbon fibre reinforced polymers. This is described further in Section 5.3.3.

Another disadvantage with steel is that there may be problems with degradation from corrosion. The relatively small area in combination with the use of high strength steel makes the tendons even more vulnerable to corrosive damage. Even a small layer of corrosion can considerably decrease the capacity of the steel due to a rather large ratio of the cross-sectional loss. This can be avoided by protecting the steel, for example by plastic sheeting.

5.3 Strengthening with fibre reinforced polymers

Fibre reinforced polymers (FRP) is a composite material consisting of fibres surrounded by a polymer matrix. The matrix is what keeps the fibres together and transfers the forces between the individual fibres, Carolin (2003). The matrix also acts as protection for the fibres. Different materials can be used as matrix, but one that is epoxy based is most commonly used.

The type of fibres can also be altered. Carbon, glass, and aramid are however the three most commonly used fibres in civil engineering, Carolin (2003). All fibres behave elastically until a brittle failure and normally have a higher tensile strength than steel. Apart from this carbon fibres have several benefits in a structural context, when compared to the alternatives, such as its high strength to weight ratio and high modulus of elasticity, Cozmanciuc et al. (2009). Carbon fibres are therefore the most common type used when strengthening buildings, creating so called carbon fibre reinforced polymers, CFRP.

At present, strengthening of concrete structures with FRP composites is not treated in the Eurocodes or any other standards, Täljsten et al. (2011). Designers who use FRP strengthening are therefore referred to available handbooks or material manufacturers. However, the structure needs to be evaluated, if it is appropriate for FRP composites before any FRP strengthening measures are taken. It is also of importance to follow the guidelines from the supplier and not to mix components from different suppliers to avoid lack of compatibility.

If the member has suffered too extensive corrosion damages, other strengthening methods might be more appropriate, Täljsten et al. (2011). Alternatively, a prior strengthening of the member, e.g. with section enlargement, can be followed by additional strengthening with CFRP.

Figure 5.4 shows typical stress-strain relations for steel and different kinds of FRP. As illustrated steel has a ductile behaviour, while FRP behave almost elastically until a brittle failure is reached. This means that the fibres will continue to carry loads even after the stress level at which the steel yields.

The behaviour of FRP can be customised. The figure for example shows that the carbon fibre reinforced polymers (CFRP) can be designed to have a high modulus of elasticity (HM) or high strength (HS). The manufacturer normally offers two or three different levels of stiffness; low, medium and high, Täljsten et al. (2011). The high tensile strength of CFRP makes them suitable for strengthening with regard to tensile forces. In compression the strength is significantly lower, since the fibres will behave similarly to the ones in timber and buckle away from each other.

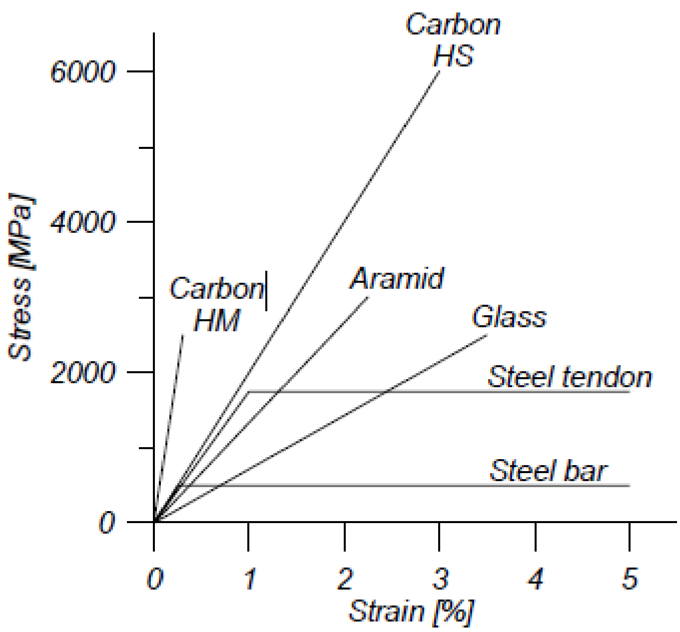

Figure 5.4 Behaviour of steel and FRPs as glued reinforcement, taken from Carolin (2003).

As mentioned above CFRP have a very high tensile strength while also being very lightweight. Furthermore, the amount and size of the fibres can be used to even further customise CFRP. When used to retrofit structural members of concrete such as columns, beams and slabs, it can add significant capacity without adding notable weight that would further increase the load on foundations and other structural members.

The durability of CFRP is in general very good. They are resistant to corrosion as well as to many chemical compounds and do not absorb water nor exhibit any creep or relaxation, Carolin (2003). The conductive property of CFRP may however damage steel if in direct contact. It should be noted that the epoxy matrix is sensitive to UV-radiation and should if needed be protected, for example by painting the surface. Another critical issue for CFRP can be how to protect them against impacts. If there is a risk of any accidental impact, for example in parking garages, extra measures are required.

The matrix is also vulnerable to elevated temperatures and fire protection might therefore be necessary. According to Blanksvärd (2013-04-08) fire protection of FRP can be quite expensive. This can be achieved by adding protective boards. For a 60 minute structural integrity requirement, an additional thickness of 50 mm can therefore be expected, Tepro (2004). Use of shotcrete may also be an option when fire protecting the material, Täljsten et al. (2011). It can be advisable to consult the manufacturer about which systems that are most suitable for the specific case and material configuration.

The orientation of the fibres can be customised; if the fibres are oriented in one direction, the CFRP become unidirectional with very high strength in the main direction, but very low strength perpendicular to the fibres. This property puts high demand on the design, since it is important not to subject the material to any destructive stresses in its weak direction. According to Täljsten et al. (2011), a 30° deviation of loading relative to the fibre direction leads to about 70 % loss of resistance. Using unidirectional CFRP is the most common method when strengthening structural members, Carolin (2003). It is however also possible to mix the directions of the fibres to get a bi- or even multi-directional material.

CFRP are normally applied to the concrete in three different ways, surface mounted (Figure 5.5a), near-surface mounted (NSM) (Figure 5.5b), or mechanically fastened (Figure 5.5c). The bond strength of glued composites is very dependent on the quality of the installation. It is therefore of great importance that the composites are applied by experienced workers, who know what they are doing, Concrete Construction (2010).

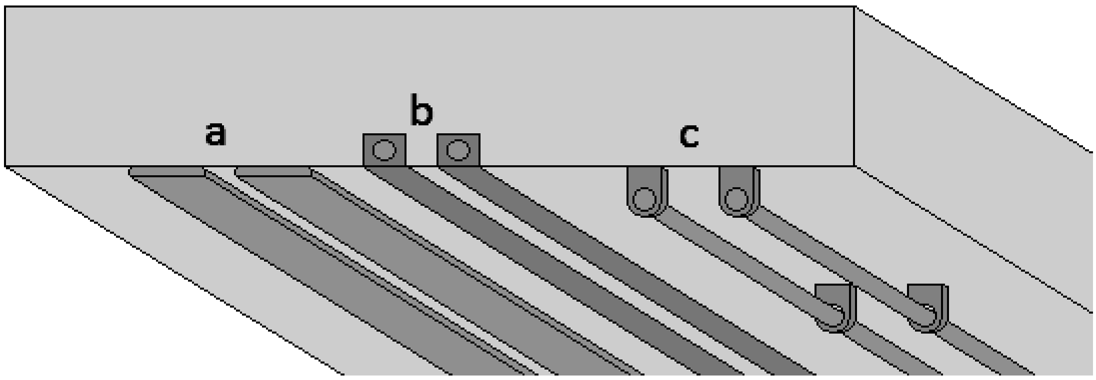

Figure 5.5 Application methods for CFRP, a) surface mounted, b) near-surface mounted and c) mechanically fastened.

The unhealthy properties of the epoxy in its uncured condition put extra demands on a good installation process, Wiberg (2003). This toxicity of the epoxy in its uncured state makes CFRP inappropriate for certain environmental certificates and is thus not always a suitable option. Research on using a cementitious matrix instead of an epoxy based has therefore been performed. The results show that using a cementitious matrix may avoid some of the problems that are present when using an epoxy based matrix, Wiberg (2003). The cementitious matrix is permeable, not hazardous and can be applied on damp surfaces. It can be modified to handle both low and high temperatures in contrast to the epoxy, Wiberg (2003), which requires a temperature above 10°C during installation and an air temperature of at least 3°C above the dew point, Carolin (2003). However, according to Wiberg (2003), the cementitious matrix cannot fully wet the carbon fibres due to its particle composition. This means that the capacity of the fibres cannot be fully utilised. A cementitious matrix also requires a longer curing time.

When strengthening a structure with CFRP, a major advantage is that it is fairly easy to transport and install the composites. There is most often no need to make holes just to transport material into structures, Samuelsson, A. (2013-05-22). It is also possible to install CFRP in areas that might otherwise be difficult to reach, for example in the vicinity of installations. Samuelsson claimed that it is here that the main advantage of strengthening with CFRP becomes evident. He said that money and time in many cases can be saved since only two people might be sufficient to carry out an adequate work without having to bring any heavy machinery.

However, the price is an important aspect to consider when evaluating CFRP as a plausible strengthening option. According to Samuelsson, A. (2013-05-22) the price of CFRP and their installation depend not only on the price of the material, but also on the extent of the contract. Strengthening of a single member with S&P Laminates CFK 150/2000 50*1.2 mm^2 may cost 1500 SEK/m. For a more extensive contract, ranging over several weeks, the cost can be reduced to 500 SEK/m for the very same material.

One of the major setbacks of strengthening with CFRP is that it can be difficult to properly anchor the forces to the supports, illustrated in Figure 5.6, Blanksvärd (2013-04-08). This is only critical for simply supported ends, since continuous structures at interior supports have their compressive zone on the lower side close the supports. This disadvantage for simply supported ends can be overcome by adding inclined ties that lift the tensile force towards the support. However, due to its superior anchorage properties, discussed in Section 5.3.2, near-surface mounted FRP show a better behaviour with regard to this, why extra anchorage may not always be necessary. It should be noted that the problem with anchorage failure might be more crucial for higher beams than for slabs.

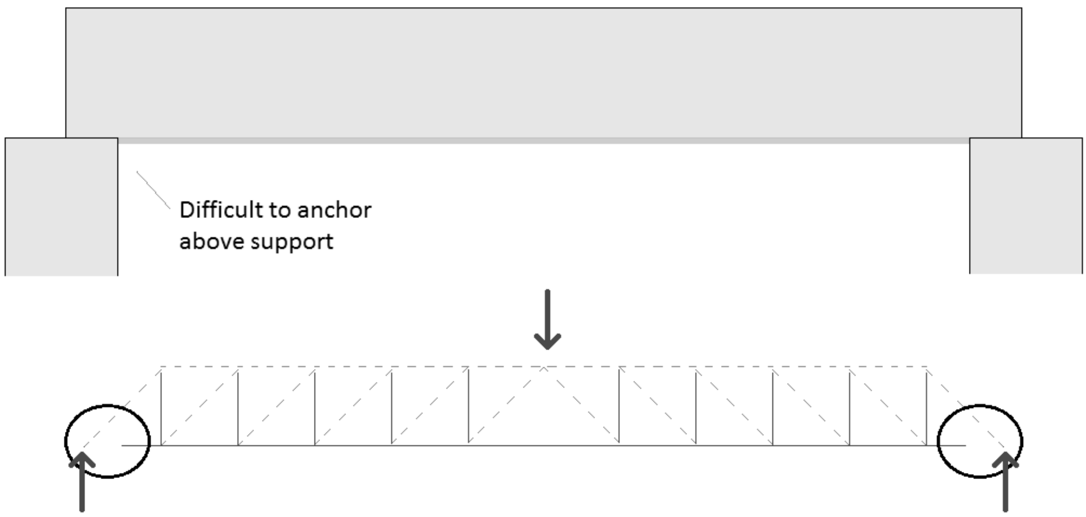

Figure 5.6 Strengthening with CFRP on the tensile side of a beam. The lack of anchorage can be seen in the lower figure.

One drawback with the less ductile properties of CFRP is that it can prevent plastic redistribution. Therefore, caution should be taken when strengthening two-way slabs or continuous members, Blanksvärd (2013-04-08).

5.3.1 Surface mounted FRP

To use surface mounted FRP is, according to EI-Hacha and Rizkalla (2004), the most adopted method for shear and flexural strengthening of slabs and beams today. It is thereby the most common of the three methods of applying FRP to concrete. Surface mounted FRP can be applied in two different configurations; either as sheets, covering large surfaces, or in strips, which only cover a limited area of the concrete. When completely covering a surface, it is important to consider that the impermeability of the epoxy may lead to premature degradation of the concrete, Wiberg (2003).

The surface mounted FRP can also be installed in two different ways; either by plate bonding or by a hand lay-up method, see Sections 5.3.1.1 and 5.3.1.2, Carolin (2003). For both alternatives surface treatment of the concrete is required to expose the aggregates to enable proper bond between the composite and the concrete. However, the surface of the concrete must be smooth enough so that the composite does not buckle and is bonded in the intended direction. This can be achieved by sandblasting the concrete and, if needed, grounding the surface. Afterwards, the surface should be cleaned from any remaining debris and visible water, since the relative humidity of the concrete has to be below 80 % during application of the epoxy.

Bond stresses between concrete and the CFRP induces tensile stresses in the concrete, and it is therefore of importance that the concrete has sufficient capacity to handle such stresses.

5.3.1.1 Plate bonding method – laminates

In the plate bonding method a prefabricated CFRP laminate is bonded to the concrete surface with a high viscosity epoxy, see Figure 5.7, Carolin (2003). To achieve a good bond any holes in the concrete surface should be mended with putty. The epoxy and the CFRP laminate are then applied in sequence and pressed against the surface. This step should be performed so that the epoxy is evenly distributed between the concrete and the plate.

The laminates are around 1-2 mm thick and can be delivered in rolled bundles, Täljsten et al. (2011). The laminate in one bundle can be up to 200 m long. This can be compared with FRP bars, which with regard to transportation are limited to a length of about 12 m.

In general, these laminates are most suitable for plane surfaces. For other situations where larger flexibility is required, the method presented in Section 5.3.1.2 is more appropriate, Täljsten et al. (2011).

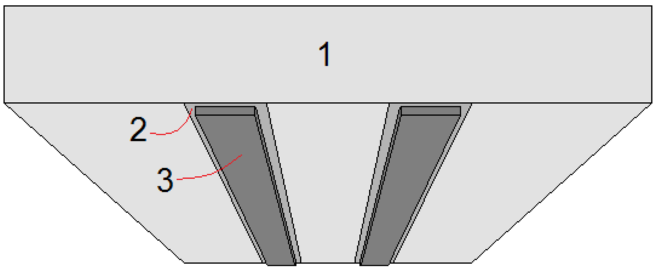

Figure 5.7 Plate bonding method for CFRP laminates.

5.3.1.2 Hand lay-up method – weaves

In this method the epoxy matrix and fibres are applied separately in sequence on the concrete surface, creating a composite at the site as illustrated in Figure 5.8. These composites are generally easy to apply and can be used on structural members of any size or shape, due to their formability. The surface treatment described in Section 5.3.1 is here followed by applying an epoxy primer, which prevents the next layer of low viscosity epoxy to be absorbed by the concrete, Alagusundaramoorthy (2002). The fibres are then pressed onto the epoxy, which thereby surrounds the fibres and act as matrix. The low viscosity of the epoxy is required to fully wet the fibres, Carolin (2003). If further layers of FRP are desired, these can be directly placed on the previous sheet with another intermediate layer of low viscosity epoxy. This process can be repeated until the total amount of fibres gives sufficient capacity. According to Täljsten et al. (2011) the number of applied layers is often limited to ten or twelve. Afterwards, the member needs to be cured for approximately seven days, Alagusundaramoorthy (2002). It is of great importance that it remains smooth and free from enclosed air. This is because the strengthening effect is directly proportional to the straightness of the fibres, Täljsten et al. (2011).

The thicknesses of the FRP weaves are around 0.1 to 0.2 mm. The thickness of the adhesive is around 0.5 mm, but the adhesive will also act as matrix for the fibres. Protection of these systems towards sunrays is therefore extra important.

Use of this type of composite requires a higher partial safety coefficient than what is used for the plate bonding method. This is due to the fact that the latter to a larger extent is manufactured in factory, which ensures fewer deviations.

The flexibility of the weave makes it more suitable for round shapes and to bend around corners than the laminates. This property makes it for example suitable for wrapping of columns, see Section 6.1.3.

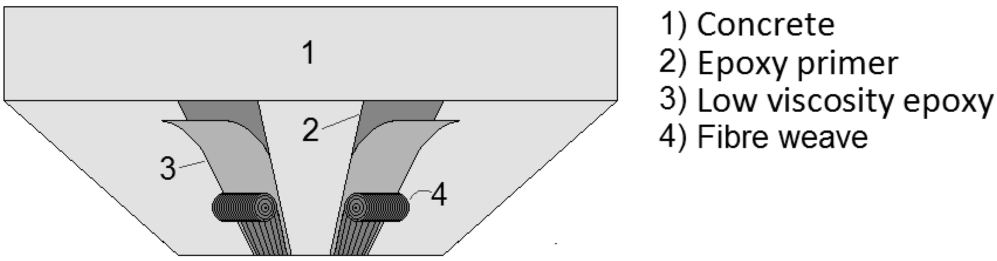

Figure 5.8 Hand lay-up method with CFRP weaves.

5.3.2 Near-surface mounted FRP

Near-surface mounted (NSM) FRP bars are very similar to laminates glued to the surface of structural members, but instead of attaching the FRP directly to the surface of the concrete, a groove or canal is cut in the concrete. The groove is then partly filled with an adhesive, either epoxy based or cementitious, in which a bar of FRP is inserted. Thereafter, the groove is completely filled and levelled by removal of excess material, El-Hacha and Rizkalla (2004). This procedure is visualised in Figure 5.9. This gives, in comparison to the glued strips, a better bond between concrete and FRP and thereby also a better material utilisation of the FRP. This increased bond can be derived from the fact that the bond occurs in more than one plane, Täljsten et al. (2011). According to Blaschko and Zilch (1999) NSM FRP also provide better ductility in the ultimate state and generate a stiffer behaviour in the service state compared to surface mounted FRP.

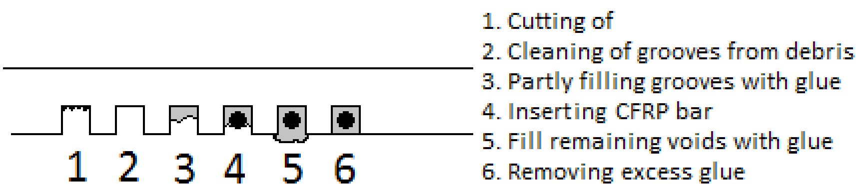

Figure 5.9 Procedure of installation of NSM FRP.

In comparison to surface bonded FRP, NSM FRP also allow quicker installation since no preparation of the surface is required, except cutting of the canals, El-Hacha and Rizkalla (2004). According to Blaschko and Zilch (1999) the cost of preparing the canals is in the same range as the cost of preparing the surface for surface mounted strips, for example by sandblasting. The layer of adhesive surrounding the NSM FRP can be quite thick, sometimes more than 5 mm. However, this adhesive is protected from fire and impacts by the surrounding concrete, Täljsten et al. (2011).

Near-surface mounted FRP is a quite newly developed method for strengthening and according to De Lorenzis and Teng (2006) one of the most promising. Steel is a possible alternative to CFRP and has been used in this manner for over 50 years, but apart from the advantages mentioned earlier in the section, NSM FRP can reduce the size and depth of the grooves due to higher strength and better durability.

Bars of FRP are very customable and can have various shapes and surface textures, El-Hacha and Rizkalla (2004). The use of slim strips gives for example very good bond resistance. Some different options are presented in Figure 5.10. It should be noted that it, as shown in the figure, also is possible to only enclose the bar from three sides and thereby reduce the depth of the canal. The choice of bars and configurations should represent the specific site conditions, while at the same time consider the cost and local availability.

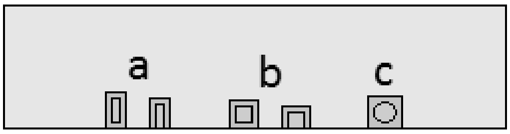

Figure 5.10 Examples of different configurations of near-surface mounted FRP.

The use of near-surface mounted FRP enables a smooth concrete surface, since the adhesive can be levelled at the surface. Members strengthened like this will have no increase in size and with some minor surface treatment such as painting, the inserted FRP can be completely concealed. The undisrupted surface that comes with this method is also very appropriate when strengthening against negative moments on the top side of beams and slabs.

A requirement for the use of near-surface mounted reinforcement is the need of a thick enough concrete cover, at least 20 mm, to allow space for the canals, Täljsten et al. (2011). It is important to bear in mind that if the groove is cut too close to the reinforcement, it can also affect the anchorage of the bars. If the cover thickness is too small, it is also possible to use T-shaped FRP bars. In this case one part of the composite is inserted into a groove, while the other part is attached as surface mounted FRP, Blaschko and Zilch (1999). This is illustrated in Figure 5.11. It is also possible to use the NSM FRP in combination with the surface mounted alternative, if desired.

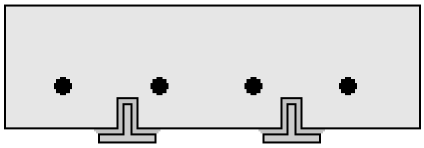

Figure 5.11 T-shaped NSM FRP bars used for strengthening a reinforced concrete slab, after Blaschko and Zilch (1999).

Compared to surface mounted FRP, NSM FRP can more easily be anchored into neighbouring elements, which increases the bond resistance even further, El-Hacha and Rizkalla (2004). This can be convenient for strengthening of frame structures with large moments at the corners or when anchoring the applied FRP over supports.

It is possible to prestress the near-surface mounted FRP and, according to De Lorenzis and Teng (2006), round bars are best suited to achieve proper anchorage. However, this effect might be difficult to achieve for structural members in an existing building, Blanksvärd (2013-04-09).

5.3.3 Mechanically fastened FRP

Mechanically fastened FRP (see Figure 5.5c) work in a similar way as externally mounted steel bars or tendons except that protection against corrosion is not required. It is however more important to consider the direction of the loading, since the fibres often are unidirectional. Anchorage zones are therefore critical areas that need careful attention, since the risk for stresses in the weak directions are especially high here. According to Nordin (2005) some further development might be necessary to achieve an anchorage system that better adapts to the material properties of FRP. An advantage with mechanically fastened FRP is that there is no curing time and therefore no need to consider unhealthy conditions.

It is possible to use fibre reinforced polymers (FRP) instead of steel for external prestressing. This will completely prevent the problem with corrosion, Nordin (2005). Carbon fibre reinforced polymers also show a better behaviour in terms of creep and relaxation than steel, which ultimately results in smaller losses of the prestressing force over time. Prestressing of FRP also enables better utilisation of the material.

6 Strengthening of structural members

In this chapter it is described how various structural members can be strengthened with regard to different types of loading. Different ways to strengthen the foundation of a structure is also treated to some extent. For information of how to treat stability issues of a structure, reference is instead made to Chapter 3. For more general information about the materials and approaches that are used to strengthen the members, see Chapter 5.

The information provided in this chapter forms the basis for the recommendations given in Chapter 8. Some of the more important strengthening methods are also investigated further and compared through calculations that are presented in Appendix D and E and described in Chapter 7.

6.1 Strengthening of columns

Columns, along with load-bearing walls, are the main members for transferring loads vertically downwards through a structure. The resistance of a column can either be determined by buckling or by its crushing. These two cases must always be considered during design. When strengthening a column, its slenderness is therefore of interest. Some approaches might be more or less appropriate depending on how large the bending moment is in comparison with the normal force.

One of the most straightforward ways to strengthen a slender column is to brace it and simply reduce its buckling length. However, this solution is not always practically possible and its aesthetical implications might be problematic.

6.1.1 Strengthening against crushing and buckling of columns by section enlargement

Increased compressive capacity of columns can be achieved by casting a new layer of reinforced concrete onto the already existing member. By increasing the thickness of the column, both the resistances towards crushing and buckling improved. The new concrete should preferably, but not necessarily, enclose the whole existing column, which then also can be complemented with stirrups, Statens råd för byggnadsforskning (1978). Figure 6.1 illustrates two cases of section enlargement of columns, one where enclosing is possible and one where section enlargement is prohibited in one direction. For the latter case, it is extra important to anchor the new part properly to the old column. To achieve a better interaction between old and new concrete, the corners of rectangular columns can be chamfered.

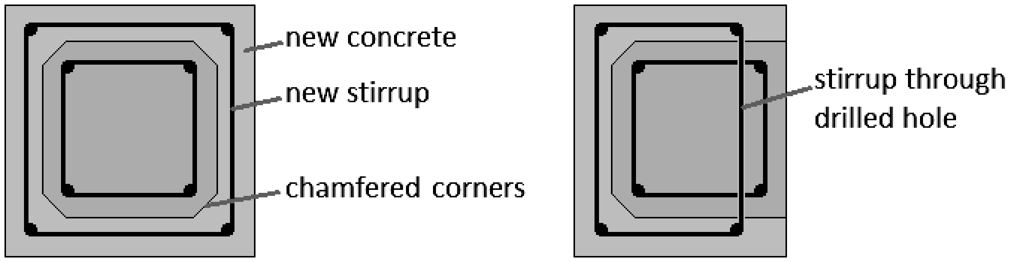

Figure 6.1 Section enlargement of columns with a) full enclosing b) partial enclosing from three sides, after Statens råd för byggnadsforskning (1978).

According to Statens råd för byggnadsforskning (1978) the thickness of the sectional enlargement should not be smaller than 50 mm for regular concrete, while only 30 mm might be sufficient for shotcrete. A disadvantage with cast in-situ concrete is the formwork required to attach the new concrete.

One important aspect when designing section enlargements for columns is how to assume that the new layer is loaded. It is beneficial if the new layer can be loaded directly from above. If it however is assumed that the load cannot go directly down into the new layer, it has to spread out through the original column, see Figure 6.2. The spreading of the load in this case depends on the interaction between the layers. It also means that the strengthened column can resist a larger normal force further down in the column than at the top. This fact might in many cases not be so critical, since the moment often is lower near the top of the column than in the mid section.

Some different approaches can come in question to ensure that the new layer is loaded directly from above; it may sometimes be required to use wedges or hydraulic jacks, but it can also be sufficient to use expanding (shrinkage compensated) concrete. It is however important to remember that it can be more difficult to directly load the new layer if the load comes from a concentrated force, e.g. a column, than if the load comes directly from the overlying slab or beam.

It is also important to account for the loading history. If no special measures are used for unloading of the original structure, such as wedges or hydraulic jacks, only the load that is added after strengthening will be able to be spread to the new layer.

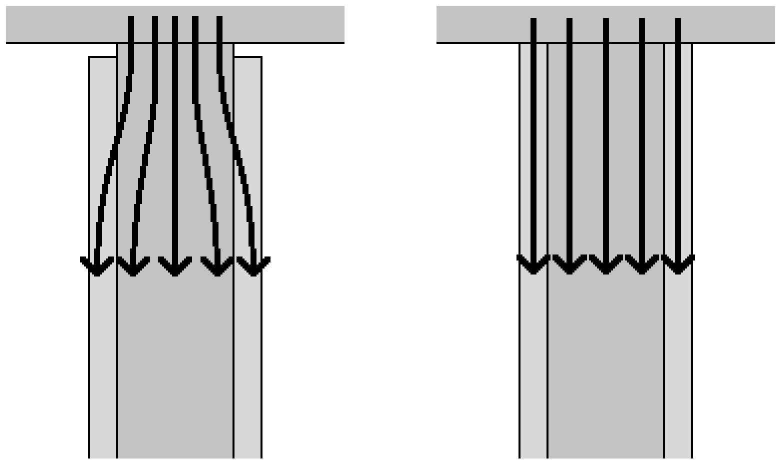

Figure 6.2 Strengthening of columns with section enlargement that is a) loaded through the original column and b) loaded directly from above.

6.1.2 Strengthening against crushing and buckling of columns by adding steel profiles on the sides

Another way to increase the capacity of columns is to add steel profiles on the sides of the column. In the same way as for the case with section enlargement, it is important to consider how the new profiles are loaded. Statens råd för byggnadsforskning (1978) differentiates two different cases, i.e. when the profiles either are intended to work as stiffeners against buckling or as vertically loaded struts. However, the real behaviour will be a combination of the two extremities in the same way as described in Section 6.1.1. Figure 6.3 illustrates how the load either can go through the original column or directly into the profiles from above. The first configuration is mainly intended to prevent buckling while the second is intended to strengthen the capacity with regard to crushing. It is of course beneficial if the profiles are connected both to the slab or beam above and to the column.

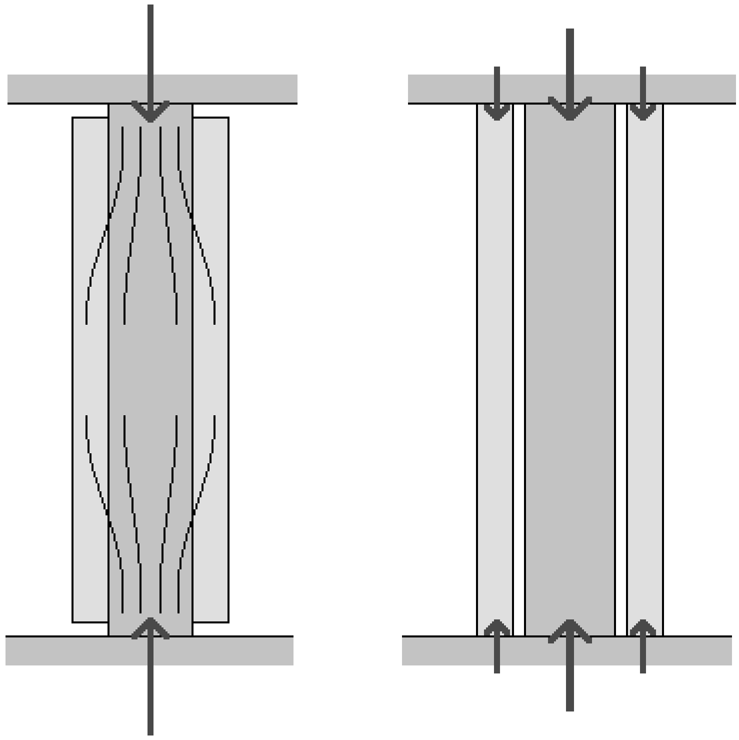

Figure 6.3 Strengthening of columns with steel profiles that a) are loaded through the original column and b) are loaded directly from above.

Strengthening against buckling can be appropriate if the column is fairly slender. Different kinds of steel profiles can in this case be fastened along the sides of the columns by for example adhesives or bolts, Statens råd för byggnadsforskning (1978). It is important that the profiles are connected to the columns at several places to ensure that they deform together as much as possible. On the other hand, the ability to resist normal force is not as critical close to the ends of these slender columns, where the bending moment is low, so it is not necessary to connect them to the slabs/beams above or beneath the column.

For the case where the profiles should be vertically loaded from above, which is especially suitable for stockier columns, the connection to the slabs/beams must be ensured. This can for example be done by hammering in flack wedges beneath the steel profiles or using a hydraulic jack, Statens råd för byggnadsforskning (1978). Using wedges or jacks in this manner also unloads the existing column before the load increase, i.e. the loading history is affected. According to Statens råd för byggnadsforskning, a prestressing force of up to 500 kN can be obtained in each profile by use of wedges. By affecting the loading history in this way, the added material can be utilised better.

6.1.3 Strengthening against crushing of columns by wrapping with CFRP

Additional compressive capacity can be achieved by wrapping the column and thereby exposing the member to constraint forces by confining the concrete with regard to expansion in the radial direction. A load increase is therefore necessary to achieve this effect. The confinement subjects the column to a triaxial stress state, thus increasing the strength and ductility properties of the concrete, ISIS Education Committee (2004). This method can also be used in countries with severe earthquakes to improve the seismic capacity, Cozmanciuc et al. (2009).

Wrapping of columns can be performed with either steel or fibre composites. A steel enclosing will be slightly thicker than an equivalent one made of FRP. A 2-4 mm thick FRP sheet will generate approximately the same capacity as a 5-10 mm steel plate, Sto (A) (2013). However, there are other factors to consider. Steel has for example problems with corrosion and has a rather high density, while at the same time being trickier to install. Using steel as means to confine concrete also gives, due to its mode of action with a distinct yield limit, a more significant increase in ductility rather than in strength, CRC for Construction Innovation (2005).

CFRP is, as described in Section 5.3, quite simple to apply and can be used on any shape and size of columns. To achieve a good result, the fibres should in general be oriented in the circumferential direction of the column, ISIS Education Committee (2004). Maximum utilisation is achieved for circular cross-sections, since the entire cross-section gets a uniform degree of confinement, as seen in Figure 6.4, Cozmanciuc et al. (2009). It is difficult to get an evenly distributed stress state for rectangular cross-sections. This is illustrated in Figure 6.5 where the confinement effect is most pronounced in the corners and the centre. The first thing that is needed with this kind of columns is to smooth the corners. Täljsten et al. (2011) recommends a corner radius of at least 30 mm. A more preferable stress state is however achieved if the shape is changed to a circular or elliptical section by enlargement, Cozmanciuc et al. (2009). However, the need for extra space and the cost of additional work may make this more or less impractical.

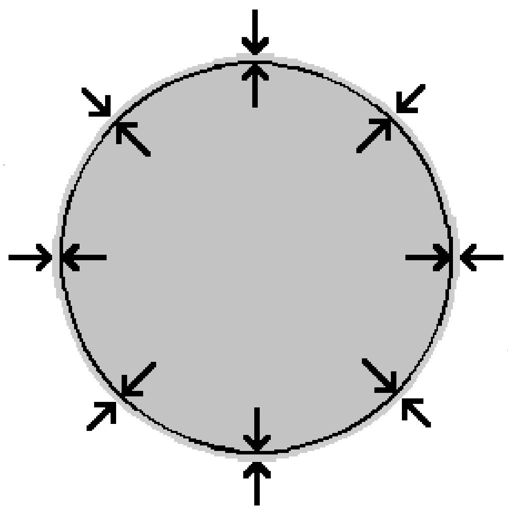

Figure 6.4 Column wrapped by CFRP and thereby subjected to a uniform degree of confinement, after Cozmanciuc et al. (2009).

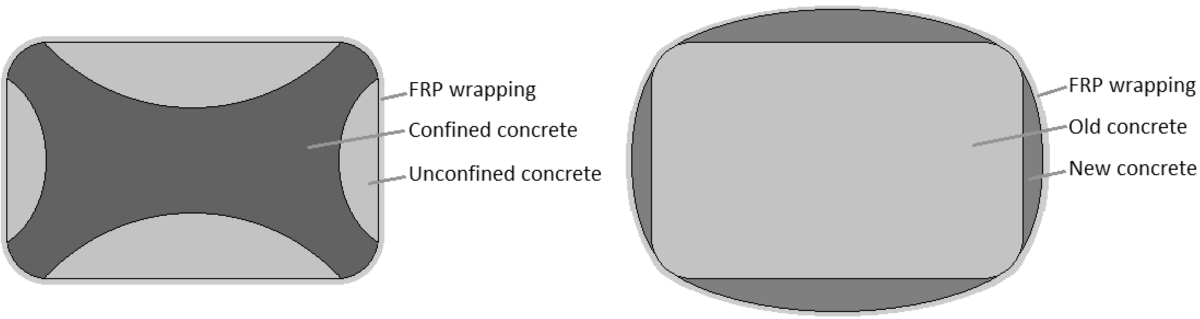

Figure 6.5 Confinement of a) rectangular cross-section and b) rectangular cross-section with a section enlargement that turns it into an elliptical shape, after Cozmanciuc et al. (2009).

The contribution from confinement is greatest when the column is subjected to centric compression and decreases as eccentricity and second order effects increase, Täljsten et al. (2011). Due to this fact, it is advantageous to strengthen columns where the compressive force is dominant and the bending moment only limited. According to Täljsten et al., it is only reasonable to strengthen concrete columns where the sectional capacity is determined by compressive failure as illustrated by the interaction diagram in Figure 6.6, i.e. the effect is better for stockier columns. This is investigated further in Section 7.1.1.

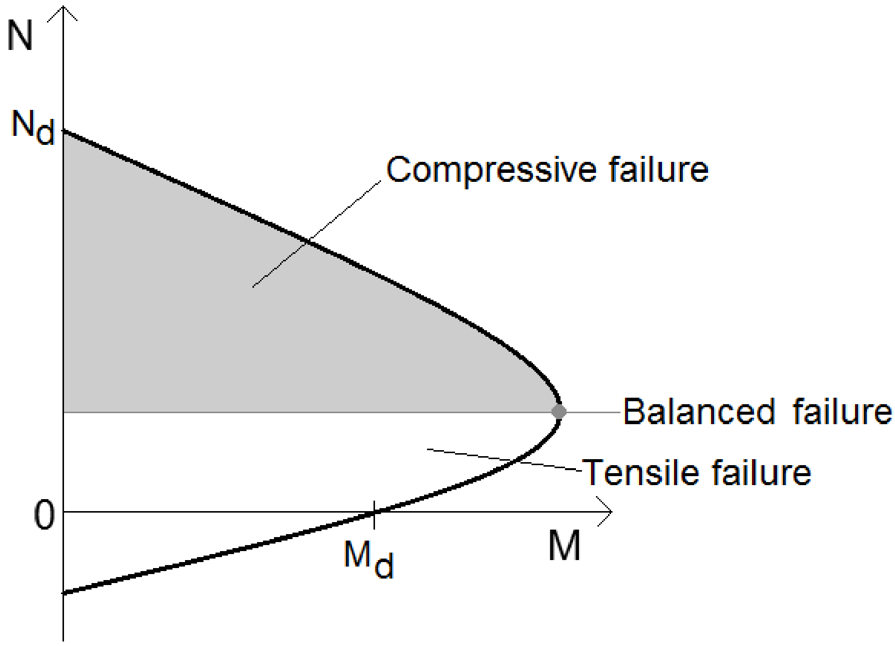

Figure 6.6 Sectional capacity of reinforced concrete sections subjected to a combination of normal force and bending moment. The grey area indicates when it can be motivated to implement confinement stresses, after Täljsten et al. (2011).

Confining concrete with CFRP will also prevent the concrete from deterioration, such as spalling. However, it should also be noted that the carbon fibre composite has to be protected if there is any risk of accidental loading. This might for example be the case for columns located in parking garages. Additional strengthening by other methods can sometimes be required to fulfil the functionality of a column. The column can for example be complemented with extra vertically mounted CFRP to increase the bending capacity. Aramid FRP wrapped around the column can according to Täljsten et al. (2011) also be used to improve the capacity towards car impact loading.

If the entire column is wrapped with FRP, some problems may arise with regard to durability, Täljsten et al. (2011). Such problems may include freeze-thaw cycles leading to frost damages, since enclosed water is prevented from diffusion through the dense composite. This can be solved by a partial coverage of the column. However, a partially wrapped column suffers losses of the confinement effect, since some areas of the column are not confined. In design, this effect can be considered through a reduction factor.

6.2 Strengthening of load-bearing walls

A wall is a member continuously supported from below, mainly acting in compression. If the member is not supported beneath the whole length and instead used to bridge a distance between supports it becomes a deep beam. A wall and a deep beam show important differences when it comes to the way they carry the load.

As mentioned in Chapter 3 it is not uncommon that walls in Sweden have an excess capacity that can be derived from the demands concerning fire and sound proofing. Strengthening of certain wall members may still become necessary for various reasons. Load-bearing walls can in a similar way as for columns experience problems with the compressive strength and/or buckling.

6.2.1 Strengthening against crushing and buckling of walls by section enlargement

As with columns it is not possible to unload an already loaded wall by adding additional concrete on the surfaces of the wall, Statens råd för byggnadsforskning (1978). This is because the existing force cannot be redirected into the new layer from the top or through the old surface. However, as for columns, it is possible to ensure that the added layers are partially utilised when the load is increased. It is however very important that the new concrete is carefully designed and applied so that the load path is continuous downwards through the entire wall.

It is more straightforward to account for an increase of the stability of the wall when new layers of concrete are applied, Statens råd för byggnadsforskning (1978). The increased thickness results in a smaller slenderness, which in turn results in a more stable wall.

6.2.2 Strengthening against crushing and buckling of walls by external struts

Unloading of walls is, due to the circumstances mentioned in the previous section, preferably achieved through external structural members, Statens råd för byggnadsforskning (1978). There are several possible ways to perform such unloading and one way is to use columns that are placed along the wall and topped with a beam. This structure must be placed within the entire loaded area and preferably on both sides of the wall, see Figure 6.7. Available space and size of the load governs the design of the external structure.

Although this method is suitable when unloading of a wall is desired, e.g. due to damage or deterioration, it can be used for strengthening with regard to an increased load as well. The unloading effect can be performed in a similar way as for columns, i.e. with wedges or hydraulic jacks.

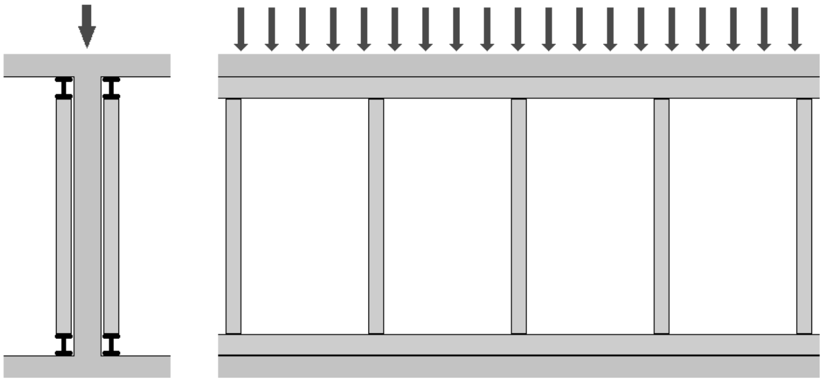

Figure 6.7 Strengthening of walls with external columns and beams, after Statens råd för byggnadsforskning (1978).

6.2.3 Strengthening against buckling of walls by vertical CFRP

If the wall is very slender, it is also possible to strengthen it with glued CFRP that are placed vertically as surface mounted or near-surface mounted strips. The bending moment resistance of the wall is in this way increased. This method can be motivated if the thickness of the wall is restricted and enlargement is not allowed.

6.3 Strengthening of beams

Beams can experience problems concerning both flexural and shear capacity. It is therefore important to understand what resistance that needs to be strengthened, since strengthening of the flexural capacity might not necessarily increase the shear capacity and vice versa. Requirements of the performance in the service state may also be decisive for the design of the beam.

The moment capacity of continuous beams can normally only be increased by strengthening the spans, since the top of the beam often is hard to reach, Statens råd för byggnadsforskning (1978). This method relies on plastic moment redistribution between the support and span sections in the ultimate state. It is then however important to check that the rotational capacity of the beam is sufficient for such redistribution.

It is sometimes possible to change the static behaviour of the entire beam by adding extra supports within its original spans. If the support is stiff enough, this will change the moment distribution and create tension in the top of the beam over the support. It is therefore critical that there is sufficient top reinforcement if such an alternative is chosen. It might however be possible to overcome this with the use of CFRP applied on the top surface of the beam.

As mentioned in Section 5.3 one common problem, when the flexural capacity is strengthened, is how to transfer the tensile forces in the added material to the supports. This can be especially critical for simply supported beams that are strengthened on the tensile side, see Figure 6.8. If good interaction between concrete and the added material is achieved, it is possible that the forces are transferred up through the concrete to the support. However, the presence of shear cracks in the concrete may disrupt this force path. If anchorage of these loads cannot be ensured, some extra measures need to be taken.

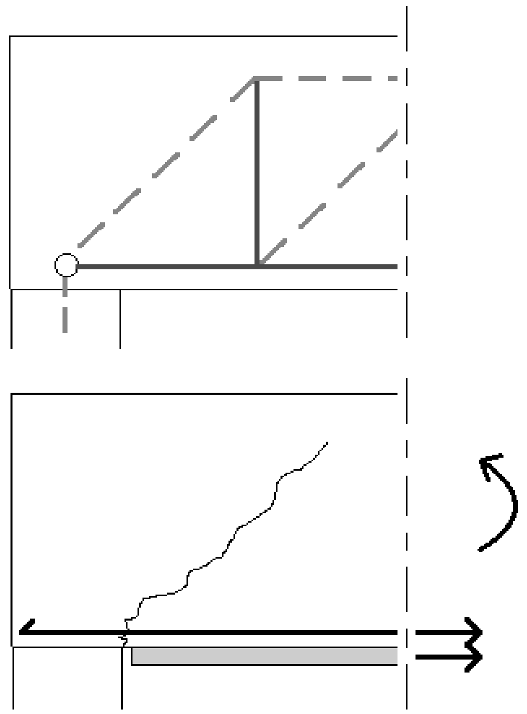

Figure 6.8 Anchorage of tensile members at simple support, a) strut and tie model before strengthening and b) anchorage problem when strengthening on the tensile side.

6.3.1 Strengthening of flexural capacity of beams by section enlargement

If there is enough space, section enlargement might be a possible option to strengthen the moment capacity of beams. An important decision is whether to strengthen the tensile zone or the compressive zone, Statens råd för byggnadsforskning (1978). However, if a section enlargement of beams is performed, the accompanying load increase from the self-weight and the corresponding increase of deflection must also be considered.

If the tensile zone is to be enlarged, extra reinforcement is necessary to achieve a notable increase of resistance. The greater the lever arm, the greater the contribution. This reinforcement is preferably surrounded by vertical stirrups, which in one way or another must be anchored to the existing beam. If the surface of the existing concrete is pre-treated, full interaction between old and new concrete can be assumed, Statens råd för byggnadsforskning (1978). The stirrups will help transferring shear stresses across the interface. These stirrups will also help to keep the longitudinal reinforcement in place before and during casting of the new concrete, which otherwise may be a problem since the tensile side often is located on the bottom side of the beam. Beams strengthened in this manner will according to Statens råd för byggnadsforskning (1978) deviate only slightly from regular beams with an equivalent composition. It should however be noted that the stiffness of the strengthened beam is lower, which will yield a larger deflection than an equivalent beam cast in one step. This is due to the fact that the new layer only is activated by the additional load.

When strengthening the compressive zone of a beam, differential shrinkage must be accounted for, which might lead to cracks in the concrete, Statens råd för byggnadsforskning (1978). These cracks should however not pose any problems in ultimate limit state, but will result in reduced stiffness. Transverse reinforcement across the joint interface might be considered if needed. Design of the shear resistance at the interface between the layers is treated in Section 5.1.1.

6.3.2 Strengthening of flexural capacity of beams by glued CFRP

Flexural strengthening of beams can be achieved by attaching CFRP on its tensile side, with the fibres oriented along the direction of the beam. Both surface mounted and near-surface mounted FRP can be used. They work quite similarly and the main differences are treated in Section 5.3.

The increased moment capacity achieved due to strengthening is followed by loss of ductility, since CFRP show no yielding before rupture, CRC for Construction Innovation (2005). If no anchorage failure occurs, flexural failure of the strengthened beam happens when either a rupture in the FRP arise or by crushing of the concrete. This failure is similar to regular bending failure in reinforced concrete beams. However, in this case the failure will be of a more brittle nature. Crushing of concrete becomes apparent when too much FRP is used. This leads to a very brittle failure and should therefore be avoided in design.

Debonding of the composite leads to a premature failure, i.e. the beam fails before the designed flexural capacity is reached, CRC for Construction Innovation (2005). Bond failures generally occur in the concrete, next to the interface between the concrete and the composite. One way to improve the anchorage of the composite is to glue additional FRP over the composite and up on the sides of the beam, Täljsten et al. (2011).

Normally, concrete beams are designed to fail by bending, but flexural strengthening may imply that failure in shear becomes governing, CRC for Construction Innovation, (2005). To avoid this failure mode, since it has a brittle behaviour, additional CFRP to increase the shear capacity of the beam might be necessary, see Section 6.3.5.

6.3.3 Strengthening of flexural capacity of beams by external prestressing

Strengthening of beams by means of constraint forces is a possible way to increase the flexural rigidity. This can be achieved by post-tensioned tendons (or single strands) that are located outside the cross-section of the beam, either in steel or FRP. The prestressing effect is transferred through contact forces in end-anchors and deviators to the concrete. The added tendon at the tensile side of the beam will also contribute to an increased bending moment capacity. Strengthening of beams with external prestressing is suitable for bridge structures, but can be used in other types of structures as well. This method is basically the same as for slabs and is further described in Section 5.2.1.

The decisions of if and how to apply prestressing depend on the type of beam, but also its availability. Multi-span beams might be especially difficult and need a different configuration of prestressing than single span beams. Space to anchor the tendons and introduce the prestressing force is also needed and can be problematic within existing buildings.

It should be observed that strengthening the bending capacity of beams using external prestressing may not always increase the shear strength, Kiang-Hwee and Tjandra (2003). Hence, the shear capacity of the beam may limit the strengthening effect.

6.3.4 Strengthening of flexural capacity of beams by adding external steel profiles

One way to strengthen beams is to add new steel profiles above, beneath or at the sides of the old concrete beam, Statens råd för byggnadsforskning (1978). If total unloading of the beam for some reason is desired, the steel profile can be placed above the concrete beam with blocks above the supports to ensure that the original beam is fully unloaded, see Figure 6.9a. This method requires that the load is diverted from the original beam to the new profiles.

Another way to perform the strengthening is, according to Statens råd för byggnadsforskning (1978), to place the new beam beneath the old and let the two beams be in contact, see Figure 6.9b. The original beam is in this way still loaded, but the steel profile helps to carry additional load. The deflection of the concrete beam can in this case be decreased by placing wedges between the beams and thereby induce a kind of prestressing, i.e. the old beam is unloaded while the new profile is loaded further.

The third way is to apply steel profiles on the two sides of the beam, as in Figure 6.9c, by the use of bolts through the concrete, Statens råd för byggnadsforskning (1978). If the profiles are fixed intermittently along the whole length, they will bend together with the original beam and increase the stiffness for additional load. According to Statens råd för byggnadsforskning (1978) it is important that the height of the profiles and the level at which they are applied is determined carefully. To fully be able to utilise the profiles' capacity in the ultimate limit state, they must be sufficiently high. Otherwise, the steel in the flanges will not yield before the beam fails.

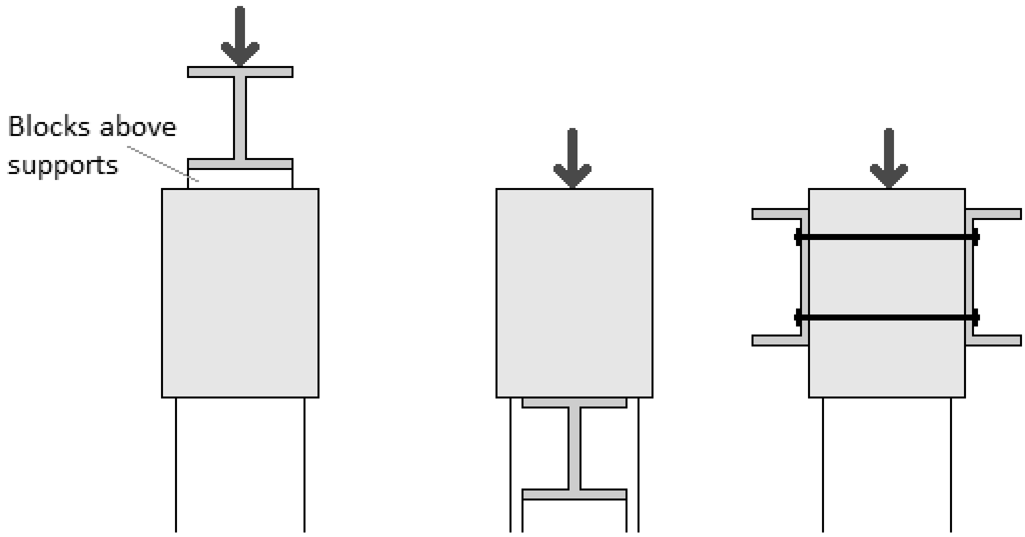

Figure 6.9 Different ways to strengthen beams with steel profiles a) above, b) beneath and c) on the sides of the original beam, after Statens råd för byggnadsforskning (1978).

6.3.5 Strengthening of shear capacity of beams by glued CFRP

Using CFRP to strengthen concrete beams with regard to shear capacity can be performed in different ways by varying the direction of the fibres as well as the type of CFRP, Westerberg (2006). The fibres can for example be strips or sheets mounted on the surface or bars mounted in the concrete cover (near-surface mounted). The main fibre direction is generally oriented perpendicular or at an angle to the main direction of the beam.

There is also the question of how much of the exterior to cover. Different solutions may suit different types of cross-sections and situations. Some possible variations of shear strengthening configurations are illustrated in Figure 6.10. It can be noted that the anchorage of FRP is especially important when strengthening the shear capacity, Sto (B) (2013). The best option is according to CRC for Construction Innovation (2005) to surround the whole section of the beam with CFRP. However, the most common way is to apply CFRP on three sides as U-wraps or bonding CFRP to both vertical sides of the member.

If further anchorage is needed, mechanical fasteners such as bolts can be used, Täljsten et al. (2011). This might be especially necessary for beams with T-sections, unless the web has sufficient height.

Shear strengthening by surface mounted FRP keeps the shear cracks together and functions similarly to internal shear reinforcement, ISIS Education Committee (2004). Furthermore, applying the fibres as U-wraps also improves anchorage of any longitudinal FRP placed underneath to strengthen the flexural resistance. When using U-wraps, the corners of the concrete beam should be rounded to avoid stress concentrations.

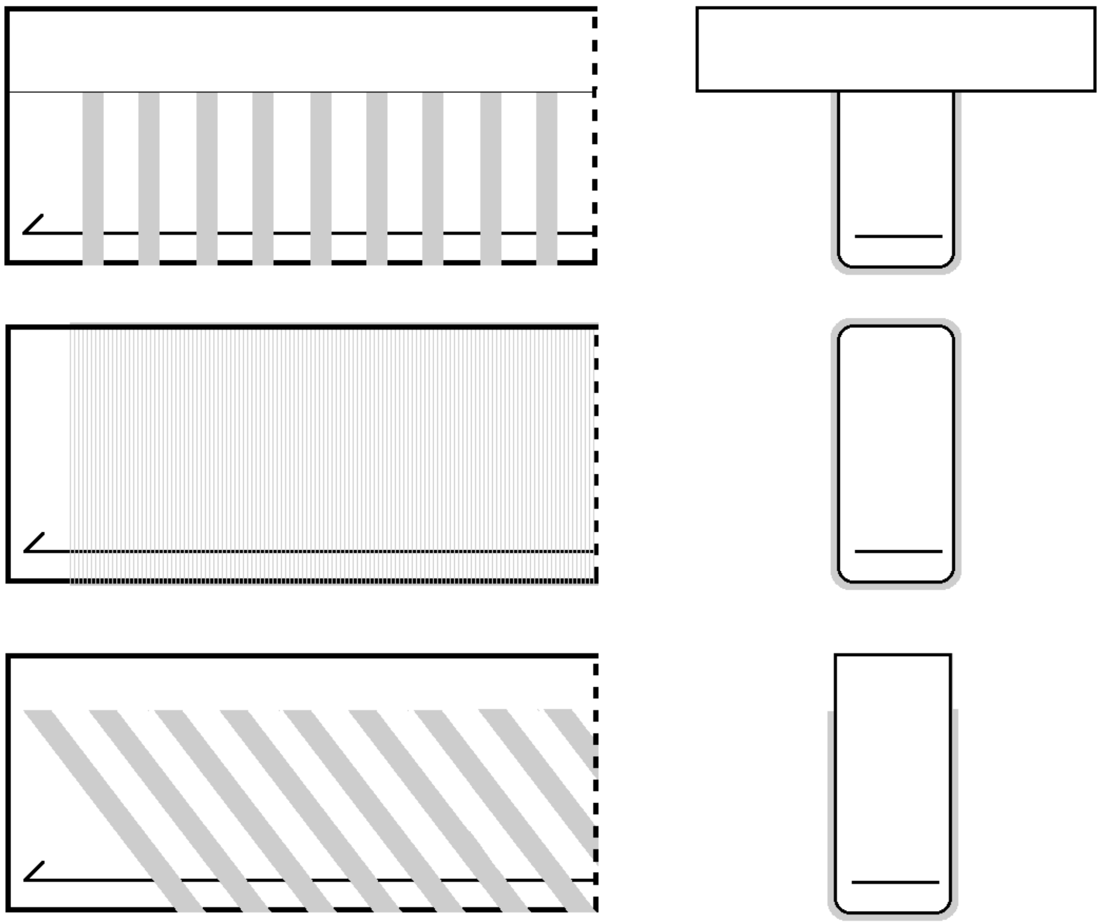

Figure 6.10 Different ways to apply FRP for shear strengthening of beams, a) fibre strips oriented perpendicularly b) fibre sheets oriented perpendicularly c) fibre strips oriented at an angle, after CRC for Construction Innovation (2005).

Shear strengthening by wrapping is less investigated in comparison with flexural strengthening, but according to test performed by Alagusundaramoorthy (2002), the shear capacity of concrete beams can be increased with up to 33 % using unidirectional carbon fibre reinforcement fabric at an angle of 45°. This can be compared with a direction perpendicular to the beam where an average increase of 18 % of the capacity was achieved. Alagusundaramoorthy also concluded that adding FRP with a fibre direction along the beam, on already applied fibres with an angle of 45° will not further increase the shear capacity, but decreases deflection at ultimate load. According to Siddiqui (2009) inclined strips provide a better shear capacity than vertical strips, which seem to be in accordance with Alagusundaramoorthy. The inclined strips also handle the crack propagation more effectively than the perpendicular CFRP-strips. This relation is analogous with the behaviour of regular stirrups.

6.3.6 Strengthening of shear capacity of beams by vertical post-tensioned steel rods

Steel rods can be placed vertically on both sides of the beam to increase the shear capacity, Statens råd för byggnadsforskning (1978). To ensure that the strengthening helps immediately when the load is increased, the bolts can be prestressed by providing the rods with threaded ends and nuts that are tightened. In this way can any existing diagonal shear cracks also be pressed together.

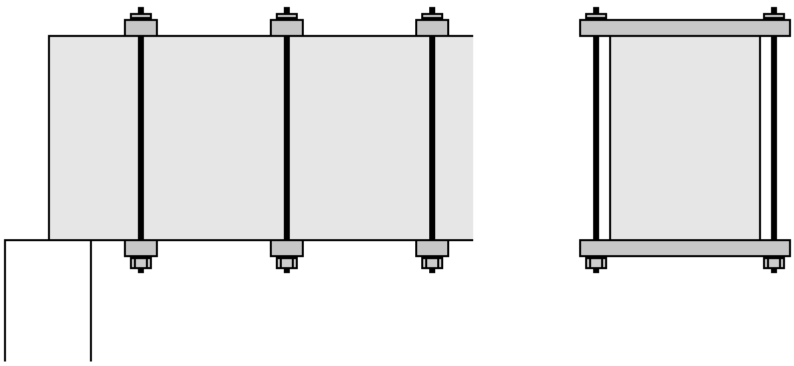

Figure 6.11 Shear strengthening of beams with vertical post-tensioned steel rods.

6.4 Strengthening of slabs

In most storey extension projects the additional load from the new storeys passes through the vertical load-bearing elements without affecting the slabs on the lower floors. However, there may be cases where these slabs are affected. Such situations arise if the old structure is designed so that the vertical load at some level is shifted horizontally through a slab. Another situation, which occurs quite often during storey extensions, is that the roof slab is subjected to new loads that it was not initially designed for. This is mostly due to the fact that it often can be difficult to place the load-bearing elements of the extension directly above the supporting elements of the old structure. An increase of imposed loads on the roof slab can also result in need of strengthening.

For the case with the roof slab it can often be advantageous to just add an extra system of joists on top of the slab to shift the loads to the existing structure. This method was used in several of the studied projects, see Section 3.1. In other situations it can however be motivated to strengthen the slab itself.

The static behaviour of the slab can in the same way as for beams be changed by introducing more supports. The main difference for slabs is that a large support area is needed (either as a wall or as several columns with or without intermittent beams). In contrast to beams, it is rather easy to reach and strengthen the upper side of the slab. This is very favourable if the static behaviour of the slab is changed, e.g. over new supports.

The main problem, concerning the ultimate limit state, when the load is increased for a normal slab is how to take the increased moment, Statens råd för byggnadsforskning (1978). The shear capacity can however be limiting for thick slabs or for normal slabs in the regions close to columns. The ability to keep deflection and crack widths at acceptable levels in the serviceability limit state can also be a problem.

The moment capacity can be limited by either a ductile or a brittle failure. The most common situation for slabs is according to Statens råd för byggnadsforskning (1978) that the reinforcement yields first. For this case it can be better to increase the reinforcement amount. However, if the slab already has a large amount of reinforcement, it is necessary to increase its height instead.

It is important to take the intended behaviour of the slab into consideration when strengthening it. For instance, a two-way slab might not be strengthened in the same manner as a one-way slab. Arrangement of reinforcement and the direction of fibres are two factors that need to be considered. However, two-way slabs have better ability to plastically redistribute the forces within itself, so it might for some cases be sufficient to strengthen the slab in only one direction.

6.4.1 Strengthening of flexural capacity of slabs by section enlargement

One way to increase the moment capacity of slabs is to just add layers of reinforced concrete. Section 5.1.1 describes the importance of interaction between the new and old concrete and how to ensure this. However, since the shear between the layers is relatively small for slabs, it is usually enough to skip anchors and just prepare the surface carefully to enable transfer of shear force and achieve interaction between new and old concrete, Statens råd för byggnadsforskning (1978). Strengthening on the compressive side is more straightforward than strengthening on the tensile side, since horizontal reinforcement is normally not required at the compressive side. It is also easily accessible, since it most often is the top side of the slab that is in compression.

When strengthening is being performed on the compressive side, it has to be checked if the original tensile reinforcement is designed to yield in ultimate limit state. This means that the increase of capacity only can be derived from the increased lever arm that follows the section enlargement. However, the increase in thickness also leads to an increased self-weight that also must be resisted by the slab and the underlying superstructure. This was experienced in the calculation presented in Section 7.2.6, where a very thick section enlargement was required.

The capacity on the tensile side can be increased by adding new reinforcement bars, Statens råd för byggnadsforskning (1978). The bars are attached to the slab and placed at least 10 mm away from the surface. To both transfer the forces and protect the steel, the bars should be embedded in concrete. Since the tensile side of the slab often is facing downwards, it can be difficult to perform regular casting with formwork. The common procedure is, according to Statens råd för byggnadsforskning (1978), to first sandblast and water the concrete and then use shotcrete. One problem with this method might be how to anchor the tensile force over the supports, in the same way as illustrated in Figure 6.8.

6.4.2 Strengthening of flexural and shear capacity of hollow core slabs by filling the cores

Hollow core slabs are quite common in existing buildings in Sweden, which gives the possibility to strengthen the slabs without increasing the thickness. Cuts can be made through the top of the slab so that reinforcement and concrete can be inserted into the cores, Statens råd för byggnadsforskning (1978). The process is illustrated in Figure 6.12. The cores are sometimes used for ventilation and installations, which may conflict this solution.

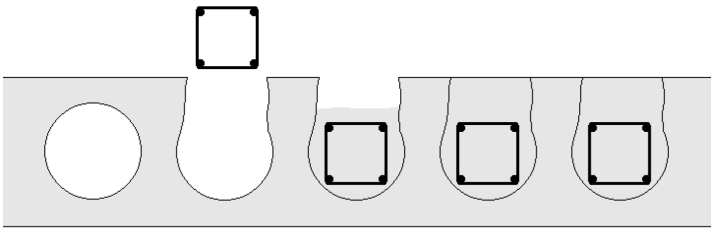

Figure 6.12 Strengthening of hollow core slab by casting reinforced concrete in the cores, after Statens råd för byggnadsforskning (1978).

This method improves the capacity with regard to both moment and shear. When strengthening against bending moment, one disadvantage may however be that the added reinforcement gets short lever arms. The ability to strengthen the shear resistance in this way can on the other hand be quite convenient, since the capacity of this kind of slab more often is limited by shear failure of the webs than for a solid slab.

6.4.3 Strengthening of flexural capacity of slabs by adding prestressing steel reinforcement

A possible way to strengthen a slab is to insert post-tensioned tendons or single strands in drilled holes. Since the method basically is the same as the one for beams, it is described more thoroughly in Section 5.2.1. An important difference from external prestressing of beams is that the steel must be placed across the width of the whole slab and not only on the sides. Inclined holes must therefore be drilled through the slab. A sketch of the method can be seen in Figure 6.13. Note that it also should be possible to strengthen a two-way slab in both directions with this method. It would however require quite extensive construction work.

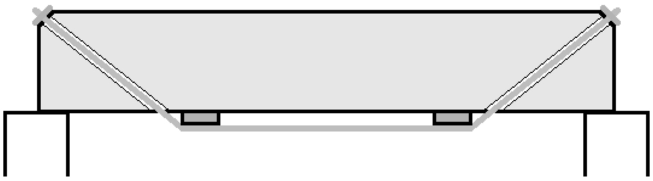

Figure 6.13 Strengthening of slabs by post-tensioned tendons, after Statens råd för byggnadsforskning (1978).

6.4.4 Strengthening of flexural capacity of slabs with glued CFRP

CFRP can also be used for strengthening of slabs and are often glued in strips on the surface as an extra layer of reinforcement, Strong Solutions (2013). The fact that CFRP strips are very strong in relation to their thickness gives this method an advantage when it comes to strengthening inside existing rooms etc.

As shown in Figure 6.14 the FRP-strips can be attached on the tensile side of the slab in a similar way as when beams are strengthened. The direction of the fibres should represent the load-carrying directions of the slab. Crossing of the fibre strips might therefore be a suitable option for two-way slabs.

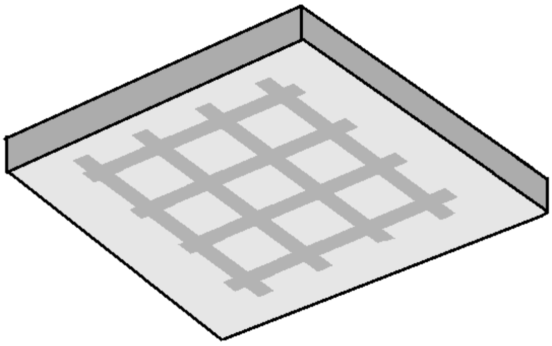

Figure 6.14 Strengthening of a two-way slab with carbon fibre strips.

Strengthening of slabs can also be done by near-surface mounted FRP. The procedure is quite similar as with surface mounted FRP, except that crossing of bars may not be feasible. However, if needed, complementary surface mounted CFRP can be used in the other direction. When strengthening slabs of low or medium concrete strength class with CFRP, the compressive strength of the member can limit the possible increase in flexural resistance, Bonaldo et al. (2008). Bonaldo et al. suggested that a thin section enlargement on the compressive side can overcome this restriction.

6.4.5 Strengthening of shear capacity of slabs by vertical post-tensioned bolts

As mentioned above, the shear forces are usually not as critical as the bending moments for a slab. There might however be cases where the shear capacity needs to be improved. If the flexural resistance is improved by section enlargement, the shear resistance increases as well, Statens råd för byggnadsforskning (1978). In cases where only the shear force is of concern, it might be possible to use bolts that are placed in drilled holes through the slab and connected to a steel plate as shown in Figure 6.15. By tensioning the bolts with nuts, the method can be useful already before the load is increased, Bohlin and Olofsson (2010). The bolts can in this way also press together any existing shear cracks.

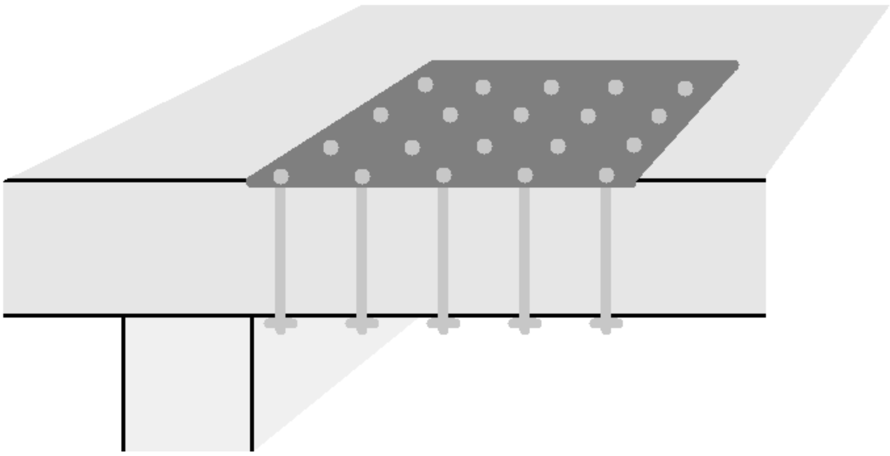

Figure 6.15 Strengthening by bolts anchored in a steel plate, after Statens råd för byggnadsforskning (1978).

In some cases, like for example residential buildings, it may be preferable to avoid affecting the top surface of the slab. In those situations, another way to perform the shear strengthening could be to use undercut anchors instead of the bolts, Bohlin and Olofsson (2010). These anchors are instead only inserted in drilled holes from the lower surface of the slab. The anchors lift the shear force in a similar way as the penetrating bolts, provided that the length of the anchors is carefully calculated so that they reach the compressive node in the truss model. This is illustrated in Figure 6.16. It is also important to design for all possible failure modes between anchors and concrete: yielding of the steel, pull-out failure of the anchor, concrete cone failure at the anchor and concrete splitting failure.

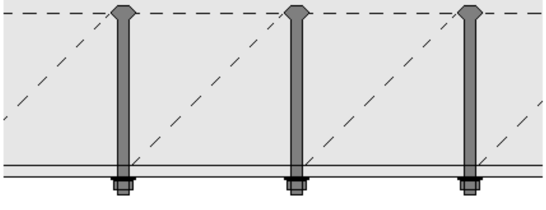

Figure 6.16 Strengthening by undercut anchors shown in a truss model, after Bohlin and Olofsson (2010).

6.4.6 Strengthening of shear capacity of slabs by vertical CFRP bars or strips

It is possible to use CFRP bars that are inserted into the slab from the lower surface in a similar way as undercut anchors, Bohlin and Olofsson (2010). The CFRP bars are not anchored mechanically, but instead glued into the drilled holes with an epoxy. It should be noted that the non-plastic behaviour of CFRP restricts the angle of the struts in the truss model used in design to 45°, which gives a larger need of shear reinforcement.

One way to solve the problem with short anchorage lengths of CFRP bars when installed vertically is to use closed loops. A test performed by Sissakis and Sheikh (2007) showed an increased resistance against punching shear failure with up to 80 % with this method. In this test, holes were drilled through the slab in the area surrounding the load from the column. Then CFRP strips were applied by a method similar to stitching. Figure 6.17 shows a sketch of how the strips were arranged. Observe that more holes and strips were used in the test than what is illustrated in the figure.

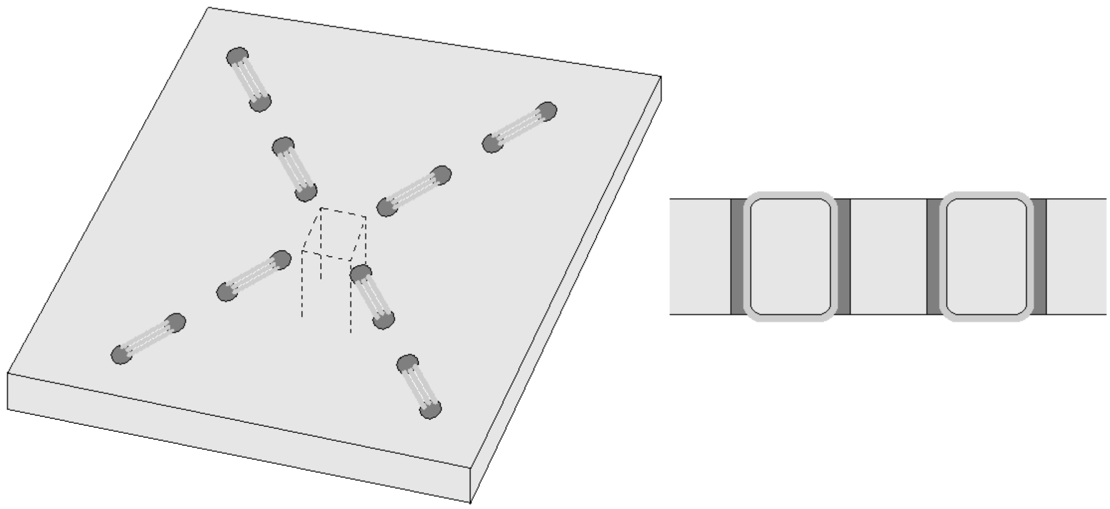

Figure 6.17 Sketch of how CFRP strips can be applied in closed loops. The right figure shows a section through the strengthened slab.

6.5 Strengthening of foundations

As presented earlier, bedrock is very favourable as foundation of heavy structures and will normally not require any strengthening measures for the increased vertical loads that follow a storey extension. This section therefore mainly focuses on strengthening of structures founded on clay, which also is common in Göteborg. Section 2.2 describes the typical geotechnical conditions in Göteborg, while Section 2.3.5 describes different types of existing foundations.

The difference in mode of action for shaft-bearing cohesion piles and end-bearing piles makes them more or less suitable for strengthening. According to Wibom (2013-04-05) it is more difficult to increase the capacity of foundations with end-bearing piles than it is for foundations with cohesion piles. This is due to the fact that the added piles need to deform before the extra capacity can be accounted for. The needed deformation is however obstructed by the already existing piles that may be in risk of crushing. The three-dimensional stress state from the surrounding soil should in theory be able to keep them together, but this cannot be accounted for in the design codes of today, Alén (2013-02-25). Crushing of existing end-bearing piles can be avoided by prestressing the new piles in relation to the existing building or by transferring the loads independently, i.e. without further loading the original piles with the extension. The last method was used in Studio 57, see Section 3.1.10.

The problem with failure of the existing piles is smaller for foundations with shaft bearing piles than for foundations with end-bearing piles, Wibom (2013-04-05). This is because the increased load mainly magnifies the settlements. These settlements can be reduced by use of additional piles. As long as these settlements are within control and evenly distributed beneath the building, they will generally not pose a big problem.

6.5.1 Strengthening with steel tube piles

According to Alén (2013-02-25) the use of hollow steel piles is the most common method when strengthening existing foundations on clay. The piles are used on both sides of the load-bearing walls or columns, which means that even if an outer wall is to be strengthened, piles have to be driven from the inside of the building. Hollow steel piles are mainly end-bearing so if shaft bearing is to be favoured due to larger thickness of the clay, winged piles (see Section 6.5.3) are to be preferred instead, Besab (A) (2012).

The piles consist of several elements that can vary in length from 1 to 6 m and be adapted to the specific conditions of the building. The machine used for indoor pile driving is rather small enabling use even in case of low roof heights, Besab (A) (2012). The elements are continuously screwed together as the piles are vibrated further down through the soil, creating a continuous pile at the site. The hole in the pile simplifies penetration through any possible dense filling material and soil layers. When the piles have been driven to solid ground, the load is transferred from the wall to the piles through a lintel. The tubes are sometimes also filled with reinforcement and concrete. A sketch of this kind of strengthening can be seen in Figure 6.18.

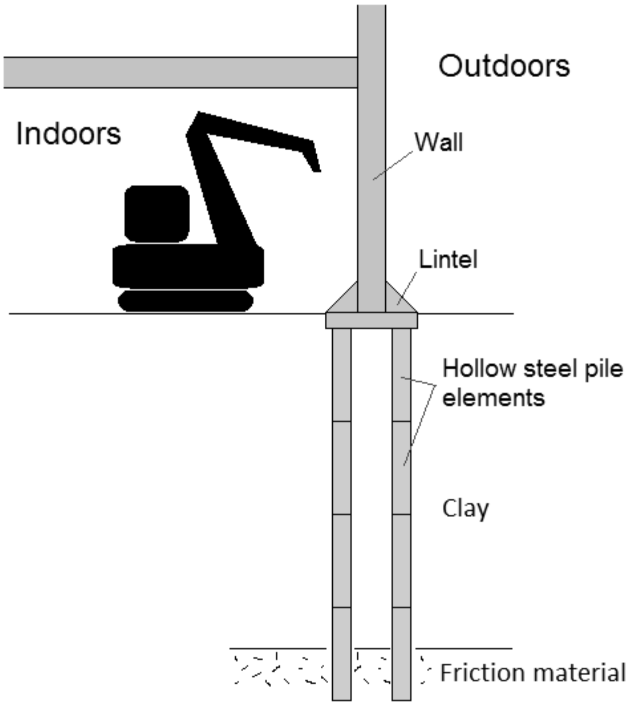

Figure 6.18 Strengthening of foundation with hollow steel piles.

Hollow steel piles can also be drilled and anchored into the bedrock. A steel tube with thick walls is in this case drilled through the ground and into the bedrock until a desired depth is reached, Ground Energy (2013). The tube is then filled with concrete and, optionally, reinforcement. If a corrosion resistant pile is used, this method can be considered proof of settlements. This approach is also useful if the bedrock is heavily inclined and when the surrounding environment is sensitive to vibrations or noise. Anchoring the piles into the bedrock also provides better capacity with regard to tensile loading.

6.5.2 Strengthening with steel core piles

If the depth to solid layers is rather small, steel core piles may be more suitable than steel tube piles, Alén (2013-02-25). The steel pile is in these cases just inserted into a pre-drilled hole. The piles can also be anchored in the bedrock and thereby be able to resist quite high tensile forces. This property was used in some of the studied projects to be able to take the new horizontal loads. One disadvantage with this method is however that it is more difficult to splice elements and still be able to account for the tensile capacity. Alén however said that tests had been made with a conical threaded connection.

6.5.3 Strengthening with winged steel piles

If the thickness of the clay is large, i.e. the distance to frictional material or bedrock is big, winged steel piles can be used instead of steel tubes, Alén (2013-02-25). The wings of the piles give them better properties with regard to cohesion, so the winged piles do not need to be driven all the way down to the friction material. In the same way as for the hollow steel piles, winged piles can be ordered in short elements so that they can be driven into the soil from the basement, Besab (A) (2012). This method was used to strengthen the foundation of the building called Odin, see Section 3.1.12.

6.5.4 Strengthening with soil injection

If the upper soil layer consists of friction material instead of clay, it might be appropriate to inject the soil with cement through high pressure, Alén (2013-02-25). Thereby something similar to concrete is created where the sand and stone particles act as aggregates. For this method to be possible, it is required that the soil has sufficient porosity to enable that the cement properly encloses the friction material.

7 Comparison of some strengthening methods by calculations

The need to strengthen slabs and columns came up quite often during the interviews presented in Chapter 3. Since the opinions about which strengthening method to use are varying, it was decided to investigate some of the methods more thoroughly by calculations. It would have been interesting to investigate all approaches for all types of structural members, but the time limit restricted the calculations to treat the resistance with regard to normal force and buckling for columns and flexural resistance for slabs. The calculations can be found in Appendix D and E and treat the different strengthening methods presented in Chapter 6. Below follows a description of how the calculations were carried out and how the results should be interpreted.

7.1 Strengthening the axial capacity of columns

As described in Chapter 6 there are several ways to strengthen columns that are loaded with a normal force. The calculations for strengthening with load-bearing steel profiles, vertically mounted steel and CFRP plates, section enlargement and CFRP wrapping are presented in Appendix D.

It is assumed that the columns only are loaded by a centric normal force that should be increased due to storey extension. However, this normal force also results in a moment due to imperfections and unintended inclination. Therefore, the analysis of the column must show that the strengthened column can withstand the combination of normal force and moment. The magnitude of this moment varies along the column with its maximum in the mid span. It should be noted that it in reality can be quite common that the load on the column has an intended eccentricity due to uneven spans etc. The moment can for these cases be significantly larger than assumed in the calculations. Horizontal loads can also affect the applied moment on columns.

7.1.1 The studied columns

Two different fictitious columns with quadratic sections were chosen to be able to compare the methods better. The first column was inspired by one of those that were strengthened in Scandic Rubinen and is therefore 4.3 m high with a square section where the sides are 710 mm wide. The second column has the same height, while the sides only are 250 mm wide, which results in a much more slender column. This more slender column was chosen in order to investigate if the methods that were unsuitable for a stockier column could be better suited for this specific case. To better examine the differences between rectangular and circular columns when strengthening with CFRP wrapping, a circular version of the large column was also created. This column was designed to have the same capacity as the large quadratic column before strengthening and therefore received a diameter of 800 mm. The calculations of the capacity of the different columns before strengthening are presented in Parts 1, 7 and 9 respectively in Appendix D. The different column sections are shown in Figure 7.1.

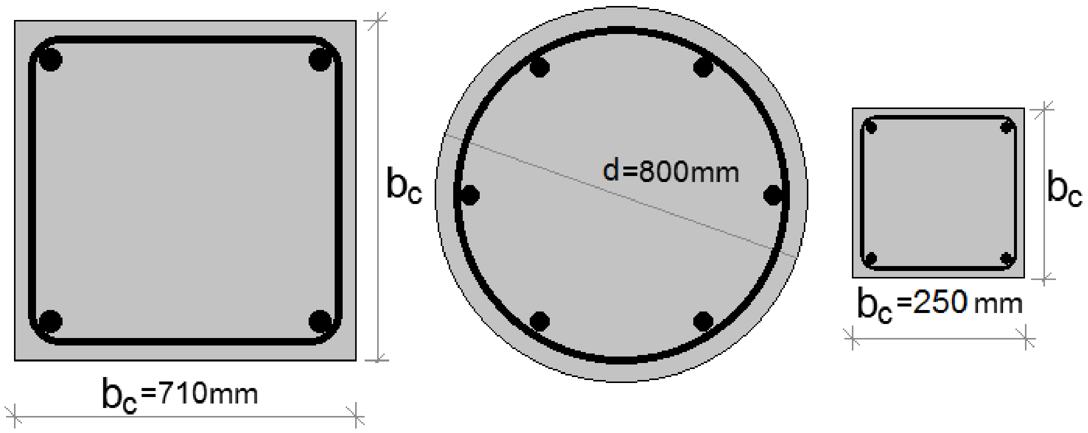

Figure 7.1 Cross-sections of the studied columns.

The difference in behaviour between the two rectangular columns before strengthening is visualised in Figure 7.2. The vertical axes in these diagrams show the relationship between the applied normal force and the axial compressive resistance in case of pure compression. The horizontal axes show the relationship between the applied moment and the moment capacity in case of pure bending. The broken lines show the capacities for the critical mid-section for different combinations of interacting normal force and moment and the circles represent the combination at which failure occurs for the studied columns. Since the column with a side length of 250 mm is much more slender than the large one, the applied normal force gives a relatively higher second order moment. This is why the circle is placed further to the right in Figure 7.2b.

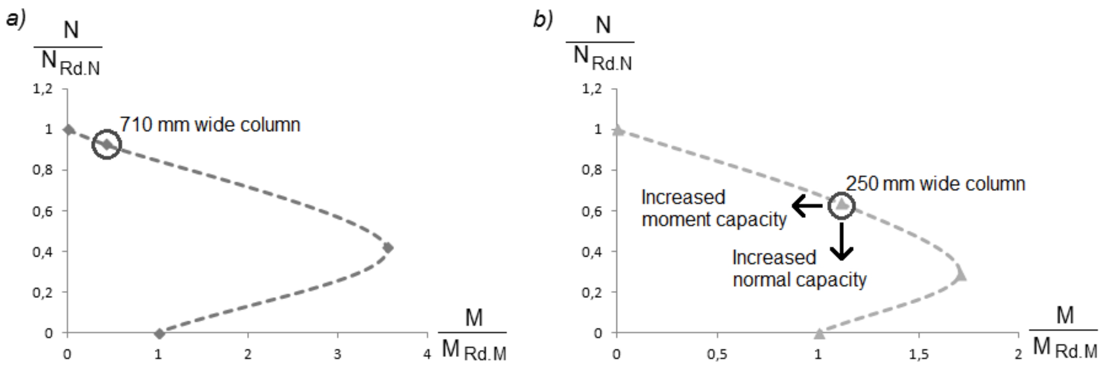

Figure 7.2 Interaction diagrams showing combinations of normal compressive force and moment that result in failure in the mid-section for the two studied rectangular columns. The broken lines are just approximated between the calculation points. Observe that the curves vary for different sections of the columns.

The diagrams in Figure 7.2 are very important to consider when different strengthening methods are chosen and evaluated. They illustrate that it is difficult to strengthen a stockier column by just increasing the moment capacity. The applied load can never be increased above the value 1.0 on the y-axis in Figure 7.2. For the column in Figure 7.2a, this increase is very small. Therefore, the compressive capacity must be increased for this case (so that the ratio is decreased). For a more slender column (or a column that is subjected to an external moment in addition to the effect of imperfections) there is on the other hand more room for an increase of the compressive capacity. Figure 7.2b shows that only 64 % of the compressive capacity is utilised when the column fails. This means that a higher capacity can be reached by just increasing the moment resistance.

7.1.2 Strengthening with load-bearing steel profiles on the sides of the column

The first strengthening method investigated is the one illustrated in Figure 7.3. As described in Section 6.1.2 steel profiles are added to the sides of the column. In this case, it was assumed that the profiles are of type HEB180 and that they are prestressed by wedges and thereafter connected to the large concrete column with the square section. This method is inspired by the one used during the project at Scandic Rubinen, see Section 3.1.3. However, the design assumptions may not be the same. The calculations can be found in Part 2 in Appendix D.

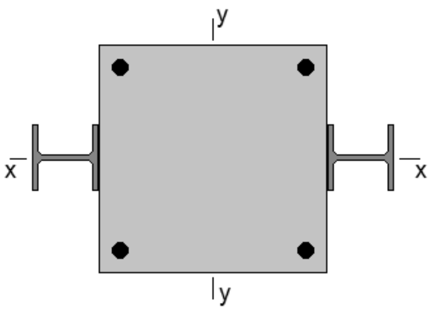

Figure 7.3 Strengthening with load-bearing steel profiles on the sides of the column.

To be able to investigate the strain difference between the original column and the steel profiles, it was assumed that the quasi-permanent load acts on the column when the profiles are added. By assuming a prestressing force of 500 kN on each profile, the difference in strain between the concrete and the steel could be calculated. The assumed magnitude of the prestressing force comes from Statens råd för byggnadsforskning (1978), where it is stated that a force of this magnitude can be reached by use of wedges that are hammered in beneath the profile. In practical applications this should be verified by measurements.

When the profiles have been connected to the original column, the load can be increased and with this assumption the new capacity in the ultimate limit state was calculated. As can be seen in the calculations, the bending stiffnesses of the profiles are only accounted for when the second order moment for the increased part of the load is calculated. This is due to the fact that the column already has a certain deflection when the profiles are applied. When the resistance of the section is calculated, it is important to ensure that the original strain difference between the concrete and the steel still is accounted for. In Figure 7.4 the model used for sectional analysis of the section is illustrated. The difference in strain between concrete and steel is visible as the difference between the lighter and darker part of the strain diagram. The calculations show that the whole sections of the steel profiles have reached yielding at the ultimate load.

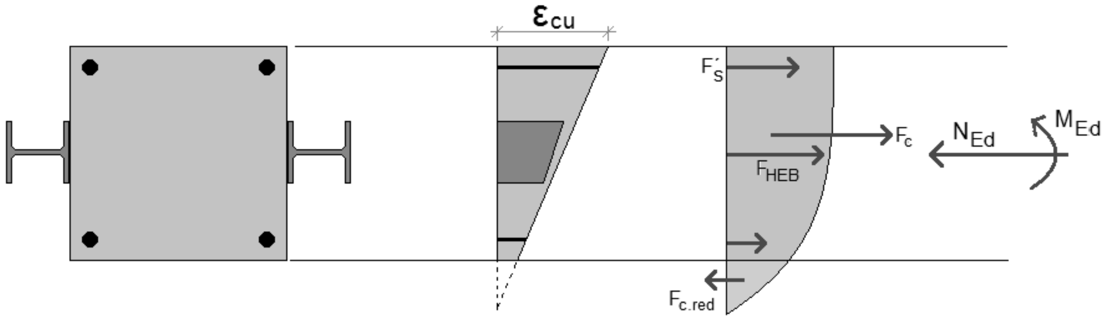

Figure 7.4 Model for sectional analysis of column section with vertically loaded prestressed steel profiles.

The results of the analysis show that the ultimate load in this case could be increased with about 28 % due to the strengthening. It is however important to ensure that the profiles are vertically loaded. The possible increase would otherwise have been very small since these profiles do not add significantly to the bending stiffness in the weak direction.

To be able to investigate the effect of prestressing of the profiles, the calculations were performed using the same method but with the assumption that no prestressing force is added to the profiles. It was however still assumed that the profiles could be loaded from above. These calculations are presented in Part 2b in Appendix D.

The calculations in Part 2b show that parts of the sections of the profiles yielded for the ultimate load. The contribution to the moment resistance therefore needed to be reduced for these parts (since elastic behaviour was assumed). This is illustrated in Figure 7.5. According to the calculations in Part 2b the ultimate load could be increased with 26 % when the steel profiles were not prestressed. This can be compared with 28 % when they were prestressed with 500 kN. It should however be noted that it was assumed that the profiles are loaded directly when the normal force is increased.

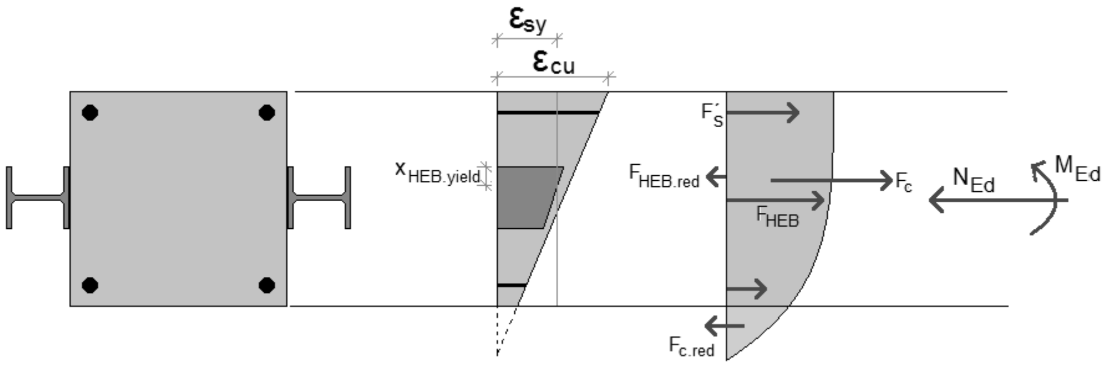

Figure 7.5 Model for sectional analysis of column section with vertically loaded steel profiles (without prestressing).

It was also investigated how big influence the bending interaction between the profiles and the column has. The calculations were therefore performed again, but with the assumption that the steel profiles act as individual columns. The load increase could thereby be calculated as the load that the profiles themselves can resist before they fail due to buckling. These calculations are presented in Part 2c in Appendix D. It was also verified that the strain difference in the concrete column between the quasi-permanent load and the ultimate load was big enough to be able to load the steel profiles as assumed. The result of the calculations in Part 2c shows that the load only could be increased with about 14 % if no bending interaction could be accounted for.

7.1.3 Strengthening with vertically mounted steel plates

The intention with these calculations was to show the influence of steel profiles that are applied to the sides of the column and mainly contribute to the bending moment capacity. The steel will in reality, as described in Section 6.1.2, also contribute to the normal force resistance (in the parts of the column where they are in interaction with the column) but this influence was neglected in the calculations. To get better moment resistance in the weak direction the HEB-profiles from the previous section were replaced by steel plates that are connected to all four sides of the column. Figure 7.6 shows the section of the concrete column with the attached steel plates.

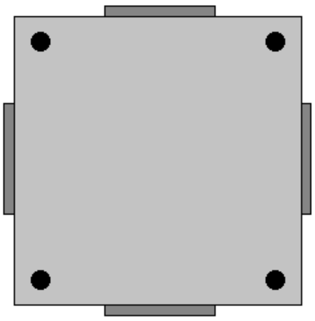

Figure 7.6 Column section strengthened by steel plates.

In the first stage it was tested if this method could be used to strengthen the large rectangular column. These calculations can be found in Part 3 in Appendix D. The same method was thereafter used to strengthen the more slender column. The calculations for the second column are presented in Part 10 in Appendix D.

To better utilise the increased moment capacity it was assumed that the columns are braced (forced to vertical alignment) before the steel plates are attached. The curvature is in this way assumed to be zero at the time of application, i.e. the strain is constant over the column section. It was also assumed that the whole load increase is applied before the bracing is removed, so that only the second order moment is applied after the steel plates are connected. In this way a sort of prestressing of the plates is created. All four steel plates were accounted for when the bending stiffness, and thereby also the second order moment, was calculated. However, during the sectional analysis, only the steel plate that is in tension was considered since it is more difficult to ensure that a compressed plate can be utilised.

During the calculations it was soon found out that this type of strengthening is unsuitable for the larger column and other columns with similar response as the one shown in Figure 7.2a. Even if large steel plates were chosen (600 mm x 10 mm), the load could only be increased with about 1 %.

This method was on the other hand more suitable for the slender column. For this case the increase in moment capacity enables a further increase in resistance with regard to normal force, refer to Figure 7.2b. Four plates of 75 mm x 6 mm gave an additional capacity of 28 %.

7.1.4 Strengthening with vertically mounted CFRP laminates

One interesting thing to investigate is the difference between steel plates and CFRP laminates when used to strengthen the resistance with regard to bending moment. The calculations described in Section 7.1.3 were therefore repeated with vertical, 1.4 mm thick, CFRP laminates (instead of steel plates) glued onto the surfaces of the column. Since it based on the calculations presented in Section 7.1.3 was concluded that strengthening of the moment resistance only can be relevant for slender columns, it was decided to only perform these calculations for the column with the small section. The calculations for this method are presented in Part 11 in Appendix D.

The results of the calculations show that this strengthening method is inappropriate for the studied column. The same load increase as before was sought, i.e. 28 %, but even if the whole surface of the column was covered by laminates, sufficient bending moment resistance could not be reached. This is because the laminates only are 1.4 mm thick compared to 10 mm for the chosen steel plates. The modulus of elasticity for the chosen FRP material is even a bit lower than for steel, so the forces in the laminates become rather small. The big advantage with CFRP is the high strength (see Figure 5.4), but since the strain at the edge of the column section is very small for the ultimate load, this benefit cannot be utilised. Strengthening with this kind of vertical CFRP laminates is therefore only beneficial for very slender columns or columns where an external bending moment is added, e.g. through intended eccentricity of the axial load.

One way to improve the behaviour for this case could be to use CFRP with higher modulus of elasticity, as illustrated in Figure 5.4. It might however be quite difficult to find a manufacturer that provides laminates with such a property. On the other hand, many manufacturers provide weaves with significantly higher modulus. The applicability of these was however not investigated.

7.1.5 Strengthening with section enlargement

Calculations for strengthening with sectional enlargement (see Section 6.1.1) were performed on the two square-shaped columns. In the first stage it was assumed that the new layer can be used to resist vertical load. These calculations are available in Appendix D, Part 4.

It was assumed that four new ϕ20 reinforcement bars were added along with a new layer of concrete that encases the whole section. The thickness of the new layer was iterated until the same increase of the load as in Part 2 in Appendix D was gained, i.e. 28 %. The enlarged section is illustrated in Figure 7.7. The same concrete strength class was for simplicity used for the new layer as in the original column.

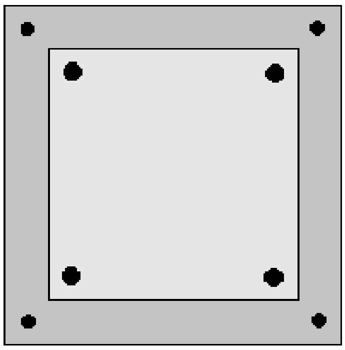

Figure 7.7 Column section after enlargement.

The strain in the unstrengthened column under the quasi-permanent load was initially calculated. The corresponding curvature before strengthening was then used to calculate the eccentricity in the critical section. This eccentricity was later used when the first order moment was calculated.

The different concrete creep coefficients for the new and the old layers was treated by calculating the creep coefficients for a homogenous column loaded at the age of 40 years and 28 days respectively. From these two values an average creep coefficient was interpolated, taking the amounts of new and old concrete into consideration. The nominal bending stiffness and the second order moment could thereafter be calculated.

The resistance of the critical section was calculated in accordance with the model shown in Figure 7.8. It was assumed that the new layer of concrete also helps to resist the combination of normal force and bending moment. The strain difference between the two layers was however regarded since the original column already had an initial strain when the new layer was cast. Figure 7.8 illustrates how the different strains were accounted for by adding together the stress blocks and then subtracting the central part of the block for the new concrete.

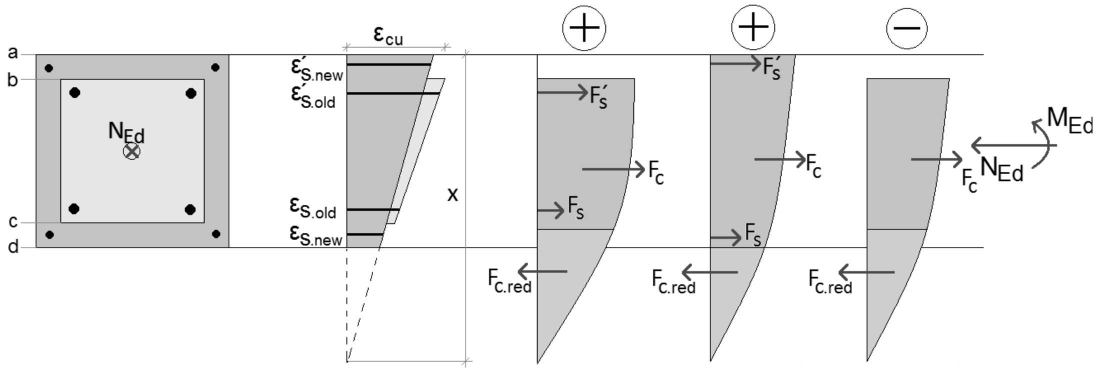

Figure 7.8 Model for sectional analysis of the column with enlarged section.

It was found out that if the same increase of the axial resistance as in Part 2 in the calculations is desired, i.e. 28 %, the thickness of the additional layer only needs to be 40 mm (corresponding to 0.52 m^3), provided that the whole layer interacts in bending. It should however be noted that it in reality is quite inappropriate to only add such a small layer. As described in Section 5.1 Statens råd för byggnadsforskning (1978) claimed that the new layer never should be less than 50 mm.

When the calculations for this strengthening method were finished, it was also investigated what the difference would be if the new layer was assumed to only contribute to an increased bending stiffness and not to the resistance of the section. The calculations for this approach are presented in Part 5 for the large column and in Part 13 for the small column.

The result of this study shows that the load could almost not be increased at all for the column with the larger section (Part 5). This can be explained by the diagram in Figure 7.2a. Since the load on the larger column cannot be increased much without improving the resistance with regard to normal force, an increased nominal bending stiffness cannot give any large effect by itself. It was however possible to account for an increased axial resistance in the more sender column (Part 13). The increased bending stiffness is in this case enough to achieve the desired capacity. When neglecting the new layer in the sectional analysis, the layer needed to be 65 mm thick if the load should be increased with 28 %. However, as described in Section 6.1.1, it would in reality be unnecessarily harsh to totally neglect the new layer in the sectional analysis since the load will spread out in the new layer even if it is not directly loaded from above.

7.1.6 Strengthening with CFRP wrapping

Calculations of strengthening with CFRP wrapping in the circumferential direction were carried out for three different columns. Strengthening of the large quadratic column is treated in Part 6 in Appendix D, while the calculations for the more slender quadratic column can be found in Part 11. Since the effect of this kind of strengthening is dependent on the shape of the column section, calculations were also carried out for a circular column. These calculations are located in Part 8.

The calculations are based on the principles in Täljsten et al. (2011) and are mainly based upon the fact that the concrete can take higher stresses if it is subjected to a triaxial stress state. To get better stress distributions it was assumed that the corners of the square columns are smoothened.

A carbon fibre weave with a layer thickness of 0.117 mm was assumed with varying number of layers. For the square 710 mm wide column it was found that 11 layers (ca. 135 m^2) were needed to be able to increase the axial resistance with 28 %. To get the same increase of capacity for the circular column only 5 layers (ca. 54 m^2) were needed. For the more slender square column not more than 5 layers (ca. 22 m^2) could be added since the concrete, which is subjected to a triaxial stress state, otherwise would crush. This limitation of the amount of CFRP-sheets resulted in that the load on the slender column only could be increased with 12 %. It can however be noted that it would have been hard to reach a load increase of 28 % for the slender column, even if this limit had been ignored. This is due to the fact that the bending moment increases drastically when the normal force is increased. The CFRP wrapping does not directly increase the moment resistance.

The results showed that strengthening with CFRP wrapping is best suited for columns mainly loaded by compression, i.e. with a behaviour similar to the one presented in Figure 7.2a. It is also much better to strengthen circular columns than square-shaped columns in this way.

7.1.7 Summary and conclusions

Table 7.1 and Table 7.2 below summarises the investigated strengthening methods used to strengthen the two different columns (with large or small cross-section). In all cases, the aim was to be able to gain an increase of the axial resistance equal to the one that was reached for the method presented in Section 7.1.2, i.e. 28 %. However, this desired capacity could not be gained with all methods.

For the larger column, the methods that increase the resistance with regard to the normal force itself are the most effective. The methods that instead add more to the bending stiffness, i.e. steel plates and section enlargement where no load is assumed to go through the new layer, are unsuitable. The big difference between the rectangular and circular columns when strengthening with CFRP wrapping is another important issue. The material usage is much lower for the circular column than the rectangular.

Table 7.1 Summary of the strengthening methods used to strengthen the column with larger section.

Strengthening method	Load increase	Material used
2. HEB-profiles (prestressing and interaction)	28 %	- 8.6 m HEB180
2b. HEB-profiles (interaction)	26 %	- 8.6 m HEB180
2c. HEB-profiles (detached)	14 %	- 8.6 m HEB180
3. Steel plates	Nothing	
4. Section enlargement (loaded)	28 %	- 0.52 m^3 Concrete - 17.2 m reinf. bars ϕ 20 mm
5. Section enlargement (assumed only to increase the bending stiffness)	Nothing	
6. CFRP wrapping (square)	28 %	- 135 m^2 S&P C-Sheet 240 (200g/m^2) - Glue etc.
8. CFRP Wrapping (circular)	28 %	- 54 m^2 S&P C-Sheet 240 (200 g/m^2) - Glue etc.

The biggest difference with the smaller column is that the methods with steel plates connected to the surfaces and section enlargement that is not accounted for in the sectional analysis both were found to be more suitable. This is due to the fact that this column is more dependent on the bending stiffness. Another interesting result is that it in this case is better to use steel plates than CFRP laminates. The laminates would however be better suited if an external moment acts on the column (CFRP need larger tensile strains to be more suitable than steel). The low load increase for the wrapping can be derived from the fact that the concrete strain reaches a critical limit before the full effect of wrapping can be utilised. Hence, wrapping with CFRP is better suited for strengthening of stockier columns.

Table 7.2 Summary of the strengthening methods used to strengthen the column with smaller section.

Strengthening method	Load increase	Material used
10. Steel plates	28 %	- 13.2 m steel plate 6x75 mm^2
11. CFRP laminates	Not recommended	
12. CFRP wrapping (rectangular)	12 %	- 22 m^2 S&P C-Sheet 240 (200 g/m^2) - Glue etc.
13. Section enlargement (assumed only to increase the bending stiffness)	28 %	- 0.35 m^3 concrete - 17.2 m reinf. bars ϕ 16 mm

7.2 Strengthening the flexural capacity of simply supported slab

As described in Chapter 3 the flexural capacity of the roof slab had to be increased in several of the studied storey extension projects. There are, as described in Section 6.4, several different ways to strengthen slabs. It was therefore considered to be relevant to compare the different methods further through calculations. In Appendix E, the calculations for strengthening with surface mounted CFRP strips, near-surface mounted CFRP bars, new steel beams on top of the slab, prestressing strands and section enlargement on the compressive side are presented. The procedures and results of the calculations are however presented and commented in this section.

In most of the studied projects the slab primarily needed to be strengthened due to the fact that the new walls could not be placed directly above the vertical members in the original structure. However, it is also quite common that the slab cannot take the increased imposed loads. It was therefore, for simplicity, decided to perform the calculations with regard to an increased evenly distributed load. Another delimitation is that the calculations only consider a simply supported one-way slab.

7.2.1 The studied slab

As described in the previous section it was decided to perform the calculations on a simply supported one-way slab. The chosen slab has a span length of 6 m and a height of 160 mm. The tensile reinforcement consists of regular reinforcement bars with a diameter of 10 mm. The concrete is of strength class C40/50. In the first step (Part 1 in Appendix E) the reinforcement amount in the mid-section of the original slab was designed for an assumed design load in the ultimate limit state. The calculations were performed in a simplified manner and only consider the ability to resist bending moment from the self-weight of the slab and the original variable load (which was chosen to 2 kN/m^2). The results showed that the reinforcement needed to be placed with a spacing of 110 mm.

Part 2 in Appendix E contains calculations for the original slab in the service state before strengthening. This analysis was needed to find the strains in the slab at the time of strengthening and was carried out in accordance to Appendix A in Täljsten et al. (2011). By first using the frequent load combination together with a roughly assumed effective creep coefficient of 2.0, it was found out that the section should be treated as flexurally cracked (state II). The strains at the time of strengthening could thereafter be calculated by assuming that the imposed loads can be removed so that only the self-weight of the slab is active, thus reducing the strains and achieving a more favourable loading case.

7.2.2 Strengthening with surface mounted CFRP laminates

The first strengthening method investigated for the simply supported slab was the one where CFRP laminates in the shape of strips are glued to the tensile surface of the slab. This method is described in Sections 5.3 and 6.4.4. The calculations for this method are presented in Part 3 in Appendix E and are mainly based on the examples in Appendix A in Täljsten et al. (2011). The same material data for the laminates was also used.

Since the slab was assumed to have a one-way behaviour, the laminates were placed with the fibres in the same direction as the span. Each FRP strip was given a width of 60 mm and a thickness of 1.2 mm. The total design load was then assumed to increase with 2.5 kN/m^2 and the needed CFRP area per unit width was estimated. To account for the horizontal cracks that can propagate along surface mounted laminates the design value of the ultimate strain in the laminates had to be limited.

When a needed CFRP area had been estimated, the spacing between each strip was chosen. The strengthened mid-section was thereafter analysed accurately by assuming that failure occurs when the design strain in the laminate is reached and that the steel yields before this stage, see Figure 7.9. The strain at the top of the concrete could then be calculated by the condition of horizontal equilibrium and it was verified that the assumptions were correct. The spacing between the laminates was thereafter iterated to better utilise the moment resistance.

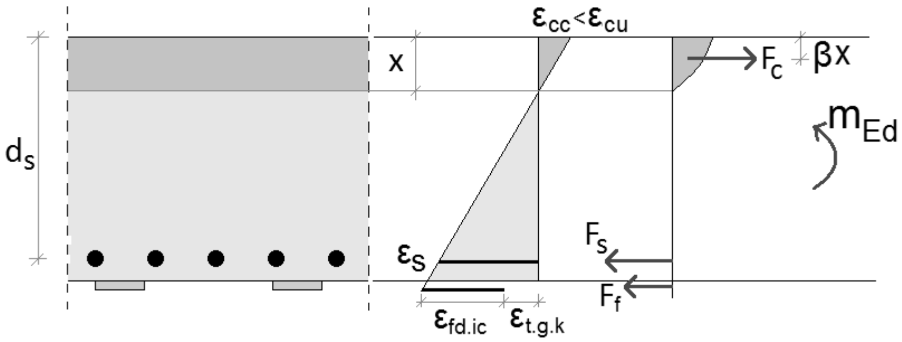

Figure 7.9 Model for analysis of the moment resistance of slab section strengthened with surface mounted CFRP strips. The capacity is reached when the steel is yielding and the laminates reach their strain limit.

When the needed amount of CFRP had been designed, the required anchorage length was calculated. This was performed by first finding the sections at which the outermost cracks due to bending moment occurs. The tensile force was thereafter determined with regard to the effect of inclined flexural shear cracks. Figure 7.10 illustrates how the tensile force increases as a result of inclined shear cracks. This increase depends on the height of the section and the inclination of the cracks and possible shear reinforcement. A higher section gives a larger effect.

Figure 7.10 Variation of tensile force along the slab without (broken line) and with (solid line) the effect of inclined shear cracks.

When the critical section had been found and the force that needed to be anchored had been calculated, the needed anchorage length was investigated. It was found out that the laminates only needed to be anchored 151 mm behind the critical section, but Täljsten et al. (2011) suggest that an anchorage length of at least 250 mm always should be used. They also state that it is advisable to anchor the strips as close to the supports as possible. Since the critical section was found to be 430 mm from the support, the anchorage of the strips is not critical in this case.

The next step of the design procedure was to check the resistance against peeling forces at the ends of the laminates. Since Täljsten et al. (2011) suggest an approach that is slightly different than the one proposed by Westerberg (2006), both methods were used. Even if the approaches gave dissimilar results, both indicated that the resistance against peeling forces is very high. In situations where the peeling forces are higher, e.g. if a higher beam is strengthened, it could however be of interest to investigate the differences between the methods more thoroughly.

The results of the calculations show that 60 mm wide strips need to be placed with a spacing of 400 mm to be able to increase the total distributed load on the slab with 2.5 kN/m^2. This gives a total cross-sectional area of 180 mm^2 CFRP per unit width of the slab. It is important that the failure mode in this case does not involve crushing of the concrete, but instead that the strain limit with regard to horizontal cracks that propagate along the laminate has been reached.

7.2.3 Strengthening with near-surface mounted CFRP bars

The design procedure for strengthening slabs with near-surface mounted CFRP bars is very similar to the method presented in the previous section. The main difference is however, as described in Sections 6.4.4 and 5.3.2, that the laminates are placed in sawn grooves at the tensile surface of the slab, which provides a better bond between laminates and concrete. The calculations for this method are presented in Part 4 in Appendix E and are mainly based on Example 2, Appendix A in Täljsten et al. (2011).

The needed area of CFRP to resist the increased load was calculated in the same way as described in Section 7.2.2, but this time assuming CFRP bars of type StoFRP Bar E10C. These bars have a square-shaped cross-section where each side is 10 mm wide. Material data for the bars can be found in Sto (2012). Another difference from the calculations described in Section 7.2.2 is that the ultimate strain in the CFRP did not need to be limited since there is no risk of horizontal cracks that can propagate along surface mounted strips.

When a needed CFRP area had been estimated, the spacing between each bar was chosen. The new moment resistance of the mid-section was thereafter analysed accurately in the same way as in Section 7.2.2 by assuming that failure occurs when the design strain in the laminate is reached and that the steel yields before this stage, see Figure 7.11. The strain at the top surface could then be calculated by the condition of horizontal equilibrium and it was verified that the assumptions were correct. The spacing between the bars was thereafter iterated to better utilise the moment resistance.

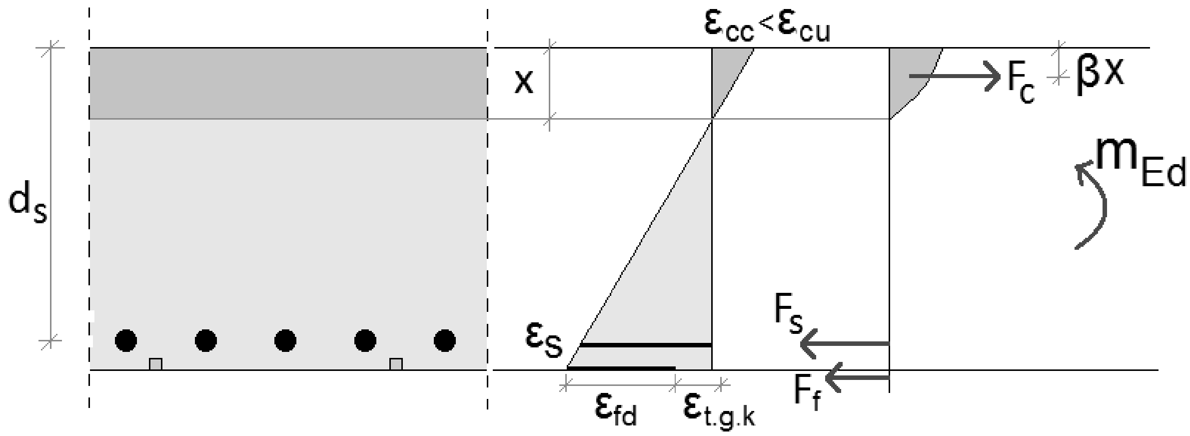

Figure 7.11 Model for analysis of the moment resistance of slab section strengthened with near-surface mounted CFRP bars. The capacity is reached when the steel is yielding and the bars reach their strain limit.

When the needed amount of CFRP had been determined, the needed anchorage length was calculated in a similar way as in Section 7.2.2. For near-surface mounted CFRP it should however be accounted for the fact that the bar is bonded to the concrete on three sides instead of just one. It was as for the previous case discovered that the needed anchorage length is sufficiently small to be able to fit into the space between the support and the sections with the outermost crack.

The results of the calculations show that the CFRP bars need to be placed with a spacing of maximum 850 mm to be able to increase the total distributed load on the slab with 2.5 kN/m^2. This gives a total cross-sectional area of 118 mm^2 CFRP per unit width of the slab (compare with 180 mm^2/m for the surface mounted). It is important that the failure mode in this case does not involve crushing of the concrete, but instead that the design value of the ultimate strain in the CFRP bars is reached.

7.2.4 Strengthening with steel beams on top of the slab

As described in Chapter 3 the problem with too low flexural capacity of the roof slab was solved by placing new steel beams on top of the slab in several of the studied projects. In most cases the beams were placed directly beneath the new walls, but in the calculations, it was chosen to use the beams to transfer an evenly distributed load to the supports. The calculations that are described in this section can be found in Part 4 in Appendix E.

Steel beams of type HEA140 were assumed to be placed on top of the original slab and an upper floor structure was assumed to distribute the imposed load transversally to the beams. When the beams tend to deflect, they press down the original slab which in its turn resists some of the load. This results in an interaction between the two members. It was however assumed that no stresses are transferred between the members, i.e. the beams slide on top of the slab, as illustrated in Figure 7.12.

Figure 7.12 Assumed interaction between beam and slab when the new load is added.

Since the steel beams are localised to lines along the slab, the whole slab will not deflect uniformly as in the case where the load is distributed evenly directly on the slab. This was regarded in a simplified way in the calculations by using an effective width of the slab as illustrated in Figure 7.13.

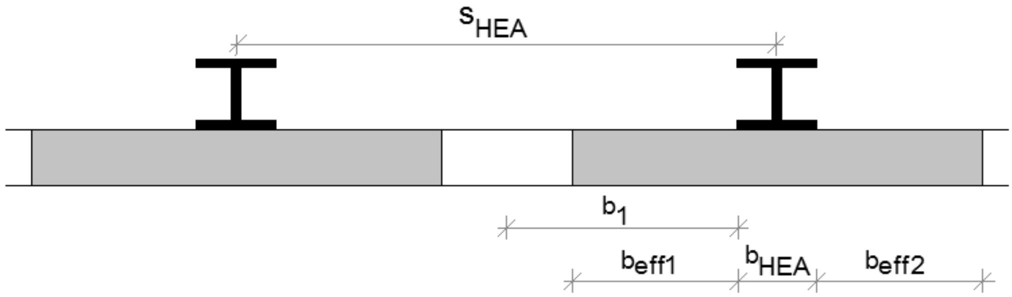

Figure 7.13 HEA-beams on top of the original slab. The grey areas of the slab show the assumed effective width for the response under added load.

The load distribution between the two different members will in the service state depend on the ratio between the flexural rigidities of the members. However, since the calculations in this project only treat the ultimate limit state, the respective capacities for the two members can be added together. This is because a hinge in the ultimate limit state will have been developed in mid-section for both members, independently of which member that yields first.

The bending moment capacity of the steel beams was first determined by use of the tabulated section modulus for the specific profile. The capacity of the slab was thereafter taken from the calculations for the original slab, but here only for the effective width beneath the steel beam. It is also important to only use the effective width of the slab when the self-weight of the slab is calculated, since the rest of the slab still can carry itself. To be able to increase the variable loads with 2.5 kN/m^2 it was found that the HEA140-beams need to be placed with a spacing of 2.6 m.

7.2.5 Strengthening with post-tensioned steel strands

As described in Section 6.4.3 it is possible to prestress slabs with strands that are placed at the tensile side and anchored in the top over the supports by tensioning them through drilled holes in the slab. The calculations for this strengthening method can be found in Part 6 in Appendix E.

The first step in the calculations was to choose amount and type of steel. As in regular prestressed members, high strength steel was assumed. It was also chosen to use single unbonded 7-wire strands with a total cross-sectional area of 100 mm^2 per strand. The spacing between the strands was first assumed and then iterated until sufficient capacity was gained.

The initial force in the strands was first calculated from the tensile strength of the steel. It was thereafter verified that this force could be applied without creating tensile cracks in the top of the section above the deviators. Such cracks would be devastating since the slab lacks reinforcement in the top. The stress at the top of the section above the deviator was calculated by use of Navier's formula, where both the moment from the self-weight of the slab and the prestressing force were regarded.

The next step was to calculate the deflection of the slab in the section of the deviator directly after tensioning. This was done to be able to investigate the increase of deflection in the ultimate limit state (calculated later) and thereby estimate the elongation of the strands. The calculation of the curvature was based on sectional analysis in state I since the whole slab is in compression due to the prestressing.

The bending moment that needs to be resisted in the ultimate limit state was thereafter calculated. This was performed by summarising the moments due to the increased distributed load, the self-weight of the slab and the contact forces that act in the locations of the anchors and deviators. Figure 7.14 shows an illustration of the forces that affect the bending moment. The forces from the prestressing steel reduce the total bending moment that needs to be resisted. In these calculations the moment from the horizontal components depends on the lever arm to the gravity centre of the slab, which in turn also depends on the deflection. The deflection in mid-span was therefore first assumed and then updated according to the results in later calculations.

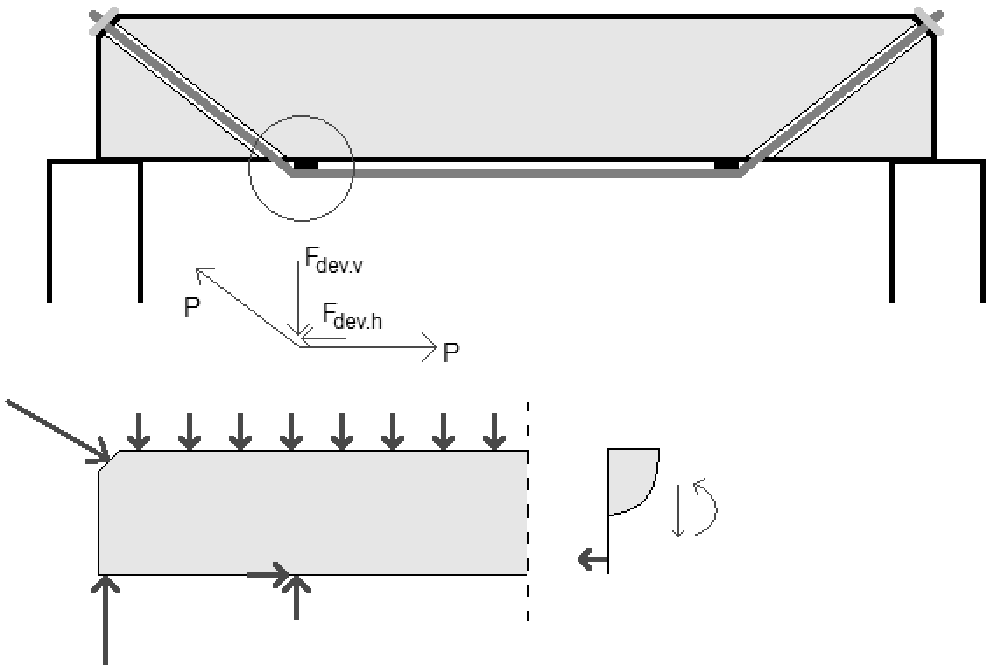

Figure 7.14 Contact forces from the anchors and deviators affect the total bending moment that needs to be resisted.

When the bending moment from all loads, including the effect of prestressing, had been calculated, the resistance of the section was investigated. This was performed with the ordinary sectional model in state II with stress block factors for the concrete compressive stresses. However, compared with earlier calculations in the ultimate limit state the horizontal equilibrium this time also included the compressive forces from the prestressing. These forces were applied in the sectional centroid since the moment from their eccentricity already had been accounted for. The moment resistance was then compared with the moment from all loads, including the effect of prestressing, and the spacing between the strands was updated until a good utilisation was reached.

If it was not for all the assumptions in the process, the calculations would have been finished when the utilisation had been verified. However, it was also needed to investigate how the force in the strands changes with time and load. The deflection at mid-section in the ultimate limit state therefore had to be calculated. This deflection was estimated by calculating the moment in each section along the span, which in turn was used to calculate the curvature distribution along the slab, support rotation and deflection. Since the moment in each section depends on the deflection in the same section (due to the horizontal loads), the calculations had to be performed iteratively so that the calculated deflection in each section was given as input data for the next iteration. One approximation in these calculations was that the whole slab was assumed to be in state II. A better approach would be to use different assumptions for different parts of the slab, i.e. all three states. However, it is generally accepted to account for elongation of external prestressing tendons by sectional analysis in state I or state II.

The last step in the calculations was to account for the difference in force in the strands between the time of application of prestress and the stage when the ultimate load is added after long time. The elongation of the strands due to deflection of the slab was first calculated. This was performed by comparing the deflection of the slab at the section of the deviator for the two situations. Other effects that also needed to be considered were relaxation, creep and anchor slip. When all effects were added together, the strand force for the design load in ultimate limit state after long time was reduced to 153 kN per unit width of the slab (compared to 172 kN under the self-weight at the time of tensioning).

When the deflection and the force in the prestressing steel had been calculated, they were reused in the calculations and all values were iterated until they fitted together. At that stage it was found out that the strands need to be placed with a spacing of 800 mm if the load should be increased with 2.5 kN/m^2. This gives a total steel amount of 125 mm^2 per unit width of the slab.

7.2.6 Strengthening with section enlargement on the compressive side

The increased capacity is in this case gained through an additional layer of concrete that is cast on top of the original slab. To simplify the calculations it was assumed that the same concrete strength as in the original slab was used for the added layer. It was also assumed that no stirrups or other transverse steel were used across the interface between the two layers. Any effects of longitudinal reinforcement in the added layer were neglected. The calculations for this strengthening method are presented in Part 7 in Appendix E.

The first issue that needed to be investigated for section enlargement was whether bending interaction between the two layers of concrete could be accounted for or not. The shear resistance in the interface was calculated according to Equation 6.25 in Eurocode 2, CEN (2004). This equation accounts for the strength of the concrete, cohesion and friction at the joint interface and possible bonded transverse steel. By assuming that the top surface could be regarded as rough (see Table 5.1), the shear resistance in the interface was calculated to 0.757 MPa. Thereafter, the shear force at the interface due to the applied loads was calculated close to the support since this section is critical. It was found out that the shear stress only reached a value of 0.255 MPa, so the resistance was high enough to ensure bending interaction between the two layers.

The load was thereafter increased and the resistance of the whole slab was investigated in the ultimate limit state. One important aspect with this strengthening method is that there will be a strain difference between new and old concrete. This difference depends on both the existing curvature in the original slab at the time of strengthening and the shrinkage in the added layer. However, since it in the calculations was found that the height of the compressive zone was small enough to fit within the added layer, this strain difference is irrelevant for the calculations in the ultimate limit state. Figure 7.15 indicates that only the top of the new layer in this case is in compression in the ultimate limit state. The rest of the concrete is disregarded in the calculations. Finally, the height of the new concrete layer was updated until enough moment resistance was gained.

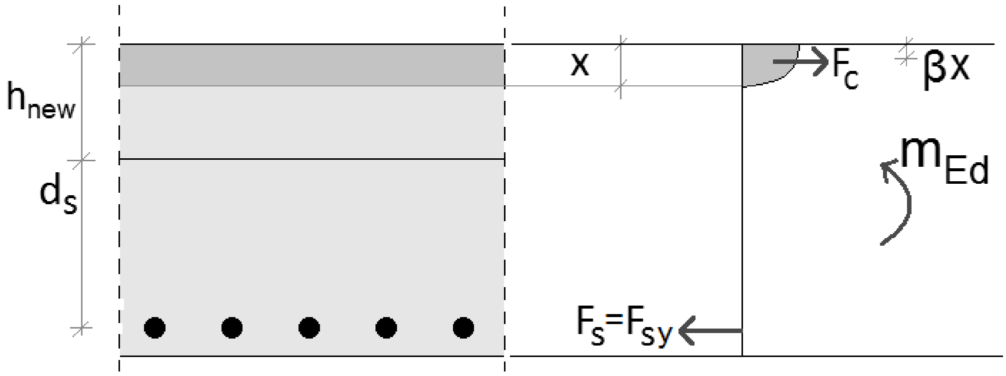

Figure 7.15 Model for sectional analysis in the ultimate limit state for the slab with section enlargement.

One conclusion from the calculations concerning this method is that a rather thick additional layer, in this case 105 mm, is needed to be able to increase the variable load with 2.5 kN/m^2. This rather high value can be derived from the fact that the reinforcement already for the original slab was designed to yield in the ultimate limit state (as is often the case). The force from the steel can therefore not increase further, which also means that the force in the compressed concrete cannot be amplified (due to the horizontal equilibrium). The moment capacity of the section can therefore only increase due to an increased lever arm between the forces. Since the self-weight also increases drastically with the thickness of the added material, a rather thick layer is needed to catch up with the increased moment.

7.2.7 Summary and conclusions

Table 7.3 below gives a summary of the strengthening methods used to strengthen the simply supported slab. Unlike the case with columns all investigated strengthening methods could be used to reach the desired capacity. However, the suitability of the different methods varies.

There are many aspects to consider when comparing the different strengthening methods and it should be kept in mind that all pros and cons cannot be discovered when just investigating a specific slab, especially since the chosen slab is a simply supported one-way slab. There are most certainly benefits with the different methods that only would be observed for other configurations. However, the calculations showed some differences that may be important when designers should choose a method.

To place steel beams directly on top of the slab seems to be a very simple and effective way to increase the capacity. However, this method is only possible if an increase of the total floor height can be allowed. It must also be verified that the supports can resist the higher concentrated loads that the large spacing between steel beams creates.

When comparing the two different methods with CFRP laminates, surface mounted and near-surface mounted, the calculations showed that a lesser amount of material is needed for the method where the laminates were placed in sawn grooves. The needed amount was 180 mm^2 CFRP per unit width for the surface mounted strips, while only 118 mm^2 for the method with near-surface mounted bars. This is of course something that favours the last method since CFRP are quite expensive. The near-surface mounted bars also ensure better anchorage, but this was not crucial for the studied slab.

The use of post-tensioned strands also showed to be a material efficient way to strengthen the slab. However, in comparison to the other methods, it seems quite expensive to drill inclined holes through the slab and anchor the prestressing forces.

To add a new layer of concrete seems to be the most ineffective way to strengthen this specific slab. The large thickness of the layer increases the self-weight quite much, which also can affect members further down in the structure. It might in some cases however be beneficial to combine strengthening on the tensile side with application of a thinner layer of concrete on the compressive side to avoid crushing of the concrete and reduce noise penetration.

Table 7.3 Summary of the strengthening methods used to strengthen the slab.

Strengthening method	Load increase	Material used per meter slab width
3. CFRP, surface mounted	2.5 kN/m^2	- 14.5 m CFRP (60x1.2 mm^2) - Glue etc.
4. CFRP, NSM	2.5 kN/m^2	- 6.8 m CFRP (10x10 mm^2) - Glue etc.
5. HEA140	2.5 kN/m^2	- 2.3 m HEA140
6. Post-tensioned strands	2.5 kN/m^2	- 7.5 m steel strands (100 mm^2) - Anchors and deviators
7. Section enlargement	2.5 kN/m^2	- 6.3 m^3 concrete - Reinforcement mesh*

* The reinforcement mesh has not been included in the calculations, but should be used due to crack control in such a thick layer of concrete.

8 Guidelines for the design process

This chapter is meant to aid the designer in the early design process. It should form a basis for how to identify common problems for various structures and design situations. Even if each storey extension project is unique, the designer can with the help of these guidelines become notified about critical issues in an early stage.

The chapter is divided in several sections, where the following steps of the process are treated:

- Section 8.1 presents general considerations concerning zoning, logistic problems, evacuation and other issues that accompany a storey extension project.

- Section 8.2 presents considerations that are more directed to specific buildings. The intention is that this information should aid the designer to evaluate the suitability to vertically extend existing buildings.

- In Section 8.3 focus is put on how the structure can be examined with the intention to find critical structural members and/or excessive capacity in certain members.

- In Section 8.4 the choice of structural system for the added part is treated.

- Section 8.5 presents information about which strengthening methods that can be suitable in different situations.

Most of the text in this chapter is based on the content in previous chapters of this report, but the discussions are in several situations influenced by the authors' view of the topic. For more detailed information, reference is often made to relevant sections in the previous part of the report. In particular, the information in Chapter 3 and Appendix B, where the reference projects are presented, is often useful. It is likely that the project at hand shows similarities with one or several of the studied projects.

8.1 General considerations before the project has started

Before any decision to investigate a possible storey extension on a specific building is taken, there are some issues that need to be considered. Below follow some of the main aspects that need to be considered in a storey extension project. It can be advantageous to be aware of them in a very early stage so that the important decisions are taken on as good basis as possible.

- Intention of the city, zoning documents
- Logistic problems
- Evacuation
- Increased need for parking and facilities
- Fire regulations
- Soil conditions
- Similar executed projects
- Actual state of existing building

One of the most important issues to consider before any storey extension is decided is the zoning documents. These represent the intention of the city and limits building heights etc. It is therefore important to be certain that a storey extension is possible. If changes in these documents are needed it may take several years. It can therefore be advantageous to appeal for needed changes very early. It is important to remember that the city council generally will not accept any changes that are not in alignment with their vision of that specific area. It can also be motivated to search for buildings that can be allowed to be extended without changes in the zoning documents.

A construction site requires space. To set up offices and work stations close to storage areas is something that needs careful consideration. However, populated areas may complicate things. People who live and work in the vicinity should be able to safely reach their destinations. This is the case every time any construction work is carried out within urban environments, but erection of new storeys on top of an existing building may even further complicate things. If people remain in the building during the storey extension, their daily procedures must be able to continue to some degree and disturbances should be kept to a minimum. Materiel, tools and equipment must also be transported vertically to the roof right from the very beginning of the construction work. All of these issues need careful planning to avoid disrupting the public as much as possible. A well coordinated logistic flow, with placing of everything needed to perform the construction, may therefore save a lot of time and money.

If and how to evacuate a structure also needs careful consideration; empty apartments etc. cost money and to arrange temporary quarters may further increase the costs. The risk of losing any tenants must also be kept in mind. If an evacuation is deemed necessary, it is advantageous to plan it carefully to reduce costs and disturbances as much as possible. Perhaps it is possible to evacuate the building partly. The possibility to only evacuate parts of the building reduces the need for temporary quarters and, if the evacuation is conducted in successive stages, the same provisional quarters can be used through the entire project. Careful considerations of evacuations also affect which erection methods might be more or less appropriate. Certain approaches may for example require more time or additional space, while other methods may reduce the produced noise and disturbances.

Increased needs of parking spaces, capacity of garbage disposal and similar are all factors that must be evaluated if more people are to be supported. On the other hand, these are the same factors that follow when erecting an entirely new building. However, it may not be possible to incorporate such additions to the same extent in an already existing building, which means that additional occupied land area might be required. For the specific case with parking lots, this problem can however often be avoided by utilising the fact that the required amount per person in many cases (at least for residential buildings) is lower today than when the original buildings were built.

To fulfil the fire safety demands is essential for all buildings. The degree of required measures may vary a lot for different storey extension projects. Different heights and number of storeys may have different influences on both the extension and the existing building. It is therefore important to be aware of the consequences that follow a specific storey extension. The effects of different building heights and number of storeys should therefore be evaluated. Since the demands often are correlated to the total number of storeys, it can be advantageous to bear the fire regulations in mind when the number of added storeys is chosen. This is treated further in Section 4.1.3.

The properties of the soil and the resistance of the foundation can have a large impact on how well a structure is suited for storey extension. Large measures may quickly become cumbersome and expensive. However, if the rest of the existing structure has excess capacity, it can be motivated to strengthen the foundation to match the needed capacity. It is quite common that projects where strengthening of the foundation is required often are rejected quite early, but it is important to properly evaluate the possibilities to strengthen the foundations before any decisions are taken. Awareness of the soil conditions and their implication is therefore very important before starting any project.

Foundations on bedrock generally pose no problems due to an increased load. This is however not the case for buildings situated on clay. Piled foundations can sometimes have reserve capacity, but additional piling is often needed beneath some critical walls. How to proceed depends on the length and type of the piles. Foundations with end-bearing piles might for example, as discussed in Section 6.5, be difficult to strengthen with additional piles.

It may also be advantageous to research if any similar projects have been carried out recently. This information can help pinpointing areas that were difficult, but also areas that went more smoothly than expected. Some examples of executed projects are provided in Chapter 3.

The state of the existing structure may also be vital for its suitability to storey extension. A more detailed analysis should be performed once it has been decided that the building is suitable for a storey extension, see Section 8.3. In this stage it can be sufficient just to perform a check of any existing damages and possibly also rough estimations of any excess capacity.

8.2 Considerations in the early state – pros and cons for existing structures

In a similar way as in the previous section early considerations are discussed and emphasised in this section. The topics are however here a bit more specific with regard to decisions that concern the choice of existing building along with its possible extensions. The common buildings that were introduced in Section 2.3 are here revisited, but now viewed more critically. Difficulties and considerations with regard to storey extensions are the focal points.

A summary of some of the more prominent positive and negative properties of different building types are presented in Table 8.1, while each property is discussed more deeply in the text that follows below the table.

Table 8.1 Summary of favourable and unfavourable factors when considering storey extensions for different building types.

	Favourable	Unfavourable
Residential buildings	Walls often have excess capacity. Robust structures, often well-braced in the transversal direction.	Many individual residents are affected. Many new elevators are often required. The budget is more often tight.
Hotel buildings	Easier to temporary evacuate the top storeys. Construction work can be carried out mostly during off-season. Storeys with hotel rooms can be quite robust. Less number of new elevators needed since corridors are used to reach the rooms (Often sufficient to extend existing elevator shafts).	Often have larger open spaces on the lower storeys with lobbies and restaurants (columns are often critical).
Office buildings	Often designed to be adaptable to future changes. It can sometimes be possible to perform the extension in the time gap after a big tenant.	Often consist of a column-based structure, with little excess capacity, both vertically and horizontally. Might be difficult to evacuate without losing the tenants.
Parking garages	Simple layout. Easier to evacuate. Little disturbance for users. Rather simple to add more vertical members, at the cost of parking spaces.	Members often in bad shape due to the severe environment. Often consist of a column-based structure, with little excess capacity, both vertically and horizontally.

8.2.1 Critical members and excess capacity for various types of existing buildings

Some members are designed with regard to other properties than their load-bearing capacity. Residential buildings often possess excess capacity. Sound and fire demands make them very robust and stable, since many of these buildings only are three to four storeys high. Bracing of these structures might however be required in the longitudinal direction. Hotel buildings have the same benefit as residential buildings, except that the lower storeys often are column-based. Strengthening might for such cases be limited to the lower storeys, making this type of building rather suitable for storey extensions.

On the other hand, parking garages and office buildings, where the vertical load-bearing members mainly consist of columns, may lack excess capacity with regard to vertical load. A column-based structure may also quite often lack excess capacity with regard to stability. Additional strengthening might for such cases be necessary to ensure sufficient resistance against increased horizontal forces.

In Chapter 3, where the studied projects are presented, it can be seen that the roof slab in many cases is problematic. In a storey extension project the roof slab becomes an intermediate slab and is therefore subjected to loads which it was not initially designed for. Either an entirely new slab must be cast, or some kind of strengthening is required. Strengthening of the roof slab is treated more in detail in Section 8.5.5.1.

8.2.2 Typical damages in various existing buildings

Parking garages with their rather harsh environment from de-icing salts etc. put high demands on a sufficient concrete cover thickness. Unfortunately, the design against this hazard has historically not always been properly performed and many of the existing parking garages in Göteborg show damages to some degree, see Section 2.3.4. Slabs and columns may therefore be in need of repair just to ensure the service life of the structure as it is. However, when this kind of renovation is being planned, it can be favourable to take it one step further and evaluate if it is possible to vertically extend the structure. The cost required just to maintain the building in service may thus be integrated with the cost of a storey extension.

No real conclusions could be drawn about typical damages for residential, hotel or office buildings when looking at the studied projects. More data would in this regard be required. However, changes in the interior layout over the years may have resulted in unintended changes of the structural system, which ultimately may lead to damages if the load is increased. This should therefore always be investigated.

8.2.3 Evacuation of various existing buildings

Both residential and office buildings require quite extensive planning with regard to evacuation and some kind of temporary quarters are most often required. The risk of losing a tenant must also be considered and it can therefore be advisable to have an early dialogue with the tenants about their needs and prospects. This can be especially important for offices, where the same tenant may rent large parts of the building. A specific case that may arise for office buildings is when the tenant requests more space, which might be addressed in form of a storey extension. It is in such cases extra important to form the terms with the tenant and to come to an agreement on how everything should proceed. For residential buildings it is important to consider that the lives of the residents are affected to a higher degree.

One of the major advantages with parking garages is that they more easily can be partly or completely closed off from use without too dire consequences. Supplementary parking spaces might be necessary depending on the degree of the construction work, but in comparison with evacuating parts of an office or residential building the costs and consequences for the affected persons are rather limited.

Evacuation of hotels can also be performed quite effortlessly; parts of the hotel can be closed, while the activities in the rest of the hotel continue almost as before. The evacuation should preferably be planned to occur mostly during off-seasons to minimise the costs. As mentioned in Section 2.1.1 the possibility to use the evacuated rooms as whereabouts and site offices for the entrepreneur makes hotel buildings extra favourable in this regard.

8.2.4 Layout of different existing buildings

The internal layout with placing and spacing of structural members will also have an impact on the building's suitability to storey extension. The positions of load-bearing members in the original structure can be limiting for the placement of the members of the extension. It can therefore, to avoid extra measures, be preferable to plan for similar activities in the extension as in the original building, since this increases the appropriateness of using a similar layout in the two parts. However, as described in Section 3.2.1, new demands and regulations can sometimes make it more or less impractical to use the same layout in the new part.

For cases when the vertical members cannot be placed straight above each other, the load must be shifted horizontally through slabs or beams, which may generate problems. The localised load needs to be diverted for such cases or some kind of strengthening of the slab or beam is necessary. Strengthening can also come in question when the load is shifted horizontally somewhere inside the original building. It is therefore important to investigate if such situations exist before it is decided to go on with the project.

When considering the accessibility demands, the general shape of residential buildings can have large influence of the suitability of the project, since it can be expensive to ensure that the these demands are fulfilled for long and narrow buildings. This is due to the fact that each stairwell most often only serves the adjacent apartments. It must therefore either be decided to build many elevators or to use external or internal corridors, which in turn may generate a need to shift loads horizontally. A square-shaped building with the stairwell placed in the centre can, in this regard, be better suited for storey extensions. More information about the influence of the general shape of residential buildings can be found in Section 2.3.1.

Internal corridors are normally used in office and hotel buildings. These types of buildings are also normally equipped with elevators that connect to all available storeys and rooms. A similar layout for a potential storey extension may therefore be advantageous to reduce the need of new elevators.

Parking garages have very open internal layouts, which may favour other structural systems for the extension. It can here be more beneficial to choose a structure that localises the loads to the original columns as much as possible. Any existing elevators are normally placed in connection with exits. Careful consideration of the internal layout of the added storeys is therefore required to utilise any existing elevators as much as possible, while also keeping the need for new elevators to a minimum.

8.3 Inspecting the state of the existing structure

In this section it is presumed that it has been decided to investigate the suitability of a storey extension for a specific building. Further knowledge about the existing building is needed before any permanent decision is taken. The key aspects of what is needed to keep in mind when examining the condition of the existing building are presented in this section.

The process of evaluation of a structure can be categorised into a preliminary assessment and a more detailed assessment, American Society of Civil Engineers (2000). The preliminary evaluation forms the basis for the detailed and can indicate which testing methods are required.

The state of the building of interest may vary a lot from one case to another. It is therefore important to perform a thorough investigation of the actual building's condition, its capacity, but also its current loading configurations. A detailed survey can normally be motivated since this may result in finding unused capacities that even may diminish the need for strengthening, Statens råd för byggnadsforskning (1978). The following issues may need to be investigated:

- Available documents, calculations and blueprints
- Size and placing of members of affected parts of the superstructure, including differences between blueprints and reality
- Previous changes of the structural system
- Structural behaviour of the existing building
- Assumed loads in the original design
- Current loads
- Material properties and concrete strength
- State of concrete and reinforcement in the existing building

- Visible strains and deformations
- Probable causes of any damages
- Presence of any excess capacity

When performing an assessment of a building it is, according to the American Society of Civil Engineers (2000), important that the people performing the evaluation are experienced and competent. They also point out that the interpretation of the results should be handled by a capable engineer with proper qualifications since this is one of the most important steps in the project.

An inventory of the available documents should be executed during the preliminary assessment, American Society of Civil Engineers (2000). These may for example include drawings, calculations and design criteria. Prior collected information may simplify the labour at the building site, since the need for some testing may diminish. An inspection of the building should be performed to check the accuracy of any documents. Indications of damages, modifications, settlements, certain weaknesses and similar should also be observed and documented.

It can in the early stages be sufficient to estimate the material properties when determining whether a storey extension is reasonable or not. By knowing what concrete strength class that normally was used at the time when the building was designed, a reasonable estimation should be possible, American Society of Civil Engineers (2000). Uncertainties in estimation may be handled by some kind of reduction factors. However, more precise knowledge of the materials is necessary in a later stage of the assessment and in-situ testing might then be needed. This can also be a way to find additional excess capacity. Today it can be fairly simple to acquire information about the reinforcement, such as amount, spacing and dimensions. This can for example be performed by some sort of scanning equipment and thus be obtained without damaging the concrete.

If an extensive assessment is to be carried out, Statens råd för byggnadsforskning (1978) claims that hours or even days can be saved if the procedure is planned carefully. Focus should lie on the critical members and their most important characteristics, such as self-weight, concrete strength, reinforcement amount, cross-sectional sizes and straightness of members.

It can also be advantageous for the engineer to study the design codes and structural systems used at the time of erection, American Society of Civil Engineers (2000). This can give a better understanding of the loads and load cases assumed at the time of design. It is important to bear in mind that in the old Swedish design codes the safety factors were treated differently than in the Eurocodes. New loads can therefore not directly be compared with old capacities.

The compressive strength of concrete is its main property with regard to structural engineering. This strength is more or less correlated to other properties of concrete and can for example be used to get an estimate of the density and modulus of elasticity, American Society of Civil Engineers (2000). It is according to Statens råd för byggnadsforskning (1978) not implausible that the strength of the concrete is higher than what is given in the original design documents. It can therefore be motivated to investigate this further, especially if strengthening is only barely needed. The loading situation might have changed as well. As described in Section 3.1.1 some safety factors used in the original design may be reduced if it can be proven that the structure was erected properly.

The state of concrete and reinforcement is also of interest, for example, extensive carbonation of concrete may mean that the reinforcement is in risk of corrosion. Crack lengths, crack widths and deflections may also be of interest. This can be indications of critical parts and possible errors in design or execution. Any detected damage or similar should be investigated to determine what may have caused it and how it can be avoided in the future.

In the end of this stage the collected information (the loading history, the current loading situation and the estimated capacity) should be sufficient to estimate the excess capacity of the building. In this stage it can also be advantageous to evaluate whether it is possible to further reduce any of the loads. Pinpointing the critical members that might require extra attention is also of interest.

One possible way to find unused capacities is to investigate the straightness of the vertical load-bearing members. This becomes more relevant for high-rise buildings. A rather harsh unintended inclination should generally be assumed in design calculations. If it can be verified that the members are standing straighter than assumed, the real inclination can be used in the calculations. It may also be possible to reduce the terrain category with regard to wind load if the built environment around the building has developed.

8.4 Evaluation of the structure and the extension

In this stage the condition of the existing building is known along with a rough estimation of any available excess capacities. The critical members are also identified It is now of relevance to determine how to proceed with this information. If the structure still is deemed suitable for storey extension, it should here be established how many storeys that are appropriate for the building at hand. With this follows choice of type of superstructure for the extension, as well as if and to what extent the building should be strengthened.

Different types of superstructures for the extension are discussed briefly in Section 4.2. The general aspect to pursue is a light superstructure since a reduced weight means a reduced stress on the underlying members. One approach might be to estimate the weight of several possible extensions with varying number of storeys and evaluate how they affect the need of strengthening. However, there are other factors to consider as well. A reduced construction time can for example be of great interest in certain projects and a way to keep the total cost down. Another aspect is the architectural characteristics of the building.

There is a large variety of structural systems and combinations that can be used as superstructure for the extension. However, whichever approach is chosen, the sound and fire demands must be fulfilled. A light timber structure will for example gain some extra weight due to fire protection and a hollow core slab might require an additional layer of concrete topping to fulfil the sound demands.

The existing building also has effects on the choice of superstructure for the extension. The decision whether or not to adapt to the original layout must be considered and taken early. Since the loads most often need to be transferred through the existing structure, the superstructure of the extension is often limited to the locations of the vertical members. A secondary structure can however be used to shift these loads horizontally if necessary. Prefabrication is also a possible option to use for the extension. However, for this to be applicable it is advantageous to be able to have the same dimensions of many elements. This requires that the spacing between members in the existing building are recurring.

There is also the possibility to completely disregard the old structure and build a new building standing independently above the existing building. This approach requires no strengthening and has therefore not been thoroughly investigated in this project. However, in Section 3.1.10, it is briefly described how this method was used at Studio 57.

Strengthening is optimally avoided or limited to as few members as possible and, in the end, it is a matter of how much money that is reasonable to spend on an extension and strengthening. It might be appropriate to evaluate how many additional storeys the building can uphold without any strengthening at all and how extensive measures are required to increase the capacities for additional storeys. A procedure like this may help to determine how many storeys are economically justifiable. A higher construction cost ultimately leads to a higher rent for the residents or tenants.

8.5 Choice of strengthening methods

In this stage it is known which members that are most critical and in need of strengthening. The increase in load is also known. This section is intended to help the designer to consider appropriate strengthening methods for the specific member at hand. This section is, to ease the use, divided after type of member in need of strengthening.

The different applications of strengthening methods described in Chapter 6 are now evaluated further. In storey extension projects specific boundary conditions can make certain methods more or less favourable.

As mentioned earlier columns and slabs (mainly the flexural capacity of the roof slabs) are in many times critical when vertically extending a structure. Therefore, extra attention is given to these two types of members.

8.5.1 Lack of axial capacity of columns

Interaction between normal forces and bending moments has a great impact on the load-bearing capacity of columns. Studying the combination of normal force and bending moment in the interaction curve for the specific member is therefore of great interest. From this, it can be determined which methods that are less suited and which are better suited.

Comparative calculations have been performed for different strengthening methods of columns and are presented in Appendix D. In Section 7.1 a short summary of the results from these calculations are presented and discussed. When strengthening columns, it is important to remember that the column cannot deform more than the ultimate strain of the original concrete (if the column buckles the ultimate strain will not be reached). It can therefore sometimes be advantageous to reduce the loads as much as possible at the time of strengthening, or even mechanically unload the column, to reduce its initial strain. Any added material can in this way be utilised better.

As mentioned earlier, load-bearing columns are often critical if the load is increased. The following methods can come in question when there is a desire to increase the capacity of columns:

1. Vertically loaded steel profiles on the sides of the column

2. Vertically mounted steel plates

3. Section enlargement – assumed to be vertically loaded

4. Section enlargement – assumed to only contribute to the bending stiffness

5. CFRP wrapping

An overview of when a specific method is suitable is presented in Table 8.2. Here, critical issues are highlighted along with the main applicability and pros and cons for each method. Below the table follows a text where each method is discussed more thoroughly

Table 8.2 Overview of applicability for various strengthening methods, critical issues are highlighted. Numbering according to the list above.

	Main applicability	**Not suited**	**Pros/cons**	**Crucial issues**
1.	Stockier columns, where compression dominates.	In parking garages or when a slender solution is desired.	+ Easy and fast to install + Easy to prestress - Impact sensitive - Logistics (large elements) - Large concentrated forces on the slab	To ensure that the profiles are loaded from above. To achieve bending interaction is beneficial.
2.	Very slender columns or when an external moment is applied.	Stockier columns.	+ Low material use + Easy and fast to install - Not contributing to the compressive capacity	To achieve tension in the steel (prestressing is good) To ensure interaction with the concrete.
3.	To resist both normal force and moment. Especially suited for slender columns.	When a slender solution is desired or for tight time plans.	+ Low material costs + Larger contact surface - Extensive labour - Curing time - Difficult to unload existing column	To ensure that the new layer is vertically loaded. Interaction between the layers. Shrinkage and creep.
4.	Slender columns or when an external moment is applied.	Stockier columns, when a slender solution is desired or for tight time plans.	+ Low material costs - Extensive labour - Curing time	Interaction between the layers.
5.	Stockier columns, where compression dominates. When a slender solution is desired and for tight time plans.	Slender columns or when an external moment is applied.	+ Small increase of thickness + Relatively fast - Expensive - Unhealthy	Very dependent on execution (skilled workers needed). Shape of the column may have a large impact on efficiency.

To add new vertical load-bearing members can be one way to avoid the difficulties that follow strengthening procedures of existing members. If the workers lack experience of strengthening procedures, it can be advantageous to just add new elements. It might however be difficult to find space inside the existing building. If the storey that contains the columns for example is a garage below a planned residential building, it might be possible to sacrifice some parking lots to place the new elements. However, for buildings where an open layout is desired, it may not be possible to just add new columns. Adding new columns also requires careful consideration about how the new members affect the behaviour of the beams or slabs above. Therefore, the placing of these columns needs careful planning.

If instead the original columns are strengthened, the problem with changed load paths in the slabs or beams can be avoided. To add new steel profiles on the sides of the column might be a fairly easy way to perform the strengthening. Which type of profiles that is best suited can vary from case to case and it can be of advantage to turn to the approach with the moment/normal force interaction curve to get an indication of which type is most appropriate. Three different variations with HEB180-profiles were examined during the comparative calculations, namely:

- Without interaction between steel and concrete

- With interaction between steel and concrete

- With interaction between steel and concrete while also prestressing the steel

If Table 7.1 is studied, it can be seen that there is a large difference for the load increase whether interaction is achieved between concrete and steel, while the additional contribution from the prestressing effect is not as great. The extra effort required to get interaction might therefore be motivated. However, perfect interaction might be complicated to accomplish. The most important aspect to consider when using steel profiles is to ensure that they get properly loaded, i.e. there is no gap to the overlying member. The extra effort needed to achieve a prestressing effect is thereafter not very great and it would be unmotivated not to utilise it. It could also be used as compensation for any difficulties in achieving a perfect interaction between concrete and steel.

Calculations with steel plates have also been executed, both for a slender and a non-slender column. These steel plates were assumed to only contribute to the bending stiffness and not carry any normal forces (which they in reality will do, see Figure 6.3). As can be seen in Table 7.1 and Table 7.2, the contribution to the stockier column is negligible even for a considerate amount of steel, while the slender column gets a drastic increase in strength with less steel. It can easily be deduced that this kind of strengthening might be a good option for very slender columns. The low material use is a great advantage for this approach. However, it is important to ensure that the plates are properly attached to the column.

Calculations with CFRP plates instead of steel plates were also performed, however, a high modulus CFRP laminate was not found among the available material and has therefore not been evaluated. The results showed that this kind of strengthening method was not a suitable option with regard to strengthening columns if no external moment is applied.

Section enlargement of the columns can be a good way to perform the strengthening, provided that enough space is available. The increased cross-section area gives improved performance towards buckling due to reduced slenderness, but also contributes to the capacity for increased loading. It is here, as mentioned earlier, important that the original concrete has possibilities to deform further to enable activation of the new concrete. A problem with section enlargement is that quite extensive construction work must be carried out inside the existing building, either by casting in formwork or use of shotcrete. Anchoring of any reinforcement may also be cumbersome. When selecting how to enclose a column, the surrounding environment needs to be considered. A complete enclosing is to be preferred, but it may however be difficult to unload the column when this method is applied. Another aspect that might favour this method is when there is risk for punching shear failure and an increase in area is required.

As discussed earlier it also requires good workmanship to ensure that the new concrete is interacting with the existing concrete and gets loaded. This should however be easier to ensure when the load is increased than if the purpose is to unload the column. As described in Section 6.1.1 one approach is to use shrinkage compensated concrete. When casting new concrete around existing columns, it is also important that the concrete fills all the voids. A self-compacting concrete may therefore be advisable. In Appendix D Part 4 calculations for a column strengthened with additional reinforced concrete can be viewed.

If it, for some reason, is difficult to ensure that the new concrete gets loaded directly from above, it can be an alternative to only assume that the sectional enlargement only increases the bending stiffness. For this to be an efficient approach it is however required that the column is rather slender or subjected to an external moment. In Part 5 and Part 13 in Appendix D, calculations for section enlargement that is assumed to only contribute to the bending stiffness are presented. It can again be noted that the new layer in reality will be loaded further down in the column if the interaction between the layers is good, see Figure 6.2.

Confining concrete with wrapping of CFRP does not claim as much space as section enlargement or adding of new elements, but requires that the columns are accessible from all sides, at least during application. Strengthening with CFRP is however quite an expensive approach with regard to material costs, but a swift and simple installation is a factor that also need to be considered.

As discussed in previous chapters the best efficiency is gained for circular columns. This is also evident from the calculations available in Appendix D, Parts 6, 8 and 12. For square cross-sections with limited additional space, a combination of CFRP and a sectional enlargement might be something to consider, i.e. first casting the column to a circular or elliptical cross-section and then wrapping it with CFRP. From the calculations it is also evident that it is more efficient to strengthen a non-slender column than a slender column.

As mentioned in Section 3.4 columns are often the critical load-bearing members, especially for parking garages and hotel buildings. These are therefore discussed more thoroughly in the two upcoming subsections.

8.5.1.1 Parking garages

The best solution when strengthening columns in a parking garage is probably quite often a regular section enlargement, especially if the concrete cover is in bad shape (or just too thin). Since the shape of the section enlargement can be chosen quite freely, it also enables a solution that can be adapted to the actual conditions and thus minimising the number of reduced parking spaces. The solid cross-section can also make the column more resistant towards accidental impacts.

Utilising confinement from carbon fibres may seem as a good option for strengthening as the additional material do not require much extra space. However, the risk for damage due to car impact can be considered quite high and, since the CFRP solution is sensitive to impacts, they cannot be used without proper protection or extra strengthening. The extra strengthening can for example consist of additional wrappings with aramid FRP, as in the example in Täljsten et al. (2011), while protection may include crash barriers.

Strengthening with steel profiles may not be a very good solution in this case. They do not add much extra area, but they need extra attention with regard to impact loading. Steel profiles are also sensitive to corrosion and the harsh environment put extra demand on a proper design and protection. Accidental impacts may also damage the corrosive protection.

8.5.1.2 Hotel buildings

The lowest storeys are often used for lobbying activities. These lobby floors require a rather open internal layout and are therefore often dominated by concrete columns as the main vertically supporting members. Increased thickness of lower floor columns should not pose a very big problem, but since these columns in many cases are visible, an aesthetic solution may therefore be favoured. Surface mounted steel profiles may therefore not be a very good choice. Additional columns may also have a negative influence on how the bottom floors are perceived and should preferably be avoided.

A rather straightforward way to strengthen the columns is to use carbon fibre wrappings since no extra space is occupied. It is however important to ensure adequate fire protection. A circular or elliptical cross-section is preferred for CFRP wrappings to be most efficient. Recasting of rectangular sections to a circular shape can be encouraged if there is a very limited space in which the columns can be increased. Otherwise, a regular section enlargement with concrete might be a more appropriate solution.

8.5.2 Lack of compressive capacity in walls

The case where the load-bearing walls are most critical is, as discussed before, rather unusual. If walls however need to be strengthened, the following methods can be considered:

- External beams and columns
- Section enlargement
- Strengthening with regard to buckling of the wall by vertical CFRP

To use a system of external beams and columns adjacent to the wall seems like a simple method, which is rather straightforward to perform. No considerations need to be taken about bond or similar. It should also be quite simple to increase the part of the load taken by the external construction just by adding wedges between the top slab and the added beam, i.e. relieving the original structure from load. How the load is transferred depends on the connections. One disadvantage is however that it is preferred to build the construction symmetrically on both sides of the wall to avoid uneven loading. This might not be possible in all cases, as for example with external walls. Another disadvantage might be the visual effects of new columns and beams along the wall or reduced net area.

Section enlargement might require more skilled labourers due to the demands about bond to the old surface and undisturbed load path from top to bottom. The concrete also needs time for curing. The result is however a smooth surface that does not affect the visual impression, apart from that the wall gets thicker. It is important to remember that the additional concrete only will be active once the load is increased.

If the wall is too slender and there is a risk of buckling, it is possible to strengthen it with glued CFRP that are placed vertically as strips or as near-surface mounted laminates. The FRP are not suitable for compressive forces, and to confine an entire wall by CFRP wrapping is not feasible. Strengthening a wall with CFRP therefore primarily increases its bending capacity. This method can be motivated if the thickness of the wall is limited.

8.5.3 Lack of flexural capacity in beams

Beams are most often not the critical members in a storey extension project. They may however need some extra attention if they are supporting a roof slab, which most often is subjected to increased loads. Beams can also become critical if the structural system is disrupted somewhere along the load path downwards so that the load from above needs to be horizontally shifted. The treated methods are as follows:

- Section enlargement on the compressive side
- Section enlargement on the tensile side
- Adding new steel profiles above, beneath or at the sides of the beam
- Glued CFRP laminates on the tensile surface
- Near-surface mounted CFRP laminates on the tensile side
- External prestressing

To add new columns beneath the beam might sometimes be possible. However, if the support is rather stiff, the top of the member can come in tension, which it most often is not designed for. Strengthening the top side to resist these tensile forces might not be possible due to the presence of an overlying slab. Strengthening on the sides might be an alternative. However, the ineffectiveness of this approach needs extra consideration. For the case where load shifts occurs further down in the building, the most straightforward way might be to place the added column directly beneath the concentrated load, but there is most often a reason for the shifting of the load in the first place (e.g. open spaces).

Section enlargement on the compressive side is best suited if the beam is over-reinforced, which is quite uncommon. It is also most often troublesome to perform the section enlargement on top of a beam due to the overlying slab. One exception is however the case with T-beams, where the top of the beam is integrated with the slab. The same pros and cons as for the slabs in Section 8.5.5.1 and Section 8.5.5.2 should be relevant for this case as well. If the height of the beam cannot be increased, either from above or below, a section enlargement on the sides might be something to consider. However, with regard to increasing the flexural capacity of the beam, this approach is not very effective due to short lever arms in bending.

To add reinforcement and concrete on the tensile side might however be possible in more situations, since it is normally located at the bottom side of the beam. A disadvantage with this approach is that a lowered room height must be permitted. As earlier discussed, the construction work for this kind of strengthening can be rather complicated and messy since it requires both attachment of new reinforcement and either casting in formwork or application of shotcrete. Another disadvantage is that it is difficult to anchor the added reinforcement bars at the end of the beam and ensure that the tensile forces are transferred to the supports.

Strengthening by adding new steel profiles seems easier than casting new layers of concrete, provided that it is logistically possible to bring the profiles inside the existing building. It should be possible to use shorter beam segments, but this approach would then require more work at the site, since splicing might be necessary. However, the possibility to strengthen the beam, either on the sides, the bottom, or the top, gives good options to adapt this method after available space. One negative aspect with this approach is the required space for the beams.

A steel profile placed above a concrete beam, unloading it partially or completely, appears to be both difficult and unnecessary in most storey extension projects. Placing the profile on new supports beneath the concrete beam has the benefits that drilling through the concrete is avoided and that any earlier deflections can be reduced. The reduced free height inside the existing building and the need of new supports can however be severe disadvantages and need great considerations. The remaining method, to bolt the profiles onto both sides of the beam, seems more appropriate. In this way, both the stiffness and the ultimate capacity can be increased without the need of extra height or supports. Unfortunately, drilling through the concrete might be necessary to attach the beams. A considerate amount of material is also put where its contribution is very small.

To glue CFRP strips onto the tensile side of the beam is a relatively simple way to strengthen with regard to flexural capacity without increasing the height. However, as with other strengthening methods, it might be difficult to transfer the increased tensile forces towards the support. If the anchorage is decisive, near-surface mounted FRP is a possible option since the improved bond properties enable better transfer of the forces to the adjacent concrete. Near-surface mounted FRP otherwise work quite similarly to surface glued FRP, but better. However, near-surface mounted FRP can only be used if there is enough cover thickness to anchor it. Alternatively, if the cover thickness is small, a hybrid T-shaped bar can be used as both surface and near-surface mounted. However, the availability at the producer needs to be taken into consideration as well. A disadvantage with FRP is the unhealthiness of the epoxy. Its poor fire resistance may also be problematic and, for cases where fire protection is required, some additional thickness can therefore be expected.

Application of prestressing steel externally might be especially suitable for beams where the deflection or crack widths are limiting. However, as discussed in Section 7.2.5, this method should also be a rather material efficient way to be able to increase the ultimate load. Even if the calculations were carried out for a slab, a similar result would probably have been achieved for a beam. One benefit for beams when compared with slabs is however that this approach does not require drilling of the inclined holes that are necessary for the slab. The labour needed to install the strands should therefore not be as extensive in this case. On the other hand, it may be difficult to install the anchors, especially if a slab lies on top of the beam. The anchorage zones also need special consideration due to the high stress concentrations. To use CFRP instead of steel strands can be a way to reduce the relaxation, risk of corrosion and cross-section area. However, it may in many cases seem more appropriate to just increase the area of the steel instead, especially since there are problems with how to anchor CFRP. Corrosion can instead be avoided by use of plastic sheathing.

8.5.4 Lack of shear capacity in beams

To strengthen beams with regard to shear failure might come into question in similar situations as strengthening for flexural resistance. In many cases, flexural strengthening also strengthens the shear resistance to some extent. However, it is also important to keep in mind that strengthening against bending moment may increase the risk of a brittle shear failure, thus requiring additional strengthening. If the shear capacity is critical, the following methods might be possible:

- Section enlargement
- Vertical post-tensioned rods
- Glued CFRP strips on the sides of the beam (vertical or at an angle)
- Glued CFRP sheets on the sides of the beam (vertical or at an angle)
- Near-surface mounted vertical CFRP bars

In the same way as for slabs an enlarged section also increases the shear resistance. This can however, as discussed before, be difficult to perform inside of an existing building. A section enlargement on the sides might be more motivated with regard to shear strengthening than flexural strengthening. This approach also avoids the problem of reduced free height.

Post-tensioned vertical rods should be easier to apply for beams than for slabs since they can be mounted on both sides of the beam. One problem might be if a slab lies on top of the beam. The designer should however be able to avoid this problem by letting the workers make small cuts in the top of the beam so that a steel plate can be inserted between the beam and the slab. On the other hand, it might be more difficult to strengthen beams with T-sections in this way since the rods must be anchored in the compressive zone.

CFRP used to strengthen a beam with regard to shear failure is best placed on its sides, with the fibre direction either vertical or at an angle with regard to the beam. CFRP at an angle generally perform better, but can be more cumbersome to install. When mounting glued CFRP strips or sheets there is the option whether or not to let the CFRP continue from side to side via the bottom of the beam. If the sheet or strips continues, any sharp corners must be smoothened which may require some extra work. A factor that disfavours surface mounted CFRP is that the surface preparation can be rather time consuming. The work can however be executed without the need to reach the top of the beam (even if the anchorage is improved when the CFRP strips are applied around the whole beam). One disadvantage with glued CFRP strips or sheets, compared to the vertical rods, is the inability to prestress the material. The ability to prestress the new shear reinforcement should however be most important if shear cracks already exist before the load increase.

Near-surface mounted FRP can also be used to increase the shear capacity of beams. These bars cannot be bent around the members as easily as surface mounted FRP, but their superior bond capacity should provide sufficient interaction. These bars then work similarly as internal shear reinforcement stirrups.

8.5.5 Lack of flexural capacity in slabs

The strengthening need with regard to bending moments in slabs can vary quite substantially, whether the slab is located on the roof or not. Therefore, this section is divided into two subsections, treating the roof slab and a slab located on other storeys separately. Some possible methods are listed below and Table 8.3 highlights their suitability along with critical issues. The methods are discussed more thoroughly in the two upcoming subsections.

1. Surface mounted CFRP laminates
2. Near-surface mounted CFRP bars
3. Steel beams on top of the slab
4. Post-tensioned steel strands
5. Section enlargement on the compressive side
6. Section enlargement on the tensile side
7. Filling of cores

Table 8.3 Overview of applicability for the investigated methods, critical issues are highlighted. Numbering according to the list above.

	Main applicability	**Not suited**	**Pros/cons**	**Crucial issues**
1.	When the height cannot be increased.	When the slab is over-reinforced.	+ Little disturbance + Easy to mount in confined spaces - Expensive - Unhealthy - Fire protection	The concrete must have sufficient strength and be in good shape. Preparation of surface. Anchorage zones.
2.	When the height cannot be increased.	When the slab is over reinforced. When the cover thickness isn't sufficient.	Same as above + Better utilisation of CFRP + Better anchorage - Only possible in one direction	The concrete must have sufficient strength and be in good shape. Preparation of grooves (that don't interfere with reinforcement).
3.	For the roof slab, especially when the structural systems are not aligned	When the height cannot be increased.	+ Easy and fast to install - The height is increased	To investigate the interaction between beams and slabs.
4.	When there is a need to reduce deflection.	When the top side cannot be reached.	+ Good material usage - Difficult and time-consuming to install	Drilling of inclined holes without damaging reinforcement. Ensure that the top of the slab does not crack at tensioning. Crushing and splitting of concrete at anchors.
5.	When the slab is over-reinforced.	When the height cannot be increased. When the self-weight needs to be kept to a minimum.	+ Inexpensive + Extra sound barrier - The slab is not often over-reinforced - Heavy alternative	Interaction between layers of concrete. Differential shrinkage and creep.
6.	When extra reinforcement is desired and it is possible to decrease the room height.	When the room height cannot be decreased. When the self-weight need to be kept at minimum.	+The new reinforcement is concealed - Work need to be carried out upside-down	Attaching and protecting reinforcement. To transfer forces from new reinforcement to supports.
7.	For hollow core slabs. When an increase in shear strength also is desired.	For other types of slabs. When installations etc. already are located in the cores.	+ No height increase - Low material efficiency in bending - Difficult to install	Finding and opening the hollows. Interaction between old and new concrete.

8.5.5.1 Lack of moment capacity in roof slabs

As described earlier the increased load on the roof slab can often pose a problem. Figure 8.1 illustrates how the evenly distributed imposed load can increase the demand on the slab (left) and how concentrated loads can arise from new elements (right).

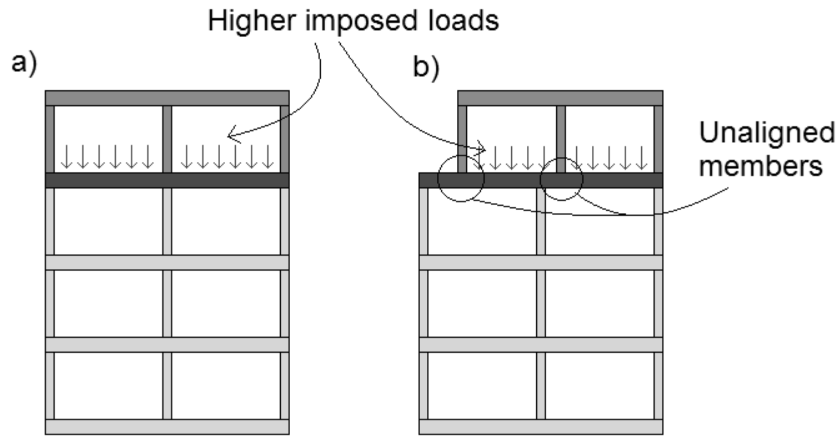

Figure 8.1 New demands on roof slabs in case of storey extension.

If a method with new columns or walls below the slab is chosen, these must be placed carefully so that they are aligned with the load-bearing members further down. This method can lead to tensile forces at the top of the roof slab in areas where the slab is designed for a positive bending moment. However, the good accessibility of the roof slab makes it possible to strengthen it with regard to these introduced tensile forces. The high concentrated forces from the columns might also create unwanted stress concentrations in walls or beams that they are placed upon. It is important to ensure that these concentrations will not lead to damages of the underlying members. Strengthening such zones with surface mounted CFRP might for example be an alternative.

Glued CFRP strips or plates on the tensile side can be a material-efficient method to strengthen roof slabs. It is also possible to concentrate the strips to areas with concentrated loading etc. to further optimise the material usage. Surface mounted CFRP builds little extra thickness, which may be an unnecessary advantage in this context where the height in many cases can be increased upwards. This method also requires construction work inside the existing building, which may be avoidable when the top of the slab already lies free. CFRP is quite an expensive material and should therefore be designed wisely to be optimally utilised. Calculations for this method have been executed and are presented in Section 7.2.2.

Near-surface mounted reinforcement works similarly to the glued CFRP strips and plates. However, crossing of bars might be impossible and it is therefore best suited for strengthening of one-way slabs or with complementary surface mounted CFRP. Another factor that may limit the usage of this method is the need for concrete cover thickness in the original slab. The near-surface mounted FRP have superior bond properties and might be an option to consider if there are any problems transferring the tensile forces to the supports. On the other hand, problems with anchorage is more common for higher beams than for slabs. In Table 7.3 it can be seen that near-surface mounted CFRP were more effective for the investigated slab than the surface mounted CFRP.

To add a new system of timber joists or steel beams on top of the roof slab was the method most commonly used in the studied projects to strengthen the roof slab. One benefit with this approach is that the beams can be placed freely to match the layout of the load-bearing elements of the extension. This can be achieved even if the new layout does not coincide with the supports of the slab. The beams can in this way add resistance to the places where it is most needed. One disadvantage might be that the beams can induce high stress concentrations on load-bearing walls if they are few. The calculations described in Section 7.2.4 show that this method is rather effective and that the original slab can be allowed to assist the beams by taking part of the load. This approach is quite straightforward, but requires the extra space necessary for the beams.

To place beams beneath the roof slab seems unnecessary unless the height needs to be minimised due to restrictions in the zoning documents. CFRP appear to be a stronger contender for such situations, since it is easier to install and will not decrease the room height.

Post-tensioning through drilled holes can be a rather material effective way to strengthen a slab, as described in Section 7.2.7. However, this method seems unnecessarily complicated for the roof slab. It can instead be more motivated to choose a method that uses the available space above the slab. The greatest setback with this method is the time and effort required to drill all the inclined holes and anchor the tendons. Prestressing is mostly suited when there is a desire to reduce the deflection of the slab, but even for such cases other options should be evaluated.

Section enlargement on the compressive side can be quite simple to perform in this case, since the slabs often are completely free and available for casting after the removal of the roof structure. Increasing the concrete thickness will, apart from improving the load-bearing capacity, also add to the performance regarding noise penetration. Since the roof structure most often has not been designed for noises originating from activities above, the additional thickness might help fulfil new noise demands. This approach was tested in the calculations presented in Section 7.2.6. The results show that a rather thick layer is required, since the reinforcement was designed to yield in the ultimate limit state already for the original load case. Therefore, the weight increase that follows the additional concrete becomes rather substantial. This can give severe consequences for the underlying structure.

Furthermore, it can be argued that section enlargement is best suited when it is the distributed loads on the slab that increases, since the capacity is increased all across the surface. If concentrated loads are horizontally shifted by the slab, a more local strengthening method might be more appropriate, for example such acquired by beams or CFRP strips.

Section enlargement on the tensile side is quite complicated, since it is mostly needed on the bottom side of the slab. This method includes casting around new reinforcement bars and it is, as mentioned in Chapter 6, only the reinforcement bars that provide the extra capacity. It might also be difficult to anchor the reinforcement at the supports without any extra measures, since the support itself is in the way. This principle is illustrated in Figure 5.6. Today there are more convenient ways to increase the tensile capacity of a slab.

Filling hollow cores inside a slab can be an alternative when the height of the slab cannot be increased further. A requirement for this method to be applicable is the use of hollow core slabs. However, an increased thickness should normally not be a problem for the roof slab since the room height above can be planned according to the new thickness. Therefore, this method seems unnecessarily complicated and ineffective for this case.

8.5.5.2 Lack of moment capacity in slabs on other storeys

If a slab on an intermediate floor, here defined as any slab not being the roof slab, requires strengthening due to storey extension, it is because the load from above is horizontally shifted at that level, see Figure 8.2. This generates a similar but yet different situation than in the case with the roof slab. Even if the same strengthening methods could be used, their suitability may differ.

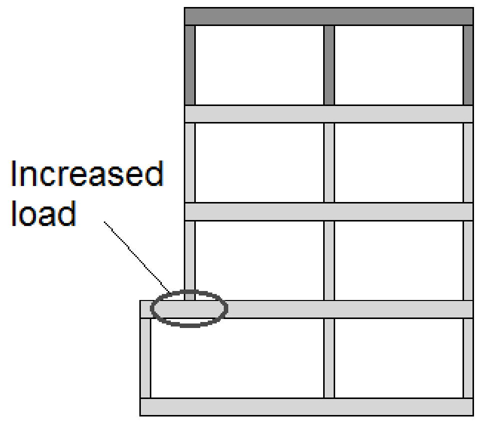

Figure 8.2 Situation when slab on lower floors can be affected by the extension through a horizontal shift).

To add new columns or walls beneath the concentrated load would in theory be the most straightforward solution. However, the activities on the lower floor would in many cases be disturbed by the new elements requiring alternative approaches. If the height of the storey is enough, it might be possible to shift the load by beams and thereby allow the additional columns to be placed in more suitable positions. When placing new supports beneath the slab, it must be considered how the behaviour of the slab is changed.

Glued CFRP strips might be a good solution in this case, since their small thickness minimises the effect on the room height. The high costs of CFRP encourage a well worked design. Near-surface mounted FRP might be a possible option to further reduce the material usage, as long as there is enough concrete cover. Fire protection of the CFRP might however be necessary and some extra thickness should therefore be accounted for.

To use steel beams for this case is probably not a suitable option. As with the roof slab, placing beams beneath the slab reduces the room height. These beams are also quite cumbersome to install. Other options are therefore more suitable.

To use post-tensioned tendons placed in drilled holes is still a complicated method, but the inapplicability of several other methods improves its relevance for this case. Prestressing of members is as mentioned however mostly suitable when a reduction of deflection is desired.

Section enlargement on the compressive side might not be as easily performed as for the case with the roof slab. Firstly, any flooring inside the building must be removed. Secondly, it might be hard to get a capacity increase in the most critical section, namely directly beneath the columns or walls where the load is increased. The increased weight that follows may instead pose a problem. The advantage about noise reduction cannot be taken into account in this case, since a slab of this kind already should be thick enough to satisfy the sound demands. Section enlargement on the tensile side shows the same pros and cons as for strengthening of roof slabs in Section 8.5.5.1.

Filling of hollow core slabs might be an alternative to consider if there are strict limitations to the height. The accompanying increase in shear capacity might be an additional factor that may favour this method. However, this approach is quite cumbersome and the material is not optimally utilised.

8.5.6 Lack of shear capacity in slabs

The shear capacity of slabs is, as described earlier, most critical near supports or localised loads, such as concentrated or line loads. The type of slab is of relevance when selecting appropriate methods, since not all methods are applicable for hollow core slabs. When shear strengthening slabs, the following strengthening methods can be relevant:

- Section enlargement
- Filling of cores in hollow core slabs
- Post-tensioned vertical bolts through the whole slab
- Undercut anchors from one side of the slab
- Vertical CFRP bars
- CFRP strips in closed loops through holes
- New supporting columns or beams beneath the slab

Section enlargement will improve the capacity with regard to shear. This approach is suitable for both solid slabs and hollow core slabs, since only the exterior of the slab is affected. As mentioned in the previous section, filling of the cores of a hollow core slab is a method to increase both the shear capacity and bending moment capacity. If the shear force in the web of the hollow core slabs is critical, this approach can be suitable. It is however an approach that requires a lot of work, especially if an extra layer of concrete for sound regulations has been cast upon the hollow core slabs. It should be observed that when hollow core slabs are designed, unnecessary material is removed, hence adding material here will not optimally utilise it. Knowledge of where the hollow cores are located is also a prerequisite, but should be possible to attain at the site if not available in any documents.

The method with post-tensioned bolts seems quite simple and straightforward to carry out, even if the positions of the bolts must be investigated thoroughly. However, it requires work from both sides of the slab. If the slab is intermediate between two storeys, the flooring must be removed to reach the surface of the slab. In the same manner as for ordinary prestressing, the relaxation in the steel bolts might be a problem. One way to minimise relaxation might be to use low-relaxation steel. It may also be possible to retighten the bolts after a while.

To use undercut anchors removes the problem with work on the top surface of the slab, which can be very beneficial. Provided that enough anchorage capacity can be achieved, i.e. the slab has sufficient thickness, this method should be more suitable.

The method with CFRP bars seems quite unfavourable in a slab, since the height of the slab often is relatively small. The bars are not anchored at the surface and must therefore gain their anchorage along the height of the slab. Even if CFRP bars are very stiff, this might be a disadvantage. Closed loops of CFRP remove the problem with anchorage length, but the problem with work on both sides of the slab reappears. This method is quite interesting and may come in question in some situations. However, it seems to be a quite expensive alternative.

Adding additional supports directly below the columns/walls may as with the roof slab directly solve the problem of strengthening. Unfortunately, it is most often not a possible solution. However, if the strengthening becomes too extensive, it might be necessary to reconsider this possibility.

9 Conclusions

Some final remarks on the project are presented in this chapter. It is discussed how well the results correspond to the expectations and how different approaches could have resulted in a better product. Finally, some recommendations for further development are presented.

9.1 Comments on the result

The result of this report is mainly presented in Chapter 8. In this section, the key aspects are highlighted and commented.

Even if each storey extension project may seem unique with very set parameters, this report shows that it is possible to draw general conclusions that are applicable on a large number of storey extension projects. However, the set parameters of the building need to be taken into account since they may greatly affect the appropriate approach.

During a storey extension project, it is important to not only focus on strengthening of the old superstructure and the extension itself, but also how the surroundings and other key aspects may affect the project. Knowledge of these issues may reduce the construction time as well as the cost.

It is also of importance to take the existing building and its users into consideration. The function of the building can also impact the choice of strengthening. This may seem quite obvious, but it is good to remember that certain strengthening measures are better suited for specific situation. This aspect is treated in this report, but there might be issues that have been overlooked or issues which impact might have been misjudged.

It can also be highlighted that structural members can be strengthened in several ways, more or less suitable for different situations. To determine the best approach for a specific case is however not a straightforward process. Instead, various parameters and their affects need to be evaluated. Some examples are that CFRP wrapping only is suited to strengthen stocky columns while vertically mounted steel plates can strengthen slender columns. Section enlargement and additional steel profiles can be used in both situations, provided that the strengthening is carefully executed. When it comes to simply supported slabs, it is often a good solution to add beams on top of the slab, provided that it is possible to increase the height. To cast a new layer of concrete on top of the slab reduces the sound penetration, but is for the most slabs an inefficient approach. To use CFRP laminates is a useful method if the height and weight of the slab should be limited and it is in this case advantageous to use near-surface mounted laminates if possible. It is also possible to use post-tensioned steel strands to strengthen the slab, but this method is regarded to be rather cumbersome in comparison to the others.

9.2 Importance of the project

The aim of this project was primarily to create guidelines that can help the designer in a storey extension project. Strengthening of structural systems and members is today a rather common problem for civil engineers. However, the subject is not treated thoroughly in the education. This, together with the lack of design handbooks in the topic, is something that argues for this project. It can be beneficial for the designer to be introduced to crucial aspects and possible solutions in a very early stage, especially if he or she is relatively inexperienced. Otherwise, some promising solutions or critical issues might easily be missed.

Throughout the interviews it was discovered that several of the designers were quite certain on which strengthening method that is best suited for a specific member. However, different designers sometimes favoured different solutions. It may therefore be important for the designers to be aware that other strengthening methods than the one that they normally use can be more suited for some specific situations. It might for some designers also be good to get an introduction to the pros and cons of newer strengthening methods, especially different variations of fibre reinforced polymers.

By reading this report the designers can also get a better understanding of which members that most often are critical and which members that often have excess capacity in different types of buildings. Even if it might be rather easy to estimate which these members are for a specific building, it can be time saving to have an understanding of what to look for and where to search.

Another important lesson that designers can learn from this project is that it is very important to plan a storey extension project carefully, maybe even more than for a normal project. It is important to collect as much data as possible before any important decisions are taken. Better knowledge and understanding lead to better suited and more cost-effective solutions. It also, to some extent, helps to avoid last minute changes.

Finally, this report will hopefully also generate some new interest for storey extension projects. With regard to structural issues there is a great potential to vertically extend many of the existing buildings. In densely populated areas the extra costs and inconveniences can often be compensated by the ability to further exploit attractive sites.

9.3 The method used in the project

This project could have been carried out in many different ways. Looking back, some other methods could have been useful. It might as an example have been good to investigate a specific building more thoroughly and design the strengthening for that particular case. The project might in that way have gotten a sharper edge. However, it can also be argued that the chosen method, to investigate many executed projects, gave a wider view of the topic. One of the most important parts of the project might be all experiences that were gathered from the many interviews. This information would not have been as extensive and varying if only a specific building had been studied in detail.

Initially, the main objective was to establish a step-wise tool, where the designer would have been guided to a suitable choice with critical details and problems to consider. Unfortunately, the vast range of specific situations in storey extension projects together with limited time resulted in that this part was revised. It was soon discovered that it would have been hard to develop such a tool with desired properties. The guidelines are instead provided as an integrated text where reference is made to different sections in the report. It would of course have been easier for the designer to just use an interactive tool than to search for information in a rather large document.

A number of calculations have been executed throughout the project to help evaluate and compare the different strengthening methods. With the results at hand, it should have been quite easy to estimate the outcome of several of the calculations beforehand. As an example, it may seem rather fundamental that it is unsuitable to improve the bending resistance of a rather stocky column loaded by a centric normal force. On the other hand, some designers may also benefit from the more simple conclusions of the calculations. The calculations also pinpointed some important issues that otherwise could have been overlooked when the strengthening methods were compared. It would have been good to compare more types of structural members by calculations, but the choice to only treat columns and slabs seems quite appropriate.

To only focus on concrete structures was quite a natural choice, mainly because a majority of the larger buildings in Sweden are concrete structures. This choice greatly affected the possible strengthening methods, but treating even more materials, such as steel or timber, would not have been feasible within the scope of this project.

9.4 Further studies and development

As described earlier the topic of strengthening buildings due to storey extensions is very vast. This project therefore only covers a small part of the subject. To build a new structure on top of an old brings many issues that differ from a regular project, so it is beneficial if the designers have access to as much important information as possible.

The most prominent thing that can be investigated more thoroughly is experiences from executed projects. It would be good for the designers to have access to an extensive collection of buildings with storey extensions. The main information about structural system, used strengthening methods and important experiences could be stated for each building. If this collection of projects was large enough, and well organised, the designer could quite easily find one or several examples that were similar to the project at hand. It could also be motivated to add information about strengthening due to other things than storey extensions to such a database.

As described in the previous section it would also be of value to investigate specific buildings more in detail and thereby examine the pros and cons with the different strengthening methods for the specific cases thoroughly. One approach could be to design the extension and needed strengthening for two different buildings, e.g. a hotel and a residential building, and analyse which strengthening methods that are best suited for the different cases. It could also be relevant to compare the total costs between the approaches more thoroughly.

Further studies can also be of relevance in some of the fields that only were investigated to a small extent in this project, e.g. strengthening against horizontal loads and strengthening of the foundation. Especially, it would be of value to let a civil engineer with a geotechnical background compare different approaches of how to strengthen (or avoid to strengthen) foundations.

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Appendices

Appendix A – Interview questions

Appendix B – The studied projects

Appendix C – Fire regulations

Appendix D – Calculations for strengthening of columns

Appendix E – Calculations for strengthening of one-way slabs

Appendix A – Interview questions

A number of interviews have been carried out during this Master's thesis project. When a meeting was difficult to arrange, phone interviews or mail correspondence has been used instead. This appendix lists the questions that were treated at the interviews concerning the investigated reference projects. It should however be noted that not all questions were asked during all interviews, but were adopted partly depending on what the project was, but also the role of the interviewed person. Many of the questions were also complemented with follow-up questions in the context of a dialogue. These are not presented here.

The original structure:

- Can you describe the structural system of the existing building?
- How did you check the state of the existing building?
- Were there any substantial damages to the building?
- Did you follow the Eurocodes or the old Swedish design codes, BKR? What is your opinion of applying new codes to older buildings?
- Which members of the original structure were most critical with regard to the load increase?
- Did any of the members have any surplus of capacity?
- Did you use any special methods to gain capacity during the calculations, assumptions for example?
- Was the structure strengthened? Where, and for what?
- How was the building stabilised with regard to horizontal loading?
- When was the original building erected, how has it been maintained since? Any notable reconstructions?

Foundation:

- How were the geotechnical conditions on the site?
- How was the building founded?
- Was any strengthening of the foundation required? Which method was used and why was this method chosen?
- How good would you say that the geotechnical conditions would have to be to promote a storey extension project?

The extension:

- How is the load-bearing system of the extension designed?
- Why did you choose this system? Were you limited in any way in that choice? Looking back, would you choose differently?
- What is (roughly) the weight of the extension, would it be possible to reduce it further? If so, how?
- Were there any details that were especially difficult to design?
- How was the building anchored to the existing building?
- What was it that promoted a storey extension project? Was there anything that was against it?

Miscellaneous:

- What sort of activities are there in the building? How does this affect the structure?
- Have the activities changed? If yes, how does this affect the project?
- What was the main reason for performing the storey extension? Any sub-reasons?
- Did you need to do any changes in the zoning documents? Any problems?
- Do you have any contact information for other persons involved that may further help us in our research?
- What services were required along with the storey extension (elevators, storage space, parking lots etc.)
- Was it possible for the residents to remain in the building during the construction process? To what degree?
- Are you satisfied with the result? Anything that exceeded your expectations? Any unexpected problems? What would you do differently in the future?

- Will you perform any new storey extension projects in the future, why? (Why not?)
- Do you have any knowledge about other storey extension projects that have been carried out recently?

Appendix B – The studied projects

This appendix forms the basis for Chapter 3, where the studied projects are presented. The information provided here is mostly gathered from interviews with persons involved in the specific projects. Some complementary information has however been collected from the Internet. Complete information has however not be identified for all projects.

The information collected from the oral interviews has been interpreted with the best intent, but as with all interviews, there is a risk for misinterpretations. It can therefore not be guaranteed that all information provided here is entirely correct.

The following projects are treated:

Hotel - Gothia Central Tower

Interviewed persons:

- Erik Samuelsson, Designer, VBK (2013-01-24)

Introduction

Gothia Central Tower was built in the vicinity of Korsvägen in Göteborg in 1984 and was 62 m high with 18 storeys. By adding six extra floors, the new building reaches 83 m.

Existing building before storey extension

Load-carrying structure	Mostly cast in-situ concrete with some minor steel columns at the façades. The general layout of the building is visible in Figure B.1. Big central core as stabilising system. Load-bearing walls between hotel rooms on most of the floors, except on the lower entrance and conference floors where concrete columns are used instead. In-situ cast concrete slabs.
Examination and condition	The condition was very good and the building stood straighter than what was assumed in the original design.
Calculations and design codes	The old Swedish design codes were used since the project was started just before the rules changed. The Eurocodes use higher values for the load cases, so Samuelsson said that the same number of storeys wouldn't be possible today. By assuming urban environment for the terrain category, the wind load could be reduced compared to when the old building was designed. The unintended inclination could also be lowered since it was found out that the building stood very straight. These two advantages resulted in that the total moment from the horizontal forces didn't change significantly even after the addition of the extension. The normal force was however increased.
Critical members	The columns on the lower floors were one of the reasons why the extension stopped at six floors.

Strengthening	Beams that were subjected to point loads from the columns needed to be strengthened by carbon fibre reinforced polymers since the new columns were placed near the edges of the beams. This was done mostly to be on the safe side. The edge of the beam was wrapped in a sheet of CFRP that spreads the load into the beam, see Figure B.2. Some slabs were strengthened with CFRP since new columns were placed beneath the slabs, thereby creating tension in the top surfaces of the slabs.

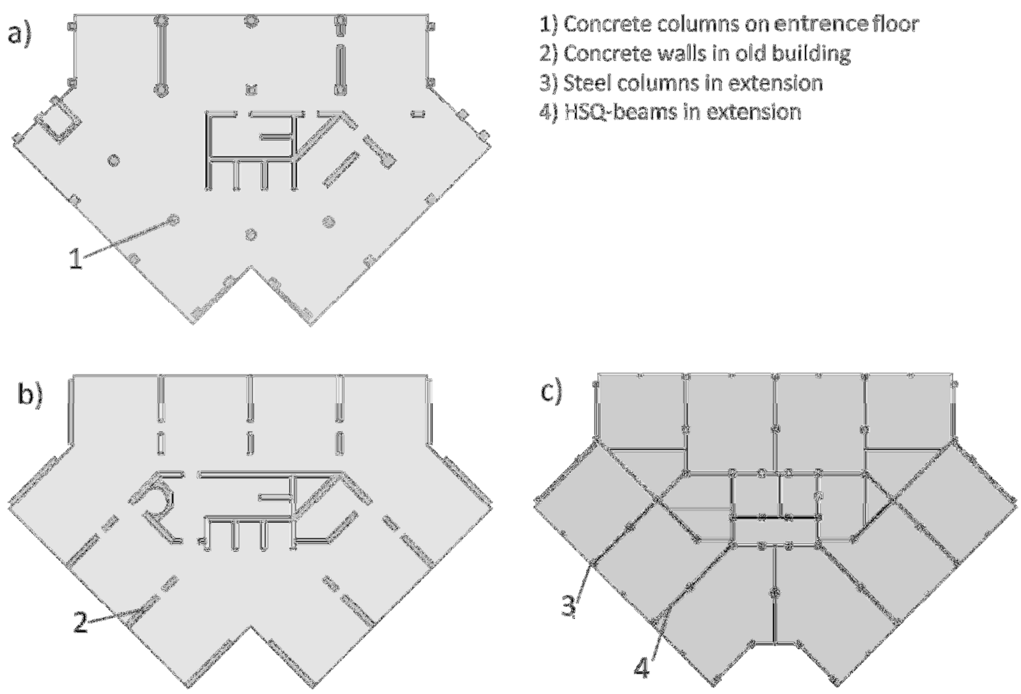


Figure B.1, Simplified illustration of plans in Gothia Central Tower; a) entrance floor, b) upper storey in old building, c) storey in the extension.

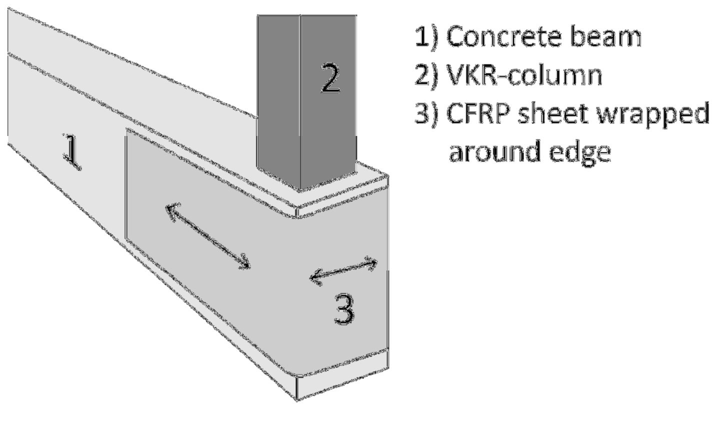


Figure B.2, CFRP-strengthened beam beneath a new column.

Soil and foundation

Soil conditions	Thin layer of clay above bedrock
Foundation	Founded mostly on plinths on top of the bedrock. Short end-bearing piles beneath one part of the building.
Strengthening	Nothing
Settlements	During the surveying, it was observed that the top of the tower was about 10 cm lower than according to the drawings. Some of this was probably due to settlements. This didn't affect the load-carrying system to any higher extent except that the connecting bridges between the towers needed to be adjusted.

The extension itself

Storeys and activities	Six new floors. They wanted to add more, but the columns on the entrance floor made it hard. The layout can be seen in Figure B.1. The added floors contain luxurious hotel rooms and a restaurant.
Load-carrying structure	Steel columns and HSQ-beams. Hollow core floor slabs with an extra in-situ cast layer of concrete and double ceilings with extra insulation to meet the high sound demands.
Alternative solutions	The choice of structure was rather limited by the layout of the underlying load-carrying walls. The choice of columns also admitted a bigger range of possible room layouts. A larger number of columns would have spread the load to the underlying walls better, but would have restricted the layout. A timber structure may have been possible, but the designers didn't have so much experience of timber structures. According to Samuelsson, it might have been hard to fulfil the sound demands with a timber structure.
Difficult details	The connection between the two cores. The new core is made of steel with diagonal ties to handle the horizontal force. The new core is connected to the existing concrete core by long steel plates (100x20 mm^2) that extend several storeys down where they are anchored. The plates are prestressed to ensure that they are activated at once. A rough sketch of the core and the steel plates can be seen in Figure B.3.

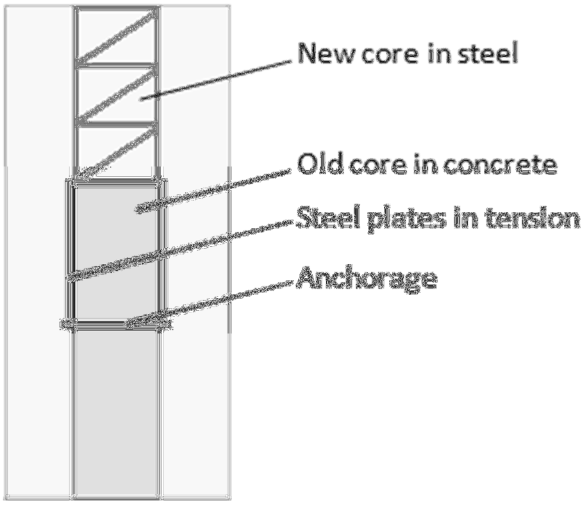


Figure B.3, Simplified sketch of the reinforced bracing in Gothia Central Tower.

Other issues

Extra constructions	A new panorama elevator along one corner of the tower. New bridges between the towers. The second highest floor in the old building was strengthened since it was changed into a spa.
Fire regulations	No major affects on the project. New bridges to the adjacent towers were installed, which facilitates the evacuation. A new elevator was installed, but that was not due to the fire regulations. According to Samuelsson, one of the elevators might have been strengthened due to fire demands.
Higher extension	The existing structure cannot take any more floors without major strengthening. Strengthening the columns on the ground floor with carbon fibres might have enabled further extension.
Important considerations	Careful surveying and inspection in a very early stage is important, since the existing structure not always is built exactly according to the drawings. When prefabricated elements are ordered, it is vital that they fit to the load-bearing structure so that loads are transported straight down through the members.

Hotel - Scandic Opalen

Interviewed persons:

- Erik Samuelsson, Designer, VBK (2013-01-24)

Introduction

The hotel Scandic Opalen was built in the beginning of the ’60s at the junction of Engelbrecktsgatan and Skånegatan in central Göteborg. The original building had eleven floors, but five extra were added in 2009.

Existing building before storey extension

Load-carrying structure	In-situ cast concrete with load-bearing walls between the hotel rooms, except on the lower entrance and conference floors where concrete columns were used instead. The walls are thinner than in Gothia Towers, but still over-dimensioned for the vertical loads. In-situ cast concrete slabs. Layout for the original building and extension can be seen in Figure B.4. The horizontal loads are first taken by the walls on each floor, but on the way down to the ground, they must go through the stabilising gable walls or the elevator shafts.
Examination and condition	They had to drill holes through the slabs to be able to see where to place the columns to get them straight above the old walls. This was due to differences between the drawings and the reality. The structure was otherwise in good condition.
Calculations and design codes	The old Swedish design codes were used since the project was started before the rules changed. The Eurocodes use higher values for the load cases, so Samuelsson said that the same number of added floors wouldn’t be possible today.
Critical members	The columns on the lower floors were one of the reasons why the extension stopped at five floors.

Strengthening	The tensile capacity in the gable walls was almost enough, but it was decided to strengthen them to be on the safe side. This was done by attaching steel plates along the gables and anchoring them in the foundation, see Figure B.5. Unlike in the Gothia Towers project, the plates were bolted into the slabs on each floor. The plates were only applied on the outer side of the gable walls even if it would have been better to place plates on the inner side of the wall as well to achieve more uniform loading. This was omitted since holes would have been needed to be cut in all slabs. The number of needed steel plates was rather small since the lever arm in the gable walls was quite large. Columns were placed inside the old ventilation room in the top of the old building since the slab wasn't built to carry imposed loads from above. These columns were rather difficult to place since they needed to be positioned between the installations.

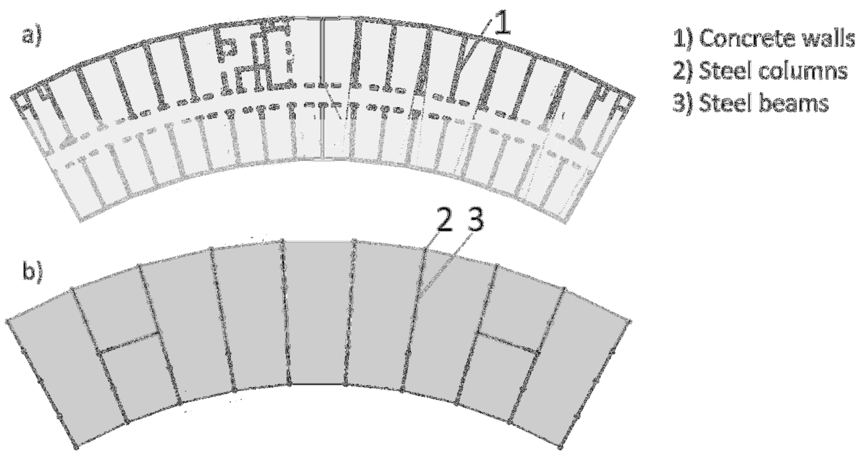


Figure B.4, Illustration of plans in Scandic Opalen: a) storey in original building, b) storey in extension.

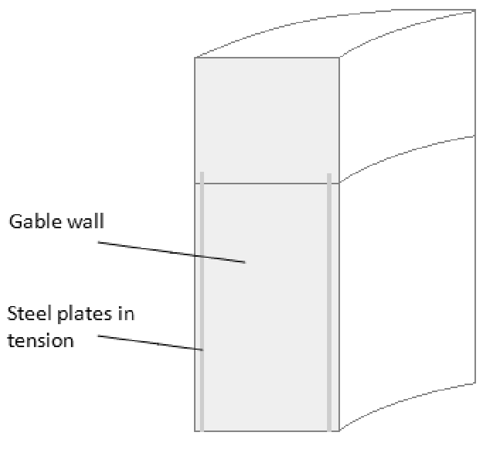


Figure B.5, Simplified sketch of the reinforced bracing in Scandic Opalen.

Soil and foundation

Soil conditions	Clay over bedrock
Foundation	End-bearing piles that rest on the bedrock. The capacity was found to be well above the limit except along one of the gables where the capacity was too low.
Strengthening	New end-bearing piles were driven to bedrock on both sides of the poorly founded gable. The gable wall was then anchored to the piles. If the whole building would have been equally poorly founded as the gable, an extension would have been almost impossible according to Samuelsson.

The extension itself

Storeys and activities	Five storeys for hotel rooms and one half-floor for installations directly above the old installation room.
Load-carrying structure	Steel columns and beams. Hollow core floor slabs with an extra in-situ cast layer of concrete to meet the sound demands. Cross-bracing walls were built above the gable walls and elevator shafts as bracing for the extension.
Alternative solutions	The aim was to make the extension light without interfering with the sound demands. Samuelsson says that the concept felt as a reasonable choice.
Difficult details	To adapt the prefabricated structure to the actual placing of the existing walls. To place the columns in the installation floor.

Other issues

Extra constructions	An extra half-floor for the installations. Faster elevators replaced the old ones.
Fire regulations	No major effects. There are staircases at the gable walls plus elevators in the middle of the building.

Higher extension	The old structure wasn't able to carry much more than the added floors without large strengthening efforts.
Important considerations	As was mentioned in the previous section, it is very important that the designer is active early in the project to be able to control the load so that it passes straight down into the load-bearing walls in the old building. Early surveys are vital so that the prefabricated structure fits to the real building.

Hotel - Scandic Rubinen

Interviewed persons:

- Dan Jarlén, Designer for the extension, VBK, (2013-03-13)

Introduction

Scandic Rubinen is a hotel at Kungsportsavenyn in central Göteborg. The original building was built in the 1960s and the storey extension will be finished in 2014.

Existing building before storey extension

Load-carrying structure	Different parts of the building have different height. The part that is going to be extended had originally at the time of writing three storeys above ground plus a basement. Another part of the building is significantly higher. The structure consists of columns (spacing 12 m) and beams that were cast in-situ. On top of the beams lie prefabricated TT-slabs. The original roof consists of a thin in-situ cast slab with a trapezoidal shape and underlying steel ties.
Examination and condition	No major damages were found. The designer claimed that in-situ cast concrete structures often are solid and in good shape.
Critical members	Two columns in the basement, TT-slabs beneath the new patio and some columns beneath these TT-slabs. See Figure B.6 (1, 4 and 6 respectively).
Strengthening	The rectangular columns (1 and 6 in Figure B.6) were strengthened by two HEB-profiles that were applied on the opposite sides of the columns. This increased the strength against buckling and crushing. The designer claimed that it is simpler and more effective to strengthen with steel profiles than a new layer of concrete. A new layer of steel beams were placed over the TT-slabs beneath the patio (4 in Figure B.6). In this way, the TT-slabs were unloaded. Along the façade-line (2 in Figure B.6), new steel columns were placed at two levels with a spacing of 4 m. These supplement the existing concrete columns with a spacing of 12 m.

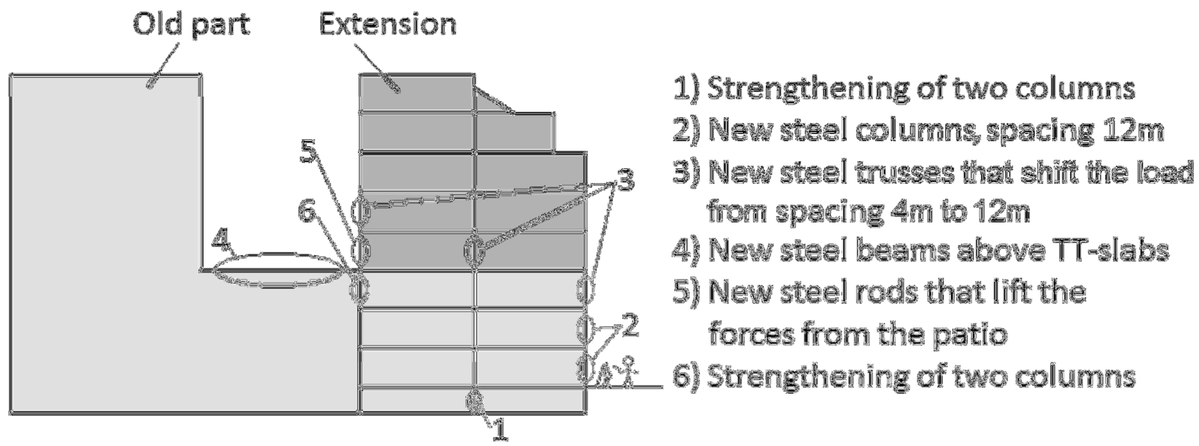


Figure B.6, Simplified sketch of a section through Scandic Rubinen.

Soil and foundation

Soil conditions	Bedrock quite near the surface.
Foundation	Large concrete piles that were cast in-situ. The piles have a diameter of about 2 m and reach down to the bedrock.
Strengthening	There was no need to strengthen the piles.

The extension itself

Storeys and activities	The extension will give five new storeys. The main activity is hotel rooms, but the highest floor will contain installations.
Load-carrying structure	Steel columns and HSQ-beams with hollow core floor slabs. The slabs have an extra 60 mm in-situ cast layer to improve the noise resistance. The steel columns stand with a spacing of 4 m to decrease the height of the HSQ-beams so that they are in level with the top of the slabs. To fit the load-carrying structure to the old columns (spacing 12m), new storey-high steel trusses were needed in the longitudinal direction of the building (3 in Figure B.6). The load from the new steel beams beneath the patio is lifted up to one of the trusses through vertical steel ties (5 in Figure B.6). This method was chosen to avoid building the truss at the same floor as the patio, which would have made the storey less open. The stabilisation will be solved mainly by connecting the structure to the existing building. This could be done since the extension meets the higher part of the old building at one end. Additional

	bracing also comes from a prefabricated gable wall and a truss.
Difficult details	It was quite difficult to design the trusses so that doors could be placed in the wall.
Weight	About 550 kg/m^2 and storey.

Other issues

Important considerations	The designer said that it is better to strengthen rectangular columns with steel profiles than adding a new layer of concrete. He was not sure that strengthening with carbon fibres would be allowed by the environmental certification that the hotel wants to get.

Office building etc. - Bonnier's Art Gallery

Interviewed persons:

- Alf Skelander, Designer, ELU (2013-02-12)

Introduction

Bonnier's Art Gallery is located in central Stockholm and was finished in 2006, ELU (2013). The house is situated on top of an old three-storey building and contains both an art gallery and offices. The project required major strengthening of the existing structure.

Existing building before storey extension

Load-carrying structure	The old building lies in a steep slope. This means that all three storeys are visible at one side of the building while the road on the other side of the building is in level with the roof of the old structure. The structure consists of in-situ cast concrete columns and slabs.
Examination and condition	To check the concrete strength, samples were taken from the existing structure, especially from the columns. No major damages were found, but some of the columns on the first floor had minor damages since that floor had been used for parking.
Calculations and design codes	BKR 2003 was used for the new structure and strengthening.
Critical members	Many of the columns were too weak for the extension. No members had any major excess capacities.
Strengthening	Many columns were strengthened by addition of an extra layer of encasing concrete since the load was increased drastically.

Soil and foundation

Soil conditions	Bedrock.
Foundation	Plinths on bedrock.
Strengthening	There was no need to strengthen the plinths. However, new drilled steel core piles were used to improve the foundation at the places where new steel cross ties were needed for stabilisation. Refer to Figure B.7 for layout of building and locatation strengthening measures.

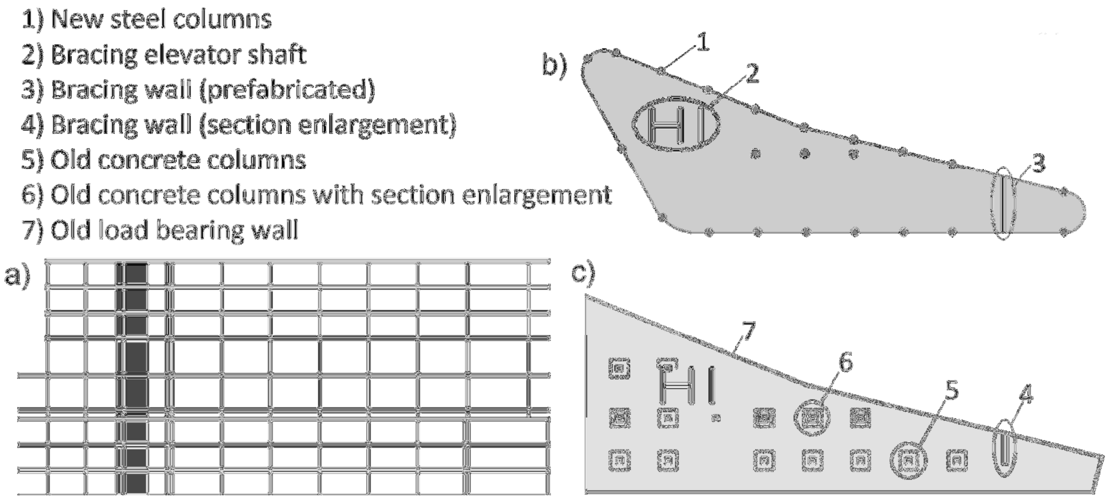


Figure B.7, Simplified illustration of Bonnier's Art Gallery; a) section, b) plan of new part and c) plan of old part.

The extension itself

Storeys and activities	Five new storeys where the first two floors contain an art gallery while the remaining levels hold offices.
Load-carrying structure	Steel columns and beams with hollow core floor slabs. The stairwells consist of prefabricated concrete.
Alternative solutions	The designers were restricted to prefabrication from the construction management. The choice of structural system was according to Skelander quite easy since the chosen method is very common today. Aspects such as time, money and lack of space were also considered.
Difficult details	The stabilisation was a critical issue. This was solved partly by a new stairwell that was clamped into the existing structure. The gable at the point of the triangular shaped building also needed to be braced with a concrete wall.
Weight	The weight of the added superstructure is about 4500 kg/m^2, which Skelander thinks is about as low as they could get.

Other issues

Extra constructions	New elevators and installations were required. The installations for the whole building lie in the original building together with garages and storage rooms.
Higher extension	A higher extension would according to the designer lead to; Need of fire classed stairwells (TR1 or TR2), that the stabilisation would be more complex and that the foundation probably would have needed to be strengthened.

Special solutions etc.	The load-carrying beams were constructed as welded box girders that also function as ventilation shafts. This solution eliminated additional ventilation tubes but increased the size of the beams significantly. The oversized beams in their turn gave the result that almost no fire proofing of the beams were needed. Skelander however thinks that installations generally not should be incorporated in the load-bearing structure.
Important considerations	The designer thinks that it is important to be careful if a new design code is applied on an old structure so that the loads are treated correctly. The Eurocodes use another way to apply safety factors than BKR, which can lead to misunderstandings. Skelander claimed that older buildings of this type seldom have major excess capacities.

Office building - HK60

Interviewed persons:

- Marcus Bågenvik, Project manager, Atrium Ljungberg (2013-03-14)
- John Jonsson, Designer, Sören Lundgren Byggkonsult AB (2013-04-17)

Introduction

HK60 is an office building in Sickla, Stockholm. It was built in 1962 and has earlier belonged to Atlas Copco. The storey extension was finished in 2013.

Existing building before storey extension

Load-carrying structure	The whole building was cast in-situ. The external walls in the longitudinal direction are load-bearing and inside the building stand two rows of columns with beams.
Critical members	The load-bearing structure was quite strong and didn't really have any critical members.
Strengthening	Every second column in the original building was removed to create more open spaces, see Figure B.8. This led to additional load on the remaining columns. These (rectangular) columns were strengthened by section enlargement. A new, 10-15 cm thick, layer was cast on one side of the columns. To only strengthen one side was chosen because the load was larger on that side. The old roof was removed.

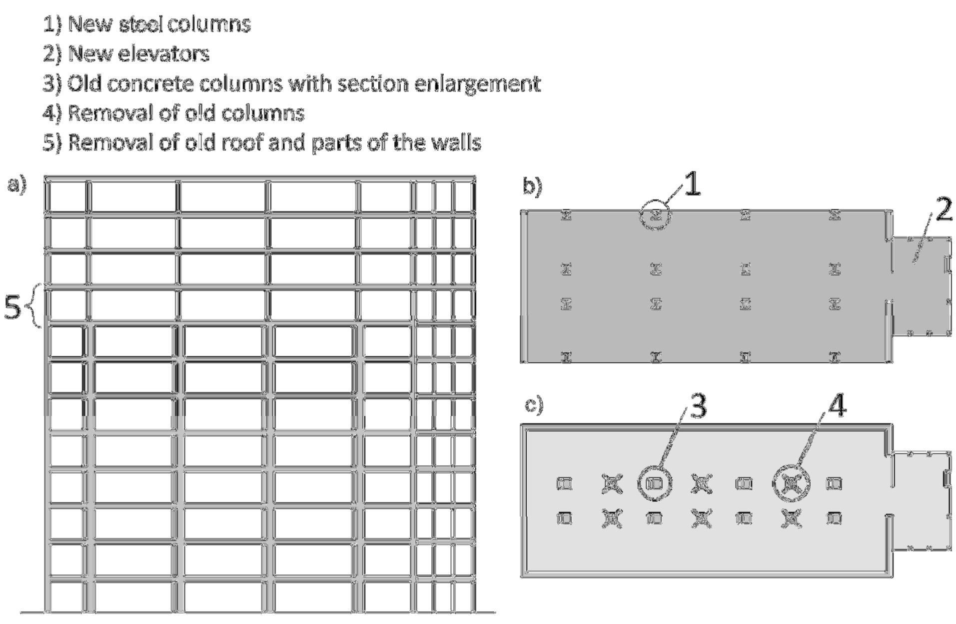


Figure B.8, Simplified illustration of HK60; a) section, b) plan of new part and c) plan of old part.

Soil and foundation

Soil conditions	Bedrock quite near the surface.
Foundation	Plinths on bedrock
Strengthening	One new plinth was cast between the new elevators. The rest of the plinths didn't need any strengthening.

The extension itself

Storeys and activities	Four new storeys containing office areas.
Load-carrying structure	Steel columns and HSQ-beams with hollow core floor slabs. The columns were placed with a spacing of 7 m. According to Bågenvik, the designer had found out that the existing walls in the façades can take the point-loads from the columns without strengthening. Bracings in the shape of steel crosses were installed.

Other issues

Extra constructions	Three new elevators were built on the gable of the building. The old elevator shaft is now used for installations.
Higher extension	A higher extension than four storeys was never investigated, but Bågenvik thinks that five storeys would have been possible without major effects.
Important considerations	The project was initiated when the old tenant moved to new offices. This made it possible to renovate the original building and avoid disturbing ongoing activities. The zoning for the area already allowed a higher building, so they didn't need to change the regulations. This was a big benefit for the project.

Residential building - Apelsinen

Interviewed persons:

- Christer Kilersjö, CEO, Eksta (2013-02-05)
- Ida Johansson, Designer, WSP (2013-01-31)

Introduction

Kvarteret Apelsinen is situated in Kungsbacka, about 30 km south of Göteborg. The storey extension has not yet been executed, but is a part of a big renovation project in the area.

The city council of Kungsbacka also wants to densify the city and is therefore positive to the project. Kvarteret Apelsinen is in need of renovation and a storey extension on one of the buildings was considered to be a good way to take the opportunity and densify the area at the same time. Several new buildings are also built in the area in the same project.

Existing building before storey extension

Basics	Built in 1976. 4 storeys with rental apartments.
Load-carrying structure	Load-carrying concrete walls placed mainly in the transverse direction with a spacing of 4 m. There are also some load-bearing walls in the longitudinal direction. The walls take both the vertical and horizontal forces. Concrete slabs. Both walls and slabs are quite thin and sparsely reinforced. Some elements are unreinforced.
Examination and condition	The building was examined by surveying and visual inspection. No major errors were found, just some frost damages.
Calculations and design codes	According to the designer, all calculations on old and new parts of the building were based on the Eurocodes since the magnitude of reconstruction is rather extensive.
Critical members	There was no real weakness in the load-bearing structure. More or less all of the members could take the same increase of load effect. The roof slab was too weak to be loaded by the extension, so beams were placed between the load-carrying walls.

Soil and foundation

Soil conditions	Varying depths of clay above bedrock.
Foundation	2/3 of the house is founded on end-bearing piles, but one end of the house is founded on plinths since the distance to the bedrock is smaller in this section.
Strengthening	Beneath some of the load-bearing walls, the foundation is too weak. This was solved by new piles with lintels that shift the load horizontally since the piles cannot be placed directly beneath the walls.

The extension itself

Load-carrying structure	Steel columns and beams with hollow core slabs since a light structure is favoured. The first plan was to use solid timber walls, but the sound demands (normal for residential buildings) made it hard to find a solution for this alternative. According to the designer, it would be hard to find a lighter alternative than the chosen.
Difficult details	An access balcony is needed to be able to use as much as possible of the area for apartments. The connection between these balconies and the structure was difficult since the building is designed for low energy use. Another difficult matter was how to plan the load-bearing structure so that the columns fit to the load-bearing walls of the existing structure. They had to change their idea about the size and layout of the new apartments.

Other issues

Extra constructions	A new ventilation room in the new attic that serves the whole building. New elevators outside of the existing house. After discussions with the municipality, it was decided that 65 % of the residents in the building get access to the elevators, which means that half of the inhabitants in the old part can use the elevators. There was no need to increase the number of parking lots or waste disposal facilities.
Higher extension	It would not be reasonable to extend further since one extra floor would have required strengthening of many load-bearing walls and a larger part of the foundation. Two floors were also considered as

	a good choice concerning the height nature of the surroundings. According to Kilersjö, adding the second floor didn't affect the economy so much as the first floor, hence building only one floor was not reasonable.
Renovation	The old part is renovated with the aim to lower the energy use and increase the standard.
Residents	The general opinion of the persons living in the existing building is so far positive according to Kilersjö. The residents will be evacuated during parts of the construction time due to the renovations. Afterwards, the rent in the old part will increase with 35 %. However, Kilersjö says that the increase is independent of the storey extension and only depends on the renovation. Persons that can't afford the increased rent will be helped to similar apartments in other areas.
Important considerations	A positive aspect with storey extensions is that much of the essential building services already are routed into the building, which can save a lot of money. A negative aspect is on the other hand that the structure becomes very dependent of the building below. Small changes in the layout can create big problems with the load paths. Another issue is the logistic problems since it is harder to build on already occupied soil. Furthermore, everything needs to be lifted into place. The rules about how to improve the accessibility in residential buildings that are reconstructed can be interpreted in various ways by the city council. This can give great consequences concerning storey extension projects. It is important that the involved consultants and entrepreneurs work close together under good guidance and that everyone understands what strengthening of the foundation etc. means for the project. Kilersjö would recommend better comunications between invovled groups and a better overview over what was going on and how it would affect others.

Residential buildings - Backa Röd

Interviewed persons:

- Cathrine Gerle, Commissioner, Poseidon Bostads AB (2013-02-12).
- Mikael Carlsson, Designer, Byggnadstekniska Byrån i Göteborg AB (2013-03-28).

Introduction

Backa Röd is situated on Hisingen in Göteborg and is a large residential area with rental apartments. It was built during the so called Million Programme (Miljonprogrammet*)* and is today a large subject for renovations. Some years ago, one of the buildings was renovated with the aim to greatly reduce the energy usage. This pilot project went very well, but the owner concluded that it is hard to make such a renovation profitable.

It was then decided that five buildings that are identical to the first one will be vertically extended with two storeys at the same time as they are renovated. In that way, Poseidon will obtain additional apartments that can be rented to the same price as in new buildings.

At the time of writing, the construction work has not yet been started, but the designer has almost finished the drawings.

Existing building before storey extension

Basics	Five buildings built in 1971. Four storeys with apartments that are built around one single stairwell. Each floor contains four flats with three rooms each.
Load-carrying structure	Load-carrying exterior and interior walls that are made of prefabricated concrete elements. Refer to Figure B.9 for more information.
Examination and condition	The main reason for the project is to renovate the buildings that are in bad shape, especially when it comes to energy efficiency and installations.

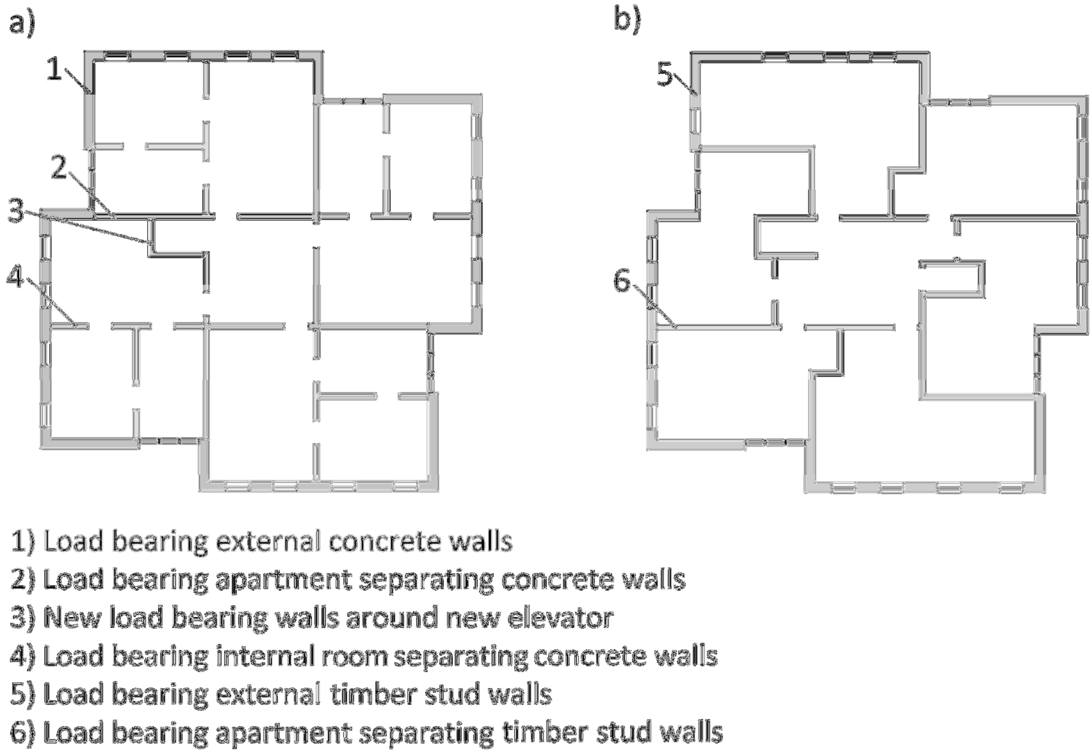


Figure B.9, Illustration of load-bearing walls in the residential buildings in Backa Röd: a) storey in original building, b) storey in extension.

Soil and foundation

Soil conditions	Deep layers of clay. The ground around the houses has settled greatly since the erection due to drainage of the ground water.
Foundation	Founded on piles. The load on the piles has probably increased since the beginning due to the large settlements.
Strengthening	They will probably need to pile beneath the new elevator if this cannot be solved with beams that take the load to the existing structure. The designer hasn't finished the calculations for the foundation yet, but he estimated that the new storeys only will add about 5-10 % on the pile loads.

The extension itself

Number of floors	Two storeys that each contain six two-room apartments. The first plan was to build three-room apartments in the same way as beneath to simplify the structure and installations etc. It was however found out that the new design codes don’t allow the same layout as in the old apartments. Bathrooms and kitchens are however placed above each other to as big extent as possible.
Load-carrying structure	Timber stud walls. The extension fulfils sound class B.

Other issues

Extra constructions	New elevators are installed inside the houses. The space needed for the new elevators is taken from storage rooms inside the old apartments. All installations are replaced to minimise the energy costs. There are also several other actions that are taken to modernise the flats.
Residents	The rent for the renovated apartments increases drastically even if it isn’t affected by the extension itself. According to Gerle, most of the residents have decided to move and are offered precedence in the queue for Poseidon’s other apartments. The ones who decide to stay have to be evacuated to prepared apartments for up to nine months. The general opinion in the neighbourhood is according to Gerle that renovations are needed. Backa is today one of Göteborgs most notorious residential areas.
Important considerations	It took about two years to change the zoning to allow the extension, so it is important to start in time. The rent for existing apartments can only be increased due to higher design code for the residents.

Residential buildings - Glasmästaregatan

Interviewed persons:

- Hans Östling, Commissioner, Bostads AB Poseidon (2013-02-06)
- Mikael Carlsson, Designer, Byggnadstekniska Byrån i Göteborg AB (2013-02-06)
- Jonas Willén, Site manager, NCC (2013-02-06)

Introduction

Glasmästaregatan is situated in the southern parts of central Göteborg at an attractive location where several buildings were built in the mid ‘60s and are in need of renovation.

At the time of writing, the construction work is in progress and some parts of the project have been finished while others still are in an early stage.

Existing building before storey extension

Basics	Two buildings built in 1965. Majorly four storeys with rental apartments.
Load-carrying structure	Load-carrying internal concrete walls that were cast in-situ and some prefabricated concrete columns in the façade. In-situ cast concrete slabs. Non load-bearing prefabricated façade walls.
Examination and condition	No damages that affected the load-bearing capacity even if there were some damages in the façade due to freeze-thaw. The buildings were in need of general renovation.
Calculations and design codes	All calculations were based on BKR since the project was started before the transition to the Eurocodes.
Critical members	Most of the structure was very robust. The roof slab was too weak to be loaded by the extension since the walls on the new floors don’t coincide with the location of the load-bearing walls in the old structure. This was solved by four longitudinal steel beams that shift the load to the walls. The few load-bearing columns might not have managed more than two extra floors without strengthening.

Soil and foundation

Soil conditions	Resting directly on bedrock

The extension itself

Number of floors	Two new floors. When the planning started, building more than two floors of timber was not usual even if it had been allowed for several years. A higher extension was therefore never really considered. It was however found economically reasonable to build two floors instead of one.
Load-carrying structure	Load-bearing walls made of double layers of timber studs with two layers of gypsum boards on each side for sound and fire demands. The choice of structure was based on its low weight and the ability to perform much of the construction work in-situ and thereby minimise the number of times that the weather protecting tent was opened. The steel beams on the old roof slab made it possible to use different layouts for the new apartments, see Figure B.10.
Difficult details	It was difficult to fit the extension to the existing building concerning elevators and installations etc.

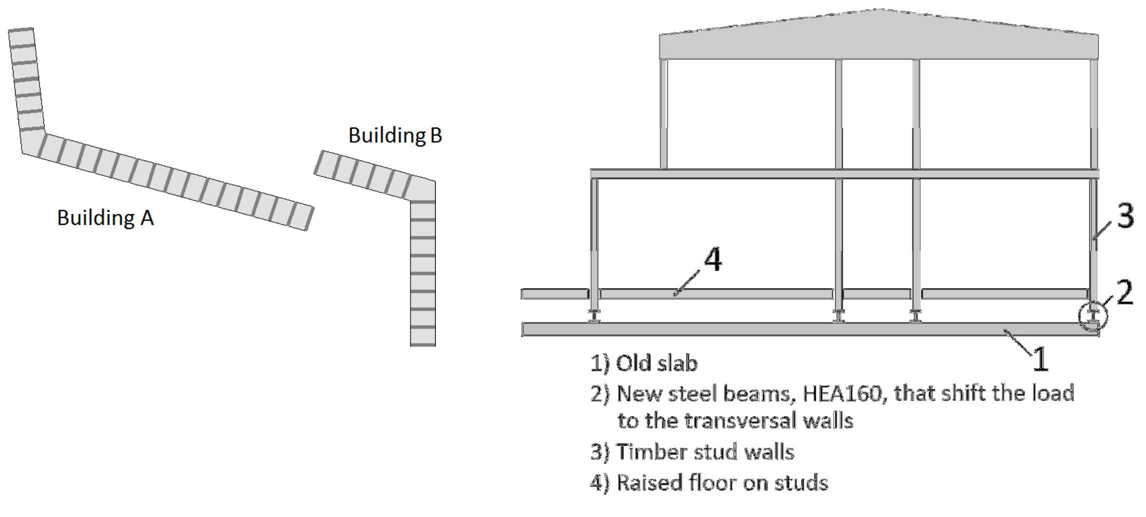


Figure B.10, Illustration of the buildings at Glasmästaregatan; a) rough sketch of the two buildings with load-bearing transversal walls, b) sketch of the load-bearing system for the extension.

Other issues

Extra constructions	After discussions with the city council, it was decided to provide every third stairwell with an elevator. The next third of the stairwells were extended to the top and the last third were left as they were before the extension. New waste disposal facilities are being built in the surroundings. New laundries are being built in the building. There was no need to increase the number of parking lots for cars, but 250 new safe bicycle deposits were needed.
Higher extension	A higher extension was never really considered, but problems might have risen about stability and the strength of the few load-bearing columns. The increase in fire demands that follows additional storeys might also have affected the design.
Renovation	The main purpose of the project was to create new apartments and in the beginning there were no plans of renovation. However, it was later found out that it was advantageous to perform a renovation at the same time.
Residents	The residents remain in their apartments except for 12 weeks when their flat is renovated. During this time they are evacuated to an already furnished apartment. In the beginning, there were a lot of objections against the project and it was appealed to the Administrative Supreme Court of Sweden. However, during time, the general opinion has improved and many are now positive to the project even if there still are some who are displeased. The rent for the old apartments will increase with 23 % due to the renovation and benefits from new facilities. The extension itself does not affect the rent of the old apartments.
Important considerations	The cooperation between commissioner and contractor is very important and in this case a solution with partnering has been successful. According to Östling, the economical benefits with storey extensions compared to new developments cannot counteract the drawbacks. Therefore, the main reason for storey extensions must be the will to densify without development of new ground rather than the will to earn more money. The extension was restricted to a layout with corridors since placing elevators in every stairwell was economically

	unreasonable. Of the same reason, it would according to Willén be more convenient to vertically extend tower blocks. According to the designer, it may have been harder to extend buildings with load-bearing brick walls since the stability of these might be more critical than in concrete buildings.

Residental buildings on garage - Studio 57

Interviewed persons:

- Björn Wibom, Designer, COWI (2013-04-05)

Introduction

Studio 57 is situated in Eriksberg in Göteborg and consists of three residential buildings built on top of a parking garage. The parking garage is relatively new since it was built during the ‘90s. The extension was finished in 2009.

Existing building before storey extension

Load-carrying structure	Columns, beams and slabs were all cast in-situ. Both the beams and the slabs were prestressed with post-tensioned tendons. The slabs were cast on top of a corrugated steel plate so that a composite slab was created.
Examination and condition	No damages that affected the load-bearing capacity were found. Wibom claimed that parking garages from this time are designed much better against de-icing salts etc. than older garages. The prestressing minimises the cracks, which leads to less damages.
Calculations and design codes	All calculations were based on BKR since the project was started before the transition to the Eurocodes.
Critical members and strengthening	Due to a very tight time table, the designer didn’t have time to investigate the excess capacity of the original structure in detail. He knew that it originally was designed to be able to carry one additional storey, but the new extension has up to four storeys. It was then decided not to use the old structure at all, but instead let the extension be carried by new columns and piles. To ensure that the casting of the columns filled the voids, a self- compacting concrete was used. The bracing for the increased horizontal loads was solved by new diagonal steel ties. These ties were needed in quite many places.

Soil and foundation

Soil conditions	About 15 m of clay above an inclined bedrock surface.
Foundation	The original structure was founded on end-bearing piles.
Strengthening	Wibom claimed that it is hard to strengthen foundations with end-bearing piles so that higher loads can be taken. He said that even if new steel piles are inserted and anchored to the pile group, they will not take load before the original piles are deformed. However, he also explained that this could be solved by prestressing the new

	piles, but the lack of time in the design process led to a decision to found the extension on new pile groups instead. In some places, they had to use steel core piles due to the inclination of the bedrock.

The extension itself

Number of floors	Two of the residential buildings have three storeys each while the third has four storeys, see Figure B.11.
Load-carrying structure	Steel columns and HSQ-beams on top of which 45 mm thick prefabricated concrete slabs were placed. An additional layer of concrete was then cast on top of the prefabricated one so that the total thickness became 230 mm. The extensions rest on top of large prestressed concrete beams that shift the load to the new columns which carry the load through the parking garage and down to the new pile groups.
Other alternatives	Timber in the superstructure was avoided due to the demands to design for fire class R60. Wibom claimed that fireproofing a timber structure needs quite extensive work.

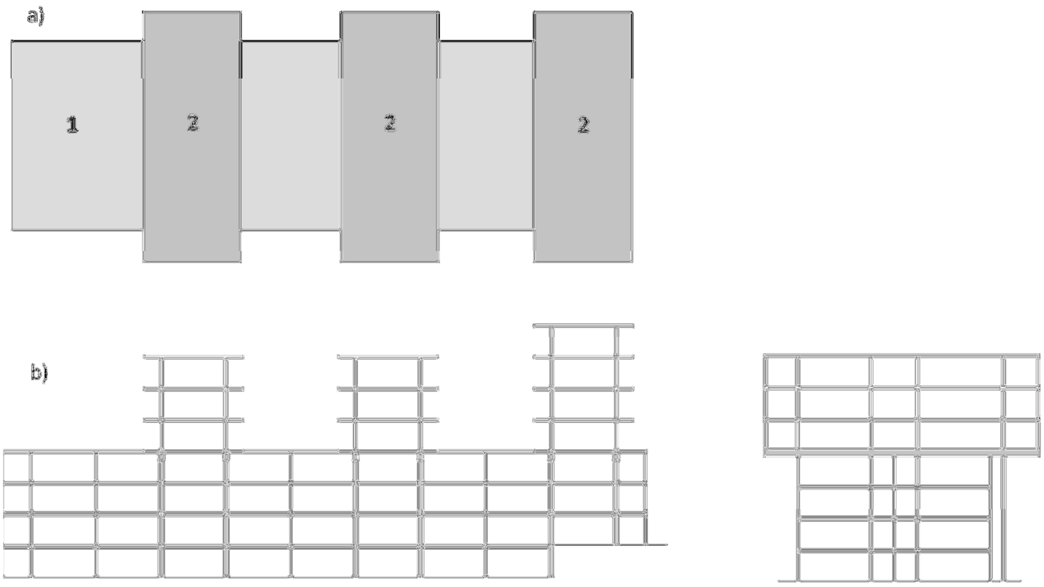


Figure B.11, Simplified illustration of Studio 57; a) overview from above where 1 shows the old garage and 2 shows the extensions b) section in longitudinal direction c) section in transversal direction.

Other issues

Renovation	The parking garage didn’t need any renovation. However, some parking lots were lost due to the new columns and had to be reallocated.
Important considerations	Wibom said that the time planned for design, which was about three months, was way too short. It led to fast decisions and “simple” solutions. On the other hand, he was unsure if more time would have resulted in another solution. He also said that it might have been better if the designer had been contacted in an earlier stage by the client instead of working for the contractor.

Student housing - Emilsborg

Interviewed persons:

- Jan Bergstrand, Designer, Piab (now working at VBK) (2013-03-01)

Introduction

Emilsborg is a student housing complex close to Chalmers University of Technology in Göteborg. A renovation and storey extension of one of the buildings was completed in 2012. The student housing layout is rather simple with internal corridors providing access to many apartments.

Existing building before storey extension

Basics	Built in the early ‘60s. Six to seven storeys including a basement, the building has a banana-like shape with apartments divided into cells along corridors. Opposing apartments are not aligned and give a tooth-like appearance if viewed from above.
Load-carrying structure	The entire building consists of in-situ cast concrete with 150 mm thick load-bearing walls between the apartments. Since they were designed according to sound demands, an unused capacity was available.
Examination and condition	The building was examined by surveying and visual inspection. No substantial damages were discovered.
Calculations and design codes	Calculations were done according to BKR.
Critical members	The interface between the load-bearing walls and the foundation walls. The roof slab also needed some attention.

Soil and foundation

Soil conditions	Inclined bedrock.
Foundation	Founded on foundations walls and concrete plinths.
Strengthening	Below the new elevators, nowhere else.

The extension itself

Load-carrying structure	The walls and slabs were semi-prefabricated. An extra layer was cast in-situ on the slabs and additional concrete was also cast between the two wall-elements. The walls of the extension are 200 mm thick compared to the 150 mm for the original building. The partial in-situ casting enabled a greater adaptability to varying measurements than an entirely prefabricated alternative. Concrete was chosen to get the same appearance as the original building without compromising the sturdiness.
Difficult details	The fire cell division was difficult due to the difference in height between different parts of the building.

Other issues

Extra constructions	New elevators were required due to accessibility. Five existing stairwells were complemented with three elevators so that full accessibility was achieved.
Higher extension	Connections between walls and foundation would need to be strengthened if more storeys were to be added. It should be observed that a quite heavy extension was used here, and the weight could have been reduced.
Renovation	The exterior walls were poor and consisted of 175 mm lightweight concrete with insufficient insulation. The original façade was torn down, insulated and replaced by a new façade. The internal plumbing was also changed.
Stability	The building had very good stability in the transverse direction. Its banana-like shape worked favourable along with the many load-bearing walls between the apartments acting as shear walls. However, in the longitudinal direction, some strengthening was required. This was done by cross-ties close to the elevators.

Student housing etc. - Odin

Interviewed persons:

- Björn Wibom, Designer, COWI (2013-04-12)

Introduction

Odin is a building situated in the vicinity of Göteborg Central Station, and contains today student housing, offices, a supermarket, a hotel and a restaurant. It also has a parking garage in the basement. It was built in 1940, and the extension was added in 2002.

Existing building before storey extension

Basics	A three storey building with a basement. Built in 1940.
Load-carrying structure	In-situ cast concrete columns, beams and slabs.
Examination and condition	The concrete quality in the original building was very poor, around C15-C20. No real damages were detected, but the state of the top floor was not very good. However, this floor was torn down to make a flat starting point for the extension.
Calculations and design codes	All calculations were based on BKR since the project was started before the transition to the Eurocodes. Wibom didn't believe that the project would have been feasible if the Eurocodes had been used for the existing building.
Critical members and strengthening	The columns were critical in this project, and many of them were strengthened on all storeys. Many columns had different cross-section and sizes, and therefore various kinds of section enlargement were implemented, see Figure B.12. Section enlargement was chosen both to increase the strength of the members, but also to reduce the risk for punching shear. Strengthening with for example steel profiles was therefore never an option. No shear reinforcement between old and new concrete was used, only surface treatment in form of sandblasting. According to Wibom, the shear strength achieved by this was more than sufficient. Strengthening of the bracing system was also required and was solved by a cross-bracing system. Anchoring this system to the original structure proved to be more troublesome than expect.

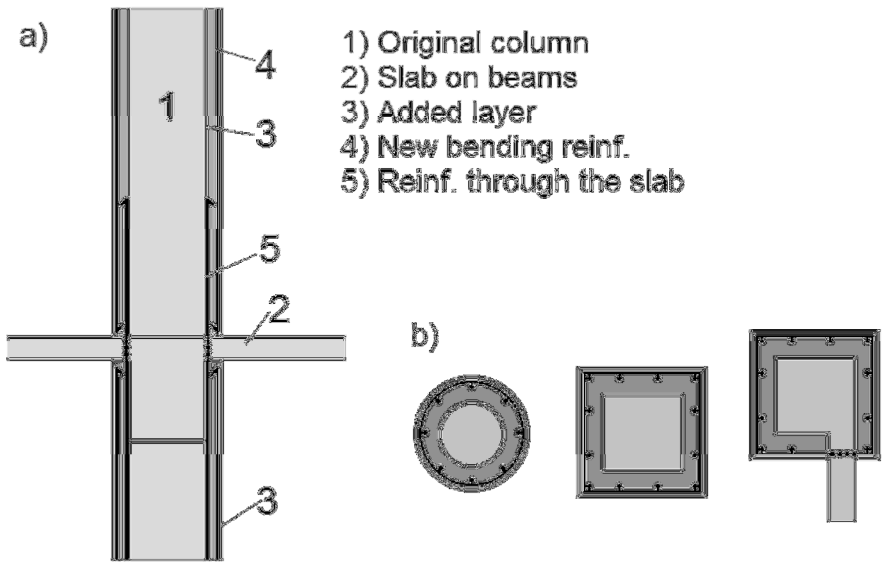


Figure B.12, Illustration of how some of the columns at Odin were strengthened.

Soil and foundation

Soil conditions	About 80 m of clay.
Foundation	The original structure was founded on cohesion piles. These were made of unjointed timber with a length of approximately 18-20 m directly cast into the concrete structure. The foundation had excess capacity due to conservative calculations. The changes in codes also lead to extra capacity.
Strengthening	The foundation was strengthened by of steel piles with wings which were connected to the load-bearing columns through steel profiles below the ground slab, see Figure B.13. The length of these were also around 20 m. Afterwards, everything was connected with concrete.

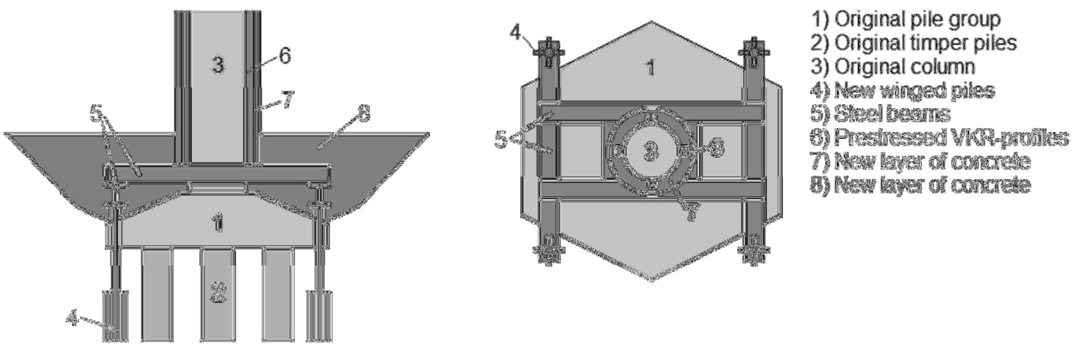


Figure B.13, Illustration of how the foundation at Odin was strengthened.

The extension itself

Number of floors	The original top storey was removed and replaced with six new storeys, see Figure B.14.
Load-carrying structure	Steel columns and HSQ-beams on top of which hollow core slabs were placed. To fulfil the sound demands, a double ceiling structure was used.

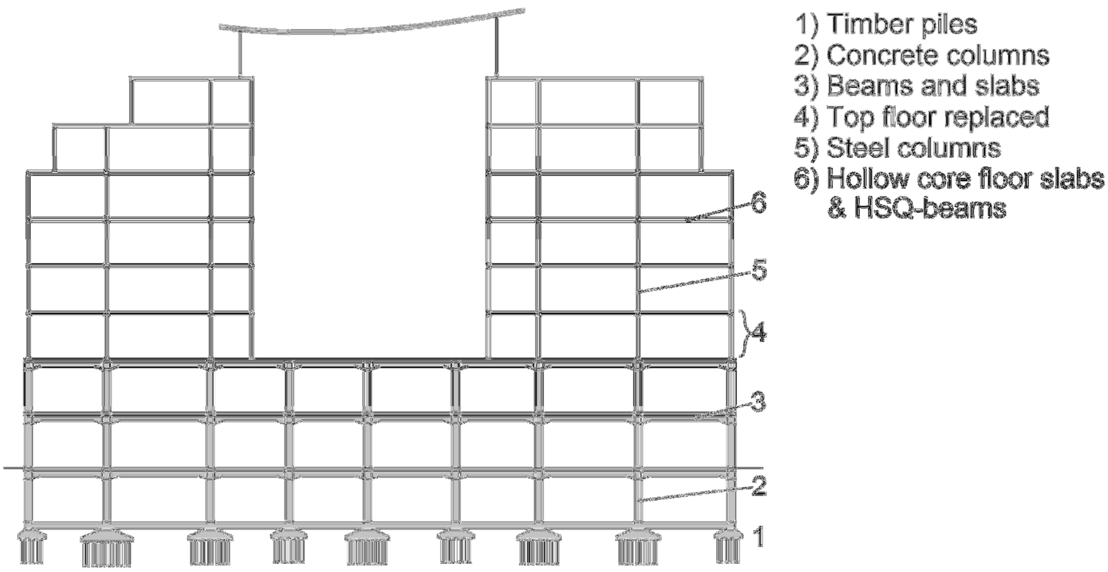


Figure B.14, Simplified illustration of section through Odin.

Other issues

Extra constructions	Installation of elevators was required. This needed extra attention at the foundation.
Important considerations	A FEM-model was established for the piles with foundation to control the degree of settlements before any strength calculations were executed.

Appendix C – Fire regulations

In this appendix, some complementary information concerning the fire regulations are presented.

The list below is based on *BBR 2012* and was mainly used to create Figure 4.2. The list explains which changes in the fire regulations that are needed to take into consideration when new heights of the building are reached. The numbers in parentheses refers to a certain section in *BBR 2012*.

Buildings with at least two storeys

- Buildings in *Activity class 4, 5A, 5B* or *5C* should be designed according to *Building class Br1* (5:22).
- Buildings in *Activity class 2B* or *2C* should be designed according to *Br1* for the second floor (5:22).
- The following buildings should at least be designed according to *Br2* (5:22):
  - Buildings intended for more than two apartments where there are living or working spaces on the attic floor.
  - Buildings with gathering halls in *Activity class 2B* or *2C* on the ground floor.
  - Buildings that have a larger area than 200 m^2 without fire cells of maximum that size.

Buildings with at least three storeys

- All buildings (except small houses) should at least be designed according to *Building class Br1* (5:22).
- For buildings in *Activity class 3,* the sprinkler system should be of type 2 (instead of type 1) (5:2522).
- Two successive light spots in stairwells should not go out due to the same error (5:342).
- The criteria for the exterior walls for buildings in *Building class Br1* change (5:551).
- Buildings that are linked together should be separated by a firewall (5:61).

Buildings with at least four storeys

- 20 minutes cannot be regarded as a quick enough response time for the rescue service considering evacuation from windows of detached residential buildings in *Activity class 3*. The normal 10 minutes can instead be used (5:323).

Buildings with at least five storeys

- Fire smoke ventilation or equivalent shall be present in every fire compartment on attics that are used for storage (5:732)

Buildings that are at least 24 m high

- Riser pipes for fire extinguishing water shall be placed in stairwells (5:733)
- Evacuation through windows should not be used if the lower edge of the opening is more than 23 m over the ground (5:323).

Buildings with at least nine storeys

- For buildings in *Activity class 3,* the sprinkler system should be of type 3 (5:2522).
- Dwellings and other areas shall have access to at least one stairwell of type *Tr2* (5:321).

- A stairwell of type *Tr2* can no longer serve as the only escape route for *Activity class 1*, (even if the conditions for satisfactory evacuation are fulfilled) (5:322)
- Emergency lighting should be installed in all stairways that serve as escape routes (5:343).
- The criteria for the exterior walls for buildings in *Building class Br1* change (5:551).

Buildings with at least eleven storeys

- At least one emergency elevator shall exist (5:734).

Buildings that are at least 40 m high

- Riser pipes for fire extinguishing water should be pressurised (5:733).

Buildings with at least seventeen storeys

- All buildings should be designed according to *Building class Br0* (5:22).
- Dwellings and other areas shall have access to at least one stairwell of type *Tr1*. Other stairwells should at least be of type *Tr2* (5:321).
- A stairwell of type *Tr2* can no longer serve as the only escape route for *Activity class 3* (5:322).
- A stairwell of type *Tr1* can no longer serve as the only escape route (5:322).

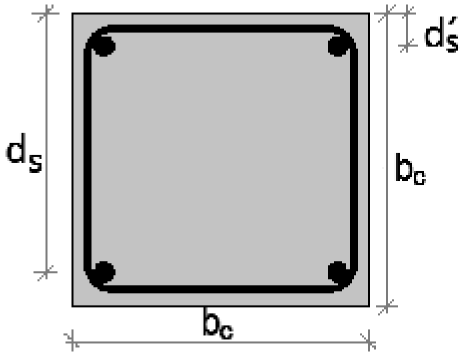
Appendix D – Calculations for strengthening of columns

To better see the differences between various ways to strengthen columns, calculations have been performed for some of the treated methods. Three different columns has been used through the calculations to get a better understanding of the effects from shape and slenderness. The calculations are presented in this appendix but described and discussed in Section 7.1.

The following subsections are treated:

*Part 1 - Capacity of existing column with rectangular section 0.71*0.71m²*

In this part of the appendix, the first column that was investigated is specified. The column is inspired by one of the columns that was strengthened during the extension of Scandic Rubinen.



Input data

Concrete:

$f_{ck} := 40MPa$ Characteristic compressive capacity

$f_{cd} := \frac{f_{ck}}{1.5} = 26.667 \cdot MPa$ Design value

$E_{cm} := 35GPa$ Mean value of modulus of elasticity

$b_c := 710mm$ Width of the column (square-shaped)

$h_c := 4.3m$ Height of the column

$A_c := b_c^2 = 0.504\,m^2$ Section area

$cover := 30mm$ Thickness of concrete cover

Reinforcement:

$E_s := 200GPa$ Modulus of elasticity

$f_{yk} := 500MPa$ Characteristic yield strength

$f_{yd} := \frac{f_{yk}}{1.15} = 434.783 \cdot MPa$ Design value

$\phi_{si} := 32mm$ Diameter of bending reinforcement

$A_{si} := \frac{\pi \cdot \phi_{si}^2}{4} = 804.248 \cdot mm^2$ Area of one reinforcement bar

$n_{si} := 2$ Number of bars at the bottom of the cross-section

$n'_{si} := 2$ Number of bars at the top of the cross-section

$A_s := n_{si} \cdot A_{si} = 1.608 \times 10^3 \cdot mm^2$ Total area of the bottom reinforcement

$A'_s := n'_{si} \cdot A_{si} = 1.608 \times 10^3 \cdot mm^2$ Total area of the top reinforcement

$\phi_{st.i} := 8mm$ Diameter of reinforcement in stirrup

Distances:

$d_s := b_c - cover - \phi_{st.i} - \frac{\phi_{si}}{2} = 0.656\,m$ Distance from compressive surface to bottom bars.

$d'_s := cover + \phi_{st.i} + \frac{\phi_{si}}{2} = 0.054\,m$ Distance from compressive surface to top bars.

Loads

The loads on the column (before strengthening) are assumed and then iterated until the resistance is slightly higher than the load effect.

$G := 6MN$ Permanent load on column, expressed as a point force

$Q := 3.73MN$ Variable load on column, expressed as a point force

$N_{Ed} := 1.35 \cdot G + 1.5 \cdot Q = 13.695 \cdot MN$ ULS combination

$\psi_2 := 0.6$ Assuming imposed load category C
(space where people may congregate)

$N_{Eqp} := G + \psi_2 \cdot Q = 8.238 \cdot MN$ Quasi-permanent SLS combination

Evaluation of slenderness

The following calculations are based on Section B11.3.2 in Al-Emrani et al. (2011)

The column is regarded as an isolated structural member with pinned connections in each end.

$l_0 := h_c = 4.3\,m$ Buckling length (assumed pinned-pinned connections)

$I_c := \frac{b_c \cdot b_c^{\,3}}{12} = 0.021\,m^4$ Second moment of inertia of the gross concrete section

$i := \sqrt{\frac{I_c}{A_c}} = 0.205\,m$ Radius of gyration

$\lambda := \frac{l_0}{i} = 20.98$ Slenderness

Rough estimation of the limit value of the slenderness:

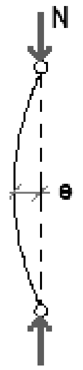
$n := \frac{N_{Ed}}{f_{cd} \cdot A_c} = 1.019$ Relative normal force

$\lambda_{lim} := \frac{10.8}{\sqrt{n}} = 10.7$ Rough value of limit

Since $\lambda > \lambda_{lim}$, the column must be designed with regard to the second order moment.

First order moment

The following calculations are based on Section B11.2 in Al-Emrani et al. (2011)



$e_0 := 0$	Intended eccentricity of load (assumed to be applied at the center of the column)
$\theta_0 := 0.005$	Base value of normal execution deviations
$\alpha_h := \dfrac{2}{\sqrt{\dfrac{h_c}{m}}} = 0.964$	Reduction factor
$m_\alpha := 1$	Only calculating with contribution from one column (and not the whole structure)
$\alpha_m := \sqrt{0.5 \cdot \left(1 + \dfrac{1}{m_\alpha}\right)} = 1$	Reduction factor
$\theta_i := \theta_0 \cdot \alpha_h \cdot \alpha_m = 4.822 \times 10^{-3}$	Normal execution deviations
$e_i := \theta_i \cdot \dfrac{l_0}{2} = 10.368 \cdot mm$	Unintended eccentricity

First order moments:

$M_{0Ed} := N_{Ed} \cdot (e_0 + e_i) = 141.993 \cdot kN \cdot m$	ULS combination
$M_{0Eqp} := N_{Eqp} \cdot (e_0 + e_i) = 85.413 \cdot kN \cdot m$	Quasi-permanent SLS combination

Nominal bending stiffness

The following calculations are based on Section B11.4.2 in Al-Emrani et al. (2011)

$\rho_{reinf} := \dfrac{A_s + A'_s}{A_c} = 6.382 \times 10^{-3}$	Reinforcement ratio
$\rho_{reinf} \geq 0.002 = 1$	OK!
$\gamma_{cE} := 1.2$	National parameter
$E_{cd} := \dfrac{E_{cm}}{\gamma_{cE}} = 29.167 \cdot GPa$	Design value of modulus of elasticity for the concrete

$$k_1 := \sqrt{\frac{\frac{f_{ck}}{MPa}}{20}} = 1.414$$

$$k_2 := \frac{N_{Ed}}{f_{cd} \cdot A_c} \cdot \frac{\lambda}{170} = 0.126$$

$RH := 50\%$ Indoor climate

$u := 4 \cdot b_c = 2.84\,m$ All sides of the column are subjected to drying

$h_0 := \frac{2 \cdot A_c}{u} = 0.355\,m$ Nominal thickness

$f_{cm} := f_{ck} + 8MPa = 48 \cdot MPa$ Mean value of compressive strength of concrete

$$\varphi_{RH} := \left[1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{\frac{h_0}{mm}}} \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.7}\right] \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.2} = 1.47$$ Creep from relative humidity

$\beta_{fcm} := 2.43$ Factor that considers the strength of the concrete

$\beta_{t0} := 0.48$ Assuming that the first load was applied after 28 days

$\varphi_{inf} := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = 1.715$ Final creep

$\varphi_{ef} := \varphi_{inf} \cdot \frac{M_{0Eqp}}{M_{0Ed}} = 1.032$ Effective creep

$$I_s := 4 \cdot A_{si} \cdot \left(\frac{b_c}{2} - cover - \phi_{st.i} - \frac{\phi_{si}}{2}\right)^2$$ Second moment of inertia for reinforcement. Simplified.

$$EI := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s = 112.35 \cdot MN \cdot m^2$$ Nominal bending stiffness

Second order moment

$N_B := \frac{\pi^2 \cdot EI}{l_0^2} = 59.97 \cdot MN$ Theoretical buckling force

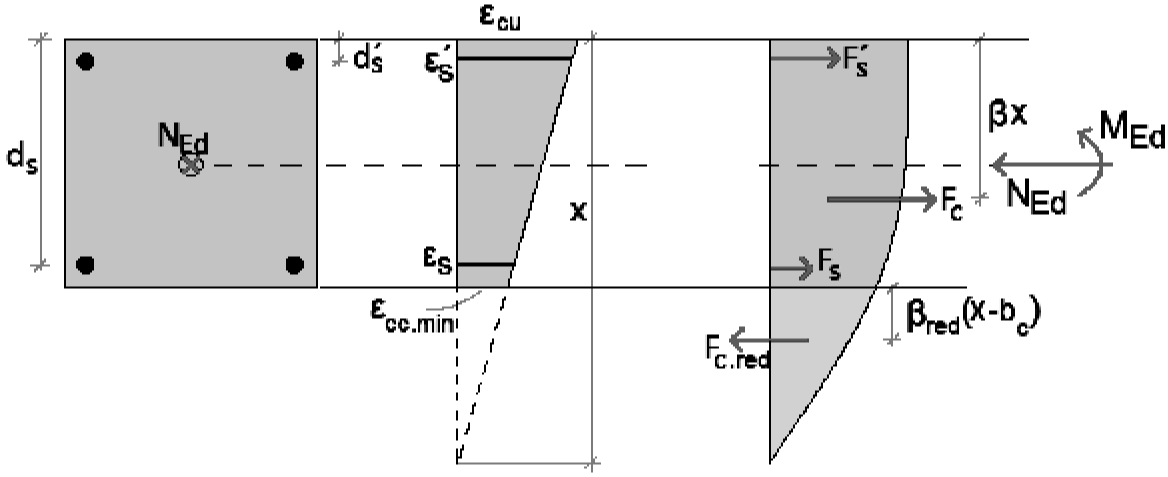
$\beta_{shape} := 1.0$ Shape factor (sinus-shaped bending moment)

$$M_{Ed} := \left(1 + \frac{\beta_{shape}}{\frac{N_B}{N_{Ed}} - 1}\right) \cdot M_{0Ed} = 184.015 \cdot kN \cdot m$$ Second order moment

Resistance of the section

The following calculations are based on Section B5.6 in Al-Emrani et al. (2010).

Assuming that the entire cross-section is in compression. This will generate a strain and stress distribution as illustrated in the figure below. To be able to use stress block factors, the part below the section needs to be removed.



$\alpha := 0.81$ Stress block factors

$\beta := 0.416$

$A_s = 1.608 \times 10^{-3}\,m^2$ Bottom reinforcement

$A'_s = 1.608 \times 10^{-3}\,m^2$ Top reinforcement

$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$ Ultimate strain for the concrete

Horizontal equilibrium:

$$\alpha \cdot f_{cd} \cdot b_c \cdot x - \alpha_{red} \cdot f_{cd} \cdot b_c \cdot \left(x - b_c\right) + \sigma'_s \cdot A'_s + \sigma_s \cdot A_s = N_{Ed}$$

$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement yields

$\sigma_s = \varepsilon_s \cdot E_s$ Assuming that bottom reinforcement doesn't yield

$\varepsilon_s = \varepsilon_{cu} \cdot \dfrac{x - d_s}{x}$ Steel strain

Calculating height of compressive zone:

$\varepsilon_{cc.min} := 0.0007664$ Assuming a value for the strain at the compressive surface

$$\alpha_{red} := 0.270 + (0.347 - 0.270) \cdot \frac{\left(\varepsilon_{cc.min} \cdot 10^3 - 0.6\right)}{(0.8 - 0.6)} = 0.334$$

$$\beta_{red} := 0.343 + (0.346 - 0.343) \cdot \frac{\left(\varepsilon_{cc.min} \cdot 10^3 - 0.6\right)}{(0.8 - 0.6)} = 0.345$$

Factors for the part of the compression block that comes below the section. α_{red} and β_{red} are in this case dependent on the strain at the the bottom of the cross-section.

$x := 0.5m$ Assuming an initial value for x

Given

$$\alpha \cdot f_{cd} \cdot b_c \cdot x - \alpha_{red} \cdot f_{cd} \cdot b_c \cdot (x - b_c) + \sigma'_s \cdot A'_s + \varepsilon_{cu} \cdot \frac{x - d_s}{x} \cdot E_s \cdot A_s = N_{Ed}$$

$x := \text{Find}(x) = 0.909\,m$ Solving x from horizontal equilibrium

Check of assumptions:

$\varepsilon_{cc.min} := \varepsilon_{cu} \cdot \frac{x - b_c}{x} = 7.664 \times 10^{-4}$ Concrete strain at "bottom side". Compare with assumption above and iterate.

$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.174 \times 10^{-3}$ Steel strain at yielding

$\varepsilon'_s := \varepsilon_{cu} \cdot \frac{x - d'_s}{x} = 3.292 \times 10^{-3}$ $\quad \varepsilon'_s \ge \varepsilon_{sy} = 1$ Upper reinf. yielding

$\varepsilon_s := \varepsilon_{cu} \cdot \frac{x - d_s}{x} = 9.743 \times 10^{-4}$ $\quad \varepsilon_s \ge \varepsilon_{sy} = 0$ Lower reinf. NOT yielding

Moment equilibrium around bottom reinforcement:

$$M_{Rd} := \alpha \cdot f_{cd} \cdot b_c \cdot x \cdot (d_s - \beta \cdot x) + \alpha_{red} \cdot f_{cd} \cdot b_c \cdot (x - b_c) \cdot [b_c - d_s + \beta_{red} \cdot (x - b_c)] \ldots = 326.73 \cdot kN \cdot m$$
$$+ \sigma'_s \cdot A'_s \cdot (d_s - d'_s) - N_{Ed} \cdot \left(d_s - \frac{b_c}{2}\right)$$

Check of resistance

$e_{min} := \max\left(\frac{b_c}{30}, 20mm\right) = 23.667 \cdot mm$ Minimum eccentricity for normal force

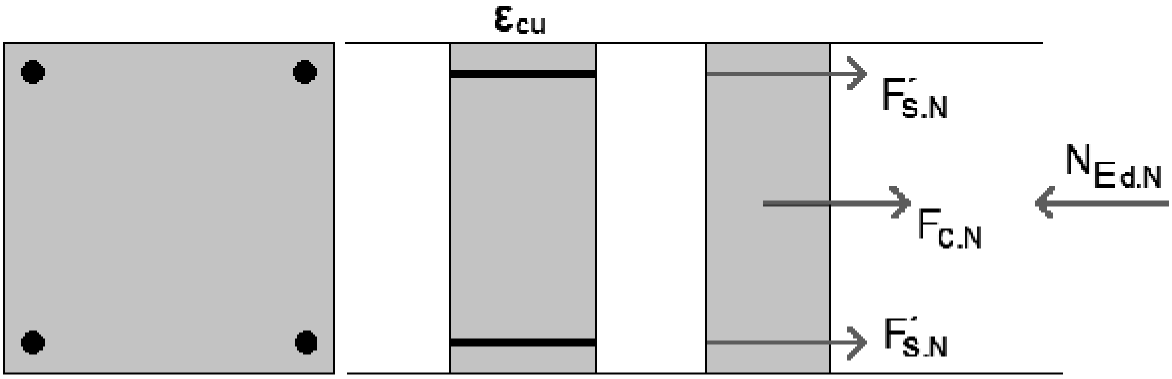
According to Section B11.4.4 in Al-Emrani et al. (2011), the following conditions must be fulfilled:

$\frac{M_{Ed}}{M_{Rd}} = 0.563$ $\quad \frac{N_{Ed} \cdot e_{min}}{M_{Rd}} = 0.992$

N-M relationship for the column

To be able to compare different columns, the relationship between normal force and moment is put into a N-M interaction diagram. To create the diagram, the capacity of the column must be calculated for the cases when it only is subjected to normal force or moment respectively.

When the column only is subjected to normal force:



$F'_{s.N} := A'_s \cdot f_{yd} = 699.346 \cdot kN$ — Force from reinforcement (yielding since $\varepsilon_{cu} > \varepsilon_{sy}$)

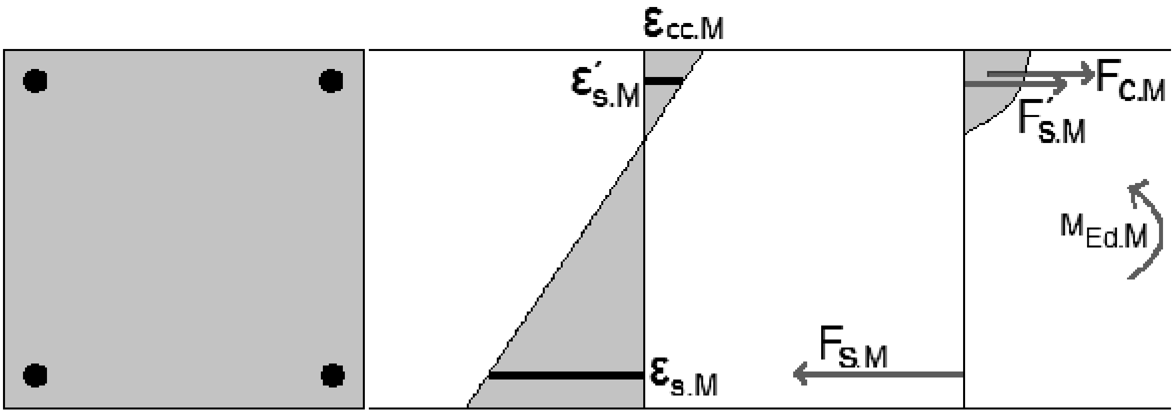
$F_{s.N} := F'_{s.N} = 699.346 \cdot kN$

$F_{c.N} := f_{cd} \cdot \left[b_c^{\,2} - \left(A'_s + A_s \right) \right] = 13.357 \cdot MN$ — Force from concrete

$N_{Rd.N} := F_{c.N} + F'_{s.N} + F_{s.N} = 14.756 \cdot MN$ — Resistance in the case of uniform compression

$\frac{N_{Ed}}{N_{Rd.N}} = 0.928$ — Relationship between maximum normal force and maximum normal force if no moment was present.

When the column only is subjected to bending moment:



$F'_{s.M} = E_s \cdot \varepsilon'_{s.M} \cdot A'_s$ — Force from reinforcement in compression (assuming that the reinforcement doesn't yield)

$\varepsilon'_{s.M} = \varepsilon_{cc.M} \cdot \frac{x_M - d'_s}{x_M}$ — Strain in top reinforcement

$F_{s.M} := f_{yd} \cdot A_s = 6.993 \times 10^5 \, N$ — Force from tensile reinforcement (assuming that the reinforcement yields)

$F_{c.M} = \alpha \cdot f_{cd} \cdot b_c \cdot x_M$ — Force from compressed concrete

$\varepsilon_{cc.M} := 0.0032$ Strain at compressed surface (assuming a value and iterating to find the highest moment resistance)

$$\alpha_M := 0.792 + (0.804 - 0.792)\cdot\frac{\left(\varepsilon_{cc.M}\cdot 10^3 - 3.2\right)}{(0.8 - 0.6)} = 0.792$$ Stress block factors

$$\beta_M := 0.410 + (0.414 - 0.410)\cdot\frac{\left(\varepsilon_{cc.M}\cdot 10^3 - 3.2\right)}{(0.8 - 0.6)} = 0.41$$

$x_M := 0.5m$ Initial guess of the height of x

Given

$$E_s\,\varepsilon_{cc.M}\cdot\frac{x_M - d'_s}{x_M}\cdot A'_s + \alpha_M\cdot f_{cd}\cdot b_c\cdot x_M - F_{s.M} = 0$$ Horizontal equilibrium

$x_M := \mathrm{Find}\left(x_M\right) = 0.051\,m$ Height of compressive zone

$$M_{Rd.M} := E_s\,\varepsilon_{cc.M}\cdot\frac{x_M - d'_s}{x_M}\cdot A'_s\cdot\left(d_s - d'_s\right) + \alpha_M\cdot f_{cd}\cdot b_c\cdot x_M\cdot\left(d_s - \beta_M\cdot x_M\right) = 446.287\cdot kN\cdot m$$

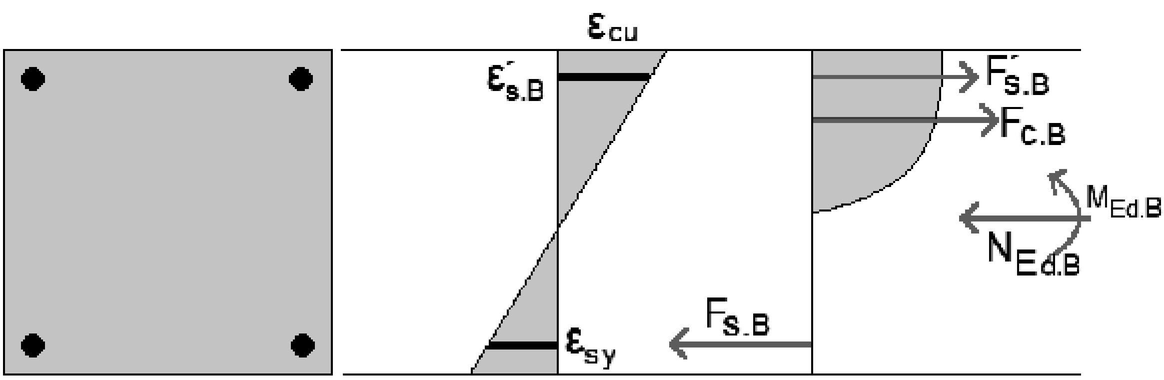
$$\varepsilon'_{s.M} := \varepsilon_{cc.M}\cdot\frac{x_M - d'_s}{x_M} = -1.971\times 10^{-4}$$ Strain in top reinforcement (neg=tension)

$$\varepsilon_{s.M} := \varepsilon_{cc.M}\cdot\frac{d_s - x_M}{x_M} = 0.038$$ Strain in bottom reinforcement

$$\frac{M_{Ed}}{M_{Rd.M}} = 0.412$$ Relation between bending moment in the actual case and moment resistance if no normal force was present

$$\frac{N_{Ed}\cdot e_{min}}{M_{Rd.M}} = 0.726$$

When the column is subjected to balanced bending moment and normal force:
This time, it is assumed that the tensile reinforcement reaches yielding at the same time as the top surface reaches ε_{cu}



$F_{s.B} := f_{yd}\cdot A_s = 699.346\cdot kN$ Force from tensile reinforcement (yielding)

$$x_B := \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}}\cdot d_s = 0.405\,m$$ Height of compressive zone

$$\varepsilon'_{s.B} := \varepsilon_{cu} \cdot \frac{x_B - d'_s}{x_B} = 3.033 \times 10^{-3}$$

Strain in compressive reinforcement

$$F'_{s.B} := \begin{cases} (f_{yd} \cdot A'_s) & \text{if } \varepsilon'_{s.B} \geq \varepsilon_{sy} \\ (E_s \cdot \varepsilon'_{s.B} \cdot A'_s) & \text{otherwise} \end{cases} = 699.346 \cdot kN$$

Force from reinforcement in compression

$$F_{c.B} := \alpha \cdot f_{cd} \cdot b_c \cdot x_B = 6.206 \cdot MN$$

Force from compressed concrete

$$N_{Rd.B} := F'_{s.B} + F_{c.B} - F_{s.B} = 6.206 \cdot MN$$

Normal force (based on horizontal equilibrium)

Moment resistance for the balanced section:

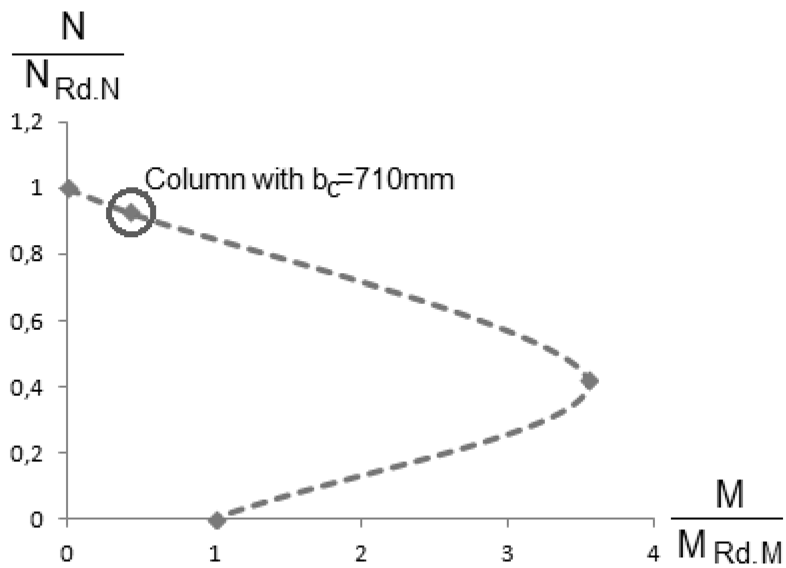
$$M_{Rd.B} := F'_{s.B} \cdot (d_s - d'_s) + F_{c.B} \cdot (d_s - \beta \cdot x_B) - N_{Rd.B} \cdot \left(d_s - \frac{b_c}{2}\right) = 1.579 \times 10^3 \cdot kN \cdot m$$

$$\frac{N_{Rd.B}}{N_{Rd.N}} = 0.421$$

Relation between normal force for the balanced section and the normal force if no moment is present

$$\frac{M_{Rd.B}}{M_{Rd.M}} = 3.539$$

Relation between moment for the balanced section and the moment if no normal force is present

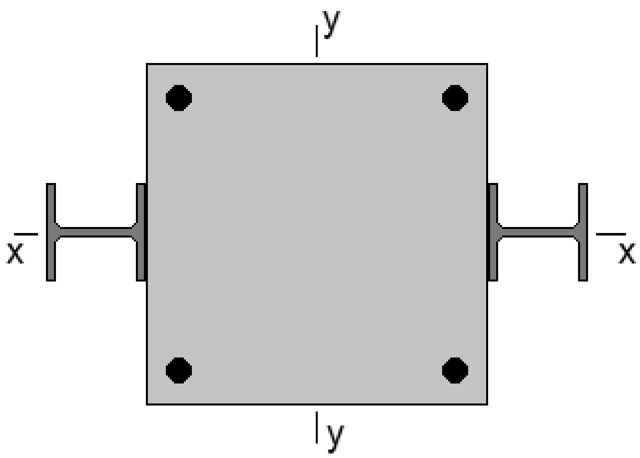


The figure above describes the N-M relation for the studied column.

Part 1 - Capacity of existing column with rectangular section 0.71*0.71m2

Part 2a - Strengthening with load-bearing steel profiles on the sides of the column (with prestressing and bending interaction)

The same column as above is strengthened with two HEB180 profiles mounted as in the figure below. In this case, it is assumed that the application is performed in such a way so that the vertical load on the top of the column also can go down into the new profiles. This can be done by inserting wedges or use of a hydraulic jack. The prestressing force is assumed to be 500kN per profile. The quasi-permanent load is put on the column before the strengthening to be able to predict the strain in the column before the profiles are added. Full bending interaction between profiles and concrete is assumed after strengthening.



$A_{HEB} := 6525mm^2$ Area of one steel profile

$I_{HEB.x} := 1363 \cdot 10^4 mm^4$ Second moment of inertia for one steel profile around its own axis

$I_{HEB.y} := 3831 \cdot 10^4 \cdot mm^4$

$f_{yk.HEB} := 355MPa$ Strength of steel profiles (steel class S355)

$\gamma_{M1} := 1.1$

$f_{yd.HEB} := \frac{f_{yk.HEB}}{\gamma_{M1}} = 322.727 \cdot MPa$ Design strength

$\varepsilon_{sy.HEB} := \frac{f_{yd.HEB}}{E_s} = 1.614 \times 10^{-3}$ Yield strain for steel profiles

Strain in column after prestressing of profiles

$N_{Eqp} = 8.238 \cdot MN$ Assuming that the quasi-permanent load is acting on the column (from Part 1)

$M_{0Eqp} = 85.413 \cdot kN \cdot m$ First order moment due to quasi-permanent load (from Part 1)

$EI = 112.35 \cdot MN \cdot m^2$ The nominal bending stiffness before strengthening is the same as in Part 1

$N_B = 59.97 \cdot MN$ Theoretical buckling force (from Part 1)

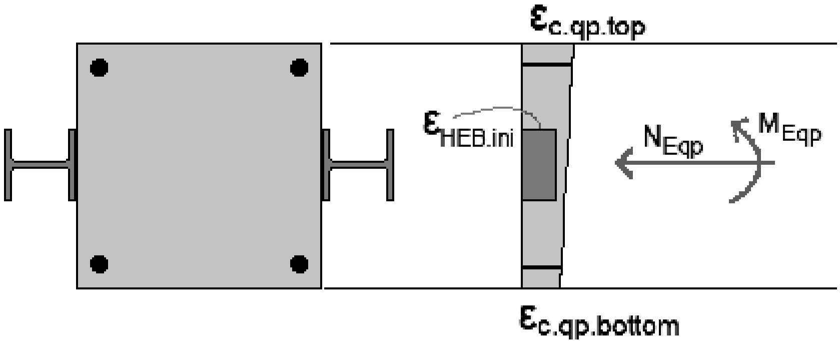
$$M_{Eqp} := \left(1 + \frac{\beta_{shape}}{\frac{N_B}{N_{Eqp}} - 1}\right) \cdot M_{0Eqp} = 99.015 \cdot kN \cdot m$$ Second order moment due to quasi-permanent load

$$F_{HEB.ini} := 500kN$$ Assumed applied prestressing force in one steel profile. 500kN can according to Statens råd för byggnadsforskning (1978) be reached by use of wedges that are hammered in beneath the profile.

$$N_{Eqp.red} := N_{Eqp} - 2 \cdot F_{HEB.ini} = 7.238 \cdot MN$$ Normal force in concrete column after the steel profiles have been prestressed.

Sectional analysis:

Assuming that the column is uncracked.



$$\alpha_I := \frac{E_s}{E_{cm}} = 5.714$$ Relationship between modulus of elasticity for reinforcement and concrete (stadium I)

$$A_I := b_c^2 + (\alpha_I - 1) \cdot (A_s + A'_s) = 0.519\,m^2$$

Second moment of inertia for the concrete column:

$$I_I := \frac{b_c \cdot b_c^3}{12} + (\alpha_I - 1) \cdot A'_s \cdot \left(\frac{b_c}{2} - d'_s\right)^2 + (\alpha_I - 1) \cdot A_s \cdot \left(d_s - \frac{b_c}{2}\right)^2 = 0.023\,m^4$$

$$\sigma_{cn.qp} := \frac{N_{Eqp.red}}{A_I} = 13.939 \cdot MPa$$ Stress in concrete due to normal force

$$\sigma_{sn.qp} := \alpha_I \cdot \frac{N_{Eqp.red}}{A_I} = 79.651 \cdot MPa$$ Stress in reinforcement due to normal force

$$\sigma_{cm.qp.top} := \frac{M_{Eqp}}{I_I} \cdot \frac{b_c}{2} = 1.559 \cdot MPa$$ Stress in concrete at top due to moment

$$\sigma_{cm.qp.bottom} := \frac{M_{Eqp}}{I_I} \cdot \frac{-b_c}{2} = -1.559 \cdot MPa$$ Stress in concrete at bottom due to moment

$$\sigma'_{sm.qp} := \alpha_I \cdot \frac{M_{Eqp}}{I_I} \cdot \left(\frac{b_c}{2} - d'_s\right) = 7.552 \cdot MPa$$ Stress in top reinforcement due to moment

$$\sigma_{sm.qp} := \alpha_I \cdot \frac{M_{Eqp}}{I_I} \cdot \left(-d_s + \frac{b_c}{2}\right) = -7.552 \cdot MPa$$ Stress in bottom reinforcement due to moment

$$\sigma_{c.qp.top} := \sigma_{cn.qp} + \sigma_{cm.qp.top} = 15.498 \cdot MPa$$ Stress at top surface

$$\sigma_{c.qp.bottom} := \sigma_{cn.qp} + \sigma_{cm.qp.bottom} = 12.38 \cdot MPa$$ Stress at bottom surface

$$\sigma'_{s.qp} := \sigma_{sn.qp} + \sigma'_{sm.qp} = 87.203 \cdot MPa$$ Stress in top reinforcement

$$\sigma_{s.qp} := \sigma_{sn.qp} + \sigma_{sm.qp} = 72.099 \cdot MPa$$ Stress in bottom reinforcement

$$\varepsilon_{c.qp.top} := \frac{\sigma_{c.qp.top}}{E_{cm}} = 4.428 \times 10^{-4}$$ Strain at top surface

$$\varepsilon_{c.qp.bottom} := \frac{\sigma_{c.qp.bottom}}{E_{cm}} = 3.537 \times 10^{-4}$$ Strain at bottom surface

$$\varepsilon'_{s.qp} := \frac{\sigma'_{s.qp}}{E_s} = 4.36 \times 10^{-4}$$ Strain in top reinforcement

$$\varepsilon_{s.qp} := \frac{\sigma_{s.qp}}{E_s} = 3.605 \times 10^{-4}$$ Strain in bottom reinforcement

$$\varepsilon_{c.qp.m} := \frac{\varepsilon_{c.qp.top} + \varepsilon_{c.qp.bottom}}{2} = 3.983 \times 10^{-4}$$ Strain at mid section

$$\varepsilon_{c.qp.flange.max} := \varepsilon_{c.qp.bottom} + \left(\varepsilon_{c.qp.top} - \varepsilon_{c.qp.bottom}\right) \cdot \frac{\frac{b_c}{2} + \frac{180mm}{2}}{b_c}$$

$$\varepsilon_{c.qp.flange.max} = 4.095 \times 10^{-4}$$ Strain in concrete at the level of the top of the steel profiles

$$\varepsilon_{c.qp.flange.min} := \varepsilon_{c.qp.bottom} + \left(\varepsilon_{c.qp.top} - \varepsilon_{c.qp.bottom}\right) \cdot \frac{\frac{b_c}{2} - \frac{180mm}{2}}{b_c}$$

$$\varepsilon_{c.qp.flange.min} = 3.87 \times 10^{-4}$$ Strain in concrete at the level of the bottom of the steel profiles

$$\sigma_{HEB.ini} := \frac{F_{HEB.ini}}{A_{HEB}} = 76.628 \cdot MPa$$ Initial stress in steel profiles

$$\varepsilon_{HEB.ini} := \frac{\sigma_{HEB.ini}}{E_s} = 3.831 \times 10^{-4}$$ Strain in the steel profiles before they are connected to the column

Loads after increase

$factor_2 := 1.28$ Increasing the load from Part 1 (iterated to find good utilisation)

$N_{Ed.2} := factor_2 \cdot N_{Ed} = 17.53 \cdot MN$ New vertical load on top of the column

$N_{Ed.2.ad} := N_{Ed.2} - N_{Eqp} = 9.292 \cdot MN$ The added load (starting from quasi-permanent)

First order moment from load increase

$e_0 = 0$

$e_i = 10.368 \cdot mm$

$M_{0.Ed.2.ad} := N_{Ed.2.ad} \cdot (e_0 + e_i) = 96.337 \cdot kN \cdot m$ Added moment

Nominal bending stiffness for the strengthened column

Adding the bending stiffness of the two steel profiles to the nominal bending stiffness:

$$EI_{2.x} := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_s \cdot I_{HEB.x} = 117.802 \cdot MN \cdot m^2$$

$$EI_{2.y} := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_s \cdot \left[I_{HEB.y} + A_{HEB} \cdot \left(\frac{b_c}{2} + \frac{180mm}{2} \right)^2 \right] = 644.519 \cdot MN \cdot m^2$$

Second order moment

$\beta_{shape} = 1$

$$N_{B.2.x} := \frac{\pi^2 \cdot EI_{2.x}}{l_0^2} = 62.881 \cdot MN$$ Buckling load around x-axis

$$N_{B.2.y} := \frac{\pi^2 \cdot EI_{2.y}}{l_0^2} = 344.032 \cdot MN$$ Buckling load around y-axis

The second order moment from the increased load is added to the second order moment before strengthening.

$$M_{Ed.2.x} := M_{Eqp} + \left(1 + \frac{\beta_{shape}}{\frac{N_{B.2.x}}{N_{Ed.2.ad}} - 1} \right) \cdot M_{0.Ed.2.ad} = 212.056 \cdot kN \cdot m$$

$$M_{Ed.2.y} := M_{Eqp} + \left(1 + \frac{\beta_{shape}}{\frac{N_{B.2.y}}{N_{Ed.2.ad}} - 1} \right) \cdot M_{0.Ed.2.ad} = 198.026 \cdot kN \cdot m$$

Resistance against bending around x-axis (weak direction)

$\alpha = 0.81$ — Stress block factors

$\beta = 0.416$

$A_s = 1.608 \times 10^3 \cdot mm^2$ — Bottom reinforcement

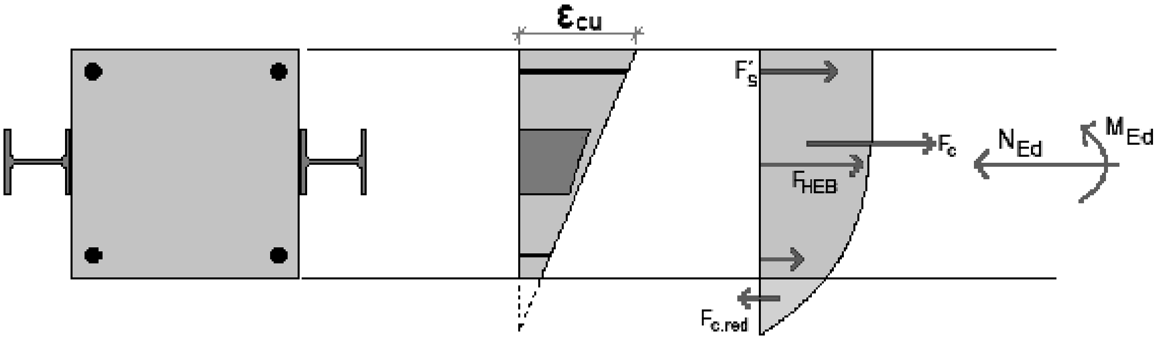
$A'_s = 1.608 \times 10^3 \cdot mm^2$ — Top reinforcement

$\varepsilon_{cu} = 3.5 \times 10^{-3}$ — Ultimate strain for the concrete

Horizontal equilibrium:

Assuming that the whole section is in compression.

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_2 - \alpha_{red.2} \cdot f_{cd} \cdot b_c \cdot (x_2 - b_c) + \sigma'_s \cdot A'_s + 2 \cdot F_{HEB} + \sigma_s \cdot A_s = N_{Ed.2}$$



$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ — Assuming that top reinforcement yields

$\sigma_s = \varepsilon_s \cdot E_s$ — Assuming that bottom reinforcement doesn't yield

$$\varepsilon_s = \varepsilon_{cu} \cdot \frac{x_2 - d_s}{x_2}$$

$F_{HEB} := f_{yd.HEB} \cdot A_{HEB} = 2.106 \cdot MN$ — Force from the steel profiles (assuming that the whole profile yields)

Calculating height of compressive zone:

$\varepsilon_{cc.min.2} := 0.0005837$

Assuming that the whole section is in compression.

$$\alpha_{red.2} := 0.187 + (0.27 - 0.187)\cdot\frac{\left(\varepsilon_{cc.min.2}\cdot 10^{3} - 0.4\right)}{(0.6 - 0.4)} = 0.263$$

$$\beta_{red.2} := 0.339 + (0.343 - 0.339)\cdot\frac{\left(\varepsilon_{cc.min.2}\cdot 10^{3} - 0.4\right)}{(0.6 - 0.4)} = 0.343$$

Factors for the part of the compression block that comes below the section. α_{red} and β_{red} is in this case dependent on the strain at the the bottom of the cross-section.

$x_2 := 0.5\text{m}$

Assuming an initial value for x_2

Given

$$\alpha\cdot f_{cd}\cdot b_c\cdot x_2 - \alpha_{red.2}\cdot f_{cd}\cdot b_c\cdot\left(x_2 - b_c\right) + \sigma'_s\cdot A'_s + 2\cdot F_{HEB} + \varepsilon_{cu}\cdot\frac{x_2 - d_s}{x_2}\cdot E_s\cdot A_s = N_{Ed.2}$$

$x_2 := \text{Find}\left(x_2\right) = 0.852\,\text{m}$

Solving x_2 from horizontal equilibrium

Check of assumptions:

$$\varepsilon_{cc.min.2} := \varepsilon_{cu}\cdot\frac{x_2 - b_c}{x_2} = 5.837\times 10^{-4}$$

Check with assumed value above and iterate

$\varepsilon_{sy} = 2.174\times 10^{-3}$

$$\varepsilon'_s := \varepsilon_{cu}\cdot\frac{x_2 - d'_s}{x_2} = 3.278\times 10^{-3} \qquad \varepsilon'_s \geq \varepsilon_{sy} = 1$$

Top reinforcement yielding

$$\varepsilon_s := \varepsilon_{cu}\cdot\frac{x_2 - d_s}{x_2} = 8.055\times 10^{-4} \qquad \varepsilon_s \geq \varepsilon_{sy} = 0$$

Bottom reinforcement NOT yielding

$$\varepsilon_{flange.max} := \varepsilon_{cu}\cdot\frac{x_2 - \frac{b_c}{2} + \frac{180\text{mm}}{2}}{x_2} - \varepsilon_{c.qp.flange.max} + \varepsilon_{HEB.ini} = 2.385\times 10^{-3}$$

$\varepsilon_{flange.max} \geq \varepsilon_{sy.HEB} = 1$

Top part of steel profile is yielding

$$\varepsilon_{flange.min} := \varepsilon_{cu}\cdot\frac{x_2 - \frac{b_c}{2} - \frac{180\text{mm}}{2}}{x_2} - \varepsilon_{c.qp.flange.min} + \varepsilon_{HEB.ini} = 1.668\times 10^{-3}$$

$\varepsilon_{flange.min} \geq \varepsilon_{sy.HEB} = 1$

Bottom part of steel profile is yielding

Moment equilibrium around tensile reinforcement:

$$M_{Rd.2.x} := \alpha \cdot f_{cd} \cdot b_c \cdot x_2 \cdot (d_s - \beta \cdot x_2) + \alpha_{red.2} \cdot f_{cd} \cdot b_c \cdot (x_2 - b_c) \cdot \left[b_c - d_s + (x_2 - b_c) \cdot \beta_{red.2}\right] \cdots$$
$$+ \sigma'_s \cdot A'_s \cdot (d_s - d'_s) + 2 \cdot F_{HEB} \cdot \left(d_s - \frac{b_c}{2}\right) - N_{Ed.2} \cdot \left(d_s - \frac{b_c}{2}\right)$$

$$M_{Rd.2.x} = 425.304 \cdot kN \cdot m$$

Check of resistance

According to B11.4.4 in Al-Emrani et al. (2011)

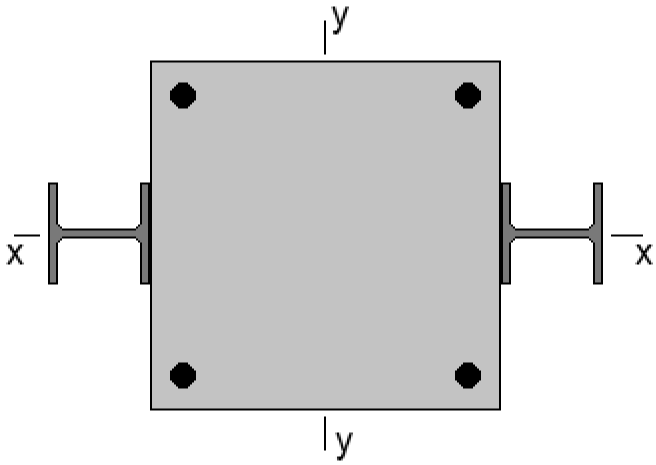
$e_{min} = 0.024\,m$ Minimum eccentricity for normal force

$$\frac{M_{Ed.2.x}}{M_{Rd.2.x}} = 0.499 \qquad \frac{N_{Ed.2} \cdot e_{min}}{M_{Rd.2.x}} = 0.975$$

Part 2a - Strengthening with load-bearing steel profiles on the sides of the column (with prestressing and bending intera..

Part 2b - Strengthening with vertically loaded steel profiles on the sides of the column (without prestressing but with bending interaction)

The column is now strengthened in the same way as in Part 2a, but it is now assumed that that the steel profiles are NOT prestressed. These calculations are performed to investigate how big influence the prestressing of the profiles has.



$A_{HEB} := 6525mm^2$ — Area of one steel profile

$I_{HEB.x} := 1363 \cdot 10^4 mm^4$ — Second moment of inertia for one steel profile around its own axis

$I_{HEB.y} := 3831 \cdot 10^4 \cdot mm^4$

$f_{yk.HEB} := 355MPa$ — Strength of steel profiles (steel class S355)

$\gamma_{M1} := 1.1$

$$f_{yd.HEB} := \frac{f_{yk.HEB}}{\gamma_{M1}} = 322.727 \cdot MPa$$

$$\varepsilon_{sy.HEB} := \frac{f_{yd.HEB}}{E_s} = 1.614 \times 10^{-3}$$ Yield strain for steel profiles

Strain in column before load increase

$N_{Eqp} = 8.238 \cdot MN$ — Assuming that the quasi-permanent load is acting on the column (from Part 1)

$M_{0Eqp} = 85.413 \cdot kN \cdot m$ — First order moment due to quasi-permanent load (from Part 1)

$EI = 112.35 \cdot MN \cdot m^2$ — Nominal bending stiffness before strengthening is the same as in Part 1

$N_B = 59.97 \cdot MN$ Theoretical buckling force (from Part 1)

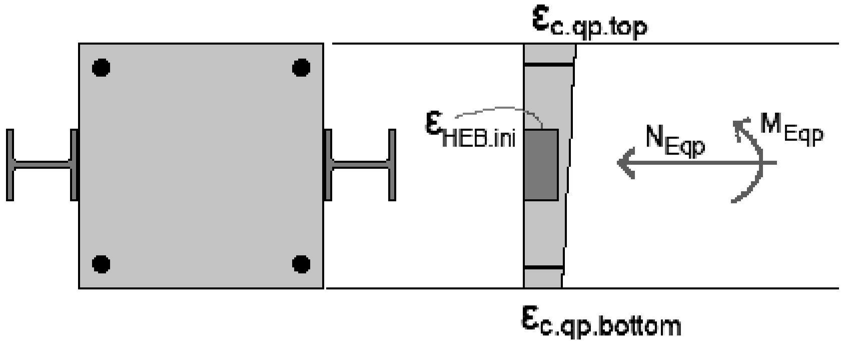
$$M_{Eqp} := \left(1 + \frac{\beta_{shape}}{\frac{N_B}{N_{Eqp}} - 1}\right) \cdot M_{0Eqp} = 99.015 \cdot kN \cdot m$$ Second order moment due to quasi-permanent load

$F_{HEB.ini} := 0kN$ Assumed applied prestressing force in one steel profile. Set to zero in this case.

$N_{Eqp.red} := N_{Eqp} - 2 \cdot F_{HEB.ini} = 8.238 \cdot MN$ Normal force in concrete column after the steel profiles have been prestressed.

Sectional analysis:

Assuming that the column is uncracked.



$$\alpha_I := \frac{E_s}{E_{cm}} = 5.714$$ Relationship between modulus of elasticity for steel and concrete

$$A_I := b_c^2 + (\alpha_I - 1) \cdot (A_s + A'_s) = 0.519 m^2$$

$$I_I := \frac{b_c \cdot b_c^3}{12} + (\alpha_I - 1) \cdot A'_s \cdot \left(\frac{b_c}{2} - d'_s\right)^2 + (\alpha_I - 1) \cdot A_s \cdot \left(d_s - \frac{b_c}{2}\right)^2 = 0.023 m^4$$

$$\sigma_{cn.qp} := \frac{N_{Eqp.red}}{A_I} = 15.865 \cdot MPa$$ Stress in concrete due to normal force

$$\sigma_{sn.qp} := \alpha_I \cdot \frac{N_{Eqp.red}}{A_I} = 90.655 \cdot MPa$$ Stress in steel due to normal force

$$\sigma_{cm.qp.top} := \frac{M_{Eqp}}{I_I} \cdot \frac{b_c}{2} = 1.559 \cdot MPa$$ Stress in concrete at top due to moment

$$\sigma_{cm.qp.bottom} := \frac{M_{Eqp}}{I_I} \cdot \frac{-b_c}{2} = -1.559 \cdot MPa$$ Stress in concrete at bottom due to moment

$$\sigma'_{sm.qp} := \alpha_I \cdot \frac{M_{Eqp}}{I_I} \cdot \left(\frac{b_c}{2} - d'_s\right) = 7.552 \cdot MPa$$ Stress in top reinforcement due to moment

$$\sigma_{sm.qp} := \alpha_I \cdot \frac{M_{Eqp}}{I_I} \cdot \left(-d_s + \frac{b_c}{2}\right) = -7.552 \cdot MPa$$ Stress in bottom reinforcement due to moment

$\sigma_{c.qp.top} := \sigma_{cn.qp} + \sigma_{cm.qp.top} = 17.423 \cdot MPa$ Stress at top surface

$\sigma_{c.qp.bottom} := \sigma_{cn.qp} + \sigma_{cm.qp.bottom} = 14.306 \cdot MPa$ Stress at bottom surface

$\sigma'_{s.qp} := \sigma_{sn.qp} + \sigma'_{sm.qp} = 98.208 \cdot MPa$ Stress in top reinforcement

$\sigma_{s.qp} := \sigma_{sn.qp} + \sigma_{sm.qp} = 83.103 \cdot MPa$ Stress in bottom reinforcement

$\varepsilon_{c.qp.top} := \dfrac{\sigma_{c.qp.top}}{E_{cm}} = 4.978 \times 10^{-4}$ Strain at top surface

$\varepsilon_{c.qp.bottom} := \dfrac{\sigma_{c.qp.bottom}}{E_{cm}} = 4.087 \times 10^{-4}$ Strain at bottom surface

$\varepsilon'_{s.qp} := \dfrac{\sigma'_{s.qp}}{E_s} = 4.91 \times 10^{-4}$ Strain in top reinforcement

$\varepsilon_{s.qp} := \dfrac{\sigma_{s.qp}}{E_s} = 4.155 \times 10^{-4}$ Strain in bottom reinforcement

$\varepsilon_{c.qp.m} := \dfrac{\varepsilon_{c.qp.top} + \varepsilon_{c.qp.bottom}}{2} = 4.533 \times 10^{-4}$ Strain at mid section

$$\varepsilon_{c.qp.flange.max} := \varepsilon_{c.qp.bottom} + \left(\varepsilon_{c.qp.top} - \varepsilon_{c.qp.bottom}\right) \cdot \frac{\frac{b_c}{2} + \frac{180mm}{2}}{b_c}$$

$\varepsilon_{c.qp.flange.max} = 4.646 \times 10^{-4}$ Strain in concrete at the level of the top of the steel profiles

$$\varepsilon_{c.qp.flange.min} := \varepsilon_{c.qp.bottom} + \left(\varepsilon_{c.qp.top} - \varepsilon_{c.qp.bottom}\right) \cdot \frac{\frac{b_c}{2} - \frac{180mm}{2}}{b_c}$$

$\varepsilon_{c.qp.flange.min} = 4.42 \times 10^{-4}$ Strain in concrete at the level of the bottom of the steel profiles

$\sigma_{HEB.ini} := \dfrac{F_{HEB.ini}}{A_{HEB}} = 0 \cdot MPa$ Initial stress in steel profiles (zero in this case)

$\varepsilon_{HEB.ini} := \dfrac{\sigma_{HEB.ini}}{E_s} = 0$ Strain in the steel profiles before they are connected to the column (zero in this case)

Loads after increase

$factor_2 := 1.26$ Increasing the load from Part 1 (integrated to get good utilisation)

$N_{Ed.2} := factor_2 \cdot N_{Ed} = 17.256 \cdot MN$ New vertical load on top of the column

$N_{Ed.2.ad} := N_{Ed.2} - N_{Eqp} = 9.018 \cdot MN$ The added load (starting from quasi-permanent)

First order moment from load increase

$e_0 = 0$

$e_i = 10.368 \cdot mm$

$M_{0.Ed.2.ad} := N_{Ed.2.ad} \cdot (e_0 + e_i) = 93.498 \cdot kN \cdot m$ Added moment

Nominal bending stiffness for the strengthened column

Adding the bending stiffnes of the two steel profiles to the nominal bending stiffness:

$$EI_{2.x} := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_s \cdot I_{HEB.x} = 117.802 \cdot MN \cdot m^2$$

$$EI_{2.y} := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_s \cdot \left[I_{HEB.y} + A_{HEB} \cdot \left(\frac{b_c}{2} + \frac{180mm}{2} \right)^2 \right] = 644.519 \cdot MN \cdot m^2$$

Second order moment

$\beta_{shape} = 1$

$$N_{B.2.x} := \frac{\pi^2 \cdot EI_{2.x}}{l_0^2} = 62.881 \cdot MN$$ Buckling load around x-axis

$$N_{B.2.y} := \frac{\pi^2 \cdot EI_{2.y}}{l_0^2} = 344.032 \cdot MN$$ Buckling load around y-axis

The second order moment from the increased load is added to the second order moment before strengthening.

$$M_{Ed.2.x} := M_{Eqp} + \left(1 + \frac{\beta_{shape}}{\frac{N_{B.2.x}}{N_{Ed.2.ad}} - 1} \right) \cdot M_{0.Ed.2.ad} = 208.166 \cdot kN \cdot m$$

$$M_{Ed.2.y} := M_{Eqp} + \left(1 + \frac{\beta_{shape}}{\frac{N_{B.2.y}}{N_{Ed.2.ad}} - 1} \right) \cdot M_{0.Ed.2.ad} = 195.029 \cdot kN \cdot m$$

Resistance against bending around x-axis (weak direction)

$\alpha = 0.81$

$\beta = 0.416$

$A_s = 1.608 \times 10^3 \cdot mm^2$ Bottom reinforcement

$A'_s = 1.608 \times 10^3 \cdot mm^2$ Top reinforcement

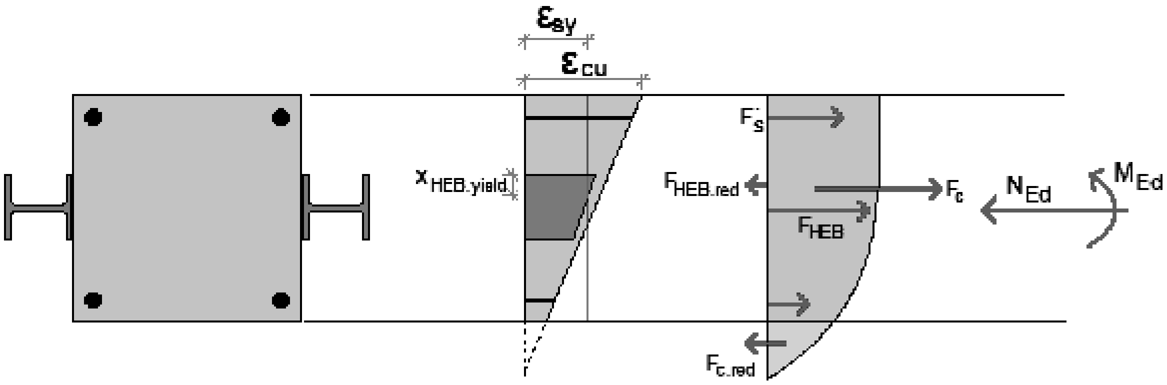
$\varepsilon_{cu} = 3.5 \times 10^{-3}$ Ultimate strain for the concrete

Horizontal equilibrium:

Assuming that the whole section is in compression.

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_2 - \alpha_{red.2} \cdot f_{cd} \cdot b_c \cdot (x_2 - b_c) + \sigma'_s \cdot A'_s + 2 \cdot F_{HEB} - 2 \cdot F_{HEB.red} + \sigma_s \cdot A_s = N_{Ed.2}$$

Where $F_{HEB.red}$ is a reduction of the force in the parts of the profiles that yield.



$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ — Assuming that top reinforcement yields

$\sigma_s = \varepsilon_s \cdot E_s$ — Assuming that bottom reinforcement doesn't yield

$$\varepsilon_s = \varepsilon_{cu} \cdot \frac{x_2 - d_s}{x_2}$$

$F_{HEB} = \sigma_{HEB} \cdot A_{HEB}$ — Force from the steel profiles (before reduction due to yielding)

$\sigma_{HEB} = E_s \cdot (\varepsilon_{c.m} - \varepsilon_{c.qp.m} + \varepsilon_{HEB.ini})$ — Mean stress in profiles (before reduction)

$$\varepsilon_{c.m} = \varepsilon_{cu} \cdot \frac{x_2 - \frac{b_c}{2}}{x_2}$$ Strain in concrete at mid section

==> $$F_{HEB} = E_s \cdot \left(\varepsilon_{cu} \cdot \frac{x_2 - \frac{b_c}{2}}{x_2} - \varepsilon_{c.qp.m} + \varepsilon_{HEB.ini} \right) \cdot A_{HEB}$$

$F_{HEB.red} = \sigma_{HEB.red} \cdot 2t_{HEB.f} \cdot x_{HEB.yield}$ — Reduction for the parts of the profiles that yield

$t_{HEB.f} := 14mm$ — Thickness of the flanges

$x_{HEB.yield} := 71mm$ — Assumed height of flanges that reach yielding (iterated)

$$\sigma_{HEB.red} = \frac{\varepsilon_{flange.max} - \varepsilon_{sy.HEB}}{2} \cdot E_s$$ Mean reduction stress in the part of the flanges that yield

$$\varepsilon_{flange.max} = \varepsilon_{cu} \cdot \frac{x_2 - \frac{b_c}{2} + \frac{180mm}{2}}{x_2} - \varepsilon_{c.qp.flange.max} + \varepsilon_{HEB.ini}$$ Strain at top of flanges

$$F_{HEB.red} = \frac{\left(\varepsilon_{cu}\cdot\frac{x_2 - \frac{b_c}{2} + \frac{180mm}{2}}{x_2} - \varepsilon_{c.qp.flange.max} + \varepsilon_{HEB.ini}\right) - \varepsilon_{sy.HEB}}{2}\cdot E_s\cdot 2t_{HEB.f}\cdot x_{HEB.yield}$$

Calculating height of compressive zone:

$\varepsilon_{cc.min.2} := 0.0004975$ Assuming that the whole section is in compression.

$$\alpha_{red.2} := 0.187 + (0.27 - 0.187)\cdot\frac{\left(\varepsilon_{cc.min.2}\cdot 10^3 - 0.4\right)}{(0.6 - 0.4)} = 0.227$$

$$\beta_{red.2} := 0.339 + (0.343 - 0.339)\cdot\frac{\left(\varepsilon_{cc.min.2}\cdot 10^3 - 0.4\right)}{(0.6 - 0.4)} = 0.341$$

Factors for the part of the compression block that comes below the section. α_{red} and β_{red} is in this case dependent on the strain at the the bottom of the cross-section.

$x_2 := 0.5m$ Assuming an initial value for x_2

Given

$$\alpha\cdot f_{cd}\cdot b_c\cdot x_2 - \alpha_{red.2}\cdot f_{cd}\cdot b_c\cdot\left(x_2 - b_c\right) + \sigma'_s\cdot A'_s \cdots$$
$$+ 2\cdot\left(\varepsilon_{cu}\cdot\frac{x_2 - \frac{b_c}{2}}{x_2} - \varepsilon_{c.qp.m} + \varepsilon_{HEB.ini}\right)\cdot E_s\cdot A_{HEB} \cdots$$
$$+ -2\cdot\left[\frac{\left(\varepsilon_{cu}\cdot\frac{x_2 - \frac{b_c}{2} + \frac{180mm}{2}}{x_2} - \varepsilon_{c.qp.flange.max} + \varepsilon_{HEB.ini}\right) - \varepsilon_{sy}}{2}\cdot E_s\cdot 2t_{HEB.f}\cdot x_{HEB.yield}\right] \cdots$$
$$+ \varepsilon_{cu}\cdot\frac{x_2 - d_s}{x_2}\cdot E_s\cdot A_s = N_{Ed.2}$$

$x_2 := \mathrm{Find}\left(x_2\right) = 0.828\,m$ Solving x_2 from horizontal equilibrium

Check of assumptions:

$\varepsilon_{cc.min.2} := \varepsilon_{cu}\cdot\frac{x_2 - b_c}{x_2} = 4.975\times 10^{-4}$ Check with assumed value and iterate

$\varepsilon_{sy} = 2.174\times 10^{-3}$

$\varepsilon'_s := \varepsilon_{cu}\cdot\frac{x_2 - d'_s}{x_2} = 3.272\times 10^{-3}$ $\varepsilon'_s \geq \varepsilon_{sy} = 1$ Top reinforcement yielding

$\varepsilon_s := \varepsilon_{cu}\cdot\frac{x_2 - d_s}{x_2} = 7.259\times 10^{-4}$ $\varepsilon_s \geq \varepsilon_{sy} = 0$ Bottom reinforcement NOT yielding

$$\varepsilon_{flange.max} := \varepsilon_{cu}\cdot\frac{x_2 - \frac{b_c}{2} + \frac{180mm}{2}}{x_2} - \varepsilon_{c.qp.flange.max} + \varepsilon_{HEB.ini} = 1.915\times 10^{-3}$$

$$\varepsilon_{flange.min} := \varepsilon_{cu} \cdot \frac{x_2 - \frac{b_c}{2} - \frac{180mm}{2}}{x_2} - \varepsilon_{c.qp.flange.max} + \varepsilon_{HEB.ini} = 1.154 \times 10^{-3}$$

$$x_{HEB.yield} := \max\left(0, 180mm \cdot \frac{\varepsilon_{flange.max} - \varepsilon_{sy.HEB}}{\varepsilon_{flange.max} - \varepsilon_{flange.min}}\right) = 71 \cdot mm$$ Height of the part of the flanges that reach yielding (iterate above)

Forces from the steel profiles:

$$F_{HEB} := E_s \cdot \left(\varepsilon_{cu} \cdot \frac{x_2 - \frac{b_c}{2}}{x_2} - \varepsilon_{c.qp.m} + \varepsilon_{HEB.ini}\right) \cdot A_{HEB} = 2.017 \cdot MN$$

$$F_{HEB.red} := \max\left(0, \frac{\varepsilon_{cu} \cdot \frac{x_2 - \frac{b_c}{2} + \frac{180mm}{2}}{x_2} - \varepsilon_{sy}}{2} \cdot E_s \cdot 2t_{HEB.f} \cdot x_{HEB.yield}\right) = 40.963 \cdot kN$$

Moment equilibrium around tensile reinforcement:

$$M_{Rd.2.x} := \alpha \cdot f_{cd} \cdot b_c \cdot x_2 \cdot (d_s - \beta \cdot x_2) + \alpha_{red.2} \cdot f_{cd} \cdot b_c \cdot (x_2 - b_c) \cdot \left[b_c - d_s + (x_2 - b_c) \cdot \beta_{red.2}\right] \ldots$$
$$+ \sigma'_s \cdot A'_s \cdot (d_s - d'_s) + 2 \cdot F_{HEB} \cdot \left(d_s - \frac{b_c}{2}\right) - 2 \cdot F_{HEB.red} \cdot \left(d_s - \frac{b_c}{2} + \frac{180mm}{2} - \frac{1}{3} \cdot x_{HEB.yield}\right) \ldots$$
$$+ -N_{Ed.2} \cdot \left(d_s - \frac{b_c}{2}\right)$$

$$M_{Rd.2.x} = 415.106 \cdot kN \cdot m$$

Check of resistance

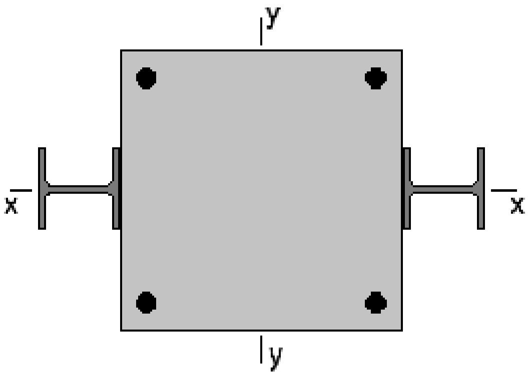
$e_{min} = 0.024\,m$ Minimum eccentricity for normal force

$$\frac{M_{Ed.2.x}}{M_{Rd.2.x}} = 0.501 \qquad \frac{N_{Ed.2} \cdot e_{min}}{M_{Rd.2.x}} = 0.984$$

Part 2b - Strengthening with vertically loaded steel profiles on the sides of the column (without prestressing but with be...

Part 2c - Strengthening with vertically loaded steel profiles (without prestressing and bending interaction)

The column is now strengthened in the same way as in Part 2, but it is now assumed that that the steel profiles are NOT prestressed and that there is NO bending interaction between the profiles and the column. This means that the additional bending moment is applied to the concrete and the profiles with a ratio that corresponds to their bending stiffness. It is however assumed that the strain due to the increased normal force becomes the same in the profiles as in the concrete. These calculations are performed to investigate how big influence the bending interaction has.



$A_{HEB} := 6525\text{mm}^2$ Area of one steel profile

$I_{HEB.x} := 1363 \cdot 10^4 \text{mm}^4$ Second moment of inertia for one steel profile around its own axis

$I_{HEB.y} := 3831 \cdot 10^4 \cdot \text{mm}^4$

$f_{yk.HEB} := 355\text{MPa}$ Strength of steel profiles (steel class S355)

$\gamma_{M1} := 1.1$

$$f_{yd.HEB} := \frac{f_{yk.HEB}}{\gamma_{M1}} = 322.727 \cdot \text{MPa}$$

Resistance of steel profiles against bending around x-axis (weak direction)

The calculations below are based on Section S4.2 in Al-Emrani et al. (2010)

Assuming that the concrete column deforms so much after the load increase that yielding can be reached in the steel profiles.

Sectional class of the web:

Assuming pure compression in the web.

$$\varepsilon_{class} := \sqrt{\frac{235}{355}} = 0.814$$

$d_{web} := 180\text{mm} - 2 \cdot 14\text{mm} = 0.152\,\text{m}$

$t_{web} := 8.5\text{mm}$

$\varepsilon_{class} \cdot 33 = 26.849$

$$\frac{d_{web}}{t_{web}} = 17.882$$ Web in class 1

Sectional class for the flanges that are in compression:

$$c_{HEB} := \frac{180mm}{2} = 90 \cdot mm$$

$$t_f := 14mm$$

$$\varepsilon_{class} \cdot 9 = 7.323$$

$$\frac{c_{HEB}}{t_f} = 6.429$$ Compressed flange in class 1

Sectional class for the flanges that have a tip that is in tension (due to bending):

$$\alpha_{class} := 0.346$$ Ratio of how big part of the flange that is in tension (Iterated)

$$\frac{9 \cdot \varepsilon_{class}}{\alpha_{class} \cdot \sqrt{\alpha_{class}}} = 35.979$$

$$\frac{c_{HEB}}{t_f} = 6.429$$ Flanges that have a tip that is in tension is in class 1

Second order moment in steel profiles:
The calculations below are based on Section S6.4 in Al-Emrani et al. (2011)

Using plastic analysis since the profile is in class 1.

$$\alpha_{bow} := 150$$ Imperfection factor (class c, plastic analysis)

$$e_{0d.HEB} := \frac{h_c}{\alpha_{bow}} = 0.029\,m$$ Initial imperfections for the steel profiles

$$N_{Ed.HEB} := 0.98MN$$ Load on one steel profile (iterated to find good utilisation)

$$N_{B.HEB} := E_s \cdot I_{HEB.x} \cdot \frac{\pi^2}{l_0^{\,2}} = 1.455 \cdot MN$$ Buckling load for one steel profile

$$M_{Ed.HEB} := N_{Ed.HEB} \cdot \left(\frac{N_{B.HEB}}{N_{B.HEB} - N_{Ed.HEB}} \cdot e_{0d.HEB} \right) = 86.044 \cdot kN \cdot m$$ Second order moment

$$A_{web} := (180mm - 2 \cdot 14mm) \cdot 8.5mm = 1.292 \times 10^{-3}\,m^2$$

$$F_{web} := A_{web} \cdot f_{yd.HEB} = 0.417 \cdot MN$$ Part of the normal force that is taken by the web.

$$y_n := \frac{N_{Ed.HEB} - F_{web}}{f_{yd.HEB} \cdot 14mm \cdot 4} = 31.154 \cdot mm$$ Distance from centre of section to tensile zone

$$M_{N.HEB} := f_{yd} \cdot 4 \cdot 14mm \cdot \left(\frac{180mm}{2} - y_n \right) \cdot \left(y_n + \frac{\frac{180mm}{2} - y_n}{2} \right) = 86.793 \cdot kN \cdot m$$

$$\frac{M_{Ed.HEB}}{M_{N.HEB}} = 0.991$$ Utilisation of the capacity of the steel column

$\alpha_{class} := \frac{y_n}{c_{HEB}} = 0.346$ Compare with above

To be able to utilize $N_{Ed.HEB}$, the strain at mid section must increase enough from the quasi-permanent load to ULS.

$\varepsilon_{sy.HEB} := \frac{f_{yd.HEB}}{E_s} = 1.614 \times 10^{-3}$ Strain when the steel in the profiles reach yielding

$\varepsilon_{c.qp.mid} := 4.533 \cdot 10^{-4}$ Strain at mid section of concrete for quasi-permanent load (from previous part)

$\varepsilon_{c.mid} := 2.133 \cdot 10^{-3}$ Strain at mid section of concrete for ultimate load (from previous part)

$\varepsilon_{c.mid} - \varepsilon_{c.qp.mid} = 1.68 \times 10^{-3}$ Strain difference at mid section of concrete

$\varepsilon_{c.mid} - \varepsilon_{c.qp.mid} \geq \varepsilon_{sy.HEB} = 1$ Check if the strain difference is big enough to allow full utilisation of the steel profiles.

Load increase

Since no bending interaction between the concrete and the steel profiles is assumed, the increased ultimate load due to the strengthening just becomes the ultimate load before strengthening plus the ultimate load on the steel columns.

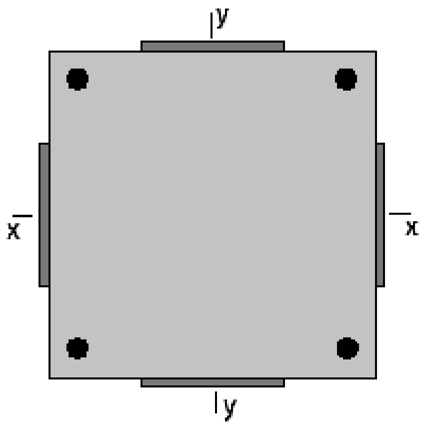
$N_{Ed.2} := N_{Ed} + 2 \cdot N_{Ed.HEB} = 15.655 \cdot MN$

$factor_2 := \frac{N_{Ed.2}}{N_{Ed}} = 1.143$ The load can in this case be increased with 14.3%

Part 2c - Strengthening with vertically loaded steel profiles (without prestressing and bending interaction)

Part 3 - Strengthening with vertically mounted steel plates

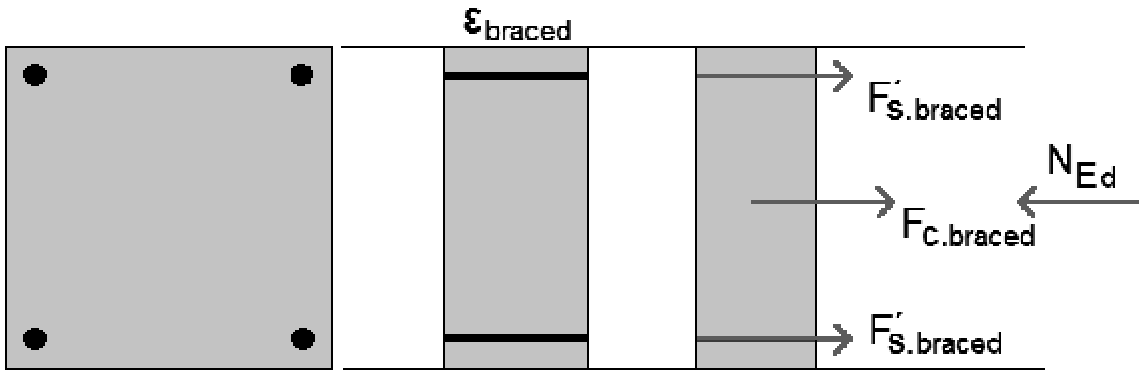
In this case, the rectangular section is strengthened with vertical steel plates that are assumed to only resist bending moment. The added load is chosen so that the column itself can take the increased compression if it is evenly distributed, but that the second order moment results in failure. The steel plates are therefore applied to increase the resistance against bending moment. The intention was to increase the load as much as for the case in Part 2a, but the N-M relationship for the column limits the load increase drastically. It is assumed that the column is braced (forced to vertical alignment) temporarily when the load is increased so that the column only is subjected to evenly distributed compression (which it can resist). The steel plates are then applied and the bracing is removed so that the second order moment is activated. Only the contribution from the steel plate that is in tension is regarded since it is more difficult to transfer compression from the plate to the concrete than tension. Full bending interaction between the plate and the concrete is assumed.



Increasing the load on the braced column before strengthening

$factor_3 := 1.01$ Increasing the load from Part 1 (iterated)

$N_{Ed.3} := factor_3 \cdot N_{Ed} = 13.832 \cdot MN$ New vertical load on top of the column



Horizontal equilibrium:

$N_{Ed.3} = F_{c.braced} + F'_{s.braced} + F_{s.braced}$

$\varepsilon_{braced} := 0.001594$ Assuming a value for the strain and iterating

$$F'_{s.braced} := \begin{cases} (\varepsilon_{braced} \cdot E_s \cdot A'_s) & \text{if } \varepsilon_{braced} < \varepsilon_{sy} \\ (f_{yd} \cdot A'_s) & \text{otherwise} \end{cases} = 512.788 \cdot kN$$

$$F_{s.braced} := \begin{cases} (\varepsilon_{braced} \cdot E_s \cdot A_s) & \text{if } \varepsilon_{braced} < \varepsilon_{sy} \\ (f_{yd} \cdot A_s) & \text{otherwise} \end{cases} = 512.788 \cdot kN$$

$\varepsilon_{c2} := 2.0 \cdot 10^{-3}$ Strain at which the concrete reaches f_{cd}

$$\sigma_{c.braced} := \begin{cases} \left[1 - \left(1 - \frac{\varepsilon_{braced}}{\varepsilon_{c2}}\right)^2\right] \cdot f_{cd} & \text{if } \varepsilon_{braced} < \varepsilon_{c2} \\ f_{cd} & \text{otherwise} \end{cases} = 25.568 \cdot MPa$$

$$F_{c.braced} := \sigma_{c.braced} \cdot (A_c - A'_s - A_s) = 12.806 \cdot MN$$

$F_{c.braced} + F'_{s.braced} + F_{s.braced} = 13.832 \cdot MN$

$N_{Ed.3} = 13.832 \cdot MN$

These equations should give the same results for horizontal equilibrium. Otherwise, ε_{braced} is updated

Steel plates

$b_{spi} := 600mm$ Width of each steel plate

$t_{spi} := 10mm$ Thickness of each steel plate

$A'_{sp} := b_{spi} \cdot t_{spi} = 6 \times 10^3 \cdot mm^2$ Area of steel plate on top surface

$A_{sp} := b_{spi} \cdot t_{spi} = 6 \times 10^3 \cdot mm^2$ Area of steel plate on bottom surface

$A_{spm} := 2 \cdot b_{spi} \cdot t_{spi} = 1.2 \times 10^4 \cdot mm^2$ Area of steel plates on side surfaces

$f_{yk.sp} := 355MPa$ Strength of steel plates (steel class S355)

$$f_{yd.sp} := \frac{f_{yk.sp}}{\gamma_{M1}} = 322.727 \cdot MPa$$

$\varepsilon_{sy.sp} := \dfrac{f_{yd.sp}}{E_s} = 1.614 \times 10^{-3}$ Yield strain for steel plates

First order moment

$e_0 = 0$

$e_i = 10.368 \cdot mm$ The eccentricity should be the same as before

$$M_{0.Ed.3} := N_{Ed.3} \cdot (e_0 + e_i) = 143.413 \cdot kN \cdot m$$

Nominal bending stiffness

$I_{spi.stiff} := \dfrac{t_{spi} \cdot b_{spi}^3}{12} = 1.8 \times 10^8 \cdot mm^4$ Second moment of inertia for one steel plate around its own axis

$$I_{spi.weak} := \frac{b_{spi} \cdot t_{spi}^3}{12} = 5 \times 10^4 \cdot mm^4$$

Adding the bending stiffness of the four steel plates to the nominal bending stiffness:

$$EI_3 := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_s \cdot I_{spi.stiff} + 2 \cdot E_s \cdot \left[I_{spi.weak} + A_{sp} \cdot \left(\frac{b_c}{2} + \frac{t_{spi}}{2} \right)^2 \right]$$

$$EI_3 = 495.41 \cdot MN \cdot m^2$$

Second order moment

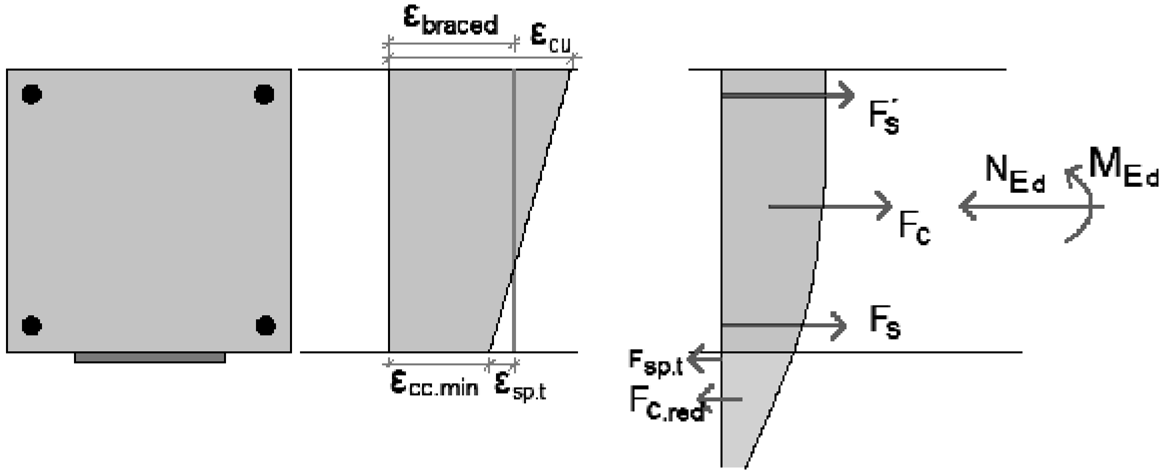
$$\beta_{shape} = 1$$

$$N_{B.3} := \frac{\pi^2 \cdot EI_3}{l_0^{\,2}} = 264.44 \cdot MN$$ Buckling load

$$M_{Ed.3} := \left(1 + \frac{\beta_{shape}}{\frac{N_{B.3}}{N_{Ed.3}} - 1} \right) \cdot M_{0.Ed.3} = 151.328 \cdot kN \cdot m$$

Resistance of the section

The bracing is removed so that the second order moment is introduced. Only the steel plate that is on the "tensile" surface is regarded in the equilibrium since it is more difficult to ensure that compression is taken by the external steel.



$$\varepsilon_{braced} = 1.594 \times 10^{-3}$$

Horizontal equilibrium:

Assuming that only the steel plate that is in tension contribute to the capacity.

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_3 - \alpha_{red.3} \cdot f_{cd} \cdot b_c \cdot (x_3 - b_c) + \sigma'_{s3} \cdot A'_s + \sigma_{s3} \cdot A_s = N_{Ed.3} + \sigma_{sp.t} \cdot A_{sp.t}$$

$$\sigma'_{s3} := f_{yd} = 434.783 \cdot MPa$$ Assuming that top reinforcement yields

$$\sigma_{s3} = \varepsilon_{s3} \cdot E_s$$ Assuming that bottom reinforcement doesn't yield

$$\varepsilon_{s3} = \varepsilon_{cu} \cdot \frac{x_3 - d_s}{x_3}$$

$$\sigma_{sp.t} = \varepsilon_{sp.t} \cdot E_s$$ Assuming that bottom reinforcement doesn't yield

$$\varepsilon_{sp.t} = \varepsilon_{braced} - \varepsilon_{cu} \cdot \frac{x_3 - b_c - \frac{t_{spi}}{2}}{x_3}$$ Tensile strain in the steel plate

$$\varepsilon_{cc.min.3} := 1.19 \cdot 10^{-3}$$ Assuming a value of the strain at the lower surface

$$\alpha_{red.3} := 0.417 + (0.48 - 0.417) \cdot \frac{\left(\varepsilon_{cc.min.3} \cdot 10^{3} - 1.0\right)}{(1.2 - 1.0)} = 0.477$$ Factors for the part of the compression block that comes below the section.

$$\beta_{red.3} := 0.35 + (0.354 - 0.35) \cdot \frac{\left(\varepsilon_{cc.min.3} \cdot 10^{3} - 1.0\right)}{(1.2 - 1.0)} = 0.354$$

$$x_3 := 1\text{m}$$

Given

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_3 - \alpha_{red.3} \cdot f_{cd} \cdot b_c \cdot \left(x_3 - b_c\right) + \sigma'_{s3} \cdot A'_s \ldots = N_{Ed.3} + \left(\varepsilon_{braced} - \varepsilon_{cu} \cdot \frac{x_3 - b_c - \frac{t_{spi}}{2}}{x_3}\right) \cdot E_s \cdot A_{sp}$$
$$+ E_s \varepsilon_{cu} \cdot \frac{x_3 - d_s}{x_3} \cdot A_s$$

$$x_3 := \text{Find}\left(x_3\right) = 1.076\text{m}$$ Height of compressive zone

Check of assumptions:

$$\varepsilon_{cc.min.3} := \varepsilon_{cu} \cdot \frac{x_3 - b_c}{x_3} = 1.19 \times 10^{-3}$$ Check with assumption and iterate

$$\varepsilon'_{s3} := \varepsilon_{cu} \cdot \frac{x_3 - d'_s}{x_3} = 3.324 \times 10^{-3}$$

$$\varepsilon'_{s3} \geq \varepsilon_{sy} = 1$$ Top reinforcement yielding

$$\varepsilon_{s3} := \varepsilon_{cu} \cdot \frac{x_3 - d_s}{x_3} = 1.366 \times 10^{-3}$$

Bottom reinforcement NOT yielding

$$\varepsilon_{s3} \geq \varepsilon_{sy} = 0$$

$$\varepsilon_{sp.t} := \varepsilon_{braced} - \varepsilon_{cu} \cdot \frac{x_3 - b_c - \frac{t_{spi}}{2}}{x_3} = 4.198 \times 10^{-4}$$

$$\varepsilon_{sp.t} \geq \varepsilon_{sy.sp} = 0$$ Steel plate in tension NOT yielding

Moment equilibrium around tensile reinforcement:

Only the contribution from the steel plate in tension is regarded.

$$M_{Rd.3} := \alpha \cdot f_{cd} \cdot b_c \cdot x_3 \cdot \left(d_s - \beta \cdot x_3\right) + \alpha_{red.3} \cdot f_{cd} \cdot b_c \cdot \left(x_3 - b_c\right) \cdot \left[b_c - d_s + \beta_{red.3} \cdot \left(x_3 - b_c\right)\right] \ldots$$
$$+ \sigma'_s \cdot A'_s \cdot \left(d_s - d'_s\right) + \varepsilon_{sp.t} \cdot E_s \cdot A_{sp} \cdot \left(b_c - d_s + \frac{t_{spi}}{2}\right) - N_{Ed.3} \cdot \left(d_s - \frac{b_c}{2}\right)$$

$$M_{Rd.3} = 332.324 \cdot \text{kN} \cdot \text{m}$$

Check of resistance

$e_{min} = 0.024\,m$ Minimum eccentricity for normal force

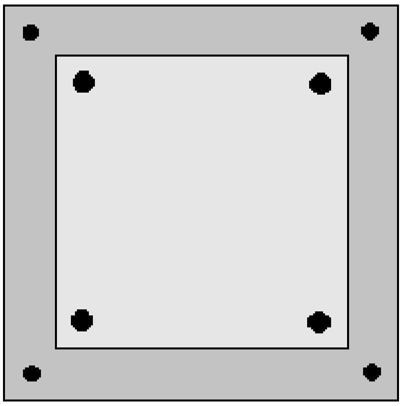
$\frac{M_{Ed.3}}{M_{Rd.3}} = 0.455$ $\frac{N_{Ed.3} \cdot e_{min}}{M_{Rd.3}} = 0.985$

The calculations show that the load only can be increased marginally before the ultimate capacity is reached.

Part 3 - Strengthening with vertically mounted steel plates

Part 4 - Strengthening with section enlargement

In this case, the column is strengthened with an additional layer of reinforced concrete that is applied symmetrically on all sides of the column. It is assumed that full interaction between the old and new concrete can be accounted for and that the new layer can be loaded directly by the normal force from above.



Behaviour of the column under quasi-permanent load before strengthening

$\varepsilon_{c.qp.bottom} := 4.087 \times 10^{-4}$ — Strain at the bottom surface of the original section (from Part 2b)

$\varepsilon_{c.qp.top} := 4.978 \times 10^{-4}$ — Strain at the top surface of the original section (from Part 2b)

$curve_{qp} := \dfrac{\varepsilon_{c.qp.top} - \varepsilon_{c.qp.bottom}}{b_c} = 1.255 \times 10^{-4} \dfrac{1}{m}$ — Curvature in the column before strengthening under quasi-permanent load.

$c_{shape} := 10$ — Factor that consider the curvature distribution

$e_{2.qp} := \dfrac{l_0^{\,2}}{c_{shape}} \cdot curve_{qp} = 0.232 \cdot mm$ — Eccentricity in critical section due to second order effects before strengthening

$e_{qp} := e_0 + e_i + e_{2.qp} = 10.6 \cdot mm$ — Eccentricity in critical section before strengthening

Loads after load increase

$factor_4 := 1.28$ — Increasing the load from Part 1 (with the same amount as in Part 2a)

$N_{Ed.4} := factor_4 \cdot N_{Ed} = 17.53 \cdot MN$ — New vertical load on top of the column, ULS

$N_{Eqp.4} := factor_4 \cdot N_{Eqp} = 10.545 \cdot MN$ — Quasi-permanent SLS combination

First order moments after the load increase

Using the eccentricity that takes the original creep into consideration. In this way, the improved nominal bending stiffness is only accounted for for the load increase.

$M_{0Ed.4} := N_{Ed.4} \cdot e_{qp} = 185.818 \cdot kN \cdot m$ — ULS combination

$M_{0Eqp.4} := N_{Eqp.4} \cdot e_{qp} = 111.776 \cdot kN \cdot m$ — Quasi-permanent SLS combination

Input data for the new layer

Using the same concrete to simplify calculations

$\phi_{si.4} := 20mm$ — Reinforcement diameter for new bars

$$A_{si.4} := \frac{\pi \cdot \phi_{si.4}^2}{4}$$ Area of one new bar

$A_{s.4} := 4 \cdot A_{si.4} + 4 \cdot A_{si} = 4.474 \times 10^{-3}\,m^2$ — Area of all bars in the section

$c_4 := 30mm$ — Concrete cover (roughly chosen)

$a_4 := 40mm$ — Thickness of new layer (iterated until the same load increase as in Part 2a is possible)

$b_{c.4} := b_c + 2 \cdot a_4 = 0.79\,m$ — Width of the new column

$A_{c.4} := 4 \cdot (b_{c.4} - a_4) \cdot a_4 = 0.12\,m^2$ — Gross area of the added layer

$A_{c.4.tot} := b_c^{\,2} + A_{c.4} = 0.624\,m^2$ — Gross area of the total section

Evaluation of slenderness

The calculations below are based on Section B11.3.2 in Al-Emrani et al. (2011)

The column is regarded as an isolated structural member with pinned connections in each end.

$l_0 = 4.3\,m$ — Buckling length (assumed pinned-pinned connections)

$$I_{c.4} := \frac{b_{c.4} \cdot b_{c.4}^{\,3}}{12} = 0.032\,m^4$$ Second moment of inertia of the gross concrete section

$$i_4 := \sqrt{\frac{I_{c.4}}{A_{c.4.tot}}} = 0.228\,m$$ Radius of gyration

$$\lambda_4 := \frac{l_0}{i_4} = 18.855$$ Slenderness

Rough estimation of the limit value of the slenderness:

$$n_4 := \frac{N_{Ed.4}}{f_{cd} \cdot A_{c.4.tot}} = 1.053$$ Relative normal force

$$\lambda_{lim.4} := \frac{10.8}{\sqrt{n_4}} = 10.523$$ Rough value of limit

Since $\lambda_4 > \lambda_{lim.4}$, the column must be designed with regard to the second order moment.

Creep for additional load if the whole section would have been cast at the same time as the original section

The calculations below are based on Section B2.1.6 in Al-Emrani et al. (2010).
It is first assumed that the whole section was cast at the same time as the original column so that it is old when the load is increased. The results are then weighted against calculations where it is assumed that the whole section is newly cast.

$t_{increase.1} := 40 \cdot 365 = 1.46 \times 10^4$ Concrete age in days at the time when the load is increased (assuming 40 years)

$RH = 50 \cdot \%$ Indoor climate

$u_4 := 4 \cdot b_{c.4} = 3.16\,m$ All sides of the column are subjected to drying

$h_{0.4} := \frac{2 \cdot A_{c.4.tot}}{u_4} = 0.395\,m$ Nominal thickness

$f_{cm} = 48 \cdot MPa$ Mean value of compressive strength of concrete

$$\varphi_{RH.4} := \left[1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{\frac{h_{0.4}}{mm}}} \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.7}\right] \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.2} = 1.452$$ Creep from relative humidity

$\beta_{fcm.4} := 2.43$ Factor that considers the strength of the concrete

$\beta_{t.increase.1} := \frac{1}{0.1 + t_{increase.1}^{0.20}} = 0.145$ Assuming that the additional load was applied after 40 years

$\varphi_{inf.increase.1} := \varphi_{RH.4} \cdot \beta_{fcm.4} \cdot \beta_{t.increase.1} = 0.511$ Final creep

Creep for additional load if the whole section was cast 28 days before the load was increased

$t_{increase.2} := 28$ Concrete age in days at the time when the load is increased (this time assuming 28 days)

$$\beta_{t.increase.2} := \frac{1}{0.1 + t_{increase.2}^{0.20}} = 0.488$$

$$\varphi_{inf.increase.2} := \varphi_{RH.4} \cdot \beta_{fcm.4} \cdot \beta_{t.increase.2} = 1.723$$

Weighting the two creep factors for the load increase

Since the two ways to calculate the creep for the added load represent the two extremities, a weighted value is calculated. This value is based on how large part of the section that consist of old and new. concrete respectively.

$$\varphi_{inf.increase} := \frac{\varphi_{inf.increase.1} \cdot b_c^2 + \varphi_{inf.increase.2} \cdot A_{c.4}}{A_{c.4.tot}} = 0.744$$

$\varphi_{ef.increase} := \varphi_{inf.increase} \cdot \frac{M_{0Eqp.4}}{M_{0Ed.4}} = 0.447$ Effective creep

Nominal bending stiffness

The calculations below are based on Section B11.4.2 in Al-Emrani et al. (2011)

$\rho_{reinf.4} := \dfrac{A_{s.4}}{A_{c.4.tot}} = 7.168 \times 10^{-3}$ Reinforcement ratio in new layer

$\rho_{reinf.4} \geq 0.002 = 1$ OK!

$\gamma_{cE} = 1.2$ National parameter

$E_{cd} := \dfrac{E_{cm}}{\gamma_{cE}} = 29.167 \cdot GPa$ Design value of modulus of elasticity for the concrete

$k_{1.4} := \sqrt{\dfrac{\dfrac{f_{ck}}{MPa}}{20}} = 1.414$ Same as before (dependent on concrete)

$k_{2.4} := \dfrac{N_{Ed.4}}{f_{cd} \cdot A_{c.4.tot}} \cdot \dfrac{\lambda_4}{170} = 0.117$

Simplified second moment of inertia for reinforcement:

$$I_{s.4} := 4 \cdot A_{si} \cdot \left(\frac{b_c}{2} - cover - \phi_{st.i} - \frac{\phi_{si}}{2} \right)^2 + 4 \cdot A_{si.4} \cdot \left(\frac{b_{c.4}}{2} - c_4 - \frac{\phi_{si.4}}{2} \right)^2 = 4.498 \times 10^{-4} \, m^4$$

$EI_4 := \dfrac{k_{1.4} \cdot k_{2.4}}{1 + \varphi_{ef.increase}} \cdot E_{cd} \cdot I_{c.4} + E_s \cdot I_{s.4} = 198.023 \cdot MN \cdot m^2$ Nominal bending stiffness

Second order moment

$N_{B.4} := \dfrac{\pi^2 \cdot EI_4}{l_0^{\,2}} = 105.701 \cdot MN$ Theoretical buckling force

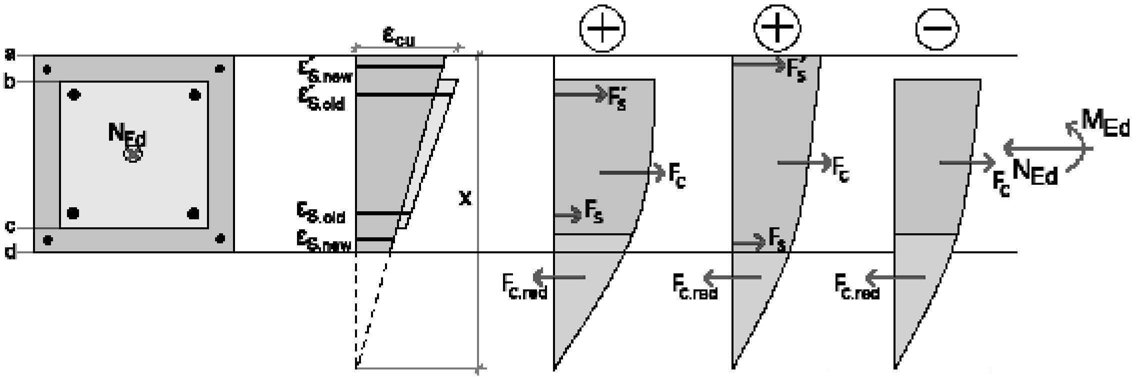
$\beta_{shape} = 1$ Due to sinus-shaped bending moment

$M_{Ed.4} := \left(1 + \dfrac{\beta_{shape}}{\dfrac{N_{B.4}}{N_{Ed.4}} - 1} \right) \cdot M_{0Ed.4} = 222.761 \cdot kN \cdot m$ Second order moment

Resistance of the section

The calculations below are based on Section B5.6 in Al-Emrani et al. (2010)

Assuming that the new layer of concrete also can help to resist the combination of normal force and bending moment. The strain difference between the two layers must however be regarded (the original column already had an initial strain when the new layer was cast). The figure below illustrates how the different strains were accounted for by adding together the stress blocks and then subtracting the central part of the block for the new concrete.



$\alpha = 0.81$

$\beta = 0.416$

$A_{s.old} := 2 \cdot A_{si} = 1.608 \times 10^3 \cdot mm^2$ Bottom reinforcement in old part

$A'_{s.old} := 2 \cdot A_{si} = 1.608 \times 10^3 \cdot mm^2$ Top reinforcement in old part

$A_{s.new} := 2 \cdot A_{si.4} = 628.319 \cdot mm^2$ Bottom reinforcement in new part

$A'_{s.new} := 2 \cdot A_{si.4} = 628.319 \cdot mm^2$ Top reinforcement in new part

$d'_{s.new} := c_4 + \frac{\phi_{si.4}}{2} = 40 \cdot mm$ Distances from the top surface to each reinforcement layer

$d'_{s.old} := a_4 + d'_s = 94 \cdot mm$

$d_{s.old} := a_4 + d_s = 696 \cdot mm$

$d_{s.new} := b_{c.4} - d'_{s.new} = 750 \cdot mm$

Calculating height of compressive zone:

$\varepsilon_{c.b.old} := \varepsilon_{cu}$

$\alpha_{c.b.old} := \alpha = 0.81$

$\beta_{c.b.old} := \beta = 0.416$

Strain in old concrete at level b (see figure above). Assuming that the old concrete reaches ε_{cu} first.

$\varepsilon_{c.b.new} := \varepsilon_{cu} - \varepsilon_{c.qp.top} = 3.002 \times 10^{-3}$ Strain in new concrete at level b

$$\alpha_{c.b.new} := 0.778 + (0.792 - 0.778) \cdot \frac{\varepsilon_{c.b.new} \cdot 10^3 - 3.0}{3.2 - 3.0} = 0.778$$

Stress block factors for the new concrete at level b

$$\beta_{c.b.new} := 0.405 + (0.41 - 0.405) \cdot \frac{\varepsilon_{c.b.new} \cdot 10^3 - 3.0}{3.2 - 3.0} = 0.405$$

$\varepsilon_{c.c.old} := 0.00152$

Strain in old concrete at level c (iterated until horizontal equilibrium is fulfilled)

$$\alpha_{c.c.old} := 0.537 + (0.587 - 0.537)\cdot\frac{\varepsilon_{c.c.old}\cdot 10^3 - 1.4}{1.6 - 1.4} = 0.567$$

Stress block factors for the old concrete at level b

$$\beta_{c.c.old} := 0.359 + (0.364 - 0.359)\cdot\frac{\varepsilon_{c.c.old}\cdot 10^3 - 1.4}{1.6 - 1.4} = 0.362$$

$$\varepsilon_{c.c.new} := \varepsilon_{c.c.old} - \varepsilon_{c.qp.bottom} = 1.111\times 10^{-3}$$

Strain in new concrete at level c

$$\alpha_{c.c.new} := 0.417 + (0.48 - 0.417)\cdot\frac{\varepsilon_{c.c.new}\cdot 10^3 - 1}{1.2 - 1} = 0.452$$

Stress block factors for the new concrete at level c

$$\beta_{c.c.new} := 0.35 + (0.354 - 0.35)\cdot\frac{\varepsilon_{c.c.new}\cdot 10^3 - 1}{1.2 - 1} = 0.352$$

$$x_4 := \frac{b_c\cdot\varepsilon_{cu}}{\varepsilon_{cu} - \varepsilon_{c.c.old}} + a_4 = 1.295\,m$$

Height of compressive zone

$$\varepsilon_{c.a.new} := \varepsilon_{c.b.new}\cdot\frac{x_4}{x_4 - a_4} = 3.098\times 10^{-3}$$

Strain in new concrete at level a

$$\alpha_{c.a.new} := 0.778 + (0.792 - 0.778)\cdot\frac{\varepsilon_{c.a.new}\cdot 10^3 - 3}{3.2 - 3} = 0.785$$

Stress block factors for the new concrete at level a

$$\beta_{c.a.new} := 0.405 + (0.41 - 0.405)\cdot\frac{\varepsilon_{c.a.new}\cdot 10^3 - 3}{3.2 - 3} = 0.407$$

$$\varepsilon_{c.d.new} := \varepsilon_{c.b.new}\cdot\frac{x_4 - b_{c.4}}{x_4 - a_4} = 1.208\times 10^{-3}$$

Strain in new concrete at level d

$$\alpha_{c.d.new} := 0.48 + (0.537 - 0.48)\cdot\frac{\varepsilon_{c.d.new}\cdot 10^3 - 1.2}{1.4 - 1.2} = 0.482$$

Stress block factors for the new concrete at level d

$$\beta_{c.d.new} := 0.354 + (0.359 - 0.354)\cdot\frac{\varepsilon_{c.d.new}\cdot 10^3 - 1.2}{1.4 - 1.2} = 0.354$$

$$\varepsilon'_{s.new} := \varepsilon_{c.a.new}\cdot\frac{x_4 - d'_{s.new}}{x_4} = 3.002\times 10^{-3} \qquad \varepsilon'_{s.new} \ge \varepsilon_{sy} = 1$$

Top reinf. in new part yielding

$$F'_{s.new} := f_{yd}\cdot A'_{s.new} = 273.182\cdot kN$$

$$\varepsilon'_{s.old} := \varepsilon_{cu}\cdot\frac{x_4 - d'_{s.old}}{x_4 - a_4} = 3.349\times 10^{-3} \qquad \varepsilon'_{s.old} \ge \varepsilon_{sy} = 1$$

Top reinf. in old part yielding

$$F'_{s.old} := f_{yd}\cdot A'_{s.old} = 699.346\cdot kN$$

$$\varepsilon_{s.old} := \varepsilon_{cu}\cdot\frac{x_4 - d_{s.old}}{x_4 - a_4} = 1.671\times 10^{-3} \qquad \varepsilon_{s.old} \ge \varepsilon_{sy} = 0$$

Bottom reinf. in old part NOT yielding

$F_{s.old} := \varepsilon_{s.old} \cdot E_s \cdot A_{s.old} = 537.428 \cdot kN$

$\varepsilon_{s.new} := \varepsilon_{c.a.new} \cdot \dfrac{x_4 - d_{s.new}}{x_4} = 1.304 \times 10^{-3}$ $\varepsilon_{s.new} \ge \varepsilon_{sy} = 0$ Bottom reinf. in new part NOT yielding

$F_{s.new} := \varepsilon_{s.new} \cdot E_s \cdot A_{s.new} = 163.842 \cdot kN$

Check of horizontal equilibrium:

$$\begin{aligned} N_{Rd.4} := {} & \alpha_{c.b.old} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4) - \alpha_{c.c.old} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4 - b_c) \ldots \quad = 17.525 \cdot MN \\ & + \alpha_{c.a.new} \cdot f_{cd} \cdot b_{c.4} \cdot x_4 - \alpha_{c.d.new} \cdot f_{cd} \cdot b_{c.4} \cdot (x_4 - b_{c.4}) \ldots \\ & + -\left[\alpha_{c.b.new} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4) - \alpha_{c.c.new} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4 - b_c)\right] \ldots \\ & + F'_{s.new} + F'_{s.old} + F_{s.old} + F_{s.new} \end{aligned}$$

$N_{Ed.4} = 17.53 \cdot MN$

$N_{Rd.4}$ and $N_{Ed.4}$ should have the same value. Otherwise, $\varepsilon_{c.c.old}$ should be updated.

Moment equilibrium around neutral layer:

$$\begin{aligned} M_{Rd.4} := {} & \alpha_{c.b.old} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4) \cdot (1 - \beta_{c.b.old}) \cdot (x_4 - a_4) \ldots \quad = 487.557 \cdot kN \cdot m \\ & + -\alpha_{c.c.old} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4 - b_c) \cdot (1 - \beta_{c.c.old}) \cdot (x_4 - a_4 - b_c) \ldots \\ & + \alpha_{c.a.new} \cdot f_{cd} \cdot b_{c.4} \cdot x_4 \cdot (1 - \beta_{c.a.new}) \cdot x_4 \ldots \\ & + -\alpha_{c.d.new} \cdot f_{cd} \cdot b_{c.4} \cdot (x_4 - b_{c.4}) \cdot (1 - \beta_{c.d.new}) \cdot (x_4 - b_{c.4}) \ldots \\ & + -\left[\begin{array}{l} \alpha_{c.b.new} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4) \cdot (1 - \beta_{c.b.new}) \cdot (x_4 - a_4) \ldots \\ + -\alpha_{c.c.new} \cdot f_{cd} \cdot b_c \cdot (x_4 - a_4 - b_c) \cdot (1 - \beta_{c.c.new}) \cdot (x_4 - a_4 - b_c) \end{array}\right] \ldots \\ & + F'_{s.new} \cdot (x_4 - d'_{s.new}) + F'_{s.old} \cdot (x_4 - d'_{s.old}) + F_{s.old} \cdot (x_4 - d_{s.old}) \ldots \\ & + F_{s.new} \cdot (x_4 - d_{s.new}) - N_{Ed.4} \cdot \left(x_4 - \frac{b_{c.4}}{2}\right) \end{aligned}$$

Check of resistance

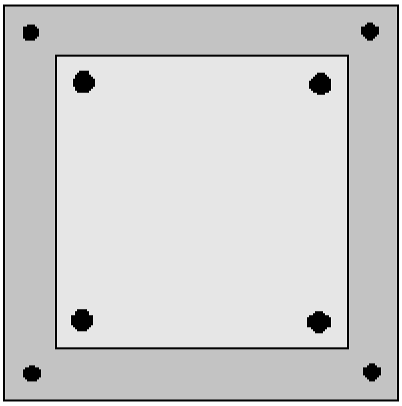
$e_{min.4} := \max\left(\dfrac{b_{c.4}}{30}, 20mm\right) = 26.333 \cdot mm$ Minimum eccentricity for normal force

$\dfrac{M_{Ed.4}}{M_{Rd.4}} = 0.457$ $\dfrac{N_{Ed.4} \cdot e_{min.4}}{M_{Rd.4}} = 0.947$

Part 4 - Strengthening with section enlargement

Part 5 - Strengthening with section enlargement - assumed to only contribute to bending stiffness

In this case, the column is strengthened in the same way as in Part 4, but the section enlargement is not accounted for in the calculation of the resistance of the critical section. The added layer is assumed to only contribute to the nominal bending stiffness.



$e_{qp} = 0.011\,m$

The eccentricity before strengthening is the same as in Part 4

Loads after increase

$factor_5 := 1.00$

$N_{Ed.5} := factor_5 \cdot N_{Ed} = 13.695 \cdot MN$

$N_{Eqp.5} := factor_5 \cdot N_{Eqp} = 8.238 \cdot MN$

First order moments after the load increase

Using the eccentricity that takes the original creep into consideration.

$M_{0Ed.5} := N_{Ed.5} \cdot e_{qp} = 145.171 \cdot kN \cdot m$ ULS combination

$M_{0Eqp.5} := N_{Eqp.5} \cdot e_{qp} = 87.325 \cdot kN \cdot m$ Quasi-permanent SLS combination

Creep after the load has been increased

The creep factor is the same as in Part 4

$\varphi_{inf.increase} = 0.744$

$$\varphi_{ef.increase} := \varphi_{inf.increase} \cdot \frac{M_{0Eqp.5}}{M_{0Ed.5}} = 0.447$$

Nominal bending stiffness

All input data for the nominal bending stiffness is the same as for Part 4 except the factor k_2 since it depends on N_{Ed}.

$$k_{2.5} := \frac{N_{Ed.5}}{f_{cd} \cdot A_{c.4.tot}} \cdot \frac{\lambda_4}{170} = 0.091$$

$$EI_5 := \frac{k_{1.4} \cdot k_{2.5}}{1 + \varphi_{ef.increase}} \cdot E_{cd} \cdot I_{c.4} + E_s \cdot I_{s.4} = 174.385 \cdot MN \cdot m^2$$

Second order moment

$$N_{B.5} := \frac{\pi^2 \cdot EI_5}{l_0^2} = 93.084 \cdot MN$$ Theoretical buckling force

$\beta_{shape} = 1$ Due to sinus-shaped bending moment

$$M_{Ed.5} := \left(1 + \frac{\beta_{shape}}{\frac{N_{B.5}}{N_{Ed.5}} - 1}\right) \cdot M_{0Ed.5} = 170.213 \cdot kN \cdot m$$ Second order moment

Resistance of the section

The calculations below are based on Section B5.6 in Al-Emrani et al. (2010)

Assuming that the new layer of concrete CANNOT help to resist the combination of normal force and bending moment.

$\alpha = 0.81$

$\beta = 0.416$

$A_s = 1.608 \times 10^{-3} m^2$ Bottom reinforcement in old part

$A'_s = 1.608 \times 10^{-3} m^2$ Top reinforcement in old part

$d_s = 656 \cdot mm$

$d'_s = 54 \cdot mm$

$\varepsilon_{cu} = 3.5 \times 10^{-3}$ Maximal strain for the concrete

Horizontal equilibrium:

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_5 - \alpha_{red.5} \cdot f_{cd} \cdot b_c \cdot (x_5 - b_c) + \sigma'_s \cdot A'_s + \sigma_s \cdot A_s = N_{Ed.5}$$

$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement in old part yields

$\sigma_s = \varepsilon_s \cdot E_s$ Assuming that bottom reinforcement in old part doesn't yield

$$\varepsilon_s = \varepsilon_{cu} \cdot \frac{x_5 - d_s}{x_5}$$

Calculating height of compressive zone:

$\varepsilon_{cc.min.5} := 0.0007687$ Assuming that the whole section is in compression.

$$\alpha_{red.5} := 0.347 + (0.417 - 0.347) \cdot \frac{\left(\varepsilon_{cc.min.5} \cdot 10^3 - 0.8\right)}{(0.8 - 0.6)} = 0.336$$

$$\beta_{red.5} := 0.346 + (0.350 - 0.346) \cdot \frac{\left(\varepsilon_{cc.min.5} \cdot 10^3 - 0.8\right)}{(0.8 - 0.6)} = 0.345$$

Factors for the part of the compression block that comes below the section. α_{red} and β_{red} is in this case dependent on the strain at the the bottom of the cross-section.

$x_5 := 0.5\text{m}$ Assuming an initial value for x_5

Given

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_5 - \alpha_{red.5} \cdot f_{cd} \cdot b_c \cdot \left(x_5 - b_c\right) + \sigma'_s \cdot A'_s + \varepsilon_{cu} \cdot \frac{x_5 - d_s}{x_5} \cdot E_s \cdot A_s = N_{Ed.5}$$

$x_5 := \text{Find}\left(x_5\right) = 0.91\,\text{m}$ Solving x from horizontal equilibrium

Check of assumptions:

$\varepsilon_{cc.min.5} := \varepsilon_{cu} \cdot \frac{x_5 - b_c}{x_5} = 7.687 \times 10^{-4}$ Concrete strain at "bottom side". Check with assumption and iterate.

$\varepsilon_{sy} = 2.174 \times 10^{-3}$ Steel strain at yielding

$\varepsilon'_s := \varepsilon_{cu} \cdot \frac{x_5 - d'_s}{x_5} = 3.292 \times 10^{-3}$ $\quad \varepsilon'_s \geq \varepsilon_{sy} = 1$ Top reinf. in old part yielding

$\varepsilon_s := \varepsilon_{cu} \cdot \frac{x_5 - d_s}{x_5} = 9.764 \times 10^{-4}$ $\quad \varepsilon_{s.old} \geq \varepsilon_{sy} = 0$ Bottom reinf. in old part NOT yielding

Moment equilibrium around bottom reinforcement in old part:

$$M_{Rd.5} := \alpha \cdot f_{cd} \cdot b_c \cdot x_5 \cdot \left(d_s - \beta \cdot x_5\right) + \alpha_{red.5} \cdot f_{cd} \cdot b_c \cdot \left(x_5 - b_c\right) \cdot \left[b_c - d_s + \beta_{red.5} \cdot \left(x_5 - b_c\right)\right] \ldots = 327.372 \cdot \text{kN} \cdot \text{m}$$
$$+ \sigma'_s \cdot A'_s \cdot \left(d_s - d'_s\right) - N_{Ed.5} \cdot \left(d_s - \frac{b_c}{2}\right)$$

Check of resistance

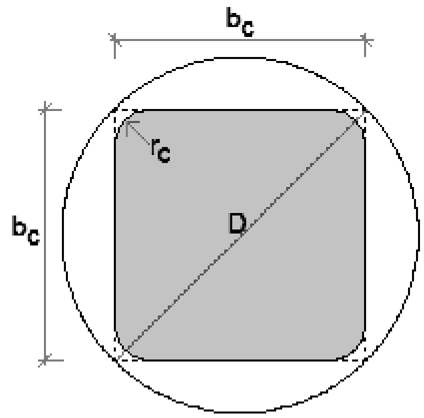
$e_{min.5} := \max\left(\frac{b_c}{30}, 20\text{mm}\right) = 23.667 \cdot \text{mm}$ Minimum eccentricity for normal force

$\frac{M_{Ed.5}}{M_{Rd.5}} = 0.52$ $\quad \frac{N_{Ed.5} \cdot e_{min.5}}{M_{Rd.5}} = 0.99$

Part 5 - Strengthening with section enlargement - assumed to only contribute to bending stiffness

Part 6 - Strengthening with CFRP wrapping - Rectangular section

In this part, the same rectangular column as in Part 1 is strengthened with CFRP sheets that are wrapped around the column so that the fibres are placed in the circumferential direction. It is assumed that the whole column is strengthened and that the corners are smoothened. The calculations are based on Alston et. al. (2011), Chapter 6.



Increased compressive strength of the concrete

$b_{1c} := b_c = 710 \cdot mm$

$b_{2c} := b_c = 710 \cdot mm$

Should be lower than 900mm to make CFRP wrapping beneficial — OK

$\dfrac{b_{2c}}{b_{1c}} = 1$

Should be lower than 2 to make CFRP wrapping beneficial — OK

$r_{c.max} := \dfrac{cover}{1 - \cos(45deg)} = 102.426 \cdot mm$

Maximum radius of the smoothened corners (at which point the stirrups will be reached)

$r_{c.min} := 30mm$

Minimum radius of the smoothened corners according to Alston et al. (2011)

$r_c := 60mm$

Chosen radius of the smoothened corners

$D := \sqrt{b_{1c}^{\,2} + b_{2c}^{\,2}} = 1.004\,m$

Diameter of the fictitious circular column

$\rho_g := \dfrac{4 \cdot A_{si}}{A_c} = 6.382 \times 10^{-3}$

Reinforcement ratio

$$A_{ce} := \left(A_c - 4 \cdot A_{si}\right) \cdot \frac{1 - \dfrac{\dfrac{b_{1c}}{b_{2c}}\left(b_{2c} - 2 \cdot r_c\right)^2 + \dfrac{b_{2c}}{b_{1c}}\left(b_{1c} - 2 \cdot r_c\right)^2}{3 \cdot A_c} - \rho_g}{1 - \rho_g} = 0.269\,m^2$$

Effective concrete area

$\kappa_a := \dfrac{A_{ce}}{A_c - 4 \cdot A_{si}} \cdot \left(\dfrac{b_{1c}}{b_{2c}}\right)^2 = 0.537$

Geometrical efficiency factor a. Lower than one since the section is non-circular

$\kappa_b := \dfrac{A_{ce}}{A_c - 4 \cdot A_{si}} \cdot \left(\dfrac{b_{1c}}{b_{2c}}\right)^{0.5} = 0.537$

Geometrical efficiency factor b

$\varepsilon_{fu} := 1.55\%$ Ultimate strain in CFRP (using sheets of type S&P Alston 240)

$\kappa_e := 0.55$ Efficiency factor concerning premature failure in CFRP due to the triaxial stress situation

$\varepsilon_{fe} := \kappa_e \cdot \varepsilon_{fu} = 8.525 \times 10^{-3}$ Effective ultimate strain in CFRP

$E_f := 240\text{GPa}$ Modulus of elasticity for CFRP

$n_f := 11$ Number of layers of sheets

$t_f := 0.117\text{mm}$ Thickness of one sheet

$f_l := \dfrac{2 \cdot E_f \cdot n_f \cdot t_f \cdot \varepsilon_{fe}}{D} = 5.245 \times 10^6 \,\text{Pa}$ Maximum wrapping pressure

$\dfrac{f_l}{f_{ck}} = 0.131$ Should at least be 0.08

$\varepsilon_{c2} := 2.0 \cdot 10^{-3}$ Strain at which the curve for the concrete becomes horizontal

$$\varepsilon_{cu.c} := \varepsilon_{c2} \cdot \left[1.50 + 12 \cdot \kappa_b \cdot \frac{f_l}{f_{ck}} \cdot \left(\frac{\varepsilon_{fe}}{\varepsilon_{c2}} \right)^{0.45} \right] = 6.243 \times 10^{-3}$$

Check of maximum strain in wrapped concrete (should be below 10‰)

$\alpha_{f.c} := 0.95$ Reduction factor to increase the safety of the model

$f_{cd.c} := f_{cd} + \alpha_{f.c} \cdot 3.3 \cdot \kappa_a \cdot f_l = 35.491 \cdot \text{MPa}$ Increased compressive strength of concrete due to triaxial stress state

Loads

$\text{factor}_6 := 1.28$ Increasing the load from Part 1

$N_{Ed.6} := \text{factor}_6 \cdot N_{Ed} = 17.53 \cdot \text{MN}$ New vertical load on top of the column

First order moment

$e_0 = 0$

$e_i = 0.01\,\text{m}$ The eccentricity should be the same as before

$M_{0.Ed.6} := N_{Ed.6} \cdot (e_0 + e_i) = 181.751 \cdot \text{kN} \cdot \text{m}$

Nominal bending stiffness

Since the column hasn't been strengthened with regard to bending resistance, the nominal bending stiffness should be the same as in Part 1.

$EI = 112.35 \cdot \text{MN} \cdot \text{m}^2$

Second order moment

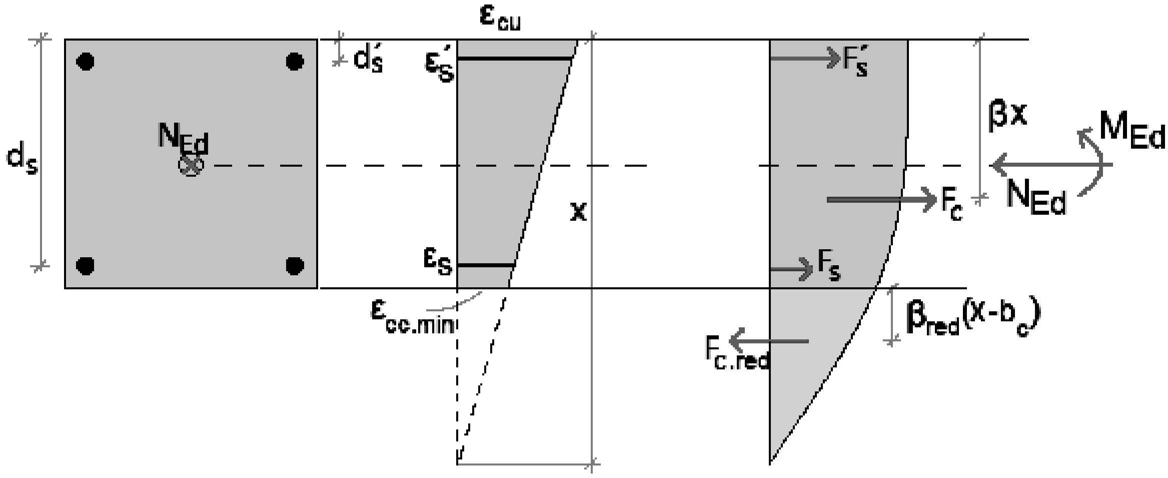
$N_{B.6} := \frac{\pi^2 \cdot EI}{l_0^{\,2}} = 59.97 \cdot MN$ Buckling load

$M_{Ed.6} := \left(1 + \frac{\beta_{shape}}{\frac{N_{B.6}}{N_{Ed.6}} - 1}\right) \cdot M_{0.Ed.6} = 256.82 \cdot kN \cdot m$

Resistance of the section

The calculations below are based on Section B5.6 in Al-Emrani et al. (2010)

Neglecting that the corners have been smoothened



$\alpha = 0.81$

$\beta = 0.416$

$A_s = 1.608 \times 10^{-3}\, m^2$ Bottom reinforcement

$A'_s = 1.608 \times 10^{-3}\, m^2$ Top reinforcement

$\varepsilon_{cu} = 3.5 \times 10^{-3}$ Maximal strain for the concrete

Horizontal equilibrium (using the increased compressive strength):

$\alpha \cdot f_{cd.c} \cdot b_c \cdot x_6 - \alpha_{red.6} \cdot f_{cd.c} \cdot b_c \cdot \left(x_6 - b_c\right) + \sigma'_s \cdot A'_s + \sigma_s \cdot A_s = N_{Ed.6}$

$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement yields

$\sigma_s = \varepsilon_s \cdot E_s$ Assuming that bottom reinforcement doesn't yield

$\varepsilon_s = \varepsilon_{cu} \cdot \frac{x_6 - d_s}{x_6}$

Calculating height of compressive zone:

$\varepsilon_{cc.min.6} := 0.0006265$ Assuming that the whole section is in compression.

$\alpha_{red.6} := 0.27 + (0.347 - 0.27) \cdot \frac{\left(\varepsilon_{cc.min.6} \cdot 10^3 - 0.6\right)}{(0.8 - 0.6)} = 0.28$

$\beta_{red.6} := 0.343 + (0.346 - 0.343) \cdot \frac{\left(\varepsilon_{cc.min.6} \cdot 10^3 - 0.6\right)}{(0.8 - 0.6)} = 0.343$

Factors for the part of the compression block that comes below the section. (Table B5.3)

$x_6 := 0.5\text{m}$ Assuming a value for x_6

Given

$$\alpha \cdot f_{cd.c} \cdot b_c \cdot x_6 - \alpha_{red.6} \cdot f_{cd.c} \cdot b_c \cdot \left(x_6 - b_c\right) + \sigma'_s \cdot A'_s + \varepsilon_{cu} \cdot \frac{x_6 - d_s}{x_6} \cdot E_s \cdot A_s = N_{Ed.6}$$

$x_6 := \text{Find}\left(x_6\right) = 0.865\,\text{m}$ Solving x_6 from horizontal equilibrium

Check of assumptions:

$\varepsilon_{cc.min.6} := \varepsilon_{cu} \cdot \frac{x_6 - b_c}{x_6} = 6.265 \times 10^{-4}$ Concrete strain at "bottom side". Check with assumption and iterate

$\varepsilon_{sy} = 2.174 \times 10^{-3}$ Steel strain at yielding

$\varepsilon'_s := \varepsilon_{cu} \cdot \frac{x_6 - d'_s}{x_6} = 3.281 \times 10^{-3}$ $\quad \varepsilon'_s \geq \varepsilon_{sy} = 1$ Top reinf. yielding

$\varepsilon_s := \varepsilon_{cu} \cdot \frac{x_6 - d_s}{x_6} = 8.45 \times 10^{-4}$ $\quad \varepsilon_s \geq \varepsilon_{sy} = 0$ Bottom reinf. NOT yielding

Moment equilibrium around tensile reinforcement:

$$M_{Rd.6} := \alpha \cdot f_{cd.c} \cdot b_c \cdot x_6 \cdot \left(d_s - \beta \cdot x_6\right) + \alpha_{red.6} \cdot f_{cd.c} \cdot b_c \cdot \left(x_6 - b_c\right) \cdot \left[b_c - d_s + \beta_{red.6} \cdot \left(x_6 - b_c\right)\right] \ldots$$
$$+ \sigma'_s \cdot A'_s \cdot \left(d_s - d'_s\right) - N_{Ed.6} \cdot \left(d_s - \frac{b_c}{2}\right)$$

$M_{Rd.6} = 490.854 \cdot \text{kN} \cdot \text{m}$

Check of resistance

$e_{min} = 0.024\,\text{m}$ Minimum eccentricity for normal force

$\frac{M_{Ed.6}}{M_{Rd.6}} = 0.523$ $\quad \frac{N_{Ed.6} \cdot e_{min}}{M_{Rd.6}} = 0.845$

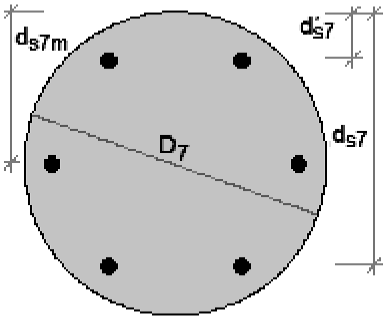
Part 6 - Strengthening with CFRP wrapping - Rectangular section

Part 7 - Capacity of column with circular section

To compare the efficiency of CFRP wrapping on a rectangular and circular columns, a column with circular section and similar capacity as the rectangular one must first be created.

Dimensions

The same concrete and steel in reinforcement as for the rectangular column is used. Six bars are used instead of four to give the column more even bending stiffness in different directions. To get approximately the same reinforcement area as in Part 1, smaller bars are used.



$D_7 := 800mm$ Diameter of the column

$A_{c.7} := \frac{\pi \cdot D_7^{\,2}}{4} = 0.503\,m^2$ Gross section area

$cover = 30 \cdot mm$ Thickness of concrete cover (same as before)

$\phi_{si7} := 26mm$ Diameter of bending reinforcement

$A'_{s7} := 2 \cdot \frac{\pi \cdot \phi_{si7}^{\,2}}{4} = 1.062 \times 10^{-3}\,m^2$ Area of the two top bars

$A_{s7m} := A'_{s7}$ Area of the two middle bars

$A_{s7} := A'_{s7}$ Area of the two bottom bars

$\phi_{st.i} = 8 \cdot mm$ Diameter of reinforcement in stirrup (same as before)

$D_{bars} := D_7 - 2 \cdot \left(cover + \phi_{st.i} + \frac{\phi_{si7}}{2} \right) = 0.698\,m$ Distance between bars on opposite sides

$d_{s7m} := \frac{D_7}{2} = 0.4\,m$ Distance from top to mid reinforcement

$d'_{s7} := \frac{D_7}{2} - \frac{D_{bars}}{2} \cdot \cos(30deg) = 97.757 \cdot mm$ Distance from top edge to top reinforcement

$d_{s7} := D_7 - d'_{s7} = 702.243 \cdot mm$ Distance from top edge to bottom reinforcement

Loads

$factor_7 := 1.0$ The load is chosen to be the same as in Part 1

$N_{Ed.7} := factor_7 \cdot N_{Ed} = 13.695 \cdot MN$ ULS combination

$N_{Eqp.7} := factor_7 \cdot N_{Eqp} = 8.238 \cdot MN$ Quasi-permanent SLS combination

Evaluation of slenderness

The calculations below are based on Section B11.3.2 in Al-Emrani et al. (2011)

The column is regarded as an isolated structural member with pinned connections in each end.

$l_0 = 4.3\,m$ Buckling length (assumed pinned-pinned connections)

$I_{c.7} := \frac{\pi}{4} \cdot \left(\frac{D_7}{2}\right)^4 = 0.02\,m^4$ Second moment of inertia of the gross concrete section

$i_7 := \sqrt{\frac{I_{c.7}}{A_{c.7}}} = 0.2\,m$ Radius of gyration

$\lambda_7 := \frac{l_0}{i_7} = 21.5$ Slenderness

Rough estimation of the limit value of the slenderness:

$n_7 := \frac{N_{Ed.7}}{f_{cd} \cdot A_{c.7}} = 1.022$ Relative normal force

$\lambda_{lim.7} := \frac{10.8}{\sqrt{n_7}} = 10.685$ Rough value of limit

Since $\lambda_7 > \lambda_{lim.7}$, the column must be designed with regard to the second order moment.

First order moment

The calculations below are based on Section B11.2 in Al-Emrani et al. (2011)

$e_i = 0.01\,m$ Unintended eccentricity (same as before)

$M_{0Ed.7} := N_{Ed.7} \cdot (e_0 + e_i) = 141.993 \cdot kN \cdot m$ ULS combination

$M_{0Eqp.7} := N_{Eqp.7} \cdot (e_0 + e_i) = 85.413 \cdot kN \cdot m$ Quasi-permanent SLS combination

Nominal bending stiffness

The calculations below are based on Section B11.4.2 in Al-Emrani et al. (2011)

$\rho_{reinf.7} := \frac{A'_{s7} + A_{s7m} + A_{s7}}{A_{c.7}} = 6.338 \times 10^{-3}$ Reinforcement ratio

$\rho_{reinf.7} \geq 0.002 = 1$ OK!

$k_1 = 1.414$ (same as before)

$k_{2.7} := \frac{N_{Ed.7}}{f_{cd} \cdot A_{c.7}} \cdot \frac{\lambda_7}{170} = 0.129$

$u_7 := \pi \cdot D_7 = 2.513\,m$ All sides of the column are subjected to drying

$h_{0.7} := \frac{2 \cdot A_{c.7}}{u_7} = 0.4\,m$ Nominal thickness

$$\varphi_{RH.7} := \left[1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{\frac{h_{0.7}}{mm}}} \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.7}\right] \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.2} = 1.449$$ Creep from relative humidity

$\beta_{fcm} = 2.43$

$\beta_{t0} = 0.48$ Assuming that the first load was applied after 28 days

$\varphi_{inf.7} := \varphi_{RH.7} \cdot \beta_{fcm} \cdot \beta_{t0} = 1.691$ Final creep

$$\varphi_{ef.7} := \varphi_{inf.7} \cdot \frac{M_{0Eqp.7}}{M_{0Ed.7}} = 1.017$$ Effective creep

$$I_{s.7} := A'_{s7} \cdot \left(\frac{D_7}{2} - d'_{s7}\right)^2 + A_{s7m} \cdot 0 + A_{s7} \cdot \left(d_{s7} - \frac{D_7}{2}\right)^2 = 1.94 \times 10^{-4}\,m^4$$ Second moment of inertia for reinforcement. Simplified.

$$EI_7 := \frac{k_1 \cdot k_{2.7}}{1 + \varphi_{ef.7}} \cdot E_{cd} \cdot I_{c.7} + E_s \cdot I_{s.7} = 91.931 \cdot MN \cdot m^2$$ Nominal bending stiffness

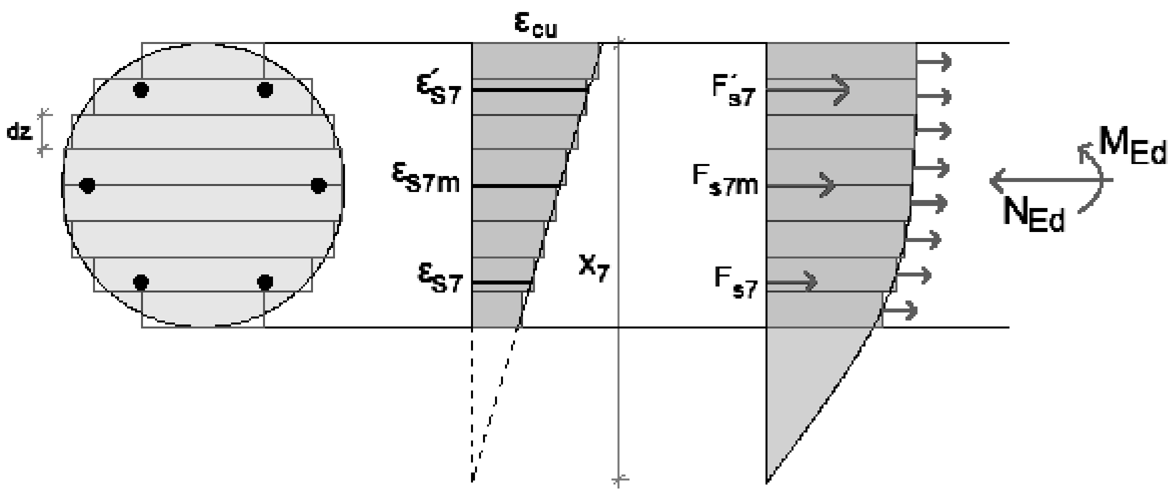
Second order moment

$$N_{B.7} := \frac{\pi^2 \cdot EI_7}{l_0^{\,2}} = 49.071 \cdot MN$$ Theoretical buckling force

$$M_{Ed.7} := \left(1 + \frac{\beta_{shape}}{\frac{N_{B.7}}{N_{Ed.7}} - 1}\right) \cdot M_{0Ed.7} = 196.962 \cdot kN \cdot m$$ Second order moment

Resistance of the section

The circular section is treated as rectangular strips according to the general approach described in Section B5.7 in Al-Emrani et al. (2010). Eight strips are used to represent the height of the section.



$dz := \frac{D_7}{8} = 100 \cdot mm$ Height of one strip

$b_{7i} = 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{7i} - \left(x_7 - \frac{D_7}{2}\right)\right]^2}$ Width of each strip where z_{7i} is the height from the neutral layer to the middle of the strip.

Horizontal equilibrium:

$$\sum_{i=1}^{8} \left(\sigma_{c7i} \cdot b_{7i} \cdot dz\right) + \sigma'_{s7} \cdot A'_{s7} + \sigma_{s7m} \cdot A_{s7m} + \sigma_{s7} \cdot A_{s7} = N_{Ed.7}$$

$\sigma_{c7i} = \begin{cases} f_{cd} & \text{if } \varepsilon_{c7i} \geq 0.0020 \\ f_{cd} \cdot \left[1 - \left(1 - \frac{\varepsilon_{c7i}}{0.0020}\right)^2\right] & \end{cases}$ Stress in one strip (depending on the behaviour of concrete under compression)

$\varepsilon_{c7i} = \frac{z_{7i}}{x_7} \cdot \varepsilon_{cu}$ Strain in one strip

$\sigma'_{s7} := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement yields

$\sigma_{s7m} = \varepsilon_{s7m} \cdot E_s$ Assuming that middle reinforcement doesn't yield

$\varepsilon_{s7m} = \varepsilon_{cu} \cdot \frac{x_7 - d_{s7m}}{x_7}$

$\sigma_{s7} = \varepsilon_{s7} \cdot E_s$ Assuming that bottom reinforcement doesn't yield

$\varepsilon_{s7} = \varepsilon_{cu} \cdot \frac{x_7 - d_{s7}}{x_7}$

Calculating height of compressive zone:

$x_7 := 0.939m$ Assuming a value for x_7

$$\sigma_{s7} := \varepsilon_{cu} \cdot \frac{x_7 - d_{s7}}{x_7} \cdot E_s = 176.496 \cdot MPa$$

$$\sigma_{s7m} := \varepsilon_{cu} \cdot \frac{x_7 - d_{s7m}}{x_7} \cdot E_s = 401.81 \cdot MPa$$

$$z_{71} := x_7 - \frac{dz}{2} = 0.889\,m \qquad b_{71} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{71} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.387\,m$$

$$z_{72} := x_7 - \left(dz + \frac{dz}{2}\right) = 0.789\,m \qquad b_{72} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{72} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.624\,m$$

$$z_{73} := x_7 - \left(2dz + \frac{dz}{2}\right) = 0.689\,m \qquad b_{73} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{73} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.742\,m$$

$$z_{74} := x_7 - \left(3dz + \frac{dz}{2}\right) = 0.589\,m \qquad b_{74} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{74} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.794\,m$$

$$z_{75} := x_7 - \left(4dz + \frac{dz}{2}\right) = 0.489\,m \qquad b_{75} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{75} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.794\,m$$

$$z_{76} := x_7 - \left(5dz + \frac{dz}{2}\right) = 0.389\,m \qquad b_{76} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{76} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.742\,m$$

$$z_{77} := x_7 - \left(6dz + \frac{dz}{2}\right) = 0.289\,m \qquad b_{77} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{77} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.624\,m$$

$$z_{78} := x_7 - \left(7dz + \frac{dz}{2}\right) = 0.189\,m \qquad b_{78} := 2\sqrt{\left(\frac{D_7}{2}\right)^2 - \left[z_{78} - \left(x_7 - \frac{D_7}{2}\right)\right]^2} = 0.387\,m$$

$$\varepsilon_{c71} := \frac{z_{71}}{x_7}\cdot\varepsilon_{cu} = 3.314\times10^{-3} \qquad \sigma_{c71} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c71} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c71}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 26.667\cdot\text{MPa}$$

$$\varepsilon_{c72} := \frac{z_{72}}{x_7}\cdot\varepsilon_{cu} = 2.941\times10^{-3} \qquad \sigma_{c72} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c72} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c72}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 26.667\cdot\text{MPa}$$

$$\varepsilon_{c73} := \frac{z_{73}}{x_7}\cdot\varepsilon_{cu} = 2.568\times10^{-3} \qquad \sigma_{c73} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c73} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c73}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 26.667\cdot\text{MPa}$$

$$\varepsilon_{c74} := \frac{z_{74}}{x_7}\cdot\varepsilon_{cu} = 2.195\times10^{-3} \qquad \sigma_{c74} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c74} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c74}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 26.667\cdot\text{MPa}$$

$$\varepsilon_{c75} := \frac{z_{75}}{x_7}\cdot\varepsilon_{cu} = 1.823\times10^{-3} \qquad \sigma_{c75} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c75} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c75}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 26.457\cdot\text{MPa}$$

$$\varepsilon_{c76} := \frac{z_{76}}{x_7}\cdot\varepsilon_{cu} = 1.45\times10^{-3} \qquad \sigma_{c76} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c76} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c76}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 24.65\cdot\text{MPa}$$

$$\varepsilon_{c77} := \frac{z_{77}}{x_7}\cdot\varepsilon_{cu} = 1.077\times10^{-3} \qquad \sigma_{c77} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c77} \ge 0.0020 \\ \left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c77}}{0.0020}\right)^2\right]\right] & \text{otherwise}\end{cases} = 20.99\cdot\text{MPa}$$

$$\varepsilon_{c78} := \frac{z_{78}}{x_7}\cdot\varepsilon_{cu} = 7.045\times10^{-4} \qquad \sigma_{c78} := \begin{cases} f_{cd} & \text{if } \varepsilon_{c78} \ge 0.0020 \\ 0 & \text{if } \varepsilon_{c78} \le 0 \\ \left[\left[f_{cd}\cdot\left[1-\left(1-\frac{\varepsilon_{c78}}{0.0020}\right)^2\right]\right]\right] & \text{otherwise}\end{cases} = 15.477\cdot\text{MPa}$$

Check of horizontal equilibrium:

$$N_{Rd.7} := \left(\sigma_{c71}\cdot b_{71} + \sigma_{c72}\cdot b_{72} + \sigma_{c73}\cdot b_{73} + \sigma_{c74}\cdot b_{74} + \sigma_{c75}\cdot b_{75} + \sigma_{c76}\cdot b_{76} + \sigma_{c77}\cdot b_{77} + \sigma_{c78}\cdot b_{78}\right)\cdot dz\ ..$$
$$+\ \sigma'_{s7}\cdot A'_{s7} + \sigma_{s7m}\cdot A_{s7m} + \sigma_{s7}\cdot A_{s7}$$

$N_{Rd.7} = 13.706\cdot\text{MN}$ (These should be the same to get horizontal equilibrium)

$N_{Ed.7} = 13.695\cdot\text{MN}$

Check of assumptions:

$\varepsilon_{sy} = 2.174 \times 10^{-3}$ Steel strain at yielding

$\varepsilon'_{s7} := \varepsilon_{cu} \cdot \dfrac{x_7 - d'_{s7}}{x_7} = 3.136 \times 10^{-3}$ $\varepsilon'_{s7} \geq \varepsilon_{sy} = 1$ Top reinf. yielding

$\varepsilon_{s7m} := \varepsilon_{cu} \cdot \dfrac{x_7 - d_{s7m}}{x_7} = 2.009 \times 10^{-3}$ $\varepsilon_{s7m} \geq \varepsilon_{sy} = 0$ Mid reinf. NOT yielding

$\varepsilon_{s7} := \varepsilon_{cu} \cdot \dfrac{x_7 - d_{s7}}{x_7} = 8.825 \times 10^{-4}$ $\varepsilon_{s7} \geq \varepsilon_{sy} = 0$ Bottom reinf. NOT yielding

Moment equilibrium around neutral layer:

$$M_{Rd.7} := dz \cdot \begin{pmatrix} \sigma_{c71} \cdot b_{71} \cdot z_{71} + \sigma_{c72} \cdot b_{72} \cdot z_{72} + \sigma_{c73} \cdot b_{73} \cdot z_{73} + \sigma_{c74} \cdot b_{74} \cdot z_{74} + \sigma_{c75} \cdot b_{75} \cdot z_{75} \cdots \\ + \sigma_{c76} \cdot b_{76} \cdot z_{76} + \sigma_{c77} \cdot b_{77} \cdot z_{77} + \sigma_{c78} \cdot b_{78} \cdot z_{78} \end{pmatrix} \cdots$$
$$+ \sigma'_{s7} \cdot A'_{s7} \cdot (x_7 - d'_{s7}) + \sigma_{s7m} \cdot A_{s7m} \cdot (x_7 - d_{s7m}) + \sigma_{s7} \cdot A_{s7} \cdot (x_7 - d_{s7}) - N_{Ed.7} \cdot \left(x_7 - \frac{D_7}{2}\right)$$

$M_{Rd.7} = 352.622 \cdot kN \cdot m$

Check of resistance

$e_{min.7} := \max\left(\dfrac{D_7}{30}, 20mm\right) = 26.667 \cdot mm$ Minimum eccentricity for normal force

$\dfrac{M_{Ed.7}}{M_{Rd.7}} = 0.559$ $\dfrac{N_{Ed.7} \cdot e_{min.7}}{M_{Rd.7}} = 1.036$

Even if the utilisation for this column is slightly above 1, it is chosen to use this design for the circular column that is to be strengthened.

Part 7 - Capacity of column with circular section

Part 8 - Strengthening with CFRP wrapping - Circular section

In this part, the same procedure as in Part 6 is carried out, but this time on the column with circular section described in Part 7. CFRP sheets are wrapped around the column so that the fibres are placed in the circumferential direction. It is assumed that the whole column is strengthened. The calculations are based on Täljsten et. al. (2011), Chapter 6.

$D_8 := D_7 = 800 \cdot mm$ — Diameter of the circular column

Increased compressive strength of the concrete

$\kappa_{a8} := 1.0$

$\kappa_{b8} := 1.0$

The geometrical efficiency factors are set to 1.0 for a circular section since the whole section will be affected by the triaxial stress state.

$\varepsilon_{fu} = 1.55 \cdot \%$ — Ultimate strain in CFRP

$\kappa_e = 0.55$ — Efficiency factor concerning premature failure in CFRP due to the triaxial stress situation

$\varepsilon_{fe} = 0.853 \cdot \%$ — Effective ultimate strain in CFRP

$E_f = 240 \cdot GPa$ — Modulus of elasticity for CFRP

$n_{f8} := 5$ — Number of layers of sheets (Chosen)

$t_f = 0.117 \cdot mm$ — Thickness of one sheet

$$f_{l.8} := \frac{2 \cdot E_f \cdot n_{f8} \cdot t_f \cdot \varepsilon_{fe}}{D_7} = 2.992 \cdot MPa$$

Maximum wrapping pressure

$$\frac{f_{l.8}}{f_{ck}} = 0.075$$

Should at least be 0.08

$$\varepsilon_{cu.c8} := \varepsilon_{c2} \cdot \left[1.50 + 12 \cdot \kappa_{b8} \cdot \frac{f_{l.8}}{f_{ck}} \cdot \left(\frac{\varepsilon_{fe}}{\varepsilon_{c2}} \right)^{0.45} \right] = 6.447 \times 10^{-3}$$

Check of maximum strain in wrapped concrete (should be below 10‰)

$\alpha_{f.c} = 0.95$ — Reduction factor to increase the safety of the model

$f_{cd.c8} := f_{cd} + \alpha_{f.c} \cdot 3.3 \cdot \kappa_{a8} \cdot f_{l.8} = 36.047 \cdot MPa$ — Increased compressive strength of concrete due to triaxial stress state

Loads

$factor_8 := 1.28$ — Increasing the load from Part 1

$N_{Ed.8} := factor_8 \cdot N_{Ed} = 17.53 \cdot MN$ — New vertical load on top of the column

First order moment

$e_0 = 0$

$e_i = 0.01\,m$ — The eccentricity should be the same as before

$M_{0.Ed.8} := N_{Ed.8} \cdot (e_0 + e_i) = 181.751 \cdot kN \cdot m$

Nominal bending stiffness

Since the column hasn't been strengthened with regard to bending resistance, the nominal bending stiffness should be the same as in Part 7.

$$EI_8 := EI_7 = 91.931\,m\cdot MN\cdot m$$

Second order moment

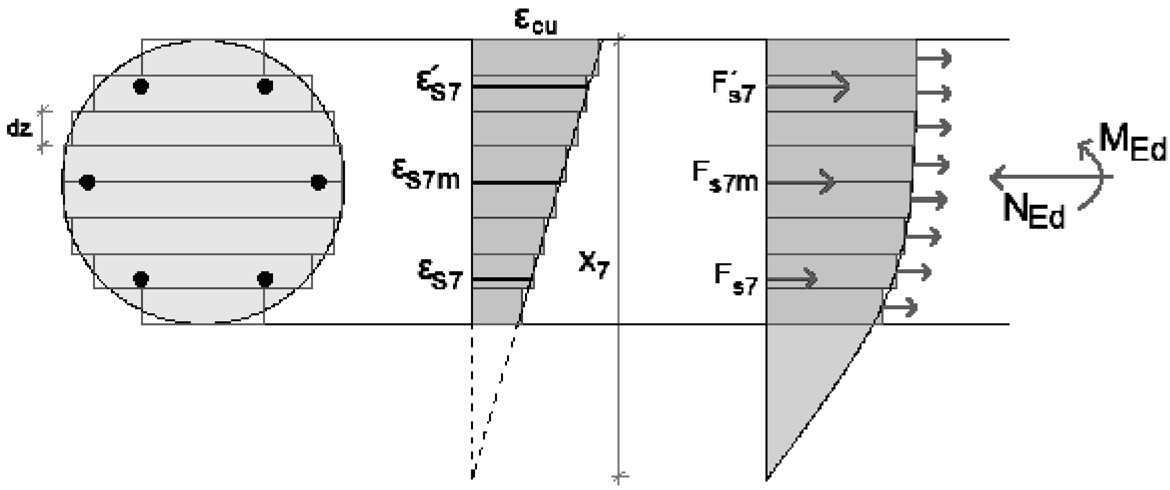
$$N_{B.8} := \frac{\pi^2 \cdot EI_8}{l_0^{\,2}} = 49.071\cdot MN$$

Buckling load

$$M_{Ed.8} := \left(1 + \frac{\beta_{shape}}{\dfrac{N_{B.8}}{N_{Ed.8}} - 1}\right)\cdot M_{0.Ed.8} = 282.762\cdot kN\cdot m$$

Resistance of the section

The circular section is treated as rectangular strips according to the general approach described in Section B5.7 in Al-Emrani et al. (2010). Eight strips are used to represent the height of the section.



$$dz = 100\cdot mm$$

Height of one strip

$$b_{8i} = 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{8i} - \left(x_8 - \frac{D_8}{2}\right)\right]^2}$$

Width of each strip where z_{7i} is the height from the neutral layer to the middle of the strip.

Horizontal equilibrium:

$$\sum_{i=1}^{8}\left(\sigma_{c8i}\cdot b_{8i}\cdot dz\right) + \sigma'_{s8}\cdot A'_{s7} + \sigma_{s8m}\cdot A_{s7m} + \sigma_{s8}\cdot A_{s7} = N_{Ed.8}$$

$$\sigma_{c8i} = \begin{vmatrix} f_{cd.c8} \quad \text{if } \varepsilon_{c8i} \geq 0.0020 \\ f_{cd.c8}\cdot\left[1 - \left(1 - \dfrac{\varepsilon_{c7i}}{0.0020}\right)^2\right] \end{vmatrix}$$

Stress in one strip (depending on the behaviour of concrete under compression)

$\varepsilon_{c8i} = \frac{z_{8i}}{x_8} \cdot \varepsilon_{cu}$ Strain in one strip

$\sigma'_{s8} := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement yields

$\sigma_{s8m} = \varepsilon_{s8m} \cdot E_s$ Assuming that middle reinforcement doesn't yield

$\varepsilon_{s8m} = \varepsilon_{cu} \cdot \frac{x_8 - d_{s7m}}{x_8}$

$\sigma_{s8} = \varepsilon_{s8} \cdot E_s$ Assuming that bottom reinforcement doesn't yield

$\varepsilon_{s8} = \varepsilon_{cu} \cdot \frac{x_8 - d_{s7}}{x_8}$

Calculating height of compressive zone:

$x_8 := 0.876m$ Assuming a value for x_7

$\sigma_{s8m} := \varepsilon_{cu} \cdot \frac{x_8 - d_{s7m}}{x_8} \cdot E_s = 380.365 \cdot MPa$

$\sigma_{s8} := \varepsilon_{cu} \cdot \frac{x_8 - d_{s7}}{x_8} \cdot E_s = 138.847 \cdot MPa$

$z_{81} := x_8 - \frac{dz}{2} = 0.826\,m$ $b_{81} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{81} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.387\,m$

$z_{82} := x_8 - \left(dz + \frac{dz}{2}\right) = 0.726\,m$ $b_{82} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{82} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.624\,m$

$z_{83} := x_8 - \left(2dz + \frac{dz}{2}\right) = 0.626\,m$ $b_{83} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{83} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.742\,m$

$z_{84} := x_8 - \left(3dz + \frac{dz}{2}\right) = 0.526\,m$ $b_{84} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{84} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.794\,m$

$z_{85} := x_8 - \left(4dz + \frac{dz}{2}\right) = 0.426\,m$ $b_{85} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{85} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.794\,m$

$z_{86} := x_8 - \left(5dz + \frac{dz}{2}\right) = 0.326\,m$ $b_{86} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{86} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.742\,m$

$z_{87} := x_8 - \left(6dz + \frac{dz}{2}\right) = 0.226\,m$ $b_{87} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{87} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.624\,m$

$z_{88} := x_8 - \left(7dz + \frac{dz}{2}\right) = 0.126\,m$ $b_{88} := 2\sqrt{\left(\frac{D_8}{2}\right)^2 - \left[z_{88} - \left(x_8 - \frac{D_8}{2}\right)\right]^2} = 0.387\,m$

$$\varepsilon_{c81} := \frac{z_{81}}{x_8}\cdot\varepsilon_{cu} = 3.3\times 10^{-3} \qquad \sigma_{c81} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c81} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c81}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 36.047\cdot\text{MPa}$$

$$\varepsilon_{c82} := \frac{z_{82}}{x_8}\cdot\varepsilon_{cu} = 2.901\times 10^{-3} \qquad \sigma_{c82} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c82} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c82}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 36.047\cdot\text{MPa}$$

$$\varepsilon_{c83} := \frac{z_{83}}{x_8}\cdot\varepsilon_{cu} = 2.501\times 10^{-3} \qquad \sigma_{c83} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c83} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c83}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 36.047\cdot\text{MPa}$$

$$\varepsilon_{c84} := \frac{z_{84}}{x_8}\cdot\varepsilon_{cu} = 2.102\times 10^{-3} \qquad \sigma_{c84} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c84} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c84}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 36.047\cdot\text{MPa}$$

$$\varepsilon_{c85} := \frac{z_{85}}{x_8}\cdot\varepsilon_{cu} = 1.702\times 10^{-3} \qquad \sigma_{c85} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c85} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c85}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 35.247\cdot\text{MPa}$$

$$\varepsilon_{c86} := \frac{z_{86}}{x_8}\cdot\varepsilon_{cu} = 1.303\times 10^{-3} \qquad \sigma_{c86} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c86} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c86}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 31.663\cdot\text{MPa}$$

$$\varepsilon_{c87} := \frac{z_{87}}{x_8}\cdot\varepsilon_{cu} = 9.03\times 10^{-4} \qquad \sigma_{c87} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c87} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c87}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 25.202\cdot\text{MPa}$$

$$\varepsilon_{c88} := \frac{z_{88}}{x_8}\cdot\varepsilon_{cu} = 5.034\times 10^{-4} \qquad \sigma_{c88} := \begin{cases} f_{cd.c8} & \text{if } \varepsilon_{c88} \ge 0.0020 \\ \left[f_{cd.c8}\cdot\left[1-\left(1-\frac{\varepsilon_{c88}}{0.0020}\right)^2\right]\right] & \text{otherwise} \end{cases} = 15.863\cdot\text{MPa}$$

Check of horizontal equilibrium:

$$N_{Rd.8} := \left(\sigma_{c81}\cdot b_{81} + \sigma_{c82}\cdot b_{82} + \sigma_{c83}\cdot b_{83} + \sigma_{c84}\cdot b_{84} + \sigma_{c85}\cdot b_{85} + \sigma_{c86}\cdot b_{86} + \sigma_{c87}\cdot b_{87} + \sigma_{c88}\cdot b_{88}\right)\cdot dz \;.. \\ + \sigma'_{s8}\cdot A'_{s7} + \sigma_{s8m}\cdot A_{s7m} + \sigma_{s8}\cdot A_{s7}$$

$N_{Rd.8} = 17.529\cdot\text{MN}$ (These should be the same to get horizontal equilibrium)

$N_{Ed.8} = 17.53\cdot\text{MN}$

Check of assumptions:

$\varepsilon_{sy} = 2.174\times 10^{-3}$ Steel strain at yielding

$\varepsilon'_{s8} := \varepsilon_{cu}\cdot\dfrac{x_8 - d'_{s7}}{x_8} = 3.109\times 10^{-3}$ $\varepsilon'_{s8} \geq \varepsilon_{sy} = 1$ Top reinf. yielding

$\varepsilon_{s8m} := \varepsilon_{cu}\cdot\dfrac{x_8 - d_{s7m}}{x_8} = 1.902\times 10^{-3}$ $\varepsilon_{s8m} \geq \varepsilon_{sy} = 0$ Mid reinf. NOT yielding

$\varepsilon_{s8} := \varepsilon_{cu}\cdot\dfrac{x_8 - d_{s7}}{x_8} = 6.942\times 10^{-4}$ $\varepsilon_{s8} \geq \varepsilon_{sy} = 0$ Bottom reinf. NOT yielding

Moment equilibrium around neutral layer:

$$M_{Rd.8} := dz\cdot\begin{pmatrix}\sigma_{c81}\cdot b_{81}\cdot z_{81} + \sigma_{c82}\cdot b_{82}\cdot z_{82} + \sigma_{c83}\cdot b_{83}\cdot z_{83} + \sigma_{c84}\cdot b_{84}\cdot z_{84} + \sigma_{c85}\cdot b_{85}\cdot z_{85} \cdots \\ + \sigma_{c86}\cdot b_{86}\cdot z_{86} + \sigma_{c87}\cdot b_{87}\cdot z_{87} + \sigma_{c88}\cdot b_{88}\cdot z_{88}\end{pmatrix} \cdots$$
$$+ \sigma'_{s8}\cdot A'_{s7}\cdot\left(x_8 - d'_{s7}\right) + \sigma_{s8m}\cdot A_{s7m}\cdot\left(x_8 - d_{s7m}\right) + \sigma_{s8}\cdot A_{s7}\cdot\left(x_8 - d_{s7}\right) - N_{Ed.8}\cdot\left(x_8 - \frac{D_8}{2}\right)$$

$M_{Rd.8} = 589.54\cdot kN\cdot m$

Check of resistance

$e_{min.8} := \max\left(\dfrac{D_8}{30}, 20mm\right) = 26.667\cdot mm$ Minimum eccentricity for normal force

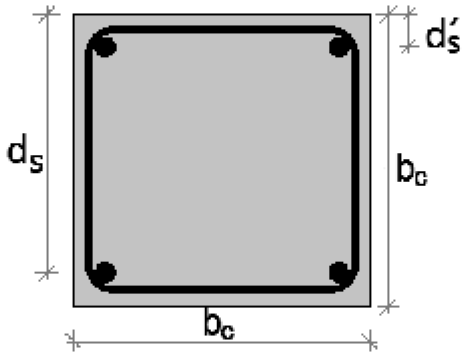
$\dfrac{M_{Ed.8}}{M_{Rd.8}} = 0.48$ $\dfrac{N_{Ed.8}\cdot e_{min.8}}{M_{Rd.8}} = 0.793$

A higher utilisation factor could not be reached since 4 layers of CFRP (instead of 5) resulted in over 100%.

Part 8 - Strengthening with CFRP wrapping - Circular section

*Part 9 - Capacity of existing column with rectangular section 0.25*0.25m²*

Some of the investigated strengthening methods are unsuitable for the column studied in Part 1. Therefore, a more slender column is also investigated.



Input data

$f_{ck} := 40MPa$	Characteristic compressive capacity
$f_{cd} := \frac{f_{ck}}{1.5} = 26.667 \cdot MPa$	Design value
$E_{cm} := 35GPa$	Mean value of modulus of elasticity
$b_c := 250mm$	Width of the column (square-shaped)
$h_c := 4.3m$	Height of the column
$A_c := b_c^{\,2} = 0.063\,m^2$	Section area
$cover := 30mm$	Thickness of concrete cover

Reinforcement:

$E_s := 200GPa$	Modulus of elasticity
$f_{yk} := 500MPa$	Characteristic yield strength
$f_{yd} := \frac{f_{yk}}{1.15} = 434.783 \cdot MPa$	Design value
$\phi_{si} := 22mm$	Diameter of bending reinforcement
$A_{si} := \frac{\pi \cdot \phi_{si}^{\,2}}{4} = 380.133 \cdot mm^2$	Area of one reinforcement bar
$n_{si} := 2$	Number of bars at the bottom of the cross-section
$n'_{si} := 2$	Number of bars at the top of the cross-section
$A_s := n_{si} \cdot A_{si} = 760.265 \cdot mm^2$	Total area of the bottom reinforcement
$A'_s := n'_{si} \cdot A_{si} = 760.265 \cdot mm^2$	Total area of the top reinforcement
$\phi_{st.i} := 8mm$	Diametre of reinforcement in stirrup

Distances:

$d_s := b_c - cover - \phi_{st.i} - \frac{\phi_{si}}{2} = 0.201\,m$ Distance from compressive surface to bottom bars.

$d'_s := cover + \phi_{st.i} + \frac{\phi_{si}}{2} = 0.049\,m$ Distance from compressive surface to top bars.

Loads

The loads on the columns are assumed and then iterated until the resistance is slightly higher than the load effect.

$G := 0.75MN$ Permanent load on column, expressed as a point force

$Q := 0.3MN$ Variable load on column, expressed as a point force

$N_{Ed} := 1.35 \cdot G + 1.5 \cdot Q = 1.462 \cdot MN$ ULS combination

$\psi_2 := 0.6$ Assuming imposed load category C
(space where people may congregate)

$N_{Eqp} := G + \psi_2 \cdot Q = 0.93 \cdot MN$ Quasi-permanent SLS combination

Evaluation of slenderness

The calculations below are based on Section B11.3.2 in Al-Emrani et al. (2011)

The column is regarded as an isolated structural member with pinned connections in each end.

$l_0 := h_c = 4.3\,m$ Buckling length (assumed pinned-pinned connections)

$I_c := \frac{b_c \cdot b_c^{\,3}}{12} = 3.255 \times 10^{-4}\,m^4$ Second moment of inertia of the gross concrete section

$i := \sqrt{\frac{I_c}{A_c}} = 0.072\,m$ Radius of gyration

$\lambda := \frac{l_0}{i} = 59.583$ Slenderness

Rough estimation of the limit value of the slenderness:

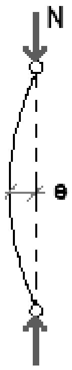
$n := \frac{N_{Ed}}{f_{cd} \cdot A_c} = 0.877$ Relative normal force

$\lambda_{lim} := \frac{10.8}{\sqrt{n}} = 11.529$ Rough value of limit

Since $\lambda > \lambda_{lim}$, the column must be designed with regard to the second order moment.

First order moment

The calculations below are based on Section B11.2 in Al-Emrani et al. (2011)



$e_0 := 0$ Intended excentricity of load (assumed to be applied at the center of the column)

$\theta_0 := 0.005$ Base value of normal execution deviations

$\alpha_h := \frac{2}{\sqrt{\frac{h_c}{m}}} = 0.964$ Reduction factor

$m_\alpha := 1$ Only calculating with contribution from one column (and not the whole structure)

$\alpha_m := \sqrt{0.5 \cdot \left(1 + \frac{1}{m_\alpha}\right)} = 1$ Reduction factor

$\theta_i := \theta_0 \cdot \alpha_h \cdot \alpha_m = 4.822 \times 10^{-3}$ Normal execution deviations

$e_i := \theta_i \cdot \frac{l_0}{2} = 0.01\,m$ Unintended excentricity

First order moments:

$M_{0Ed} := N_{Ed} \cdot (e_0 + e_i) = 15.164 \cdot kN \cdot m$ ULS compination

$M_{0Eqp} := N_{Eqp} \cdot (e_0 + e_i) = 9.642 \cdot kN \cdot m$ Quasi-permanent SLS combination

Nominal bending stiffness

The calculations below are based on Section B11.4.2 in Al-Emrani et al. (2011)

$\rho_{reinf} := \frac{A_s + A'_s}{A_c} = 0.024$ Reinforcement ratio

$\rho_{reinf} \geq 0.002 = 1$ OK!

$\gamma_{cE} := 1.2$ National parameter

$E_{cd} := \frac{E_{cm}}{\gamma_{cE}} = 29.167 \cdot GPa$ Design value of modulus of elasticity for the concrete

$$k_1 := \sqrt{\frac{\frac{f_{ck}}{MPa}}{20}} = 1.414$$

$$k_2 := \frac{N_{Ed}}{f_{cd} \cdot A_c} \cdot \frac{\lambda}{170} = 0.308$$

$RH := 50\%$ Indoor climate

$u := 4 \cdot b_c = 1\,m$ All sides of the column are subjected to drying

$$h_0 := \frac{2 \cdot A_c}{u} = 0.125\,m$$ Nominal thickness

$f_{cm} := f_{ck} + 8MPa = 48 \cdot MPa$ Mean value of compressive strength of concrete

$$\varphi_{RH} := \left[1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{\frac{h_0}{mm}}} \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.7}\right] \cdot \left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.2} = 1.691$$ Creep from relative humidity

$\beta_{fcm} := 2.43$

$\beta_{t0} := 0.48$ Assuming that the first load was applied after 28 days

$\varphi_{inf} := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = 1.973$ Final creep

$$\varphi_{ef} := \varphi_{inf} \cdot \frac{M_{0Eqp}}{M_{0Ed}} = 1.254$$ Effective creep

$$I_s := 4 \cdot A_{si} \cdot \left(\frac{b_c}{2} - cover - \phi_{st.i} - \frac{\phi_{si}}{2}\right)^2$$ Second moment of inertia for reinforcement. Simplified.

$$EI := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s = 3.588 \cdot MN \cdot m^2$$ Nominal bending stiffness

Second order moment

$$N_B := \frac{\pi^2 \cdot EI}{l_0^2} = 1.915 \cdot MN$$ Theoretical buckling force

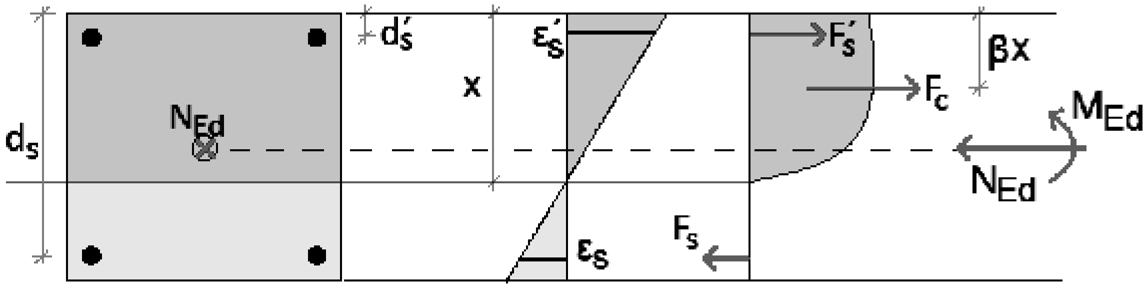
$\beta_{shape} := 1.0$ Due to sinus-shaped bending moment

$$M_{Ed} := \left(1 + \frac{\beta_{shape}}{\frac{N_B}{N_{Ed}} - 1}\right) \cdot M_{0Ed} = 64.139 \cdot kN \cdot m$$ Second order moment

Resistance of the section

The calculations below are based on Section B5.6 in Al-Emrani et al. (2010)

Assuming that the lower surface is in tension at ULS (unlike in the case with the bigger cross-section)



$\alpha := 0.81$
$\beta := 0.416$

$A_s = 7.603 \times 10^{-4}\,m^2$ Bottom reinforcement

$A'_s = 7.603 \times 10^{-4}\,m^2$ Top reinforcement

$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$ Ultimate strain for the concrete

Horizontal equilibrium:

$\alpha \cdot f_{cd} \cdot b_c \cdot x + \sigma'_s \cdot A'_s + \sigma_s \cdot A_s = N_{Ed}$

$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement yields

$\sigma_s = \varepsilon_s \cdot E_s$ Assuming that bottom reinforcement doesn't yield

$\varepsilon_s = \varepsilon_{cu} \cdot \frac{d_s - x}{x}$

Calculating height of compressive zone:

$x := 0.5m$ Assuming an initial value for x
Given

$\alpha \cdot f_{cd} \cdot b_c \cdot x + \sigma'_s \cdot A'_s + \varepsilon_{cu} \cdot \frac{x - d_s}{x} \cdot E_s \cdot A_s = N_{Ed}$

$x := Find(x) = 0.207\,m$ Solving x from horizontal equilibrium

Check of assumptions:

$\varepsilon_{cc.min} := \varepsilon_{cu} \cdot \frac{b_c - x}{x} = 7.304 \times 10^{-4}$ Concrete strain at "bottom side" (tension)

$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.174 \times 10^{-3}$ Steel strain at yielding

$\varepsilon'_s := \varepsilon_{cu} \cdot \frac{x - d'_s}{x} = 2.671 \times 10^{-3}$ $\quad \varepsilon'_s \geq \varepsilon_{sy} = 1$ Upper reinf. yielding

$\varepsilon_s := \varepsilon_{cu} \cdot \frac{x - d_s}{x} = 9.88 \times 10^{-5}$ $\quad \varepsilon_s \geq \varepsilon_{sy} = 0$ Lower reinf. NOT yielding

Moment equilibrium around tensile reinforcement:

$$M_{Rd} := \alpha \cdot f_{cd} \cdot b_c \cdot x \cdot (d_s - \beta \cdot x) + \sigma'_s \cdot A'_s \cdot (d_s - d'_s) - N_{Ed} \cdot \left(d_s - \frac{b_c}{2}\right) = 67.49 \cdot kN \cdot m$$

Check of resistance

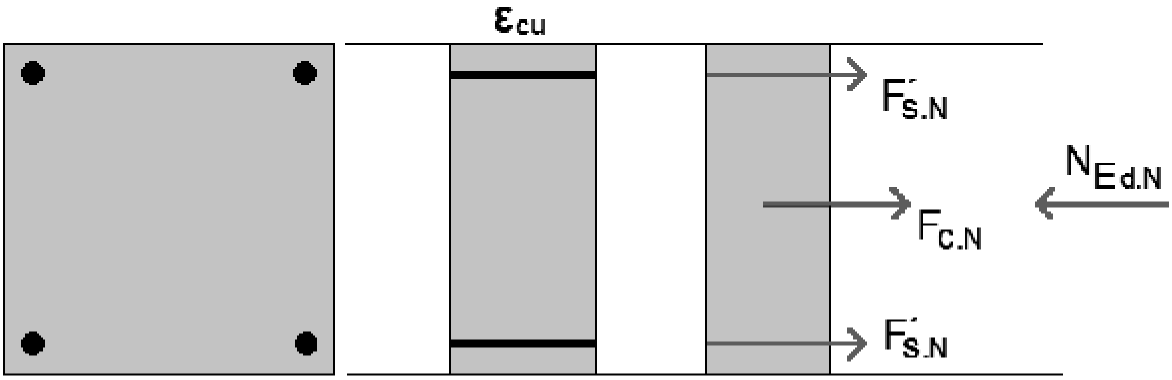
$$e_{min} := \max\left(\frac{b_c}{30}, 20mm\right) = 20 \cdot mm$$ Minimum excentricity for normal force

$$\frac{M_{Ed}}{M_{Rd}} = 0.95 \qquad \frac{N_{Ed} \cdot e_{min}}{M_{Rd}} = 0.433$$

N-M relationship for the column

To be able to compare different columns, the relationship between normal force and moment is put into a N-M interaction diagram. To create the diagram, the capacity of the column must be calculated for the cases when it only is subjected to normal force and moment respectively.

Only normal force:



$$F'_{s.N} := A'_s \cdot f_{yd} = 330.55 \cdot kN$$ Force from reinforcement (yielding since $\varepsilon_{cu} > \varepsilon_{sy}$)

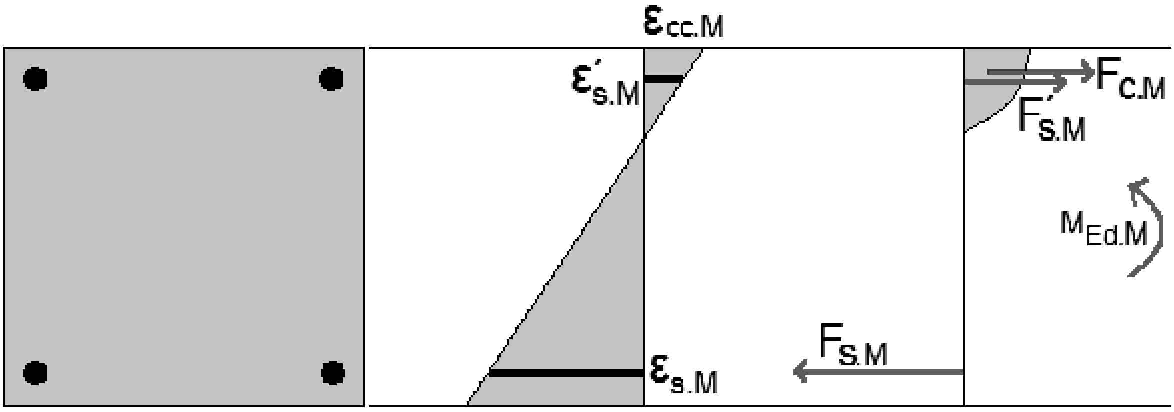
$$F_{s.N} := F'_{s.N} = 330.55 \cdot kN$$

$$F_{c.N} := f_{cd} \cdot \left[b_c^{\ 2} - (A'_s + A_s)\right] = 1.626 \cdot MN$$ Force from concrete

$$N_{Rd.N} := F_{c.N} + F'_{s.N} + F_{s.N} = 2.287 \cdot MN$$ Resistance in the case of uniform compression

$$\frac{N_{Ed}}{N_{Rd.N}} = 0.639$$ Relationship between maximum normal force and maximum normal force if no moment is present.

Only moment:
Since the amount of reinforcement is the same in the top and bottom, the top reinforcement doesn't yield when the maximum moment is reached. To be able to fulfil the horizontal equilibrium, the force from the top reinforcement must be smaller than the one from the bottom reinforcement (since it should be added together with the force from the compressed concrete).



$F'_{s.M} = E_s \cdot \varepsilon'_{s.M} \cdot A'_s$ Force from top reinforcement (not yielding)

$$\varepsilon'_{s.M} = \varepsilon_{cc.M} \cdot \frac{x_M - d'_s}{x_M}$$

$F_{s.M} := f_{yd} \cdot A_s = 3.306 \times 10^5\,N$ Force from bottom reinforcement (yielding)

$F_{c.M} = \alpha_M \cdot f_{cd} \cdot b_c \cdot x_M$ Force from compressed concrete

$\varepsilon_{cc.M} := 0.00308$ Assumed strain at top surface (iterated to reach as high moment capacity as possible)

$$\alpha_M := 0.778 + (0.792 - 0.778) \cdot \frac{\left(\varepsilon_{cc.M} \cdot 10^3 - 3.0\right)}{(0.8 - 0.6)} = 0.784$$

$$\beta_M := 0.405 + (0.410 - 0.405) \cdot \frac{\left(\varepsilon_{cc.M} \cdot 10^3 - 3.0\right)}{(0.8 - 0.6)} = 0.407$$

$x_M := 0.5m$

Given

$$E_s \varepsilon_{cc.M} \cdot \frac{x_M - d'_s}{x_M} \cdot A'_s + \alpha_M \cdot f_{cd} \cdot b_c \cdot x_M - F_{s.M} = 0$$ Horizontal equilibrium

$x_M := \mathrm{Find}\left(x_M\right) = 0.054\,m$

$$M_{Rd.M} := E_s \varepsilon_{cc.M} \cdot \frac{x_M - d'_s}{x_M} \cdot A'_s \cdot \left(d_s - d'_s\right) + \alpha_M \cdot f_{cd} \cdot b_c \cdot x_M \cdot \left(d_s - \beta_M \cdot x_M\right) = 57.876 \cdot kN \cdot m$$

$$\varepsilon'_{s.M} := \varepsilon_{cc.M} \cdot \frac{x_M - d'_s}{x_M} = 3.053 \times 10^{-4}$$ Check of strain in top reinforcement

$$\varepsilon_{s.M} := \varepsilon_{cc.M} \cdot \frac{d_s - x_M}{x_M} = 8.302 \times 10^{-3}$$ Check of strain in bottom reinforcement

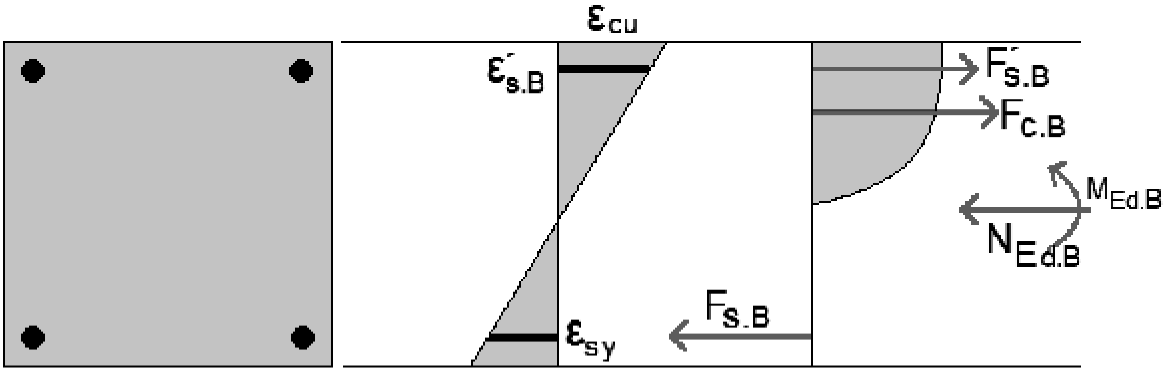
$$\frac{M_{Ed}}{M_{Rd.M}} = 1.108$$

Ratio between maximum moment and maximum moment if no normal force is present.

$$\frac{N_{Ed} \cdot e_{min}}{M_{Rd.M}} = 0.505$$

Balanced cross-section:

This time, it is assumed that the tensile reinforcement reaches yielding at the same time as the top surface reaches ε_{cu}



$$F_{s.B} := f_{yd} \cdot A_s = 330.55 \cdot kN$$

Force from bottom reinforcement (yielding)

$$x_B := \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d_s = 0.124 m$$

$$\varepsilon'_{s.B} := \varepsilon_{cu} \cdot \frac{x_B - d'_s}{x_B} = 2.117 \times 10^{-3}$$

$$F'_{s.B} := \left| \begin{array}{l} (f_{yd} \cdot A'_s) \text{ if } \varepsilon'_{s.B} \geq \varepsilon_{sy} \\ (E_s \cdot \varepsilon'_{s.B} \cdot A'_s) \text{ otherwise} \end{array} \right. = 321.867 \cdot kN$$

Force from bottom reinforcement

$$F_{c.B} := \alpha \cdot f_{cd} \cdot b_c \cdot x_B = 0.67 \cdot MN$$

Force from compressed concrete

$$N_{Rd.B} := F'_{s.B} + F_{c.B} - F_{s.B} = 0.661 \cdot MN$$

Normal force from horizontal equilibrium

$$M_{Rd.B} := F'_{s.B} \cdot (d_s - d'_s) + F_{c.B} \cdot (d_s - \beta \cdot x_B) - N_{Rd.B} \cdot \left(d_s - \frac{b_c}{2}\right) = 98.742 \cdot kN \cdot m$$

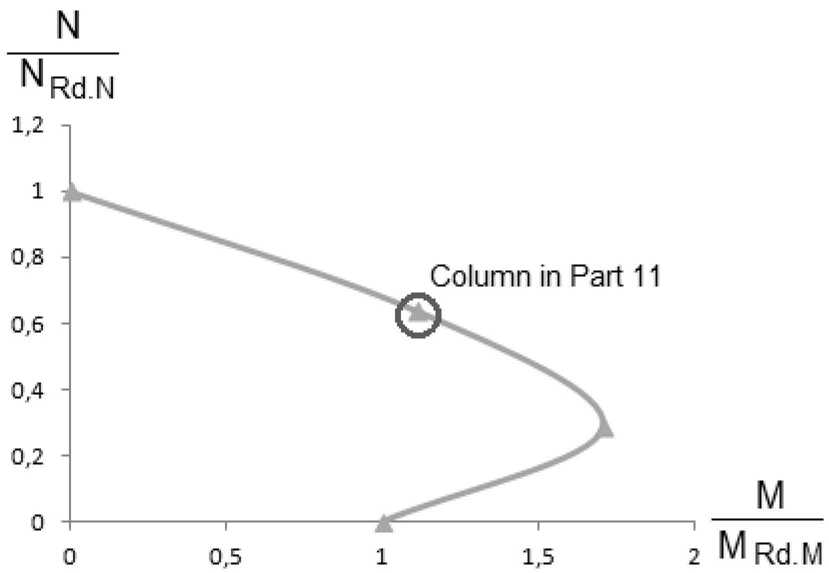
Maximum moment

$$\frac{N_{Rd.B}}{N_{Rd.N}} = 0.289$$

Ratio between normal forced in balanced situation and when no moment is present.

$$\frac{M_{Rd.B}}{M_{Rd.M}} = 1.706$$

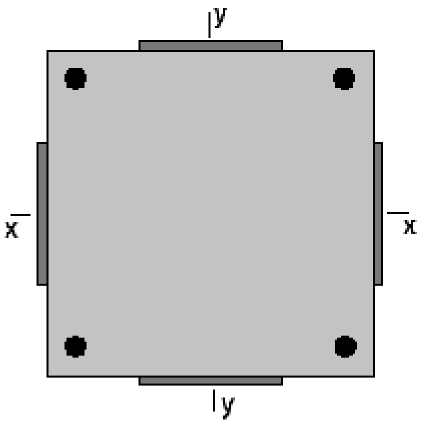
Ratio between moment in balanced situation and when no normal force is present.



Part 9 - Capacity of existing column with rectangular section 0.25*0.25m2

Part 10 - Strengthening with vertically mounted steel plates

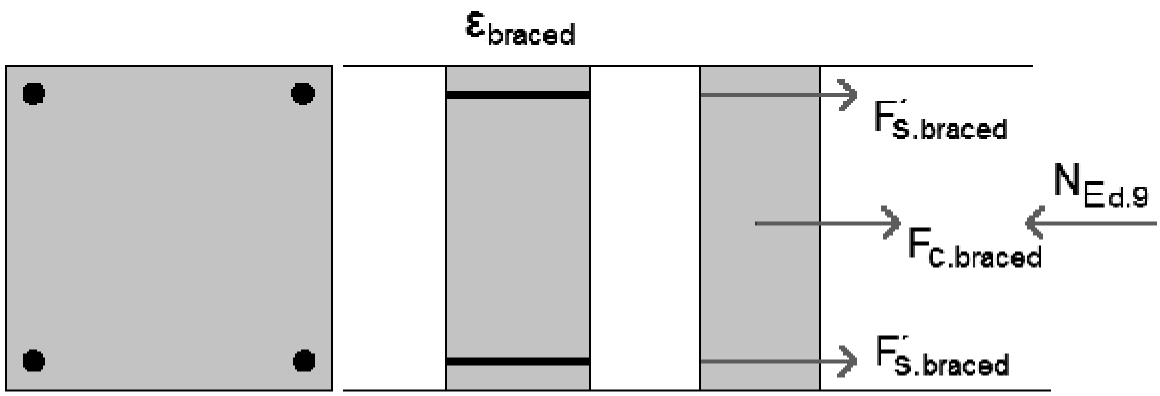
In this case, the rectangular section is strengthened with vertical steel plates that are assumed to only resist bending moment. It is assumed that the concrete by itself can take the increased compression if it is evenly distributed, but that the second order moment results in failure. The steel plates are therefore applied to increase the resistance against bending moment. The amount of steel is chosen so that the same increase of capacity as in Part 2 is reached, i.e. 28%. It is assumed that the column is braced (forced to vertical alignment) temporarily when the load is increased so that the column only is subjected to evenly distributed compression (which it can resist). The steel plates are then applied and the bracing is removed so that the second order moment is activated. Only the contribution from the steel plate that comes in tension is regarded since it is more difficult to transfer compression from the plate to the concrete than tension.



Increasing the load on the braced column before strengthening

$factor_{10} := 1.28$ Increasing the load from Part 9

$N_{Ed.10} := factor_{10} \cdot N_{Ed} = 1.872 \cdot MN$ New vertical load on top of the column



Horizontal equilibrium:

$N_{Ed.10} = F_{c.braced} + F'_{s.braced} + F_{s.braced}$

$\varepsilon_{braced} := 0.001359$ Assuming a value for the strain and iterating

$$F'_{s.braced} := \begin{cases} (\varepsilon_{braced} \cdot E_s \cdot A'_s) & \text{if } \varepsilon_{braced} < \varepsilon_{sy} \\ (f_{yd} \cdot A'_s) & \text{otherwise} \end{cases} = 206.64 \cdot kN$$

$$F_{s.braced} := \begin{cases} (\varepsilon_{braced} \cdot E_s \cdot A_s) & \text{if } \varepsilon_{braced} < \varepsilon_{sy} \\ (f_{yd} \cdot A_s) & \text{otherwise} \end{cases} = 206.64 \cdot kN$$

$$\varepsilon_{c2} := 0.002$$

$$\sigma_{c.braced} := \begin{cases} \left[1 - \left(1 - \frac{\varepsilon_{braced}}{\varepsilon_{c2}}\right)^2\right] \cdot f_{cd} & \text{if } \varepsilon_{braced} < \varepsilon_{c2} \\ f_{cd} & \text{otherwise} \end{cases} = 23.927 \cdot MPa$$

$$F_{c.braced} := \sigma_{c.braced} \cdot (A_c - A'_s - A_s) = 1.459 \cdot MN$$

$$F_{c.braced} + F'_{s.braced} + F_{s.braced} = 1.872 \cdot MN$$

$$N_{Ed.10} = 1.872 \cdot MN$$

These should be the same for horizontal equilibrium. Otherwise, ε_{braced} is updated.

Steel plates

$b_{spi} := 75mm$ — Width of each steel plate

$t_{spi} := 6mm$ — Thickness of each steel plate

$A'_{sp} := b_{spi} \cdot t_{spi} = 450 \cdot mm^2$ — Area of steel plate on top surface

$A_{sp} := b_{spi} \cdot t_{spi} = 450 \cdot mm^2$ — Area of steel plate on bottom surface

$A_{spm} := 2 \cdot b_{spi} \cdot t_{spi} = 900 \cdot mm^2$ — Area of steel plates on side surfaces

First order moment

$e_0 = 0$

$e_i = 10.368 \cdot mm$ — The excentricity should be the same as before

$M_{0.Ed.10} := N_{Ed.10} \cdot (e_0 + e_i) = 19.409 \cdot kN \cdot m$

Nominal bending stiffness

$$I_{spi.stiff} := \frac{t_{spi} \cdot b_{spi}^3}{12} = 2.109 \times 10^5 \cdot mm^4$$

Second moment of inertia for one steel plate around its own axis

$$I_{spi.weak} := \frac{b_{spi} \cdot t_{spi}^3}{12} = 1.35 \times 10^3 \cdot mm^4$$

Adding the bending stiffnes of the four steel plates to the nominal bending stiffness:

$$EI_{10} := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_s \cdot I_{spi.stiff} + 2 \cdot E_s \cdot \left[I_{spi.weak} + A_{sp} \cdot \left(\frac{b_c}{2} + \frac{t_{spi}}{2}\right)^2\right]$$

$$EI_{10} = 6.622 \cdot MN \cdot m^2$$

Second order moment

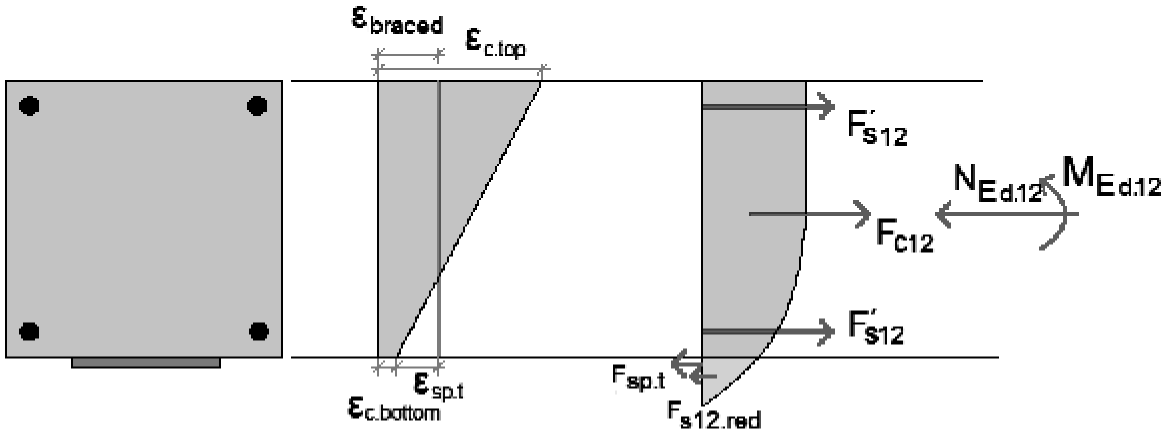
$\beta_{shape} = 1$

$$N_{B.10} := \frac{\pi^2 \cdot EI_{10}}{l_0^{\,2}} = 3.535 \cdot MN$$ Buckling load

$$M_{Ed.10} := \left(1 + \frac{\beta_{shape}}{\dfrac{N_{B.10}}{N_{Ed.10}} - 1}\right) \cdot M_{0.Ed.10} = 41.26 \cdot kN \cdot m$$

Resistance of the section

The bracing is removed so that the second order moment is introduced. Only the steel plate that is on the "tensile" surface is regarded in the equilibrium since it is more difficult to ensure that compression is taken by the external steel.



$\varepsilon_{braced} = 1.359 \times 10^{-3}$

$\varepsilon_{c.top} := \varepsilon_{cu} = 3.5 \times 10^{-3}$ New strain on the top surface

$\varepsilon_{c.bottom} := 0.381 \cdot 10^{-3}$ Assuming strain in bottom (compression)

$$x_{10} := \frac{\varepsilon_{c.top} \cdot b_c}{\varepsilon_{c.top} - \varepsilon_{c.bottom}} = 0.281\,m$$

$$\varepsilon'_{s10} := \varepsilon_{c.top} \cdot \frac{x_{10} - d'_s}{x_{10}} = 2.889 \times 10^{-3}$$

$$\varepsilon_{s10} := \varepsilon_{c.top} \cdot \frac{x_{10} - d_s}{x_{10}} = 9.923 \times 10^{-4}$$

$\varepsilon_{sp.t} := \varepsilon_{braced} - \varepsilon_{c.bottom} = 9.78 \times 10^{-4}$ Tensile strain in steel plate

$F_{sp.t} := A_{sp} \cdot E_s \cdot \varepsilon_{sp.t} = 88.02 \cdot kN$ Force in the steel plate that is in tension

$$\alpha_{red.10} := 0.097 + (0.187 - 0.097)\cdot\frac{\left(\varepsilon_{c.bottom}\cdot 10^3 - 0.2\right)}{(0.4 - 0.2)} = 0.178$$ Stress block factors for the part below the section

$$\beta_{red.10} := 0.336 + (0.339 - 0.336)\cdot\frac{\left(\varepsilon_{c.bottom}\cdot 10^3 - 0.2\right)}{(0.4 - 0.2)} = 0.339$$

$$N_{Rd.10} := \alpha\cdot f_{cd}\cdot b_c\cdot x_{10} - \alpha_{red.10}\cdot f_{cd}\cdot b_c\cdot\left(x_{10} - b_c\right) + f_{yd}\cdot A'_s + \varepsilon_{s10}\cdot E_s\cdot A_s - F_{sp.t} = 1.872\cdot MN$$

$$\frac{N_{Ed.10}}{N_{Rd.10}} = 1$$ Should be 1 (update $\varepsilon_{c.bottom}$)

$$M_{Rd.10} := \alpha\cdot f_{cd}\cdot b_c\cdot x_{10}\cdot\left(d_s - \beta\cdot x_{10}\right) + \alpha_{red.10}\cdot f_{cd}\cdot b_c\cdot\left(x_{10} - b_c\right)\cdot\left[b_c - d_s + \beta_{red.10}\cdot\left(x_{10} - b_c\right)\right] \dots$$
$$+ f_{yd}\cdot A'_s\cdot\left(d_s - d'_s\right) + F_{sp.t}\cdot\left(b_c - d_s\right) - N_{Ed.10}\cdot\left(d_s - \frac{b_c}{2}\right)$$

$$M_{Rd.10} = 42.141\cdot kN\cdot m$$

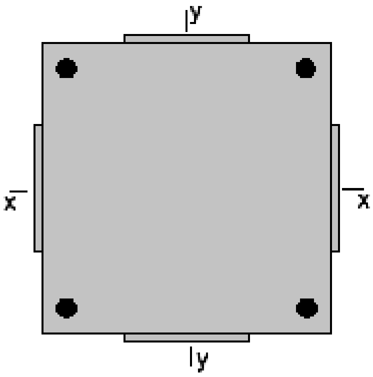
$$M_{Ed.10} = 41.26\cdot kN\cdot m$$

$$\frac{M_{Ed.10}}{M_{Rd.10}} = 0.979 \qquad \frac{N_{Ed.10}\cdot e_{min}}{M_{Rd.10}} = 0.888$$ Utilization of moment resistance

Part 10 - Strengthening with vertically mounted steel plates

Part 11 - Strengthening with vertical surface mounted CFRP laminates

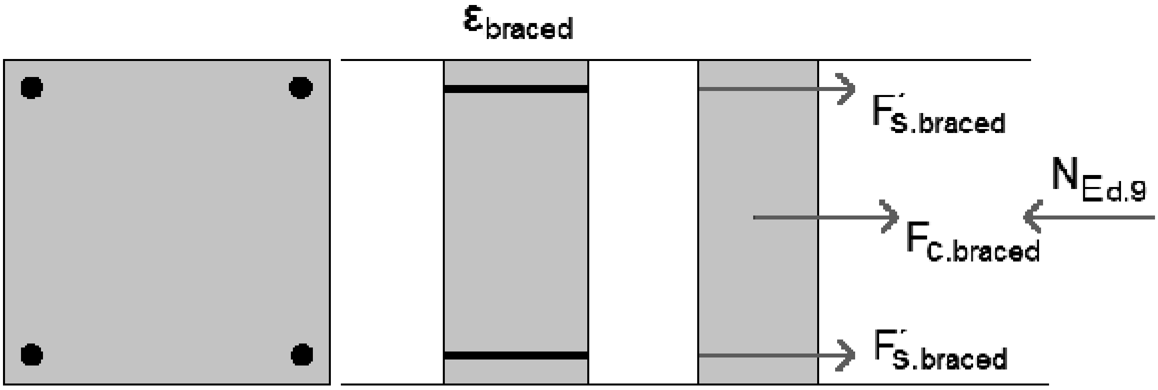
In this case, the rectangular section is strengthened with vertical surface mounted CFRP laminates that are assumed to only resist bending moment. These calculations are performed to be able to compare the results with the ones for the case when steel plates are used (see Part 10). The same assumptions as for Part 10 are made, i.e. that the column is braced (forced to vertical alignment) temporarily when the load is increased so that the column only is subjected to evenly distributed compression (which it can resist). The laminates are then applied and the bracing is removed so that the second order moment is activated. Only the contribution from the laminate that comes in tension is regarded since it is more difficult to transfer compression from the laminate to the



Increasing the load on the braced column before strengthening

$factor_{11} := 1.28$ Increasing the load from Part 11 (the same increase as before is sought)

$N_{Ed.11} := factor_{11} \cdot N_{Ed} = 1.872 \cdot MN$ New vertical load on top of the column



Horizontal equilibrium:

$N_{Ed.11} = F_{c.braced} + F'_{s.braced} + F_{s.braced}$

$\varepsilon_{11.braced} := 0.00136$ Assuming a value for the strain and iterating

$$F'_{s.11.braced} := \begin{cases} (\varepsilon_{11.braced} \cdot E_s \cdot A'_s) & \text{if } \varepsilon_{11.braced} < \varepsilon_{sy} \\ (f_{yd} \cdot A'_s) & \text{otherwise} \end{cases} = 206.792 \cdot kN$$

$$F_{s.11.braced} := \begin{cases} (\varepsilon_{11.braced} \cdot E_s \cdot A_s) & \text{if } \varepsilon_{11.braced} < \varepsilon_{sy} \\ (f_{yd} \cdot A_s) & \text{otherwise} \end{cases} = 206.792 \cdot kN$$

$\varepsilon_{c2} = 2 \times 10^{-3}$

$$\sigma_{c.11.braced} := \begin{cases} \left[1 - \left(1 - \dfrac{\varepsilon_{11.braced}}{\varepsilon_{c2}}\right)^2\right] \cdot f_{cd} & \text{if } \varepsilon_{11.braced} < \varepsilon_{c2} \\ f_{cd} & \text{otherwise} \end{cases} = 23.936 \cdot MPa$$

$$F_{c.11.braced} := \sigma_{c.11.braced} \cdot \left(A_c - A'_s - A_s\right) = 1.46 \cdot MN$$

$$F_{c.11.braced} + F'_{s.11.braced} + F_{s.11.braced} = 1.873 \cdot MN$$

$$N_{Ed.11} = 1.872 \cdot MN$$

These should be the same for horizontal equilibrium

Laminates

$b_{fi} := 250mm$ — Width of each laminate (the with cannot be bigger than the with of the column)

$t_f := 1.4mm$ — Thickness of each laminate

$E_{fm} := 165GPa$ — Modulus of elasticity for CFRP laminates

$A'_f := b_{fi} \cdot t_f = 350 \cdot mm^2$ — Area of laminate on top surface

$A_f := b_{fi} \cdot t_f = 350 \cdot mm^2$ — Area of laminate on bottom surface

$A_{fm} := 2 \cdot b_{fi} \cdot t_f = 700 \cdot mm^2$ — Area of laminates on side surfaces

First order moment

$e_0 = 0$

$e_i = 10.368 \cdot mm$ — The excentricity should be the same as before

$$M_{0.Ed.11} := N_{Ed.11} \cdot \left(e_0 + e_i\right) = 19.409 \cdot kN \cdot m$$

Nominal bending stiffness

$$I_{fi.stiff} := \frac{t_f \cdot b_{fi}^3}{12} = 1.823 \times 10^6 \cdot mm^4$$

Second moment of inertia for one laminate around its own axis

$$I_{fi.weak} := \frac{b_{fi} \cdot t_f^3}{12} = 57.167 \cdot mm^4$$

Adding the bending stiffnes of the four laminates to the nominal bending stiffness:

$$EI_{11} := \frac{k_1 \cdot k_2}{1 + \varphi_{ef}} \cdot E_{cd} \cdot I_c + E_s \cdot I_s + 2 \cdot E_{fm} \cdot I_{fi.stiff} + 2 \cdot E_{fm} \cdot \left[I_{fi.weak} + A_f \cdot \left(\frac{b_c}{2} + \frac{t_f}{2}\right)^2\right]$$

$$EI_{11} = 6.015 \cdot MN \cdot m^2$$

Second order moment

$\beta_{shape} = 1$

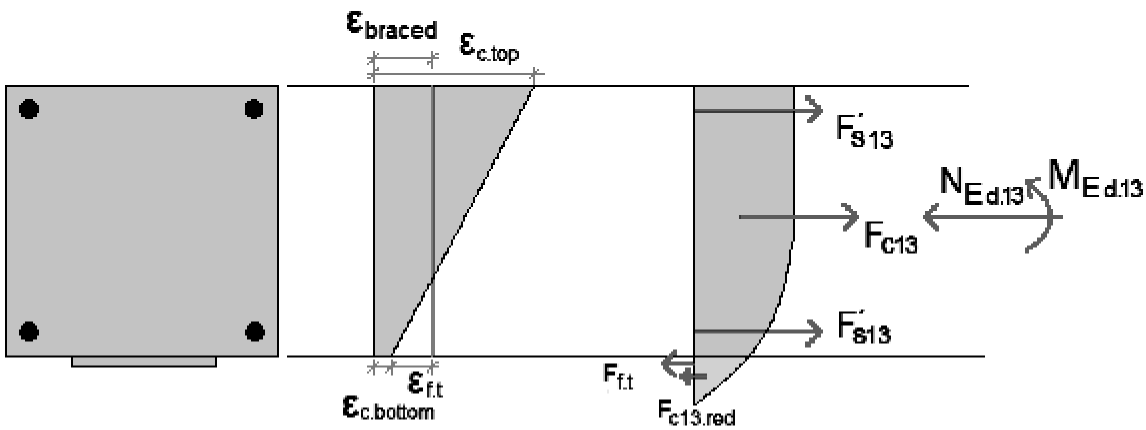
$$N_{B.11} := \frac{\pi^2 \cdot EI_{11}}{l_0^2} = 3.211 \cdot MN$$

Buckling load

$$M_{Ed.11} := \left(1 + \frac{\beta_{shape}}{\frac{N_{B.11}}{N_{Ed.11}} - 1}\right) \cdot M_{0.Ed.11} = 46.554 \cdot kN \cdot m$$

Resistance of the section

The bracing is removed so that the second order moment is introduced. Only the laminate that is on the "tensile" surface is regarded in the equilibrium since it is more difficult to ensure that compression is taken by the external laminates.



$$\varepsilon_{11.braced} = 1.36 \times 10^{-3}$$

$$\varepsilon_{c.11.top} := \varepsilon_{cu} = 3.5 \times 10^{-3}$$ New strain on the top surface

$$\varepsilon_{c.11.bottom} := 0.315 \cdot 10^{-3}$$ Assuming strain in bottom (compression)

$$x_{11} := \frac{\varepsilon_{c.11.top} \cdot b_c}{\varepsilon_{c.11.top} - \varepsilon_{c.11.bottom}} = 0.275\,m$$

$$\varepsilon'_{s11} := \varepsilon_{c.11.top} \cdot \frac{x_{11} - d'_s}{x_{11}} = 2.876 \times 10^{-3}$$

$$\varepsilon_{s11} := \varepsilon_{c.11.top} \cdot \frac{x_{11} - d_s}{x_{11}} = 9.393 \times 10^{-4}$$

$$\varepsilon_{f.t} := \varepsilon_{11.braced} - \varepsilon_{c.11.bottom} = 1.045 \times 10^{-3}$$ Tensile strain in laminate

$$F_{f.t} := A_f \cdot E_{fm} \cdot \varepsilon_{f.t} = 60.349 \cdot kN$$ Force in the laminate that is in tension

$$\alpha_{red.11} := 0.097 + (0.187 - 0.097) \cdot \frac{\left(\varepsilon_{c.11.bottom} \cdot 10^3 - 0.2\right)}{(0.4 - 0.2)} = 0.149$$ Stress block factors for the part below the section

$$\beta_{red.11} := 0.336 + (0.339 - 0.336) \cdot \frac{\left(\varepsilon_{c.11.bottom} \cdot 10^3 - 0.2\right)}{(0.4 - 0.2)} = 0.338$$

$$N_{Rd.11} := \alpha \cdot f_{cd} \cdot b_c \cdot x_{11} - \alpha_{red.11} \cdot f_{cd} \cdot b_c \cdot \left(x_{11} - b_c\right) + f_{yd} \cdot A'_s + \varepsilon_{s11} \cdot E_s \cdot A_s - F_{f.t} = 1.872 \cdot MN$$

$$\frac{N_{Ed.11}}{N_{Rd.11}} = 1$$ Should be 1

$$M_{Rd.11} := \alpha \cdot f_{cd} \cdot b_c \cdot x_{11} \cdot \left(d_s - \beta \cdot x_{11}\right) + \alpha_{red.11} \cdot f_{cd} \cdot b_c \cdot \left(x_{11} - b_c\right) \cdot \left[b_c - d_s + \beta_{red.11} \cdot \left(x_{11} - b_c\right)\right] \dots$$
$$+ f_{yd} \cdot A'_s \cdot \left(d_s - d'_s\right) + F_{f.t} \cdot \left(b_c - d_s\right) - N_{Ed.11} \cdot \left(d_s - \frac{b_c}{2}\right)$$

$$M_{Rd.11} = 40.977 \cdot kN \cdot m$$

$$M_{Ed.11} = 46.554 \cdot kN \cdot m$$

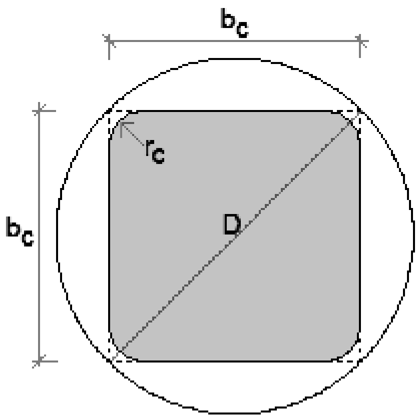
$$\frac{M_{Ed.11}}{M_{Rd.11}} = 1.136 \qquad \frac{N_{Ed.11} \cdot e_{min}}{M_{Rd.11}} = 0.914$$ Utilization of moment resistance

Enough capacity could not be gained from this strengthening method.

Part 11 - Strengthening with vertical surface mounted CFRP laminates

Part 12 - Strengthening with CFRP wrapping - Rectangular section

In this part, the same rectangular column as in Part 9 is strengthened with CFRP sheets that are wrapped around the column so that the fibres are placed in the circumferential direction. It is assumed that the whole column is strengthened and that the corners are smothened. The calculations are based on Täljsten et. al. (2011), Chapter 6.



Increased compressive strength of the concrete

$b_{1c} := b_c = 250 \cdot mm$

$b_{2c} := b_c = 250 \cdot mm$

Should be lower than 900mm to make CFRP wrapping beneficial — OK

$\frac{b_{2c}}{b_{1c}} = 1$

Should be lower than 2 to make CFRP wrapping beneficial — OK

$r_{c.max} := \frac{cover}{1 - \cos(45deg)} = 102.426 \cdot mm$

Maximum radius of the smoothened corners (at which point the stirrups will be reached)

$r_{c.min} := 30mm$

Minimum radius of the smoothened corners according to Täljsten et al. (2011)

$r_c := 60mm$

Chosen radius of the smoothened corners

$D := \sqrt{b_{1c}^2 + b_{2c}^2} = 0.354\,m$

Diameter of the fictitious circular column

$\rho_g := \frac{A'_s + A_s}{A_c} = 0.024$

Reinforcement ratio

$$A_{ce} := \left(A_c - 4\cdot A_{si}\right)\cdot\frac{1 - \dfrac{\dfrac{b_{1c}}{b_{2c}}\left(b_{2c} - 2\cdot r_c\right)^2 + \dfrac{b_{2c}}{b_{1c}}\left(b_{1c} - 2\cdot r_c\right)^2}{3\cdot A_c} - \rho_g}{1 - \rho_g} = 0.05\,m^2$$

$$\kappa_a := \frac{A_{ce}}{A_c - 4\cdot A_{si}}\cdot\left(\frac{b_{1c}}{b_{2c}}\right)^2 = 0.815$$ Geometrical efficiency factor a. Lower than one since the section is non-circular

$$\kappa_b := \frac{A_{ce}}{A_c - 4\cdot A_{si}}\cdot\left(\frac{b_{1c}}{b_{2c}}\right)^{0.5} = 0.815$$ Geometrical efficiency factor b

$\varepsilon_{fu} := 1.55\%$ Ultimate strain in CFRP

$\kappa_e := 0.55$ Efficiency factor concerning premature failure in CFRP due to the triaxial stress situation

$\varepsilon_{fe} := \kappa_e\cdot\varepsilon_{fu} = 8.525\times 10^{-3}$ Effective ultimate strain in CFRP

$E_f := 240GPa$ Modulus of elasticity for CFRP

$n_f := 5$ Number of layers of sheets

$t_f := 0.117mm$ Thickness of one sheet

$$f_l := \frac{2\cdot E_f\cdot n_f\cdot t_f\cdot\varepsilon_{fe}}{D} = 6.771\cdot MPa$$ Maximum wrapping preassure

$$\frac{f_l}{f_{ck}} = 0.169$$ Should at least be 0.08

$\varepsilon_{c2} = 2\times 10^{-3}$ Strain at which the curve for the concrete stress becomes horizontal

$$\varepsilon_{cu.c} := \varepsilon_{c2}\cdot\left[1.50 + 12\cdot\kappa_b\cdot\frac{f_l}{f_{ck}}\cdot\left(\frac{\varepsilon_{fe}}{\varepsilon_{c2}}\right)^{0.45}\right] = 9.359\times 10^{-3}$$ Check of maximum strain in wrapped concrete (should be below 10‰)

$\alpha_{f.c} := 0.95$ Reduction factor to increase the safety of the model

$f_{cd.c} := f_{cd} + \alpha_{f.c}\cdot 3.3\cdot\kappa_a\cdot f_l = 43.971\cdot MPa$ Increased compressive strength of concrete due to triaxial stress state

Loads

$factor_{12} := 1.121$ Increasing the load from Part 11

$N_{Ed.12} := factor_{12}\cdot N_{Ed} = 1.639\cdot MN$ New vertical load on top of the column

First order moment

$e_0 = 0$

$e_i = 0.01\,m$ The excentricity should be the same as before

$M_{0.Ed.12} := N_{Ed.12}\cdot\left(e_0 + e_i\right) = 16.998\cdot kN\cdot m$

Nominal bending stiffness

Since the column hasn't been strengthened with regard to bending resistance, the nominal bending stiffness should be the same as in Part 9.

$EI = 3.588 \cdot MN \cdot m^2$

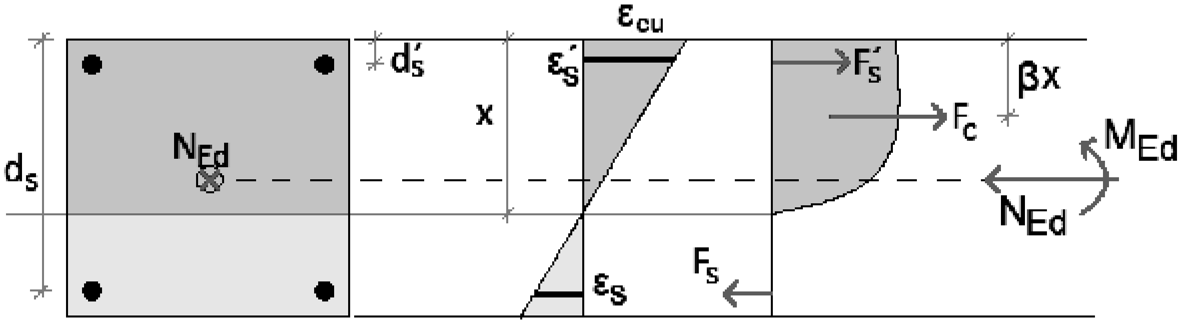
Second order moment

$N_{B.12} := \frac{\pi^2 \cdot EI}{l_0^2} = 1.915 \cdot MN$ Buckling load

$$M_{Ed.12} := \left(1 + \frac{\beta_{shape}}{\frac{N_{B.12}}{N_{Ed.12}} - 1}\right) \cdot M_{0.Ed.12} = 118.026 \cdot kN \cdot m$$

Resistance of the section

The calculations below are based on Section B5.6 in Al-Emrani et al. (2010).
Neglecting that the corners have been smoothened.



$\alpha = 0.81$

$\beta = 0.416$

$A_s = 7.603 \times 10^{-4} m^2$ Bottom reinforcement

$A'_s = 7.603 \times 10^{-4} m^2$ Top reinforcement

$\varepsilon_{cu} = 3.5 \times 10^{-3}$ Maximal strain for the concrete

Horizontal equilibrium (using the increased compressive strength):

$\alpha \cdot f_{cd.c} \cdot b_c \cdot x_{12} + \sigma'_s \cdot A'_s = N_{Ed.12} + \sigma_s \cdot A_s$

$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement yields

$\sigma_s = \varepsilon_s \cdot E_s$ Assuming that bottom reinforcement doesn't yield

$$\varepsilon_s = \varepsilon_{cu} \cdot \frac{d_s - x_{12}}{x_{12}}$$

Calculating height of compressive zone:

$x_{12} := 0.15m$ Assuming a value for x_{12}

Given

$$\alpha \cdot f_{cd.c} \cdot b_c \cdot x_{12} + \sigma'_s \cdot A'_s = N_{Ed.12} + \varepsilon_{cu} \cdot \frac{d_s - x_{12}}{x_{12}} \cdot E_s \cdot A_s$$

$x_{12} := \text{Find}(x_{12}) = 0.162\,m$ Solving x_{12} from horizontal equilibrium

Check of assumptions:

$\varepsilon_{sy} = 2.174 \times 10^{-3}$ Steel strain at yielding

$\varepsilon'_s := \varepsilon_{cu} \cdot \frac{x_{12} - d'_s}{x_{12}} = 2.439 \times 10^{-3}$ $\varepsilon'_s \geq \varepsilon_{sy} = 1$ Top reinf. yielding

$\varepsilon_s := \varepsilon_{cu} \cdot \frac{d_s - x_{12}}{x_{12}} = 8.539 \times 10^{-4}$ $\varepsilon_s \geq \varepsilon_{sy} = 0$ Bottom reinf. NOT yielding

Moment equilibrium around tensile reinforcement:

$$M_{Rd.12} := \alpha \cdot f_{cd.c} \cdot b_c \cdot x_{12} \cdot (d_s - \beta \cdot x_{12}) + \sigma'_s \cdot A'_s \cdot (d_s - d'_s) - N_{Ed.12} \cdot \left(d_s - \frac{b_c}{2}\right) = 118.123 \cdot kN \cdot m$$

Check of resistance

$e_{min} = 0.02\,m$ Minimum excentricity for normal force

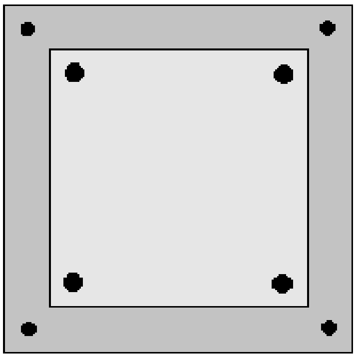
$\frac{M_{Ed.12}}{M_{Rd.12}} = 0.999$ $\frac{N_{Ed.12} \cdot e_{min}}{M_{Rd.12}} = 0.278$

The calculations showed that the load only could be increased with 12.1 % since the concrete in triaxial stress state otherwise would be crushed.

Part 12 - Strengthening with CFRP wrapping - Rectangular section

Part 13 - Strengthening with section enlargement - assumed to only contribute to bending stiffness

In this case, the slender column is strengthened in the same way as the stockier column was in Part 5. The section enlargement is not accounted for in the calculation of the resistance of the critical section so the added layer is assumed to only contribute to the nominal bending stiffness.



Behaviour of the column under quasi-permanent load before strengthening

$N_{Eqp} = 0.93 \cdot MN$ — Assuming that the quasi-permanent load is acting on the column (from Part 9)

$M_{0Eqp} = 9.642 \cdot kN \cdot m$ — First order moment due to quasi-permanent load (from Part 9)

$EI = 3.588 \cdot MN \cdot m^2$ — Nominal bending stiffness before strengthening is the same as in Part 9

$N_B = 1.915 \cdot MN$ — Theoretical buckling force (from Part 9)

$$M_{Eqp} := \left(1 + \frac{\beta_{shape}}{\dfrac{N_B}{N_{Eqp}} - 1}\right) \cdot M_{0Eqp} = 18.744 \cdot kN \cdot m$$ Second order moment due to quasi-permanent load

Sectional analysis:

Assuming that the column is uncracked.

$$\alpha_I := \frac{E_s}{E_{cm}} = 5.714$$ Relationship between modulus of elasticity for reinforcement and concrete

$$A_I := b_c^{\,2} + (\alpha_I - 1) \cdot (A_s + A'_s) = 0.07\,m^2$$ Transformed section area in stadium I

$$I_I := \frac{b_c \cdot b_c^{\,3}}{12} + (\alpha_I - 1) \cdot A'_s \cdot \left(\frac{b_c}{2} - d'_s\right)^2 + (\alpha_I - 1) \cdot A_s \cdot \left(d_s - \frac{b_c}{2}\right)^2 = 3.669 \times 10^{-4}\,m^4$$

$$\sigma_{cn.qp} := \frac{N_{Eqp}}{A_I} = 13.349 \cdot MPa$$ Stress in concrete due to normal force

$$\sigma_{sn.qp} := \alpha_I \cdot \frac{N_{Eqp}}{A_I} = 76.28 \cdot MPa$$ Stress in steel due to normal force

$\sigma_{cm.qp.top} := \frac{M_{Eqp}}{I_I} \cdot \frac{b_c}{2} = 6.385 \cdot MPa$ Stress in concrete at top due to moment

$\sigma_{cm.qp.bottom} := \frac{M_{Eqp}}{I_I} \cdot \frac{-b_c}{2} = -6.385 \cdot MPa$ Stress in concrete at bottom due to moment

$\sigma_{c.qp.top} := \sigma_{cn.qp} + \sigma_{cm.qp.top} = 19.734 \cdot MPa$ Stress at top surface

$\sigma_{c.qp.bottom} := \sigma_{cn.qp} + \sigma_{cm.qp.bottom} = 6.964 \cdot MPa$ Stress at bottom surface

$\varepsilon_{c.qp.top} := \frac{\sigma_{c.qp.top}}{E_{cm}} = 5.638 \times 10^{-4}$ Strain at top surface

$\varepsilon_{c.qp.bottom} := \frac{\sigma_{c.qp.bottom}}{E_{cm}} = 1.99 \times 10^{-4}$ Strain at bottom surface

$curve_{qp} := \frac{\varepsilon_{c.qp.top} - \varepsilon_{c.qp.bottom}}{b_c} = 1.46 \times 10^{-3} \frac{1}{m}$ Curvature in the column before strengthening under quasi-permanent load.

$c_{shape} := 10$ Factor that considers the curvature distribution

$e_{2.qp} := \frac{l_0^{\,2}}{c_{shape}} \cdot curve_{qp} = 2.699 \cdot mm$ Eccentricity in critical section due to second order effects before strengthening

$e_{qp} := e_0 + e_i + e_{2.qp} = 13.067 \cdot mm$ Eccentricity in critical section before strengthening

Loads after increase

$factor_{13} := 1.28$ The same load increase as for Part 2 is sought

$N_{Ed.13} := factor_{13} \cdot N_{Ed} = 1.872 \cdot MN$

$N_{Eqp.13} := factor_{13} \cdot N_{Eqp} = 1.19 \cdot MN$

First order moments after the load increase

Using the eccentricity that takes the original creep into consideration.

$M_{0Ed.13} := N_{Ed.13} \cdot e_{qp} = 24.461 \cdot kN \cdot m$ ULS combination

$M_{0Eqp.13} := N_{Eqp.13} \cdot e_{qp} = 15.555 \cdot kN \cdot m$ Quasi-permanent SLS combination

Input data for the new layer

Using the same concrete to simplify calculations

$\phi_{si.13} := 16mm$ — Reinforcement diameter for new bars

$$A_{si.13} := \frac{\pi \cdot \phi_{si.13}^{2}}{4}$$ Area of one new bar

$A_{s.13} := 4 \cdot A_{si.13} + 4 \cdot A_{si} = 2.325 \times 10^{-3}\,m^2$ — Area of all bars in the section

$c_{13} := 30mm$ — Concrete cover (roughly chosen)

$a_{13} := 65mm$ — Thickness of new layer (iterated until good utilisation is reached)

$b_{c.13} := b_c + 2 \cdot a_{13} = 0.38\,m$ — Width of the new column

$A_{c.13} := 4 \cdot \left(b_{c.13} - a_{13}\right) \cdot a_{13} = 0.082\,m^2$ — Gross area of the added layer

$A_{c.13.tot} := b_c^{2} + A_{c.13} = 0.144\,m^2$ — Gross area of the total section

Evaluation of slenderness

The calculations below are based on Section B11.3.2 in Al-Emrani et al. (2011)

The column is regarded as an isolated structural member with pinned connections in each end.

$l_0 = 4.3\,m$ — Buckling length (assumed pinned-pinned connections)

$$I_{c.13} := \frac{b_{c.13} \cdot b_{c.13}^{3}}{12} = 1.738 \times 10^{-3}\,m^4$$ Second moment of inertia of the gross concrete section

$$i_{13} := \sqrt{\frac{I_{c.13}}{A_{c.13.tot}}} = 0.11\,m$$ Radius of gyration

$$\lambda_{13} := \frac{l_0}{i_{13}} = 39.199$$ Slenderness

Rough estimation of the limit value of the slenderness:

$$n_{13} := \frac{N_{Ed.13}}{f_{cd} \cdot A_{c.13.tot}} = 0.486$$ Relative normal force

$$\lambda_{lim.13} := \frac{10.8}{\sqrt{n_{13}}} = 15.49$$ Rough value of limit

Since $\lambda_{13} > \lambda_{lim.13}$, the column must be designed with regard to the second order moment.

Creep for additional load if the whole section would have been cast at the same time as the original section

The calculations below are based on Section B2.1.6 in Al-Emrani et al. (2010).
It is first assumed that the whole section was cast at the same time as the original column so that it is old when the load is increased. The results are then weighted against calculations where it is assumed that the whole section is newly cast.

$t_{increase.1} := 40 \cdot 365 = 1.46 \times 10^{4}$ Concrete age in days at the time when the load is increased (assuming 40 years)

$RH = 50 \cdot \%$ Indoor climate

$u_{13} := 4 \cdot b_{c.13} = 1.52\,m$ All sides of the column are subjected to drying

$h_{0.13} := \dfrac{2 \cdot A_{c.13.tot}}{u_{13}} = 0.19\,m$ Nominal thickness

$f_{cm} = 48 \cdot MPa$ Mean value of compressive strength of concrete

$$\varphi_{RH.13} := \left[1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{\dfrac{h_{0.13}}{mm}}} \cdot \left(\frac{35}{\dfrac{f_{cm}}{MPa}}\right)^{0.7}\right] \cdot \left(\frac{35}{\dfrac{f_{cm}}{MPa}}\right)^{0.2} = 1.593$$ Creep from relative humidity

$\beta_{fcm.13} := 2.43$ Factor that considers the strength of the concrete

$\beta_{t.increase.1} := \dfrac{1}{0.1 + t_{increase.1}^{\;0.20}} = 0.145$ Assuming that the additional load was applied after 40 years

$\varphi_{inf.increase.1} := \varphi_{RH.13} \cdot \beta_{fcm.13} \cdot \beta_{t.increase.1} = 0.561$ Final creep

Creep for additional load if the whole section was cast 28 days before the load was increased

$t_{increase.2} := 28$ Concrete age in days at the time when the load is increased (this time assuming 28 days)

$$\beta_{t.increase.2} := \frac{1}{0.1 + t_{increase.2}^{\;0.20}} = 0.488$$

$$\varphi_{inf.increase.2} := \varphi_{RH.13} \cdot \beta_{fcm.13} \cdot \beta_{t.increase.2} = 1.891$$

Weighting the two creep factors for the load increase

Since the two ways to calculate the creep for the added load represent the two extremities, a weighted value is calculated. This value is based on how large part of the section that consist of old and new. concrete respectively.

$$\varphi_{inf.increase} := \frac{\varphi_{inf.increase.1} \cdot b_c^{\,2} + \varphi_{inf.increase.2} \cdot A_{c.13}}{A_{c.13.tot}} = 1.315$$

$\varphi_{ef.increase} := \varphi_{inf.increase} \cdot \dfrac{M_{0Eqp.13}}{M_{0Ed.13}} = 0.836$ Effective creep

Nominal bending stiffness

The calculations below are based on Section B11.4.2 in Al-Emrani et al. (2011)

$\rho_{reinf.13} := \dfrac{A_{s.13}}{A_{c.13.tot}} = 0.016$ Reinforcement ratio in new layer

$\rho_{reinf.13} \geq 0.002 = 1$ OK!

$\gamma_{cE} = 1.2$ National parameter

$E_{cd} := \dfrac{E_{cm}}{\gamma_{cE}} = 29.167 \cdot GPa$ Design value of modulus of elasticity for the concrete

$$k_{1.13} := \sqrt{\dfrac{\dfrac{f_{ck}}{MPa}}{20}} = 1.414$$

$$k_{2.13} := \frac{N_{Ed.13}}{f_{cd} \cdot A_{c.13.tot}} \cdot \frac{\lambda_{13}}{170} = 0.112$$

Simplified second moment of inertia for reinforcement:

$$I_{s.13} := 4 \cdot A_{si} \cdot \left(\frac{b_c}{2} - cover - \phi_{st.i} - \frac{\phi_{si}}{2} \right)^2 + 4 \cdot A_{si.13} \cdot \left(\frac{b_{c.13}}{2} - c_{13} - \frac{\phi_{si.13}}{2} \right)^2 = 2.736 \times 10^{-5} \, m^4$$

$EI_{13} := \dfrac{k_{1.13} \cdot k_{2.13}}{1 + \varphi_{ef.increase}} \cdot E_{cd} \cdot I_{c.13} + E_s \cdot I_{s.13} = 9.848 \cdot MN \cdot m^2$ Nominal bending stiffness

Second order moment

$N_{B.13} := \dfrac{\pi^2 \cdot EI_{13}}{l_0^{\,2}} = 5.257 \cdot MN$ Theoretical buckling force

$\beta_{shape} = 1$ Due to sinus-shaped bending moment

$M_{Ed.13} := \left(1 + \dfrac{\beta_{shape}}{\dfrac{N_{B.13}}{N_{Ed.13}} - 1} \right) \cdot M_{0Ed.13} = 37.99 \cdot kN \cdot m$ Second order moment

Resistance of the section

The calculations below are based on Section B5.6 in Al-Emrani et al. (2010)

Assuming that the new layer of concrete CANNOT help to resist the combination of normal force and bending moment.

$\alpha = 0.81$

$\beta = 0.416$

$A_s = 7.603 \times 10^{-4} \, m^2$ Bottom reinforcement in old part

$A'_s = 7.603 \times 10^{-4} \, m^2$ Top reinforcement in old part

$d_s = 201 \cdot mm$ Distances from top (of old part) to reinforcement

$d'_s = 49 \cdot mm$

$\varepsilon_{cu} = 3.5 \times 10^{-3}$ Maximal strain for the concrete

Horizontal equilibrium:

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_{13} - \alpha_{red.13} \cdot f_{cd} \cdot b_c \cdot \left(x_{13} - b_c\right) + \sigma'_s \cdot A'_s + \sigma_s \cdot A_s = N_{Ed.13}$$

$\sigma'_s := f_{yd} = 434.783 \cdot MPa$ Assuming that top reinforcement in old part yields

$\sigma_s = \varepsilon_s \cdot E_s$ Assuming that bottom reinforcement in old part doesn't yield

$$\varepsilon_s = \varepsilon_{cu} \cdot \frac{x_{13} - d_s}{x_{13}}$$

Calculating height of compressive zone:

$\varepsilon_{cc.min.13} := 0.0001796$ Assuming that the whole section is in compression.

$$\alpha_{red.13} := 0 + (0.097 - 0) \cdot \frac{\left(\varepsilon_{cc.min.13} \cdot 10^{3} - 0\right)}{(0.2 - 0)} = 0.087$$

$\beta_{red.13} := 0.336$

Factors for the part of the compression block that comes below the section. α_{red} and β_{red} are in this case dependent on the strain at the the bottom of the cross-section.

$x_{13} := 0.5m$ Assuming an initial value for x_{13}

Given

$$\alpha \cdot f_{cd} \cdot b_c \cdot x_{13} - \alpha_{red.13} \cdot f_{cd} \cdot b_c \cdot \left(x_{13} - b_c\right) + \sigma'_s \cdot A'_s + \varepsilon_{cu} \cdot \frac{x_{13} - d_s}{x_{13}} \cdot E_s \cdot A_s = N_{Ed.13}$$

$x_{13} := \mathrm{Find}\left(x_{13}\right) = 0.264 m$ Solving x from horizontal equilibrium

Check of assumptions:

$$\varepsilon_{cc.min.13} := \varepsilon_{cu} \cdot \frac{x_{13} - b_c}{x_{13}} = 1.796 \times 10^{-4}$$

Concrete strain at "bottom side". Check with assumption and iterate.

$\varepsilon_{sy} = 2.174 \times 10^{-3}$ Steel strain at yielding

$$\varepsilon'_s := \varepsilon_{cu} \cdot \frac{x_{13} - d'_s}{x_{13}} = 2.849 \times 10^{-3} \qquad \varepsilon'_s \geq \varepsilon_{sy} = 1$$

Top reinf. in old part yielding

$$\varepsilon_s := \varepsilon_{cu} \cdot \frac{x_{13} - d_s}{x_{13}} = 8.304 \times 10^{-4} \qquad \varepsilon_s \geq \varepsilon_{sy} = 0$$

Bottom reinf. in old part NOT yielding

Moment equilibrium around bottom reinforcement in old part:

$$M_{Rd.13} := \alpha \cdot f_{cd} \cdot b_c \cdot x_{13} \cdot \left(d_s - \beta \cdot x_{13}\right) + \alpha_{red.13} \cdot f_{cd} \cdot b_c \cdot \left(x_{13} - b_c\right) \cdot \left[b_c - d_s + \beta_{red.13} \cdot \left(x_{13} - b_c\right)\right] \ldots$$
$$+ \sigma'_s \cdot A'_s \cdot \left(d_s - d'_s\right) - N_{Ed.13} \cdot \left(d_s - \frac{b_c}{2}\right)$$

$$M_{Rd.13} = 38.42 \cdot kN \cdot m$$

Check of resistance

$$e_{min.13} := \max\left(\frac{b_c}{30}, 20mm\right) = 20 \cdot mm$$ Minimum eccentricity for normal force

$$\frac{M_{Ed.13}}{M_{Rd.13}} = 0.989 \qquad \frac{N_{Ed.13} \cdot e_{min.13}}{M_{Rd.13}} = 0.974$$

Part 13 - Strengthening with section enlargement - assumed to only contribute to bending stiffness

Appendix E – Calculations for strengthening of one-way slabs

To better see the differences between various ways to strengthen slabs, calculations have been performed for some methods. The investigated member is a simply supported one-way slab upon which the distributed load is increased. The calculations are presented in this appendix but described and discussed in Section 7.1.

The following subsections are treated:

Part 1 - Capacity of original slab

Input data

$L := 6m$	Length of span
$h := 160mm$	Assumed height of slab
$\phi_s := 10mm$	Diameter of reinforcement bars
$s_s := 110mm$	spacing between reinforcement bars (iterated to get enough resistance)
$A_{s.m} := \frac{\pi \cdot \phi_s^2}{4} \cdot \frac{1m}{s_s} = 713.998 \cdot mm^2$	Total area of reinforcement bars per meter
$d_s := h - 30mm = 130 \cdot mm$	Distance from top surface to reinforcement (assuming that the bars lie 30mm from the surface)
$f_{ck} := 40MPa$	Assumed value of concrete strength
$f_{cd} := \frac{f_{ck}}{1.5} = 26.667 \cdot MPa$	Design value of concrete strength
$f_{ctm} := 3.5MPa$	Tensile strength of concrete
$E_{cm} := 35GPa$	Mean value of modulus of elasticity for the concrete
$f_{yk} := 500MPa$	Assumed value of yield strength of reinforcement
$f_{yd} := \frac{f_{yk}}{1.15} = 434.783 \cdot MPa$	Design value
$E_s := 200GPa$	Assumed value of modulus of elasticity for reinforcement
$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.174 \times 10^{-3}$	Steel strain at yielding

Loads

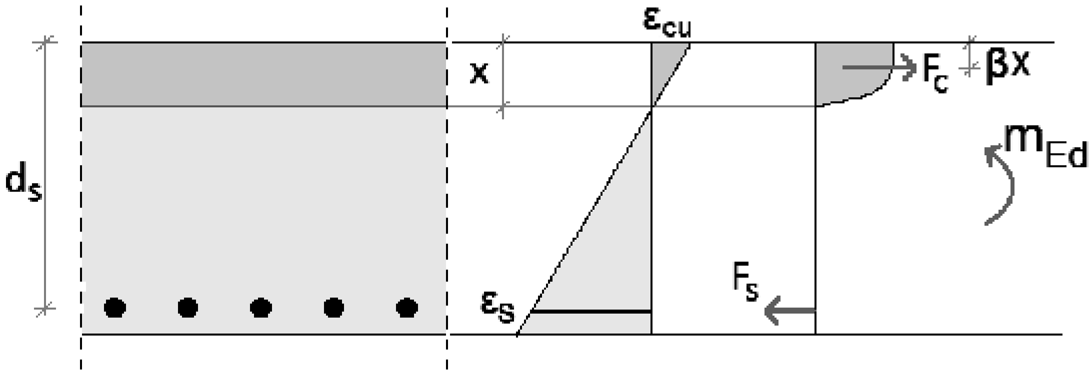
$q_k := 2 \frac{kN}{m^2}$	Assumed value of variable distributed loads before strengthening
$g_k := \frac{25kN}{m^3} \cdot h = 4 \cdot \frac{kN}{m^2}$	Simplified calculation of self-weight of the slab
$q_d := 1.35 \cdot g_k + 1.5 \cdot q_k = 8.4 \cdot \frac{kN}{m^2}$	ULS combination before load increase
$q_{qp} := g_k + 0.3 \cdot q_k = 4.6 \cdot \frac{kN}{m^2}$	Quasi-permanent combination before increase
$q_{freq} := g_k + 0.5 \cdot q_k = 5 \cdot \frac{kN}{m^2}$	Frequent combination before load increase

Calculation of resistance in ULS

$m_{Ed} := \dfrac{q_d \cdot L^2}{8} = 37.8 \cdot \dfrac{kN \cdot m}{m}$ Moment that must be resisted in mid-section in ULS

Horizontal equilibrium:

$f_{cd} \cdot \alpha \cdot x \cdot 1m = \sigma_s \cdot A_{s.m}$



$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$ Maximum strain in concrete

$\alpha := 0.81$ Stress block factors

$\beta := 0.416$

$\sigma_s = f_{yd}$ Assuming that the reinforcement yields

$x := 100mm$ First guess of height of the compressive zone

Given

$f_{cd} \cdot \alpha \cdot x \cdot 1m = f_{yd} \cdot A_{s.m}$ Horizontal equilibrium

$x := Find(x) = 14.372 \cdot mm$ Height of compressive zone

$\varepsilon_s := \varepsilon_{cu} \cdot \dfrac{d_s - x}{x} = 0.028$ Strain in reinforcement

$\varepsilon_s \geq \varepsilon_{sy} = 1$ Reinforcement is yielding as assumed

Moment equilibrium around tensile reinforcement:

$m_{Rd} := f_{cd} \cdot \alpha \cdot x \cdot \left(d_s - \beta \cdot x\right) = 38.5 \cdot \dfrac{kN \cdot m}{m}$ Moment resistance

$m_{Ed} = 37.8 \cdot \dfrac{kN \cdot m}{m}$

$\dfrac{m_{Ed}}{m_{Rd}} = 0.982$ Utilisation of moment capacity

Part 1 - Capacity of original slab

Part 2 - Original slab in service state before load increase

The behaviour before strengthening is analysed according to Täljsten et al. (2011).

$\varphi_{ef} := 2$ Creep factor, very roughly assumed in the same way as Täljsten et al.

$\alpha_s := \dfrac{E_s \cdot (1 + \varphi_{ef})}{E_{cm}} = 17.143$ Proportionality factor between steel and concrete

$y_0 := \dfrac{1m \cdot h \cdot \frac{h}{2} + (\alpha_s - 1) \cdot A_{s.m} \cdot d_s}{1m \cdot h + (\alpha_s - 1) \cdot A_{s.m}} = 83.36 \cdot mm$ Neutral layer for the section before cracking of concrete (stadium I), i.e. the mass centre

$A_I := h \cdot 1m + (\alpha - 1) \cdot A_{s.m} = 0.16\, m^2$

Second moment of inertia in stadium I:

$$I_I := \frac{1m \cdot h^3}{12} + 1m \cdot h \cdot \left(y_0 - \frac{h}{2}\right)^2 + (\alpha_s - 1) \cdot A_{s.m} \cdot (d_s - y_0)^2 = 3.682 \times 10^8 \cdot mm^4$$

$m_{freq} := \dfrac{q_{freq} \cdot L^2}{8} = 22.5 \cdot \dfrac{kN \cdot m}{m}$ Maximum moment from frequent load

$\sigma_{ct.freq} := \dfrac{m_{freq} \cdot 1m}{I_I} \cdot (h - y_0) = 4.683 \cdot MPa$ Tensile stress on bottom surface

$\dfrac{\sigma_{ct.freq}}{f_{ctm}} = 1.338$ Utilisation of tensile capacity of concrete

The utilisation is above 1, so the concrete has cracked. Therefore, the calculations should be performed for concrete in stadium II.

Horizontal equilibrium:

$F_c = F_s$

$F_c = 1m \cdot \dfrac{x_{freq}}{2} \cdot \varepsilon_{cc} \cdot E_{c.eff}$

$F_s = A_{s.m} \cdot \alpha_s \cdot E_{c.eff} \cdot \varepsilon_s$

$\varepsilon_s = \varepsilon_{cc} \cdot \dfrac{d_s - x_{qp}}{x_{qp}}$

==> $1m \cdot \dfrac{x_{qp}}{2} \cdot \varepsilon_{cc} \cdot E_{c.eff} = A_{s.m} \cdot \alpha_s \cdot E_{c.eff} \cdot \varepsilon_{cc} \cdot \dfrac{d_s - x_{freq}}{x_{freq}}$

$x_{freq} := y_0$

Given

$1m \cdot x_{freq} \cdot \dfrac{x_{freq}}{2} = \alpha_s \cdot A_{s.m} \cdot (d_s - x_{freq})$

$x_{freq} := \mathrm{Find}(x_{freq}) = 45.485 \cdot mm$ Distance from top surface to neutral layer under frequent load combination in stadium II.

Second moment of inertia in stadium II:

$$I_{II} := \frac{1m \cdot x_{freq}^{3}}{12} + 1m \cdot x_{freq} \cdot \left(\frac{x_{freq}}{2}\right)^{2} + (\alpha_s - 1) \cdot A_{s.m} \cdot (d_s - x_{freq})^{2} = 1.137 \times 10^{8} \cdot mm^{4}$$

Initial strain and stress relations

$$\sigma_{cc.freq} := \frac{1m \cdot m_{freq}}{I_{II}} \cdot x_{freq} = 9.001 \cdot MPa$$ Stress in compressed concrete

$$\sigma_{s.freq} := \alpha_s \cdot \frac{1m \cdot m_{freq}}{I_{II}} \cdot (d_s - x_{freq}) = 286.718 \cdot MPa$$ Stress in steel

Assuming that only the characteristic value of the self-weight of the slab acts at the time of strengthening (all other loads are removed before strengthening)

$$m_{g.k} := \frac{g_k \cdot L^{2}}{8} = 18 \cdot \frac{kN \cdot m}{m}$$ Maximum moment at the time of strengthening

$$\sigma_{cc.g.k} := \frac{1m \cdot m_{g.k}}{I_{II}} \cdot x_{freq} = 7.201 \cdot MPa$$ Stress at compressed surface

$$\sigma_{s.g.k} := \alpha_s \cdot \frac{1m \cdot m_{g.k}}{I_{II}} \cdot (d_s - x_{freq}) = 229.375 \cdot MPa$$ Stress in steel

$$\varepsilon_{cc.g.k} := \frac{\sigma_{cc.g.k}}{E_{cm}} \cdot (1 + \varphi_{ef}) = 6.172 \times 10^{-4}$$ Strain on compressed side

$$\varepsilon_{s.g.k} := \frac{\sigma_{s.g.k}}{E_s} = 1.147 \times 10^{-3}$$ Strain in reinforcement

$$\varepsilon_{t.g.k} := \varepsilon_{s.g.k} \cdot \frac{h - x_{freq}}{d_s - x_{freq}} = 1.554 \times 10^{-3}$$ Strain on tensile surface

Part 2 - Original slab in service state before load increase

Part 3 - Strengthening with surface mounted CFRP strips

The flexural resistance of the slab is strengthened by CFRP strips that are glued to the bottom surface of the slab. The fibres are placed in the same direction as the span.

The calculations in this part are based on Täljsten et al. (2011). Especially, the calculations in *Appendix A - Exempel 1. Böjning* have been used.

$q_{k.add.3} := 2.5 \frac{kN}{m^2}$ Additional distributed load on top of the slab

$q_{d3} := 1.35 \cdot g_k + 1.5 \cdot (q_k + q_{k.add.3}) = 12.15 \cdot \frac{kN}{m^2}$ New design value (ULS)

$m_{Ed.3} := \frac{q_{d3} \cdot L^2}{8} = 54.675 \cdot \frac{kN \cdot m}{m}$ New moment in mid-section

Estimated need of strengthening

In the first stage, simplified calculations are used to estimate the needed amount of CFRP strips.

Input data for CFRP laminates:

The same values as Täljsten et al. (2011) are used for the CFRP strips.

$\varepsilon_{fk} := 15 \cdot 10^{-3}$ Characteristic value of ultimate strain in CFRP, from Täljsten et al. (2011)

$\gamma_{frp} := 1.2$

$\varepsilon_{fd} := \frac{\varepsilon_{fk}}{\gamma_{frp}} = 0.0125$ Design value

$E_{fk} := 160GPa$ Characteristic value of elastic modulus for CFRP

$E_{fd} := \frac{E_{fk}}{\gamma_{frp}} = 133.333 \cdot GPa$ Design value

$t_{frp} := 1.2mm$ Thickness of laminates

$n := 1$ Number of layers of CFRP laminates

$\varepsilon_{fd.ic} := \min\left(0.41 \cdot \sqrt{\frac{f_{cd}}{n \cdot E_{fd} \cdot \frac{t_{frp}}{mm}}}, 0.9 \cdot \varepsilon_{fd}\right) = 5.293 \times 10^{-3}$ Reduction of allowed strain due to horizontal cracks that propagate along the laminate

$A_{frp.prel} := \frac{\frac{m_{Ed.3} \cdot 1m}{0.9} - A_{s.m} \cdot f_{yd} \cdot d_s}{\varepsilon_{fd.ic} \cdot E_{fd} \cdot h} = 180.604 \cdot mm^2$ Estimated needed area of CFRP per meter slab

$b_{frp.prel} := \frac{A_{frp.prel}}{t_{frp}} = 150.503 \cdot mm$ Estimated needed width of CFRP per meter slab

Using 60mm wide laminates with a spacing of 40cm.

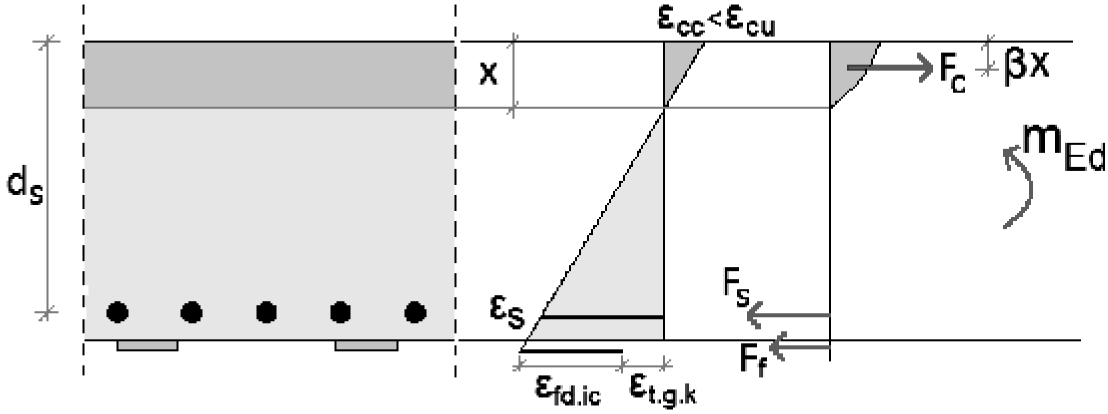
$s_{frp} := 40cm$

$b_{frp} := \frac{60mm}{s_{frp}} = 150 \cdot \frac{mm}{m}$ Chosen width of CFRP per meter slab

$A_{frp} := b_{frp} \cdot t_{frp} = 180 \cdot \frac{mm^2}{m}$ Chosen area of CFRP per meter slab

New moment capacity

Assuming that the reinforcement is yielding and that $\varepsilon_{fd.ic}$ restricts how large part of the strain in the laminates that can be accounted for. Assuming a value for the strain at the top surface and iterating until horizontal equilibrium is reached.



$\varepsilon_{cc.3} := 0.0015153$ Strain at top surface

$\alpha_3 := 0.537 + (0.587 - 0.537) \cdot \frac{\left(\varepsilon_{cc.3} \cdot 10^3 - 1.4\right)}{(1.6 - 1.4)} = 0.56$ Stress block factors

$\beta_3 := 0.359 + (0.364 - 0.359) \cdot \frac{\left(\varepsilon_{cc.3} \cdot 10^3 - 1.4\right)}{(1.6 - 1.4)} = 0.362$

$\varepsilon_{ct.3} := \varepsilon_{fd.ic} + \varepsilon_{t.g.k} = 6.8471 \times 10^{-3}$ Strain at lower surface

$x_3 := h \cdot \frac{\varepsilon_{cc.3}}{\varepsilon_{cc.3} + \varepsilon_{ct.3}} = 28.993 \cdot mm$ Height of compression zone

Horizontal equilibrium:

$F_{c.3} = F_{s.3} + F_{f.3}$

$F_{s.3} := f_{yd} \cdot A_{s.m} = 310.434 \cdot kN$

$F_{f.3} := \varepsilon_{fd.ic} \cdot E_{fd} \cdot A_{frp} \cdot 1m = 127.034 \cdot kN$

$F_{s.3} + F_{f.3} = 437.468 \cdot kN$ Iterating $\varepsilon_{cc.3}$ until these equations give the same result

$F_{c.3} := \alpha_3 \cdot f_{cd} \cdot x_3 \cdot 1m = 437.463 \cdot kN$

Check of assumptions:

$$\varepsilon_{s.3} := \varepsilon_{cc.3} \cdot \frac{d_s - x_3}{x_3} = 0.528 \cdot \%$$ Steel strain

$\varepsilon_{sy} = 0.217 \cdot \%$

$\varepsilon_{s.3} \geq \varepsilon_{sy} = 1$ Reinforcement is yielding

Calculating moment resistance:

$$m_{Rd.3} := \frac{A_{s.m}}{m} \cdot f_{yd} \cdot \left(d_s - \beta_3 \cdot x_3\right) + \varepsilon_{fd.ic} \cdot E_{fd} \cdot A_{frp} \cdot \left(h - \beta_3 \cdot x_3\right) = 56.092 \cdot \frac{kN \cdot m}{m}$$

$$\frac{m_{Ed.3}}{m_{Rd.3}} = 0.975$$ Utilisation of moment capacity

Check of ductility

This check is done to ensure that the concrete isn't crushed.

$\lambda := 0.8$ For a square-shaped stress block

$$\omega_{bal} := \frac{\lambda}{1 + \frac{\varepsilon_{fd.ic} + \varepsilon_{t.g.k}}{\varepsilon_{cu}}} = 0.271$$

$$\omega := \frac{A_{s.m} \cdot f_{yd} + A_{frp} \cdot m \cdot \varepsilon_{fd.ic} \cdot E_{fd}}{1m \cdot h \cdot f_{cd}} = 0.103$$

$\omega_{bal} > \omega = 1$ If $\omega_{bal} > \omega$, the concrete will not be crushed before full utilisation of the reinforcement.

Check of required anchorage length

Calculating at which section (x_{cr} from the support) where the last crack in the concrete occurs. For simplicity, only the bending stiffness for the concrete section without reinforcement is used

$$I_c := \frac{1m \cdot h^3}{12} = 3.413 \times 10^8 \cdot mm^4$$

$$W_c := \frac{I_c}{0.5 \cdot h} = 4.267 \times 10^6 \cdot mm^3$$

$$m_{x.cr} := \frac{W_c \cdot f_{ctm}}{1m} = 14.933 \cdot \frac{kN \cdot m}{m}$$

$$m_x(x) = R_{A.3} \cdot x - factor_3 \cdot q_d \cdot \frac{x^2}{2}$$

$$R_{A.3} := \frac{q_{d3} \cdot L}{2} = 36.45 \cdot \frac{kN}{m}$$

$x_{cr} := 1m$

Given

$$m_{x.cr} = R_{A.3} \cdot x_{cr} - q_{d3} \cdot \frac{x_{cr}^2}{2}$$

$$x_{cr} := \mathrm{Find}(x_{cr}) = 442.298 \cdot mm$$

Calculation of increased moment due to displacement between the moment curve and the curve for the tensile force. This displacement depends on the inclined cracks from the shear force.

$a_l := 0.45 \cdot d_s = 58.5 \cdot mm$ Assuming cracks in 45deg

$$m_{xa} := R_{A.3} \cdot (x_{cr} + a_l) - q_{d3} \cdot \frac{(x_{cr} + a_l)^2}{2} = 16.73 \cdot \frac{kN \cdot m}{m}$$

Calculating the needed tensile force in the CFRP to be able to resist m_{xa}

$$F_{frp.xa} := \frac{\frac{m_{xa}}{0.9 \cdot h}}{1 + \frac{E_s \cdot \frac{A_{s.m}}{1m}}{E_{fd} \cdot A_{frp}} \cdot \left(\frac{d_s}{h}\right)^2} = 23.577 \cdot \frac{kN}{m}$$

Checking if the force in the CFRP is sufficiently low to enable anchorage to the concrete.

$$k_b := \sqrt{\frac{2 - \max(b_{frp}, 0.33)}{1 + \max(b_{frp}, 0.33)}} = 1.121$$

$$G_f := 0.03mm \cdot k_b \cdot \sqrt{f_{ck} \cdot f_{ctm}} = 0.398 \cdot \frac{N \cdot mm}{mm^2}$$

$$\varepsilon_{f.x} := \sqrt{\frac{2G_f}{E_{fd} \cdot t_{frp}}} = 2.23 \times 10^{-3}$$

$$F_{fe} := \varepsilon_{f.x} \cdot A_{frp} \cdot E_{fd} = 53.515 \cdot \frac{kN}{m}$$

$F_{frp.xa} \le F_{fe} = 1$ The force is small enough to be anchored. Otherwise, a new location for anchorage would have needed to be chosen.

Calculating anchorage length.

$$l_{ef} := \left(\sqrt{\frac{E_{fd} \cdot \frac{t_{frp}}{mm}}{2 \cdot f_{ctm}}}\right) mm = 151.186 \cdot mm$$

Needed anchorage length. Täljsten et al. however recommend that the anchorage length never should be smaller than 250mm.

$x_{cr} = 442.298 \cdot mm$

Since x_{cr} is quite much larger than 250mm, there is no problem to anchor the strips. To get a more favourable stress state, it is chosen to anchor the strips maximum 100mm from the support.

$x_{frp.end} := \min\left(x_{cr} - l_{ef}, x_{cr} - 250mm, 100mm\right) = 100 \cdot mm$ Distance from support to end of strip

Check of peeling forces at the end of the strip

$E_a := 12.8GPa$	Modulus of elasticity for the adhesive
$\nu_a := 0.3$	Poisson's ratio for the adhesive
$G_a := \frac{E_a}{2 \cdot (1 + \nu_a)} = 4.923 \cdot GPa$	Shear modulus for the adhesive
$l_1 := \frac{L}{2} - x_{frp.end} = 2.9 \times 10^3 \cdot mm$	Length of the laminates from mid-section to end
$n := \frac{1m}{s_{frp}} = 2.5$	Number of laminates (calculating on a 1m wide strip of the slab)
$z_0 := h - x_3 = 131.007 \cdot mm$	Height from neutral axis to laminates
$W_c = 4.267 \times 10^{-3} \cdot m^3$	Sectional modulus for the concrete section

$$\lambda_b := \sqrt{\frac{\frac{G_a}{MPa} \cdot \frac{b_{frp} \cdot 1m}{mm}}{n} \cdot \left(\frac{1}{\frac{E_{fd}}{MPa} \cdot \frac{A_{frp} \cdot 1m}{mm^2}} + \frac{1}{\frac{E_{cm}}{MPa} \cdot \frac{h \cdot 1m}{mm^2}} + \frac{\frac{z_0}{mm}}{\frac{E_{cm}}{MPa} \cdot \frac{W_c}{mm^3}} \right)} = 0.112$$

$$\tau_{max} := \frac{\frac{q_{d3} \cdot 1m}{\frac{kN}{m}}}{2} \cdot \frac{\frac{G_a}{MPa}}{n \cdot \frac{E_{cm}}{MPa} \cdot \frac{W_c}{mm^3}} \cdot \frac{\left[\left(\frac{x_{frp.end}}{mm} \right)^2 + 2 \cdot \frac{x_{frp.end} \cdot l_1}{mm^2} \right] \cdot \lambda_b + \frac{l_1}{mm}}{\lambda_b^2} \cdot MPa = 0.439 \cdot MPa$$

Calculating the tensile stress difference in the concrete surface at the end of the laminates (the concrete is uncracked since $a<x_{cr}$):

$\Delta q_3 := q_{d3} - g_k = 8.15 \cdot \frac{kN}{m^2}$	Load increase from the application of the laminates to ULS
$\Delta R_A := \frac{\Delta q_3 \cdot L}{2} = 24.45 \cdot \frac{kN}{m}$	Increased load on support
$\Delta M_a := \Delta R_A \cdot x_{frp.end} - \Delta q_3 \cdot \frac{x_{frp.end}^2}{2} = 2.404 \cdot \frac{kN \cdot m}{m}$	Increased moment in the section where the laminates end

$$\Delta\sigma_a := \frac{\Delta M_a \cdot 1m}{I_I} \cdot (h - y_0) = 0.5 \cdot MPa$$ Increased tensile stress

Calculating the principle stress in that section:

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

The calculations are on the safe side if it is assumed that $\sigma_y = \tau_{xy} = \tau_{max}$

$$\sigma_1 := \frac{\Delta\sigma_a + \tau_{max}}{2} + \sqrt{\left(\frac{\Delta\sigma_a - \tau_{max}}{2}\right)^2 + \tau_{max}^2} = 0.91 \cdot MPa$$

$$\frac{\sigma_1}{f_{ctm}} = 0.26$$ Check of utilisation

Check of peeling forces at the end of the strip (SIKA)

Westerberg (2006), presented another approach to treat the peeling forces then Täljsten et al. (2011). Therefore, the calculations for the peeling forces were performed once more with the approach that Westerberg presented.

$$t_a := 2mm$$

$$c := \sqrt{t_a \cdot t_{frp} \cdot \frac{E_{fd}}{G_a}} = 8.062 \times 10^{-3} m$$

$$R_A := \frac{q_{d3} \cdot L}{2} = 36.45 \cdot \frac{kN}{m}$$

$$V_a := R_A \cdot 1m - q_{d3} \cdot 1m \cdot x_{frp.end} = 35.235 \cdot kN$$

$$\tau_{max} := V_a \cdot \left(\frac{x_{frp.end}}{c} + 1\right) \cdot \frac{E_{fd}}{E_{cm}} \cdot \frac{t_{frp}}{W_c} = 0.506 \cdot MPa$$

$$\frac{\tau_{max}}{f_{ctm}} = 0.145$$

The results show that the two methods give different utilisation factors. However, both of them indicate that the resistance against peeling forces is enough.

Part 3 - Strengthening with surface mounted CFRP strips

Part 4 - Strengthening with near-surface mounted CFRP bars

This time, the slab is strengthened in the same way as in Part 3, but near surface mounted CFRP bars are used instead.

The calculations in this part are based on Täljsten et al. (2011). Especially, the calculations in *Appendix A - Exempel 2. Böjning NSM* have been used.

$q_{k.add.4} := 2.5\frac{kN}{m^2}$ Additional distributed load on top of the slab

$q_{d4} := 1.35 \cdot g_k + 1.5 \cdot \left(q_k + q_{k.add.4}\right) = 12.15 \cdot \frac{kN}{m^2}$ New design value (ULS)

$m_{Ed.4} := \frac{q_{d4} \cdot L^2}{8} = 54.675 \cdot \frac{kN \cdot m}{m}$ New moment in mid-section

Estimation of need of strengthening

Input data for NSM CFRP:

StoFRP bar E10C are used.

$h_{frp.4} := 10mm$ Cross-sectional height of the CFRP bar

$t_{frp.4} := 10mm$ Cross-sectional width of the CFRP bar

$s_{frp.4} := 85cm$ Spacing between CFRP bars

$\varepsilon_{fk.4} := 0.012$ Characteristic value of ultimate strain in CFRP

$\gamma_{frp.4} := 1.35$

$\varepsilon_{fd.4} := \frac{\varepsilon_{fk.4}}{\gamma_{frp.4}} = 8.889 \times 10^{-3}$ Design value. This time, there is no need to reduce the allowed strain in the CFRP due to horizontal cracks that propagate along the laminate

$E_{fk.4} := 160GPa$ Characteristic value of elastic modulus for CFRP

$E_{fd.4} := \frac{E_{fk.4}}{\gamma_{frp.4}} = 118.519 \cdot GPa$ Design value

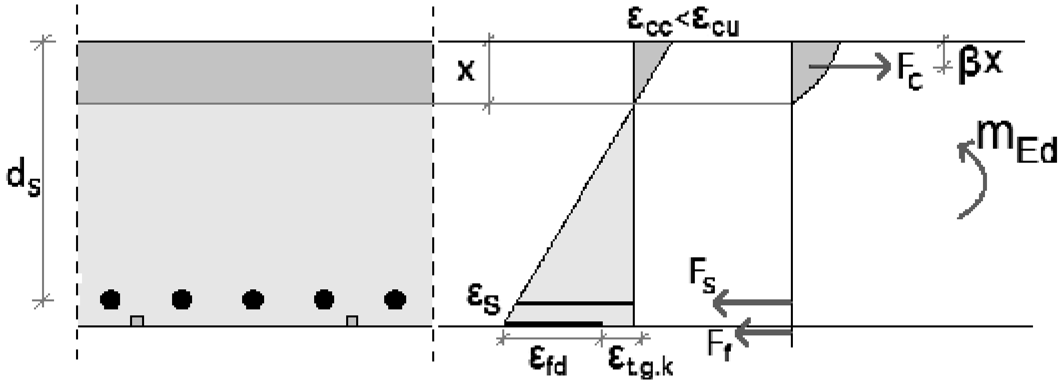
$A_{frp.4.prel} := \frac{\frac{m_{Ed.4} \cdot 1m}{0.9} - A_{s.m} \cdot f_{yd} \cdot d_s}{\varepsilon_{fd.4} \cdot E_{fd.4} \cdot h} \cdot \frac{1}{m} = 120.987 \cdot \frac{mm^2}{m}$ Estimated need of CFRP per meter width of the slab

$A_{frp.4} := \frac{h_{frp.4} \cdot t_{frp.4}}{s_{frp.4}} = 117.647 \cdot \frac{mm^2}{m}$ Chosen area of CFRP per meter width of the slab

New moment capacity

Horizontal equilibrium:

Assuming that the steel is yielding and that the maximum allowed strain in the CFRP is reached. Assuming a value for the strain at the top surface and iterating until horizontal equilibrium is reached.



$\varepsilon_{cc.4} := 0.0019271$ Strain at top surface

$$\alpha_4 := 0.63 + (0.667 - 0.63) \cdot \frac{\left(\varepsilon_{cc.4} \cdot 10^3 - 1.8\right)}{(2.0 - 1.8)} = 0.654$$ Stress block factors

$$\beta_4 := 0.369 + (0.375 - 0.369) \cdot \frac{\left(\varepsilon_{cc.4} \cdot 10^3 - 1.8\right)}{(2.0 - 1.8)} = 0.373$$

$\varepsilon_{ct.4} := \varepsilon_{fd.4} + \varepsilon_{t.g.k} = 0.0104$ Strain at lower surface

$$x_4 := h \cdot \frac{\varepsilon_{cc.4}}{\varepsilon_{cc.4} + \varepsilon_{ct.4}} = 24.926 \cdot mm$$ Height of compression zone

$F_{c.4} = F_{s.4} + F_{f.4}$

$F_{s.4} := f_{yd} \cdot A_{s.m} = 310.434 \cdot kN$

$F_{f.4} := \varepsilon_{fd.4} \cdot E_{fd.4} \cdot A_{frp.4} \cdot 1m = 123.941 \cdot kN$

$F_{s.4} + F_{f.4} = 434.375 \cdot kN$ Iterating $\varepsilon_{cc.4}$ until these equations give the same result

$F_{c.4} := \alpha_4 \cdot f_{cd} \cdot x_4 \cdot 1m = 434.389 \cdot kN$

Check of assumptions:

$$\varepsilon_{s.4} := \varepsilon_{cc.4} \cdot \frac{d_s - x_4}{x_4} = 0.812 \cdot \%$$ Steel strain

$\varepsilon_{sy} = 0.217 \cdot \%$

$\varepsilon_{s.4} \geq \varepsilon_{sy} = 1$ Reinforcement is yielding

Calculating moment resistance:

$$m_{Rd.4} := \frac{A_{s.m}}{m} \cdot f_{yd} \cdot (d_s - \beta_4 \cdot x_4) + \varepsilon_{fd.4} \cdot E_{fd.4} \cdot A_{frp.4} \cdot (h - \beta_4 \cdot x_4) = 56.15 \cdot \frac{kN \cdot m}{m}$$

$\frac{m_{Ed.4}}{m_{Rd.4}} = 0.974$ Utilisation of moment capacity

Check of ductility

This check is done to ensure that the concrete isn't crushed.

$\lambda = 0.8$ For a square-shaped stress block

$$\omega_{bal.4} := \frac{\lambda}{1 + \frac{\varepsilon_{fd.4} + \varepsilon_{t.g.k}}{\varepsilon_{cu}}} = 0.201$$

$$\omega_4 := \frac{A_{s.m} \cdot f_{yd} + A_{frp.4} \cdot 1m \cdot E_{fd.4} \cdot \varepsilon_{fd.4}}{1m \cdot h \cdot f_{cd}} = 0.102$$

$\omega_{bal.4} > \omega_4 = 1$ If $\omega_{bal} > \omega$, the concrete will not be crushed before full utilisation of the reinforcement.

Check of required anchorage length

Calculating at which section (x_{cr} from the support) where the last crack in the concrete occurs. For simplicity, only the bending stiffness for the concrete section without reinforcement is used (on the safe side). Since the load on the slab is increased with the same factor as in Part 3, the calculations for the critical section (where the last crack occurs) are the same as in Part 3. They are therefore not presented here.

Calculating the needed tensile force in the CFRP to be able to resist m_{xa}:

$$F_{frp.xa.4} := \frac{\frac{m_{xa}}{0.9 \cdot h}}{1 + \frac{E_s \cdot A_{s.m}}{E_{fd.4} \cdot A_{frp.4} \cdot 1m} \cdot \left(\frac{d_s}{h}\right)^2} = 14.97 \cdot \frac{kN}{m}$$

Checking if the force in the CFRP is sufficiently low to enable anchorage to the concrete.

$b_g := 12mm$ Depth of the sawn groove

$t_g := 14mm$ Width of the sawn groove

$L_{per} := 2 \cdot b_g + t_g = 38 \cdot mm$ Perimeter of the surface in the groove

$$\tau_f := \left[0.54 \cdot \sqrt{\frac{f_{cd}}{MPa}} \cdot \left(\frac{h_{frp.4}}{mm}\right)^{0.4} \cdot \left(\frac{t_{frp.4}}{mm}\right)^{0.4}\right] \cdot MPa = 17.595 \cdot MPa$$ Shear stress

$$\delta_f := 0.78 \cdot \frac{\left(\frac{f_{cd}}{MPa}\right)^{0.27}}{\left(\frac{t_g}{mm}\right)^{0.3}} \cdot mm = 0.858 \cdot mm$$

Displacement

$$\lambda_f := \sqrt{\frac{\frac{\tau_f}{MPa} \cdot \frac{L_{per}}{mm}}{\frac{\delta_f}{mm} \cdot \frac{E_{fd.4}}{MPa} \cdot \frac{A_{frp.4}}{\frac{mm^2}{m}}}} = 7.478 \times 10^{-3}$$

Relationship between L_{per}, $\tau_{f,}$ d_f E_{fd} and $A_{frp.4}$

$$L_e := \frac{\pi}{2 \cdot \lambda_f} \cdot mm = 210.066 \cdot mm$$

Required anchorage length (but never use less than 250mm)

$$a_{max.4} := x_{cr} - \max(L_e, 250mm) = 192.298 \cdot mm$$

Maximum distance from support to end of the CFRP bars.

Part 4 - Strengthening with near-surface mounted CFRP bars

Part 5 - Strengthening with steel beams on top of the slab

This time, the distributed load is directed to HEA-beams that lie on top of the original slab. When the beams deflect, they push down the slab which in its turn resists the deflection. It is however assumed that the beams glide on top of the slab (no bending interaction).

Since the calculations only treat ULS, the respective capacities for the two members can be added together. A hinge will in ULS have been developed in mid-section for both members, independently of which member that yields first.

Input data for steel beams, HEA140:

$b_{HEA} := 140mm$ Width of steel beams

$h_{HEA} := 133mm$

$t_{w.HEA} := 5.5mm$

$t_{f.HEA} := 8.5mm$

$g_{HEA} := 24.7\frac{kg}{m}\cdot 9.82\frac{m}{s^2} = 0.243\cdot\frac{kN}{m}$ Self-weight of steel beams

$I_{x.HEA} := 10.33\cdot 10^6 mm^4$ Second moment of inertia for steel beams

$f_{yk.HEA} := 355MPa$ Yield strength

$\gamma_{M.1} := 1.1$

$f_{yd.HEA} := \frac{f_{yk.HEA}}{\gamma_{M.1}} = 322.727\cdot MPa$

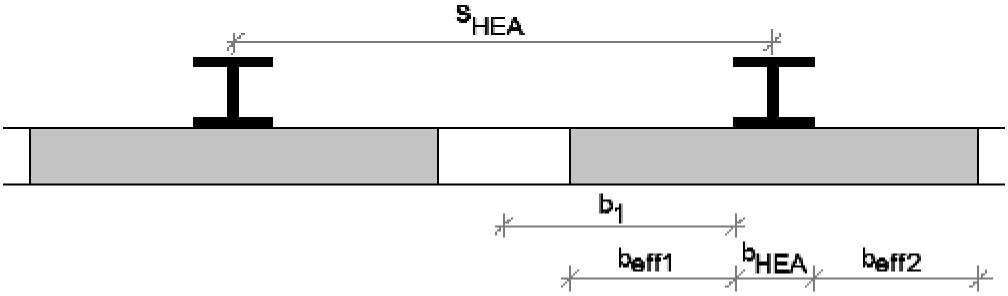
$E_{s.HEA} := 210GPa$

$s_{HEA} := 2.6m$ Spacing between steel beams (iterated)

$\gamma_{M1} := 1.1$

Effective width of slab beneath beams:

The calculations below are based on Section B1.2.5 in Al-Emrani et al. (2010)



$b_1 := \frac{s_{HEA} - b_{HEA}}{2} = 1.23\,m$

$L_0 := L = 6\,m$ Length between zero-moment sections (same as the span length since the slab is simply supported)

$b_{eff.1} := \min(0.2 \cdot b_1 + 0.1 \cdot L_0, 0.2 \cdot L_0, b_1) = 0.846\,m$

$b_{eff.2} := b_{eff.1} = 0.846\,m$

$b_{eff} := b_{eff.1} + b_{eff.2} + b_{HEA} = 1.832\,m$ Effective width of slab beneath each steel beam

Resistance of steel beams

Assuming that the beams only are allowed to bend elastically.

$W_{pl.HEA} := 173 \cdot 10^3 mm^3$ Section modulus for steel beams if plastic behaviour is allowed

$\chi_{LT} := 1.0$

$M_{Rd.HEA} := \chi_{LT} \cdot W_{pl.HEA} \cdot f_{yd.HEA} = 55.832 \cdot kN \cdot m$

$q_{d.HEA} := \frac{M_{Rd.HEA} \cdot 8}{L^2} = 12.407 \cdot \frac{kN}{m}$ Distributed load that the beams themselves can transport to the support (including yielding).

Resistance of slab

$m_{Rd} \cdot b_{eff} = 70.533 \cdot kN \cdot m$ Moment resistance from Part 1 (taken for the effective width)

$q_{d.5.slab} := \frac{m_{Rd} \cdot b_{eff} \cdot 8}{L^2} = 15.674 \cdot \frac{kN}{m}$ Load that the effective width of the slab can resist in stadium III.

Total resistance of the two members

$g_{d.slab.eff} := 1.35 \cdot g_k \cdot b_{eff} = 9.893 \cdot \frac{kN}{m}$ Self-weight of the slab within the effective width

$g_{d.HEA} := 1.35 g_{HEA} = 0.327 \cdot \frac{kN}{m}$ Self-weight of one steel beam

Load that the two members together can resist from above:

$q_{k.5.line} := \frac{1}{1.5} \cdot (q_{d.HEA} - g_{d.HEA} + q_{d.5.slab} - g_{d.slab.eff}) = 11.907 \cdot \frac{kN}{m}$

$q_{k.5} := \frac{q_{k.5.line}}{s_{HEA}} = 4.58 \cdot \frac{kN}{m^2}$

$q_{k.5.add} := q_{k.5} - q_k = 2.58 \cdot \frac{kN}{m^2}$ Load increase that is possible due to the strengthening (should be $2.5 kN/m^2$)

Buckling of the web in the steel beams

Since the web of the steel beam is loaded vertically by both the load that the beam resist and the load that the slab resist, it can be good to investigate if there is any risk of buckling of the web.

$h_{HEA.web} := h_{HEA} - 2 \cdot t_{f.HEA} = 116 \cdot mm$ — Height of the web

$h_{0.HEA} := 0.5 \cdot h_{HEA.web} = 58 \cdot mm$ — Buckling length of the web (assuming that the web has fixed ends.

$\nu := 0.3$ — Poisson's ratio

$$\sigma_{cr} := \frac{\pi^2 \cdot E_{s.HEA}}{12 \cdot \left(1 - \nu^2\right) \cdot \left(\frac{h_{0.HEA}}{t_{w.HEA}}\right)^2} = 1.707 \times 10^3 \cdot MPa$$

Critical buckling stress for the web

$$q_{cr} := \sigma_{cr} \cdot t_{w.HEA} = 9.387 \times 10^3 \cdot \frac{kN}{m}$$

Critical buckling load

$$\frac{q_{k.5.line} \cdot 1.5}{q_{cr}} = 1.903 \times 10^{-3}$$

Since this ratio is far below 1, the web in the steel beam will not buckle

Part 5 - Strengthening with steel beams on top of the slab

Part 6 - Strengthening with post-tensioned steel strands

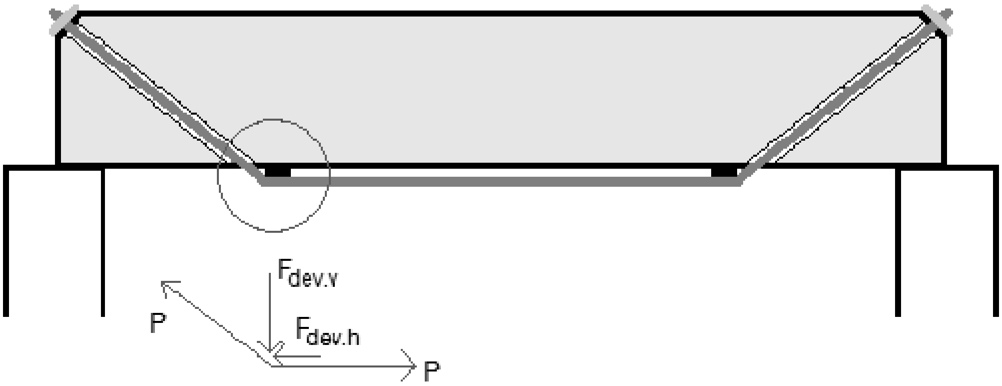
Steel strands in plastic covers are placed in drilled holes through the slab and anchored in the top at the ends of the slab. The strain in the strands depend on the initial prestressing and the difference in length of the strands.

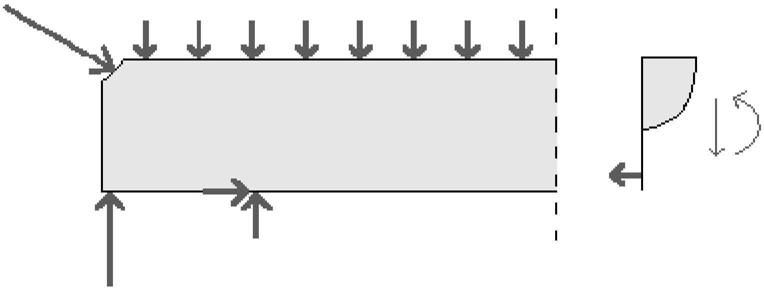
Input data:

$s_{strand} := 80cm$	Spacing between strands
$n_{wires} := 7$	Number of wires per strand
$d_{strand} := 13mm$	Diameter of one strand
$A_{pi} := 100mm^2$	Steel area of one strand
$d_p := h + \frac{d_{strand}}{2} = 166.5 \cdot mm$	Distance from top surface to prestressing steel
$L_1 := 50cm$	Distance from support to deviator
$\alpha_{strand} := \operatorname{atan}\left(\frac{h}{L_1}\right) = 17.745 \cdot deg$	Angle of strands before deviator
$f_{puk} := 1860MPa$	Ultimate strength of the prestresssing steel
$f_{p0.1k} := 1580MPa$	0.1% proof-stress for the prestressing steel
$\gamma_s := 1.15$	
$f_{pd} := \frac{f_{p0.1k}}{\gamma_s} = 1.374 \times 10^3 \cdot MPa$	Design value for tensile strength
$E_p := 190GPa$	Modulus of elasticity
$P_i := f_{pd} \cdot A_{pi} = 137.391 \cdot kN$	Initial force in one strand before anchorage
$\sigma_{pi} := \frac{P_i}{A_{pi}} = 1.374 \times 10^3 \cdot MPa$	Initial stress in prestressing steel
$P_{i.m} := \frac{P_i}{s_{strand}} = 171.739 \cdot \frac{kN}{m}$	Initial strand force per meter width of the slab

Maximal strand force with regard to tensile failure of concrete over the deviator while tensioning

Since there is a lack of top reinforcement, there is a risk of cracking of the top side of the slab above the deviators when applying the prestressing force.





$P_{h.i} := P_{i.m} \cdot \cos(\alpha_{strand}) = 163.568 \cdot \frac{kN}{m}$ Horizontal component of the initial prestressing force

$P_{v.i} := P_{i.m} \cdot \sin(\alpha_{strand}) = 52.342 \cdot \frac{kN}{m}$ Vertical component of the initial prestressing force

$F_{dev.h.i} := P_{i.m} - P_{i.m} \cdot \cos(\alpha_{strand}) = 8.171 \cdot \frac{kN}{m}$ Horizontal component of the initial force at the deviator

$F_{dev.v.i} := P_{i.m} \cdot \sin(\alpha_{strand}) = 52.342 \cdot \frac{kN}{m}$ Vertical component of the initial force at the deviator

$R_{A.i} := 1.0 \cdot g_k \cdot \frac{L}{2} = 12 \cdot \frac{kN}{m}$ Force from the support

Moment in the section where the deviator is (where the tension in the top surface is at its highest):

$$M_{6.L1.i} := R_{A.i} \cdot L_1 - P_{v.i} \cdot L_1 + P_{h.i} \cdot y_0 - F_{dev.h.i} \cdot (h - y_0) - \frac{1.0 g_k \cdot L_1^{\,2}}{2} = -7.662 \cdot \frac{kN \cdot m}{m}$$

The tension in the top surface over the deviator is calculated by Navier's formula:

$$\sigma_{c.L1.i} := \frac{-P_{h.i} \cdot 1m - F_{dev.h.i} \cdot 1m}{A_I} + \frac{M_{6.L1.i} \cdot 1m}{I_I} \cdot -y_0 = 0.66 \cdot MPa$$

$f_{ctm} = 3.5 \cdot MPa$ Tensile strength of the concrete

$$\frac{\sigma_{c.L1.i}}{f_{ctm}} = 0.189$$ Utilisation of tension in top surface

Since there is no risk that the top of the slab cracks, the strands can be tensioned up to the design value of their tensile strength.

Moment in mid-span at the time of tensioning

Assuming that the whole slab is in stadium I (simplifying the calculations by neglecting the influence of deflection on the moment)

$$M_{6.mid.i} := R_{A.i}\cdot\frac{L}{2} - P_{v.i}\cdot\frac{L}{2} + P_{h.i}\cdot y_0 - F_{dev.h.i}\cdot(h - y_0) + F_{dev.v.i}\cdot\left(\frac{L}{2} - L_1\right) - \frac{1.0 g_k\cdot\left(\frac{L}{2}\right)^2}{2}$$

$$M_{6.mid.i} = 4.838\cdot\frac{kN\cdot m}{m}$$

$$\sigma_{c.mid.i} := \frac{-P_{h.i}\cdot 1m - F_{dev.h.i}\cdot 1m}{A_I} + \frac{M_{6.mid.i}\cdot 1m}{I_I}\cdot(h - y_0) = -0.067\cdot MPa$$ Stress in lower surface in mid-span

Since the lower surface is in compression, the whole slab is in stadium I

Approximate calculation of deflection at deviators at the time of tensioning

$$x := 0, 0.1m .. \frac{L}{2}$$

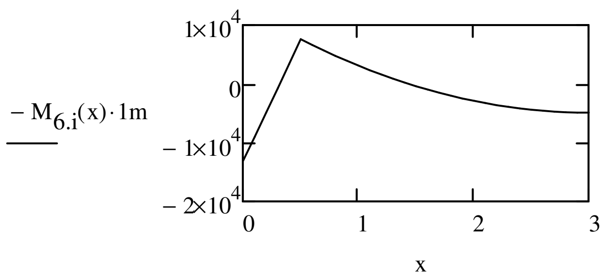
Moment equation valid to the left of the deviator:

$$M_{6.i.1}(x) := R_{A.i}\cdot x - P_{v.i}\cdot x + P_{h.i}\cdot y_0 - F_{dev.h.i}\cdot(h - y_0) - \frac{1.0 g_k\cdot x^2}{2}$$

Moment equation valid to the right of the deviator:

$$M_{6.i.2}(x) := R_{A.i}\cdot x - P_{v.i}\cdot x + P_{h.i}\cdot y_0 - F_{dev.h.i}\cdot(h - y_0) + F_{dev.v.i}\cdot(x - L_1) - \frac{1.0 g_k\cdot x^2}{2}$$

$$M_{6.i}(x) := \text{if}\left(x < L_1, M_{6.i.1}(x), M_{6.i.2}(x)\right)$$



$$\kappa_{appr.i}(x) := \frac{M_{6.i}(x)\cdot 1m}{E_{cm}\cdot I_I}$$ Curvature along the span (stadium I)

$$\theta_{A.appr.i} := \int_0^{\frac{L}{2}} \kappa_{appr.i}(x)\,dx = 2.378 \times 10^{-4}$$

Rotation at the support

$$f_{dev.i} := \theta_{A.appr.i} \cdot L_1 - \int_0^{L_1} \kappa_{appr.i}(x) \cdot (L_1 - x)\,dx = 0.059 \cdot mm$$

The positive sign on the deflection at the deviator indicates that the deflection is downwards.

Loads

$$q_{k.add.6} := 2.5 \frac{kN}{m^2}$$

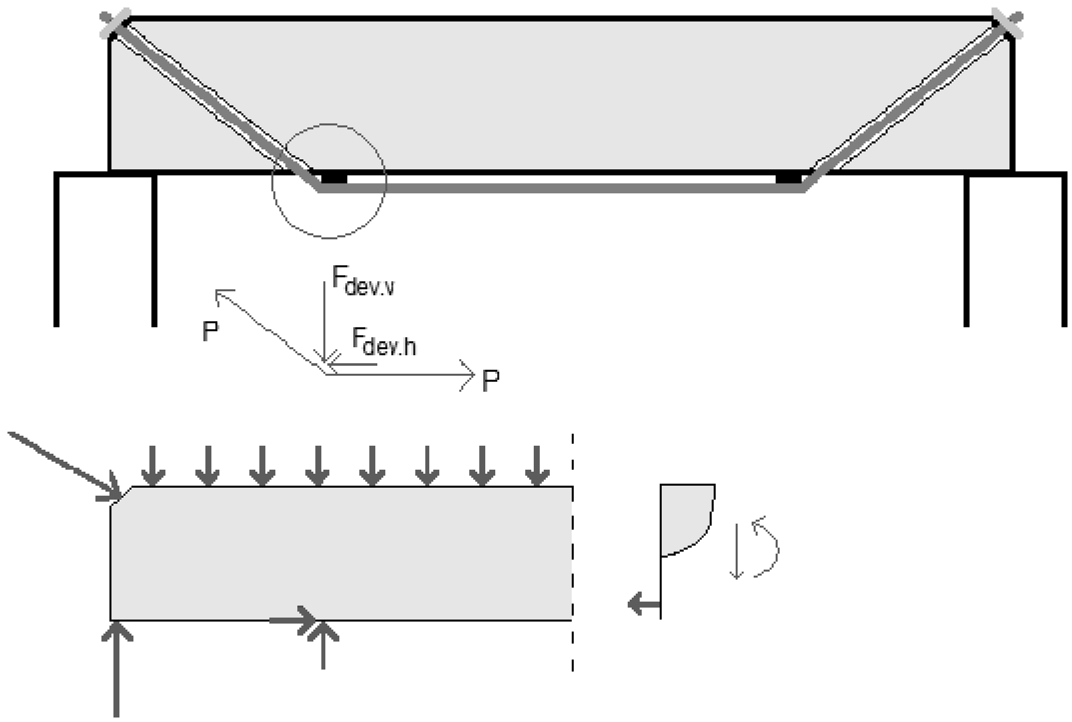
Additional load

$$q_{d.6} := 1.35 \cdot g_k + 1.5 \cdot (q_k + q_{k.add.6}) = 12.15 \cdot \frac{kN}{m^2}$$

New design value (ULS)

Moment from loads in ULS

Apart from the self-weight and the variable loads, the prestressing steel creates point forces at the anchors and deviators. For simplicity, it is assumed that the stress in the strands is constant along the length of the steel (in reality, the friction in the deviators creates differences along the strands). Both the vertical and horizontal components of these forces are regarded when the moment is calculated.



$$P_{ULS} := 153 \frac{kN}{m}$$

Prestressing force in ULS after long time (the difference from the initial value is calculated furthe down, i.e. the value is iterated)

$F_{dev.v.ULS} := P_{ULS} \cdot \sin(\alpha_{strand}) = 46.631 \cdot \frac{kN}{m}$

$F_{dev.h.ULS} := P_{ULS} - P_{ULS} \cdot \cos(\alpha_{strand}) = 7.279 \cdot \frac{kN}{m}$

$f_{mid.ULS} := 33.9mm$ Deflection in mid section in ULS (iterated below)

$x_6 := 21.455mm$ Height of compressive zone (from below)

$$m_{Ed.6} := \frac{q_{d.6} \cdot L^2}{8} + P_{ULS} \cdot \cos(\alpha_{strand}) \cdot (x_6 + f_{mid.ULS}) - P_{ULS} \cdot \sin(\alpha_{strand}) \cdot \frac{L}{2} \dots$$
$$+ F_{dev.v.ULS} \cdot \left(\frac{L}{2} - L_1\right) - F_{dev.h.ULS} \cdot \left[d_p - (x_6 + f_{mid.ULS})\right]$$

$m_{Ed.6} = 38.617 \cdot \frac{kN \cdot m}{m}$

Sectional analysis in mid section in ULS

The concrete section in mid-span will act in the same way as before the strengthening, but with the exception that the section also must resist the horizontal load from the anchors and deviators. This load is placed in the neutral layer since the moment from eccentricity is regarded in $m_{Ed.6}$ instead.

Horizontal equilibrium:

$f_{cd} \cdot \alpha \cdot x \cdot 1m = \sigma_s \cdot A_{s.m} + (P_{ULS} \cdot \cos(\alpha_{strand}) + F_{dev.h.ULS}) \cdot 1m = \sigma_s \cdot A_{s.m} + P_{ULS} \cdot 1m$

$\varepsilon_{cu} = 3.5 \times 10^{-3}$ Maximum strain in concrete

$\alpha = 0.81$ Stress block factors

$\beta = 0.416$

$\sigma_s = f_{yd}$ Assuming that the reinforcement yields

Given

$f_{cd} \cdot \alpha \cdot x_6 \cdot 1m = f_{yd} \cdot A_{s.m} + P_{ULS} \cdot 1m$

$x_6 := \mathrm{Find}(x_6) = 21.455 \cdot mm$ Height of compressive zone

$\varepsilon_{s6} := \varepsilon_{cu} \cdot \frac{d_s - x_6}{x_6} = 0.018$ Strain in reinforcement

$\varepsilon_{s6} \geq \varepsilon_{sy} = 1$ Reinforcement is yielding as assumed

Moment equilibrium around tensile reinforcement:

$$m_{Rd.6} := f_{cd} \cdot \alpha \cdot x_6 \cdot (d_s - \beta \cdot x_6) - P_{ULS} \cdot (d_s - x_6) = 39.503 \cdot \frac{kN \cdot m}{m}$$

Check of moment resistance:

$$\frac{m_{Ed.6}}{m_{Rd.6}} = 0.978$$

Calculation of deflection in ULS

To be able to calculate m_{Ed} (see above), the deflection in mid-section must be calculated. It is assumed that the whole slab is in stadium II, which is a rather rough simplification.

$$x := 0 \cdot m, 0.1 \cdot m .. \frac{L}{2}$$ Vector dividing half of the span into segments

$$R_{A.6} := \frac{q_{d.6} \cdot 1m \cdot L}{2} = 36.45 \cdot kN$$ Support reaction

Moments in each section along half of the span:
Since the moment from the horizontal forces depend on the distance to the neutral layer, the deflection in each section must be regarded when the moment in that section is calculated. This is done by the vector f_{guess}, which is iterated.

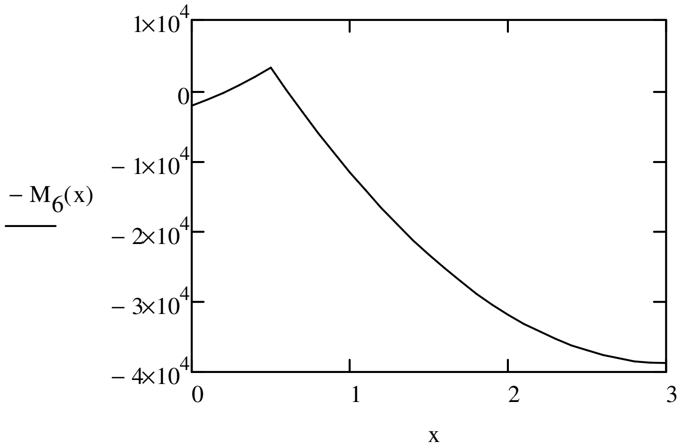
Equation valid to the left of the deviator:

$$M_{6.1}(x) := R_{A.6} \cdot x - P_{ULS} \cdot 1m \cdot \sin(\alpha_{strand}) \cdot x + P_{ULS} \cdot 1m \cdot \cos(\alpha_{strand}) \cdot \left(x_6 + f_{guess_{\frac{x}{0.1m}}} \right) \cdots$$
$$+ -F_{dev.h.ULS} \cdot 1m \cdot \left(h - x_6 - f_{guess_{\frac{x}{0.1m}}} \right) - \frac{q_{d.6} \cdot 1m \cdot x^2}{2}$$

Equation valid to the right of the deviator:

$$M_{6.2}(x) := R_{A.6} \cdot x - P_{ULS} \cdot 1m \cdot \sin(\alpha_{strand}) \cdot x + P_{ULS} \cdot 1m \cdot \cos(\alpha_{strand}) \cdot \left(x_6 + f_{guess_{\frac{x}{0.1m}}} \right) \cdots$$
$$+ F_{dev.v.ULS} \cdot 1m \cdot (x - L_1) - F_{dev.h.ULS} \cdot 1m \cdot \left(h - x_6 - f_{guess_{\frac{x}{0.1m}}} \right) - \frac{q_{d.6} \cdot 1m \cdot x^2}{2}$$

$$M_6(x) := \mathrm{if}(x < L_1, M_{6.1}(x), M_{6.2}(x))$$



$$\kappa_{appr}(x) := \frac{M_6(x)}{E_{cm} \cdot I_{II}}$$

Curvature along half of the span (assuming stadium II)

$\kappa_{appr}(x) =$

$5.322 \cdot 10^{-4}$	$\frac{1}{m}$
$3.22 \cdot 10^{-4}$	
$8.108 \cdot 10^{-5}$	
$-1.904 \cdot 10^{-4}$	
$-4.923 \cdot 10^{-4}$	
$-8.246 \cdot 10^{-4}$	
$-1.525 \cdot 10^{-5}$	
$7.599 \cdot 10^{-4}$	
$1.524 \cdot 10^{-3}$	
$2.219 \cdot 10^{-3}$	
$2.922 \cdot 10^{-3}$	
$3.555 \cdot 10^{-3}$	
$4.197 \cdot 10^{-3}$	
$4.77 \cdot 10^{-3}$	
$5.351 \cdot 10^{-3}$	
...	

Rotation at the support:

$$\theta_{A.appr} := \sum_{i=0}^{30} \left(\kappa_{appr_i} \cdot 0.1m \right) = 0.01588$$

Deflection in mid-span:

$$x_0 := \frac{L}{2} = 3\ m$$

Section where the maximum deflection is located

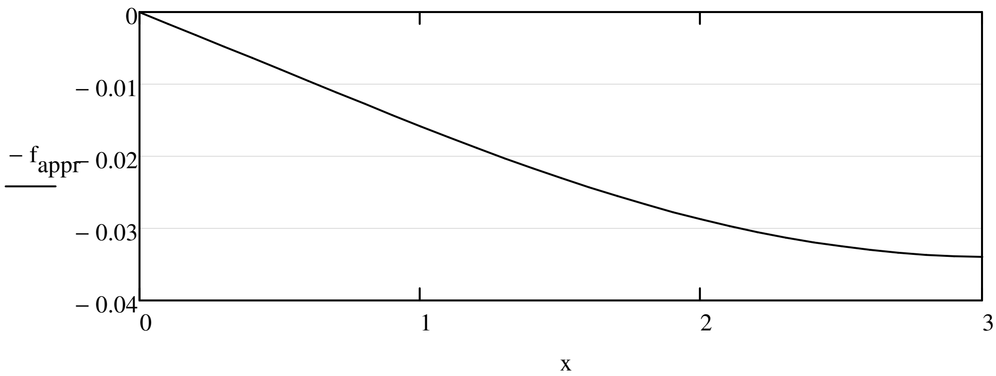
$$f_{max.appr} := \theta_{A.appr} \cdot x_0 - \sum_{i=0}^{30} \left[\kappa_{appr_i} \cdot 0.1m \cdot \left[x_0 - (i \cdot 0.1m) \right] \right] = 33.915 \cdot mm$$

Deflection in each section along half of the span:

$$f_{appr} := \text{for } i \in 0..30$$
$$f_{appr_i} \leftarrow \theta_{A.appr} \cdot x_i - \sum_{j=0}^{i} \left[\kappa_{appr_j} \cdot 0.1m \cdot \left[x_i - (j \cdot 0.1m) \right] \right]$$

$f_{appr} =$ (m)

	0
0	0
1	$1.582704 \cdot 10^{-3}$
2	$3.162189 \cdot 10^{-3}$
3	$4.740862 \cdot 10^{-3}$
4	$6.32144 \cdot 10^{-3}$
5	$7.90694 \cdot 10^{-3}$
6	$9.500687 \cdot 10^{-3}$
7	$1.109459 \cdot 10^{-2}$
8	$1.268089 \cdot 10^{-2}$
9	$1.425195 \cdot 10^{-2}$
10	$1.580082 \cdot 10^{-2}$
11	$1.732047 \cdot 10^{-2}$
12	$1.880457 \cdot 10^{-2}$
13	$2.02467 \cdot 10^{-2}$
14	$2.164113 \cdot 10^{-2}$
15	...



Deflection through half of the span

Calculation of the prestressing force in ULS after long time

The initial prestressing force must be reduced due to anchor slip, relaxation and creep in the concrete. The force is however increased again when the loads for the ultimate limit state are regarded.

Elongation of prestressing steel due to deflection

The strands are only connected to the slab at the anchors and deviators. By calculating the difference in deflection at the section of the deviator, the elongation of the strands can be calculated.

$$L_{strand.i} := 2\cdot\left[\sqrt{L_1^{\,2} + \left(h + f_{dev.i}\right)^2} + \left(L - 2L_1\right)\right] = 11.04999\,m$$

$$L_{strand.ULS} := 2\cdot\left[\sqrt{L_1^{\,2} + \left(h + f_{appr_5}\right)^2} + \left(L - 2L_1\right)\right] = 11.0549\,m$$

$$\Delta\varepsilon_{strand.ULS} := \frac{L_{strand.ULS} - L_{strand.i}}{L} = 8.152\times 10^{-4}$$

Reduction due to relaxation:

The relaxation of the strands is calculated according to Section 3.2 in Engström (2011).

$\chi_{1000} := 0.08$ Basic relaxation factor (ordinary prestressing steel, class 1)

$t_{inf} := 500000$ Time that, according to Eurocode 2, can be used to estimate the final relaxation (in hours)

$$\mu := \frac{\sigma_{pi}}{f_{puk}} = 0.739$$

$$\chi_{inf} := 5.39\cdot\chi_{1000}\cdot e^{6.7\cdot\mu}\cdot\left(\frac{t_{inf}}{1000}\right)^{0.75\cdot(1-\mu)}\cdot 10^{-3} = 0.206$$ Final relaxation

$$\sigma_{p.inf} := \sigma_{pi} - \chi_{inf}\cdot\sigma_{pi} = 1.091\times 10^3\cdot MPa$$

Reduction due to creep:

$$\varphi_{inf} = \varphi_{RH}\cdot\beta_{fcm}\cdot\beta_{t0}$$

$$RH := 50\%$$

$$h_0 := 2\cdot\frac{1m\cdot h}{1m + 1m} = 0.16\,m$$

$$f_{cm} := f_{ck} + 8MPa = 48\cdot MPa$$

$$\varphi_{RH} := \left[1 + \frac{1 - RH}{0.1\cdot\sqrt[3]{\frac{h_0}{mm}}}\cdot\left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.7}\right]\cdot\left(\frac{35}{\frac{f_{cm}}{MPa}}\right)^{0.2} = 1.632$$

$$\beta_{fcm} := 2.43$$

$t_0 := 40\cdot 365 = 1.46\times 10^4$ Age of the concrete at the time of strengthening (assumed to be 40 years)

$$\beta_{t0} := \frac{1}{0.1 + t_0^{0.2}} = 0.145$$

$$\varphi_{inf} := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = 0.574$$

$$\varepsilon_{c.creep} = \varphi_{inf} \cdot \frac{\sigma_{c.prestress}}{E_{cm}}$$

Simplifying by only taking the compressive part of the contribution from the prestressing effect, moment from loading is also ignored in this context. To compensate these simplifications, the higher initial prestressing force is used.

$$\sigma_{c.prestress} := \frac{\left(P_{h.i} + F_{dev.h.i}\right) \cdot 1m}{h \cdot 1.m} = 1.073 \cdot MPa$$

$$\varepsilon_{c.creep} := \varphi_{inf} \cdot \frac{\sigma_{c.prestress}}{E_{cm}} = 1.761 \times 10^{-5}$$

Reduction due to anchor slip:

It is assumed that an anchor slip of 1mm occurs when the hydraulic jack is removed.

$$\Delta s_{anchor} := 1mm$$

Prestressing force in ULS after long time:

$\sigma_{pi} = 1.374 \times 10^{3} \cdot MPa$ Initial prestressing

$$\Delta\sigma_p := -\chi_{inf} \cdot \sigma_{pi} - \varepsilon_{c.creep} \cdot E_p - \frac{\Delta s_{anchor}}{L_{strand.i}} \cdot E_p + \Delta\varepsilon_{strand.ULS} \cdot E_p = -148.125 \cdot MPa$$

$P_{m.ULS} := \left(\sigma_{pi} + \Delta\sigma_p\right) \cdot \frac{A_{pi}}{s_{strand}} = 153.224 \cdot \frac{kN}{m}$ Force from prestressing steel in ULS after long time (this force is used to calculate the utilisation of the moment capacity above)

Part 6 - Strengthening with post-tensioned steel strands

Part 7 - Strengthening with section enlargement on the compressive side

This time, the increased capacity is gained through an additional layer of concrete that is cast on top of the slab. To simplify the calculations, it is assumed that the same concrete strength is chosen for the added layer. It is also assumed that no stirrups or bolts are used to increase the interaction between the two layers.

Loads

$h_{new} := 105mm$ Height of new layer of concrete

$g_{new} := \frac{25kN}{m^3} \cdot h_{new} = 2.625 \cdot \frac{kN}{m^2}$ Self-weight of the new concrete layer

$q_{k.add.7} := 2.5 \frac{kN}{m^2}$ Additional distributed load on top of the slab

$q_{d7} := 1.35 \cdot (g_k + g_{new}) + 1.5 \cdot (q_k + q_{k.add.7}) = 15.694 \cdot \frac{kN}{m^2}$ New design value (ULS)

$m_{Ed.7} := \frac{q_{d7} \cdot L^2}{8} = 70.622 \cdot \frac{kN \cdot m}{m}$ New moment in mid-section

Shear resistance at the interface between concrete cast at different times

The shear resistance is according to CEN (2004), equation (6.25) defined as:

$$v_{Rdi} = \min\left[c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot (\mu \cdot \sin(\alpha) + \cos(\alpha)), 0.5 \cdot \nu \cdot f_{cd}\right]$$

By assuming that the top surface can be regarded as rough, i.e. that it has at least 3mm roughness at a spacing of about 40mm, the following values for cohesion and friction can be assumed:

$c := 0.45$ Cohesion between the surfaces

$\mu := 0.7$ Friction between surfaces

$f_{ctk.0.05} := 2.5MPa$

$f_{ctd} := \frac{f_{ctk.0.05}}{1.5} = 1.667 \cdot MPa$ Tensile strength of concrete

$\sigma_n := q_{d7} - 1.35 \cdot g_k = 10.294 \cdot \frac{kN}{m^2}$ Normal stress between the layers. Since the shear capacity is calculated for ULS, the design values of the self-weight and the variable load are taken.

$\rho := 0$ Since no steel passes through the interface, the term that includes ρ disappears.

$\nu := 0.6 \cdot \left(1 - \frac{\frac{f_{ck}}{MPa}}{250}\right) = 0.504$ Strength reduction factor

$$v_{Rdi} := \min(c \cdot f_{ctd} + \mu \cdot \sigma_n, 0.5 \cdot \nu \cdot f_{cd}) = 0.757 \cdot MPa$$

Shear effect at the interface between concrete cast at different times

According to CEN (2004), equation (6.24), the design value of the shear stress in the interface is:

$$v_{Edi} = \frac{\beta_{long} \cdot V_{Ed.h}}{z \cdot b_i}$$

Where β_{long} is the ratio between the longitudinal force in the new layer and the total longitudinal force in the compression (or tension) zone. $V_{Ed.h}$ is the shear force between the layers, z is the inner lever arm for the composite section and b_i is the width of the interface.

Shear force from bending:

$$\tau = \frac{S(z) \cdot V}{I \cdot b}$$ Shear due to load

$$V_{Ed} := q_{d7} \cdot 1m \cdot \frac{L}{2} = 47.081 \cdot kN$$ Vertical shear force near the support for the ultimate load

$$S_{new} := h_{new} \cdot 1m \cdot \left(\frac{h + h_{new}}{2} - \frac{h_{new}}{2} \right) = 8.4 \times 10^{-3} \cdot m^3$$ First moment of area

$$I_{tot} := \frac{1m \cdot (h + h_{new})^3}{12} = 1.551 \times 10^{-3} m^4$$

==> $$\tau_{Ed} := \frac{S_{new} \cdot V_{Ed}}{I_{tot} \cdot 1m} = 0.255 \cdot MPa$$ Shear in the interface between the two layers from the load (at the support section)

Since τ_{Ed} is the shear stress in the interface, the equation for v_{Edi} can be rewritten:

$$v_{Edi} = \beta_{long} \cdot \tau_{Ed}$$

Since the critical section is at the support (where the longitudinal force is zero), the value for β_{long} becomes 1.0

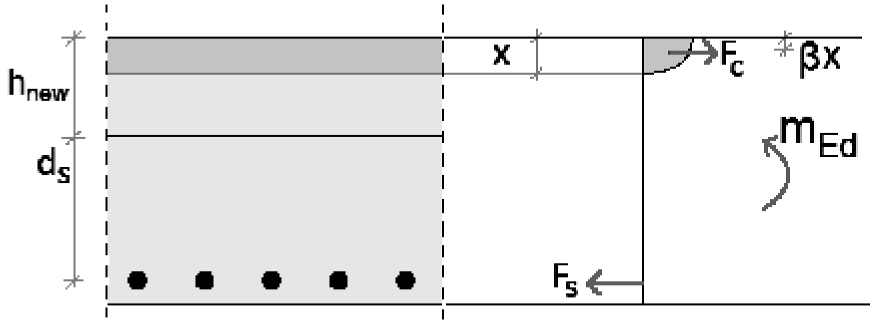
$$v_{Edi} := 1.0 \cdot \tau_{Ed} = 0.255 \cdot MPa$$

$$\frac{v_{Edi}}{v_{Rdi}} = 0.337$$ Utilisation of the shear in the interface

Since the interface between the layers can transfer the shear force, bending interaction can be accounted for.

Calculation of resistance in ULS

If it can be assured that the shear in the interface is small enough, the resistance of the slab can be calculated with interaction between the two layers. There will however be a strain difference between new and old concrete, but this aspect can be disregarded for calculations ir ULS if it can be shown that the compressive zone fits within the height of the new layer.



$\varepsilon_{c.new.top} := \varepsilon_{cu}$

$F_{s.7} := f_{yd} \cdot A_{s.m} = 310.434 \cdot kN$ — Force from reinforcement, assuming that the reinforcement yields.

$F_{c.new} := F_{s.7} = 310.434 \cdot kN$ — Force from compressed concrete in new layer (should be equal to the force from the reinforcement due to horizontal equilibrium)

$$x_7 := \frac{F_{c.new}}{\alpha \cdot f_{cd} \cdot 1m} = 14.372 \cdot mm$$

$$\varepsilon_{s.7} := \varepsilon_{cu} \cdot \frac{d_s + h_{new} - x_7}{x_7} = 0.0537$$

Strain in reinforcement

$$\frac{\varepsilon_{s.7}}{\varepsilon_{sy}} = 2472 \cdot \%$$

The reinforcement is yielding, OK

To calculate $\varepsilon_{s.7}$ from ε_{cu} and x_7 is a simplification since the strain difference between the layers is neglected. However, since the strain in the steel is well above the yield limit, the simplification is valid.

$$m_{Rd.7} := \frac{F_{c.new}}{1m} \cdot \left(d_s + h_{new} - \beta \cdot x_7\right) = 71.096 \cdot \frac{kN \cdot m}{m}$$

$$m_{Ed.7} = 70.622 \cdot \frac{kN \cdot m}{m}$$

$$\frac{m_{Ed.7}}{m_{Rd.7}} = 0.993$$

Utilisation of moment capacity

Part 7 - Strengthening with section enlargement on the compressive side