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DTU Civil Engineering
Department of Civil Engineering

Study of Concrete Structures in Older Residential Buildings for Renovation Purposes

A Case Study Concerning Multistory Buildings in Greenland

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Abstract

In this thesis, renovation possibilities of the concrete structure in older Greenlandic multistory buildings are presented. Renovation proposals are investigated based on a case study of two buildings in Sisimiut, that have undergone a condition assessment and a structural analysis respectively.

The external concrete elements of one block has been analysed by a visual registration of the damages and based on this analysis, representative elements have been chosen for further testing. The results show that the condition based on a visual inspection of an element corresponds well with the actual condition determined by testing. This indicates that it is possible to perform an overall condition assessment of a building purely from a visual registration, which is less comprehensive than performing tests on all elements. Consequently, it can be assumed that the reinforcement will be intact in elements with a good visual appearance, on the other hand, the reinforcement will have corroded in elements with a poor visual appearance. The extend of corrosion of the reinforcement will be unknown from a visual registration, and therefore this parameter should be determined by tests.

An analysis of the bearing capacity of certain structural elements have been conducted. The consoles have been analysed in terms of changing the current location of the facade and balconies. The effect of corroded reinforcement in the consoles has also been investigated. Results show, depending on the amount of corroded reinforcement, that the consoles have sufficient capacity to allow a new facade to be moved to the outside of the consoles and built-in balconies can be constructed. If a more extensive renovation design is wanted the consoles can be strengthened with a tension rod and thereby external balconies can be placed on the outside of the new facade. The bearing walls have been analysed in relation to creating new openings and adding an extra floor to the building. In both cases the bearing walls of the building have the sufficient capacity and the changes barely affect the current stresses in the walls.

It can be concluded that the concrete structure has a high potential for undergoing an extensive renovation. The results can be applied to other buildings of the same type and the best renovation method should be chosen based on a condition assessment of that specific building.

Preface

This master's thesis was prepared at the department of Civil Engineering at the Technical University of Denmark in fulfillment of the requirements for acquiring a Master of Science degree in Civil Engineering.

The master's thesis includes a project report and an appendix. The appendix contains photo material and data from a field testing in Sisimiut as well as detailed calculations and results that have not been included in the report. The original drawings and a digital model will also be included as an appendix. All photos and illustrations are produced by the author, unless other is stated.

The thesis is based on Laursen's thesis template and it should be read in a two page view. Due to the photos, illustrations, and graphical results it is recommended to read the thesis in colors.

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CHAPTER 1

Introduction

1.1 Background

In the 1960s' and 1970s' multistory concrete housing was build in several cities along the west coast of Greenland. In general the buildings have not been maintained and therefore the condition of the buildings is very poor today. According to the Greenlandic sector plans from 2016 many buildings of this type have a remaining life span of less than 20 years. *The White Blocks* in Sisimiut will, according to the sector plans, be demolished within 0-5 years and *Sletten* in Nuuk is planned to be demolished within 10-20 year from year 2016. A few buildings have been renovated and the cost have turned out to be more expensive than what was first anticipated. It is therefore assumed by the Government of Greenlandic that it is cheaper to demolish the buildings and construct new buildings compared to renovating the existing buildings.

There is an extensive lack of housing in the major cities in Greenland and it is therefore important to keep the existing buildings to combat the housing shortage. If the concrete blocks are demolished or renovated, rehousing will be needed and therefore the construction time must be short. It is anticipated that it will be less time consuming to renovate an existing building compared to constructing a new building. This is partly due to the fact that the bearing structure of the building already exists and the infrastructure, sewerage and installations are already established around the building. The concrete buildings are all build from a standard design, only varying in height and length. Therefore the renovation method can be standardised for all the buildings, which in the end will reduce the renovation costs.

Renovating the existing buildings will be a more sustainable choice compared to constructing new buildings and a life cycle analysis that confirms this statement has been made [1]. Transportation of new building materials to Greenland will be costly both economically and for the environment. By reusing the concrete structure only the non-structural components of the building will be new. Usually a building is constructed to last for +50 years according to the building regulations, however in order for a building to be sustainable the life span of a building should be increased to at least 100 years considering ongoing maintenance. The total life span of the buildings is currently 45-70 years, which means that a renovation of the buildings must be sufficient enough to let the buildings last for at least another 50 years in order to meet the sustainability demands.

This thesis is a part of the Arctic Building and Construction (ABC) research project that investigates construction in Greenland and the Arctic from a holistic point of view, including the possibilities of recycling the concrete construction of multistory buildings. The focus in this thesis will only be the load bearing concrete structure, which is the main element of a building to be reused in a renovation. Other topics such as the thermal envelope, life cycle analysis, environmentally hazardous materials, and renovation costs has been investigated elsewhere. The aim of the research project is to document the benefits of renovating the concrete buildings and hopefully avoid their planned demolitions.

1.2 Project Outline

The title and the problem statement for this master thesis is defined as follows:

Title:	Study of concrete structures in older residential buildings for renovation purposes
Problem statement:	The project concerns residential concrete housing in Greenland, constructed in the 1960s and 1970s. The load bearing concrete structure will be analyzed with the aim to evaluate whether the building is suitable for renovation. Damages in the structure will be analyzed and assessed, where after proposals for repairs will be made. The load bearing capacity of the structure will be calculated. It will be investigated how much of the structure can be removed in order to create alternative room divisions, and whether the structure will have the necessary capacity to obtain additional structural components, floors etc. A proposal for a comprehensive solution will be outlined as to how a concrete building can be renovated with emphasis on the structure.

The project is a Master Thesis in Civil Engineering at DTU and the work is equal to 32.5 ECTS point.

1.3 Scope

The scope of the thesis is to investigate renovation possibilities of the concrete structure. In order to limit the scope, specific elements of the concrete structure have been chosen for in depth analysis, since these elements will be essential during a renovation and investigating every detail in the concrete structure will be too extensive. Furthermore the analysis is a case study which means that not every building design will be considered, however the buildings are very similar and it is the intention that the results can be applied to all buildings. The renovation proposals will be suggested as concepts and detailed solutions will not be provided. The foundation of the buildings will not be investigated in this project since the bearing capacity of the foundation depends on the ground below a specific site. It is therefore assumed in this project, that the foundation of a building has the sufficient capacity needed.

The project consists of two main elements, namely a condition assessment and a structural analysis of the concrete blocks. A representative building in Sisimiut has been investigated in order to determine the current condition of the concrete structure. The condition assessment is limited to the concrete structure on the outside of the building envelope. The condition assessment will form the basis of the structural analysis since it will provide information about the condition of the concrete elements. A structural analysis will be performed for certain chosen elements of the building in order to investigate renovation proposals. The structural analysis will be based on the original drawing material of the building and the scenario of concrete elements being in a good condition will be investigated as well as the effect of corroded reinforcement.

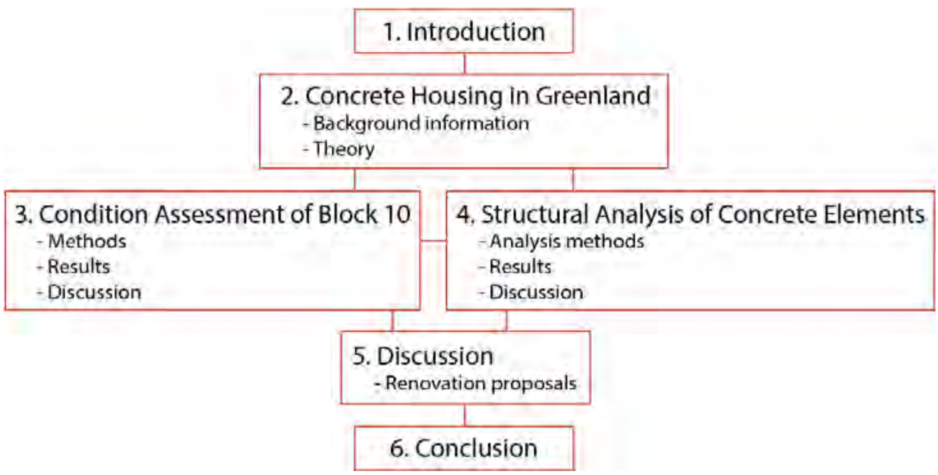


Figure 1.1: Reading guide.

The structure of the thesis illustrated in Figure 1.1 and it can be used as a reading guide. The project begins with an introduction describing the project formalities. Then a chapter containing background information and theory regarding concrete housing in Greenland will follow in order to give the reader an overview of the case study. The project contains two main elements which have been analysed and therefore each element will be described separately. First a chapter about the condition assessment will be presented containing several sections describing the methods used followed by the results and a discussion. Secondly a chapter regarding the bearing capacity of the concrete structure will be presented. Just as the previous chapter this chapter will consist of several sections describing the analysis methods and results from the analysis of the bearing capacity as well as a discussion. The condition assessment creates the foundation of the structural analysis and therefore this order is chosen. A discussion will then combine these two chapters into a final discussion where renovation proposals will be presented. Lastly the main points of this thesis will be summarised in a conclusion.

CHAPTER 2

Concrete Housing in Greenland

2.1 History

In the early days until the middle of the 20th century peat houses were the main buildings in Greenland according to the Inuit and Norse building tradition. When the colonization of Greenland started in 1700 the Scandinavian building tradition was introduced. Stone houses where built with local materials and timber was imported in order to built wooden houses. After the 2nd World War there was a big need of more housing in the cities due to the industrialisation of the fishing industry. In order to secure quality housing a few standardised buildings were designed by GTO (Grønlands Tekniske Organisation) that was responsible for the construction of all new buildings [2]. A new building tradition was therefore introduced in the 1960s with townhouses and multistory buildings made of concrete. The buildings investigated in this project are one of the standard GTO buildings designed in this time. The most famous building is block P in Nuuk, that was the largest residential building in Greenland, built in 1965 and demolished in 2012, see Figure 2.1.

Figure 2.1: The iconic block P in Nuuk [3].

The new buildings increased the health of the residents since sewage and running water was installed. At the same time the living quality was decreased for people living in multistory buildings since the residents did not have access to an outdoor area anymore. Hunting and fishing is very big part of the Greenlandic tradition and this requires having an outdoor area for sled dogs and drying fish. The new types of buildings did therefore not only change the living ways of the Greenlandic people, but it also separated them from their traditions.

Most of the multistory concrete buildings have not been properly maintained since they were built and therefore the living conditions today are very poor. The amount of uninhabitable apartments is rising, which creates a negative spiral since the income from the apartments will decrease and thus maintenance can not be afforded. The buildings are commonly known as *The Blocks* and the residents are socioeconomically mixed. However many residents, depending on the geographical location, belong to the lower classes of the society and therefore it is important that the buildings will be renovated in order to increase the living conditions and thereby the life quality of the residents.

According to the sector plans, published by The Government of Greenland in 2016, 42 of the considered blocks will be demolished within 16 years from now. A list showing the amount of buildings and the time for demolition in different cities is given in Table 2.1. Many buildings have already been demolished including Block P in Nuuk and the buildings used for the case study in this project. It seems unlikely that new affordable housing will be build in the next 16 years, that will be able to accommodate all the residents of the existing blocks, when the population rise in the cities is also taken into account.

Table 2.1: Amount of buildings and time for demolition according to the sector plans from 2016 [4].

City	0-5 years	5-10 year	10-20 years	Total
Qaqortoq	2	2	-	4
Paamiut	7	-	-	7
Nuuk	1	-	10	11
Maniitsoq	4	-	1	5
Sisimiut	6	-	7	13
Ilulissat	-	-	2	2
Total	20	2	20	42

2.2 Design of Concrete Apartment Blocks

The concrete blocks investigated in this project consist of apartments with balconies on each side of the building spanning the whole length of the facade. On one side of the building the balconies are connected to staircases and they thereby form the access ways to the apartments. The balconies found on the other side of building are narrow private balconies, which can be seen in Figure 2.2, and these balconies are mainly used for storage and laundry. The function of the ground levels of the buildings vary and they can used for apartments, storage, laundry and so on. The remaining levels of the buildings only consist of apartments.

Figure 2.2: A typical design of the concrete blocks.

The amount of floors vary between three and six floors. The lengths of the blocks also vary, where the longest building was Block P. Despite the varying sizes of the blocks they are all built according to the same standard design provided by GTO. The construction is made of reinforced concrete walls and slabs. Typically the structures are in-situ cast, however a few blocks in Nuuk are made from pre-fabricated elements, that were produced at a local concrete element factory in Nuuk [1]. The bearing walls are placed in order to create 2.7m and 3.6m wide modules in the building. These modules form the basis of the apartments that can either consist of 1, 2 or 3 modules.

Some blocks in Nuuk and Sisimiut have been renovated and the renovations have mainly been conducted for the exterior of the buildings, by altering the balconies and facade, and some blocks have been renovated internally. The renovation costs have been larger than first anticipated and therefore renovations of more blocks have stopped. The reason for the larger costs could be that Greenland has a lack of experience in regards to renovations. It is known that currently the balcony railings on blocks in Nuuk are being maintained due to the danger of concrete breaking off and a block in Sisimiut is being internally renovated due to mold. Despite the renovations the demolition of all the mentioned buildings is planned.

The renovated blocks in Sisimiut have been designed in many different colors, just as other traditional colorful houses. The colors create a more interesting area and the buildings become more personal due to the different colors, see Figure 2.3.

Figure 2.3: Renovated blocks in Sisimiut.

In addition to the changing colors the balcony design of the renovated blocks are also varying. In some blocks rounded steel balconies have been installed, which creates a larger balcony area, see Figure 2.4. A more extensive renovation has also been performed for some blocks where columns have been built in order to support larger asymmetrical concrete balconies, see Figure 2.5. The use of wood for the balcony railings creates a more warm facade and there is a good dynamic between the wood, concrete and steel materials.

Figure 2.4: Rounded steel balconies.

Figure 2.5: Asymmetrical concrete balconies.

For all buildings, both the renovated and non-renovated buildings, the consoles underneath the balconies are placed on the outside of the facade and therefore thermal bridges around the consoles are created. This has a negative effect on the indoor climate as well as the energy consumption of the building and it is therefore wanted to solve the issue in a thorough renovation. This problem has been investigated in the master's thesis *Renovation of Greenlandic apartment blocks constructed in the 1960s and 1970s, with a focus on improving the thermal envelope* [5]. The thesis implies that a solution would be to move the facade of the building to the outside of the consoles and thereby include them in the indoor environment. If the facade is moved to the outside of the consoles the living area will increase since the balcony area has been included. To compensate for this other types of balconies will be installed. Due to a lack of insulation, the bottom level of the buildings is unsuitable as a living area and therefore the building envelope of the building can be placed between the bottom floor and the 1st floor. The bottom floor will in the future be used as a common area where meat from hunting can be cut, equipment can be stored and it can be used as a workshop.

In the city of Herning concrete blocks in the area Brændegårdsparken, have undergone a renovation which has completely changed the appearance of the buildings. The buildings are interesting since the static system of the buildings is similar to the one found in the Greenlandic blocks. The monotonous look of the existing blocks has been altered by a new facade and balcony design. Figure 2.6 and 2.7 shows the blocks before and after the renovation. A dynamic facade is made with both asymmetrical and built-in balconies which has also resulted in more functional balconies.

Figure 2.6: Brændegårdsparken before the renovation [6].

Figure 2.7: Brændegårdsparken after the renovation [7].

A renovation proposal for the area of Blokland in Albertslund has been made, however the renovation is not yet completed [8]. The area consist of many concrete blocks and the appearance, as well as the condition of the blocks, are very similar to the concrete blocks in Greenland, see Figure 2.8. As a renovation proposal it is suggested that the facade of a block will vary in color and texture as in Figure 2.9. Some blocks also have a varying height and some have been divided into two blocks in order to create better urban areas around the buildings.

Figure 2.8: Blokland before the renovation [8].

Figure 2.9: Renovation proposal of Blokland [8].

2.3 Material Properties

The blocks were built from the mid 1960s through to the mid of the 1970s. During this time the valid building standards were *DS411:1949 Beton- og jernbetonkonstruktioner* (Concrete and Reinforced Concrete Structures) and *DS410:1945 Belastningsforskrifter* (Load Requirements)[9]. According to the standards partial coefficients were not added to the loads, but only to the strength of the material. This meant that the design strength of the material would, in general, be smaller in order to take the load uncertainties into account.

In this thesis the rules according to the Eurocodes will be applied and therefore a correlation of the material strengths between the old standards and the Eurocodes will be determined. It is very important that correct design strengths are found since they create the basis for all calculations. If the strengths are not correct the results of the calculations will be invalid. The strength conversion will be based on three different papers defined below:

- Building regulation of Greenland, 1982 [10]
- Regulation for concrete structures in Greenland, 1996 [11]
- Handbook for bridges, 2017 [12]

According to the *Special Work Description for the Execution of all Works* [13], regarding the construction of building no. 10, 11, and 12 in Sisimiut, the concrete used is defined by:

- Concrete Type 200
- Rapid cement content min. 300kg/m^3
- Air content 3-4%

- Slump 3-6cm
- Cylinder strength 200kg/cm^2 = cube strength 240kg/cm^2 = beam fracture strength 300 kg/cm^2

The concrete definition correspond to concrete type 1:2:3 used for reinforced concrete. The characteristic concrete strengths for this concrete type is given in Table 2.2 according to the three different papers.

Table 2.2: Characteristic strengths of concrete from different sources.

Building regulation of Greenland, 1982:	$f_{ck} = 150\text{kg/cm}^2 = 14.7\text{MPa}$
Regulation for concrete structures, 1996:	$f_{ck} = 15\text{MPa}$
Handbook for bridges, 2017:	$f_{ck} = 0.8 \cdot 0.8 \cdot \sigma_T = 15.1\text{MPa}$

Where the cube strength σ_T of the concrete in MPa is defined by:

$$\sigma_T = 240\text{kg/cm}^2 \cdot 9.81\text{m/s}^2 = 23.52\text{MPa} \tag{2.1}$$

From Table 2.2 it can be seen that the characteristic concrete strengths vary a little, however if they are rounded to the nearest whole number the characteristic concrete strength will be f_{ck}=15MPa in all three cases. This is a very low characteristic concrete strength compared to the concrete strengths used today, especially for concrete used in structures that are exposed to a harsh climate.

According to the *Handbook for bridges*, the characteristic concrete strength can be increased by 25% for concrete structures built before 1990. This is because the chemical reactions in the concrete continue for many years, which means that the concrete keeps on becoming stronger as time passes. The extra characteristic concrete strength will not be considered unless it is needed in the calculations. The concrete properties are summarized in Table 2.3.

Table 2.3: Material properties of concrete.

Characteristic concrete strength:	$f_{ck} = 15\text{MPa}$
Partial coefficient, concrete:	$\gamma_c = 1.45$
Design concrete strength:	$f_{cd} = f_{ck}/\gamma_c = 10.3\text{MPa}$
Concrete strength, 25% extra capacity:	$f_{cd,125\%} = f_{cd} \cdot 1.25 = 12.9\text{MPa}$
Characteristic concrete tensile strength:	$f_{ctk,0.05} = 0.7 \cdot 0.3 \cdot f_{ck}^{2/3} = 1.28\text{MPa}$

The reinforcement used in the buildings is either normal round rebars or tentor steel. Round rebars can easily be bend and they are e.g. used in the consoles, whereas tentor steel is used as straight reinforcement in e.g. the walls. The round rebars have a smooth surface and they therefore do not anchor as well with the concrete compared to tentor steel. In the project round rebars are referred to as R and tentor steel is defined by T. The strength of both types of steel is identical and it only varies with the diameter of the reinforcement. The reinforcement properties are given in Table 2.4 according to the *Handbook for bridges*.

Table 2.4: Material properties of reinforcement.

Characteristic steel strength st.37, $d \leq$16mm:	$f_{yk,d\leq 16\text{mm}} = 235\text{MPa}$
Characteristic steel strength st.37, $d \geq$16mm:	$f_{yk,d\geq 16\text{mm}} = 225\text{MPa}$
Partial coefficient, steel:	$\gamma_s = 1.2$
Design steel strength st.37, $d \leq$16mm:	$f_{yd,d\leq 16\text{mm}} = f_{yk}/\gamma_s = 195.8\text{MPa}$
Design steel strength st.37, $d \geq$16mm:	$f_{yd,d\geq 16\text{mm}} = f_{yk}/\gamma_s = 187.5\text{MPa}$
Modulus of elasticity:	$E_s = 200 \cdot 10^3\text{MPa}$

2.3.1 Blasting Stones Used as Aggregates

When in-situ casting a building in Greenland the concrete is made with local aggregates. The rock aggregates are produced by blasting mountains and then crushing the rock into smaller stones. Greenlandic blasting stones are made of granite or gneiss, which are the same types of rock found on the danish island Bornholm. The quality of the rock in Greenland compared to Bornholm is unknown, but typically blasting stones are of a better quality compared to sea stones or field stones. The shape of blasting stones are angular compared to sea stones which are rounded. Concrete made with rounded stones will be easier to cast and a reduced amount of cement is needed due to a smaller surface area compared to concrete made with angular stones [14]. Therefore using blasting stones can have an influence on the concrete properties, e.g. the flow ability might be lower and the concrete might not be evenly distributed around the reinforcement. If a concrete with blasting stones is mixed properly and cast correctly then the concrete produced will be strong and of a very good quality.

2.4 Damages of Reinforced Concrete in Greenland

In this section typical damages of reinforced concrete will be presented in relation to the Greenlandic climate.

2.4.1 Climate

The climate in Greenland plays an important role regarding the damages that occur on concrete structures. Greenland is placed in an arctic climate where the temperature on average does not exceed 10°C and the average temperature is below 0°C for more than six months of the year. The humidity is around 70-80% in Greenland throughout the whole year. Graphs of the annual temperature and humidity in Sisimiut is shown in Figure 2.10 and 2.11 for year 2019 [15].

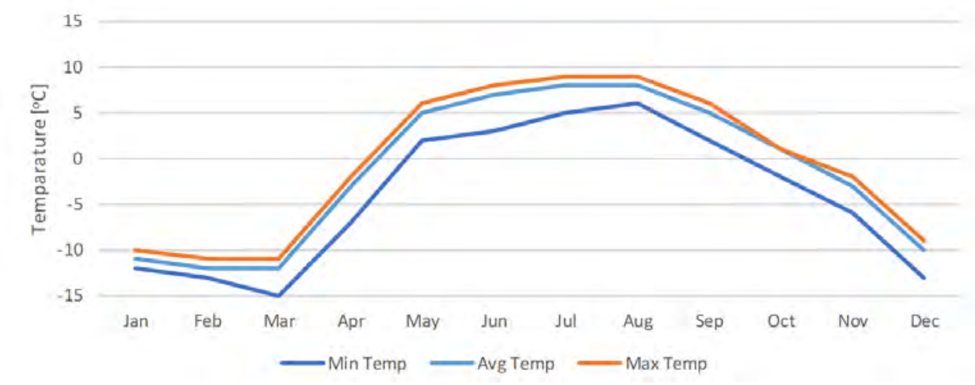


Figure 2.10: Temperature data for Sisimiut in 2019.

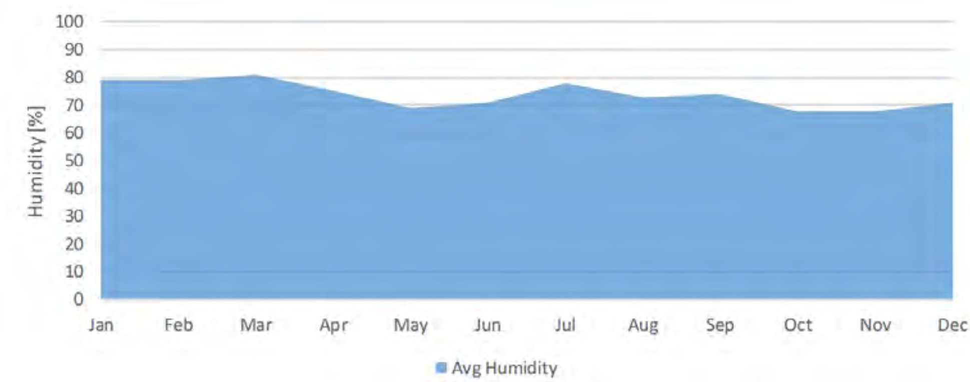


Figure 2.11: Average humidity data for Sisimiut in 2019.

2.4.2 Carbonation

Uncarbonated concrete has a high pH value, usually pH 12-14, and it is therefore very alkaline. When the cement in concrete comes into contact with CO_2 a reaction will take place that results in calcium and water.

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

The reaction causes the pH value of the concrete to drop almost to a neutral pH 7-9. This reaction creates the basis for what is known as carbonation.

Reinforcement that is cast in concrete is protected against corrosion when the pH value of the concrete is very high. When carbonation occurs and the concrete pH becomes neutral the protective properties of the concrete ceases and therefore corrosion of the reinforcement can occur. Carbonation first occurs at the surface of the concrete and then slowly the carbonation depth will continue to increase as long as the concrete surface is in contact with atmospheric air. Typical signs of carbonation damage are large cracks and spalling of concrete, since the reinforcement volume expands when corrosion occurs [16]. Figure 2.12 shows a carbonation damage where the reinforcement has expanded due to corrosion and thereby the concrete cover layer has broken off.

Figure 2.12: Typical damage due to carbonation of the concrete.

The carbonation reaction can only take place when water is present in the concrete since water is needed in order to transport CO_2 into the concrete. Carbonation happens most rapidly when the relative humidity is 40-70% and if the concrete is either completely wet or dry carbonation does almost not occur [16]. Therefore carbonation is not a problem in indoor environments where the air usually is warm and dry. For concrete placed outside the carbonation rate will be faster in concrete that is covered against direct rain compared to concrete subjected to direct rain. The reason for this is that the covered concrete is always subjected to a constant humidity and the carbonation depth will steadily increase. Concrete subjected to direct rain will most of the time be very wet and carbonation will therefore not occur until the concrete has dried, which means that the carbonation conditions are only optimal for a short period of time before the next rainfall will occur. The relative humidity also depends on the air temperature and the carbonation rate is slowed down for low temperatures. If the water is frozen the carbonation reaction will basically stop and water inside concrete usually freezes around a temperature of -4°C.

Besides the relative humidity and the temperature another very important parameter in relation to carbonation is the compaction degree of the concrete. For compact and dense concrete the carbonation rate will be very small since the transportation of CO_2 into the concrete is prevented. A high water/cement ratio will cause a faster carbonation, since CO_2 travels more easily in fluids, and therefore a w/c ratio of less than 55% is desired [16]. The last important factor is the cover layer thickness of the concrete and this layer should be thicker than the carbonation depth in order for the reinforcement not to corrode. The carbonation process is irreversible, but if the process is stopped, e.g. with special paint that protects the surface, then re-alkalizing can occur. This means that the pH value around the reinforcement can increase and thereby the reinforcement can again be passivated.

2.4.3 Freeze-Thaw

If concrete structures are exposed to cycles of freezing and thawing then damages can occur over time. The damages occur when water inside the concrete freezes, since it expands about 9%, and thereby a pressure larger than the tensile strength of the concrete can be produced. Freeze-thaw damage results in scaling, crumbling and cracking of the concrete surface. In relation to this the concrete cover layer will be reduced and thereby the reinforcement is more exposed. Freeze-thaw damages are usually avoided if the water/cement ratio is low and air entrainment is used [17]. An example of a freeze-thaw damage can be seen in Figure 2.13 where the concrete surface is crumbling.

Figure 2.13: Typical damage due to freeze-thaw cycles.

2.4.4 Chloride Attack

Chloride attack occurs when concrete is in touch with de-icing salt or seawater, which will cause corrosion of the reinforcement. For the specific buildings that are investigated in this project chloride attacks are not relevant, since de-icing salt is not used in Greenland and the buildings are not in touch with seawater.

2.4.5 Corrosion Rate

The amount of corroded reinforcement depends on the surrounding climate and the time that the reinforcement has been exposed to conditions where corrosion can occur. As previously described the reinforcement, in reinforced concrete, is not protected against corrosion when the surrounding concrete is carbonated. The corrosion rate is a very uncertain parameter to determine since it depends on the relative humidity, temperature, pollution and weather conditions such as sun and rain. It has not been possible to find information about a corrosion rate in Greenland, however a corrosion rate will still be roughly estimated. A correct corrosion rate for Greenland can only be obtained by measurements.

From *DS/EN ISO 12944-2* [18] the atmospheric corrosivity category in Greenland is determined to be C3 with medium corrosivity, which corresponds to an urban and/or industrial area with moderate pollution and low salinity. According to this standard the corrosion rate for C3 is >25-50μm/year, meaning that 25-50μm depth of a steel surface will corrode every year. For cold climates the corrosion rate will be lower compared to a temperate climate and corrosion barely takes place for temperatures less than 0°C. Water in concrete freezes around -4°C and therefore it is assumed that the corrosion will almost stop below this temperature. A typical temperature graph for Greenland will show that for approximately six months of the year the average temperature is below -4°C, which suggests that corrosion only occurs during half of the year. Therefore in theory the corrosion rate should be divided by two resulting in 12.5-25 μm/year.

An article about corrosion of metals in cold and very cold climates describes corrosion rates in many locations in Russia and Antarctica [19]. Tests performed in a subarctic climate in Russia near the border to Norway results in corrosion rates varying from 8-75μm/year. The corrosion rates measured in Antarctica, where the climate is very cold, are all below <9μm/year. Greenland is, like Antarctica, placed in an arctic climate, but the temperatures in Greenland are however more similar to the temperatures found in the subarctic climate in Russia. Therefore an average value of the two maximum corrosion rates found for Antarctica and Russia is determined to be 42μm/year.

Two approximate corrosion rates have been estimated. The first corrosion rate is estimated from *DS/EN ISO 12944-2* to be 12.5-25 μm/year where the low temperatures in Greenland has been considered. The second corrosion rate is estimated from an article concerning corrosion in cold climates, however the Greenlandic climate has not been investigated in this article, and therefore a corrosion rate based on the average rate in Antarctica and Russia is found to be $42\mu m/year$. A rough estimate of the corrosion rate in Greenland is determined as the average value of the two maximum corrosion rates resulting in $34\mu m/year$. This value is a rough estimate and it is not validated, therefore measurements of the corrosion rate in Greenland should be performed to make a better estimate of the value. The value, however, can give an idea of how much steel has corroded which would otherwise be very hard to estimate. E.g. if steel has corroded for 20 years the amount of steel that has corroded is 0.6mm. If a reinforcement bar is considered the diameter of the bar would be reduced by 2x0.6mm=1.2mm, which results in quite a significant area especially for reinforcement bars with small diameters. When steel corrodes it can expand up to 7 times, meaning that 1.2mm steel can expand to a width of 10.1mm and this definitely will cause damage to the surrounding concrete.

CHAPTER 3

Condition Assessment of Block 10

The White Blocks in Sisimiut currently consist of 13 blocks, where 7 blocks have been lightly renovated and 6 blocks have never been renovated. One of the non-renovated blocks, namely block 10 (B-891), is planned to be demolished from November 2019 to September 2020. The remaining 5 non-renovated blocks are scheduled for demolition in the near future. Block 10 was empty in October 2019 and therefore it was ideal to perform a condition assessment of this block since it was possible to access the whole building without bothering the residents. Figure 3.1 shows an image of block 10 after it was abandoned. The interior parts of the apartments that could be used elsewhere had been removed and, as it can be seen from the image, some windows and doors have been covered with plywood.

Figure 3.1: South facade of block 10 in Sisimiut.

The main focus of a condition assessment is the outdoor area of the building, because it is assumed that the concrete inside the existing building envelope is in good condition. The external concrete structure consists of consoles, balcony plates and the gable walls. The most interesting elements to examine are the consoles which are cast together with the internal bearing walls. The consoles are therefore more important to retain for renovation purposes than the easily replaceable balcony plates. The gable walls have always been covered by insulation whereas the gable columns have been exposed to the outdoor climate. It is therefore very relevant to check the condition of the columns since they are stabilizing the gable walls.

3.1 Method for Visual Registration

In the visual registration of block 10 all damages on consoles, balcony plates and gables were registered. This was done in order to get an overview of the damages and to see whether there was any general tendency related to the damages. The tendency could either be the placement of the damage or the type of damage. A brief visual registration was also performed for block 7 (B-914) in Sisimiut to compare the relative degradation of the two buildings.

The condition of the concrete elements was documented, which resulted in more than 1000 pictures. The pictures were analysed and the damages found were recorded for each element. The elements were then divided into three damage categories, depending on the severeness of the damages, in order to get a simplified overview of the overall condition of the building. The elements investigated are the bottom part of consoles, the top part of consoles, and balcony plates, see Figure 3.2.

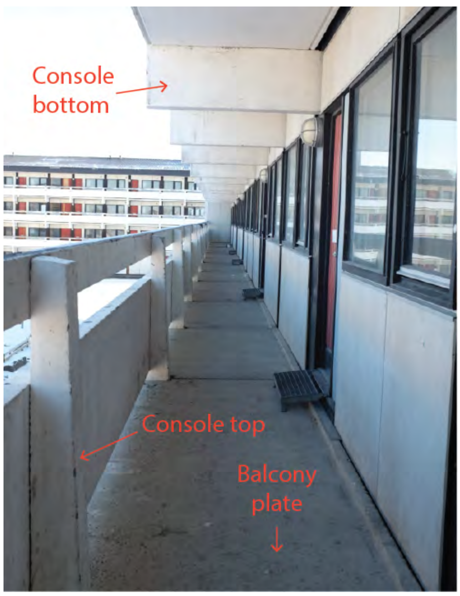


Figure 3.2: Definition of investigated elements.

Block 10 consists of four floors which are named level 0-3. The block contains 14 transverse walls in total and the gable walls are placed towards west and east. On the north facade 12 long consoles can be found and 12 short consoles are placed on the south facade. Balcony plates are placed on top of the consoles in level 1-3. Figure 3.3 and 3.4 show the elevation and plan drawing of the building, where the grid lines 1-14 and A-E are illustrated. In the remaining part of this chapter the placement of the elements will refer to these drawings.

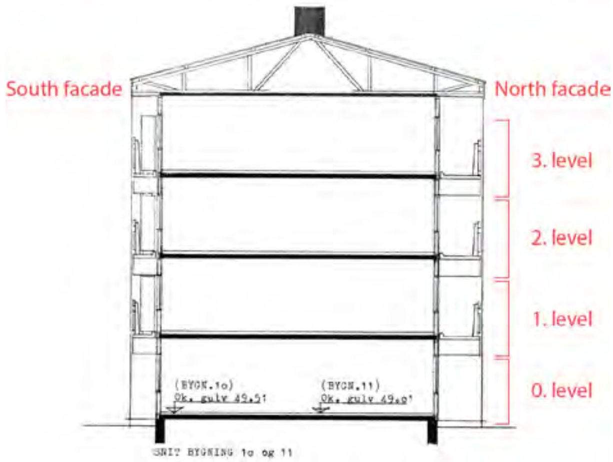


Figure 3.3: Elevation drawing of block 10.

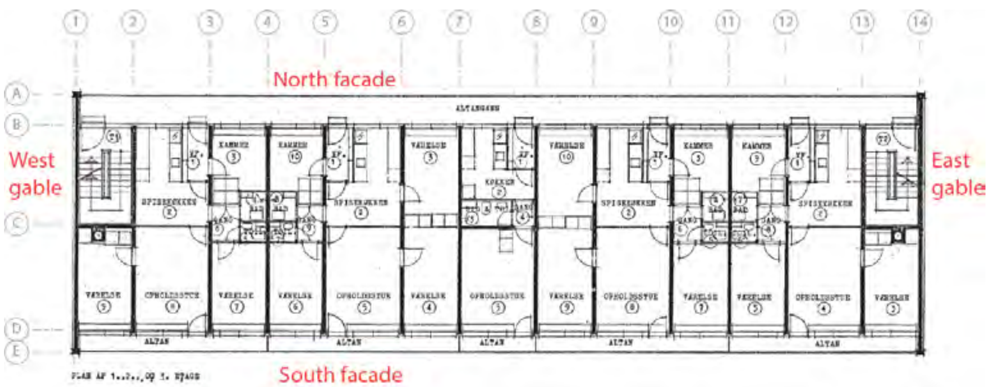


Figure 3.4: Plan drawing of block 10.

3.2 Method for Field Testing

From the visual registration concrete elements in a good and bad visual condition had been chosen for further testing. The purpose of choosing elements based on the visual registration was to see whether there was a correlation between a visual registration and testing of the elements. In order to obtain valid data it was chosen to test 3 elements of the same type in the same condition. This resulted in 3 tests of the consoles in good condition, 3 tests of the consoles in bad condition, 3 tests of the plates in good condition and 3 tests of the plates in bad condition. In addition to the consoles and plates the gable column in the north-east corner of the building was tested and a test was also performed on the gable wall.

All tests were performed on elements on the north side of the building which is where the wide balconies were placed. It would not have been possible to perform tests on the south side of the building because of the lack of space. Elements in level 1 were not tested due to the inconvenience of working from a ladder on the outside ground surface.

3.2.1 Cover Layer Thickness

The cover layer thickness of the concrete was measured with a PROFOMETER 5^+ Model S which is shown in Figure 3.5. The cover meter can locate rebars, measure the cover layer thickness and determine the bar diameter [20].

Figure 3.5: Cover meter PROFOMETER 5^+ Model S.

Figure 3.6: Measurement of cover layer.

The first step when using the cover meter is to locate the correct reinforcement. Usually both longitudinal and transverse reinforcement will be present in a concrete element as well as stirrups. In this project it was important to measure the cover layer thickness to the main reinforcement. The reinforcement was located by looking at drawings showing the reinforcement design and by detecting the reinforcement with the cover meter in two directions perpendicular to each other, see usage of cover meter in Figure 3.6. The cover meter made a sound when reinforcement was detected and it displayed the cover layer thickness as well as the diameter of the bars. The diameters measured with the cover meter were however not very reliable, since they varied a lot, especially when the reinforcement was closely placed in an element. In this project the diameter of the reinforcement was not relevant and only information about the cover layer thickness was needed. In order to obtain a good data-set of the cover layer thickness at least 10 measurements had to be performed for each individual element.

3.2.2 Carbonation Depth

For each element three carbonation tests were performed in order to compare the results and the validity of the tests. The tests were performed by drilling a 5mm deep hole with a Hilti rotary hammer using a 10mm drill, see Figure 3.7. The indicator thymolphthalein was then dripped into the hole with a pipette and the color of the indicator was checked. The indicator thymolphthalein is colorless, but it changes color to dark blue when it comes in contact with materials that has a pH value of 9-10.5. The pH value of carbonated concrete is 7-9 and the indicator would therefore detect fully carbonated concrete. If the indicator dripped into the hole remained colorless another 5mm was drilled in order to obtain a 10mm deep hole. The indicator was then dripped into the hole again and the process of drilling 5mm at a time was continued until the indicator turned dark blue, see Figure 3.8. The intersection between carbonated and un-carbonated concrete was then found, the so called carbonation depth. Due to the drilling process the carbonation depth was found in an interval of 5mm.

Figure 3.7: Drilling of holes for carbonation test.

Figure 3.8: Carbonation indication with thymolphthalein.

One of the main errors that occurred whilst performing this test was that the drilled depth might vary a few mm. A distance measurement tool was not available for the rotary hammer and therefore the measurements where made manually by placing tape on the drill. This distance error combined with the probability of drilling into a stone were the main reasons for performing three tests on each element. If a more detailed carbonation depth measurement is required then a drilled core from an element can be analysed in a laboratory, however it was not possible for this field testing. The advantage of drilling holes in many elements with a rotary hammer is that quantitative data is obtained and it gives a good overview of the carbonation depth around the building.

3.2.3 Concrete Strength

The strength of the concrete was measured with a Schmidt Hammer and the Schmidt Hammer used for the tests is shown in Figure 3.9. The Schmidt Hammer is a mechanical instrument that impacts the concrete and thereby measures a rebound value. For each tested element 10 measurements should be conducted with the Schmidt Hammer in order to obtain reliable data [21]. An average of the 10 values is found and then the concrete strength can be found from a conversion graph, see Appendix B.4. The strength of the concrete determined from the rebound value depends on whether the impact is performed vertical downward, vertical upward or horizontal, since human strength is needed to perform the impacts. The concrete strengths found might not be correct, since there is a lot of uncertainty when using the Schmidt Hammer. Despite this it is still possible to compare the magnitude of the strengths for each element in order to see which elements contain the strongest concrete.

Figure 3.9: Schmidt Hammer.

3.2.4 Tested Elements

Consoles

The consoles are pre-cast and thereafter they are cast together with the inner bearing walls. The bottom part of the console is therefore the most interesting part, since the top part of the console can always be removed and is not needed in a renovation. The consoles are subjected to a vertical downward facing load and therefore the console will be in tension along the top edge. It was chosen to measure the cover layer thickness of the main reinforcement in the top of the cross section which consist of four horizontal bars. The cover layer thickness was measured ten times on each side of the console, resulting in twenty measurements. Three horizontal holes in the bottom part of the console were drilled in order to determine the carbonation depth of the concrete. The concrete strength was, for practical reasons, measured on the top part of the console by taking random measurements on all sides of the element. The placements of the tests performed on a console are illustrated in Figure 3.10.

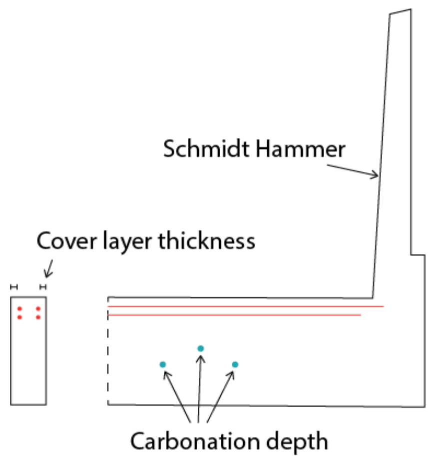


Figure 3.10: Console measurements.

Plates

The balcony plates contain reinforcement in both the longitudinal direction and in the transverse direction. The longitudinal reinforcement placed in the bottom of the cross section is most important since it will be in tension when the plates are subjected to a vertical load. The cover layer thickness was therefore measured from the bottom of the plates to the longitudinal reinforcement. The carbonation depth was measured by drilling three holes in both the bottom and the top of the plate, since the depth might vary in these two locations. The concrete strength was measured on the top of the plate with vertical downward facing impacts from the Schmidt Hammer, see Figure 3.11.

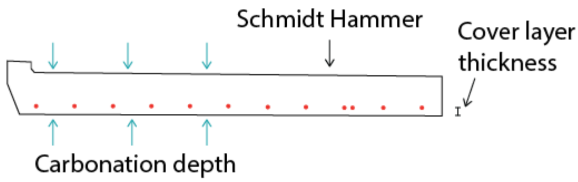


Figure 3.11: Plate measurements.

Gable

A gable column was tested and the tests were performed at each level. The cover layer thickness was measured to the main vertical reinforcement bars placed in the corners of the column. The carbonation depth was measured in three places on top of each other and ten Schmidt Hammer measurements were conducted, an illustration is given in Figure 3.12. Since the gable wall was covered with insulation and cladding it was not possible to perform tests on the wall, however it was possible to make one small hole in the cladding which resulted in ten Schmidt Hammer measurements and one measurement of the carbonation depth.

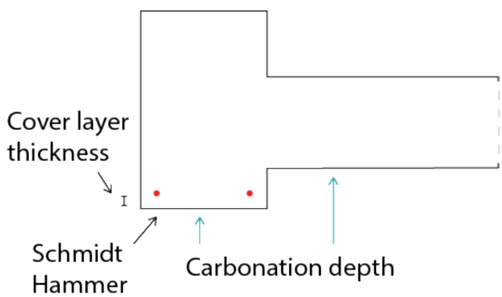


Figure 3.12: Gable column and wall measurements.

3.2.5 Carbonation Age

The carbonation age is a measure that can either be used to determine how long reinforcement has been subjected to carbonated concrete or when the carbonation front will reach the reinforcement. The carbonation age is measured in years and the results depends on the measured carbonation depths and cover layer thicknesses as well as the amount of years that the concrete has been subjected to atmospheric air. It is assumed that the concrete elements have never been protected against carbonation and therefore the amount of years that carbonation has occurred is assumed to be the same as the age of the concrete. Block 10 was build in 1975 and therefore the age of the concrete was 44 years in 2019. The carbonation age will be determined for all elements by the use of equation 3.1 [16]. The equation states that the carbonation depth x is proportional to the time t squared. The factor K is a constant that is determined individually for each element.

$$x = K \cdot \sqrt{t} \tag{3.1}$$

In order to use the equation the procedure will now be described. The carbonation depth x_{carb} for an element is known from the tests and the exposure time is $t_{44} = 44$years, therefore the constant K can be determined for the specific element:

$$K = \frac{x_{carb}}{\sqrt{t_{44}}} \tag{3.2}$$

Now that the constant is known the measured cover layer thickness x_{cover} and the constant K can be inserted in the equation in order to determine t_{front}:

$$t_{front} = \left(\frac{x_{cover}}{K}\right)^2 \tag{3.3}$$

The time t_{front} is in this case the age of the concrete when the carbonation front reaches the reinforcement. The difference $\Delta t = |t_{front} - t_{44}|$ will therefore define the amount of years that the concrete around the reinforcement has already been carbonated or in how many years from now the carbonation front will reach the reinforcement.

3.3 Results of Visual Registration

A visual registration was performed for block 10 in Sisimiut and in this section the results will be presented. The visual condition of block 7 and the renovated blocks in Sisimiut will also be considered.

Figure 3.13: North facade of block 10 in Sisimiut.

3.3.1 Damage Categories

In order evaluate the amount of damage in the concrete structure the damages in each concrete element has been divided into three damage categories. The categories are defined below.

- Category 1: No severe damage, smooth surfaces.
- Category 2: Semi severe damage, loose/uneven concrete surfaces, visible aggregates, concrete spalling, small cracks.
- Category 3: Severe damage, visible corroded reinforcement, large cracks, large amount of broken off concrete.

The damage category for an element is determined purely from a visual registration and therefore the category is based on surface damages of the concrete elements. Surface damages are often caused by internal damage, however internal damage might not be visible on the surface. Examples of the three damage categories can be seen from Figure 3.14 and more images representing each damage category can be found in Appendix A.1.

(a) Damage category 1, console bottom. Smooth surface, intact console.

(b) Damage category 1, console top. Slightly uneven top surface, no signs of cracks.

(c) Damage category 2, plate. Spalling of concrete, loose concrete surface, small cracks.

(d) Damage category 2, console bottom. Uneven concrete surface patch, signs of cracks.

(e) Damage category 3, console top. Large cracks, broken off concrete, visible reinforcement.

(f) Damage category 3, console bottom. Large cracks, will lead to broken off concrete.

Figure 3.14: Examples of damage categories.

From a visual registration of block 10 damages on external concrete elements were registered and the registration can be found in Appendix B.1. The registered elements consisted of the top and bottom of the consoles as well as the balcony plates. The damage category for each element is illustrated in Figure 3.15 and 3.16, for the north and south facade respectively, where the categories have been given colors corresponding to the category. Category 1 with no severe damage is marked with green, category 2 with semi severe damage is marked with orange and category 3 with severe damage is marked with red.

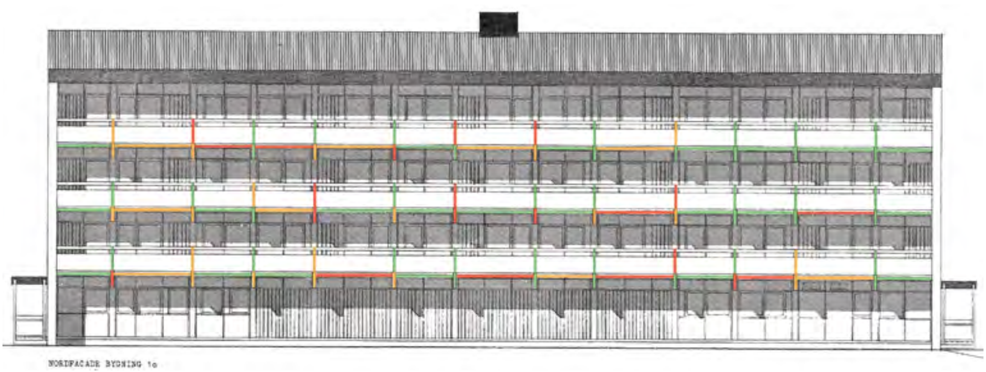


Figure 3.15: Damage categories, north facade. Category A = green, category B = orange and category C = red.

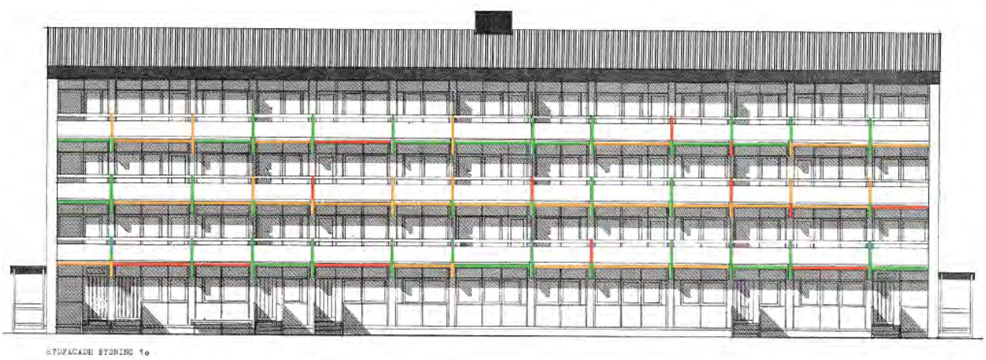


Figure 3.16: Damage categories, south facade. Category A = green, category B = orange and category C = red.

It is difficult to find a pattern in the placement of the damages from Figure 3.15 and 3.16. In general the elements placed closer to the gable walls belong to damage category 1 or 2 and damage category 3 is more frequently found in the middle section of the building. Besides this observation no other connection is found between the damage category and the placement of the elements. In order to get an overview of the amount of damages the percentage of elements belonging to each damage category has been found for plates, the top of consoles and the bottom of consoles, see Table 3.1, 3.2, and 3.3. From the results it can be seen that in general most elements belong to damage category 1 with no severe damage, therefore implying that the general condition of the building is quite good. It should also be noted that the amount of elements belonging to damage category 3 is less for the south facade compared to the north facade which means that the overall condition of the south facade is slightly better.

Table 3.1: Damage categories in percentage for plates.

Plates	North facade	South facade	Total
Category 1	56%	31%	44%
Category 2	23%	54%	38%
Category 3	21%	15%	18%

Table 3.2: Damage categories in percentage for top of consoles.

Consoles, top	North facade	South facade	Total
Category 1	64%	64%	64%
Category 2	17%	22%	19%
Category 3	19%	14%	17%

Table 3.3: Damage categories in percentage for bottom of consoles.

Consoles, bot	North facade	South facade	Total
Category 1	45%	72%	58%
Category 2	39%	22%	30%
Category 3	16%	6%	12%

For the plates on the south facade 54% of them belong to damage category 2 and this number stands out. The balcony plates have an uneven concrete surface where scaling of the concrete has occurred. It is very likely that the loose concrete surfaces are caused by freeze-thaw damage. Photos of an even and uneven concrete surface are shown in Figure 3.17 and 3.18.

Figure 3.17: Even concrete surface.

Figure 3.18: Uneven concrete surface.

3.3.2 Gable Columns

An example of a damage on a gable column is given in Figure 3.19. The damage is placed on the gable column towards south-east which is shown in Figure 3.20.

Figure 3.19: Damages on column.

Figure 3.20: Gable column south-east.

The damage which can be seen in Figure 3.19 is a typical carbonation damage where the concrete is cracked and broken off due to corrosion of the reinforcement. In general the columns of block 10 only have few damages on each column and images of all the gable columns are given in Appendix A.2. The damages on the gable columns of the other non-renovated blocks in Sisimiut were in a worse condition compared to block 10. Large amounts of concrete has broken off which makes the reinforcement very visible and therefore the blocks are currently going through a light renovation where the entire columns are covered with a thin U-shaped metal sheet.

3.3.3 Foundation

The foundation of block 10 consists of prefabricated beam elements. The beam elements placed along the edges of the building are not in a good condition and corroded reinforcement is visible on most of the elements, see Figure 3.21. The beams have over time been displaced and some of the beams are so displaced that they are not supporting the building anymore as seen in Figure 3.22. The bearing foundation is not visible and the condition is unknown. However due to soil displacement large holes under the building can be found and it is possible to see the bearing piles in some places, see Figure 3.23. The holes have been filled with large rocks as in Figure 3.24.

Figure 3.21: Corroded reinforcement along the foundation.

Figure 3.22: Displaced foundation beam.

Figure 3.23: View of bearing pile under building.

Figure 3.24: Hole under building filled with large rocks.

Block 10 is being demolished due to settlements, however no settlement damages were found. On the south side of the building a swampy area is located that most likely consist of permafrost clay and on the north side of the building rock appears approximately 20m away from the building. This highly suggests that the whole building is sinking towards south. A test was made with a spirit level in order to investigate whether the building was tilting or not. The spirit level was placed in many different places along the foundation, see Figure 3.25 and 3.26, in order to observe the angle of the building. In no location did the spirit level show any signs of displacement of the building, however the displacements might be so small that the spirit level could not record them.

Figure 3.25: Placement of spirit level on the foundation.

Figure 3.26: Result from spirit level on foundation.

Block 10 is the only block in Sisimiut which has a prefabricated foundation, the rest of the blocks have in-situ cast foundations with no visible reinforcement. A picture of the foundation of block 7 is shown in Figure 3.27, the foundation is in a good condition and it is in-situ cast. Many of the blocks are surrounded by rock which most likely means that the foundations are placed directly on rock, which creates a much more stable foundation. Block 11 in Sisimiut has a foundation built on rock, which means that the bottom level of the building is not continuous, see Figure 3.28.

Figure 3.27: In-situ cast foundation of block 7.

Figure 3.28: Solid rock foundation of block 11.

3.3.4 Comparison with Block 7

Block 7 was in October 2019 undergoing a renovation due to mold in 13 out of 21 apartments. It was therefore possible to get access to the building and a brief visual registration of the building was performed. Block 7 has four floors like block 10 and it consists of 16 modules which is 3 modules more than block 10. The facade with long consoles are placed towards west and the facade with short consoles are placed towards east. A picture of the building can be seen in Figure 3.29 and it can also be seen that one of the concrete railings have been covered with plywood to avoid falling concrete.

Figure 3.29: Block 7, Sisimiut.

The amount of damage have been counted for each facade instead of registering them for each element as it was done for block 10. The reason for this choice was that a full registration would have been too extensive to perform. The amount of damages in percentage for each of the two facades (west and east) are given in Table 3.4. The elements were checked for damages such as visible reinforcement and broken off concrete, and in general the percentages are very low. For the plates the percentage of visible reinforcement is very high and the reason could be that the transverse reinforcement have been placed too close to the edge of the plates when they were cast. This results in visible reinforcement along the edge of the plates as seen in Figure 3.30. Block 7 had previously been tested by drilling out cores from the concrete elements, see Figure 3.31. The holes had not been filled out after the testing and it was noticed that the reinforcement inside the holes have corroded.

Table 3.4: Amount of damages on block 7 seen from outside the building.

Block 7	West facade long consoles	East facade short consoles
Plates - visible reinforcement	42%	48%
Plates - broken off concrete	13%	4%
Consoles - visible reinforcement	18%	2%
Consoles - broken off concrete	22%	11%
Consoles - cracks	13%	4%

Figure 3.30: Visible reinforcement at edge of balcony plates.

Figure 3.31: Holes in the elements due to previous drill core testing.

In general the condition of block 7 is much better than the condition of block 10. The main difference seems to be the type of concrete used, since in block 7 all of the concrete surfaces are smooth and hard. The amount and size of the aggregates are also larger compared to block 10, see Figure 3.32 and 3.33.

Figure 3.32: Aggregates in concrete, block 7.

Figure 3.33: Aggregates in concrete, block 10.

3.3.5 Comparison with Renovated Blocks

The condition of the renovated blocks in Sisimiut is not known. The blocks showed no visible signs of damage and this could be because the condition of the elements is very good or because the damages were hidden. The renovation method is unknown and therefore is it not known whether damaged elements have been fully or superficially renovated. Figure 3.34 shows the elongation of a console. It can be seen that the connection is skew and it is unknown how the elongation of the console had been made. The elongated console is supported by a column and therefore the skew connection might not have an effect on the bearing capacity of the console. Figure 3.35 shows a facade with long consoles and no damages can be seen on any of the consoles or plates.

Figure 3.34: Renovated block, elongation of console.

Figure 3.35: Renovated block, facade with long consoles.

3.4 Results of Field Testing

The chosen representative elements from the visual registration are illustrated in Figure 3.36, where the color green represents elements in a good condition and the color red represents elements in a bad condition. Pictures of all the tested elements are shown in Appendix A.3.

Figure 3.36: Tested elements, north facade. Good condition = green and bad condition = red.

For each concrete element three different tests were performed as described earlier in section 3.2. All test data is given in Appendix B.2 for measurements of cover layer thickness, Appendix B.3 for measurements of the carbonation depth and Appendix B.4 for measurements of the concrete strength. In order to summarise the results from the testing the average values will be presented. For the strength measurements outliers are not included in the results and they are marked with red in the data. If a measurement of the concrete strength was not performed correctly it has not been included in the data. To estimate the homogeneity of the cover layer thickness and the carbonation depth the standard deviations have been calculated, these are also presented in Appendix B.2 and B.3. It is worth noticing that at least three carbonation tests were performed for every element, which resulted in a total of 61 carbonation tests. Only in one test did the drill collide with a rock in the concrete and the test was stopped. This could be a big coincidence, however it either seems like there was a lack of aggregates in the concrete or the aggregates had been unevenly distributed, see Figure 3.33. Another explanation could be that the quality of the aggregates was very poor and therefore there was no big difference in strength between the cement and the aggregates.

The average results for tests performed on the chosen consoles are shown in Table 3.5, where the average cover layer thickness, carbonation depth and concrete strength is given for each element. The difference between the cover layer thickness and the carbonation depth is also defined, where a positive difference means that the cover layer thickness is larger than the carbonation depth and a negative difference means that the carbonation depth is larger than the cover layer thickness. From the results it can be seen that the carbonation depth is smaller than the cover layer layer thickness for all the elements in a good visual condition, this means that the reinforcement has not corroded. For elements in a bad visual condition the reinforcement in 2 out of 3 elements will have started corroded since the carbonation depth exceeds the cover layer thickness. The element in level 2, line 8 turns out to be in a good condition despite the visual registration of it being in a bad condition. From the concrete strength measurements it can be seen that the strength is slightly higher for elements in a good condition compared to the elements in a bad condition.

Table 3.5: Results of average measured data for consoles.

Consoles	Good visual condition			Bad visual condition		
	level 2, line 3	level 3, line 7	level 3, line 11	level 2, line 8	level 2, line 10	level 3, line 9
Cover layer thickness [mm]	27.9	25.2	26.9	26.8	27.1	26.8
Carbonation depth [mm]	24.2	24.2	15.8	14.2	37.5	52.5
Concrete strength [MPa]	31	31	37	26	20	22
Difference [mm]	+3.7	+1.0	+11.1	+12.6	-10.4	-25.7

Table 3.6 sums up the average results for tested balcony plates. In the table the symbol b stands for the bottom of the plate and the symbol t stands for the top of the plate. It should be noticed that the carbonation depth was not tested in the bottom side of two of the plates. The cover layer thickness to the bottom longitudinal reinforcement is very similar for all the measured elements. When comparing the carbonation depth of the plate tops with the plate bottoms it can be seen that the carbonation depth is always smaller on the top of the plates. For elements in a good visual condition the carbonation depth is very small, especially on the top of the plates, and it is quite large for the elements in a bad visual condition. The difference in depth is calculated for the top and the bottom of the plates and it can be seen that the carbonation front has not reached the reinforcement in elements with a good visual condition whereas the reinforcement in elements with a bad visual condition has begun to corrode. It can also be seen that the concrete strength is significantly higher for elements in a good condition compared to elements in a bad condition.

Table 3.6: Results of average measured data for plates.

Plates	Good visual condition			Bad visual condition		
	level 2, line 4-5	level 2, line 6-7	level 3, line 2-3	level 2, line 2-3	level 2, line 12-13	level 3, line 11-12
Cover layer thickness [mm]	28.3	28.2	22.5	25.0	24.0	26.9
Carbonation depth, b [mm]	7.5	-	19.2	-	37.5	32.5
Carbonation depth, t [mm]	2.5	2.5	2.5	34.2	30.8	29.2
Concrete strength [MPa]	38	40	43	27	30	29
Difference, b [mm]	+20.8	-	+3.3	-	-13.5	-5.6
Difference, t [mm]	+25.8	+25.7	+20.0	-9.2	-6.8	-2.3

The results concerning the east gable wall and the gable column in the north-eastern corner of the building are presented in Table 3.7. The results show that in all cases the carbonation depth is very small compared to the cover layer thickness, which implies that the reinforcement has not begun to corrode. However carbonation damages are visible on the columns and therefore the wall and column are not in a good condition despite the results. Lastly it can be seen that the concrete strengths are larger than the strengths measured for the consoles and plates.

Table 3.7: Results of average measured data for the gable wall and gable column.

Gable	Wall	Gable columns			
	level 3	level 0	level 1	level 2	level 3
Cover layer thickness [mm]	50.0	32.5	37.2	36.0	42.8
Carbonation depth [mm]	2.5	2.5	12.5	2.5	10.8
Concrete strength [MPa]	50	52	38	46	48
Difference [mm]	+47.5	+30.0	+25.0	+33.5	+32.0

3.4.1 Carbonation Age

The carbonation age has been determined for all the tested elements and Table 3.8, 3.9 and 3.10 sum up the results. If the remaining time until the carbonation front reaches the reinforcement is greater than 150 years then it has been noted >150. The results are very interesting since they vary a lot in size. For two of the consoles the concrete around the reinforcement has already been carbonated for many years and the rest of the elements still have a few years left before the carbonation front reaches the reinforcement, however they should be protected against further carbonation. The reinforcement in the plates with a bad visual condition have all be exposed to carbonated concrete for 7 to 26 years already. The plate elements in a good visual condition must be made of very dense concrete since the time until the carbonation front reaches the reinforcement is very high. The same fact applies to the gable and gable wall where the largest amount of years calculated is >15,000 years for the gable wall. For the console and plate elements where the reinforcement is surrounded by carbonated concrete the average amount of years since the carbonation front reached the reinforcement is 20 years.

Table 3.8: Carbonation age for tested consoles.

Consoles	Good visual condition			Bad visual condition		
	level 2, line 3	level 3, line 7	level 3, line 11	level 2, line 8	level 2, line 10	level 3, line 9
Δt [years]	14	4	84	113	-21	-33

Table 3.9: Carbonation age for tested plates.

Plates	Good visual condition			Bad visual condition		
	level 2, line 4-5	level 2, line 6-7	level 3, line 2-3	level 2, line 2-3	level 2, line 12-13	level 3, line 11-12
Δt [years] (t)	>150	>150	>150	-20	-17	-7
Δt [years] (b)	>150	-	16	-	-26	-14

Table 3.10: Carbonation age for tested gable wall and gable columns.

Gable	Wall	Gable columns			
	level 3	level 0	level 1	level 2	level 3
Δt [years]	>150	>150	>150	>150	>150

3.5 Discussion of Condition Assessment

A visual registration of external concrete elements of block 10 has been performed and each element has been divided into three damage categories depending on the severeness of the damages. In general most elements belonged to damage category 1 with no severe damages and this indicates that the overall state of the concrete is good. It was expected that a damage pattern for a whole facade could be found and that elements would be placed next to or on top of each other if the damages e.g. had occurred due to accumulation of water, however no damage pattern has been found. This highly suggests that the damages has occurred due to insufficient concrete quality in some elements or a varying cover layer thickness. It was a general observation that the surface texture of the concrete varied, since some surfaces were smooth and other surfaces were crumbling, which could be a result of freeze-thaw damage. Given that the building is in-situ cast there might not have been a strict supervision of the concrete mixing and the casting, which is why the concrete quality can be questionable.

It was found that elements on the south facade in general were in a better condition compared to the north facade. The only exception being the balcony plates on the south facade, where a lot of them are subjected to freeze-thaw damages. An explanation could be the effect of the sun, which causes temperature changes and most likely also drying of the elements on the south side, whereas elements on the north facade will be subjected to fever temperature changes and a more constant humidity. The main wind direction in Sisimiut comes from north meaning that the north facade would be more subjected to rain. Therefore it would be expected that less damages due to carbonation would be found on the north facade, however this was not observed.

The actual condition of the foundation of block 10 is unknown. The visible foundation along the edge of the building is made with prefabricated elements and these elements are either displaced or damaged. However the foundations of the other non-renovated buildings in Sisimiut are in-situ cast and the conditions are much better. The bearing capacity of the foundation depends on the type of concrete foundation and the ground conditions, since the buildings can either be placed on rock or permafrost. The foundations will therefore vary for each building and the capacities will have to be investigated separately. The foundation capacity has not been investigated in this project, but since the concrete structure is massively built it is assumed that the concrete foundation also is over dimensioned.

Block 7 in Sisimiut was briefly investigated and the main observation was that the concrete seems to be in a much better condition compared to the concrete in block 10. The amount and sizes of the aggregates in the concretes were different for the two blocks. Other parameters in the concrete mixture might also vary and therefore the condition of each individual building should be determined separately.

Elements in a good and bad condition according to the visual registration were chosen for further testing to determine the actual condition of the elements. The carbonation depth and cover layer thickness were measured in order to investigate whether the reinforcement was corroded or not. The results show that the carbonation depth is smaller than the cover layer thickness for elements in a good visual condition and vice versa. The measured cover layer thicknesses for the consoles are smaller than the described cover layer thickness according to the construction drawings. For consoles the cover layer thickness to the main reinforcement should be 30mm and the measurements vary from 25.2-27.9mm. For plates, according to the drawings, the cover layer thickness should be 20mm and this requirement is met. In general the measured cover layer thicknesses are smaller than what they should be, according to the standards, since the concrete elements are placed outside in a harsh climate. The carbonation depth was measured on the top and the bottom of the plates and the results show that the carbonation depth on the top of the plates is lower compared to the bottom of the plates for all the tested elements. This corresponds very well with the theory regarding the carbonation rate being faster for surfaces not directly subjected to direct rain, since a constant humidity will be present for these elements. Elements subjected to direct rain, like the top of the plates, will be very moist most of the time and therefore optimal conditions for carbonation is not often present. A measurement of the strength was also performed and the results showed that elements in a good condition also had a higher strength compared to elements in a bad condition. This can be due to the fact that, as discussed earlier, the concrete quality varies and thereby also the strength.

The test results for the gable column do not match the visual condition of the columns. The test results show that the carbonation depths are very small for the column, between 2.5-12.5mm, and that the cover layer thickness to the main reinforcement is at least 32.5mm. This suggests that carbonation is not causing the damages even though the damages look like typical carbonation damages. The tests are performed on one side of the column and therefore the cover layer thickness might vary on the other sides. Also the cover layer thickness will be smaller for the horizontal stirrups compared to the main reinforcement. When examining the damages on the columns it is noticed that the concrete is often missing around the stirrups and that the damages do no occur on all sides of the column. This suggest that the reinforcement might not placed centrally in the columns. Therefore the cause of the damages can most likely be carbonation due to a small cover layer. The condition of the columns for block 10 are quite good compared to the columns of the other non-renovated buildings in Sisimiut. Here the damages are very severe since large amounts of concrete are missing around the reinforcement. It is a general observation that the columns are the elements in the worst condition and the reason could be that when casting the columns the concrete was not compacted enough, which means that the concrete texture was loose and therefore carbonation can occur more rapidly. If at the same time the reinforcement is not placed centrally in the columns this can cause the cover layer thicknesses to be too small.

Many measurements have been performed which has resulted in a big amount of quantitative data. This type of data is very useful for creating an overview of the situation. Errors in the data can be uncertainty in the depth of the drilled holes due to the equipment used, the cover meter values can be measured for the wrong or overlapping reinforcement, and the strength measurements with the Schmidt Hammer depends on the manual force used to create an impact. In order to take the errors into account many tests have been performed for each element in order to validate the results. If more qualitative results are wanted then a drilled core can be tested in a laboratory, however this method will not take the varying concrete quality in different elements into account and it will therefore not create an overview of the condition. The test would however be able to determine the amount of aggregates and density of the concrete.

From the test results it can be seen that elements with a good visual condition are also in a good condition according to the tests. The same can be seen for elements in a bad visual condition, except for one console element that turned out to be in a good condition despite the visual registration. Twelve representative consoles and plates were investigated in this manner, the results are consistent and therefore very reliable. This means that the condition of the concrete elements can be determined purely from a visual registration, which makes it easy to obtain an overview of the overall condition of the building. This is a quick and easy method that can be used to determine the condition of a building before the planning of a renovation. It is however recommended for future studies that a few concrete samples from other buildings will be tested in the same manner to confirm this observation.

The types of damages found are usually spalling of concrete, cracks and visible reinforcement. The primary cause of deterioration is therefore carbonation and freeze-thaw. The damages depend on several factors such as the w/c ratio and air entrainment of the concrete as well as the temperature and humidity of the air. The combination of carbonated concrete and a thin cover layer result in broken off concrete and corrosion of the reinforcement. Therefore carbonation is the main deterioration cause for both the reinforcement and the concrete. In some places, mainly the plates, the concrete surfaces are scaling from freeze-thaw damage however this damage is not as severe as the carbonation damage.

CHAPTER 4

Structural Analysis of Concrete Elements

In this chapter the load bearing capacity of the concrete elements will be analysed. The calculations will be performed for block 12 (B-871), due to the availability of the original drawing material, however the aim is that the results should be applicable to all buildings of the same type. Block 12 was demolished in 2015 and Figure 4.1 shows the concrete structure of the building during the demolition. The load bearing capacity of several structural elements of the building have been analysed in order to investigate the relative effect of different renovation proposals. This means that a complete structural analysis of the entire building has not been performed, but the overall stability of the building was checked. Calculations, in the case of fire, have not been considered since it is assumed that all structural elements will be covered by 2 x 12.5mm gypsum plates that meet the fire requirements. The standards used for the calculations are the Eurocodes and the Greenlandic National Annexes.

Figure 4.1: Block 12 during demolition (photo provided by Egil Borchersen).

4.1 Static System

Block 12 is an in-situ cast building where the main structural elements are walls and slabs. Other structural elements present are four columns placed in each corner of the building as well as the consoles that are cast together with the transverse walls. The block consists of four levels and twelve transverse walls, see plan and elevation drawing in Figure 4.3 and 4.2.

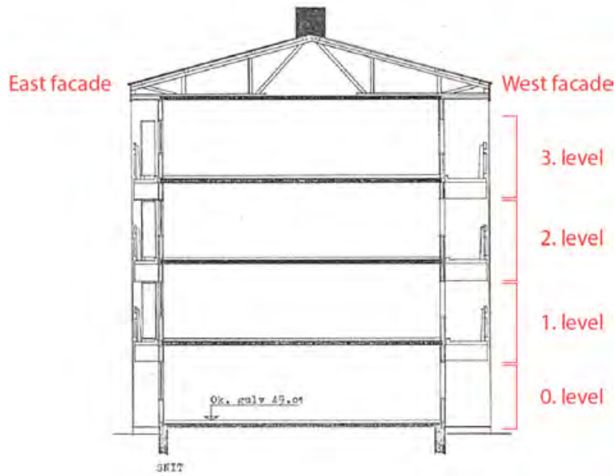


Figure 4.2: Elevation drawing of block 12.

Figure 4.3: Plan drawing of block 12.

Vertical load, including live load, snow load, and dead load, will be distributed from the decks out to the bearing walls, which will then transfer the load straight down to the foundation. Horizontal load that in this case is wind load or seismic load, will act as a distributed load either on the facade or on the gable wall. The load will then be transferred to the decks and eventually it will be transferred through the transverse or the longitudinal walls in order for the load to end up at the foundation. The gable walls are attached to a column on each side of the wall in order to stabilize the wall from buckling, since the walls are not supported by decks where the balconies are placed. The concrete structure of block 12 is illustrated in Figure 4.4 where the grid lines according to the original drawings are also shown. The original drawings from GTO that are used for the calculation can be found in Appendix E.

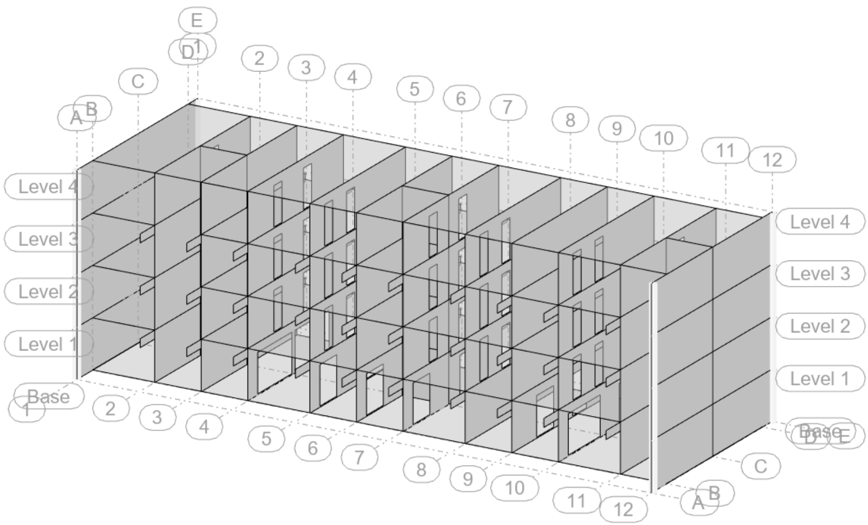


Figure 4.4: Concrete structure of block 12.

A building consisting of plates will be stable if it contains a set of 3 walls that span along the full height of the building and these walls must be attached to the decks at all levels. At least two of the walls should not be parallel and the walls must be separately stable. The stabilizing walls in block 12 could be the two gable walls and one of the longitudinal walls. However, there are several more stabilizing walls within the building and therefore it can be concluded, that the conditions regarding stability are fulfilled for the analysed building. No extra stability checks will therefore be performed as long as the above mentioned conditions hold true.

4.2 Description of Renovation Proposals

The bearing capacity will be investigated for three different renovation cases. The different cases are firstly the design of new facades and balconies, secondly the possibility of creating openings in the bearing walls, and lastly the effect of adding an extra floor to the building. For all renovation proposals it is not wanted to create any extra structural elements, this includes new foundations and columns, in order to simplify the renovation. The top part of the consoles will not be considered in the renovation proposals since it is assumed that they will be removed. Each renovation case will be described in the following sections.

Furthermore it is assumed that new access ways to the apartments will be implemented as external structures. The external structures will consist of stairs and elevators and, at each level of the building, they will be connected to the apartments. The structures will have their own foundation and they will not influence the static system of the building. Since the entrance towers are external structures they will not be considered any further in this report.

4.2.1 New Facade and Balcony Design

One of the main problems with the current placement of the facade is the thermal bridges around the consoles. Therefore, two of the new renovation proposals will include the consoles in the indoor environment. The facade should thus be placed on the outside of the current balconies and consoles. The facade will only extend to the bottom of level 1 and not all the way to the ground level, since this area will not be included in the indoor environment. Another thermal bridge is found between the foundation and the bottom level of the building and therefore it has been chosen not to place any apartments in level 0. The building envelope will be placed just under level 1 in order to secure a good indoor climate in all apartments. The new facade ends at the bottom of level 1 and, since no new foundations and columns will be constructed, this means that the consoles must carry the entire load arising from the facade and balconies. The bearing capacity of the consoles will be checked in the following three different design scenarios.

Proposal 1: In the first design proposal the original balcony and facade design is maintained, see Figure 4.5. The original design is kept in this proposal in order to compare the results to the other proposals. Furthermore it will be investigated whether the balcony areas in this proposal can be increased. This will be the most simple renovation of the three outlined proposals, since only the facade is changed, however there will still be problems with thermal bridges around the consoles.

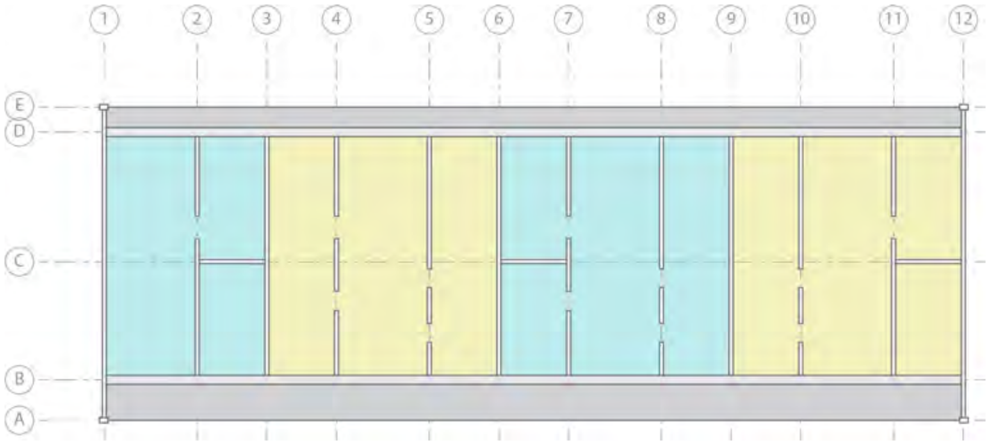


Figure 4.5: Proposal 1 - original balconies.

Proposal 2: In the second design proposal most of the balcony plates and consoles will be included in the residential area, this means that a new facade will be placed on the outside of the consoles. In some modules built-in balconies will be placed, see Figure 4.6. The residential area will be larger and the apartments will have more privacy due to the change of balcony design.

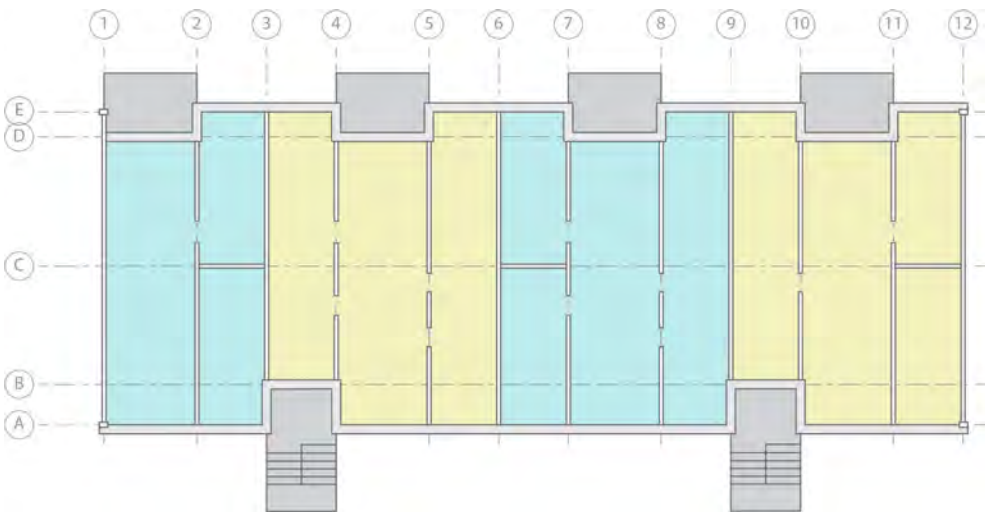


Figure 4.6: Proposal 2 - built-in balconies.

In order to enter an apartment it will no longer be necessary to walk past the outside of other apartments and the balconies will, as mentioned, be more private since they will no longer be connected. Another positive side effect is that the new balcony space will be more useful than the old narrow balconies. The built-in balconies can be constructed on either side of the building, depending on the orientation of the building. For now they will be placed on the side with narrow balconies, since the current entrances are placed on the side with wide balconies.

Proposal 3: Lastly in the third design proposal the facade will be placed on the outside of all consoles and balcony plates. It will be investigated whether cantilevered balconies can be attached to the outside of the new facade, see Figure 4.7. The external balconies are illustrated as rectangles, but the idea is that the balconies can have any desired shape, which will create a more interesting and dynamic facade for the building. This design proposal has a more simple facade than proposal 2 since it is linear and does not bend in any angles. The living area in this design proposal is largest since all the original balconies are included in the living area.

Figure 4.7: Proposal 3 - external balconies.

4.2.2 New Openings in Bearing Walls

In this design proposal the possibilities of creating new openings in the bearing walls will be investigated. The layout of the existing apartments are very well made, however if changes will be made to the floor plans then new openings must be made in the walls. The old staircases will, in the renovation proposal, be included in the living area. Therefore new openings will be made in order to connect these areas, see

Figure 4.8 where the current floor plan is also showed. The size of the apartments will increase if the facade is moved to the outside of the consoles and this creates new possibilities in designing the floor plan. A systematic analysis of creating new openings in the bearing walls will be made in order to create useful results for all buildings and not only block 12.

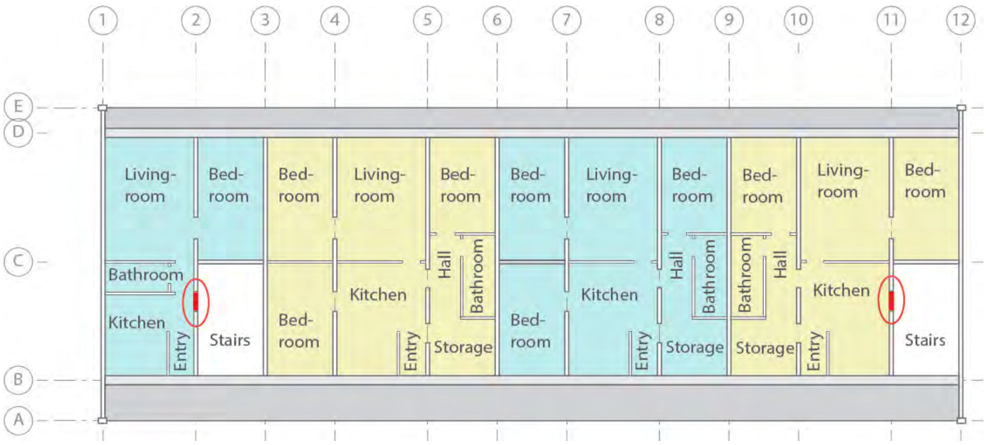


Figure 4.8: Proposal - new openings in bearing walls.

4.2.3 Addition of Extra Floor

The possibility of adding an extra floor or several floors on top of the building will be investigated in this design proposal. Adding an extra floor will have several benefits. First of all there is a lack of housing in Greenland and by adding an extra floor to the buildings new apartments will be created without having to construct a foundation and sewerage system. Also if the building envelope is moved then existing apartments on the ground level will be closed down and an additional floor would compensate for this. Secondly if the building is to be renovated the residents must be rehoused and due to the lack of apartments, the residents could perhaps be rehoused on the extra floor while their apartments are being renovated. It might not be a solution to be rehoused in the same building that is being renovated, but if several buildings were renovated one after another then residents could be rehoused on the extra floor on top of the other buildings. If temporary rehousing is needed then container apartments could be attached to the roof since this would be a fast and simple solution. Otherwise a permanent extra floor will be made of a light construction and the floor area will be slightly smaller, compared to the other levels, in order to avoid extra load on the consoles. Therefore the top floor would have a balcony area similar to the original balcony design.

4.3 Loads and Load Combinations

4.3.1 Dead Load

The dead load is a permanent load which includes the self weight from the bearing structure and non-structural elements. This includes walls, decks, facades, non-bearing walls, roof, cladding on walls, flooring, insulation, balconies etc. The self weights are based on the densities found in *DS/EN 1991-1-1 Annex A* [22] and otherwise they are roughly estimated. If a more detailed calculation were to be made, then the exact materials used in the building should be known, in order to calculate precise values of the self weights.

The characteristic self weights of the structural reinforced concrete components are defined in Table 4.1, where the density of reinforced concrete $\gamma_{RC} = 25.0\text{kN/m}^3$ is used. The top part of the consoles will not be included in the calculations since it is assumed that this structural element will be removed in a renovation.

Table 4.1: Characteristic self weight, structural components.

Element	Dimension	Self weight $g_{k,structure}$
Wall	$t = 150\text{mm}$	3.8kN/m^2
Gable wall	$t = 150\text{mm}$	3.8kN/m^2
Floor	$t = 120\text{mm}$	3.0kN/m^2
Balcony plate	$t = 125\text{mm}$	3.1kN/m^2
Console, bottom part	$h \text{ x } b = 480\text{x}150\text{mm}$	1.8kN/m
Gable column	$b \text{ x } t = 200\text{x}330\text{mm}$	1.7kN/m

The self weights of the non-structural components are estimated and the values are defined in Table 4.2. No extra self weight is added to the walls, balcony plates and consoles since it is assumed that these elements are only painted.

Table 4.2: Characteristic self weight, non-structural elements.

Element	Self weight, others $g_{k,other}$
Gable wall (insulation, cladding)	2.0kN/m^2
Floor (installations, ceiling, light walls)	1.5kN/m^2
Roof (wooden structure, insulation, cladding, installations, ceiling)	3.0kN/m^2

The total sum of the self weights are defined in Table 4.3. The total sum is the sum of the structural elements and the non-structural elements. The weights are given as line loads for elevated elements, such as walls, and as uniformly distributed loads for plan elements.

Table 4.3: Total characteristic self weight (structure + others).

Element	Dimension	Self weight, total $g_{k,tot}$
Wall	$h = 2.80$ m	10.5kN/m
Gable wall	$h = 2.80$ m	16.1kN/m
Floor	-	4.5kN/m^2
Roof (incl. floor plate)	-	6.0kN/m^2
Balcony plate	-	3.1kN/m^2
Console	-	1.8kN/m
Gable column	-	1.7kN/m

Finally the self weights of the new non-structural elements will be defined. The elements will be used in the renovation proposals and they consist of a new facade, inner walls, flooring and balconies, the self weights are defined in Table 4.4. The new floor consist of a balcony plate and the non-structural floor elements, therefore the weight is slightly larger compared to the floor inside the building. It is chosen to keep the original concrete balcony plates in the new floor design, but if the large weight is a problem the concrete plates can be substituted with lighter plates. The new facade and inner wall will be light structures and the self weights are estimated in Appendix C.2. It is estimated that a steel balcony will weight approximately 100kg/m^2. This is a large self weight for a balcony, but it is assumed that the balcony must be very strong and stable to endure the harsh climatic conditions.

Table 4.4: Characteristic self weight, new non-structural elements.

Element	Self weight, new $g_{k,new}$
Floor (concrete plate, installations, ceiling)	4.6kN/m^2
Facade, $h = 2.8$m (including windows)	4.3kN/m
Inner wall	0.8kN/m
Steel balcony	1.0kN/m^2

4.3.2 Live Load

Live loads are imposed loads that occur due to the movement of people or things. For buildings consisting of several floors the imposed load from several floors acting on a wall can be reduced with a factor α_n. Since the residential blocks do not consist of many floors, usually less than five, it is chosen not to take the reduction factor into account in the calculations. The building functions as a residential building and therefore the category of use is Category A according to *DS/EN 1991-1-1*.The roof of the building is only accessible for normal maintenance and repair, therefore the category for the roof is Category H. The imposed loads for Category A and H, defined by *EN 1991-1-1 GL NA* [23], are shown in Table 4.5.

Table 4.5: Imposed loads according to the National Annex of Greenland [23].

Category	q_k [kN/m^2]	Q_k [kN]
Category A - housing		
- A1 residential area and internal access roads	1.5	2.0
- A4 stairs	3.0	2.0
- A5 balconies	2.5	2.0
Category H - roof	0.0	1.5

4.3.3 Snow Load

According to *EN 1991-1-3 GL NA* [24] exceptional snow loads are not used in Greenland and exceptional drifting of snow does not occur either. Therefore the design situation is normal according to *EN 1991-1-3* [25]. The snow load on roofs for persistent and transient design situations is defined by:

$$s = \mu_i C_e C_t s_k \tag{4.1}$$

where
μ_i is the snow load shape coefficient,
C_e is the exposure coefficient,
C_t is the thermal coefficient,
s_k is the characteristic value of snow load on the ground.

The snow load shape coefficient μ_i is for a pitched roof with an inclination $\alpha < 30°$ given by $\mu_1 = 0.8$. The topography in Greenland is assumed to be normal, since nothing else is stated in the GL NA, therefore the exposure coefficient is $C_e = 1.0$. The roof of the building is a cold roof, meaning that there is no significant heat loss through the roof that would otherwise result in melting of the snow. For this reason the thermal coefficient is $C_t = 1.0$. The characteristic value of snow load on the ground is $s_k = 1.8\text{kN/m}^2$ according to GL NA. The magnitude of the undrifted snow load on the roof and on the balconies then becomes:

$$s = \mu_i s_k = 0.8 \cdot 1.8\text{kN/m}^2 = 1.44\text{kN/m}^2 \tag{4.2}$$

In the calculations the snow load determined for the roof will also be applied to the balconies. Usually two snow load situations are taken into account, namely the undrifted situation and the case where snow has drifted. In the calculations the situation with drifted snow will not be considered. The reason for this choice is that when the snow load is used for the console calculations the largest vertical load will be most critical and that is the case with undrifted snow. For the calculations of the bearing walls a load combination with dominating wind will be most critical and in this load combination the snow load is not included.

4.3.4 Wind Load

The fundamental basic wind velocity $v_{b,0}$ and the basic velocity pressure q_b are given in *EN 1991-1-4 GL NA* [26] for most cities and villages in Greenland. According to the Greenlandic NA simplified rules must be applied for buildings with a height below 20m, since the terrain in Greenland varies a lot and therefore it does not make sense to let the wind load vary with the height. The buildings investigated in this project are all below 20m and therefore these simplified rules will be applied. The mean wind velocity v_m therefore becomes equal to the fundamental basic wind velocity $v_{b,0}$ and the peak velocity pressure q_p is equal to the basic wind pressure q_b, all the values are independent of the height above ground level. Values of the mean wind velocity v_m and the peak velocity pressure q_p for chosen cities in Greenland, see Table 2.1, are summed up in Table 4.6. A peak velocity pressure of $q_p = 1.6\text{kN/m}^2$ will be used in the further calculations since this value represents almost all cities where the residential blocks can be found.

Table 4.6: Peak velocity pressures and mean wind velocities for cities in Greenland (building height < 20m) [23].

City	q_p [kN/m^2]	$v_{b,0}$ [m/s]
Qaqortoq	1.6	40
Paamiut	1.6	40
Nuuk	1.6	40
Maniitsoq	1.2	35
Sisimiut	1.2	35
Ilulissat	1.2	35

The wind load can either be perpendicular to the facade or the gable wall, and the magnitude of the load depends on the geometry of the building. The calculations of the wind pressures can be found in Appendix C.3 and it turns out that five different combinations of wind pressure applies. Four cases for wind perpendicular to the facade and one case with wind perpendicular to the gable, see below. The terms suction and pressure define the wind conditions on the roof of the building.

- Wind LC1 $\perp$ facade: Suction - Suction
- Wind LC2 $\perp$ facade: Pressure - Pressure
- Wind LC3 $\perp$ facade: Suction - Pressure
- Wind LC4 $\perp$ facade: Pressure - Suction
- Wind LC5 $\perp$ gable: Suction

4.3.5 Seismic Load

Seismic load can occur during an earthquake or during other types of shock, e.g. the impact of a big truck. A seismic load is a horizontal load that depends on the mass of the building. The partial coefficient (importance factor) on seismic load is set to $\gamma_I = 1.0$ according to *DS/EN 1998-1* [27], since the building is an ordinary building. The design seismic load involves the self weight and the live load, it is defined by:

$$A_{Ed} = 1.5\% \left(\sum G_{k,j} + \sum \psi_{2,i} Q_{j,i}\right) \tag{4.3}$$

The seismic load is calculated for each level of the building and it acts as a horizontal line load along each floor. The seismic load can both act in a direction parallel to the facade or the gable wall, which results in different load sizes. Basically the seismic load is defined as the SLS quasi-permanent load which is multiplied with 1.5%. In order to simplify the calculations the self weight and live load for one floor has been calculated based on Table 4.3 and 4.5 and as an extra precaution a point load of 40kN is applied to each console. The calculations are shown in Appendix C.4 and the seismic load for one level is given by:

$$A_{Ed} = 1.5\% \, (3562\text{kN} + 0.2 \cdot 480\text{kN}) = 55\text{kN} \tag{4.4}$$

The seismic load is divided by either the length of the gable (12.14m) or the length of the facade (33.3m). The linear loads for each floor is defined in Table 4.7. It is assumed that the seismic load acts as a linear horizontal load along each floor, for the ground floor and the roof the load will be divided by two.

Table 4.7: Seismic loads A_{Ed}.

Level	Seismic load A_{Ed} [kN/m] perpendicular to gable	Seismic load A_{Ed} [kN/m] perpendicular to facade
4	2.3	0.8
3	4.5	1.7
2	4.5	1.7
1	4.5	1.7
0	2.3	0.8

4.3.6 Load Combinations

The following load combinations are defined in accordance with *DS/EN 1990* [28]. The consequence class is CC2, which is applicable for residential buildings. This means that the factor concerning reliability differentiation is $K_{FI} = 1.0$. The Ψ-factors are given in Table 4.8 according to *EN 1990 GL NA* [29]. It should be noted that the values for snow load are, according to the Greenlandic NA, smaller than the recommended values for other Nordic countries.

Table 4.8: Ψ-factors according to *EN 1990 GL NA*.

Action	Ψ_0	Ψ_1	Ψ_2
Category A - domestic, residential areas	0.5	0.3	0.2
Snow load	0.3	0.2	0.0
Wind load	0.3	0.2	0.0

The partial factors for each load combination is given in Table 4.9. It should be noted that a load combination with full snow load and live load could occur on the balconies, but this scenario is not further investigated since it is not defined in the Eurocodes.

Table 4.9: Partial factors according to *EN 1990 GL NA*.

Dominating action	Self weight γ_G	Variable load $\gamma_{Q,A}$	Snow load $\gamma_{Q,s}$	Wind load $\gamma_{Q,w}$
Self weight	1.2	-	-	-
Variable load	1.0	1.5	0.45	0.45
Snow load	1.0	1.5	1.5	0.45
Wind load	1.0	1.5	0.0	1.5

The structure will be checked for five different load combinations, where four of them apply in ultimate limit states (ULS) and one in serviceability limit state (SLS). The load combinations are defined below.

Load combination 1: dominating self weight (structure and others)

$$E_d = \sum_{j\geq 1} \gamma_G G_{k,j} \quad j \geq 1 \tag{4.5}$$

Load combination 2: dominating variable load

$$E_d = \sum_{j\geq 1} \gamma_G G_{k,j} + \gamma_{Q,A} Q_{k,A} + \gamma_{Q,s} \Psi_{0,s} Q_{k,s} + \gamma_{Q,w} \Psi_{0,w} Q_{k,w} \quad j \geq 1 \tag{4.6}$$

Load combination 3: dominating snow load

$$E_d = \sum_{j\geq 1} \gamma_G G_{k,j} + \gamma_{Q,s} Q_{k,s} + \gamma_{Q,w} \Psi_{0,w} Q_{k,w} + \gamma_{Q,A} \Psi_{0,A} Q_{k,A} \quad j \geq 1 \tag{4.7}$$

Load combination 4: dominating wind load

$$E_d = \sum_{j\geq 1} \gamma_G G_{k,j} + \gamma_{Q,w} Q_{k,w} + \gamma_{Q,A} \Psi_{0,A} Q_{k,A} \quad j \geq 1 \tag{4.8}$$

Load combination 5: seismic load combination (SLS)

$$E_d = \sum_{j\geq 1} G_{k,j} + A_{Ed} + \Psi_{2,i} Q_{k,A} \quad j \geq 1 \tag{4.9}$$

4.4 Analysis of Consoles

In relation to the renovation proposals concerning a new facade and balcony design an analysis of the consoles will be performed. The bearing capacity of the consoles will be investigated for each design proposal where the condition of the consoles will be taken into account.

4.4.1 Console Design

The load bearing capacity of a long and a short console will be determined. The bearing capacity will be determined for a console in good condition with intact reinforcement and for a console in a bad condition with corroded reinforcement. For consoles with corroded reinforcement, the bearing capacity will be determined by reducing the diameter of the reinforcement. Figure 4.9 and 4.10 shows the reinforcement design in a long console and a short console. The drawings are standard drawings from GTO and as it can be seen from the drawings the reinforcement design is very similar for the two consoles. The only difference besides the length of the consoles is that the long console has four longitudinal reinforcement bars in the top of the cross section, whereas the short console only has two longitudinal reinforcement bars in the top of the cross section.

The dimensions of the original intact consoles, long and short, are summed up in Table 4.10. All dimensions are based on the GTO drawings in Figure 4.9 and 4.10. Despite the varying cover layer thicknesses and diameters measured in the field testing, see section 3.4, the calculations will be based on the original design dimensions provided by GTO.

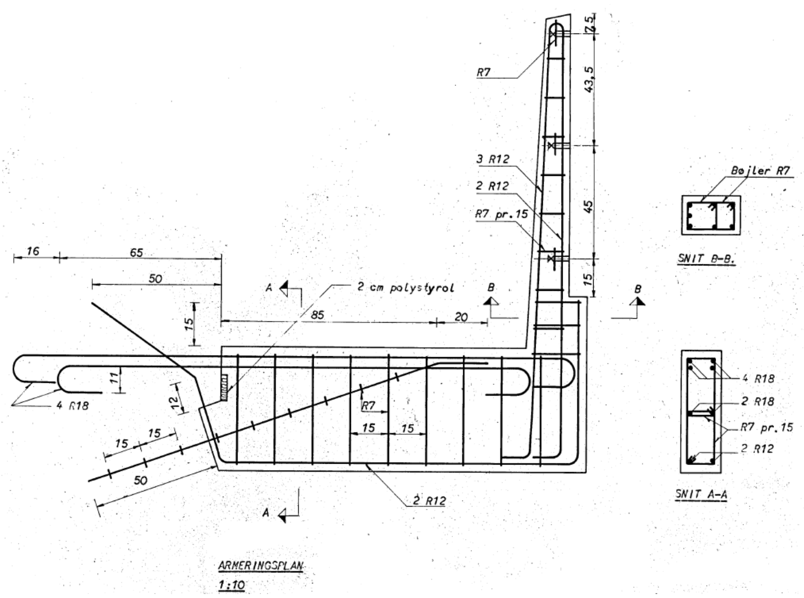


Figure 4.9: GTO drawing, long console, length $l_c = 1.45$m.

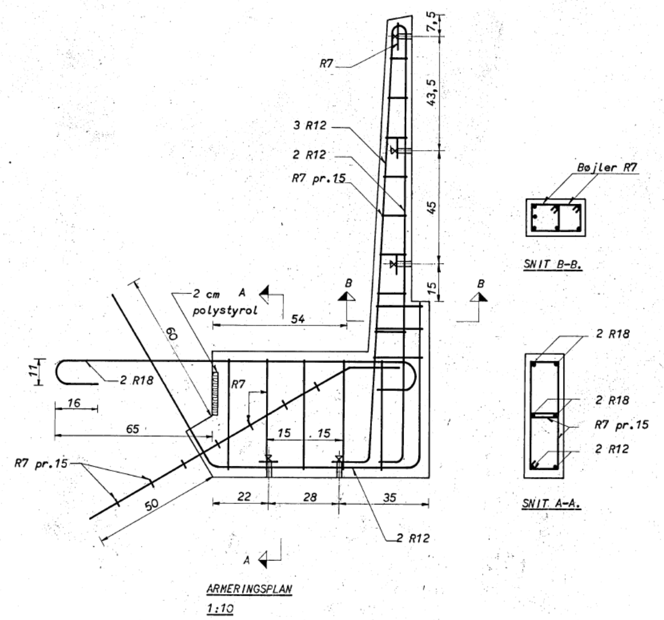


Figure 4.10: GTO drawing, short console, length $l_c = 0.85$m.

Table 4.10: Console dimensions, intact reinforcement.

Consoles with intact reinforcement		Long console	Short console
Height [mm]:	h	480	480
Width [mm]:	b	150	150
Console length [mm]:	l_c	1450	850
Diameter, tension reinforcement [mm]:	d_s	18	18
Diameter, compression reinforcement [mm]:	d_{sc}	12	12
Diameter, stirrups [mm]:	d_{sw}	7	7
Number of rebars, tension [-]:	n_s	4	2
Number of rebars, compression [-]:	n_{sc}	2	2
Area, tension reinforcement [mm^2]:	A_s	1018	509
Area, compression reinforcement [mm^2]	A_{sc}	226	226
Area, stirrups (x2) [mm^2]	A_{sw}	77	77
Distance to tension reinforcement [mm]:	d	423	441
Distance to compression reinforcement [mm]:	d_0	36	36
Distance between stirrups [mm]:	s	150	150
Cover layer to main reinforcement [mm]:	c	30	30

Since the corrosion rate is a very uncertain parameter, the bearing capacity of the consoles will be determined as a function of the reinforcement diameter. If x is the amount of steel that has corroded, then the new diameter becomes $d - 2x$, see Figure 4.11 where the corroded reinforcement is illustrated. It is assumed that the the corrosion thickness x is identical for the tension reinforcement, compression reinforcement and the stirrups, even though the distance from the surface to the reinforcement varies. For consoles with corroded reinforcement the reinforcement dimensions as a function of x are given in Table 4.11. The reduced diameters will also result in reduced reinforcement areas A. The rest of the parameters are unchanged and can be seen in Table 4.10.

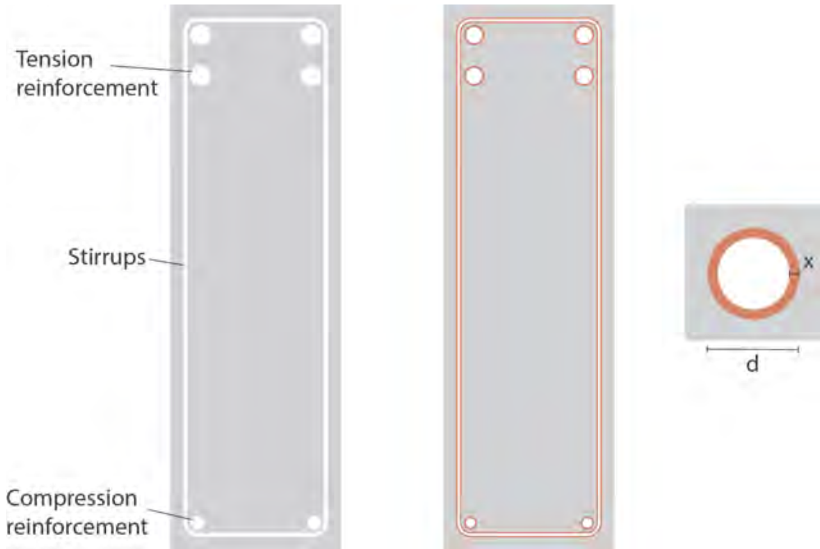


Figure 4.11: Cross section of a long console with un-corroded and corroded reinforcement.

Table 4.11: Console dimensions, corroded reinforcement.

Consoles with corroded reinforcement		Long console	Short console
Diameter, tension reinforcement [mm]:	d_s	$18-2x$	$18-2x$
Diameter, compression reinforcement [mm]:	d_{sc}	$12-2x$	$12-2x$
Diameter, stirrups [mm]:	d_{sw}	$7-2x$	$7-2x$

4.4.2 Load Cases

Three design proposals will be investigated and for each design the loads acting on the consoles will be determined. The consoles are fixed to the bearing walls and therefore they will act as a cantilevered beam. The consoles will be subjected to a uniform line load p and a point load P at the end of the beam, see Figure 4.12. Depending on the design situation the uniform line load can consist of the self weights from the floor, balcony, light inner wall, and facade as well as the variable loads acting on the floor and balcony. The point load can consist of the weight of the facade as well as the weight of the balcony that extends beyond this point.

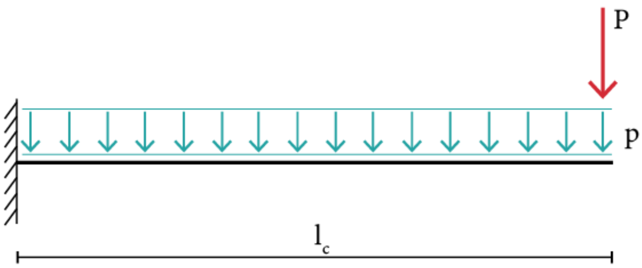


Figure 4.12: Uniform load p and point load P on cantilevered beam.

The consoles will be checked for three different load combinations where the dominant action is either self weight, variable load or snow load. Wind load will in all cases be neglected and since it is a horizontal load it will be on the safe side to neglect this load. When the linear load p and the point load P are determined for a given load combination then the shear force and the maximum moment can be determined by superposition:

$$V_{Ed} = pl_c + P \tag{4.10}$$

$$M_{Ed} = 1/2pl_c^2 + Pl_c \tag{4.11}$$

A torsion moment can occur from uneven loading caused by different magnitudes of loads on each side of the console. A linearly distributed torsion load p_T and a torsion point load P_T is determined and it is assumed that the torsion loads act at the edge of the console, therefore the eccentricity from the loading is $b/2$ from the center line of the console. The torsion moment can thereby be defined as:

$$T_{Ed} = (p_T l_c + P_T)\frac{b}{2} \tag{4.12}$$

For each design proposal the design shear force, moment and torsion moment will be determined for the three load combinations and the utilization of the console will be based on the worst load combination. The worst case load combination might vary for the 3 different design proposals.

4.4.3 Bearing Capacity

The bearing capacity of a console will now be determined according to the Eurocodes [30]. Usually consoles are very short and the length is less than twice the height $l < 2h$. Besides that, consoles per definition do not contain stirrups [31]. Since these requirements are not fulfilled, except for a short console with $l = 0.85\text{m} < 2h = 0.94\text{m}$, it is chosen to base the calculations on the assumption that the consoles act as cantilevered beams. In the calculations it is assumed that the concrete cover layer is cracked which means that the longitudinal reinforcement will take all the tension from the loading. Figure 4.13 illustrates the stresses and symbols used in the calculations. Since the consoles are cantilevered the top reinforcement will be subjected to tension and the bottom reinforcement will be subjected to compression, which can also be seen from Figure 4.13. The consoles will be checked for a combination of bending, shear, and torsion and the anchoring of the consoles to the bearing walls will also be checked.

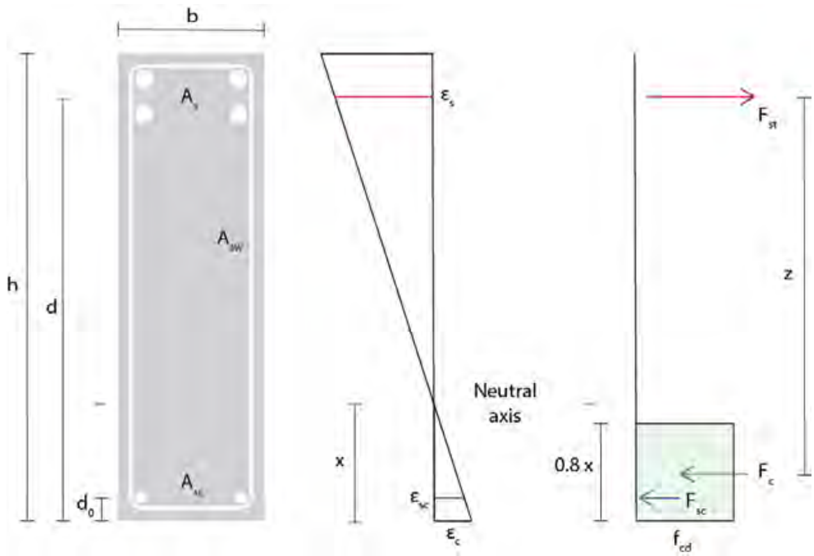


Figure 4.13: Calculation symbols.

Bending

The cross section of the console contains both tension reinforcement and compression reinforcement. The moment capacity will now be determined taking the compression reinforcement into account, which will give a larger moment capacity. It is assumed that the concrete is cracked, meaning that the concrete is not subjected to tension and therefore only the tension reinforcement is subjected to tension. First it should be checked whether the stress in the compression reinforcement is smaller than the design yield strength:

$$\sigma_{sc} = \varepsilon_{sc} E_s \leq f_{yd} \tag{4.13}$$

where:

$$\varepsilon_{sc} = \varepsilon_{cu3} \frac{x - d_0}{x} \tag{4.14}$$

Since the design yield strength of the steel is very small, $f_{yd} = 187.5\text{MPa}$ for $d \geq 16\text{mm}$, the inequality is not satisfied for any load case. Therefore the design yield strength of the steel f_{yd} will be used in order to determine the height of the compression zone x:

$$x = \frac{1}{0.8} \cdot \frac{A_s f_{yd,_{d \geq 16\text{mm}}} - A_{sc} f_{yd,_{d \leq 16\text{mm}}}}{b f_{cd}} \tag{4.15}$$

The diameter of the tension and compression reinforcement are not identical and therefore two different values of the design yield strength f_{yd} is used, see section2.3. The height of the compression zone x has determined and it must now be checked whether the cross section is normally reinforced:

$$\varepsilon_{yd} < \varepsilon_s = \varepsilon_{cu3} \, \frac{d - x}{x} \tag{4.16}$$

If the above expression is fulfilled the moment capacity can be determined:

$$M_{Rd} = \lambda \; x \left(d - \frac{1}{2} \lambda \; x \right) b f_{cd} + A_{sc} f_{yd,_{d \leq 16\text{mm}}} (d - d_0) \tag{4.17}$$

The following constant parameters for concrete with strength C12-C50 are used in the calculations:

$$\lambda = 0.8 \quad , \quad \varepsilon_{cu3} = 0.35\% \quad , \quad \varepsilon_{yd} = f_{yd,_{d \geq 16\text{mm}}} / E_s = 0.094\%$$

Shear

The shear forces in the consoles will be transferred through the console to the bearing wall by the help of stirrups. First the concrete stress must be checked by the following inequality:

$$\sigma_c < \nu_v f_{cd} \tag{4.18}$$

Where the efficiency factor for shear is given by:

$$\nu_v = 0.58 + \frac{0.6 - 0.58}{5} f_{ck} = 0.64 \tag{4.19}$$

The concrete stress is defined by:

$$\sigma_c = \tau_{Ed} \left(\cot\theta + \frac{1}{\cot\theta} \right) \tag{4.20}$$

The concrete compression angle is chosen to be:

$$\cot\theta = 2.5 \tag{4.21}$$

The design shear stress is defined by:

$$\tau_{Ed} = \frac{V_{Ed}}{bz} \tag{4.22}$$

Where V_{Ed} is the actual shear force in the console. The inner moment arm z is then given by:

$$z = d\left(1 - \frac{1}{2}\omega\right) \tag{4.23}$$

Where ω is the actual degree of reinforcement:

$$\omega = \frac{A_s f_{yd,_{d\geq 16\text{mm}}}}{bdf_{cd}} \tag{4.24}$$

If equation (4.18), concerning the concrete stress, is satisfied the amount of shear reinforcement will be checked, which in this case are stirrups. The stirrups are in both types of consoles placed with a distance of:

$$s = 150\text{mm} \tag{4.25}$$

The following conditions must be satisfied:

$$s \leq 0.75d \tag{4.26}$$

$$s \leq 15.9\frac{A_{sw}}{b}\frac{f_{yk,_{d\leq 16\text{mm}}}}{\sqrt{f_{ck}}} \tag{4.27}$$

$$s \leq \frac{A_{sw}}{\tau_{Ed}b} f_{yd,_{d\leq 16\text{mm}}} \cot\theta \tag{4.28}$$

The shear capacity is then given by:

$$V_{Rd} = \tau_{Rd} b\ z \tag{4.29}$$

where:

$$\tau_{Rd} = \frac{A_{sw}}{s\ b}\ f_{yd,_{d\leq 16\text{mm}}} \cot\theta \tag{4.30}$$

Torsion

The torsion moment arising from uneven loading is usually not very large and therefore the utilization of the torsion capacity will be small.

The effective thickness of the console cross section is defined by:

$$t_{ef} = \max\left(\frac{A}{u}, 2\left(c + d_{sw} + \frac{d_{sc}}{2}\right)\right) \tag{4.31}$$

Where A is the total area:

$$A = bh \tag{4.32}$$

The total circumference u is given by:

$$u = 2b + 2h \tag{4.33}$$

The concrete cover to the stirrups is defined by:

$$c = 30\text{mm} - d_{sw} = 23\text{mm} \tag{4.34}$$

The cross section area within the mid line is given by:

$$A_k = (b - t_{ef})(h - t_{ef}) \tag{4.35}$$

The efficiency factor for torsion is defined as:

$$\nu_t = 0.7\left(0.7 - \frac{f_{ck}}{200}\right) = 0.44 \tag{4.36}$$

The torsion bearing capacity for a console is then defined by:

$$T_{Rd} = \frac{2\ \nu_t f_{cd}\ t_{ef} A_k}{\cot\theta + (1/\cot\theta)} \tag{4.37}$$

Anchoring Length

The anchoring length for one reinforcement bar depends on the diameter as well as the design yield strength and it is given by:

$$l_{bd} = \frac{d_s f_{yd,_{d\geq 16\text{mm}}} \gamma_c}{9 f_{ctk,0.05}} \cdot \frac{1}{0.7} = 608\text{mm} \tag{4.38}$$

For all consoles, both long and short, the reinforcement has a minimum anchoring length of:

$$l_{b,Ed} = (650 + 110 + 160)\text{mm} = 920\text{mm} \tag{4.39}$$

This is the length of the reinforcement supported by the wall, including the length of the hooks, see Figure 4.14. The actual anchoring length should be greater than the design anchoring length:

$$l_{b,Ed} \geq l_{bd} \tag{4.40}$$

If the anchoring length was critical detailed calculations could be made in order to reduced the required anchoring length. However the anchoring length is sufficient for both types of consoles and therefore this matter will not be investigated further.

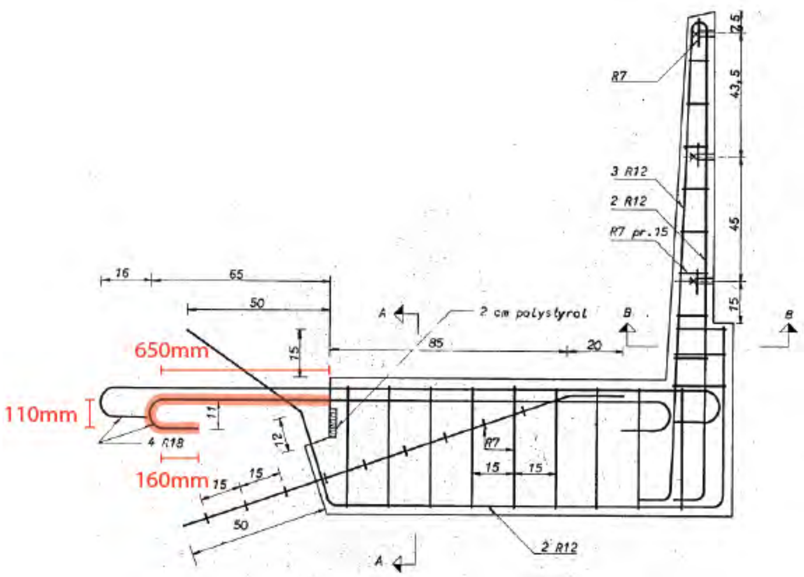


Figure 4.14: Illustration of minimum anchoring length.

Check

The longitudinal tension reinforcement is not used to obtain the shear force and therefore the moment capacity and shear capacity are not checked together. The moment and shear capacities are checked separately in combination with the torsion capacity. The following two checks must be made in order to determine the utilization of the console, where the largest utilization will be the crucial utilization:

$$\frac{M_{Ed}}{M_{Rd}} + \frac{T_{Ed}}{T_{Rd}} \leq 1 \tag{4.41}$$

$$\frac{V_{Ed}}{V_{Rd}} + \frac{T_{Ed}}{T_{Rd}} \leq 1 \tag{4.42}$$

4.5 Analysis of Bearing Walls

The effect of creating new openings in the bearing walls will be investigated systematically. The analysis of the bearing walls will be performed by the use of a finite element method (FEM) program. The program used is *Robot Structural Analysis Professional* provided by Autodesk. By using the FEM program *Robot* it is possible to run many calculations and it is possible to receive the results in a graphical manner. The desired results from *Robot* are stress maps showing σ_{xx}, σ_{yy} and σ_{xy}. The stresses will then be used in order to check whether the given reinforcement in the building is sufficient as well as the concrete strength. The amount of reinforcement will be checked around openings in the wall, in the area where the consoles are attached and in a plane wall area.

4.5.1 Wall Design

In level 1-3 all openings in the bearing walls are identical and they have the dimension bxh=0.81m x 2.22m. Openings in level 0 are very large and it is chosen not to add any new openings in this level. The main focus will therefore be walls in level 1-3 which is where the apartments are placed. All bearing walls with existing door openings are surrounded by a 2.7m and 3.6m module. Walls without door openings divide the apartments and it is chosen not to create openings in these walls since the apartment layout is well made already. Four different placements of door openings exist in the building and they have been named A, B, C and D. The openings are combined two and two, which means that opening A and B will occur in the same wall and opening C and D will be placed in the same wall. This means that the walls are identical throughout the building and throughout all the buildings of this type. Four different types of walls exist and they are walls with opening A and B, walls with opening C and D, walls in connection with the staircase with opening A and intact walls between the apartments. The existing openings and wall types are illustrated in Figure 4.15.

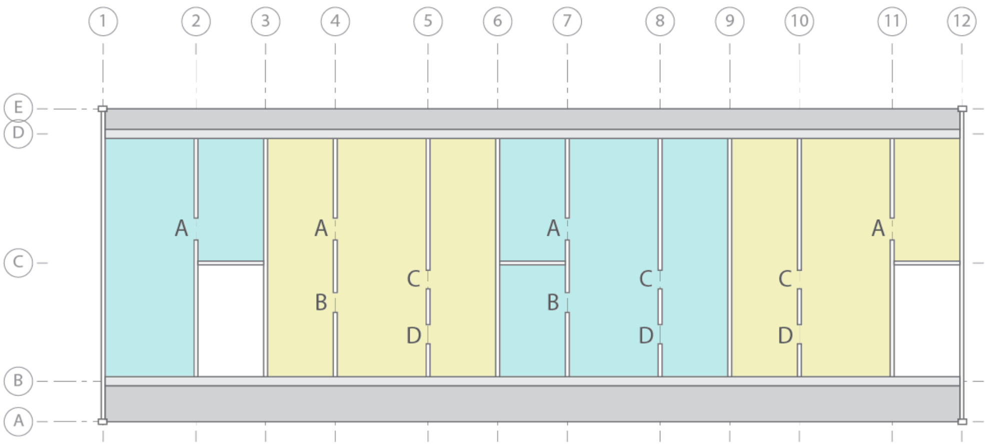


Figure 4.15: Existing openings in bearing walls.

Although the walls have the same type of openings the reinforcement is not necessarily identical. Table 4.12 describes the wall type for each bearing wall and Table 4.13 describes the amount of reinforcement in each wall type. It can be seen that some walls in level 3 do not contain any reinforcement (A0) and it can be seen that the reinforcement amount increases for the lower levels of the building. Besides the uniform reinforcement in the walls every single opening is surrounded by extra reinforcement and this reinforcement will be included in the calculations if needed.

Table 4.12: Wall type for each bearing wall and level.

Wall	2	3	4	5	6	7	8	9	10	11
Level 3	A1	A1	A0	A0	A1	A1	A0	A0	A0	A1
Level 2	A1	A1	A1	A1	A1	A1	A1	A1	A1	A1
Level 1	A1	A1	A2	A2	A2	A2	A1	A1	A2	A1
Level 0	A2	A2	A2	A2	A2	A2	A1	A2	A2	A2

Table 4.13: Reinforcement in the bearing walls.

Wall type	No. of net	Reinforcement	
		Vertical	Horizontal
A0	0	0	0
A1	1	T10/20	T8/13
A2	1	T10/10	T8/13
A3	1	T10/20	T10/20
A4	2	T10/25	T8/15
A5	1	T10/10	T10/10

4.5.2 FEM Model

The building modelled in *Robot* is exactly as block 12 and the original grid lines are used. In the following comments about the model will be presented in order to fully understand the model. The comments concern the geometry, the definition of loads and load combinations, as well as how to interpolate the results from *Robot*.

Geometry

- Door openings and other large openings in bearing walls are modelled and small openings for e.g. installations have not been taken into account.
- Consoles are modelled as walls with height $h = 480\text{mm}$ in order to assure a correct transfer of loads to the bearing walls.

- The floors are modelled as one floor per level since it is assumed that the floor is cast on top of the walls creating a rigid floor.
- The balcony plates are not modelled since they are not a part of the static system.
- The areas where the old stairs cases were placed have been modelled without a floor plate assuming that a new floor pate will not contribute to the static system.

Loads

- The structural self weight of the walls, gable walls, columns, consoles and floors are defined according to Table 4.3.
- The self weight of the consoles is added as a line load on the bottom edge of the console.
- A point load of 40kN is added at the end of each console and a factor of 1.0 is multiplied with this load. This is the maximum design load a console can carry which includes self weight (facade, floor, balcony), live load and snow load.
- Where floor plates are missing (due to old stair cases) the same loads are applied as if the plates existed, since it is assumed that a concrete floor plate will be installed.

Load Combinations

- LC2 Variable load: $1.0 \cdot G + 1.5 \cdot Q + 0.135 \cdot S + 0.135 \cdot W$
- LC3 Snow load: $1.0 \cdot G + 1.5 \cdot S + 0.135 \cdot W + 0.75 \cdot Q$
- LC4 Wind load X: $1.0 \cdot G + 1.5 \cdot W_X + 0.75 \cdot Q$
- LC4 Wind load Y (S-S): $1.0 \cdot G + 1.5 \cdot W_{Y,S-S} + 0.75 \cdot Q$
- LC4 Wind load Y (P-P): $1.0 \cdot G + 1.5 \cdot W_{Y,P-P} + 0.75 \cdot Q$
- LC4 Wind load Y (S-P): $1.0 \cdot G + 1.5 \cdot W_{Y,S-P} + 0.75 \cdot Q$
- LC4 Wind load Y (P-S): $1.0 \cdot G + 1.5 \cdot W_{Y,P-S} + 0.75 \cdot Q$
- LC5 Seismic X: $G + A_X + 0.2 \cdot Q$
- LC5 Seismic Y: $G + A_Y + 0.2 \cdot Q$

Interpretation of Results from Robot

In order to understand the output from *Robot* a simple model has been made. The model consist of a wall and two consoles, modelled with the same dimensions as in the actual building. A linear line load of p_z = 10kN/m has been added along the entire length of the consoles and wall. The global and local coordinate system is defined in Figure 4.16, which both applies to the simple model and the transverse walls in the building.

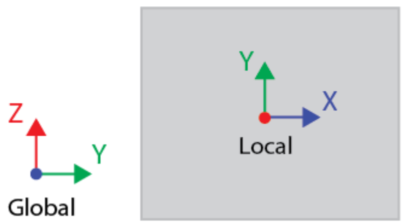


Figure 4.16: Global and local coordinate system for transverse walls.

The vertical stresses σ_{yy} are illustrated in Figure 4.17 and a check is now made to verify the results. The compression stress in the center of the wall is calculated, using the load p_z and wall thickness t = 150mm:

$$\sigma_{yy} = \frac{p_z}{t} = 0.066\text{MPa} \tag{4.43}$$

It can be seen that the compression stress calculated falls under the lightest blue color spanning 0.0MPa $> \sigma_{yy} = -0.066$MPa $\geq$ -0.15MPa, which is the color of the main part of the wall. The stresses are thereby verified.

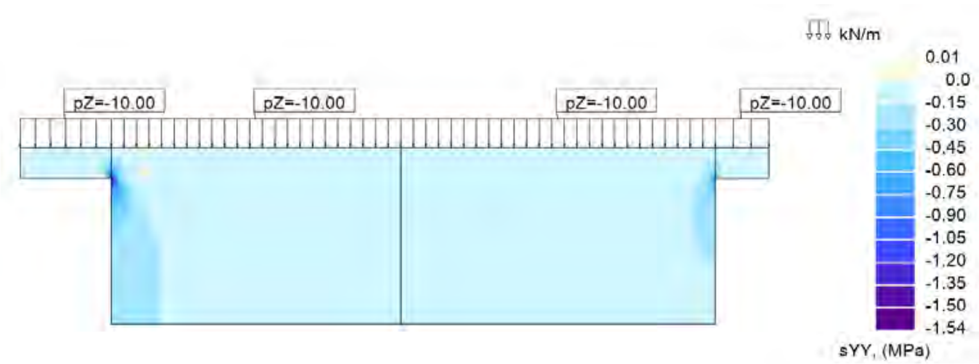


Figure 4.17: Illustration of vertical stresses σ_{yy}. Negative blue numbers = compression. Positive orange numbers = tension.

Figure 4.18 illustrates the horizontal stresses σ_{xx}. The tensile stress at the top of the left console will now be verified. The left console has a length of $l = 1.45\text{m}$ and a height $h = 0.48\text{m}$. According to Saint-Venant's principle the well known beam theory will only give correct stress results at a distance of approximately one beam height away from the load [32]. This means that the horizontal stress will be determined at a distance of $1.45\text{m} - 0.48\text{m} = 0.97\text{m}$ from the left corner of the console, which is approximately a distance of $2/3l$. The maximum tensile stress on the top of the console at this point will be found in order to compare it to the stress map. The maximum moment M and moment of resistance W are determined in this point:

$$M = \frac{1}{2} p_z (0.97\text{m})^2 = 4.70\text{kNm} \tag{4.44}$$

$$W = \frac{1}{6} th^2 = 0.0058\text{m}^3 \tag{4.45}$$

The maximum horizontal stress σ_{xx} is then found:

$$\sigma_{xx} = \frac{M}{W} = 0.81\text{MPa} \tag{4.46}$$

From the stress map it can be seen that this value corresponds very well with the stress determined by *Robot*. The stress falls under the second orange color 0.14MPa $\leq \sigma_{xx} = 0.81\text{MPa} < 0.86\text{MPa}$. The horizontal stresses are thereby verified.

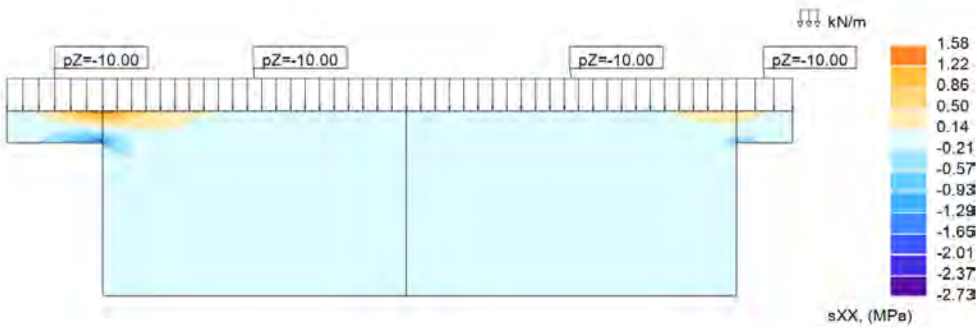


Figure 4.18: Illustration of horizontal stresses σ_{xx}. Negative blue numbers = compression. Positive orange numbers = tension.

4.5.3 Required Reinforcement

The required reinforcement in the bearing walls will be calculated based on the plasticity theory [33]. The walls are treated as disks and plane stress yield conditions are applied. The strength of the concrete is also checked and the design concrete strength

$f_{cd} = 10.3\text{MPa}$ must be reduced by an effectiveness factor ν. The effectiveness factor ν may on the safe side be taken as [33]:

$$\nu = 0.7 - \frac{f_{cd}}{200} = 0.65 \geq 0.5 \tag{4.47}$$

In case the reduced concrete strength is not sufficient compared to the concrete stresses a more precise value of the effectiveness factor can be used in order to increase the strength. E.g. for pure compression the effectiveness factor is $\nu = 1.0$, however this value will not be used in the calculations. Otherwise, as mentioned in section 2.3, the concrete strength can be increased with 25% because the concrete structure was build before 1990. Increasing the concrete strength of the walls would be acceptable, since the walls are placed in an indoor environment and thereby the condition of the walls is assumed to be good.

The amount of required reinforcement in the bearing walls depend on the local horizontal stresses σ_{xx}, vertical stresses σ_{yy} and shear stresses σ_{xy} which are provided by *Robot*. Depending on the size of the stresses and whether they are tensile or compressive stresses five different cases apply and the conditions are defined in Table 4.14.

Table 4.14: Case numbers and stress conditions.

<table>
<tr><th>Case</th><th colspan="2">Condition</th></tr>
<tr><td>1</td><td colspan="2">$\sigma_{xx} \geq -|\sigma_{xy}|$
$\sigma_{yy} \geq -|\sigma_{xy}|$</td></tr>
<tr><td>2</td><td rowspan="2">$\sigma_{xx} \leq \sigma_{yy}$
$\sigma_{xx} < -|\sigma_{xy}|$</td><td>$\sigma_{xx}\sigma_{yy} \leq \sigma_{xy}^2$</td></tr>
<tr><td>3</td><td>$\sigma_{xx}\sigma_{yy} > \sigma_{xy}^2$</td></tr>
<tr><td>4</td><td rowspan="2">$\sigma_{xx} \geq \sigma_{yy}$
$\sigma_{xx} > -|\sigma_{xy}|$</td><td>$\sigma_{xx}\sigma_{yy} \leq \sigma_{xy}^2$</td></tr>
<tr><td>5</td><td>$\sigma_{xx}\sigma_{yy} > \sigma_{xy}^2$</td></tr>
</table>

From Table 4.15 the required reinforcement and concrete stresses are defined for each case. The required reinforcement areas A_{sx} and A_{sy} can then be determined for each case and the concrete stresses can be checked, e.g. for case 1 the following applies:

$$A_{sx} = (\sigma_{xx} + |\sigma_{xy}|)\frac{t}{f_{yd}} \tag{4.48}$$

$$A_{sy} = (\sigma_{yy} + |\sigma_{xy}|)\frac{t}{f_{yd}} \tag{4.49}$$

$$\sigma_c = 2|\sigma_{xy}| \quad \leq \nu f_{cd} \tag{4.50}$$

Table 4.15: Reinforcement and concrete stresses for different cases.

Case	$(A_{sx} f_{yd})/t$	$(A_{sy} f_{yd})/t$	$\sigma_c(\leq \nu f_{cd})$
1	$\sigma_{xx} + \lvert\sigma_{xy}\rvert$	$\sigma_{yy} + \lvert\sigma_{xy}\rvert$	$2\lvert\sigma_{xy}\rvert$
2	0	$\sigma_{yy} + \dfrac{\sigma_{xy}^2}{\lvert\sigma_{xx}\rvert}$	$\lvert\sigma_{xx}\rvert\left(1+\left(\dfrac{\sigma_{xy}}{\sigma_{xx}}\right)^2\right)$
3	0	0	$\left\lvert\frac{1}{2}(\sigma_{xx}+\sigma_{yy}) - \sqrt{\frac{1}{4}(\sigma_{xx}-\sigma_{yy})^2 + \sigma_{xy}^2}\right\rvert$
4	$\sigma_{xx} + \dfrac{\sigma_{xy}^2}{\lvert\sigma_{yy}\rvert}$	0	$\lvert\sigma_{yy}\rvert\left(1+\left(\dfrac{\sigma_{xy}}{\sigma_{yy}}\right)^2\right)$
5	0	0	$\left\lvert\frac{1}{2}(\sigma_{xx}+\sigma_{yy}) - \sqrt{\frac{1}{4}(\sigma_{xx}-\sigma_{yy})^2 + \sigma_{xy}^2}\right\rvert$

4.6 Analysis of Adding an Extra Floor

The effect of adding an additional floor will be investigated. For reasons of simplicity an extra load will be added to the *Robot* model that corresponds to the weight of an additional floor. The stresses in the walls will then be compared to stresses in the case without an extra floor. If it turns out that the extra load does not make a big difference to the stresses then the situation will not be investigated further. However if the stresses turn out to be more critical then a detailed analysis will be made where e.g. the extra wind load will be taken into account. The analysis will be performed for the design proposal with new openings in the bearing walls.

4.7 Results for Consoles

The results from the analysis of the bearing capacity of the consoles will now be presented. The consoles have been examined in regards to design proposal 1, 2 and 3 concerning original balconies, built-in balconies and external balconies. The consequences of corroded reinforcement has also been taken into account in the results.

4.7.1 Bearing Capacity of Consoles

First the bearing capacity of a long and a short console will be determined according to section 4.4.3. The geometry used is defined in Table 4.10 for a console with intact reinforcement. The bearing capacity of a console, which includes the moment, shear, and torsion capacity, is given in Table 4.16. It can be seen from the results that the moment capacity for a long console is almost twice as big as the moment capacity for a short console. This is due to the fact that the long consoles contain twice the amount of longitudinal tension reinforcement compared to the short consoles. The shear capacity is slightly larger for a short console, but very similar to the shear capacity for a long console. The reason for this similarity is that the consoles contain the same amount of stirrups per unit length. The torsion capacity is the same for both consoles, since it is determined from the concrete cross section and stirrups, that are identical in both cases.

Table 4.16: Bearing capacity of intact consoles.

	Long console	Short console
Moment capacity M_{Rd} [kNm]:	72.2	39.7
Shear capacity V_{Rd} [kN]:	90.8	103.1
Torsion capacity T_{Rd} [kNm]:	7.15	7.15

The bearing capacity will now be determined for a cross section with reduced reinforcement in order to investigate the effect of corrosion. The results are illustrated in Figure 4.19 where the moment, shear, and torsion capacity are shown in relation to the diameter of the tension reinforcement. It has been chosen to illustrate the results, as a function of the main tension reinforcement diameter, for simplicity reasons. All the following results, where corrosion has been taken into account, will also be presented for this diameter. The capacities are illustrated for a short and a long console, with dashed and full lines respectively.

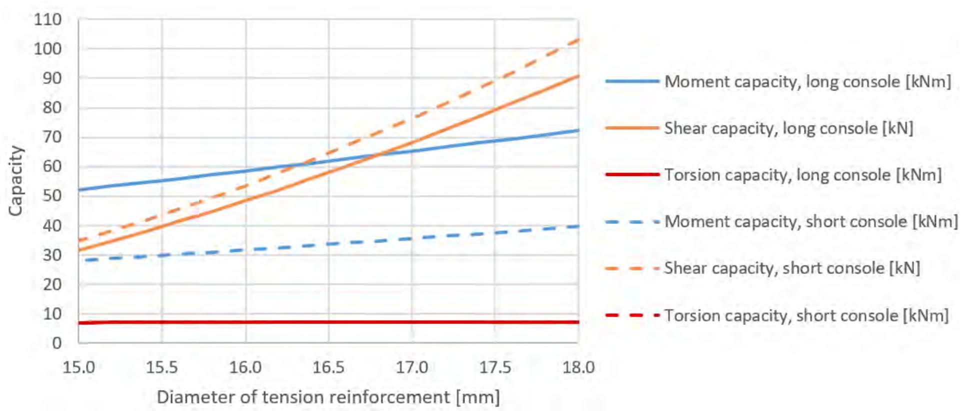


Figure 4.19: Moment, shear and torsion capacity for a long and short console compared to the diameter of the tension reinforcement.

From the graph it can be seen that the capacity of a console varies almost linearly with the diameter size. The moment capacity (blue) is reduced with a small inclination, whereas the shear capacity (orange) is reduced with a large inclination when the diameter is decreased. This is due to the fact that the shear capacity depends on the cross sectional area of the stirrups. The diameter of the stirrups (7mm) is much smaller than the diameter of the longitudinal reinforcement (18mm) which means that the reinforcement area of the stirrups is decreasing a lot faster compared to the reinforcement area of the longitudinal reinforcement. The graph implies that for large diameters the moment capacity will define the bearing capacity and for small diameters the shear capacity will most likely be the defining factor in relation to the checks described in section 4.4.3. The torsion capacity does not depend on the diameters of the reinforcement.

4.7.2 Proposal 1 - Original Balconies

For design proposal 1 the placement of the facade is the original placement which means that the consoles are not covered by the facade and they will therefore only carry the weight of the balconies. In the calculations the balcony plates will be made of concrete like the existing plates and the weight of a railing will be neglected. First the utilization will be determined for the existing size of the balconies and then it will be investigated how large the balcony plates can be until the consoles are fully utilized. Both a long and a short console will be investigated in both cases. The console that carries the most weight is surrounded by a 3.6m and a 2.7m wide concrete plate, meaning that the length of the loaded area is $l_p = 1/2(3.6\text{m} + 2.7\text{m}) = 3.15\text{m}$. The console will be subjected to a uniform line load consisting of the self weight from the console and the concrete plates, see Figure 4.20.

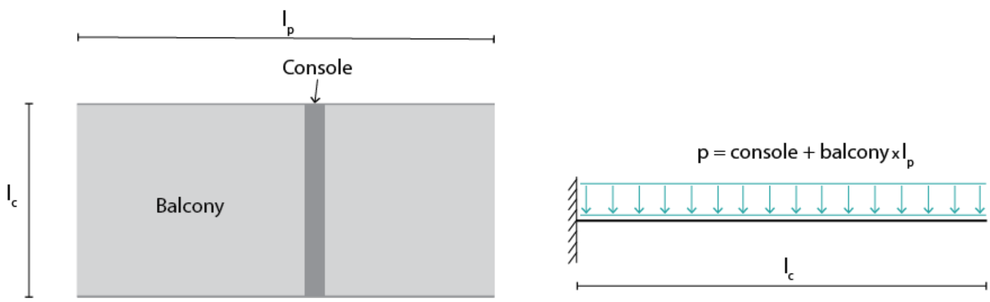


Figure 4.20: Illustration of loads for design proposal 1, plan view and cantilevered beam.

The design loads are calculated for load combination LC1, LC2 and LC3 and the results are given up in Appendix C.5.1. For both a long and a short console the worst load combination is LC3 where snow load is dominating. The design loads for LC3 are summed up in Table 4.17.

Table 4.17: Design loads for proposal 1 (LC3) - original balconies.

	Long console	Short console
Moment M_{Ed} [kNm]:	25.6	8.8
Shear force V_{Ed} [kN]:	35.3	20.7
Torsional moment T_{Ed} [kNm]:	0.4	0.2

For the given design loads in Table 4.17 the utilization of both consoles have been calculated and the results are given in Table 4.18. From the results it can be seen that the consoles are less than 50% utilized which is a very small utilization. In the calculations of the bearing capacity of the consoles, see section 4.4.3, it is assumed that the concrete in the top of the cross section has cracked in order for the reinforcement to yield. For this loading situation the concrete might not have cracked, however, with the larger loads that will be added to the consoles in the different proposals, the concrete will most likely crack. For design proposal 1 cracks will be a problem, since the consoles are placed in the outdoor environment. Cracks will not be critical, in terms of corrosion of the reinforcement, for the two other proposals since the consoles will be placed inside the building envelope.

Table 4.18: Utilization results - original balconies.

	Long console	Short console
Utilization	44%	25%

Although there is a risk of cracks and corroded reinforcement an extra load will be determined in order for the consoles to be 100% utilized. In Table 4.19 an extra point load P is given, which is placed at the end of the consoles. The consoles are 100% utilized when the extra load P is applied. The equivalent extra depth of the concrete balcony is also defined in Table 4.19. A load combination with dominant snow load has been taken into account when calculating the extra balcony area. If the concrete plates where substituted with balconies of a lighter material the balcony area could be increased even further. It should be noted that railings have not been included in the calculations.

Table 4.19: Extra capacity 100% utilization - original balconies.

	Long console	Short console
Extra point load P	18.3kN	18.0kN
Equivalent extra depth of balcony	0.75m	0.74m

4.7.3 Proposal 2 - Facade on the Outside of the Consoles

For design proposal 2 and 3 the new facade will be placed on the outside of the consoles. A console will carry the self weight of the console, a light inner wall, the floor and a new facade at the end of the console, which is illustrated in Figure 4.21.

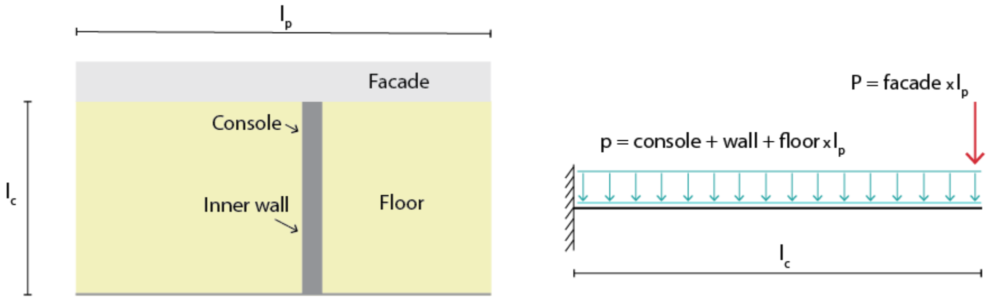


Figure 4.21: Illustration of loads for design proposal 2 with a facade, plan view and cantilevered beam.

The utilization will be determined for a short and a long console, both surrounded by a 2.7m and a 3.6m module. The consoles will be checked for load combination LC1 and LC2, for dominating self weight and live load respectively, see Appendix C.5.2. The loads are very similar for the two load combinations and it is therefore chosen that the worst load combination is LC1 with dominant self weight. The reason for this choice it that the self weight of the new elements is the most uncertain parameter. Therefore the self weight might be larger than assumed, which will cause larger design loads. The design loads for LC1 are summed up in Table 4.20.

Table 4.20: Design loads for proposal 2 (LC1) - new facade.

	Long console	Short console
Moment M_{Ed} [kNm]:	45.1	21.2
Shear force V_{Ed} [kN]:	46.1	33.7
Torsional moment T_{Ed} [kNm]:	0.4	0.3

The utilizations of a long and short console based on the design loads are defined in Table 4.21. The maximum utilization is 70%,for a long console, and it is therefore possible to increase the weight of the new facade if needed. From the results is can also be concluded that keeping the heavy concrete plates in the new floor design will not be a problem.

Table 4.21: Utilization results - new facade.

	Long console	Short console
Utilization	70%	58%

Figure 4.22 shows the utilization of a long and a short console compared to the diameter of the tension reinforcement. For a console with intact reinforcement the moment capacity will define the crucial utilization. For corroded reinforcement, and thereby smaller diameters, the shear capacity will result in the most critical utilization.

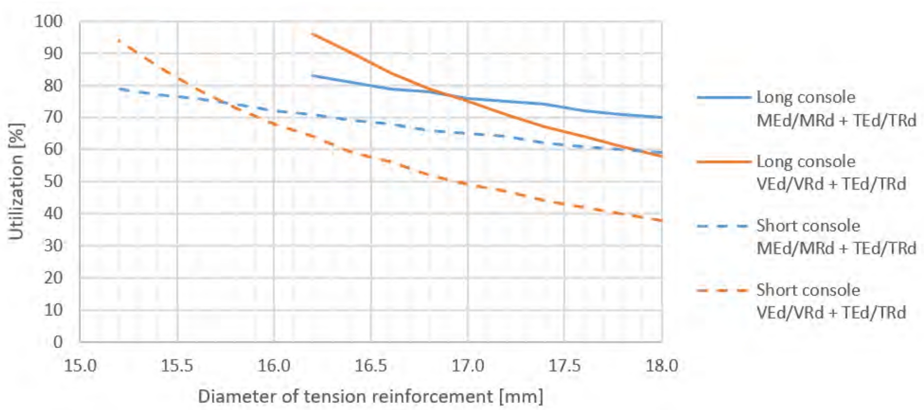


Figure 4.22: Utilization of long and short consoles as a function of the diameter of the tension reinforcement.

The minimum allowable diameter of the tension reinforcement, when the console is 100% utilized, is defined in Table 4.22. It should be noted that the diameter reduction of the stirrups is most critical, however for reasons of simplicity, the results are shown for the tension reinforcement.

Table 4.22: Diameter of longitudinal reinforcement for 100% utilization - new facade.

	Long console	Short console
Diameter, 100% utilization	16.2mm	15.2mm

4.7.4 Proposal 2 - Built-in Balconies

For design proposal 2 the facade is moved to the outside of the consoles and in the wide modules of 3.6m built-in balconies will be installed. In the design proposal built-in balconies are only placed on the facade with short consoles, but the calculations will be performed for both short and long consoles. The existing concrete balconies will be replaced by new light weight balconies, in order to obtain a larger balcony area. The size of the balconies will be determined for consoles with intact and corroded reinforcement. A console will carry the self weight of the console itself and the facade along the console, as well as the self weight of the floor and the facade spanning the length of $l_f = 2.7\text{m}/2 = 1.35\text{m}$ on one side and on the other side the weight of half the balcony spanning $l_b = 3.6\text{m}/2 = 1.8\text{m}$, see Figure 4.23.

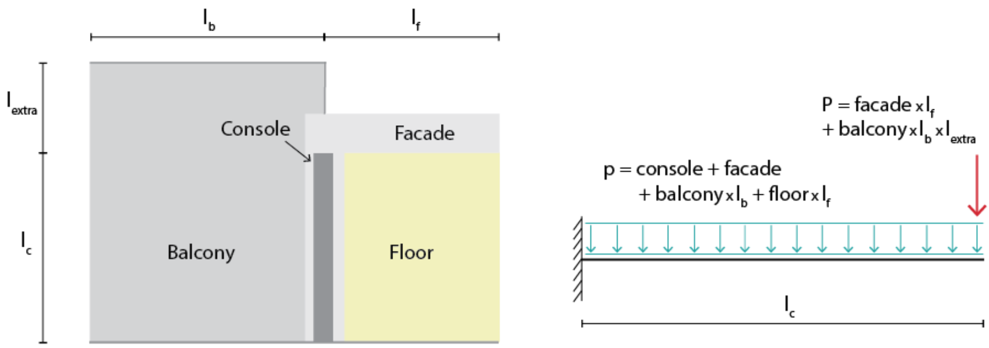


Figure 4.23: Illustration of loads for design proposal 2 with facade and balcony, plan view and cantilevered beam.

The consoles are checked for load combination LC1, LC2 and LC3, see loads in Appendix C.5.3. It turns out that the most critical load combination is LC2 for dominating live load. In the calculations the extra depth of a balcony l_{extra} will be determined which is the length of the balcony that extends beyond the end of the console. For a console with intact reinforcement the extra balcony length causing the consoles to be 100% utilized is 1.15m for a long console and 1.05m for a short console. The design loads for these balcony sizes are given in Table 4.23 for LC2. The theoretical dimensions of the balconies becomes 2.6m x 3.6m = 9.4m^2 on the facade with long consoles and 1.9m x 3.6m = 6.8m^2 on the facade with short consoles. Even though the balconies are placed in a 3.6m wide module the actual width of the balconies will be reduced by 2x0.3m=0.6m due to the placement of the facade on the outside of the consoles. The reduction has not been taken into account in the calculations since it is on the safe side to assume a larger balcony area and thereby a larger load.

Table 4.23: Design loads for proposal 2 (LC2) - built-in balconies.

	Long console	Short console
Moment M_{Ed} [kNm]:	67.1	37.5
Shear force V_{Ed} [kN]:	51.2	35.7
Torsional moment T_{Ed} [kNm]:	0.3	0.2

The possible extra depth of a balcony is investigated in the case of corroded reinforcement, since the balcony size is reduced when the diameter is reduced. In Figure 4.24 the results are given for a long and a short console, blue and orange respectively. The diameter of the tension reinforcement is compared to the maximum extra balcony depth for consoles that are 100% utilized. Both the bearing capacity of the console and the load from the balcony are reduced when the diameter and balcony depth are reduced. For a tension reinforcement diameter of less than 16.0mm the long consoles do not have the sufficient bearing capacity to carry any extra balcony length. The reason for this change around d =16mm is that the critical bearing capacity changes from depending on the moment capacity to the shear capacity, see Figure 4.19. From the figure it can also be seen that the shear capacity drops more radically compared to the moment capacity when the diameter is reduced for a long console. For short consoles the moment capacity is still the most critical until $d < 15$mm and therefore the short consoles still have the capacity to carry an extra balcony depth even though the diameter is decreasing.

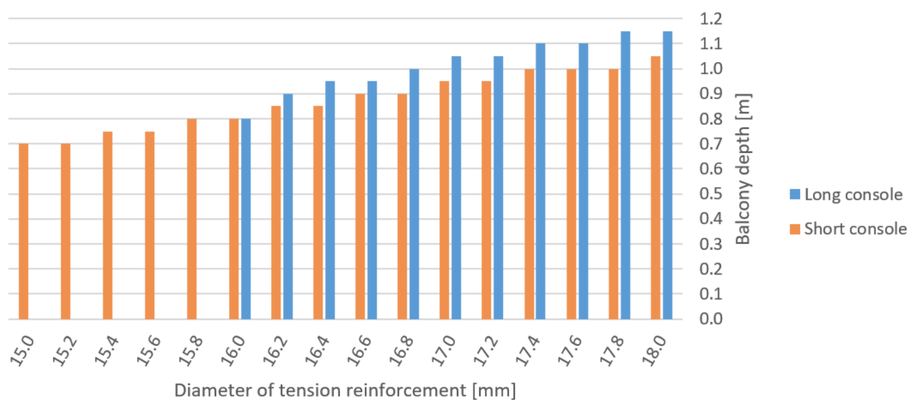


Figure 4.24: Extra depth of balcony compared to diameter of tension reinforcement.

If a console does not have a sufficient amount of bearing capacity, either due to larger balcony than suggested or due to corroded reinforcement causing a smaller diameter, there is an option for strengthening the console. This can be done by attaching a tension rod to the console which will be anchored in the bearing wall above the console.

4.7.5 Proposal 3 - External Balconies

Earlier the utilization was found for a long and short console when a new facade was placed on the outside of the consoles, see section 4.7.3. The utilization was 70% for a long console and 58% for a short console. In design proposal 3 it is chosen to place the balconies on the outside of the new facade. This means that the extra loads occurring from the balconies will act as a point load at the end of the consoles. It is assumed that a balcony will span along a short and a wide module of the facade, which means that the heaviest loaded console is the one in the middle of the balcony which carries the load of $l_p = (2.7\text{m} + 3.6\text{m})/2 = 3.15\text{m}$, see Figure 4.25.

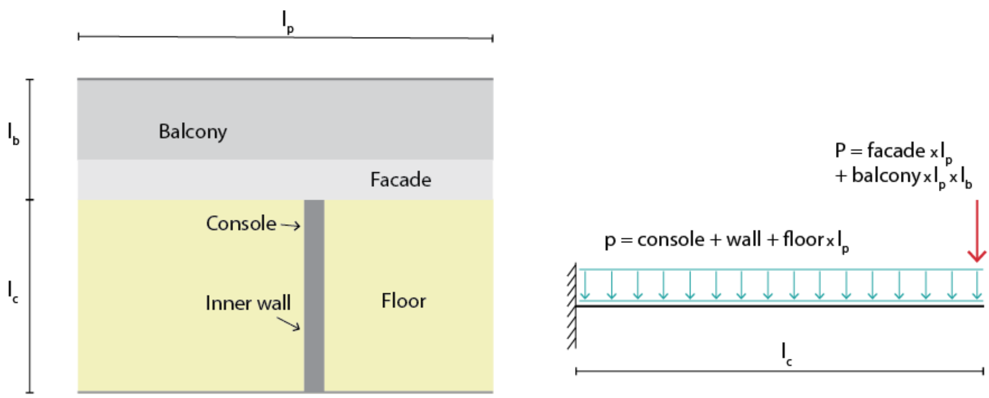


Figure 4.25: Illustration of loads for design proposal 3, plan view and cantilevered beam.

The design proposal has been investigated for load combination LC1, LC2 and LC3, see Appendix C.5.4. The worst load combination in this scenario is LC2 with dominating live load. The depth of the external balconies are calculated until the consoles are 100% utilized and the results are given in Table 4.24. It can be seen from the results that the depths of the balconies are very small and the balconies would loose their purpose given these dimension.

Table 4.24: Depth of external balcony for 100% utilization - external balcony.

	Long console	Short console
Balcony depth, 100% utilization	0.47m	0.50m

The consoles will be strengthened with a tension rod in order to be able to attach larger balconies to the outside of the facade. The tension rod will be attached to the end of the console and it will be anchored to the bearing wall which creates an inclined tension rod, it is illustrated in Figure 4.26. The anchoring of the tension rod in the bearing wall is not further investigated, it should however be anchored below the console above in order to not interfere with the anchoring of the console.

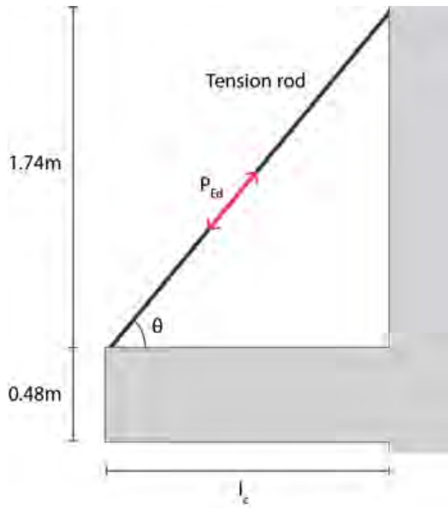


Figure 4.26: Inclined tension rod attached between a console and the bearing wall.

For the calculations a balcony of depth 2.0m will be investigated, which will create a total balcony area of 12.6m^2. This area does not necessarily need to be rectangular, it can be any shape. It is chosen that the console will carry the load from the indoor area, the facade and 0.45m of the balcony, resulting in the consoles almost being fully utilized. The loads are calculated for LC2 and are given in Table 4.25.

Table 4.25: Design loads for proposal 3 (LC2) - external balconies.

	Long console	Short console
Moment M_{Ed} [kNm]:	64.5	35.4
Shear force V_{Ed} [kN]:	55.7	41.1
Torsional moment T_{Ed} [kNm]:	0.6	0.4

The tension rod should therefore carry the load of 2.00m-0.45m=1.55m balcony depth in order to obtain a 2.0m deep balcony. The load from 1.55m balcony is calculated for LC2, since this was the most critical load combination for the consoles. The load P is given in Table 4.26. From the inclinations of the tension rods Θ the design tension forces P_{Ed} are calculated for both consoles and the results are also defined in Table 4.26. A tension rod with diameter $d = 12$mm and characteristic yield strength $f_{yk} = 355$MPa is chosen. This results in a design tension capacity of:

$$F_{\mathrm{Rd}} = \frac{\pi}{4} d^2 \cdot \frac{f_{yk}}{1.2} = 33.5\mathrm{kN} \tag{4.51}$$

The utilization of the tension rod is defined in Table 4.26 and it can be seen that the chosen tension rod has a sufficient bearing capacity. If a larger balcony is wanted or if the reinforcement in the consoles has corroded then a tension rod with a larger diameter can be chosen.

Table 4.26: Design loads for load combination 3 - external balconies.

	Long console	Short console
Load 1.55m balcony P [kN]:	24.1	24.1
Inclination θ [°]:	58	70
Design tension force P_{Ed} [kN]:	28.5	25.7
Utilization $P_{\mathrm{Ed}}/F_{\mathrm{Rd}}$ [%]:	85	77

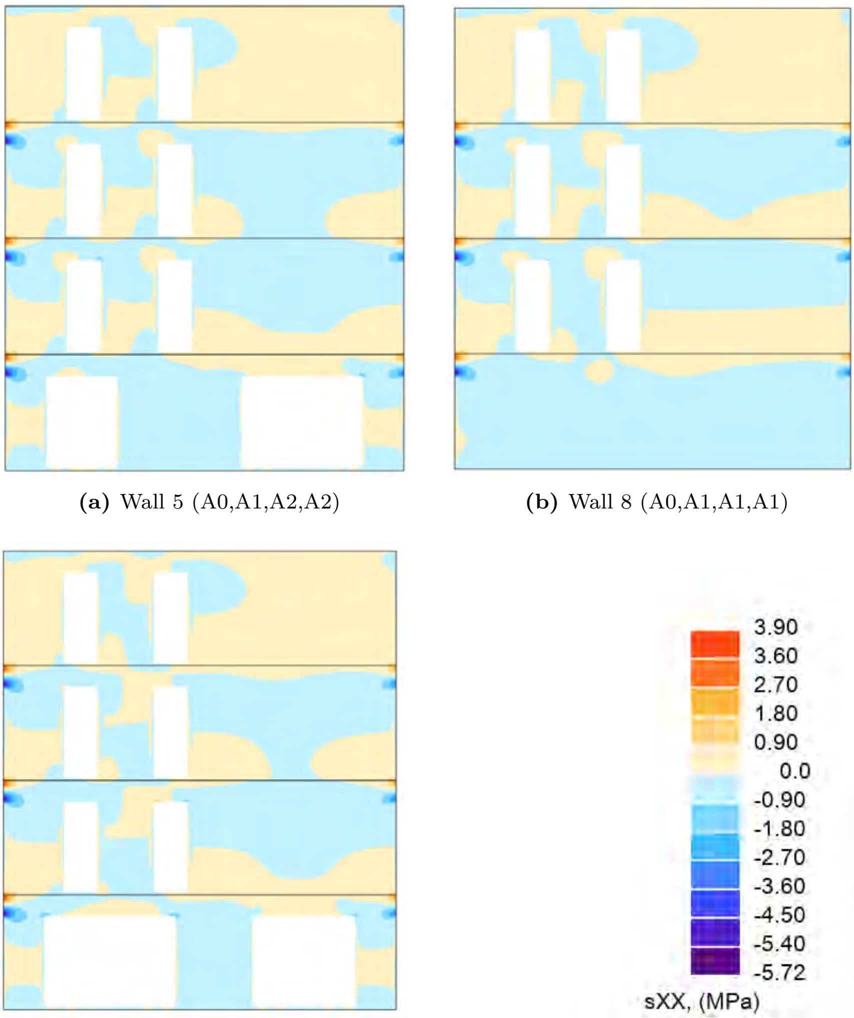
4.8 Results for Bearing Walls

The bearing walls will be systematically analysed in relation to creating new openings in the walls. Only the transverse walls will be investigated since it is chosen not to create openings in the longitudinal walls. The structure has been checked for all load combinations and the results are very similar for all combinations. It is chosen to base the investigation on load combination LC4 with wind being the dominant load. The direction of the wind is perpendicular to the facade and the roof is subjected to pressure and suction. The stress maps for the walls will be shown without the consoles, since the stresses in the consoles are larger compared to the walls and therefore the stress maps for the walls will not be very detailed if they are included. The stresses in the consoles calculated by *Robot* will not be used since the bearing capacity of the consoles have already been checked, see section 4.4.3.

4.8.1 Comparison of Identical Openings

First it is checked whether walls of the same type are subjected to identical stresses or if there is a difference in the stresses due to the placement of the walls. All results from *Robot* for the original walls are given in Appendix D.1 where walls with the same types of openings are compared. The results for walls with opening C and D are shown in Figure 4.27 and 4.28 for local horizontal stresses σ_{xx} and vertical stresses σ_{yy}. From Figure 4.27 it can be seen that the horizontal stresses are close to zero everywhere except in the area where the consoles are attached. In Figure 4.28 it can be seen that most of the walls are subjected to a small compression stress and it should be noted that the vertical stresses do not change much throughout the levels except around the big openings in the bottom level.

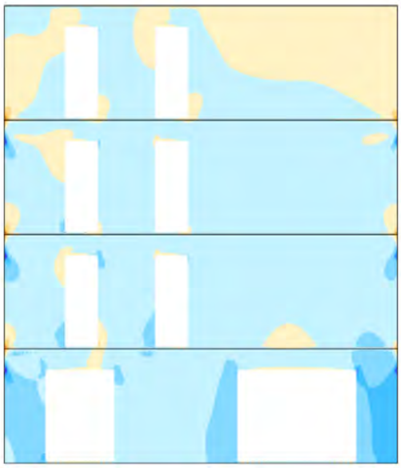
From all the results it can be seen that there is no significant difference in the stresses when comparing the same types of walls. The stresses are also fairly similar when all the walls are compared, see Appendix D.1, and the largest stresses occur around the anchoring of the consoles. Even though the stresses are very similar it should be noted that the amount of reinforcement varies in every wall and therefore an exact comparison can not be made. It can be seen from Figure 4.27 and 4.28 that wall 5 and 10 have identical reinforcement, whereas wall 8 contains less reinforcement in the two bottom floors, most likely due to the lack of large openings.



(a) Wall 5 (A0,A1,A2,A2) **(b)** Wall 8 (A0,A1,A1,A1)

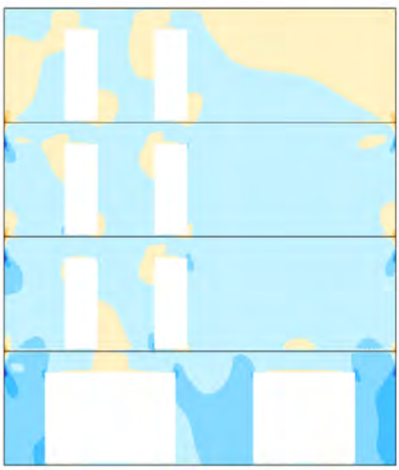
(c) Wall 10 (A0,A1,A2,A2) **(d)** Legend, σ_{xx}

Figure 4.27: Walls with opening C and D. Horizontal stresses σ_{xx} [MPa].

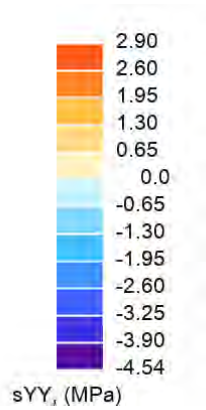


(a) Wall 5 (A0,A1,A2,A2)

(b) Wall 8 (A0,A1,A1,A1)



(c) Wall 10 (A0,A1,A2,A2)



(d) Legend, σ_{yy}

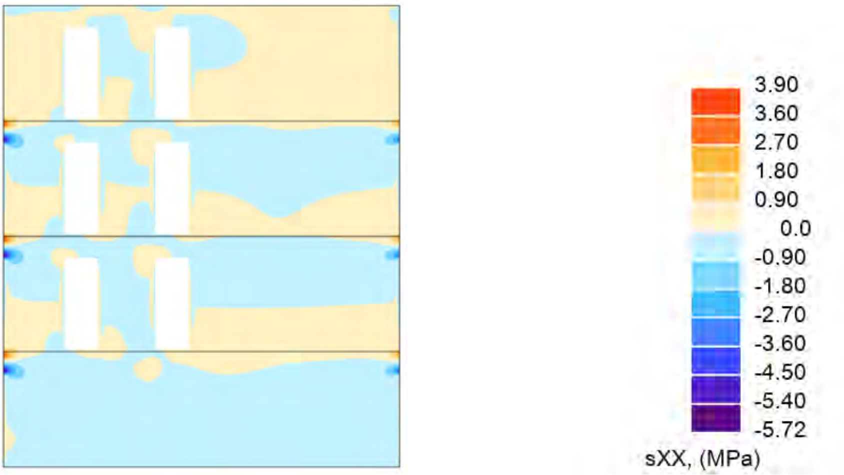
Figure 4.28: Walls with opening C and D. Vertical stresses σ_{yy} [MPa].

4.8.2 The Influence of Creating a New Opening

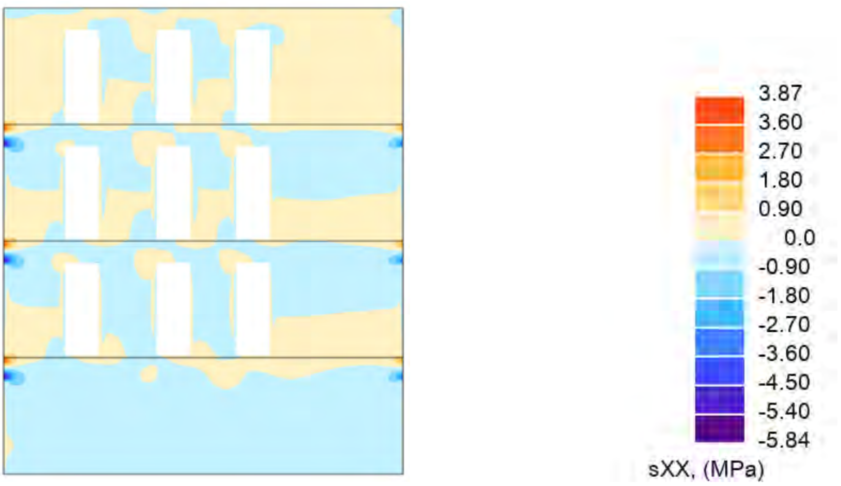
It will be examined whether making a new opening in one wall will affect the stresses in the neighbouring walls and whether it will affect the stresses around the original openings in that specific wall. This is very relevant to check because if new openings do affect the other walls, then new openings would have to be designed specifically for each building in order to secure the overall stability of the building. If, however new openings do not effect other walls, then one analysis can be performed for each type of wall and these results can be directly implemented in other buildings. Since the bearing capacity of each wall has already been checked a stability check will not be necessary.

A new opening of type A is made in wall 8, where opening C and D already exists. The stresses before and after the opening is implemented will be compared for wall 8 and the neighbouring walls 7 and 9. The stress maps for wall 7, 8, and 9 are shown in Appendix D.2. From the results it can be seen that the new opening in wall 8 has no influence on the stresses in wall 7 and 9. Furthermore the new opening A in wall 8 does not effect the stresses around the existing openings in wall 8, see Figure 4.29 and 4.30. The stresses are within the same range in both situations and the maximum stresses have been slightly modified.

A new opening in wall 8 makes no significant changes to the stresses and therefore it is assumed that this is a general tendency. This means that openings in a wall will only have an effect on the wall itself and not on the surrounding walls.

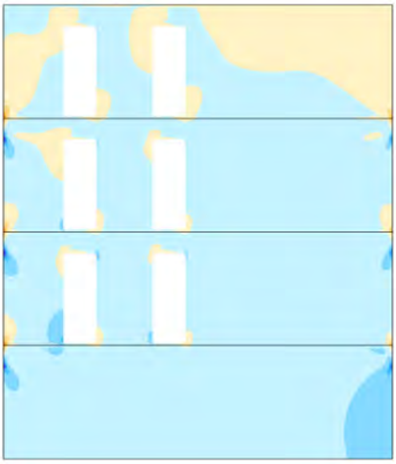


(a) Wall 8 without opening A **(b)** Wall 8 without opening A, legend σ_{xx}

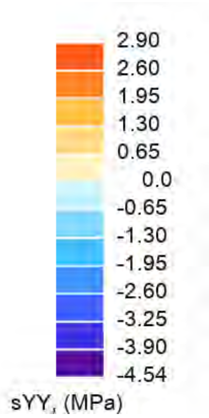


(c) Wall 8 with opening A **(d)** Wall 8 with opening A, legend σ_{xx}

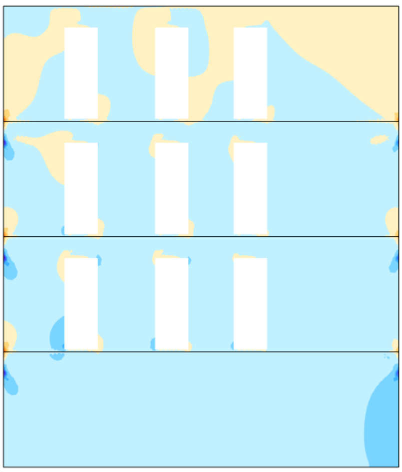
Figure 4.29: Wall 8, without and with opening A. Horizontal stresses σ_{xx} [MPa].



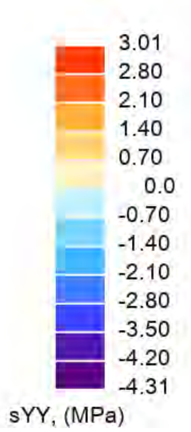
(a) Wall 8 without opening A



(b) Wall 8 without opening A, legend σ_{yy}



(c) Wall 8 with opening A



(d) Wall 8 with opening A, legend σ_{yy}

Figure 4.30: Wall 8, without and with opening A. Vertical stresses σ_{yy} [MPa].

4.8.3 Placement of New Openings

Since new openings in a wall does not affect other walls it has been decided to design new openings throughout all the walls in the building, so that no two walls will be identical. This means that only one analysis will be made in *Robot*. The stresses will then be checked for all the walls and a required reinforcement area will be calculated and compared to the actual reinforcement in the walls.

All new openings in the walls are designed in order to create new useful floor plans of the apartments. Since no walls will be identical the apartments will likewise not be identical. This is done in order to be able to use the results in most ways. When renovating a building the floor plan of an apartment can be designed specifically for that apartment and standard solutions can be avoided. This means that a bigger diversity is created within the building.

The original apartments span either 1, 2 or 3 modules. All apartments, independent of the size, consist of one 3.6m module and the rest of them being 2.7m modules. This is a general tendency throughout all the building plans that have been examined. In general the apartment plans are very well made and it has been chosen not to make any large changes to the original plans, see Figure 4.8. Therefore no openings will be implemented in the bearing walls that separate the apartments. New room divisions which requires new openings in the bearing walls inside the apartments will now be proposed for the three types of apartments. The new floor plan layout for block 12 is illustrated in Figure 4.31.

1-module apartments: The smallest apartments are placed in a 3.6m module and no changes will be made to the bearing walls, since these walls separate the apartment from the neighboring apartments. Block 12 does not contain any 1-module apartments.

2-module apartments: The medium apartments will consist of a kitchen and a living room in the 3.6m module. In the 2.7m module the apartments will contain an entrance, a bathroom, and a bedroom. Block 12 contains one 2-module apartment and in this apartment the old staircase is included in the living area. This results in a new large entrance area and bathroom, and two new openings in the bearing wall should be created in order to connect the rooms with the rest of the apartment. The door opening between the living room and the bedroom is also widened so it will consists of double doors. This opening will create the feeling of a larger living room and bedroom, since light will come through the opening.

3-module apartments: The large 3-module apartments are very similar to the 2-module apartments, the only difference being the extra 2.7m module. The extra module can be used for two bedrooms, keeping the existing door entrances. In between the rooms a storage room can be built which means that one extra entrance must be created in the bearing wall. The hallway outside the old bathrooms will be included in the area of the new bathrooms and a new opening from the bathroom will be created. The 3-module apartments usually have three bedrooms and one living room. However another option would be to combine a bedroom with the living room which will create a larger living room area that can also be used as a dining room area or office area. For this solution a large opening between the living room and a bedroom must be created in the bearing wall.

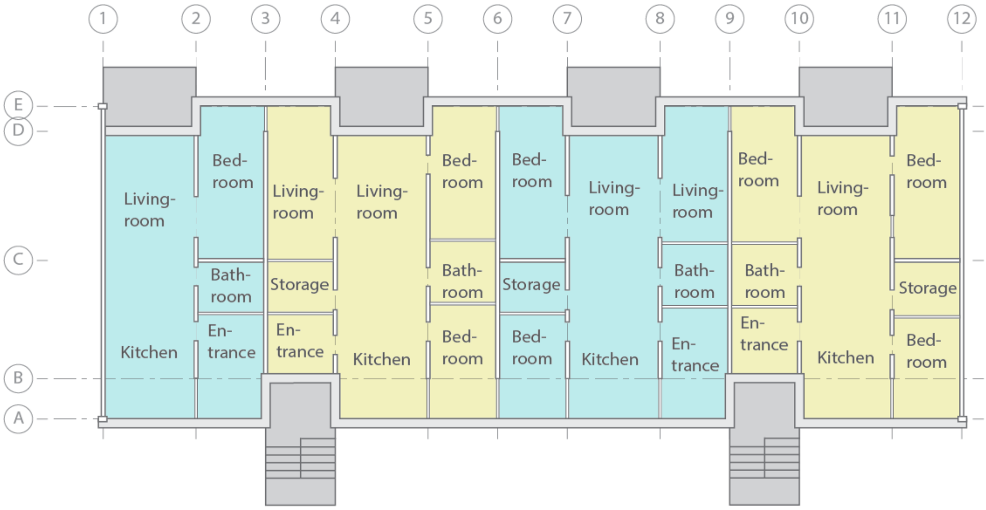


Figure 4.31: Suggestion of room division where no apartments are identical.

It is chosen to place the new openings in the same manner as the old ones, therefore the new openings can be of type A, B, C or D. From the apartment layout it is also chosen to create a larger type of opening A namely A2 which is 2 x A and an opening A3 which is 3 x A. A new door opening E and F will likewise be created and they will be placed a distance of 1.0m from the edge of the bearing wall in order not to interfere with the anchoring of the consoles. The existing openings for doors are 81cm x 222cm and it is chosen to keep this dimension for the new openings.

The openings in the walls are illustrated in Figure 4.32 and defined in Table 4.27. Wall 1 and 12 are the gable walls and no openings will be made in these walls. Wall 3, 6 and 9 are bearing walls dividing the apartments and no openings will be created in these walls. The openings will be placed identically in level 1, 2 and 3 and no changes will be made to the openings in level 0.

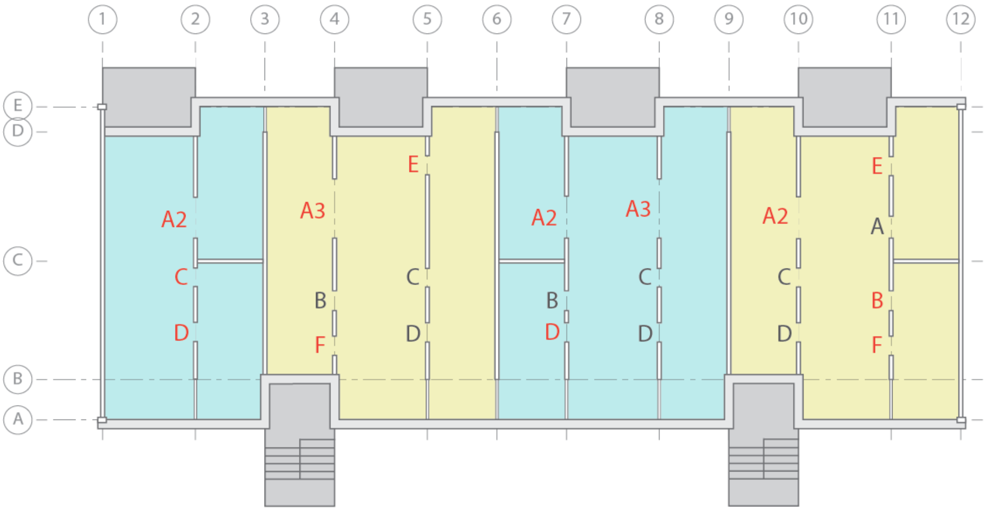
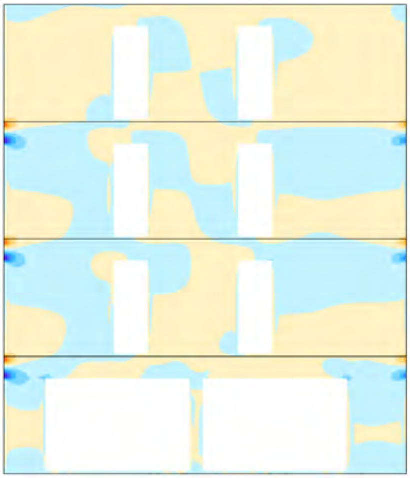


Figure 4.32: Illustration of existing and new openings in walls. New openings are marked with red.

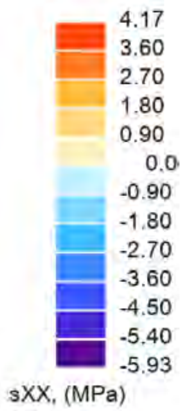
Table 4.27: Overview of openings in walls. Openings marked with black are existing, openings marked with red are new, openings in ( ) will be sealed.

Wall	2	3	4	5	6	7	8	9	10	11
A	A		A			A	A		A	(A)
A2	A2		A2			A2	A2		A2	
A3			A3				A3			
B			B			B				B
C	C			C			C		C	
D	D			D		D	D		D	
E				E						E
F			F							F

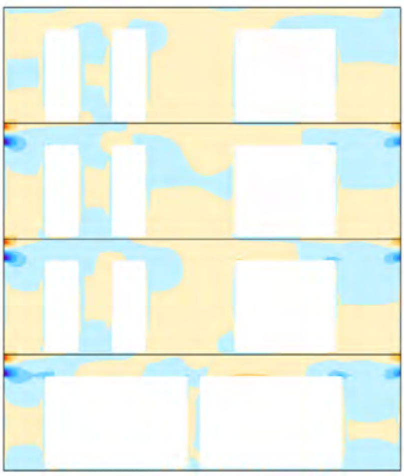
An analysis of the openings have been made in *Robot* and the results of the stress maps are shown in Appendix D.3. Despite the fact that 13 new openings have been made in every 3 levels there are no significant changes in the stress maps compared to the original openings. This is very interesting since the amount of openings have been more than doubled in level 1-3 and some openings have been made 3 times larger. The stress maps for wall 4 with and without new openings are shown in Figure 4.33 and 4.34. From the stress maps it can be seen that stresses around the openings have not changed much, except in the top right corner of opening A3 where there is a slightly larger compression stress in both the x and y direction.



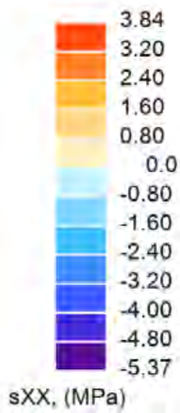
(a) Wall 4 without new openings



(b) Wall 4 without new openings, legend σ_{xx}

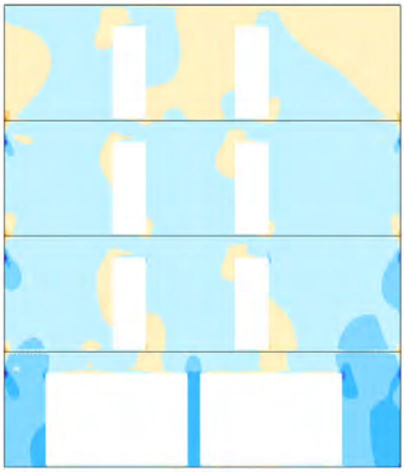


(c) Wall 4 with new openings

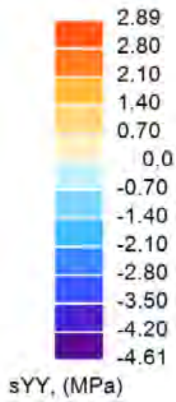


(d) Wall 4 with new openings, legend σ_{xx}

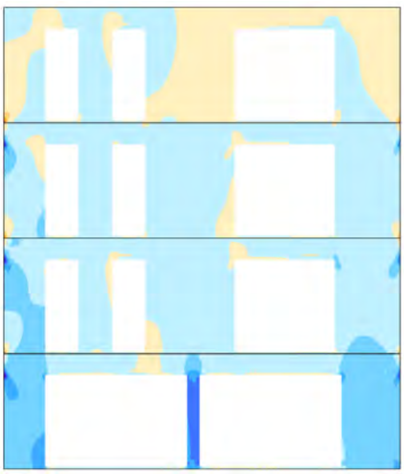
Figure 4.33: Wall 4, without and with opening A3 and F. Horizontal stresses σ_{xx} [MPa].



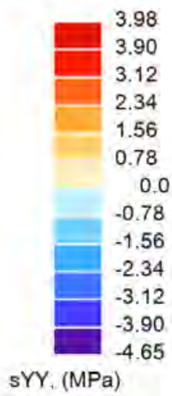
(a) Wall 4 without new openings



(b) Wall 4 without new openings, legend σ_{yy}



(c) Wall 4 with new openings



(d) Wall 4 with new openings, legend σ_{yy}

Figure 4.34: Wall 4, without and with opening A3 and F. Vertical stresses σ_{yy} [MPa].

4.8.4 Check of Reinforcement

The stresses are very similar in every wall and therefore it is chosen only to investigate one wall. The amount of required reinforcement will be determined for wall 4, level 1 with new openings, since the stresses are largest in this level compared to level 2 and 3. Different locations on the wall have been chosen for further analysis. The locations are in the middle of a plane wall area between opening F and B, in the area around the door opening A3 and in the area where the long console is attached. The areas are chosen since the stresses and the reinforcement layout vary in these places. In the area where the console is attached both large tensile and compression stresses occur, therefore two analysis will be made around the console. Besides the amount of required reinforcement the stresses in the concrete will also be checked. The stress maps are shown for wall 4, level 1 in Figure 4.35, 4.36 and 4.37 where a precise value of the stresses in the four locations are shown.

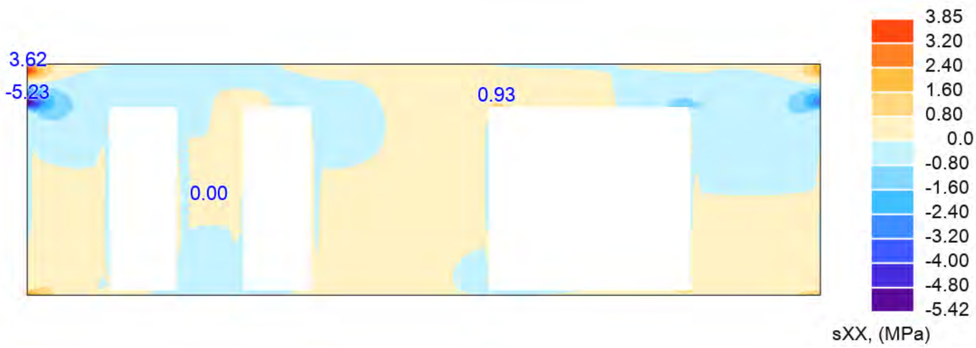


Figure 4.35: Horizontal stresses σ_{xx} [MPa] in wall 4, level 1, with new openings.

Figure 4.36: Vertical stresses σ_{yy} [MPa] in wall 4, level 1, with new openings.

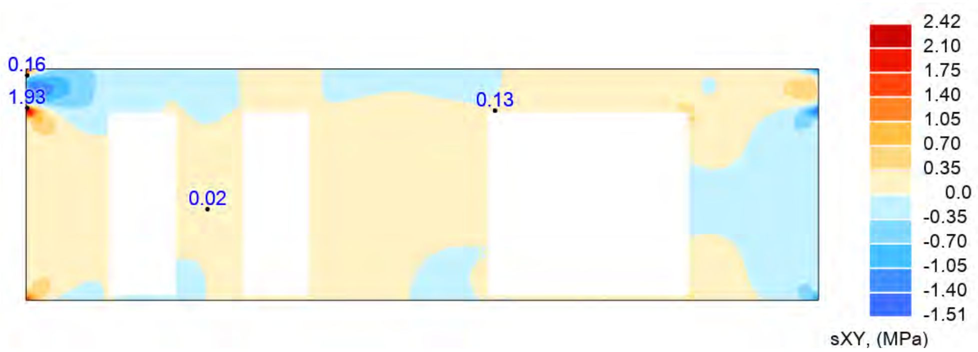


Figure 4.37: Shear stresses σ_{xy} [MPa] in wall 4, level 1, with new openings.

The stresses from the stress maps are summed up in Table 4.28 and the relevant stress cases according to the conditions in Table 4.14 are shown. For the wall area case 4 applies which means that no vertical reinforcement is needed since the wall is in compression in this direction. Around the door opening and in the top corner where the console is attached only tensile stresses are present and reinforcement is needed in both the vertical and horizontal direction, in this situation the case number is 1. The load from the console also creates an area with compression stresses in the wall which corresponds to case number 3. In this area no reinforcement is needed but it is however very important to check the concrete stress.

Table 4.28: Stresses in wall 4, level 1 - new openings.

	σ_{xx} [MPa]	σ_{yy} [MPa]	σ_{xy} [MPa]	Case
Wall	0.00	-0.13	0.02	4
Door opening	0.93	0.11	0.13	1
Console, tension	3.84	3.62	0.16	1
Console, compression	-5.23	-3.78	1.93	3

The reinforcement areas and concrete stresses are determined from the equations in Table 4.15 and the results are defined in Table 4.29. From the results it can be seen that reinforcement is only necessary around the door opening and in the area where the console is anchored.

Table 4.29: Required reinforcement areas and concrete stresses for wall 4, level 1 - new openings.

	A_{sx} [mm^2/m]	A_{sy} [mm^2/m]	σ_c [MPa]
Wall	0	0	0.13
Door opening	81	18	0.26
Console, tension	306	290	0.32
Console, compression	0	0	6.57

As mentioned earlier the amount of reinforcement in the walls vary around the building. The area of reinforcement in [mm^2/m] that is present in each wall type is defined in Table 4.30 based on the type of reinforcement given in Table 4.13.

Table 4.30: Reinforcement in the bearing walls.

Wall type	A_{sx} [mm^2/m]	A_{sy} [mm^2/m]
A0	0	0
A1	387	393
A2	387	785

Wall 4 consists of wall type A0 (level 3), A1 (level 2), A2 (level 1) and A2 (level 0). The required reinforcement areas in Table 4.29 are calculated for level 1, however level 2 and 3 will also be checked for the same required reinforcement since the stresses are very similar in all levels.

When comparing the required reinforcement in Table 4.29 with the actual amount of reinforcement in Table 4.30 it can be seen that wall type A1 and A2 has a sufficient amount of reinforcement. The amount of reinforcement by the console is sufficient without taking the extra reinforcement into account. This means that the wall in this area still has more bearing capacity. Furthermore it can be concluded that new large openings with width 2.43m in walls A1 and A2 can be created without strengthening of the openings since the existing reinforcement in the walls is enough.

Wall type A0 will only be checked for stresses in the wall and around the door opening since no console is attached to the top level of the building. The wall contains no reinforcement which means that the new door opening of width 2.43m in wall A0 will have to be strengthened. Extra reinforcement is present around the existing door openings, but since door opening A has been expanded to opening A3 the existing reinforcement will not be enough. The new door opening F is not subjected to any significant tensile stresses and therefore it is not necessary to strengthen this opening. This means that either large openings, with widths greater than 0.81m, should not be made in walls of type A0 or the large openings should be strengthened.

The largest concrete stress occurs in the area next to the long console. The requirement in Table 4.15 should be met:

$$\sigma_c = 6.57\text{MPa} \leq \nu f_{cd} = 6.70\text{MPa} \tag{4.52}$$

The requirement is fulfilled, however the concrete is fully utilized. This means that if the compression stresses are slightly larger the requirement will not be met and there is a risk of failure in the concrete. As mentioned earlier the value $\nu = 0.65$ was chosen as a conservative value which means that extra capacity of the concrete can be found if a more precise value of the effectiveness factor is found. The value is found empirically and therefore it will be difficult to determine a precise value for this specific situation. Another solution is to assume that the concrete has 25% extra capacity, since the buildings were built before 1990, which gives:

$$1.25\nu f_{cd} = 8.38\text{MPa} \tag{4.53}$$

The capacity of the concrete is now much larger and the concrete will be utilized around 80% in this case. Alternatively the wall area by the consoles will need to be strengthened or the loading on the console must be reduced. This can be done by attaching steel plates to both sides of the wall in that area, in order for the steel plates to take some of the compression stresses. An alternative solution is to remove the concrete in that area and then replace it with new concrete with higher strength, but this is however a complicated renovation proposal. If the load can not be reduced the consoles can be attached to a tension rod in order to remove some of the load from the consoles and transfer it to the bearing wall. This solution will reduce the compression stresses and actually add tension to the area due to the tension rod.

4.9 Results of Adding an Extra Floor

The effect of adding an extra level to the building will now be investigated. The load from one floor will be added as an extra uniform load on top of the roof in the *Robot* model. The *Robot* model with new openings in the bearing walls will be used for the purpose. The weight of a floor in the existing building is determined in Appendix C.4 since it was used to find the seismic load. The self weight G_{floor} and the live load Q_{floor} for one floor are given by:

$$G_{floor} = 3562\text{kN} \tag{4.54}$$

$$Q_{floor} = 480\text{kN} \tag{4.55}$$

The load combination used for the investigations is LC4 with dominating wind load, therefore the design load for the extra floor is defined by:

$$E_{d,floor} = \gamma_G G_{floor} + \gamma_{Q,A} \Psi_{0,A} Q_{floor} = 3922\text{kN} \tag{4.56}$$

The load is divided with the area of the roof in order to determine a uniformly distributed load:

$$e_{d,floor} = \frac{E_{d,floor}}{9.6\text{m} \cdot 33.3\text{m}} = 12.3\text{kN/m}^2 \tag{4.57}$$

The new level will most likely be build of light materials and not concrete, therefore the load added is conservatively larger than what is necessary. It is expected that the load will increase the vertical compression stresses and it will increase the horizontal tension stresses on the top edge of openings. In order to compare the effect of adding another floor to the building the stress maps for wall 4, level 1, are shown in Figure 4.38, 4.39 and 4.40.

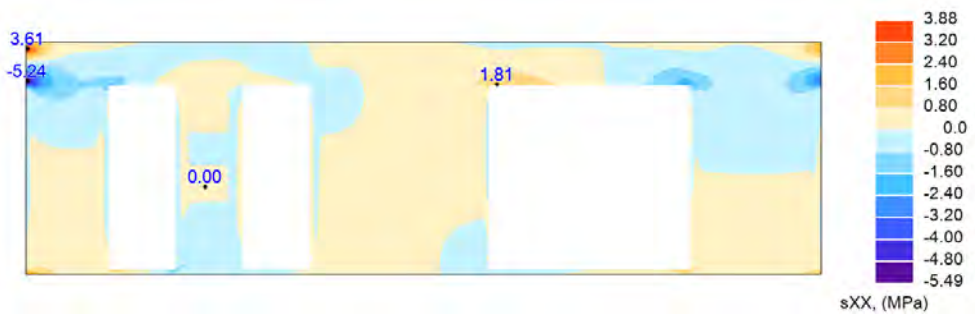


Figure 4.38: Horizontal stresses σ_{xx} [MPa] in wall 4, level 1, with extra load from new floor.

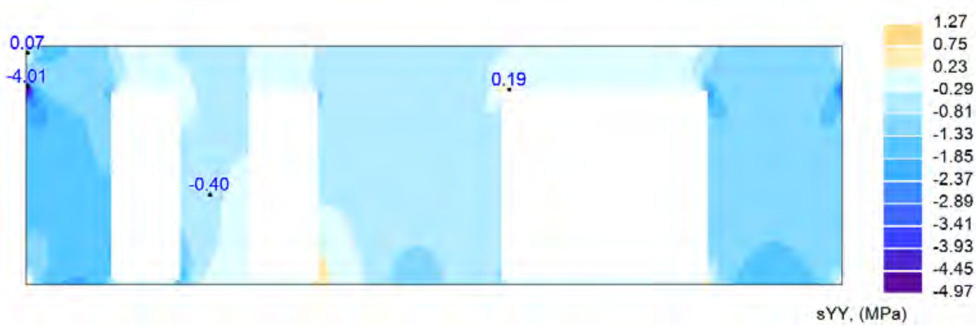


Figure 4.39: Vertical stresses σ_{yy} [MPa] in wall 4, level 1, with extra load from new floor.

Figure 4.40: Shear stresses σ_{xy} [MPa] in wall 4, level 1, with extra load from new floor.

The stresses are summed up in Table 4.31 and the required reinforcement areas and concrete stresses are defined in Table 4.32. When comparing the results to Table 4.29 it can be seen that the required reinforcement areas and concrete stress are slightly larger in the wall and in the area around an opening. The requirements are still fulfilled despite the larger values, except for wall type A0, as discussed earlier. When comparing the results for the area around the console it can be seen that adding an extra floor has a positive effect on the stresses in that area. The amount of required reinforcement has been reduced a lot and the concrete stress has been slightly reduced.

Table 4.31: Stresses in wall 4, level 1 - extra floor.

	σ_{xx} [MPa]	σ_{yy} [MPa]	σ_{xy} [MPa]	Case
Wall	0.00	-0.40	-0.09	4
Door opening	1.81	0.19	0.24	1
Console, tension	3.61	0.07	0.25	1
Console, compression	-5.24	-4.01	1.77	3

Table 4.32: Required reinforcement areas and concrete stresses for wall 4, level 1 - extra floor.

	A_{sx} [mm^2/m]	A_{sy} [mm^2/m]	σ_c [MPa]
Wall	2	0	0.42
Door opening	157	33	0.48
Console, tension	296	25	0.50
Console, compression	0	0	6.50

4.10 Discussion of Structural Analysis

The bearing capacity of chosen concrete elements have been investigated in relation to different renovation proposals. The calculations are performed for block 12 in Sisimiut, but the results will be applicable to other buildings of the same type as well. The bearing capacity is calculated in a simple static manner and detailed calculations of connection details have not been investigated. The elements investigated are the consoles and the bearing walls. The bearing capacity of the consoles is determined based on the classical beam theory and the bearing walls have been analysed in a FEM program called *Robot*.

The bearing capacity of the consoles has been investigated for three different design cases. In the first design case the existing balcony and facade design has been kept and it turns out that the consoles are 44% and 25% utilized for a long and short console respectively. The consoles have therefore not been very utilized throughout their entire lifetime. This implies that the concrete on the top of the cross section might not be cracked and the top reinforcement has been protected against corrosion, which corresponds well with what was observed in the visual registration. The consoles will now be placed inside the building envelope and therefore it is acceptable if the concrete will crack since the steel will not corrode in these conditions.

In the second design case a new facade is placed on the outside of the consoles. The utilization becomes 70% for a long console and 58% for a short console. If it is assumed that the reinforcement has corroded then the minimum required diameter of the tension reinforcement is 16.2mm for a long console and 15.2mm for a short console, in comparison the diameter of the intact reinforcement is 18.0mm. In the second design case it is also wished to place built-in balconies in the large modules of 3.6m. The concrete balconies are replaced by light weight steel balconies and the depth of the balconies are calculated for both short and long consoles as a function of the diameter of the tension reinforcement. If the diameter of the tension reinforcement in the long console is less than 16.0mm the console does not have the capacity to carry a new balcony whereas the diameter can be less than 15.0mm for the short consoles. If the reinforcement is intact the total balcony depth will be 2.6m for the long consoles and 1.9m for the short consoles.

In the third design case the facade is still placed on the outside of the columns and external balconies will be placed on the outside of the facade. The consoles do not have the sufficient bearing capacity for this design case and therefore they will be strengthened with a steel tension rod that is attached to the balcony and anchored in the bearing wall above. The balconies in this proposal can span along several modules and if a depth of 2.0m is desired the tension rod must be 12mm in diameter of quality S355. This is not a very big tension rod and therefore, if needed, the diameter can be increased or several tension rods can be applied.

In the calculations of the bearing capacity of the consoles it is assumed that the diameter of corroded reinforcement is reduced the same amount for all reinforcement. In reality the stirrups will be subjected to carbonation earlier than the longitudinal reinforcement and therefore corrosion will have occurred longer for these elements. In the calculations it is also assumed that the anchoring between the corroded reinforcement and the concrete is unchanged compared to the situation with intact reinforcement. This assumption is made due to the fact that steel expands when it corrodes and therefore no void will be present between the steel and the concrete.

The bearing walls have been examined in relation to creating new openings in the wall in order to create new room divisions in the apartments. The building contains four different types of transverse walls, that depend on the existing openings, and these walls are used in all buildings since the apartment design is standardised. From the analysis it has been found that walls of the same type are subjected to the same stresses no matter where the walls are placed in the building. A new opening was then created in a wall and the results showed that the new opening did not effect the stresses in the neighbouring walls neither the stresses around the existing openings in that wall. It was therefore decided to create new openings in all walls with existing openings in order to see the effect. None of the walls were identical, however each opening has been chosen in order to be able to create a good room division in the apartments. Despite doubling the amount of openings in the walls there were no significant changes in the stresses.

A representative wall was then chosen in order to check the stresses in relation to the amount of reinforcement in the walls. The results showed that for bearing walls without any reinforcement small openings of width 0.81m could be created with no problems but larger openings would have to be reinforced. For the remaining walls the reinforcement is sufficient. The largest stresses in the walls occur where the consoles are attached and here large compression stresses occur. The concrete has sufficient strength in this area, but this area is definitely the most critical area in the walls and the concrete is almost fully utilized. It is therefore important if further calculations are made of the bearing walls that this area will be checked thoroughly if the loads are changed. If needed the concrete strength can be increased by 25% due to the fact that the buildings where built before 1990.

Lastly the effect of adding an extra floor to building is checked and from the results it is clear that the extra load does not change the stresses in any significant way. The only main difference is that the critical concrete area is now less critically loaded due to the extra vertical load. A further investigation could be conducted where the effect of creating different openings in each level in the same wall would be investigated, since this might create more concentrated stress areas.

CHAPTER 5

Discussion

The concrete structure of residential buildings in Greenland have been analyzed in terms of a condition assessment and an evaluation of the bearing capacity. Results from each of the two investigations have already been discussed in the intermediate discussions in the former two chapters. In this chapter the results from the two investigations will be combined in an overall discussion of the renovation proposals.

5.1 Prerequisites and Limitations

Not all structural elements of the building have been investigated in relation to renovation. Therefore the prerequisites and limitations will be discussed in this section.

For all renovation proposals it is assumed that the staircases inside the building will be removed and the space will be used for apartments. A new stair case and elevator will be made outside the building as a separate structure. The structure will require its own foundation and access bridges will be constructed to the apartments. Further detailing will not be made for this solution since the access towers are external structures. It is also assumed that no new structural elements such as a foundation or columns will be made, only structural elements that can strengthen existing elements will be used, such as tension rods.

The bearing capacity of the concrete balcony plates have not been investigated. It is assumed however that the plates have sufficient capacity since the loading situation will not change significantly in the future. The plates are very thick and the span is very short, therefore it is also assumed that the plates still have sufficient bearing capacity despite the fact that the reinforcement has started to corrode. In the design proposals the balconies will be a part of the indoor area and therefore corrosion will stop, leaving the plates with their current capacity. If the plates are in a very bad shape they can easily be replaced with new plates, but it is however recommended to keep the plates if possible. The concrete plates are massive and very heavy, they can therefore be replaced by lighter plates if it is necessary to reduce the load on the consoles.

The top part of the consoles have not been taken into account in the design proposals. It is assumed that the console tops will be either completely or partly removed during a renovation. A part of the console top might be kept if they will simplify the attachment of a new facade or a tension rod. However the capacity of the console tops in this situation have not been analysed and therefore they should be investigated if this solution is chosen.

The gable columns were investigated in the condition assessment, however no analysis of the bearing capacity has been performed. The columns are very damaged and therefore it would not make sense to perform calculations of the columns, since the bearing capacity would be individual for each column. It is assumed that the columns, despite the damages, will not effect the overall stability of the building since the gable walls are massively built and the condition of the walls is good. For a renovation it is recommended to repair the columns and prevent further carbonation of the concrete.

In the structural analysis of the consoles a reduction of the reinforcement diameter has been taken into account, due to corrosion of the reinforcement. A reduced concrete strength has not been considered, since the reinforcement in the cross section is the dimensioning parameter. However it is assumed that the condition of the concrete is sufficiently good and if the concrete is very damaged it should be repaired.

5.2 Renovation Proposals

In general the bearing capacity of the concrete structure is better than expected. The consoles and bearing walls are not very utilized in the existing buildings and therefore the possibilities for renovations are great. The renovation proposals can be combined or applied separately depending on the specific building that will be renovated. The chosen renovation method depends on the current condition of the building as well as the desired future purpose of the building.

From a visual condition assessment of the consoles in a given building the elements with a good visual appearance are assumed to have intact reinforcement and therefore the full capacity of these elements can be utilized. For elements in a bad condition the reinforcement will have started to corrode and therefore the diameter of the reinforcement will be reduced. Based on the calculated carbonation ages from the field tests and the estimated corrosion rate, an approximate reduction in the diameter can be determined. For concrete elements in a bad condition the amount of years since the carbonation front reached the reinforcement was found to be between 7-33 years. The average amount of years ago is 20 years and if the age is multiplied with the estimated corrosion rate of 34μm/year a reduction of the reinforcement diameter becomes 2x0.6mm=1.2mm. This results in a diameter of the tension reinforcement of d = 16.8mm. When comparing this diameter with the console results it is found that the damaged console has sufficient capacity to carry the original balconies and to carry a new facade on the outside of the console. If a built-in balcony is attached to the console the depth of the balcony can be 2.45m for a long console and 1.75m for a short console. It will also be possible to attach an external balcony if the console is strengthened with a tension rod. However these calculations are based on an average carbonation age. When a console is checked for a carbonation age of 33 years, the results show that the console would not have a sufficient capacity to carry a new facade and balconies, therefore strengthening would be necessary.

The corrosion rate is an uncertain parameter which is unknown for Greenlandic climate conditions. It would therefore be very relevant to determine the corrosion rate of the reinforcement in a console placed in Greenland. The best way forward would be to measure the diameter of the reinforcement in a console where the concrete has been removed and the reinforcement has been cleaned. This diameter should then be compared with the carbonation depth in that certain element in order to determine the corrosion rate. The corrosion rate earlier estimated is used to get an overview of the amount of corrosion in order to estimate whether there is any steel left in a console. Some of the elements in block 10 that are in a very poor condition contain cracks in the concrete due to the expansion of corroded reinforcement and the displacement of the concrete is very little. Knowing that 1mm of steel can expand up to 7mm when it corrodes, the corrosion rate determined seems reasonable.

In general the consoles should in the future be protected against further carbonation. This is done by either placing the consoles in a indoor environment, where carbonation almost does not occur, or by painting the consoles with protective paint. If the consoles are subjected to minor damages then these should be locally repaired. If the condition of the consoles is very bad it is recommended to remove all the concrete, clean the reinforcement and thereafter cast the console with new concrete of the same strength. If this repair method is not possible then the consoles in a very bad condition should be removed and replaced with new steel consoles. If most consoles are in a good condition the consoles should be kept and the few consoles in a bad condition should be repaired or strengthened.

Depending on the visual condition of the consoles several renovation suggestions will be made. The pros and cons of each renovation proposal will be discussed, and the final decision should be made individually for each building. It is however recommended, for all buildings, that the consoles will be incorporated in the indoor environment to improve the thermal insulation and to avoid thermal bridges. Therefore a renovation proposal, where the concrete consoles are placed on the outside of the facade, will not be suggested.

1. In the first proposal it is assumed that most of the consoles are in a very bad condition It is therefore recommended that they should all be removed, then a new facade should be installed where the existing facade is placed and new steel balconies can then be attached directly to the bearing walls. Thermal bridges will be created where the balconies are attached, however the contact area between the steel balconies and the bearing walls will be smaller compared

to the concrete consoles. It will also be possible to place isolation around the connections in order to minimize the thermal bridges. The living area will in this proposal be the same as the current living area, but the balconies have become private and the function of the balconies will be of better use compared to the existing balconies. This renovation proposal will most likely be the cheapest and it can also be performed for consoles with a good visual appearance, however it would be unfortunate not to keep the consoles in a renovation if the condition is good.

2. The second proposal is suggested for buildings where most of the consoles are in a semi good or good condition, since the full capacity of the consoles is not needed. The new facade will be moved to the outside of the consoles and built-in balconies will be installed in every 3.6m modules. The full capacity of an intact console is not needed to carry the facade and the size of the built-in balconies will depend on the condition of the consoles. If a few consoles are in a bad condition these can be repaired and strengthened individually. The new facade is not linear, due to the built-in balconies, and therefore it is important to investigate the connection details of the facade in the corners. The living area for this proposal is increased since some of the old balcony plates are incorporated in the apartments. The proposal will result in a more exiting appearance of the building, compared to the current one, and the new built-in balconies can both be rectangular or they can have an organic shape. This solution does not require strengthening of the consoles, if the condition of the consoles is semi good or good, which reduces the overall workload and thereby also the cost.

3. For the third proposal the consoles will also be kept and the condition of the consoles should be semi good or good. A facade covering the consoles will be placed and then external balconies will be installed on the outside of the new facade. An external balcony can span one, two or three modules, depending on the wish. This solution will create the most interesting and unique facade, since the geometry of the balconies can vary as well as the placement and size of the balconies. This proposal has the largest living area since all of the old balconies will be included in the apartments. This solution, however, requires strengthening of the consoles with tension rods. It will result in more work load and therefore the solution is recommended for buildings where the consoles would need strengthening in any case.

A design proposal combining proposal 2 and 3 is also a possibility. Here the facade will both have built-in balconies and external balconies. It is assumed that the new facade will be made of prefabricated facade elements that can easily be installed. First a steel frame should be attached to the consoles and thereby connect all the consoles. The facade elements and balconies can then be attached to the steel frame.

It is assumed that the condition of the concrete inside the existing building is in a good condition and it is assumed that the reinforcement has not corroded. Therefore the apartment layouts can be freely chosen, either based on the suggested designs or new designs, since the bearing walls have the sufficient capacity for creating new openings in the walls. The construction of an additional floor spanning the whole length of the building is also possible. If it is wished to construct several floors or if the additional floor is not evenly distributed on the roof then new calculations should be performed.

CHAPTER 6

Conclusion

The concrete structure of residential buildings in Greenland constructed in the 1960s and 1970s has been investigated and several renovation proposals have been suggested. The renovation proposals depend on the current condition and the desired future use of a building.

A case study has been made for block 10 in Sisimiut where the condition of the concrete structure has been assessed. The condition assessment concerned concrete elements on the outside of the building envelope, mainly consoles and plates, since these elements will be important in a thorough renovation. First a visual registration of the damages were made and, based on this registration, elements with a good and bad visual appearance were chosen for further testing. The testing showed that concrete elements with a good visual condition contain intact reinforcement, whereas corrosion of the reinforcement has begun in elements with a bad visual condition. Therefore when comparing the results, from the visual registration and the field testing, it is indicated that the condition of the elements can be determined purely from a visual registration. This is a very important observation since it creates an easy method for obtaining an overview of the condition of a building before a renovation starts.

The bearing capacity of the concrete structure has been analysed based on several design proposals. The design proposals concern the placement of the facade and balconies, the alteration of the bearing walls in terms of creating new openings, and finally the possibility of creating an additional floor on top of the building. Due to problematic thermal bridges around the consoles a new facade will, in the renovation proposals, be placed on the outside of the consoles. From the analysis it can be concluded that the consoles have the sufficient bearing capacity to carry the load from a new facade on the outside of the consoles. It is also possible to install built-in balconies without strengthening the consoles. The bearing capacity of the consoles is reduced if the reinforcement has corroded and this scenario has been taken into account in the analysis. If a console does not have the sufficient bearing capacity it can be strengthened with a tension rod that is anchored to the bearing wall. It is also possible to strengthen all the consoles with tension rods and thereby place external balconies on the outside of the new facade. This solution will create the largest apartments and it will create the most unique facade of the building. The chosen renovation proposal depends on the current state of the consoles since it greatly influences the cost of the renovation. Therefore if most of the consoles are found to be in a bad condition the best solution will be to remove the consoles, keep the current placement of the facade, and attach new balconies directly to the bearing walls. The transverse bearing walls have been systematically analysed in relation to create new openings for alternative room divisions in the apartments. It turns out that creating new openings barely affects the stresses in the walls and therefore new openings can freely be made. The only exception is in walls without reinforcement where openings, larger than a single door opening, will have to be strengthened. Lastly it has been found that applying an extra load to the building that corresponds to an additional floor does not have an effect on the bearing capacity of the walls. The walls have plenty of capacity and therefore there are many possibilities for the alternation of these walls.

In general the bearing capacity of the investigated elements is better than expected and therefore the buildings are very suitable for an extensive renovation, if the condition of the concrete permits. A renovation proposal should therefore be based on the condition of the concrete elements, since the overall condition will define the most economic renovation type. The investigations have been based on two specific buildings, however the results can be applied to other buildings of the same type as well.

Bibliography

[1] Tove Lading. *Statement from supervisor Tove Lading.*

[2] *Grønland.* Gyldendalske Boghandel, Nordisk Forlag A/S, Copenhagen, 1975.

[3] *Arktiske Anmeldelser.* https://arktiskeanmeldelser.wordpress.com/2013/12/05/udstilling-blok-p/. Accessed: 03-03-2020.

[4] *Sektorplaner for boligområdet.* https://naalakkersuisut.gl/da/Naalakkersuisut/Departementer/Boliger-og-Infrastruktur/Bolig-og-ejendomsafdelingen/Sektorplaner. Accessed: 09-03-2020.

[5] Andreas Plaugborg Pedersen & Charlotte Kloster Haugvaldstad. *Renovation of Greenlandic apartment blocks constructed in the 1960s and 1970s, with a focus on improving the thermal envelope.* 2019.

[6] *Asymmetriske altaner frembringer dynamisk facadeudtryk.* http://inspirationskatalog.dk/projekter/bygning-bolig/asymmetriske-altaner-frembringer-dynamisk-facadeudtryk/. Accessed: 09-03-2020.

[7] *Brændegårdsparken Herning.* https://www.jfp.dk/referencer/referencevisning/braendgaardsparken-herning-4. Accessed: 09-03-2020.

[8] *Afd. 7 Blokland Helhedsplan 2014.* https://albertslund.dk/Umbraco/openpublic/EsdhFile/Download?id=248&url=Bilag/Punkt_7_Bilag_3_Blokland__helhedsplan.pdf. Accessed: 14-03-2020.

[9] Jørgen Munch-Andersen & Jørgen Nielsen. *Sammenligning af normer for betonkonstruktioner 1949 og 2006.* Journal nr. 721-072. Statens Byggeforskningsinstitut, Aalborg Universitet, 2007.

[10] Grønlands Hjemmestyre & Grønlands Tekniske Organisation. *Foreløbige forskrifter udfærdigede i henhold til Grønlands Bygningsrelement 1982.* Namminersornerullutik Oqartussat - Grønlands Hjemmestyre, 1983.

[11] Grønlands Hjemmestyre & Bygge- og Anlægsstyrelsen. *Forskrift for betonkonstruktioner.* Sanaartortitsinermut Aqutsisoqarfik, 1986.

[12] Vejregler. *Vejledning til belastnings- og beregningsgrundlag for broer.* Transport-, Bygnings og Boligministeriet, 2017.

[13] Leif Eriksen & Vagn Thorsmark Arkitekter & M. Folmer Andersen Rådgivende Ingeniørfirma. *Særlig arbejdsbeskrivelse for udførelse af samtlige arbejder (Bygn. 10, 11 & 12).* 1972.

[14] Christian Munch-Petersen. *Betonhåndbogen - 3.2 Tilslag*. Dansk Betonforening, 2013.

[15] *Average weather data in Sisimiut*. `https://www.worldweatheronline.com/sisimiut-weather-averages/vestgronland/gl.aspx`. Accessed: 10-03-2020.

[16] Jens Mejer Frederiksen. *Betonhåndbogen - 19.2 Karbonatisering*. Dansk Betonforening, 2018.

[17] *Freeze Thaw Damage of Concrete – Effects & Preventive Measures*. `https://civildigital.com/freeze-thaw-damage-concrete-effects-preventive-measures/`. Accessed: 12-03-2020.

[18] Danish Standards Association. *Paints and varnishes - Corrosion protection of steel structures by protective paint systems - Part 2: Classification of environments (ISO 12944-2:2017)*. Dansk Standard, 2017.

[19] A.A. Mikhailov & P.V. Strekalov & Yu.M. Panchenko. *Atmospheric Corrosion of Metals in Regions of Cold and Extremely Cold Climate (a Review)*. Pleiades Publishing Ltd., 2007.

[20] *Operating Instructions Rebar Detection System Model S / SCANLOG*. Proceq SA Switzerland, 2007.

[21] *Operating Instructions Concrete Test Hammer*. Proceq SA Switzerland, 2006.

[22] Danish Standards. *DS/EN 1991-1-1:2007 - Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings*. 2nd edition. Dansk Standard, 2007.

[23] Danish Standards. *EN 1991-1-1 GL NA:2010 - Greenlandic national annex to Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings*. Preliminary edition, rev.1. Dansk Standard, 2010.

[24] Danish Standards. *EN 1991-1-3 GL NA:2010 - Greenlandic national annex to Eurocode 1: Actions on structures - Part 1-3: General actions - Snow loads*. Preliminary edition, rev.1. Dansk Standard, 2010.

[25] Danish Standards. *DS/EN 1991-1-3:2007 - Eurocode 1: Actions on structures - Part 1-3: General actions - Snow loads*. 2nd edition. Dansk Standard, 2007.

[26] Danish Standards. *EN 1991-1-4 GL NA:2010 - Greenlandic national annex to Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions*. Preliminary edition, rev.1. Dansk Standard, 2007.

[27] Danish Standards. *DS/EN 1998-1 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*. 1st edition. Dansk Standard, 2005.

[28] Danish Standards. *DS/EN 1990:2007 - Eurocode 0: Basis of Structural Design*. 2nd edition. Dansk Standard, 2007.

[29] Danish Standards. *EN 1990 GL NA:2010 - Greenlandic national annex to Eurocode 0: Basis of Structural Design*. Preliminary edition, rev.1. Dansk Standard, 2010.

[30] *Teknisk Ståbi*. 23rd edition. PRAXIS - Nyt Teknisk Forlag, 2015.

[31] Bjarne chr. Jensen. *Betonkonstruktioner efter DS/EN 1992-1-1*. 2nd edition. Nyt Teknisk Forlag, 2012.

[32] Egil Borchersen & Henning Larsen. *Skivebygningers Statik*. 2nd edition. Den polytekniske Læreanstalt, Danmarks tekniske Højskole, 1992.

[33] Mogens Peter Nielsen and Linh Cao Hoang. *Limit analysis and concrete plasticity*. 3rd edition. CRC Press, Taylor & Francis Group, 2011. ISBN: 978-1-4398-0396-7.

[34] Danish Standards. *DS/EN 1991-1-4:2007 - Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions*. 2nd edition. Dansk Standard, 2007.

APPENDIX A

Photos from Visual Registration

A.1 Damage Categories

The following images representing the three damage categories, where category 1 is no severe damage, category 2 is semi severe damage and category 3 is severe damage. All the images are from the north facade of the building, unless otherwise stated in the figure caption.

Figure A.1: Category 1. Smooth surface, intact console. Console 2, level 2, bottom part.

Figure A.2: Category 1. Slightly uneven top surface, no signs of cracks. Console 12, level 2, top part.

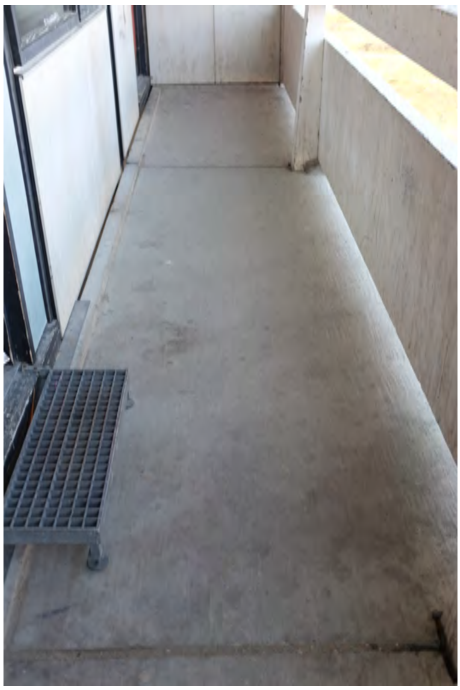


Figure A.3: Category 1. Smooth plate surface, no visible aggregates. Plate 2-3, level 3.

Figure A.4: Category 2. Small cracks. Console 11, level 2, top part.

Figure A.5: Category 2. Uneven concrete surface patch, signs of cracks. Console 10, level 3, bottom part.

Figure A.6: Category 2. Spalling of concrete, loose concrete surface, small cracks. Plate 5-6, level 3.

Figure A.7: Category 2. Loose/uneven concrete surface, visible aggregates. Plate 5-6, level 3, south.

Figure A.8: Category 3. Large cracks, will lead to broken off concrete. Console 7, level 2, bottom part.

Figure A.9: Category 3. Large cracks, broken off concrete, visible reinforcement. Console 10, level 2, top part.

Figure A.10: Category 3. Spalling of concrete surface, visible reinforcement. Console 10, level 3, south, top part.

Figure A.11: Category 3. Large cracks, uneven concrete surface. Console 9, level 3, bottom part.

Figure A.12: Category 3. Loose concrete surface, visible reinforcement. Plate 3-4, level 1.

Figure A.13: Category 3. Broken off concrete, visible reinforcement. Plate 9-10, level 1.

A.2 Gable Columns

Figure A.14: Gable column north-east.

Figure A.15: Gable column north-west.

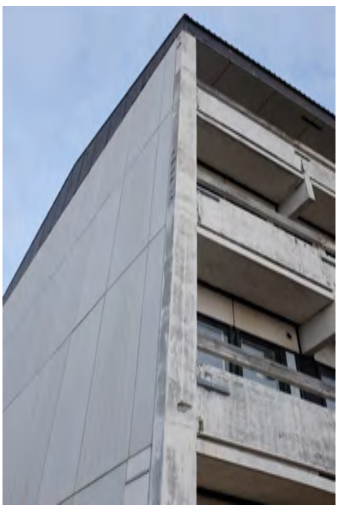


Figure A.16: Gable column south-west.

Figure A.17: Gable column south-east.

A.3 Tested Elements

Pictures of each of the tested structural elements are showed in this section.

A.3.1 Consoles

Figure A.18: Console 3, level 2, bottom part, good condition.

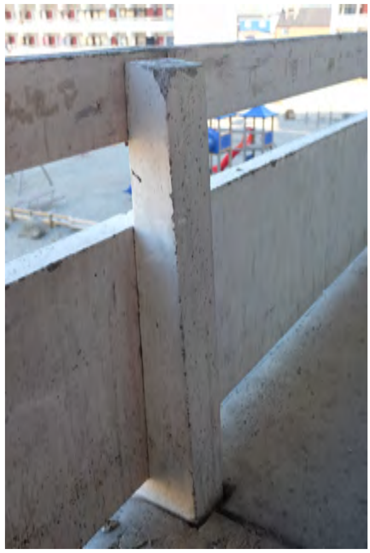


Figure A.19: Console 3, level 2, top part.

Figure A.20: Console 8, level 2, bottom part, bad condition.

Figure A.21: Console 8, level 2, top part.

Figure A.22: Console 10, level 2, bottom part, bad condition.

Figure A.23: Console 10, level 2, top part.

Figure A.24: Console 7, level 3, bottom part, good condition.

Figure A.25: Console 7, level 3, top part.

Figure A.26: Console 9, level 3, bottom part, bad condition.

Figure A.27: Console 9, level 3, top part.

Figure A.28: Console 11, level 3, bottom part, good condition.

Figure A.29: Console 11, level 3, top part.

A.3.2 Plates

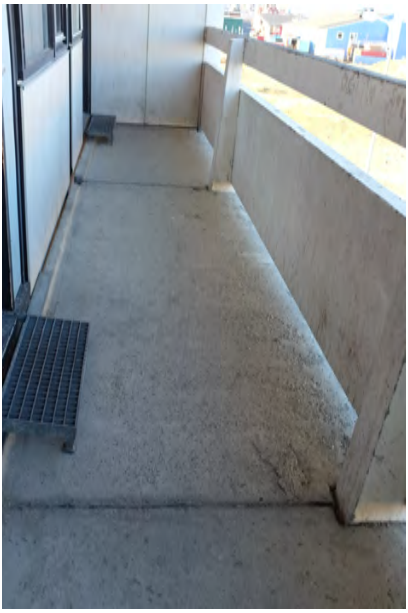


Figure A.30: Plate 2-3, level 2, bad condition.

Figure A.31: Plate 2-3, level 2, damage.

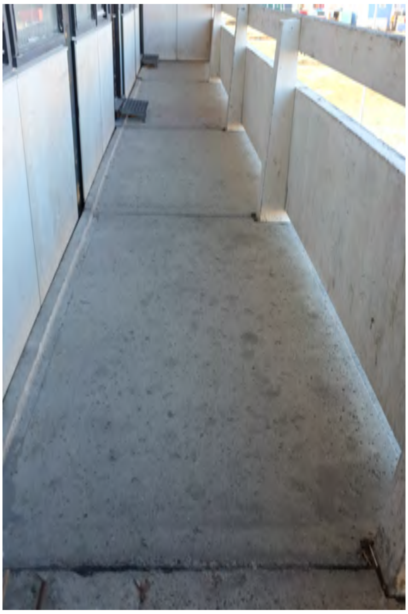


Figure A.32: Plate 4-5, level 2, good condition.

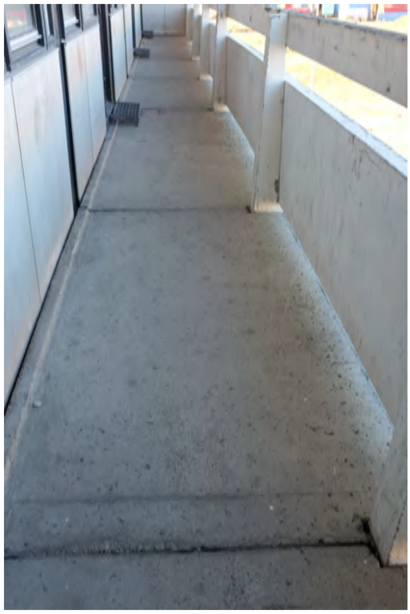


Figure A.33: Plate 6-7, level 2, good condition.

Figure A.34: Plate 12-13, level 2, bad condition.

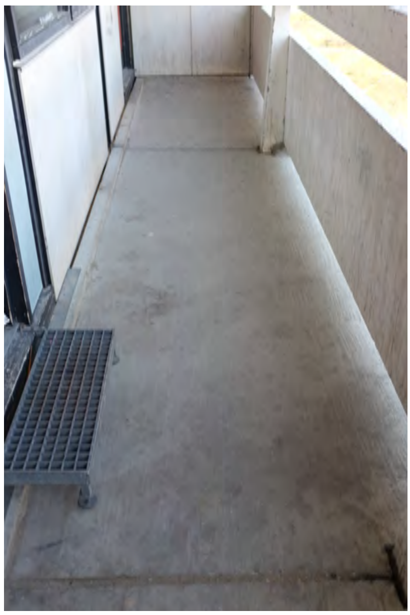


Figure A.35: Plate 2-3, level 3, good condition.

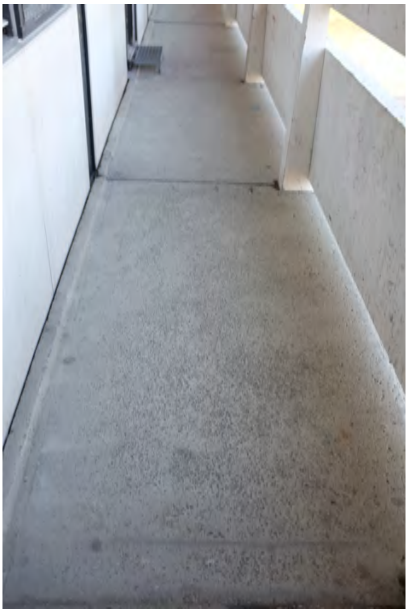


Figure A.36: Plate 11-12, level 3, bad condition.

Figure A.37: Plate 11-12, level 3, damage.

A.3.3 Gable Column and Gable Wall

Figure A.38: Gable wall, level 3.

Figure A.39: Gable column, level 0.

Figure A.40: Gable column, level 1.

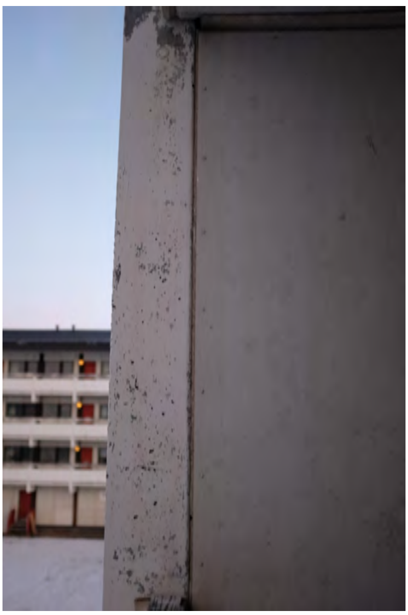


Figure A.41: Gable column, level 2.

Figure A.42: Gable column, level 3.

APPENDIX B

Field Work Data

All field work data from block 10 is presented in the following sections.

B.1 Damages

From the visual registration the damages for each structural element has been defined in the following tables. The tables describe where the element is placed (facade, floor and line), what type of element it is (type, drawing), the damage category and finally which type of damages that are found on the element.

Table B.1: Registered damages, north facade, top part of consoles.

Facade	Floor	Line	Type	Drawing	Damage category	Visible reinforce-ment	Concrete spalling	Uneven concrete surface	Cracks
north	1	2	large	1.18(26)	1			1	
north	1	3	large	1.18(26)	2		1	1	
north	1	4	large	1.18(26)	1				
north	1	5	large	1.18(26)	3	1	1	1	
north	1	6	large	1.18(26)	1				
north	1	7	large	1.18(26)	1				
north	1	8	large	1.18(26)	1		1		
north	1	9	large	1.18(26)	1				
north	1	10	large	1.18(26)	2		1	1	
north	1	11	large	1.18(26)	1				
north	1	12	large	1.18(26)	2		1	1	1
north	1	13	large	1.18(26)	1				
north	2	2	large	1.18(26)	1				
north	2	3	large	1.18(26)	1		1	1	
north	2	4	large	1.18(26)	1			1	
north	2	5	large	1.18(26)	3	1			1
north	2	6	large	1.18(26)	1				
north	2	7	large	1.18(26)	1				
north	2	8	large	1.18(26)	3	1	1		1
north	2	9	large	1.18(26)	1		1		
north	2	10	large	1.18(26)	3	1	1	1	1
north	2	11	large	1.18(26)	2				1
north	2	12	large	1.18(26)	1			1	
north	2	13	large	1.18(26)	1		1	1	
north	3	2	large	1.18(26)	1		1	1	
north	3	3	large	1.18(26)	1			1	
north	3	4	large	1.18(26)	1		1		
north	3	5	large	1.18(26)	2			1	
north	3	6	large	1.18(26)	1				
north	3	7	large	1.18(26)	3	1	1	1	1
north	3	8	large	1.18(26)	3	1	1		
north	3	9	large	1.18(26)	1		1		
north	3	10	large	1.18(26)	1			1	
north	3	11	large	1.18(26)	1				
north	3	12	large	1.18(26)	3	1	1	1	1
north	3	13	large	1.18(26)	2		1	1	
					Total:	7	17	17	7
					Total [%]:	19	47	47	19

Damage category	1	2	3
Total:	23	6	7
Total [%]:	64	17	19

Table B.2: Registered damages, north facade, bottom part of consoles.

Facade	Floor	Line	Type	Drawing	Damage category	Visible reinforce-ment	Concrete spalling	Uneven concrete surface	Cracks
north	1	2	large	1.18(26)	1				
north	1	3	large	1.18(26)	2		1	1	
north	1	4	large	1.18(26)	3				1
north	1	5	large	1.18(26)	1				
north	1	6	large	1.18(26)	1				
north	1	7	large	1.18(26)	1				
north	1	8	large	1.18(26)	1				
north	1	9	large	1.18(26)	2				1
north	1	10	large	1.18(26)	2				1
north	1	11	large	1.18(26)	2		1		
north	1	12	large	1.18(26)	2				1
north	1	13	large	1.18(26)	3				1
north	2	2	large	1.18(26)	1	1	1		
north	2	3	large	1.18(26)	1		1		
north	2	4	large	1.18(26)	1			1	
north	2	5	large	1.18(26)	2				
north	2	6	large	1.18(26)	2			1	
north	2	7	large	1.18(26)	3		1		1
north	2	8	large	1.18(26)	3		1	1	1
north	2	9	large	1.18(26)	2		1	1	1
north	2	10	large	1.18(26)	3		1		1
north	2	11	large	1.18(26)	1				
north	2	12	large	1.18(26)	2				1
north	2	13	large	1.18(26)	2			1	
north	3	2	large	1.18(26)	1		1		1
north	3	3	large	1.18(26)	1				1
north	3	4	large	1.18(26)	1				
north	3	5	large	1.18(26)	1		1		
north	3	6	large	1.18(26)	1				
north	3	7	large	1.18(26)	3				1
north	3	8	large	1.18(26)	1				
north	3	9	large	1.18(26)	3		1	1	1
north	3	10	large	1.18(26)	2		1	1	
north	3	11	large	1.18(26)	1			1	
north	3	12	large	1.18(26)	2		1		
north	3	13	large	1.18(26)	2			1	1
					Total:	1	13	10	15
					Total [%]:	3	36	28	42

Damage category	1	2	3
Total:	16	13	7
Total [%]:	44	36	19

Table B.3: Registered damages, north facade, plates.

Facade	Floor	Deck	Type	Drawing	Damage category	Visible reinforce-ment	Broken off concrete, edges	Loose concrete surface	Cracks
north	1	1-2	corner	201.20(26)	1				
north	1	2-3	large	1.1(26)	2				
north	1	3-4	small	1.2(26)	3	1	1	1	
north	1	4-5	small	1.2(26)	1		1		
north	1	5-6	large	1.1(26)	3	1	1		1
north	1	6-7	small	1.2(26)	2		1		1
north	1	7-8	large	1.1(26)	3	1		1	1
north	1	8-9	small	1.2(26)	1				
north	1	9-10	large	1.1(26)	3	1	1	1	1
north	1	10-11	small	1.2(26)	1				
north	1	11-12	small	1.2(26)	1				
north	1	12-13	large	1.1(26)	2			1	
north	1	13-14	corner	201.21(26)	1				
north	2	1-2	corner	201.20(26)	1				
north	2	2-3	large	1.1(26)	3	1	1	1	1
north	2	3-4	small	1.2(26)	1				
north	2	4-5	small	1.2(26)	1				
north	2	5-6	large	1.1(26)	3	1			
north	2	6-7	small	1.2(26)	1				
north	2	7-8	large	1.1(26)	1				
north	2	8-9	small	1.2(26)	1				
north	2	9-10	large	1.1(26)	1				
north	2	10-11	small	1.2(26)	2				
north	2	11-12	small	1.2(26)	1				
north	2	12-13	large	1.1(26)	2			1	1
north	2	13-14	corner	201.21(26)	1				
north	3	1-2	corner	201.20(26)	1				
north	3	2-3	large	1.1(26)	1				
north	3	3-4	small	1.2(26)	1				
north	3	4-5	small	1.2(26)	1				
north	3	5-6	large	1.1(26)	2	1			1
north	3	6-7	small	1.2(26)	1				
north	3	7-8	large	1.1(26)	2			1	1
north	3	8-9	small	1.2(26)	1				
north	3	9-10	large	1.1(26)	2				
north	3	10-11	small	1.2(26)	3	1	1		
north	3	11-12	small	1.2(26)	3	1		1	1
north	3	12-13	large	1.1(26)	2	1	1		
north	3	13-14	corner	201.21(26)	1				
					Total:	10	8	8	9
					Total [%]:	26	21	21	23

Damage category	1	2	3
Total:	22	9	8
Total [%]:	56	23	21

Table B.4: Registered damages, south facade, top part of consoles.

Facade	Floor	Line	Type	Drawing	Damage category	Visible reinforce-ment	Concrete spalling	Uneven concrete surface	Cracks
south	1	2	small	1.19(26)	1				
south	1	3	small	1.19(26)	1				
south	1	4	small	1.19(26)	1				
south	1	5	small	1.19(26)	1				
south	1	6	small	1.19(26)	1				
south	1	7	small	1.19(26)	1				
south	1	8	small	1.19(26)	1				
south	1	9	small	1.19(26)	3	1	1		
south	1	10	small	1.19(26)	1			1	
south	1	11	small	1.19(26)	2				1
south	1	12	small	1.19(26)	2			1	
south	1	13	small	1.19(26)	1			1	
south	2	2	small	1.19(26)	1			1	
south	2	3	small	1.19(26)	1			1	
south	2	4	small	1.19(26)	2	1			1
south	2	5	small	1.19(26)	3	1	1		1
south	2	6	small	1.19(26)	2			1	1
south	2	7	small	1.19(26)	2				1
south	2	8	small	1.19(26)	3				1
south	2	9	small	1.19(26)	1				
south	2	10	small	1.19(26)	1				
south	2	11	small	1.19(26)	3				1
south	2	12	small	1.19(26)	2			1	
south	2	13	small	1.19(26)	2				1
south	3	2	small	1.19(26)	2				1
south	3	3	small	1.19(26)	2				1
south	3	4	small	1.19(26)	1				
south	3	5	small	1.19(26)	1				
south	3	6	small	1.19(26)	1				
south	3	7	small	1.19(26)	2				1
south	3	8	small	1.19(26)	1				
south	3	9	small	1.19(26)	1				
south	3	10	small	1.19(26)	3	1			1
south	3	11	small	1.19(26)	1				
south	3	12	small	1.19(26)	1				
south	3	13	small	1.19(26)	1				
					Total:	4	2	7	12
					Total [%]:	11	6	19	33

Damage category	1	2	3
Total:	21	10	5
Total [%]:	58	28	14

Table B.5: Registered damages, south facade, bottom part of consoles.

Facade	Floor	Line	Type	Drawing	Damage category	Visible reinforce-ment	Concrete spalling	Uneven concrete surface	Cracks
south	1	2	small	1.19(26)	2			1	
south	1	3	small	1.19(26)	1		1		
south	1	4	small	1.19(26)	1				
south	1	5	small	1.19(26)	1				
south	1	6	small	1.19(26)	1				
south	1	7	small	1.19(26)	2		1		
south	1	8	small	1.19(26)	1				
south	1	9	small	1.19(26)	1				
south	1	10	small	1.19(26)	1				
south	1	11	small	1.19(26)	1				
south	1	12	small	1.19(26)	1				
south	1	13	small	1.19(26)	1				
south	2	2	small	1.19(26)	1				
south	2	3	small	1.19(26)	1				
south	2	4	small	1.19(26)	1				
south	2	5	small	1.19(26)	2			1	
south	2	6	small	1.19(26)	2			1	
south	2	7	small	1.19(26)	1				
south	2	8	small	1.19(26)	1				
south	2	9	small	1.19(26)	1				
south	2	10	small	1.19(26)	1				
south	2	11	small	1.19(26)	2		1		1
south	2	12	small	1.19(26)	3				1
south	2	13	small	1.19(26)	1		1		
south	3	2	small	1.19(26)	1				
south	3	3	small	1.19(26)	2				1
south	3	4	small	1.19(26)	1				
south	3	5	small	1.19(26)	3		1		
south	3	6	small	1.19(26)	1				
south	3	7	small	1.19(26)	2				1
south	3	8	small	1.19(26)	1				
south	3	9	small	1.19(26)	1				
south	3	10	small	1.19(26)	1				
south	3	11	small	1.19(26)	3				1
south	3	12	small	1.19(26)	2			1	
south	3	13	small	1.19(26)	1				
					Total:	0	5	4	5
					Total [%]:	0	14	11	14

Damage category	1	2	3
Total:	25	8	3
Total [%]:	69	22	8

Table B.6: Registered damages, south facade, plates.

Facade	Floor	Deck	Type	Drawing	Damage category	Visible reinforce-ment	Broken off concrete, edges	Loose concrete surface	Cracks
South	1	1-2	corner	201.23(26)	2			1	
South	1	2-3	large	1.3(26)	3	1		1	
South	1	3-4	small	1.4(26)	3	1			
South	1	4-5	small	1.4(26)	1				
South	1	5-6	large	1.3(26)	3	1	1		
South	1	6-7	small	1.4(26)	2				
South	1	7-8	large	1.3(26)	1		1		
South	1	8-9	small	1.4(26)	2				
South	1	9-10	large	1.3(26)	2		1		
South	1	10-11	small	1.4(26)	2		1		
South	1	11-12	small	1.4(26)	1			1	
South	1	12-13	large	1.3(26)	3	1			1
South	1	13-14	corner	201.22(26)	1				
South	2	1-2	corner	201.23(26)	1				
South	2	2-3	large	1.3(26)	2			1	
South	2	3-4	small	1.4(26)	2			1	
South	2	4-5	small	1.4(26)	2			1	
South	2	5-6	large	1.3(26)	2			1	
South	2	6-7	small	1.4(26)	2		1	1	
South	2	7-8	large	1.3(26)	2			1	
South	2	8-9	small	1.4(26)	2				
South	2	9-10	large	1.3(26)	2			1	
South	2	10-11	small	1.4(26)	2			1	
South	2	11-12	small	1.4(26)	2			1	
South	2	12-13	large	1.3(26)	2			1	
South	2	13-14	corner	201.22(26)	3	1		1	
South	3	1-2	corner	201.23(26)	1				
South	3	2-3	large	1.3(26)	2			1	
South	3	3-4	small	1.4(26)	1			1	
South	3	4-5	small	1.4(26)	2			1	
South	3	5-6	large	1.3(26)	3	1		1	
South	3	6-7	small	1.4(26)	1				
South	3	7-8	large	1.3(26)	1				
South	3	8-9	small	1.4(26)	1				
South	3	9-10	large	1.3(26)	2			1	
South	3	10-11	small	1.4(26)	1				
South	3	11-12	small	1.4(26)	1				
South	3	12-13	large	1.3(26)	2			1	
South	3	13-14	corner	201.22(26)	2				
					Total:	6	5	20	1
					Total [%]:	15	13	51	3

Damage category	1	2	3
Total:	12	21	6
Total [%]:	31	54	15

B.2 Cover Layer Measurements

The cover layer thickness was measured with a Covermeter which also measured the diameter of the reinforcement. In general ten measurements were taken on every element, but for consoles twenty measurements were conducted, ten on each side of the console. The average cover layer thickness and diameter are given in the tables, as well ass the standard deviation.

Table B.7: Cover layer measurements, north facade, consoles 2. floor.

2. floor, console 3, north, good

Average cover layer [mm]: 27.9
Standard deviation [mm]: 2.3

Average diameter [mm]: 20.6
Standard deviation [mm]: 2.5

Coverlayer c [mm]	Diameter d [mm]	Notes
27	20.6	east
31	26.3	east
26	19.1	east
28	20.0	east
25	18.7	east
26	18.6	east
27	22.0	east
25	18.2	east
23	24.8	east
26	20.1	east
31	24.7	east
33	20.5	east
28	23.8	west
29	20.4	west
30	19.0	west
28	18.7	west
29	17.6	west
28	18.0	west
28	22.8	west
30	19.0	west

2. floor, console 8, north, bad

Average cover layer [mm]: 26.8
Standard deviation [mm]: 4.3

Average diameter [mm]: 19.9
Standard deviation [mm]: 4.0

Coverlayer c [mm]	Diameter d [mm]	Notes
25	20.1	east
25	21.5	east
24	18.1	east
25	17.5	east
33	17.9	east
23	17.0	east
23	21.3	east
26	17.5	east
22	22.5	east
24	22.5	east
31	26.5	east
31	26.9	east
28	22.9	west
15	7.0	west
28	20.3	west
33	20.1	west
28	18.8	west
30	18.7	west
29	20.7	west
32	19.9	west

2. floor, console 10, north, bad

Average cover layer [mm]: 27.1
Standard deviation [mm]: 3.0

Average diameter [mm]: 16.4
Standard deviation [mm]: 3.0

Coverlayer c [mm]	Diameter d [mm]	Notes
25	18.0	east
25	22.2	east
23	17.7	east
25	18.4	east
24	18.0	east
23	17.9	east
23	22.1	east
23	17.7	east
25	18.7	east
26	18.2	east
30	14.5	east
30	16.0	east
30	13.7	west
30	12.6	west
31	13.2	west
31	11.5	west
30	14.4	west
30	12.2	west
28	16.9	west
29	14.2	west

Table B.8: Cover layer measurements, north facade, consoles 3. floor.

3. floor, console 7, north, good

Average cover layer [mm]: 25.2
Standard deviation [mm]: 2.5

Average diameter [mm]: 21.2
Standard deviation [mm]: 2.3

Coverlayer c [mm]	Diameter d [mm]	Notes
21	18.2	east
20	18.3	east
23	18.0	east
22	18.3	east
26	22.8	east
25	18.8	east
27	23.8	east
28	25.0	east
28	24.5	east
30	20.1	east
27	20.1	east
28	21.2	east
26	19.4	west
27	20.8	west
24	22.5	west
26	20.4	west
23	24.0	west
25	23.1	west
23	23.9	west
25	20.7	west

3. floor, console 9, north, bad

Average cover layer [mm]: 26.8
Standard deviation [mm]: 2.7

Average diameter [mm]: 19.1
Standard deviation [mm]: 2.2

Coverlayer c [mm]	Diameter d [mm]	Notes
23	17.6	east
22	20.0	east
24	17.0	east
24	18.8	east
28	19.0	east
26	19.1	east
29	20.8	east
28	18.3	east
27	22.0	east
27	18.5	east
27	21.8	east
21	13.7	east
26	20.1	west
27	20.7	west
27	17.7	west
29	18.3	west
30	22.8	west
30	22.2	west
30	16.9	west
31	17.1	west

3. floor, console 11, north, good

Average cover layer [mm]: 26.9
Standard deviation [mm]: 2.3

Average diameter [mm]: 20.0
Standard deviation [mm]: 2.0

Coverlayer c [mm]	Diameter d [mm]	Notes
30	22.7	east
25	25.2	east
25	18.6	east
25	21.2	east
34	22.4	east
28	17.5	east
28	19.5	east
27	17.8	east
29	19.5	east
27	19.1	east
27	18.7	east
28	22.4	east
26	19.1	west
26	20.6	west
25	18.0	west
26	18.3	west
23	20.1	west
24	17.0	west
27	21.4	west
28	20.0	west

Table B.9: Cover layer measurements, north facade, plates.

2. floor, plate 2-3, long, bad

Average cover layer [mm]: 28.2
Standard deviation [mm]: 2.1

Average diameter [mm]: 11.3
Standard deviation [mm]: 2.7

Coverlayer c [mm]	Diameter d [mm]
27	15.2
28	15.2
29	14.8
28	10.4
27	9.7
25	10.3
29	9.9
33	8.2
30	7.3
26	11.7

2. floor, plate 4-5, short, good

Average cover layer [mm]: 28.3
Standard deviation [mm]: 2.3

Average diameter [mm]: 11.2
Standard deviation [mm]: 3.4

Coverlayer c [mm]	Diameter d [mm]
29	8.5
30	10.0
32	19.3
30	7.4
30	14.4
28	10.8
28	12.8
25	10.2
24	10.9
27	7.7

2. floor, plate 6-7, short, good

Average cover layer [mm]: 28.2
Standard deviation [mm]: 2.1

Average diameter [mm]: 11.3
Standard deviation [mm]: 2.7

Coverlayer c [mm]	Diameter d [mm]
27	15.0
28	15.2
29	14.8
28	10.4
27	9.7
25	10.3
29	9.9
33	8.2
30	7.3
26	11.7

2. floor, plate 12-13, long, bad

Average cover layer [mm]: 24.0
Standard deviation [mm]: 5.1

Average diameter [mm]: 16.3
Standard deviation [mm]: 2.8

Coverlayer c [mm]	Diameter d [mm]
24	15.0
23	15.6
22	15.8
33	14.8
33	15.9
19	14.8
17	11.5
26	17.8
22	21.1
21	20.8

3. floor, plate 2-3, long, good

Average cover layer [mm]: 22.5
Standard deviation [mm]: 3.5

Average diameter [mm]: 15.0
Standard deviation [mm]: 1.5

Coverlayer c [mm]	Diameter d [mm]
25	15.8
25	13.5
24	16.4
30	13.8
24	11.9
21	16.3
19	16.5
19	15.5
19	14.5
19	16.0

3. floor, plate 11-12, short, bad

Average cover layer [mm]: 26.9
Standard deviation [mm]: 2.7

Average diameter [mm]: 9.8
Standard deviation [mm]: 2.9

Coverlayer c [mm]	Diameter d [mm]
31	10.8
29	15.5
27	7.0
24	8.3
22	7.0
30	12.9
28	7.1
28	8.7
25	7.6
25	12.6

Table B.10: Cover layer measurements, north-east gable columns.

0. floor, gable column 14, A

Average cover layer [mm]: 32.5
Standard deviation [mm]: 3.6

Average diameter [mm]: 10.3
Standard deviation [mm]: 1.0

Coverlayer c [mm]	Diameter d [mm]
27	8.9
31	10.4
35	9.3
35	10.0
33	10.9
27	11.9
35	11.1
39	11.0
30	8.9
33	10.1

1. floor, gable column 14, A

Average cover layer [mm]: 37.2
Standard deviation [mm]: 4.3

Average diameter [mm]: 10.0
Standard deviation [mm]: 1.5

Coverlayer c [mm]	Diameter d [mm]
38	9.8
34	9.3
37	9.4
30	9.7
39	12.9
39	8.7
41	10.9
35	10.7
46	7.2
33	11.4

2. floor, gable column 14, A

Average cover layer [mm]: 36.0
Standard deviation [mm]: 4.3

Average diameter [mm]: 10.0
Standard deviation [mm]: 2.1

Coverlayer c [mm]	Diameter d [mm]
26	15.3
39	12.0
37	9.5
37	8.4
36	10.1
34	9.3
42	9.5
34	8.1
41	8.7
34	8.6

3. floor, gable column 14, A

Average cover layer [mm]: 42.8
Standard deviation [mm]: 8.7

Average diameter [mm]: 16.9
Standard deviation [mm]: 9.6

Coverlayer c [mm]	Diameter d [mm]
39	10.5
52	7.5
34	10.5
49	10.9
28	19.4
44	8.3
36	28.4
54	27.0
38	10.6
54	35.8

B.3 Carbonation Depth Tests

In order to determine the carbonation depth three holes were drilled in each element. The depth of each hole is given in the following tables.

Table B.11: Carbonation depth, north, consoles.

2. floor, console bottom 3, A-B, good, drill 10mm, east side		
Test	Depth	Notes
1	20-25mm	Blue
2	20-25mm	Blue
3	25-30mm	Blue
Average: 24.2mm		Standard deviation: 2.4mm

2. floor, console bottom 8, A-B, bad, drill 10mm, east side		
Test	Depth	Notes
1	15-20mm	Blue
2	5-10mm	Blue
3	15-20mm	Blue
Average: 14.2mm		Standard deviation: 4.7mm

2. floor, console bottom 10, A-B, bad, drill 10mm, east side		
Test	Depth	Notes
1	45-50mm	Blue
2	60-65mm	Not blue, experiment was stopped
3	0-5mm	Very blue
Average: 37.5mm		Standard deviation: 25.5mm

3. floor, console bottom 7, A-B, good, drill 8mm, east side		
Test	Depth	Notes
1	25-30mm	Very blue
2	20-25mm	A little blue in the middle
3	20-25mm	Half blue, but definetely blue
Average: 24.2mm		Standard deviation: 2.4mm

3. floor, console bottom 9, A-B, bad, drill 10mm, west side		
Test	Depth	Notes
1	50-55mm	It seems a little bit blue, test stopped
2	25-30mm	Test was stopped, there was a stone in the way
3	50-55mm	Definetely blue
Average: 44.2mm		Standard deviation: 11.8mm

3. floor, console bottom 11, A-B, good, drill 10mm, east side		
Test	Depth	Notes
1	15-20mm	Blue
2	5-10mm	Blue
3	20-25mm	A little blue
Average: 15.8mm		Standard deviation: 6.2mm

Table B.12: Carbonation depth, north, top of plates.

2. floor, plate top 2-3, A-B, long, bad, drill 10mm		
Test	Depth	Notes
1	35-40mm	Blue
2	30-35mm	Blue
3	30-35mm	Blue
Average: 34.2mm		Standard deviation: 2.4mm

2. floor, plate top 4-5, A-B, short, good, drill 10mm		
Test	Depth	Notes
1	0-5mm	Blue
2	0-5mm	Blue
3	0-5mm	Blue
Average: 2.5mm		Standard deviation: 0.0mm

2. floor, plate top 6-7, A-B, short, good, drill 10mm		
Test	Depth	Notes
1	0-5mm	Blue
2	0-5mm	Blue
3	0-5mm	Blue
Average: 2.5mm		Standard deviation: 0.0mm

2. floor, plate top 12-13, A-B, long, bad, drill 10mm		
Test	Depth	Notes
1	30-35mm	Blue
2	35-40mm	Blue
3	20-25mm	Blue
Average: 30.8mm		Standard deviation: 6.2mm

3. floor, plate top 2-3, A-B, long, good, drill 10mm		
Test	Depth	Notes
1	0-5mm	Blue
2	0-5mm	Blue
3	0-5mm	Blue
Average: 2.5mm		Standard deviation: 0.0mm

3. floor, plate top 11-12, A-B, short, bad, drill 10mm		
Test	Depth	Notes
1	20-25mm	Blue
2	30-35mm	Blue
3	30-35mm	Blue
Average: 29.2mm		Standard deviation: 4.7mm

Table B.13: Carbonation depth, north, bottom of plates.

2. floor, plate bottom 4-5, A-B, short, good, drill 10mm		
Test	Depth	Notes
1	5-10mm	Blue
2	10-15mm	Blue in the middle
3	0-5mm	Very blue
Average: 7.5mm		Standard deviation: 4.1mm

2. floor, plate bottom 12-13, A-B, long, bad, drill 10mm		
Test	Depth	Notes
1	35-40mm	Blue
2	35-40mm	Blue
3	35-40mm	Blue
Average: 37.5mm		Standard deviation: 0.0mm

3. floor, plate bottom 2-3, A-B, long, good, drill 10mm		
Test	Depth	Notes
1	10-15mm	Partly blue
2	20-25mm	A little blue
3	20-25mm	Blue
Average: 19.2mm		Standard deviation: 4.7mm

3. floor, plate bottom 11-12, A-B, short, bad, drill 10mm		
Test	Depth	Notes
1	25-30mm	Partly blue
2	30-35mm	A little blue
3	35-40mm	Blue
Average: 32.5mm		Standard deviation: 4.1mm

Table B.14: Carbonation depth, north-east gable.

3. floor, gable 14 east, A-B, drill 10mm		
Test	Depth	Notes
1	0-5mm	Blue
Average: 0.5mm		Standard deviation: 0.0mm

0. floor, gable column 14, A, drill 10mm		
Test	Depth	Notes
1	0-5mm	Blue
2	0-5mm	Blue
3	0-5mm	Blue
Average: 2.5mm		Standard deviation: 0.0mm

1. floor, gable column 14, A, drill 10mm		
Test	Depth	Notes
1	10-15mm	Blue
2	10-15mm	Blue
3	10-15mm	Blue
Average: 12.5mm		Standard deviation: 0.0mm

2. floor, gable column 14, A, drill 10mm		
Test	Depth	Notes
1	0-5mm	A little blue
2	0-5mm	Blue
3	0-5mm	Blue
Average: 2.5mm		Standard deviation: 0.0mm

3. floor, gable column 14, A, drill 10mm		
Test	Depth	Notes
1	10-15mm	Blue
2	10-15mm	Blue
3	5-10mm	Blue
Average: 10.8mm		Standard deviation: 2.4mm

B.4 Strength Measurements

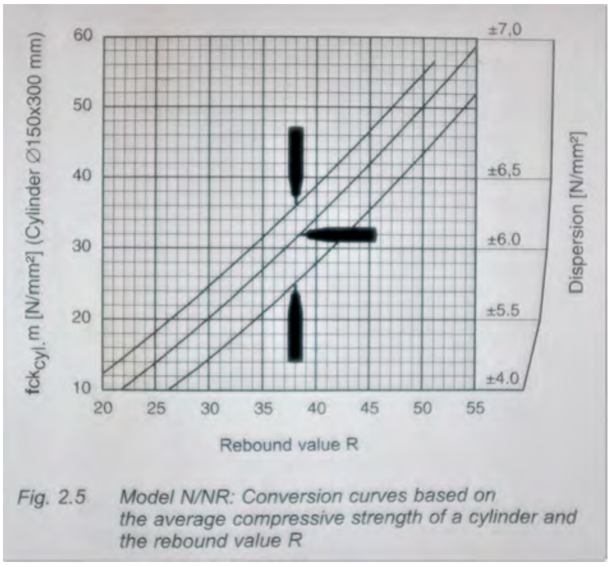


Figure B.1: Schmidt Hammer conversion curve [21].

Table B.15: Concrete strength, north, consoles.

2. floor, console top 3, A, horizontal, good											
Rebound values R		34	34	34	35	37	39	39	41	42	42
Mean value	37.7										
Strength [MPa]	31										
Median value	38.0										
Strength [MPa]	31.5										

2. floor, console top 8, A, horizontal, bad											
Rebound values R		29	31	32	32	33	36	36	37	38	39
Mean value	34.3										
Strength [MPa]	26										
Median value	34.5										
Strength [MPa]	26										

2. floor, console top 10, A, horizontal, bad											
Rebound values R		27	28	28	29	29	30	32	32	32	35
Mean value	30.2										
Strength [MPa]	20										
Median value	29.5										
Strength [MPa]	19.5										

3. floor, console top 7, A, horizontal, good											
Rebound values R		30	34	34	36	38	40	40	41	42	43
Mean value	37.8										
Strength [MPa]	31										
Median value	39.0										
Strength [MPa]	33										

3. floor, console top 9, A, horizontal, bad											
Rebound values R		22	25	26	30	32	32	33	34	36	36
Mean value	31.6										
Strength [MPa]	22										
Median value	32.0										
Strength [MPa]	23										

3. floor, console top 11, A, horizontal, good											
Rebound values R		34	35	37	40	43	43	44	45	48	48
Mean value	41.7										
Strength [MPa]	37										
Median value	43.0										
Strength [MPa]	39										

Table B.16: Concrete strength, north, plates.

2. floor, plate top 2-3, A-B, vertical, bad											
Rebound values R		25	26	30	30	31	31	32	36	37	38
Mean value	31.6										
Strength [MPa]	27										
Median value	31.0										
Strength [MPa]	26										

2. floor, plate top 4-5, A-B, vertical, good											
Rebound values R		36	37	38	38	38	38	39	40	45	45
Mean value	39.4										
Strength [MPa]	38										
Median value	38.0										
Strength [MPa]	36										

2. floor, plate top 6-7, A-B, vertical, good											
Rebound values R		33	37	39	40	40	41	42	43	44	49
Mean value	40.8										
Strength [MPa]	40										
Median value	40.5										
Strength [MPa]	40										

2. floor, plate top 12-13, A-B, vertical, bad											
Rebound values R		29	30	32	33	33	34	36	37	38	40
Mean value	34.2										
Strength [MPa]	30										
Median value	33.5										
Strength [MPa]	30										

3. floor, plate top 2-3, A-B, vertical, good											
Rebound values R		40	40	41	41	42	43	43	44	45	49
Mean value	42.8										
Strength [MPa]	43										
Median value	42.5										
Strength [MPa]	43										

3. floor, plate top 11-12, A-B, vertical, bad											
Rebound values R		28	28	31	32	33	33	34	35	37	39
Mean value	33.0										
Strength [MPa]	29										
Median value	33.0										
Strength [MPa]	29										

Table B.17: Concrete strength, north-east gable.

3. floor, gable wall east 14, A-B, horizontal											
Rebound values R		43	48	48	50	50	50	51	52	52	53
Mean value	49.7										
Strength [MPa]	50										
Median value	50.0										
Strength [MPa]	50										

0. floor, gable column north-east 14, A, horizontal											
Rebound values R		45	48	48	50	51	51	51	54	54	55
Mean value	50.7										
Strength [MPa]	52										
Median value	51.0										
Strength [MPa]	52										

1. floor, gable column north-east 14, A, horizontal											
Rebound values R		33	35	40	41	41	42	46	46	47	51
Mean value	42.2										
Strength [MPa]	38										
Median value	41.5										
Strength [MPa]	37										

2. floor, gable column north-east 14, A, horizontal											
Rebound values R		43	44	45	47	48	48	49	50	50	51
Mean value	47.5										
Strength [MPa]	46										
Median value	48.0										
Strength [MPa]	47										

3. floor, gable column north-east 14, A, horizontal											
Rebound values R		44	45	46	46	46	48	49	50	53	53
Mean value	48.0										
Strength [MPa]	48										
Median value	47.0										
Strength [MPa]	48										

APPENDIX C

Calculations

C.1 Basis for Calculations

Eurocodes:

- DS/EN 1990:2007 - Eurocode 0: Basis of structural design
- DS/EN 1991-1-1:2007 - Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings
- DS/EN 1991-1-3:2007 - Eurocode 1: Actions on structures - Part 1-3: General actions - Snow loads
- DS/EN 1991-1-4:2007 - Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions
- DS/EN 1992-1-1:2008 - Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings

Greenlandic national annexes:

- EN 1990 GL NA:2010 - Greenlandic national annex to Eurocode 0: Basis of structural design
- EN 1991-1-1 GL NA:2010 - Greenlandic national annex to Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings
- EN 1991-1-3 GL NA:2010 - Greenlandic national annex to Eurocode 1: Actions on structures - Part 1-3: General actions - Snow loads
- EN 1991-1-4 GL NA:2010 - Greenlandic national annex to Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions
- DS/EN 1992-1-1 GL NA:2009 - Greenlandic national annex to Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings

Books:

- Teknisk Ståbi [30]

- Betonkonstruktioner efter DS/EN 1992-1-1 [31]
- Limit analysis and concrete plasticity [33]

C.2 Self Weight of New Elements

C.2.1 Facade

The design of the new facade is based on a design from another thesis project concerning the thermal envelope [5]. The weight of the facade will now be determined. The design of the facade is described below and the facade densities are defined in Table C.1.

External cladding - 10mm

Ventilated air cavity - 10mm

CEMBRIT Windstopper - 9mm

Insulation - 45mm
(45x45mm horizontal timber laths, c/c 600mm)

Insulation - 200mm
(200x45mm vertical timber laths, c/c 600mm)

OSB plate - 12mm

Vapor barrier

Insulation - 45mm
(45x45mm horizontal timber laths, c/c 600mm)

Gypsum board - 2x12.5mm

Table C.1: Material densities, facade.

Material	Dimension	Self weight
Wood cladding[1]:	t =10mm	38.0N/m^2
Wood 45x45mm[1]:	b =45mm x h =45mm	7.7N/m
Wood 45x200mm[1]:	b =45mm x h =200mm	34.2N/m
CEMBRIT windstopper:	t =9mm	134.5N/m^2
ISOVER facade insulation 32:	t =45mm	14.6N/m^2
ISOVER facade insulation 32:	t = 200mm	64.8N/m^2
OSB flake board[22]:	t =12mm	84.0N/m^2
Gyproc gypsum board:	t = 12.5mm	65.8N/m^2
Velfac window 3-layers:	3 x t =4mm	2946.0N/m^2
[1]Timber, strength class C18[22]:		38.0kN/m^3

The modules in the building are either 2.7 m or 3.6 m wide. Therefore the facade elements will be either 0.9 m or 1.2 m wide. The window area and wall area of each of the two elements (1.2m and 0.9m) are given in Table C.2.

Table C.2: Areas of wall elements.

	Element 1.2m	Element 0.9m
A_{tot}:	$1.2\text{m} \cdot 2.8\text{m} = 3.36\text{m}^2$	$0.90\text{m} \cdot 2.8\text{m} = 2.52\text{m}^2$
A_{window}:	$1.11\text{m} \cdot 1.11\text{m} = 1.23\text{m}^2$	$0.81\text{m} \cdot 1.11\text{m} = 0.90\text{m}^2$
A_{wall}:	$A_{tot} - A_{window} = 2.13\text{m}^2$	$A_{tot} - A_{window} = 1.62\text{m}^2$

The dimensions of each material are defined in Table C.3 and Table C.4 for the two different elements respectively. The measurements of the elements are illustrated in Figure C.1 and C.2.

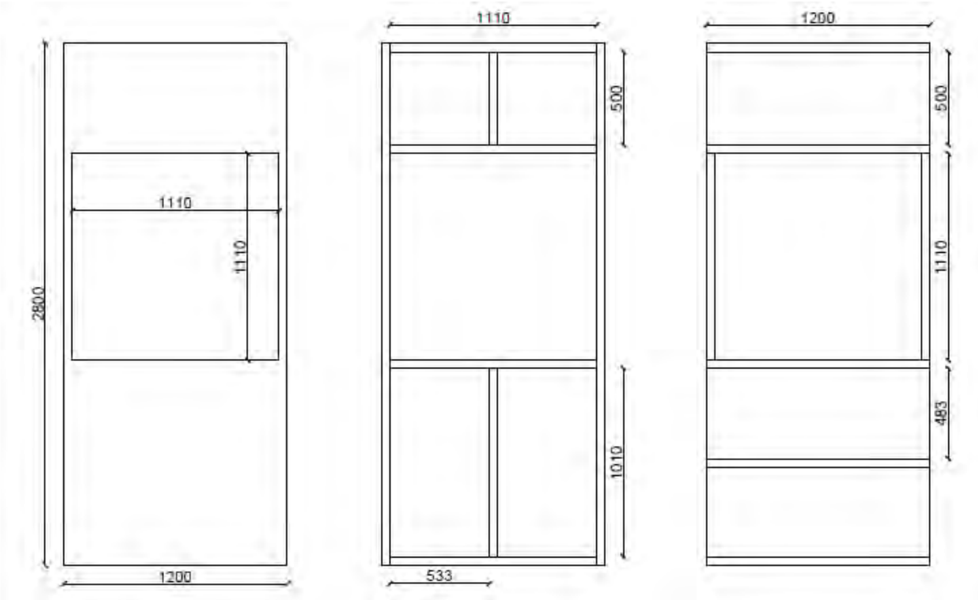


Figure C.1: Facade dimensions [mm] for 1.2m element. Wall area, placement of 45x45 timber laths and placement of 200x45mm timber laths.

Table C.3: Self weight, facade element 1.2m.

Material	Dimension	Self weight
Wood cladding:	A_{wall}	0.08kN
Wood 45x45mm:	2 x 8.22m	0.13kN
Wood 45x200mm:	11.55m	0.40kN
CEMBRIT windstopper:	A_{wall}	0.29kN
ISOVER facade insulation 45 mm:	2 x 1.76m^2	0.05kN
ISOVER facade insulation 200 mm:	1.61m^2	0.10kN
OSB flake board[22]:	A_{wall}	0.18kN
Gyproc gypsum board:	2 x A_{wall}	0.28kN
Velfac window 3-layers:	A_{window}	3.63kN
Total:		5.13kN

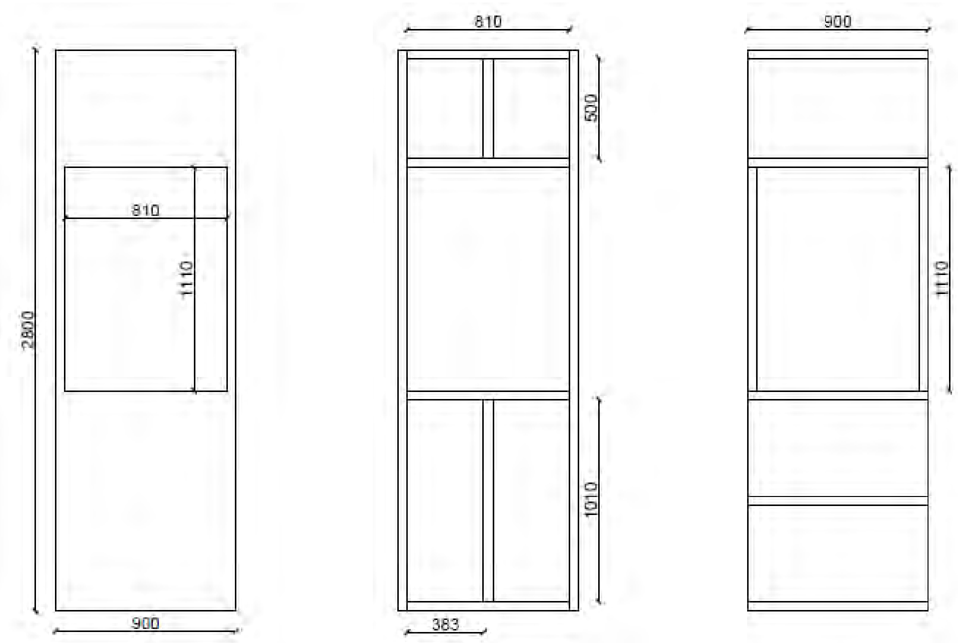


Figure C.2: Facade dimensions [mm] for 0.9m element. Wall area, placement of 45x45 timber laths and placement of 200x45mm timber laths.

Table C.4: Self weight, facade element 0.9m.

Material	Dimension	Self weight
Wood cladding:	A_{wall}	0.06kN
Wood 45x45mm:	2 x 6.72m	0.10kN
Wood 45x200mm:	10.35m	0.35kN
CEMBRIT windstopper:	A_{wall}	0.22kN
ISOVER facade insulation 45 mm:	2 x 1.32m^2	0.04kN
ISOVER facade insulation 200 mm:	1.16m^2	0.07kN
OSB flake board[22]:	A_{wall}	0.14kN
Gyproc gypsum board:	2 x A_{wall}	0.21kN
Velfac window 3-layers:	A_{window}	2.65kN
Total:		3.85kN

The loads for the two different elements are compared in Table C.5 and it can be seen that the uniformly distributed loads are the same for each of the two elements.

Table C.5: Self weights of facade elements.

	Element 1.2m	Element 0.9m
G_{facade}	5.13kN	3.85kN
g_{facade}	1.53kN/m^2	1.53kN/m^2

C.2.2 Inner Wall

The new light inner walls will be placed in continuation of the consoles, see Figure C.3. The material densities are defined in Table C.6.

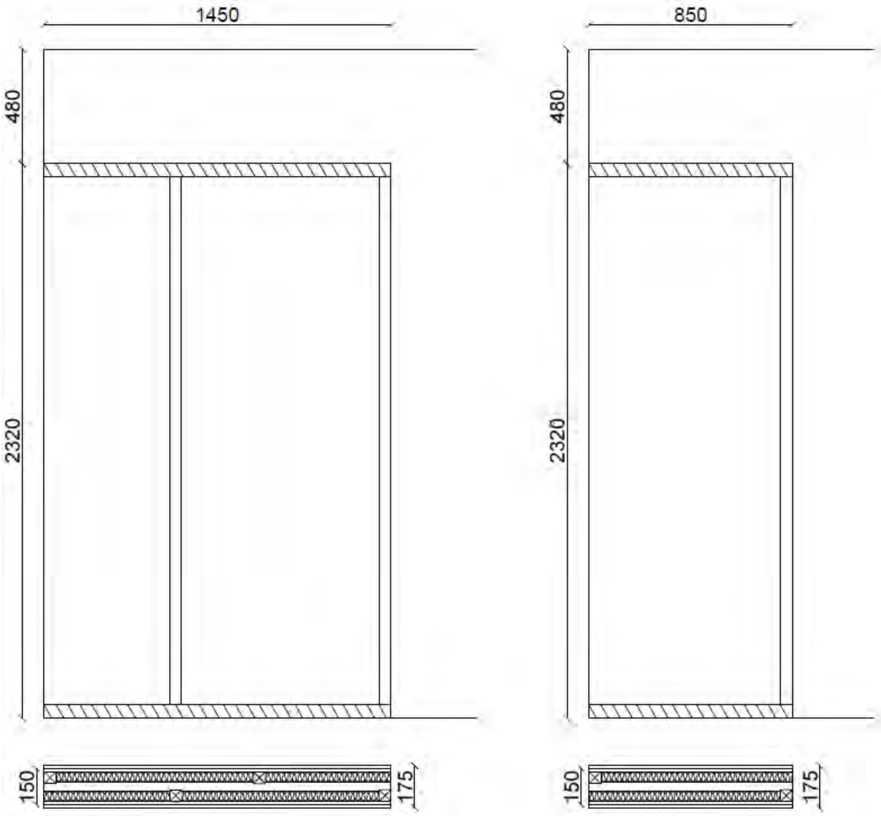


Figure C.3: Inner wall for a long and short console [mm].

Table C.6: Material densities, inner wall.

Material	Dimension	Self weight
Wood 50x50mm[1]:	50x50 mm	9.5 N/m
ISOVER facade insulation 32:	45 mm	14.6 N/m^2
Gyproc gypsum board:	12.5 mm	65.8 N/m^2
U-profile:	55x55 mm	8.5 N/m^2
[1]Timber, strength class C18[22]:		38.0 kN/m^3

The self weight of an inner wall in elongation of a long console is calculated in Table C.7 and it is calculated for a short console in Table C.8. It can be seen that the line loads for each case are very similar.

Table C.7: Self weight inner wall, long console 1.45m.

Material	Dimension	Self weight
Wood 50x50mm:	4 x 2.32m	0.09kN
ISOVER facade insulation 32:	2 x 3.13m^2	0.09kN
Gyproc gypsum board:	2 x (3.36m^2+4.06m^2)	0.98kN
U-profile 55x55mm:	4 x 1.45m	0.05kN
Total:		1.21kN
Line load:		0.83kN/m

Table C.8: Self weight inner wall, short console 0.85m.

Material	Dimension	Self weight
Wood 50x50mm:	2 x 2.32m	0.04kN
ISOVER facade insulation 32:	2 x 1.86m^2	0.05kN
Gyproc gypsum board:	2 x (1.97m^2+2.38m^2)	0.57kN
U-profile 55x55mm:	4 x 0.85m	0.03kN
Total:		0.70kN
Line load:		0.82kN/m

C.3 Wind Load

C.3.1 External Wind Pressure Coefficients

The peak velocity pressure of $q_p = 1.6\text{kN/m}^2$ will be used to determine the wind pressure coefficients. The external wind pressure coefficients will be multiplied with the peak velocity pressure in order to find the actual wind pressure on the building. Since the velocity pressure does not vary over the height of the building, the shape profile of the velocity pressure is uniform over the whole height of the building.

The external pressure coefficient depends on the dimensions of the building and the wind direction. The height of the building including the roof is (approximately) $h = 4 \cdot 2.8\text{m} + 2.5\text{m} = 13.7\text{m}$. The calculations are based on the assumption that the roof is a duo pitch roof, just as the original roof. The depth and the width of the building depends on the wind direction, as shown in Figure C.4.

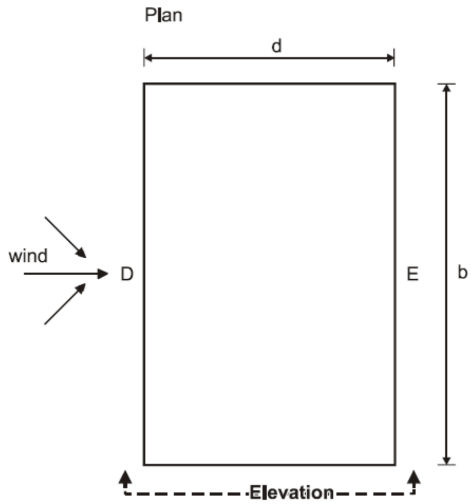


Figure C.4: Zones for wind pressure coefficients, plan view [34].

For wind perpendicular to the longitudinal facade of the building the depth and width of the building is given by $d = 12.1\text{m}$ and $b = 33.3\text{m}$. The value e should be taken as the smallest of b or $2h$. In this case $e = 2h = 27.4\text{m}$. Since $e \geq d$ the pressure zones for the elevation is defined as in Figure C.5.

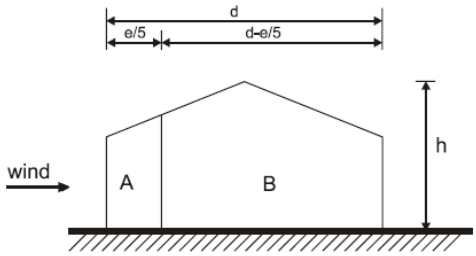


Figure C.5: Wind pressure zones for vertical walls, $e \geq d$ [34], wind perpendicular to facade.

For a wind direction perpendicular to the gable wall the depth and width will be the opposite which is $d = 33.3$m and $b = 12.1$m. In this case $e = b = 12.1\text{m} < d$ and the vertical pressure zones for this situation are illustrated in Figure C.6.

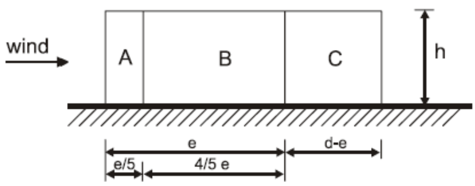


Figure C.6: Wind pressure zones for vertical walls, $e \leq d$ [34], wind perpendicular to gable.

The external pressure coefficients in each zone depends on the ratio h/d. The external pressure coefficients are defined in EN 1991-1-4 and are summed up in Table C.9 where the height to depth ratio has been taken into account. The coefficients $c_{pe,10}$ are used since all areas are above 10m^2.

Table C.9: External pressure coefficient for vertical walls, $c_{pe,10}$.

Zone	A	B	C	D	E
$\perp$ Facade ($h/d = 1.1$)	-1.2	-0.8	-0.5	+0.8	-0.5
$\perp$ Gable ($h/d = 0.4$)	-1.2	-0.8	-0.5	+0.72	-0.34

The external pressure coefficients on the roof will now be defined. The roof is a duopitch roof and the pressure zones are given in Figure C.7 for wind perpendicular to the facade and in Figure C.8 for wind perpendicular to the gable.

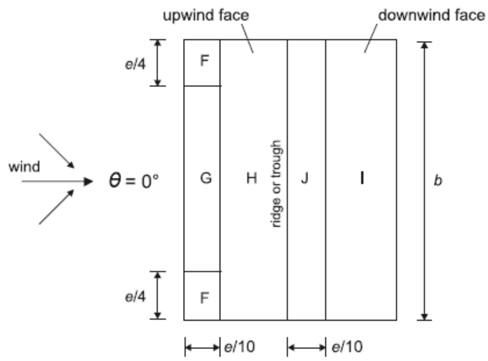


Figure C.7: Wind pressure zones for roof, wind perpendicular to facade [34].

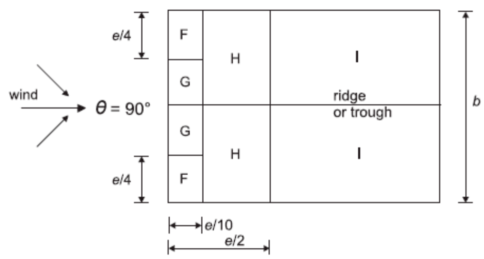


Figure C.8: Wind pressure zones for roof, wind perpendicular to gable [34].

The coefficients depend on the angle of the roof. The pitch angle of the original roof is approximately $\alpha = 22°$. The value is approximate since the exact height of the roof is unknown. For wind perpendicular to the facade four different load cases should be considered since both positive and negative wind pressure can occur. For wind perpendicular to the gable only one wind case should be considered. The external pressure coefficients for the different zones of the roof are given by EN 1991-1-4 and they are summed up in Table C.10 where the angle α has been taken into account.

Table C.10: External pressure coefficient for roof, $c_{pe,10}$.

Zone	F	G	H	I	J
$\perp$ Facade ($\theta = 0°$)	-0.71	-0.66	-0.25	-0.4	-0.77
	+0.43	+0.43	+0.29	+0.0	+0.0
$\perp$ Gable ($\theta = 90°$)	-1.21	-1.35	-0.71	-0.5	-

Five different combinations of wind pressure applies. Four cases for wind perpendicular to the facade and one case with wind perpendicular to the gable.

- Wind LC1 $\perp$ facade: Suction - Suction
- Wind LC2 $\perp$ facade: Pressure - Pressure
- Wind LC3 $\perp$ facade: Suction - Pressure
- Wind LC4 $\perp$ facade: Pressure - Suction
- Wind LC5 $\perp$ gable: Suction

C.3.2 Friction Coefficients

A friction force is only present for wind action perpendicular to the gable walls. The friction area is located beyond a distance of min(2b,4h) away from where the wind hits the building. Only the surfaces parallel to the wind direction should be taken into account. For the case of wind perpendicular to the gable the friction area will be located at a distance of $2b$=24.2m to 33.3m away from the gable wall. The friction force is parallel to the wind force and it is applied to both the facade and the roof. The largest friction coefficient of $c_{fr} = 0.04$ is used. The friction force becomes:

$$f_{fr} = c_{fr} q_p = 0.064\text{kN/m}^2 \tag{C.1}$$

The friction force is applied as a uniform load on the roof and as line loads along the facade.

C.4 Seismic Load

Self weight of one floor is based on weights in Table 4.3.

$$\begin{aligned} G =& g_{wall} \cdot (2 \cdot 12.14\text{m} + 10 \cdot 9.6\text{m}) + g_{floor} \cdot 9.6\text{m} \cdot 33.3\text{m} \\ &+ g_{console} \cdot 10 \cdot (1.45\text{m} + 0.85\text{m}) + g_{column} \cdot 4 \cdot 2.8\text{m} + 40\text{kN} \cdot 20 \\ =& 3562\text{kN} \end{aligned} \tag{C.2}$$

Live load on one floor:

$$Q = 1.5\text{kN/m}^2 \cdot 9.6\text{m} \cdot 33.3\text{m} = 480\text{kN} \tag{C.3}$$

Seismic load on one floor:

$$A = 1.5\% \cdot (G + 0.2 \cdot Q) = 55\text{kN} \tag{C.4}$$

C.5 Loads on Consoles

C.5.1 Proposal 1 - Original Balconies

Table C.11: Design loads for long console - original balconies.

Long console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	14.7	25.3	25.6
Shear force V_{Ed} [kN]:	20.2	34.9	35.3
Torsional moment T_{Ed} [kNm]:	0.2	0.4	0.4

Table C.12: Design loads for short console - original balconies.

Short console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	5.0	8.7	8.8
Shear force V_{Ed} [kN]:	11.9	20.4	20.7
Torsional moment T_{Ed} [kNm]:	0.1	0.2	0.2

Table C.13: Design loads for long console - original balconies with extra balcony length.

Long console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	37.3	64.3	65.0
Shear force V_{Ed} [kN]:	30.6	52.7	53.3
Torsional moment T_{Ed} [kNm]:	0.3	0.5	0.5

Table C.14: Design loads for short console - original balconies with extra balcony length.

Short console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	21.5	37.0	37.4
Shear force V_{Ed} [kN]:	22.2	38.2	38.7
Torsional moment T_{Ed} [kNm]:	0.2	0.4	0.4

C.5.2 Proposal 2 - Facade on the Outside of the Consoles

Table C.15: Design loads for long console - new facade.

Long console	LC1 Self weight	LC2 Variable load
Moment M_{Ed} [kNm]:	45.1	45.1
Shear force V_{Ed} [kN]:	46.1	48.7
Torsional moment T_{Ed} [kNm]:	0.4	0.5

Table C.16: Design loads for short console - new facade.

Short console	LC1 Self weight	LC2 Variable load
Moment M_{Ed} [kNm]:	21.2	20.2
Shear force V_{Ed} [kN]:	33.7	34.1
Torsional moment T_{Ed} [kNm]:	0.3	0.3

C.5.3 Proposal 2 - Built-in Balconies

Extra balcony length for long console: 1.15m.
Extra balcony length for short console: 1.05m.

Table C.17: Design loads for long console - built-in balconies.

Long console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	42.3	67.1	66.2
Shear force V_{Ed} [kN]:	34.0	51.2	49.4
Torsional moment T_{Ed} [kNm]:	0.9	0.3	0.5

Table C.18: Design loads for short console - built-in balconies.

Short console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	23.6	37.5	37.3
Shear force V_{Ed} [kN]:	23.6	35.7	34.8
Torsional moment T_{Ed} [kNm]:	0.7	0.2	0.4

C.5.4 Proposal 3 - External Balconies

Length of balcony carried by consoles: 0.45m.
Length of balcony carried by tension rod: 1.55m.

Table C.19: Design loads for long console - external balconies.

Long console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	55.7	64.5	61.0
Shear force V_{Ed} [kN]:	47.8	55.7	50.7
Torsional moment T_{Ed} [kNm]:	0.5	0.6	0.5

Table C.20: Design loads for short console - external balconies.

Short console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Moment M_{Ed} [kNm]:	30.7	35.4	34.3
Shear force V_{Ed} [kN]:	35.4	41.1	38.2
Torsional moment T_{Ed} [kNm]:	0.4	0.4	0.4

Table C.21: Design loads for tension rod - external balconies.

Long console	LC1 Self weight	LC2 Variable load	LC3 Snow load
Vertical load P :	5.9kN	24.1kN	24.6kN
Inclined load, long console $\theta = 58°$:	6.9kN	28.5kN	29.0kN
Inclined load, short console $\theta = 70°$:	6.4kN	25.7kN	26.2kN

APPENDIX D

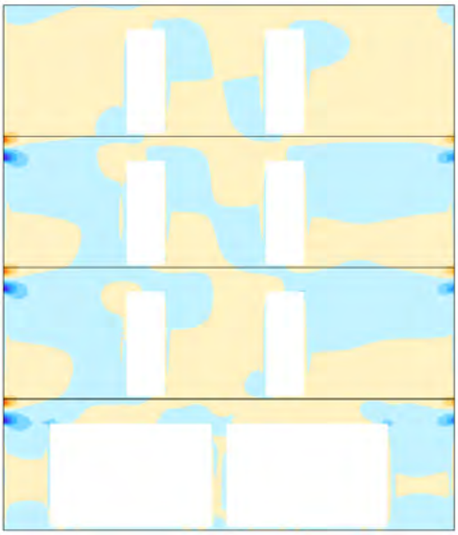
Results from Robot

In this appendix all stress maps from Robot will be shown. Selected stress maps have also been presented in the report.

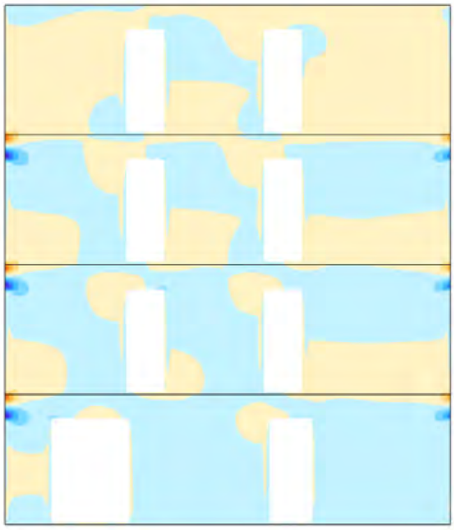
In the stress maps for wall 2 and wall 11 larger stresses occur in certain areas around level 0 and level 1. These walls are connected to the former staircases and therefore the walls are not supported by a deck on one side. However, loads are still applied in this area, where the decks is missing, and it is unknown why *Robot* distributes the loads in this manner. The stresses are not very large and therefore they are not critical.

D.1 Original Openings in Walls

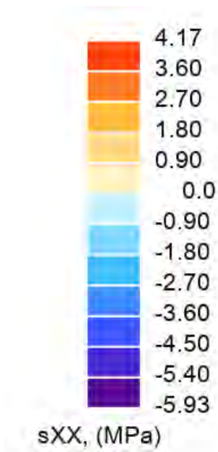
D.1.1 Walls with Opening A and B



(a) Wall 4 (A0,A1,A2,A2)

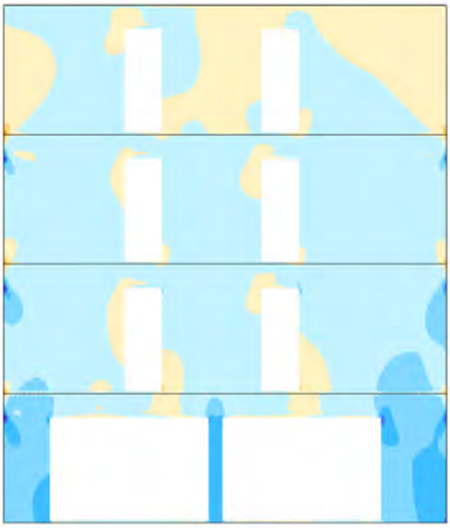


(b) Wall 7 (A1,A1,A2,A2)

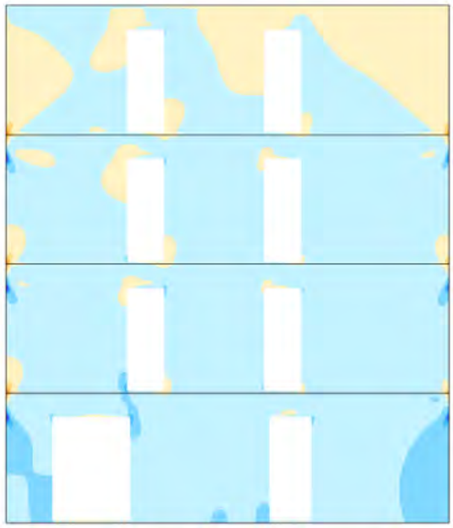


(c) Legend, σ_{xx}

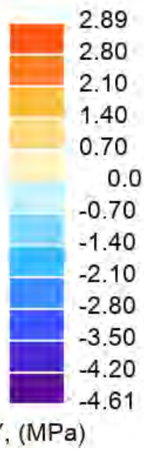
Figure D.1: Walls with opening A and B. Horizontal stresses σ_{xx} [MPa].



(a) Wall 4 (A0,A1,A2,A2)

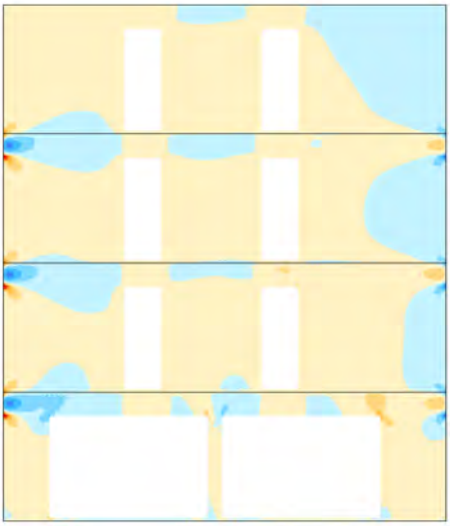


(b) Wall 7 (A1,A1,A2,A2)

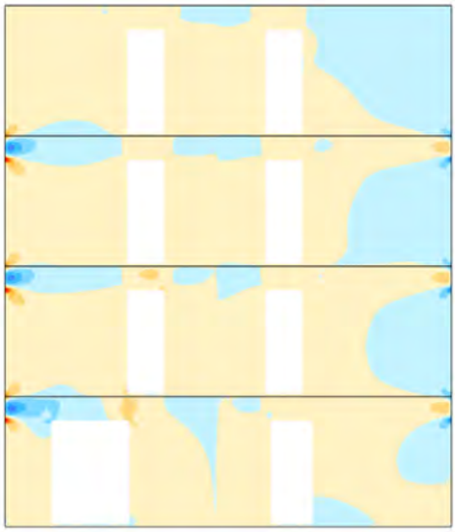


(c) Legend, σ_{yy}

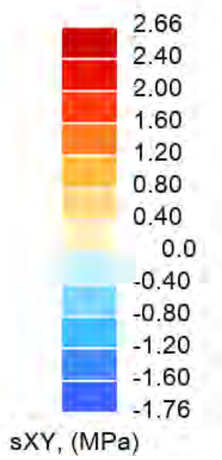
Figure D.2: Walls with opening A and B. Vertical stresses σ_{yy} [MPa].



(a) Wall 4 (A0,A1,A2,A2)



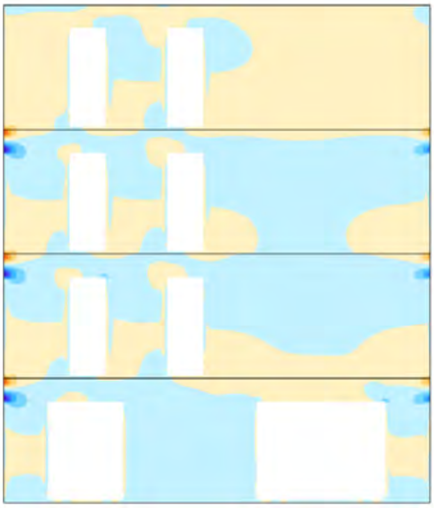
(b) Wall 7 (A1,A1,A2,A2)



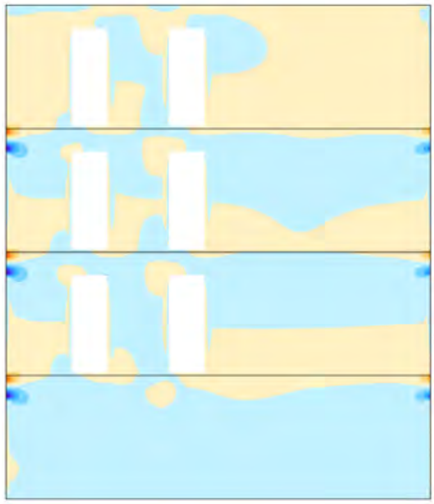
(c) Legend, σ_{xy}

Figure D.3: Walls with opening A and B. Shear stresses σ_{xy} [MPa].

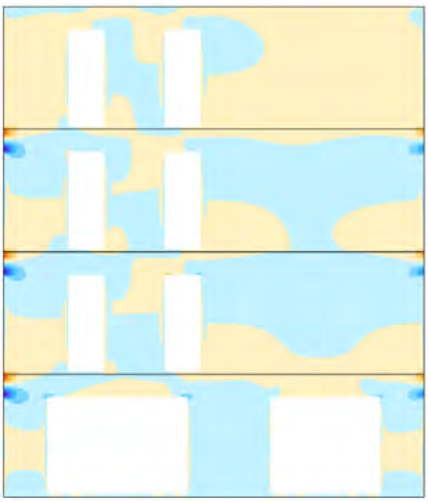
D.1.2 Walls with Opening C and D



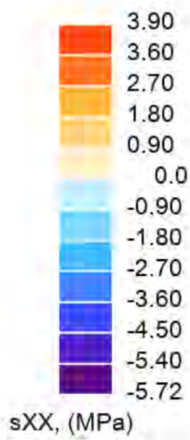
(a) Wall 5 (A0,A1,A2,A2)



(b) Wall 8 (A0,A1,A1,A1)

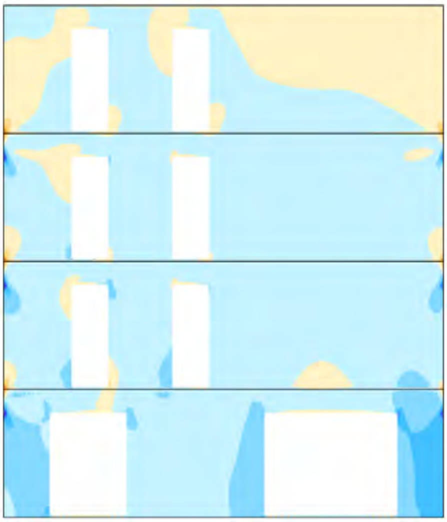


(c) Wall 10 (A0,A1,A2,A2)

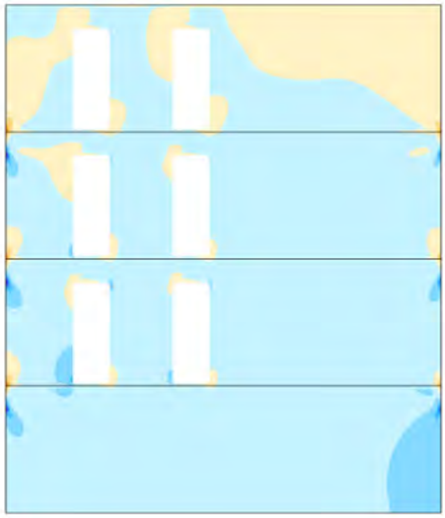


(d) Legend, σ_{xx}

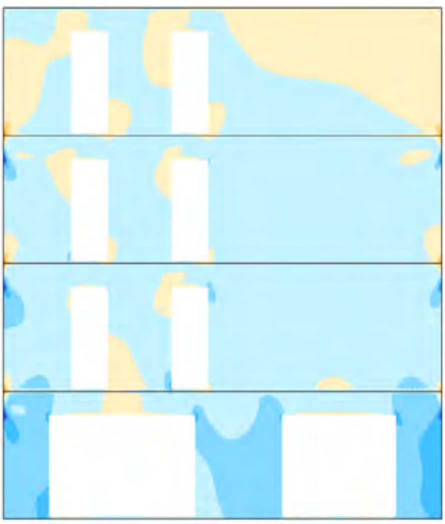
Figure D.4: Walls with opening C and D. Horizontal stresses σ_{xx} [MPa].



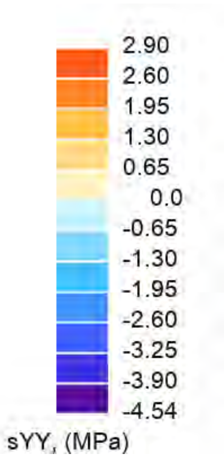
(a) Wall 5 (A0,A1,A2,A2)



(b) Wall 8 (A0,A1,A1,A1)

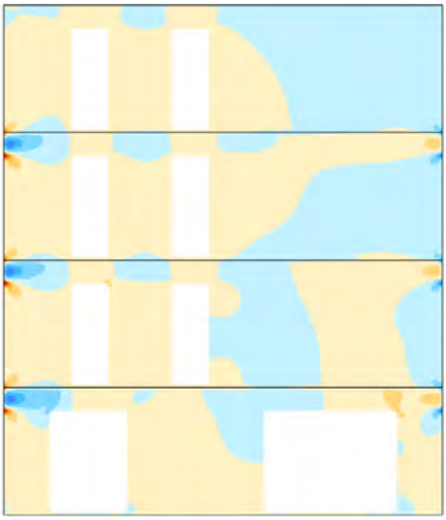


(c) Wall 10 (A0,A1,A2,A2)

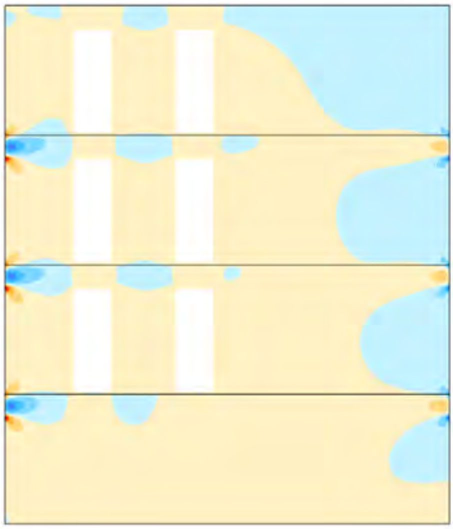


(d) Legend, σ_{yy}

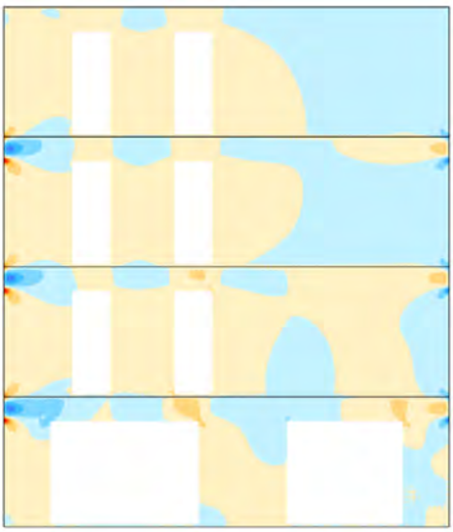
Figure D.5: Walls with opening C and D. Vertical stresses σ_{yy} [MPa].



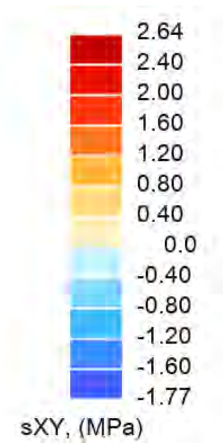
(a) Wall 5 (A0,A1,A2,A2)



(b) Wall 8 (A0,A1,A1,A1)



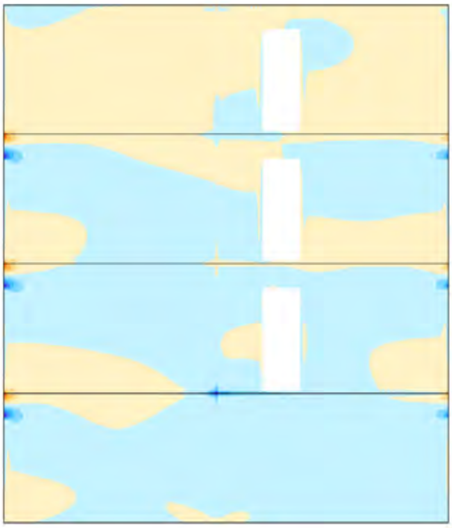
(c) Wall 10 (A0,A1,A2,A2)



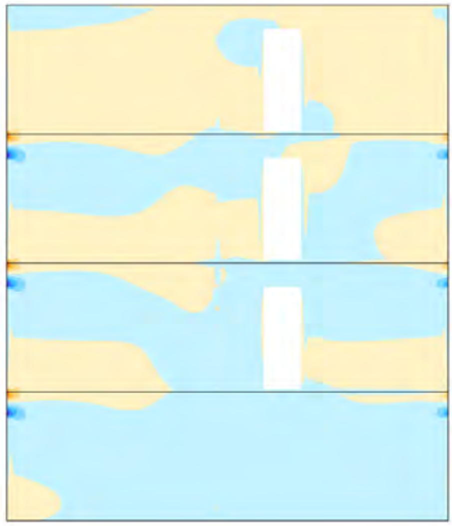
(d) Legend, σ_{xy}

Figure D.6: Walls with opening C and D. Shear stresses σ_{xy} [MPa].

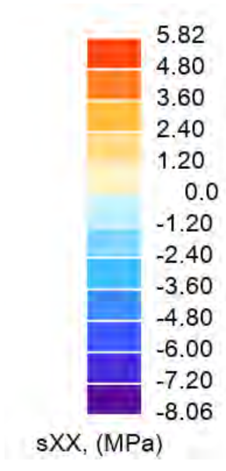
D.1.3 Walls by a Staircase with Opening A



(a) Wall 2 (A1,A1,A1,A2)

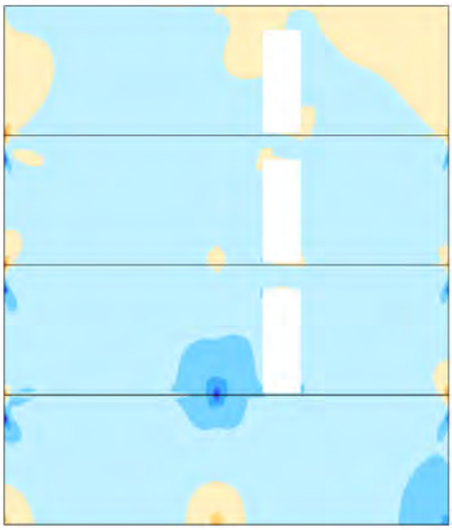


(b) Wall 11 (A1,A1,A1,A2)

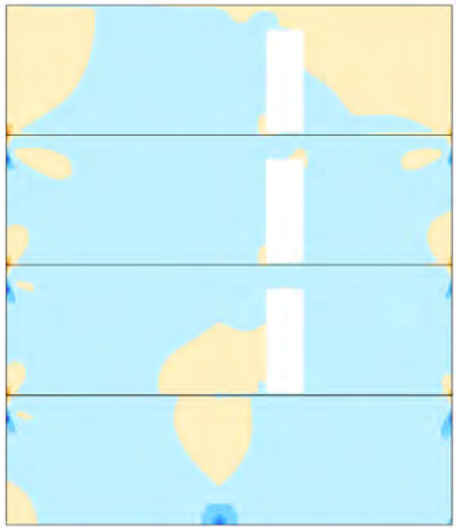


(c) Legend, σ_{xx}

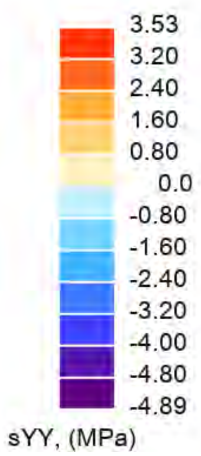
Figure D.7: Walls with opening A. Horizontal stresses σ_{xx} [MPa].



(a) Wall 2 (A1,A1,A1,A2)

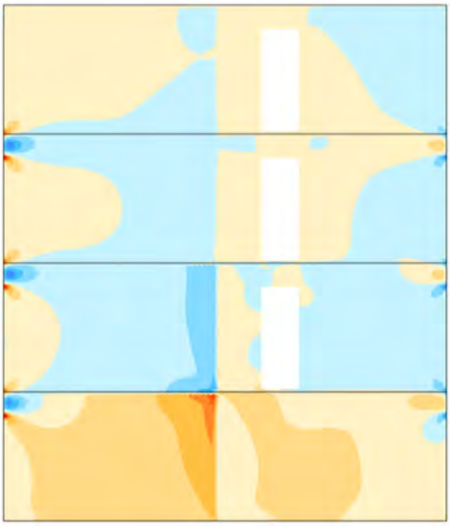


(b) Wall 11 (A1,A1,A1,A2)

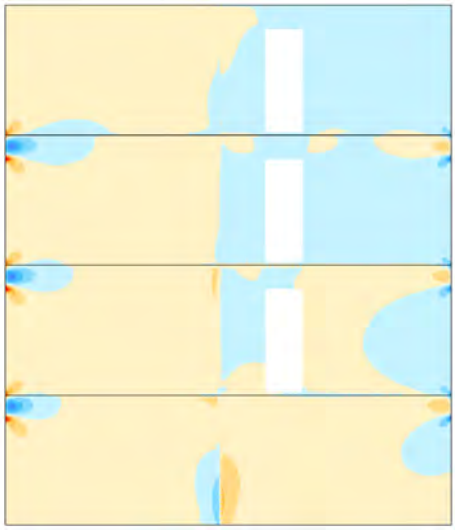


(c) Legend, σ_{yy}

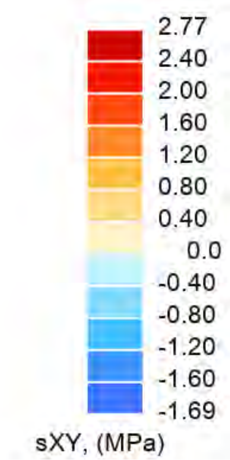
Figure D.8: Walls with opening A. Vertical stresses σ_{yy} [MPa].



(a) Wall 2 (A1,A1,A1,A2)



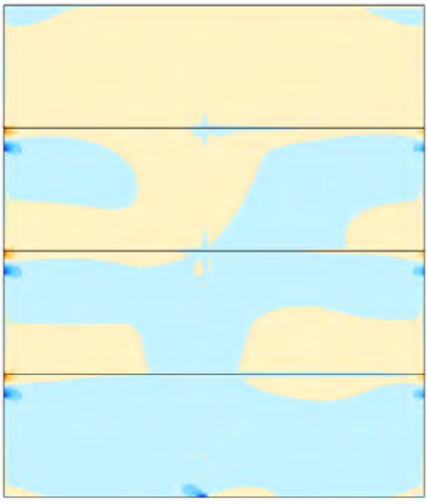
(b) Wall 11 (A1,A1,A1,A2)



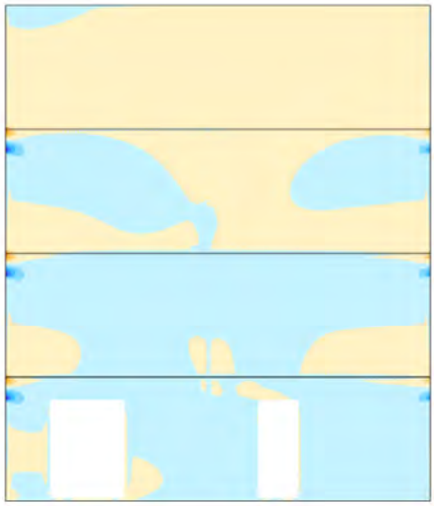
(c) Legend, σ_{xy}

Figure D.9: Walls with opening A. Shear stresses σ_{xy} [MPa].

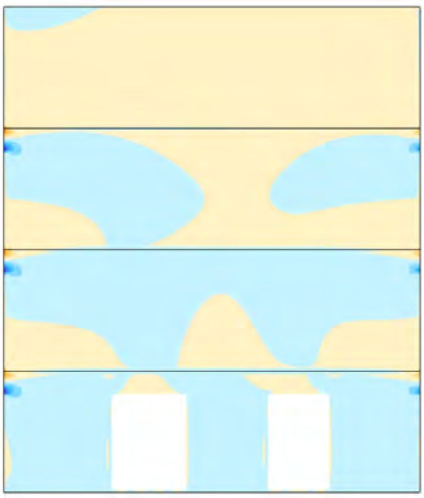
D.1.4 Walls without Openings



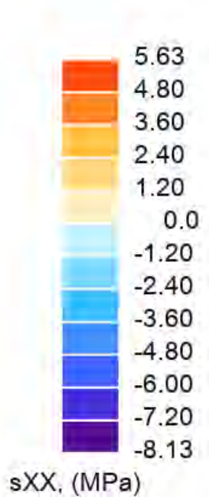
(a) Wall 3 (A1,A1,A1,A2)



(b) Wall 6 (A1,A1,A2,A2)

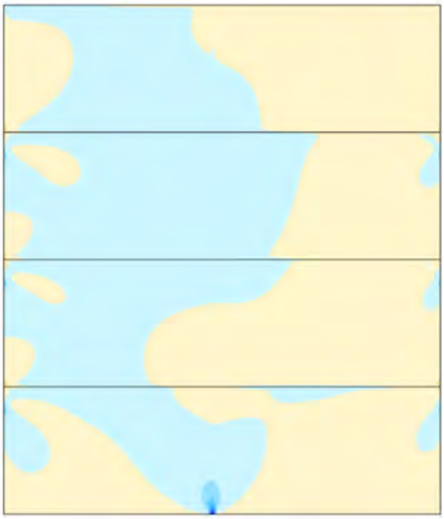


(c) Wall 9 (A0,A1,A1,A2)

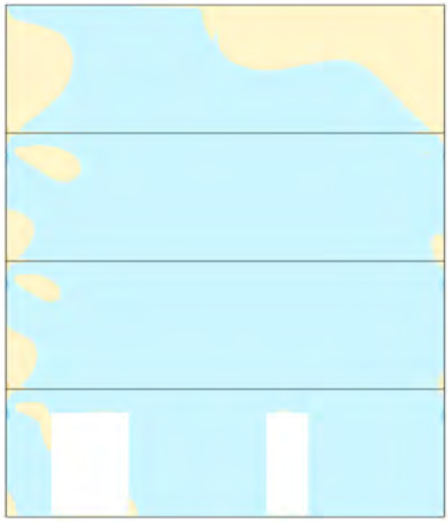


(d) Legend, σ_{xx}

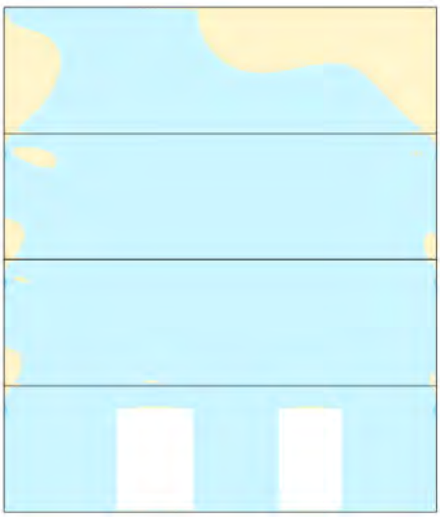
Figure D.10: Walls without openings. Horizontal stresses σ_{xx} [MPa].



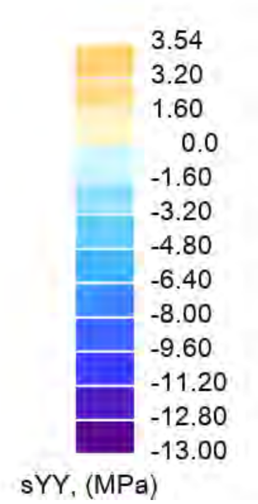
(a) Wall 3 (A1,A1,A1,A2)



(b) Wall 6 (A1,A1,A2,A2)

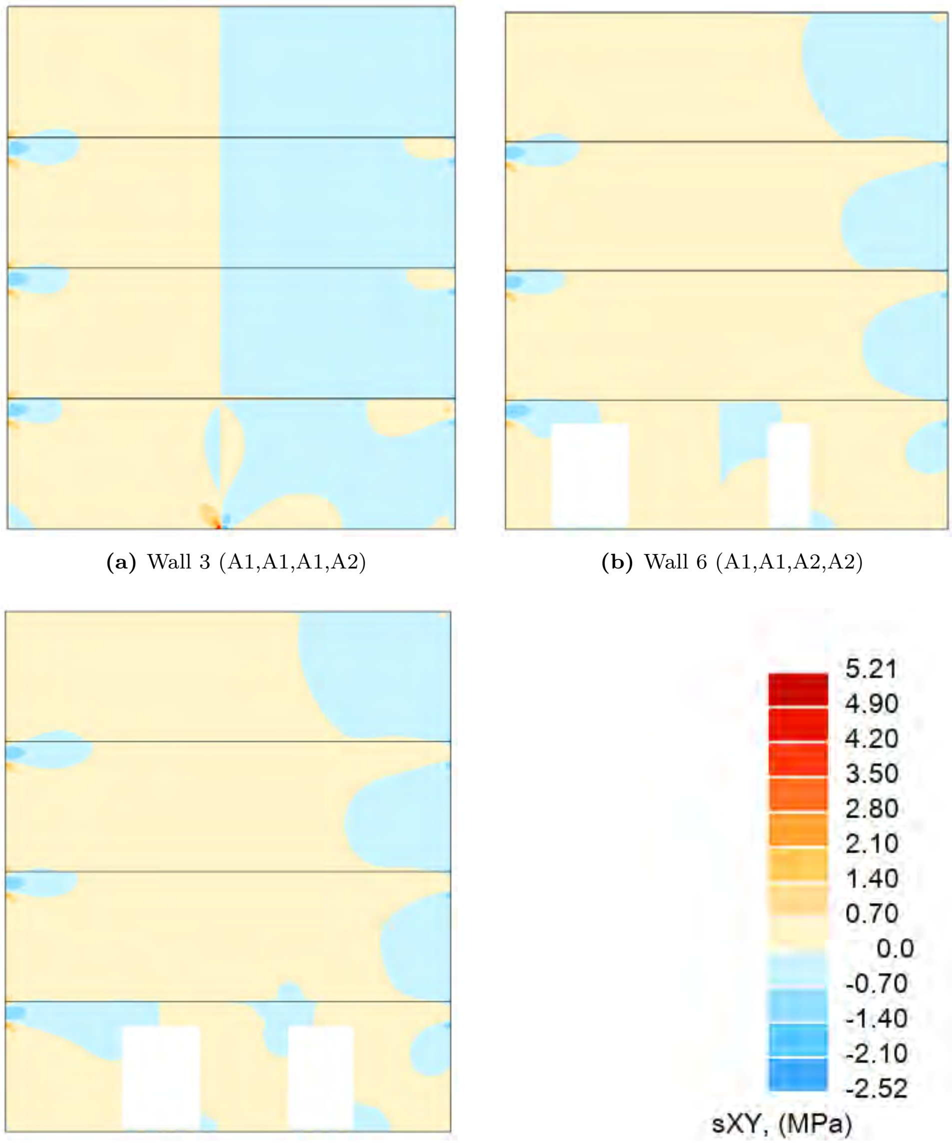


(c) Wall 9 (A0,A1,A1,A2)



(d) Legend, σ_{yy}

Figure D.11: Walls without openings. Vertical stresses σ_{yy} [MPa].



(a) Wall 3 (A1,A1,A1,A2)

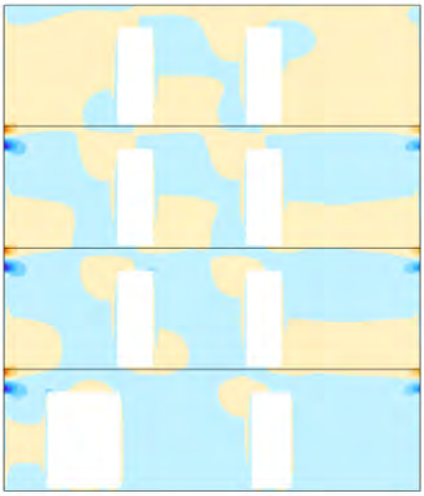
(b) Wall 6 (A1,A1,A2,A2)

(c) Wall 9 (A0,A1,A1,A2)

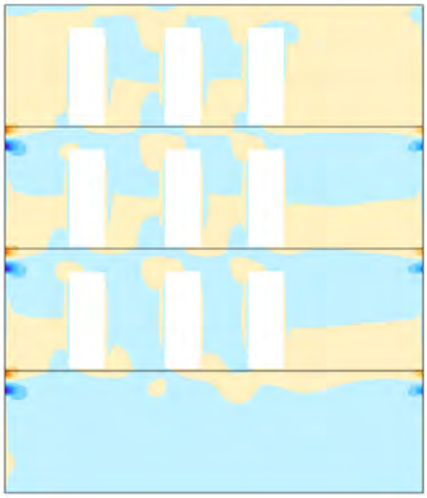
(d) Legend, σ_{xy}

Figure D.12: Walls without openings. Shear stresses σ_{xy} [MPa].

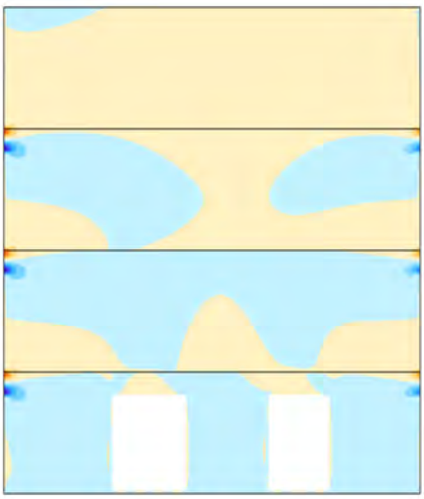
D.2 New Opening A in Wall 8



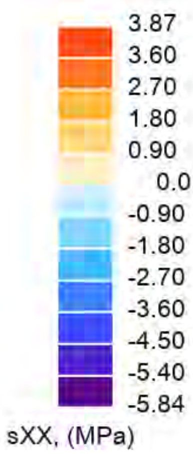
(a) Wall 7, opening A and B (A1,A1,A2,A2)



(b) Wall 8, opening C and D, new opening A (A0,A1,A1,A1)

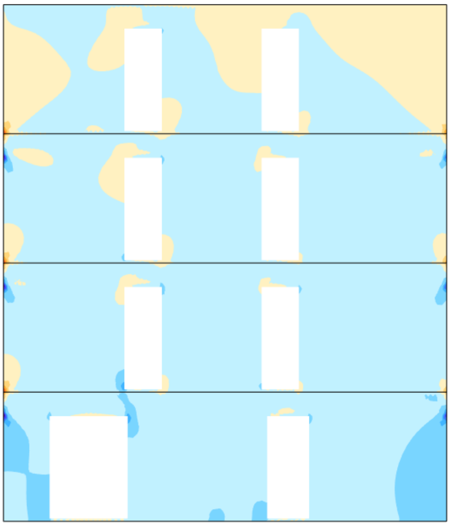


(c) Wall 9, no openings (A0,A1,A1,A2)

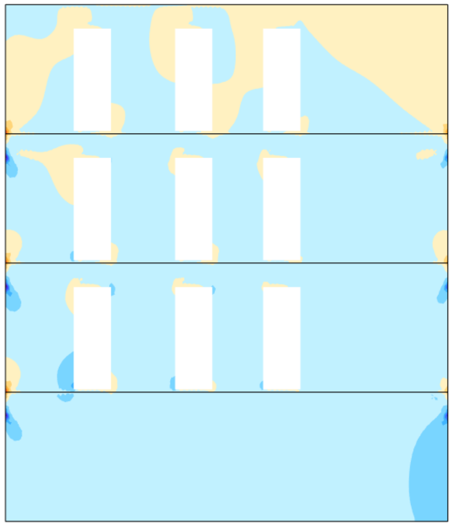


(d) Legend, σ_{xx}

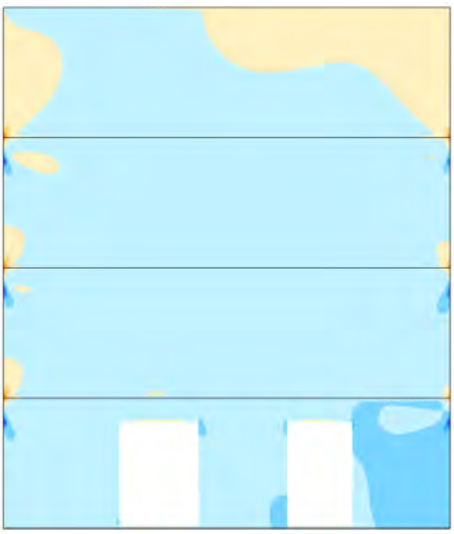
Figure D.13: New opening A in wall 8. Horizontal stresses σ_{xx} [MPa].



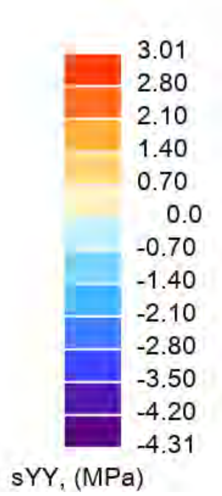
(a) Wall 7, opening A and B (A1,A1,A2,A2)



(b) Wall 8, opening C and D, new opening A (A0,A1,A1,A1)

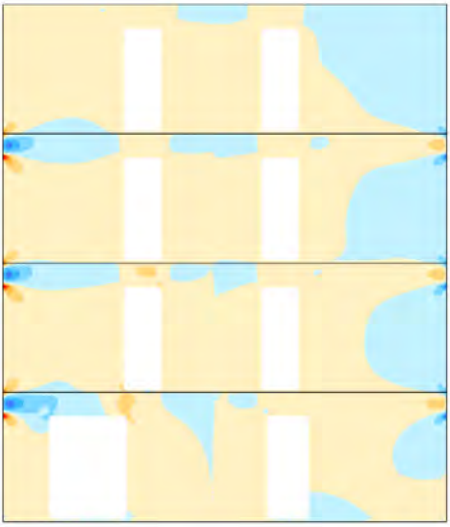


(c) Wall 9, no openings (A0,A1,A1,A2)

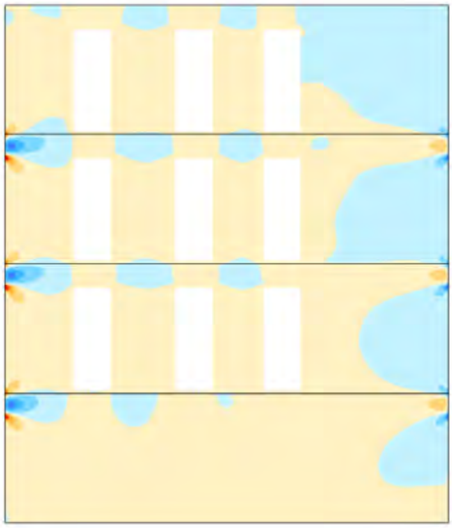


(d) Legend, σ_{yy}

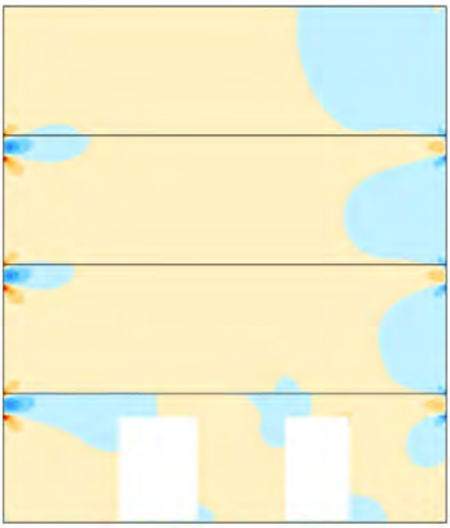
Figure D.14: New opening A in wall 8. Vertical stresses σ_{yy} [MPa].



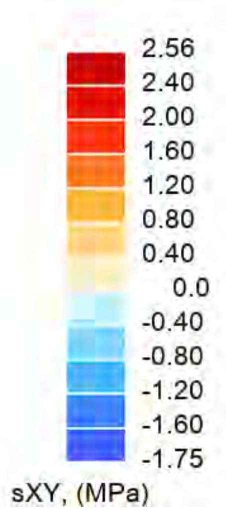
(a) Wall 7, opening A and B (A1,A1,A2,A2)



(b) Wall 8, opening C and D, new opening A (A0,A1,A1,A1)



(c) Wall 9, no openings (A0,A1,A1,A2)

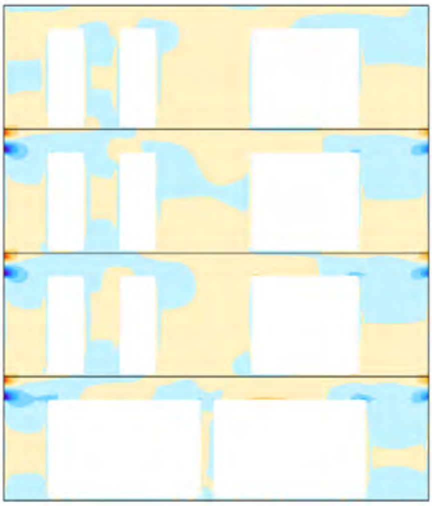


(d) Legend, σ_{xy}

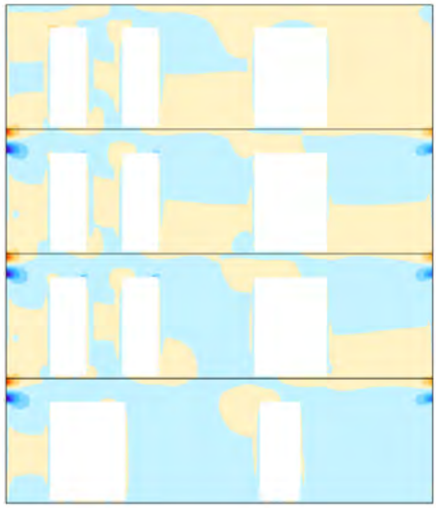
Figure D.15: New opening A in wall 8. Shear stresses σ_{xy} [MPa].

D.3 New Openings in All Walls

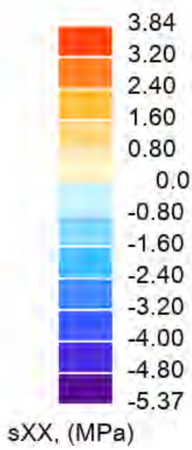
D.3.1 Walls with Original Opening A and B



(a) Wall 4 (A0,A1,A2,A2)

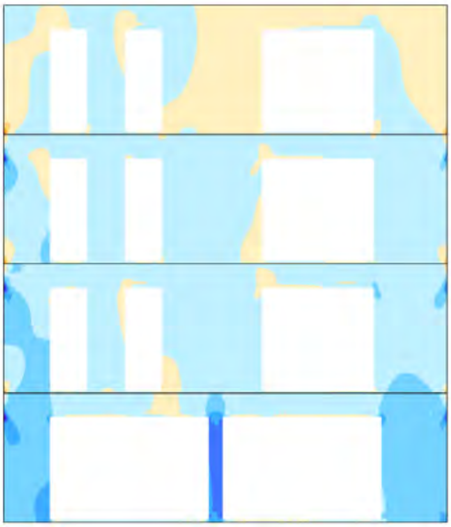


(b) Wall 7 (A1,A1,A2,A2)

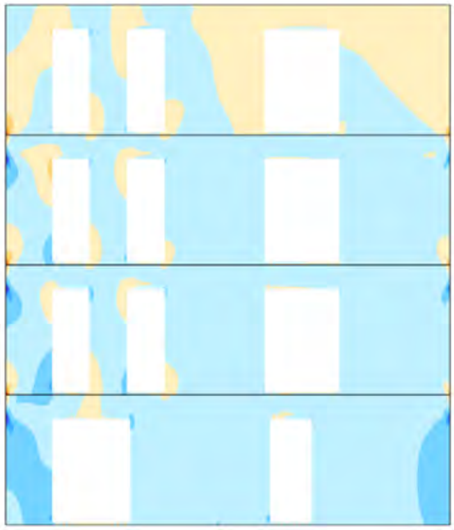


(c) Legend, σ_{xx}

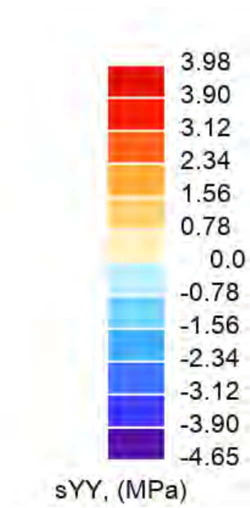
Figure D.16: Walls with original opening A and B, new openings. Horizontal stresses σ_{xx} [MPa].



(a) Wall 4 (A0,A1,A2,A2)

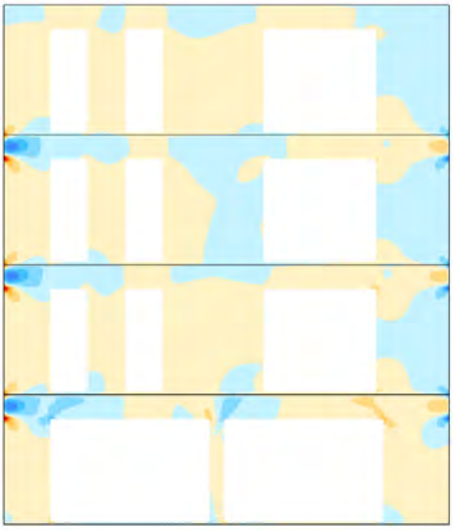


(b) Wall 7 (A1,A1,A2,A2)

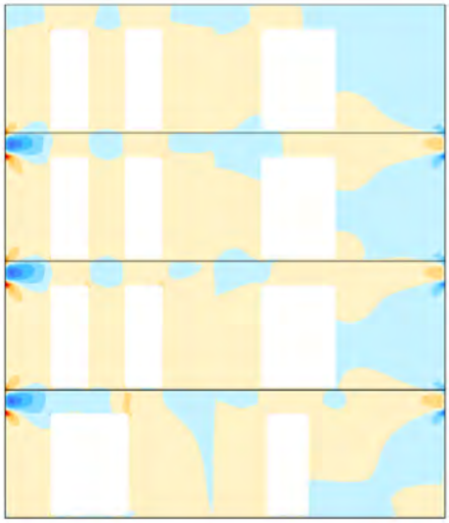


(c) Legend, σ_{yy}

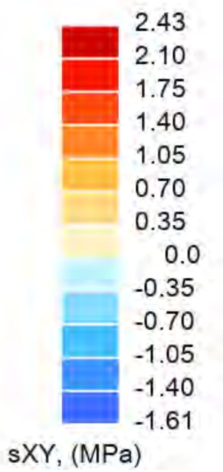
Figure D.17: Walls with original opening A and B, new openings. Vertical stresses σ_{yy} [MPa].



(a) Wall 4 (A0,A1,A2,A2)



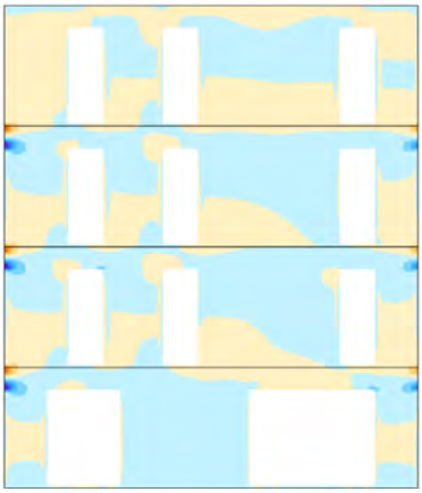
(b) Wall 7 (A1,A1,A2,A2)



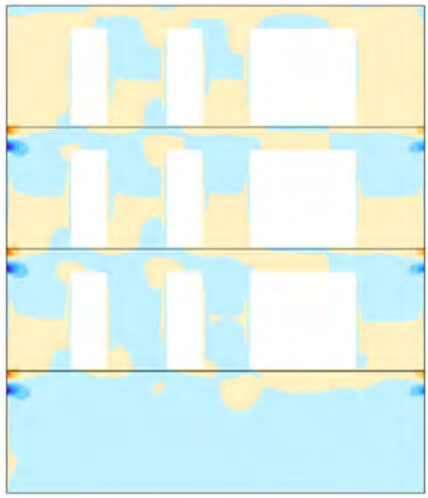
(c) Legend, σ_{xy}

Figure D.18: Walls with original opening A and B, new openings. Shear stresses σ_{xy} [MPa].

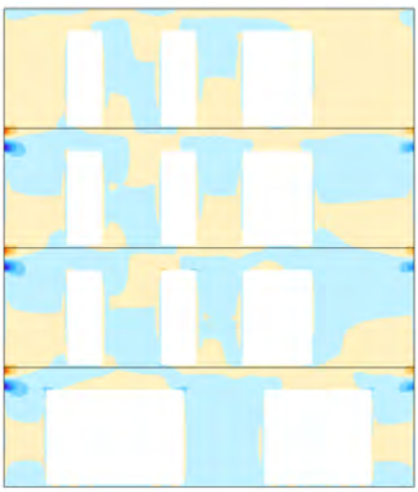
D.3.2 Walls with Original Opening C and D



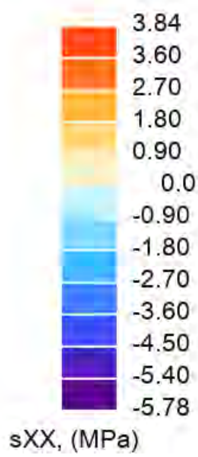
(a) Wall 5 (A0,A1,A2,A2)



(b) Wall 8 (A0,A1,A1,A1)

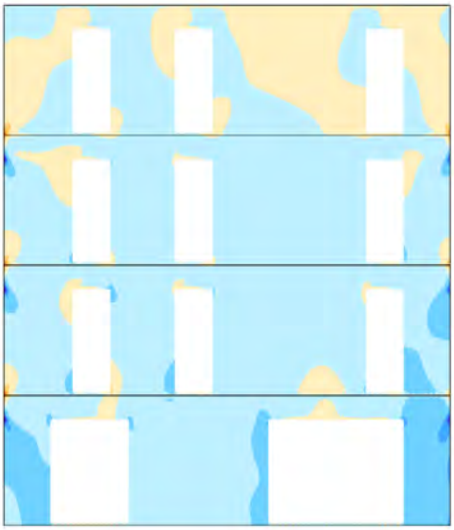


(c) Wall 10 (A0,A1,A2,A2)

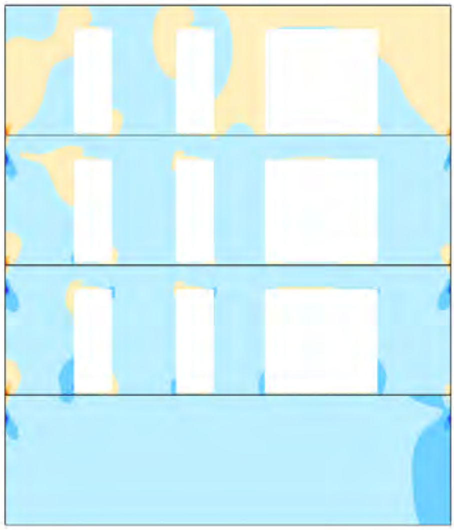


(d) Legend, σ_{xx}

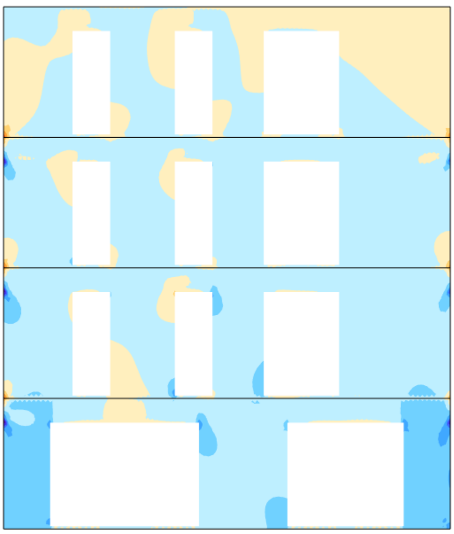
Figure D.19: Walls with original opening C and D, new openings. Horizontal stresses σ_{xx} [MPa].



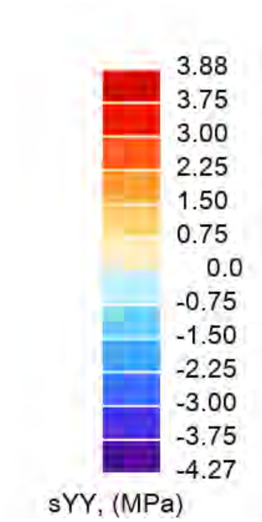
(a) Wall 5 (A0,A1,A2,A2)



(b) Wall 8 (A0,A1,A1,A1)

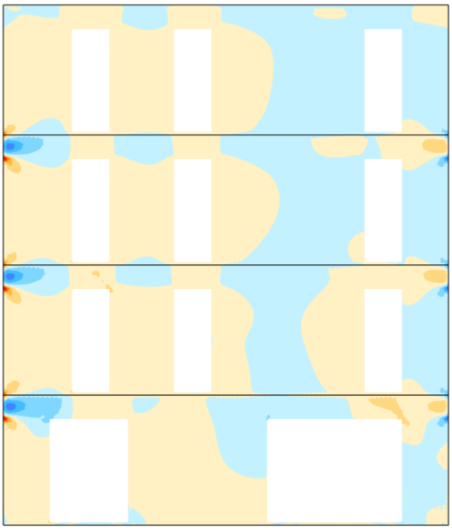


(c) Wall 10 (A0,A1,A2,A2)

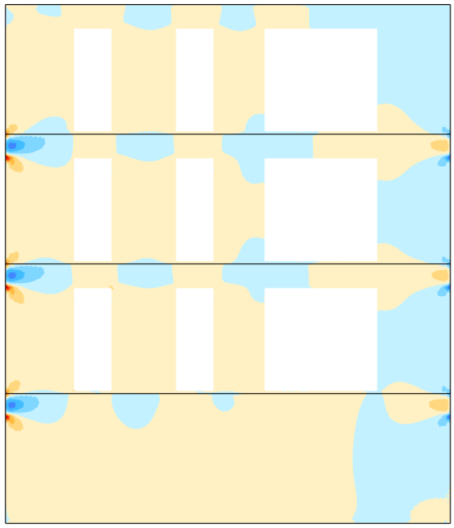


(d) Legend, σ_{yy}

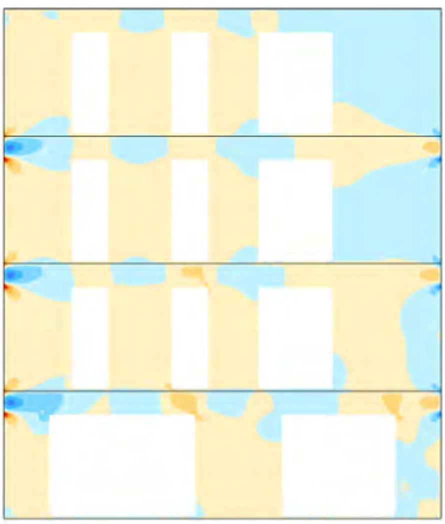
Figure D.20: Walls with original opening C and D, new openings. Vertical stresses σ_{yy} [MPa].



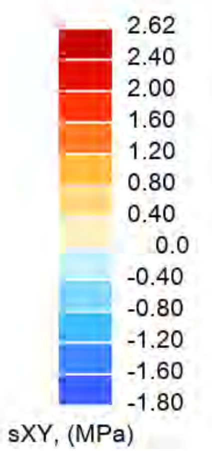
(a) Wall 5 (A0,A1,A2,A2)



(b) Wall 8 (A0,A1,A1,A1)



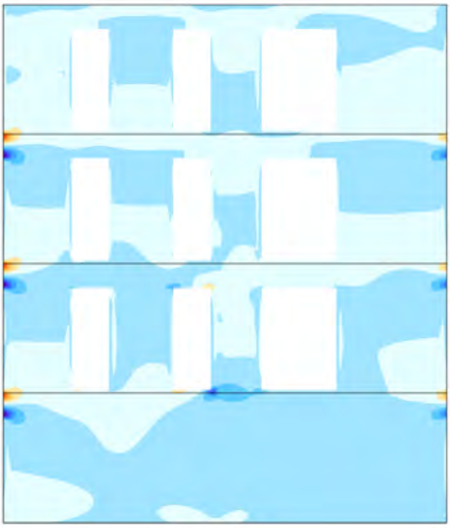
(c) Wall 10 (A0,A1,A2,A2)



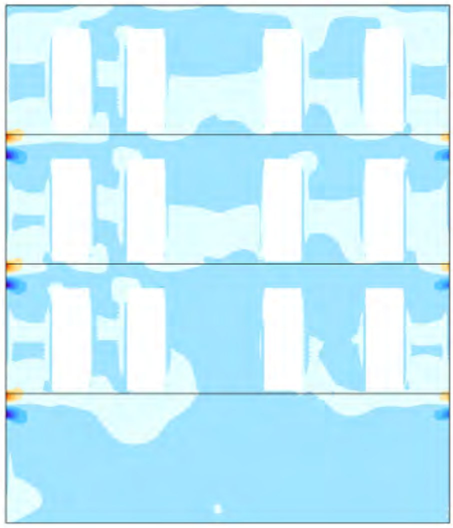
(d) Legend, σ_{xy}

Figure D.21: Walls with original opening C and D, new openings. Shear stresses σ_{xy} [MPa].

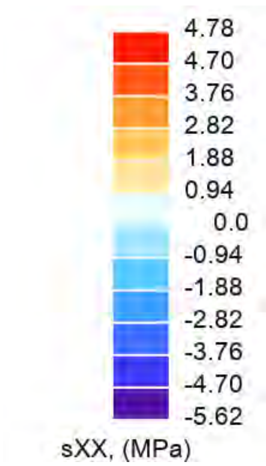
D.3.3 Walls by a Staircase with Original Opening A



(a) Wall 2 (A1,A1,A1,A2)

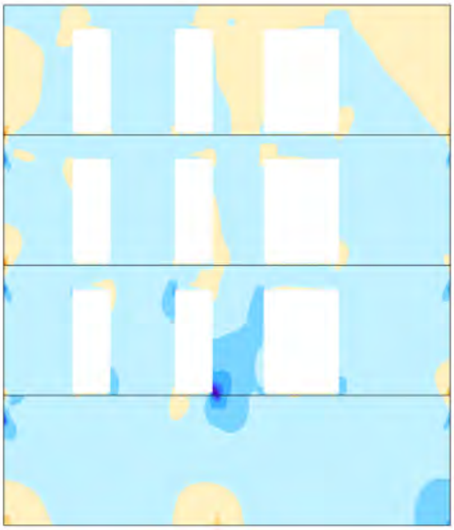


(b) Wall 11 (A1,A1,A1,A2)

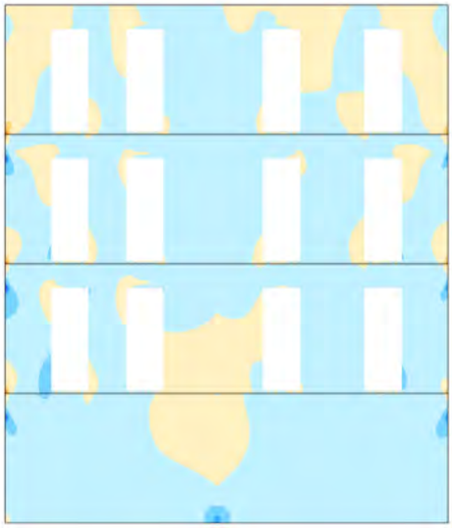


(c) Legend, σ_{xx}

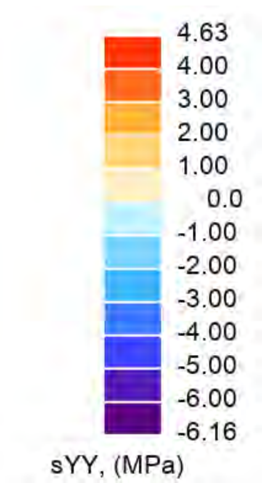
Figure D.22: Walls with original opening A, new openings. Horizontal stresses σ_{xx} [MPa].



(a) Wall 2 (A1,A1,A1,A2)

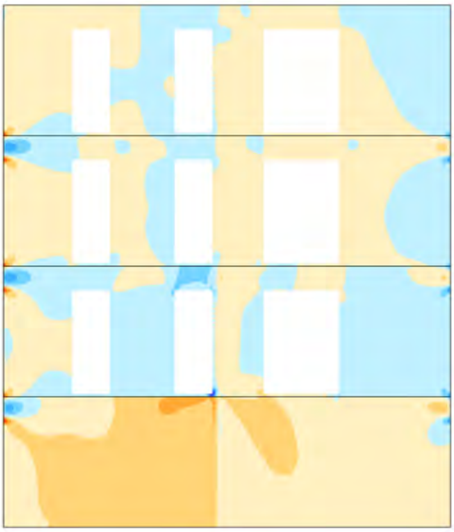


(b) Wall 11 (A1,A1,A1,A2)

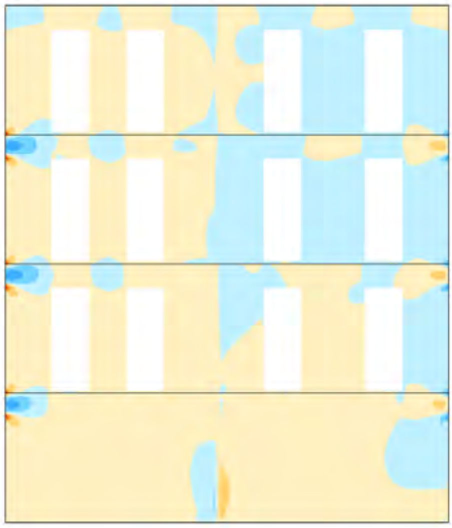


(c) Legend, σ_{yy}

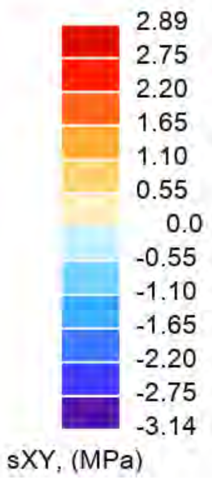
Figure D.23: Walls with original opening A, new openings. Vertical stresses σ_{yy} [MPa].



(a) Wall 2 (A1,A1,A1,A2)



(b) Wall 11 (A1,A1,A1,A2)



(c) Legend, σ_{xy}

Figure D.24: Walls with original opening A, new openings. Shear stresses σ_{xy} [MPa].

APPENDIX E

Original Drawings from GTO

<u>Group (9): Layout drawings</u>

1.10(9)	Building no. 10, plan of ground floor, layout drawing
2.10(9)	Building no. 10, plan of 1st, 2nd and 3rd floor, layout drawing
3.10(9)	Building no. 10, north facade
4.10(9)	Building no. 10, south facade
5.10-5.11(9)	Building no. 10 and 11, east and west gable
1.12(9)	Building no. 12, plan of ground floor, layout drawing
2.12(9)	Building no. 12, plan of 1st floor, layout drawing
3.12(9)	Building no. 12, plan of 2nd and 3rd floor, layout drawing
4.12(9)	Building no. 12, east facade
5.12(9)	Building no. 12, west facade
6.12(9)	Building no. 12, north and south gable

<u>Group (21): Detail drawings, gable</u>

201-204(21)	Reinforcement details by gable walls

<u>Group (22): Building component drawings, walls</u>

201.26-201.30(22)	Building no. 12, walls in line 2-6
201.31-201.35(22)	Building no. 12, walls in line 7-11
201.40-201.44(22)	Building no. 12, longitudinal walls in line C
201-203(22)	Reinforcement details in transverse walls
204-206(22)	Reinforcement details in transverse walls

Group (23): Building component drawings, decks

201.6(23)	Building no. 12, deck above 1st and 2nd floor
201.9(23)	Building no. 12, deck above 3rd floor

Group (26): Building component drawings, balconies

1.1(26)	Balcony plate, wide, large module
1.2(26)	Balcony plate, wide, small module
1.3(26)	Balcony plate, narrow, large module
1.4(26)	Balcony plate, narrow, small module
1.18(26)	Balcony console for wide balcony
1.19(26)	Balcony console for narrow balcony
201.20(26)	Balcony plate, wide, small module, by gable
201.22(26)	Balcony plate, narrow, small module, by gable
201.24(26)	Balcony plate, wide, large module, by gable
201.25(26)	Balcony plate, narrow, large module, by gable