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Buildability and Assembly of the WoodSol Concept

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Buildability and Assembly of the WoodSol Concept

Byggbarhet og montering av WoodSol-konseptet

BY:

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SUMMARY:

This master thesis is part of the research program *WoodSol*, and aims to check the buildability and assembly aspects of WoodSol, as well as the economic, transportational and environmental aspect. In this process, assumptions are made based on the previous work of the WoodSol project.

The buildability for an introduced reference building is investigated. The main areas considered in the buildability aspect are the size and rotational stiffness of the foundations and the variation of the different components impact on the final rotational stiffness. The obtainable rotational stiffness of the foundation is in the range of 3315-11460 [kNm/Rad]. This is found numerically, and checked for the serviceability limit state for the reference building. The size of the foundations are found considering the forces acting on the columns. These forces are found both numerically and analytically. The necessary foundation size varies with regards to the soil stiffness, but for loose gravel and eight stories the proposed solution is a strip foundation with a width of 3300 millimeters and height of 500 millimeters. The volume of the concrete foundation for the reference building is found to be 30-42% lower than the foundation for an equivalent concrete building, when built on fine sand or loose gravel.

For the assembly aspect, the cost of different cranes is compared depending on the time rented. For the WoodSol project it is concluded that a mobile crane would be the most economic because of the rapid erection time. Since the columns do not have capacity to stand by them self after mounting, one deck need to connect four columns as soon as possible after the sufficient number of columns have been erected. The erecting time of the bearing structure is only five days, after the foundations are finished.

The saving of kg CO_2-eq polluted, for the reference building built in timber compared to concrete, are 286.082 kg, 433.087 kg, 576.008 kg, for four, six and eight stories respectively. 576.008 kg is equivalent with driving one million new Volvo cars 4,5 kilometers. Costs of the foundations are 30-40% less depending on the stiffness of the ground when building in timber compared to concrete. This is based on the 44% reduction of concrete needed.

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Preface

The present master thesis have been submitted to the Norwegian University of Science and Technology (NTNU) as an end of a 2 year master's degree. The thesis is written for the Department of Structural Engineering over a period of 20 weeks, from January to June 2018 and is weighted with 30 credits per student. The thesis is part of the research project "*WoodSol* – Wood frame solutions for free space design in urban building", led by Professor Kjell Arne Malo from the Department of Structural Engineering.

As the authors have chosen two different paths after the last semester at the university, one is to start at a construction company and one at a consulting company, finding a thesis that could fit both was of interest. After a meeting with Kjell Arne Malo and Haris Stamatopoulos the authors found the present thesis to be very interesting, combining the competence from both theory and practical work. This resulted in a wide thesis containing everything from transport to buildability and assembly which has been very educational.

We would like to thank our supervisors, Professor Kjell Arne Malo and Postdoctoral Fellow Haris Stamatopoulos, for granting us such an interesting and inspiring thesis. A great deal of gratitude is given to the supervisors, as their guidance, inspiration and motivation has made this work possible. Not to mention the great atmosphere in the meetings. We would also like to say thank you to the workers at the different companies who spent their precious time answering our e-mails.

Trondheim, June 5th 2018.

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Abstract

This master thesis is part of the research program *WoodSol,* and aims to check the buildability and assembly aspects of WoodSol, as well as the economic, transportational and environmental aspect. In this process, assumptions are made based on the previous work of the WoodSol project.

The buildability for an introduced reference building is investigated. The main areas considered in the buildability aspect are the size and rotational stiffness of the foundations and the variation of the different components impact on the final rotational stiffness. The obtainable rotational stiffness of the foundation is in the range of 3315-11460 [kNm/Rad]. This is found numerically, and checked for the serviceability limit state for the reference building. The size of the foundations are found considering the forces acting on the columns. These forces are found both numerically and analytically. The necessary foundation size varies with regards to the soil stiffness, but for loose gravel and eight stories the proposed solution is a strip foundation with a width of 3300 millimeters and height of 500 millimeters. The volume of the concrete foundation for the reference building is found to be 30-42% lower than the foundation for an equivalent concrete building, when built on fine sand or loose gravel.

Considering the transportational aspect of the project, the deck elements used will be transported by semi-trucks, transporting six elements simultaneously. The columns will be transported by an extendable semi-truck, transporting 14 columns each delivery. This transport will need a police escort and is costly. Other elements and materials have a standardized transportation.

For the assembly aspect, the cost of different cranes is compared depending on the time rented. For the WoodSol project it is concluded that a mobile crane would be the most economic because of the rapid erection time. The lifting process for the different elements is discussed and it is figured out that the columns will be lifted by the pre-installed connectors meant for the deck elements. While the decks will need to have installed eyebolts to make it possible to lift with an angle for easy mounting. Since the columns do not have capacity to stand by them self after mounting, the top deck need to connect four columns as soon as possible after the sufficient number of columns have been erected. The erecting time of the bearing structure is only five days, after the foundations are finished.

The saving of kg CO_2-eq polluted, for the reference building built in timber compared to concrete, are 286.082 kg, 433.087 kg, 576.008 kg, for four, six and eight stories respectively. 576.008 kg is equivalent with driving one million new Volvo cars 4,5 kilometers. Costs of the foundations are 30-40% less depending on the stiffness of the ground when building in timber compared to concrete. This is based on the 44% reduction of concrete needed.

Sammendrag

Denne masteroppgaven er en del av forskningsprosjektet *WoodSol*, og har som mål å sjekke byggbarheten og monteringsprosessen til WoodSol, så vel som aspekter innen økonomi, transport og miljø. I denne prosessen gjøres det antagelser som baserer seg på tidligere arbeid gjort i WoodSol-prosjektet.

Byggbarheten er undersøkt for et presentert referansebygg. Fokusområdene vurdert i byggbarhetsaspektet er størrelsen på, og rotasjonsstivheten i fundamentene. I tillegg til innvirkningen variasjon av forskjellige komponenter i fundamentene har på den totale rotasjonsstivheten. Den oppnåelige rotasjonsstivheten av fundamentet spenner fra 3315-11460 [kNm/Rad]. Dette er funnet numerisk, og er sjekket for bruksgrensetilstanden for referansebygget. Størrelsen av fundamentene er funnet basert på kreftene som virker på søylene. Disse kreftene er funnet både numerisk og analytisk. Den nødvendige fundamentstørrelsen varierer med tanke på jordstivheten, men for grus og åtte etasjer er den foreslåtte løsningen et stripefundament med bredde 3300 millimeter og høyde 500 millimeter. Volumet av betongfundament for referansebygget virker å være 30-42% lavere enn fundamentene for en tilsvarende betongbygning, når bygget står på fin sand eller grus.

Med tanke på transportaspektet av prosjektet vil dekkeelementene transporteres med semi-trailere, som kan transportere seks elementer per tur. Søylene vil transporteres med en uttrekkbar semi-trailer, og denne kan transportere 14 søyler per tur. Denne transporten trenger politieskorte og er kostbar. Andre elementer vil ha en standardisert transport.

For monteringsaspektet er kostnadene for forskjellige kraner sammenlignet, med tanke på tiden de leies. For WoodSol-prosjektet er det konkludert med at en mobilkran vil være det mest økonomiske på grunn av den raske monteringstiden. Løfteprosessen for de forskjellige elementene er diskutert og det er funnet ut at søylene vil løftes ved bruk av de pre-monterte koblingspunktene ment for dekkeelementene. Mens dekkene må få montert øyebolter slik at det er mulig å løfte dekkene i vinkel for enkel montering. På grunn av at søylene ikke har kapasitet til å stå alene etter montering, må det øverste dekket monteres for å koble 4 søyler sammen så fort som mulig etter at nok søyler er reist. Monteringstiden av bæresystemet er kun fem dager regnet fra etter at fundamentene er klare.

For referansebygget i tre sammenlignet med betong, er den mulige reduksjonen av utslipp av CO_2-ekvivalenter 286.082 kg, 433.087 kg og 576.008 kg for henholdsvis fire, seks og åtte

etasjer. 576.008 kg tilsvarer å kjøre en million nye Volvo biler 4,5 kilometer. Kostnader for fundamenter er 30-40% mindre, avhengig av jordstivheten, for referansebygget bygget i tre sammenlignet med betong. Dette er basert på en reduksjon på 44% for nødvendig betong.

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1. Introduction

1.1. Background

This master thesis is a part of the WoodSol project. The project is carried out by the institutes of NTNU and SINTEF in cooperation with other qualified partners.

Sustainable development is universally quoted as that which "*meets the needs of the present, without compromising the ability of future generations to meet their own needs*". Engineers have a responsibility to contribute to the sustainability agenda by promoting sustainable methods of construction (Mosley et al., 2012). More massive timber constructions may be a great contributor to a more sustainable future.

For more than a century urban skylines world over have been built with the unsustainable materials steel and concrete. These materials have outstanding structural properties and have for a long time been the appropriate choices for multi-story buildings in urban areas. Unfortunately, these materials do not fulfil one of the most important criteria of modern development, the criteria of environmental sustainability.

In Norway, as in the rest of Europe, the building sector is responsible for approximately 40% of the land-based energy consumption as well as 40% of emission of greenhouse gases (WoodSol, 2016). For the world to have any chance on reaching the goal of a temperature increase below 2°C relative to pre-industrial levels the energy and pollution from the building sector need to be dramatically decreased (Skullestad, 2016). There are two ways to address climate change. One way is to reduce the CO_2 and other greenhouse emissions, the other way is to find ways to store these gasses. Wood can contribute to both (Green and Karsh, 2012).

Over the last decades the forest, especially in parts of Scandinavia, have had a rapid growth. The number of trees has almost tripled in Norway (WoodSol, 2016) while the Swedish forest have doubled (Green and Karsh, 2012). This have laid the ground work for more sustainable harvest and processing of this wood to be used for buildings, replacing much of the steel and concrete used today.

One of the reasons for replacing steel and concrete in large buildings is that timber has a so-called zero-emission of CO_2 as well as other attributes which makes it highly attractive as a structural material. Due to the environmental issues and modern technology, the development of timber products such as GLT, CLT, LVL and other wood composite materials have accelerated, and the dream of modern high-rise timber buildings taking a bigger share of the building sector seems to be within grasp. Timber structures, especially high-rise buildings have developed a lot this decade, but there is still need for a more functional structural system in order to be able to compete with concrete and steel buildings. Several of the high-rise timber buildings built to date are structures with a very large story height compared to similar buildings in steel and concrete. This is not favorable in for example housing projects.

WoodSol research project is a project to develop industrialized structural solutions based on rigid wooden frames for use in urban high-rise buildings up to ten stories with a large architectural flexibility. The WoodSol project started in January 2016 and is expected to be finished with the structural system and hopefully a prototype at the ending of December 2019 (WoodSol, 2016). The structural solution is based on prefabricated timber elements for columns, decks, and walls. This results in rapid construction, low pollution and a high degree of safety during the construction phase.

With a highly functional prefabricated timber system that allows large open spaces and multiple stories, the industrialized world will hopefully see an increase in multi-story timber structures. This may be a step towards decreasing the high CO_2 pollution in the building sector.

1.2. Scope

The WoodSol project is built up of seven work packages, and this master thesis is a part of work package 2 (WP2), production and assembly of structural system and components.

The authors have taken on the task to check in what degree the WoodSol concept is buildable, and find solutions to make it more buildable. In addition to this, the authors have tried to uncover challenges that may make the concept unbuildable. In doing so, the following aspects have been evaluated:

- Buildability
- Transportation
- Assembly
- Environment
- Economy

Within WP2, the authors have put the most emphasis on the erection process and the constructional details that will be important for the concept to be doable and favorable. The authors of this thesis have focused on the foundations of the columns, stability of the columns during the erection process, and the final stiffness and stability of the structure. Calculations and models are based on a reference building, made by the authors. This building is very general and not at all complex.

Considering the WoodSol project mainly focuses on the bearing structure, this thesis will not focus on final completion of the building or installations such as electrical systems and piping. The groundworks are a significant part of a building project, and will demand a lot of time and work. Therefore groundworks have not been prioritized in this thesis.

1.3. Structure of the report

Chapter 2

In this chapter the necessary theory that substantiate the thesis, and previous work on the focus areas are presented.

Chapter 3

A brief overview of the goals, visions and build-up of the WoodSol project is presented.

Chapter 4

The reference building is introduced. Different elements of the building is dimensioned to check if the reference building is buildable with the WoodSol concept. The rotational stiffness in the foundations, capacity of the columns and stiffness of the entire structure and their challenges are presented, along with proposed solutions.

Chapter 5

The transportation of the different structural elements is discussed. Different rules inflicting on the transportation and the number of transportations needed for the different elements is shown. The type of vehicle used to transport elements is also discussed.

Chapter 6

The assembly of the structure is explained in detail. Everything from what cranes to be used to how to mount the columns is discussed. Different erection methods for the structure are also considered. An estimation of the building time for the bearing structure is shown.

Chapter 7

The environmental benefits of a timber building compared to a concrete building is discussed, as well as the differences in pollution under transportation. The possibilities for savings of pollution is presented.

Chapter 8

The cost of different part of the structure as well as transportation and cranes is estimated. The cost of different solutions is compared and the influence of the dimensioning of components is shown.

Chapter 9

A summary of the results found in earlier chapters are presented, as well as proposals for further work.

2. Use of timber in construction

2.1. History of timber constructions

Shelter against wind, rain, and cold is one of the three basic needs for humankind, and since ancient times wood has been one of the most important building materials. Much has happened to the timber structures since the ancient times and up to this date, but even 3000 years BC, they made longhouses in Central Europe. The longhouses that have been found is estimated to have had a length and a width of approximately 45 meters and 7 meters (Thelandersson and Larsen, 2003).

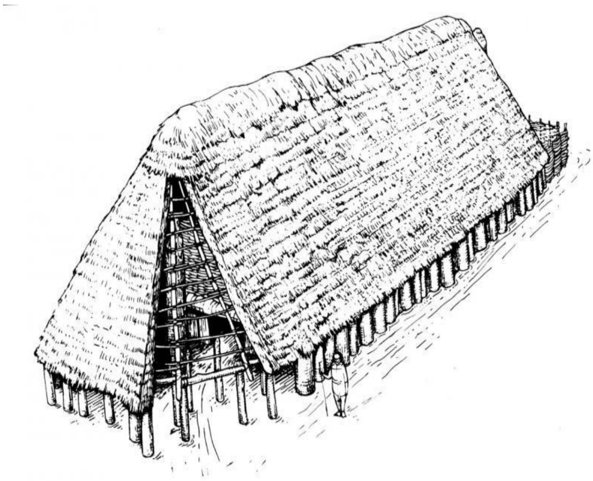


Figure 2.1: Long house from 3000 BC (scottishheritagehub, 2017).

From the longhouses with one story, the evolution of timber developed further on to multistoried buildings. Timber constructions have even been made with multiple stories for centuries, as the five-story pagoda in Japan shown in *Figure 2.2* from the year 730 AD shows. The pagodas had short spans, but the architectural remarks were outstanding. What might be even more impressive is that these padogas still stand today in a high seismic and damp environment (Thelandersson and Larsen, 2003).

Figure 2.2: Five story pagoda, Japan (Frech, 2011).

The maximum dimensions of solid timber sawn directly from logs is in the order of 300 millimeters or less. This makes the largest possible span of structural timber beams to about 5 – 7 meters without trusses, at least before the development of glulam in the early nineteen hundreds. The development of glulam beams is still competitive today, and by creating curved glulam for arch beams the possible span for timber structures increased drastically. This made it possible for large, open spaces with the use of timber for single story buildings. As can be seen in *Figure 2.3*, Stockholm railway station was made using curved glulam in 1925 (Thelandersson and Larsen, 2003).

Figure 2.3: Stockholm central railway station, built 1925 (Thelandersson and Larsen, 2003).

The use of wood in multi-story buildings, more than 2 stories, was not allowed in urban areas in Norway in the period 1907 to 1997. This resulted in a slow development of multi-story timber structures in that period (Thelandersson and Larsen, 2003). From 1997 to this date, the interest and the structural solutions have developed a lot, and modern multi-story buildings such as "Treet" in Bergen, which has 14 stories, have been getting a lot of attention (TekniskUkeblad, 2015). The recent decades have given the opportunity of really revolutionizing timber buildings, but structural solutions for a more rapid erection have to be developed for economic reasons as well as functionality during the construction phase.

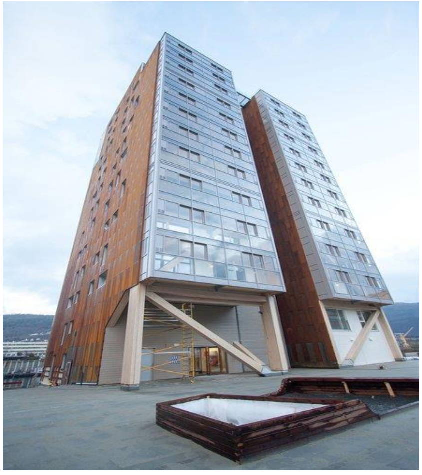


Figure 2.4: «Treet» Bergen (TekniskUkeblad, 2015).

2.2. Wood as a construction material

Wood is an orthotropic material, meaning it has different properties in the three different directions, radial, tangential and longitudinal. The stiffness is for example 10 to 15 times higher in the longitudinal direction than in the radial direction and up to 30 times higher than in the tangential direction (Kristian, 2009). Timber denotes wood which is suitable for building or carpentry, and for various other engineering and construction purposes. In this thesis timber is used to refer to any stage of the wood after the tree has been cut down.

Wood in itself is a complicated material, containing hollow cells in the longitudinal direction capable of transporting water and nutrition. Wood contains about 50% carbon, 6 % hydrogen and 44 % oxygen in the form of cellulose, hemicellulose and lignin (Bjørge and Kristoffersen, 2017).

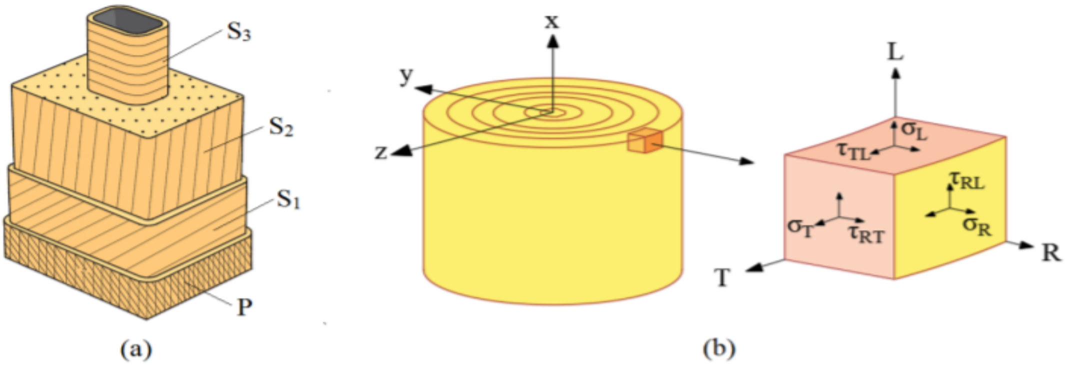


Figure 2.5:
a) Buildup of the cells b) Directions of stresses (Bjørge and Kristoffersen, 2017).

Dissimilar to concrete and steel, timber is not composed of a man-made recipe. Timber specimens are made in and by nature, and therefore properties of the timber specimens are highly influenced by the environment of which the timber is collected. Everything from the quality of the soil to the amount of wind and sunlight has an impact on the properties of the given tree. This gives timber a high degree of variability of properties.

Compared to its weight timber has high strength and stiffness. The properties of timber result in a low self-weight in the construction which is beneficial in urban areas as it may reduce the size of the foundation. It may also make it easier to add stories on existing buildings (Klund et al., 2017).

Timber materials are often referred to as being "carbon-neutral", due to the wood's ability to temporary store CO_2. The CO_2 released by timber materials due to decay or incineration was once removed from the atmosphere through photosynthesis. However, if the global biomass stock is reduced due to timber production, the carbon concentration in the atmosphere would increase, and thus, carbon-neutrality would not be achieved (Skullestad, 2016). Therefore, an important prerequisite for obtaining carbon-neutrality for the timber materials, is a sustainable harvest of wood, where new biomass is added to uphold the capacity for storing CO_2. This thesis assumes sustainable harvest when talking about the climate impact of timber as a building material.

2.3. Wood compared to steel and concrete

Compared to its weight wood have a high strength and stiffness, and its specific stiffness E/ρ is almost as high as for steel, even though the modulus of elasticity (E) is low compared to steel and concrete.

Material	E [MPa]	ρ [kg/m^3]	Specific stiffness E/ρ
Steel	210000	7800	27
Concrete	35000	2400	14
Wood (C24)	11000	420	26

Table 2.1: Relative stiffness of steel, concrete and wood (Klund et al., 2017).

Wood carries several benefits in addition to its strength/weight ratio, which makes it an excellent construction material. One such benefit is its thermal properties giving it a resistance against high temperatures, unlike steel. Steel expands, and can even collapse in high heat. Wood, on the other hand, dries out and becomes even stronger as the heat increases. In addition, wood has a low heat conductivity in comparison to steel, which makes wood applicable for wall coverings and ceilings (understandconstruction.com, 2017).

In the thesis "Høyhus i tre som et klimatiltak", by Skullestad (Skullestad, 2016), three different approaches are used to assess the impact wood has on the climate as a building material, compared to concrete and steel. In approach 3 she assumes that 90% of the timber material is incinerated with heat recovery to replace natural gas as an energy source, after destruction of the construction. This allows the values of climate impact from timber to be negative, when sustainable harvest is assumed.

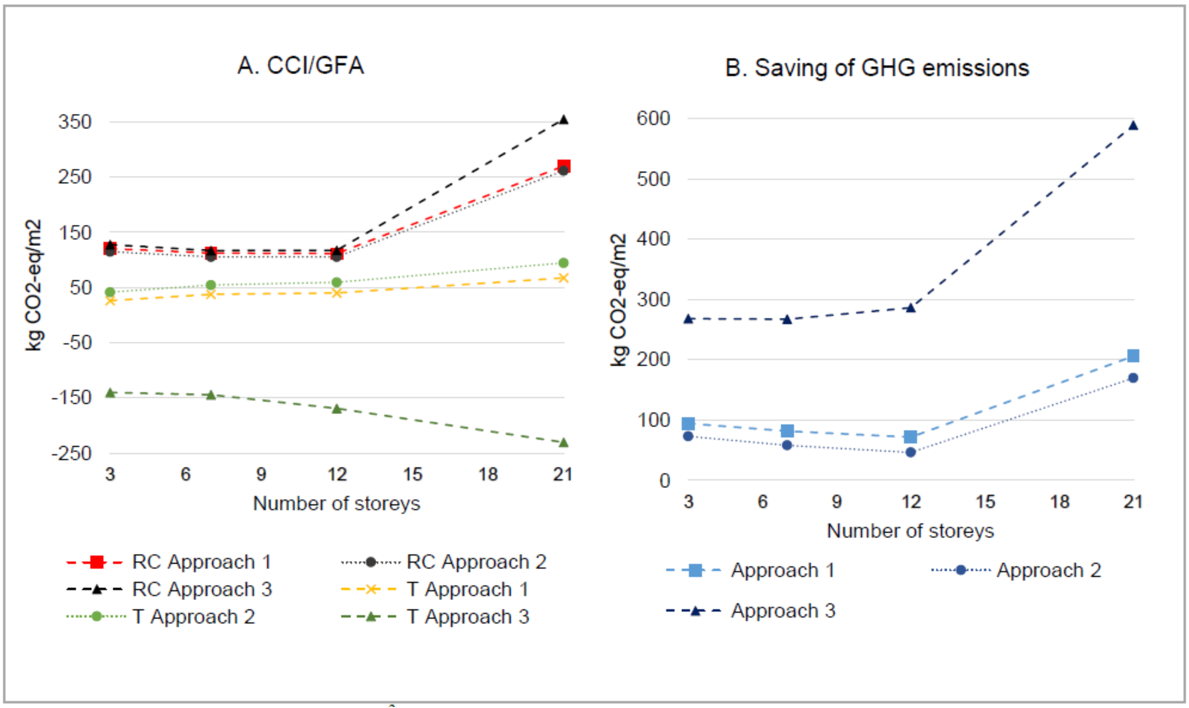


Figure 2.6: On the left: Difference in kg CO_2-eq/m^2 gross floor area for three different calculation methods for reinforced concrete (RC) and timber (T). On the Right: The potential saving of greenhouse gasses with the three different calculation methods (Skullestad, 2016).

The increasing urbanization have created a demand for more high-rise buildings, but the challenges with climate change require engineers to see to more eco-friendly solutions (Klund et al., 2017). The WoodSol project has a goal of creating a competitive solution in the environmental aspect as well as the structural aspect. The solution is thought to be from five to ten stories. As can be seen from *Figure 2.5* there are great differences between the reinforced concrete solution and the timber solution up to ten stories. But the differences are even greater, and rises even more from 12 to 21 stories. This can be an incentive for future work, to create a solution that can reach even higher. This may be a good way to ensure environmental friendly construction of even taller buildings in the future.

Even though hollow core concrete decks have a lower cost financially, the kg CO_2-eq/m^2 is about three times bigger than the decks suggested by Bjørge and Kristoffersen (Bjørge and Kristoffersen, 2017) (EPD-Norge, 2014). The decks suggested have continuous Kerto-Q plates on the top and bottom, and is not the most environmental friendly model considered, with regards to development, but Bjørge and Kristoffersen found this to be the best alternative over all.

2.4. Prefabrication of timber

Prefabrication is in the construction business referred to as smaller or bigger parts of the structure which is created off site, where the environment is controlled and stable. This gives the structure elements a higher degree of preciseness. Eliminating most of the chances for human errors and exposure to harsh environments during fabrication can help avoid moisture problems and other errors. The fabrication off site can be ongoing as another part of the building process is taking place on site, and therefore, contribute to a more efficient process. It may also eliminate the area needed to store materials on the building sites. The erection time on site is shown to be considerably shorter when prefabricated elements are used, which again leads to a faster return on the investment. With a well-engineered prefab solution, a lot of the ladder works and heavy lifting by personnel at site can be disregarded. This means a safer work environment for the workers on site. One of the big goals of many contractors are to get a safer work environment and get a lower injury rate. Prefabrication of elements of the structure can contribute to achieve this goal (Hartley and Blagden, 2007).

With the same plans being constantly built the manufacturer has records of exactly the amount of materials needed for a given task. According to the UK group WRAP the waste can be reduced by up to 90% by using prefabricated elements for construction instead of everything being built on site (Hartley and Blagden, 2007).

Now, larger timber structures are on the rise, and here as well, the preciseness and effectiveness can benefit from prefabrication. From the 1980's the prefabrication companies made use of technology to be able to optimize the elements and modules to the costumers wishes and needs (Thue, 2018).

Prefabrication of elements and modules are now used in the some of the largest timber buildings in the world. "Treet" in Bergen has a load carrying structure of glulam columns and consists of prefabricated modules. The shafts are made of prefabricated CLT-elements and the foundation is made of concrete. This structure was the tallest timber structure in the world when it was completed (Abrahamsen and Malo, 2014).

3. Project WoodSol

The WoodSol project is a research project financed by the Research Council of Norway and the consortium partners. With a planned ending date 31.12.2019 and startup in 2015, the project duration is 4 years. The WoodSol project is coordinated by NTNU, Department of Structural Engineering, with project leader Professor Kjell Arne Malo. The Department of Structural Engineering is the grant holder, while Sintef Byggforsk and NTNU Department of Civil and Transport Engineering is sub-contracting partners (WoodSol, 2016). Some of the other partners are Moelven Limtre AS, SWECO Norge AS and ÅF Advancia AS.

The main goal of the project is to develop industrialized structural solutions based on rigid wooden frames for use in urban buildings up to ten stories, with large architectural flexibility. When timber is used in taller structures, it is often because it is specified by the builder, even though the project will be more costly and perhaps less practical. By finding a better solution for a wood-based structural system giving larger spans and more open spaces. This may increase the competitiveness of high-rise timber buildings, as they are not very competitive in the Nordic countries at the moment.

In order to facilitate industrial production, the load bearing structure should primarily be based on grids and repetitions. Architectural flexibility requires floors without too closely placed load bearing elements. To accomplish such a structure, the WoodSol project focuses on three substantial targets:

- The extension of the floor span length without increased story height.
- The horizontal stabilization of the building by moment resisting frames.
- The development of prefabricated couplings to allow rapid erection on site.

The project has a strong focus on the practical documentation of the developed solutions. Hence, the erection of a demonstration building is part of the project. Several articles, master theses and other publications are available on the project website, www.woodsol.no.

The WoodSol project contains seven work packages:

- Project management
- Production and assembly
- Moment resisting frames
- Flooring systems
- Acoustics
- Prototype
- Dissemination

Within these work packages, the main subjects are; Production and assembly of structural systems and components, moment resisting frames, flooring systems and acoustics.

As mentioned earlier, the structural system is based on grids and repetitions. This makes it possible to place the inner walls freely and helps to achieve architectural flexibility. Therefore, the current basis of the structural system is as shown in *Figure 3.1*.

Figure 3.1: The current basis of the structural system.

The flooring system is made up of decks between the columns. These decks are made by top and bottom Kerto-Q plates, with integrated glulam beams between. This wood-box principle gives a very high stiffness (Bjørge and Kristoffersen, 2017). The decks are mounted to the columns with high rotational stiffness in the connections, which reduces the need for additional bracing. *Figure 3.2* shows a simple illustration of the principle of the composite decks.

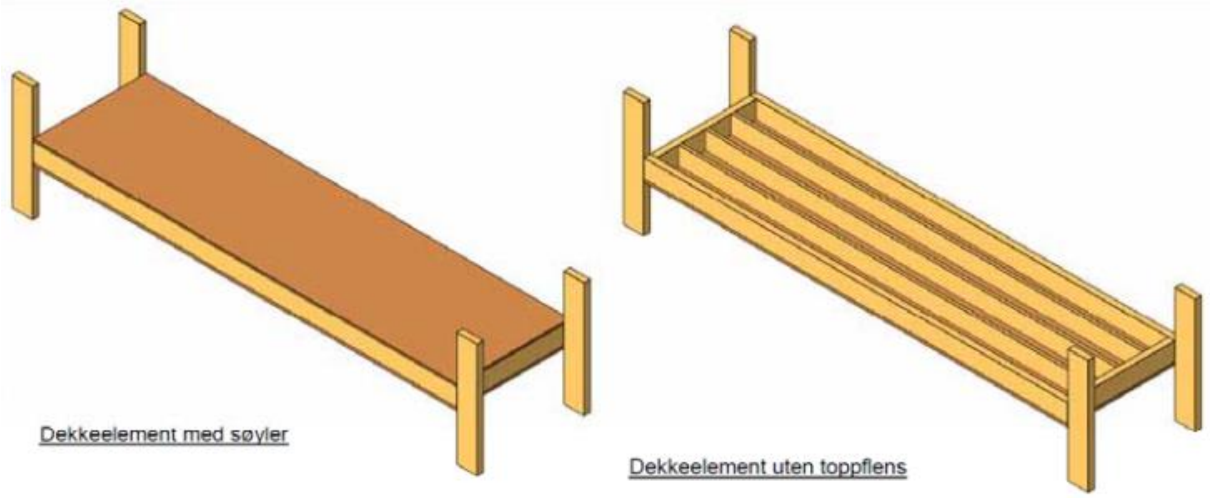


Figure 3.2: Principle of the composite decks.

The connections between the decks and columns uses threaded rods to accomplish the necessary rotational stiffness. The rods work with an angle inside the decks and columns, to the connectors, which optimizes the capacity of the joints. *Figure 3.3* shows the solution of the connections.

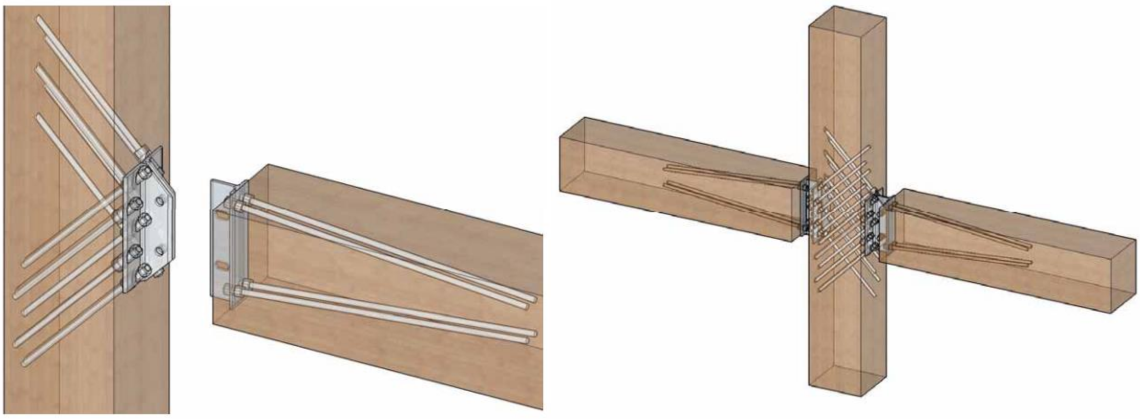


Figure 3.3: Connections between columns and decks.

By addressing the challenges of stiffness in the joints, effective deck elements, easy mounting and prefabrication, hopefully a solution can be found that gives open spaces, open facades, flexible use and longer spans with moment resisting frames.

This master thesis is a part of Work Package 2, production and assembly. Therefore, the themes in this thesis will be buildability aspects, transportational aspects, assembly aspects, environmental aspects and economical aspects of the WoodSol project.

4. Buildability aspects

4.1. Reference building

The authors have made a reference building, which is used as a basis for all the calculations in this thesis. This building is a simple structure, with two rows of 11 columns, and 8 stories. The rows of columns are spaced with 9 meters between, and each column in a row is spaced 2,4 meters from the next one. The story height is 3,5 meters, making the total height 28 meters. Each part of the structure is modelled from what can be expected to be used in the WoodSol concept. The columns are GL30c 400x400 [mm], with all the corresponding material properties. The decks between the columns, which together with the columns make up the moment resisting frames, are modelled from the decks described in the master thesis by Bjørge and Kristoffersen (Bjørge and Kristoffersen, 2017). Their deck is the one considered for the WoodSol concept. The transversal direction of the building or along the length of the decks, i.e. the moment resisting frames, is henceforth referred to as the frame direction. The foundations used in the reference building are subject for calculations in chapter 4.3.

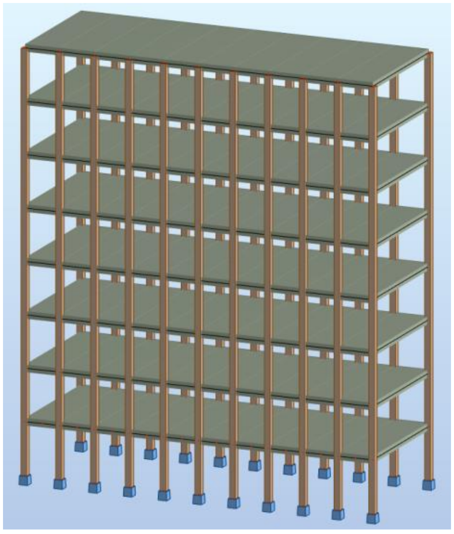


Figure 4.1: Reference building for this master thesis.

4.2. Types of foundations

For the foundations of the columns there are three concepts that are considered. These three are spot foundation for each column, strip foundation and sole foundation. All of these have different properties and usage, and which one to use is a consideration depending on the structure to be built. For economic and environmental reasons, one goal for the WoodSol project is to keep the volume of the foundations to a minimum. Therefore, the hope is that the spot or strip foundations will give sufficient stiffness and strength to be used for most structures.

Spot foundations

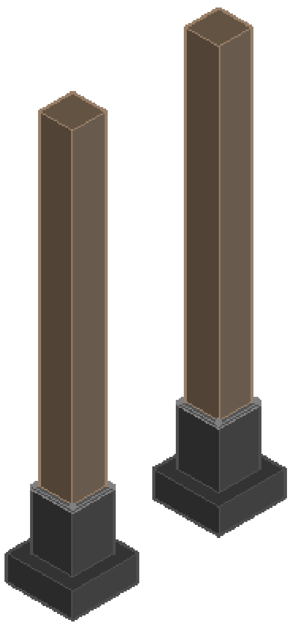


Figure 4.2: Spot foundation.

The columns will be exposed to large moments as well as compression and horizontal forces. These forces must be distributed to the ground. This will happen through the foundations. One type of foundation that can be used is a spot foundation. A spot foundation is a smaller foundation under each column. With the use of spot foundation the need for concrete might be reduced. In Appendix A.1 you can see the spreadsheet used when calculating the foundations. The results are presented in chapter 4.3. If the spot foundations under the columns become so large that the foundations merge between the columns, a strip or sole foundation will be the better solution.

Strip foundation

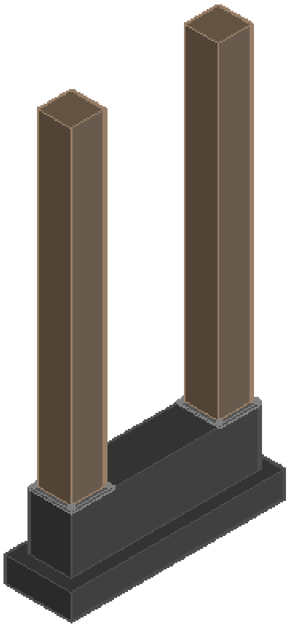


Figure 4.3: Strip foundation.

A strip foundation is a continuous foundation under a row of columns. This can give a high stiffness and stability in the direction of the foundation. Therefore, the optimal direction for a strip foundation, with stiffness and stability in mind, would be in the frame direction, between two columns. Because of the large moments and forces in the bottom of the columns, the foundations still need a certain width. If this width is too large, the foundations will still merge between the columns in one row. This is a challenge when laying the strip foundation in the frame direction. Therefore, one solution may be to lay the strip foundation perpendicular to the frame direction. The solution is presented in chapter 4.3.

Sole foundation

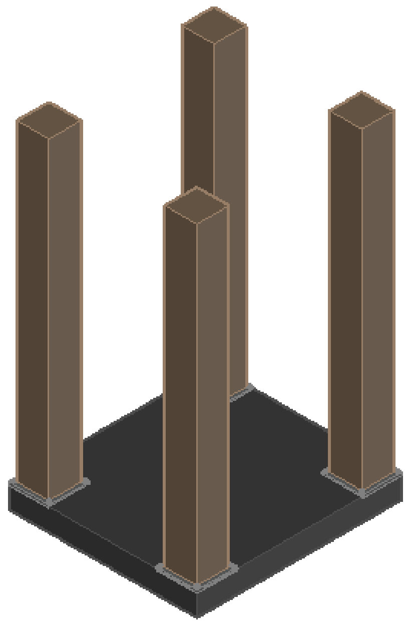


Figure 4.4: Sole foundation.

The sole foundation is a widely used foundation. It consist of a wide area of concrete that covers the whole print of the building, giving support to all the columns and walls. In a building using the WoodSol concept, a goal will be to reduce the need for concrete. This type of foundation will therefore be a less attractive solution, and will try to be avoided.

4.3. Dimensioning of foundations

As mentioned earlier, the sizes of the foundations are of great importance for the CO_2 emissions and the cost of the WoodSol structures. The foundation will be the only part of the structure containing concrete. The reduced amount of concrete and foundation works is a contributing factor towards making timber structures more economical, see chapter 8.4.

Dimensioning process

The dimensioning of the foundations is done by the use of a spreadsheet made by Tumcivil (Tumcivil, 2018). The calculations used are based on the structural rules from the American concrete institute (ACI), which is a leading authority and resource for worldwide development and distribution of standards (AmericanConcreteInstitute, 2018). The safety-factors may vary some from the European codes, but the results achieved from the spreadsheet should be applicable. The results have been checked analytically in Mathcad, see Appendix A.10. When comparing the results from the spreadsheet to the results in Mathcad, the spreadsheet results in conservative dimensions in most cases. When comparing the safety factors used in the spreadsheet with the safety factors for load combinations from the National Annex of Eurocode 0 Table NA.A1.2(A), it can be seen that the factors in the spreadsheet lead to a more conservative result (CEN, 2008a).

The spreadsheet is made so that when the different parameters such as forces, concrete strength and approximate dimensions are put in, the sheet says if the foundation is ok or not by doing calculations according to ACI. The different forces for the different buildings are plotted into the spreadsheet, varying the soil stiffness. Then the necessary foundation sizes are plotted into tables, and a foundation type is chosen for each of the four different soil stiffnesses shown in the tables below. The spreadsheet can be found in Appendix A.1.

Bearing capacity of soil

The bearing capacity of the soil is vital as this tells how much pressure the ground can withstand per m^2. The higher the bearing capacity is, the smaller the foundations can be. The soil bearing capacity found in *Table 4.1* is found in the lecture “design of shallow foundations” by NPTEL (NPTEL, 2017).

Soil	**Safe bearing capacity (kN/m^2)**
Rock	3240
Gravel	440
Loose gravel	245
Fine sand	100

Table 4.1: Safe bearing capacity for different soils.

Eight story timber building

For the eight story building the forces taken by the foundation is:

$$Moment = 259\ kNm \quad Axial\ pressure = 695\ kN \quad Shear = \ 57\ kN$$

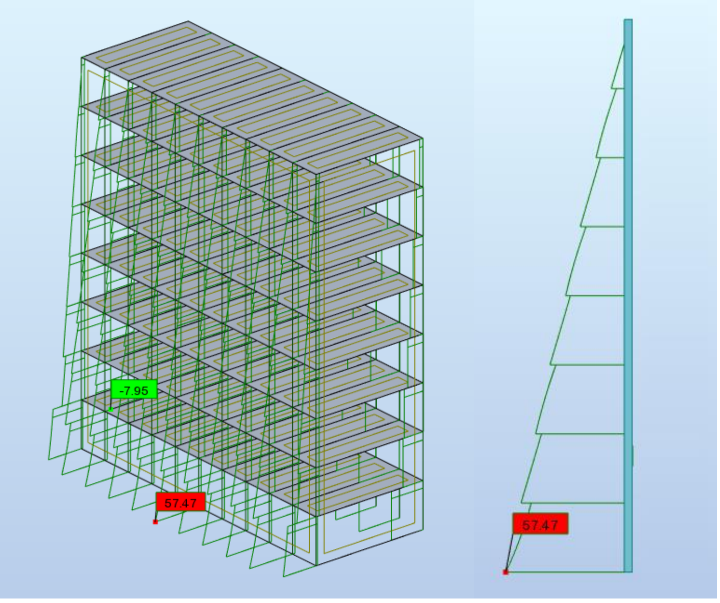


Figure 4.5: Shear forces for reference building with eight stories.

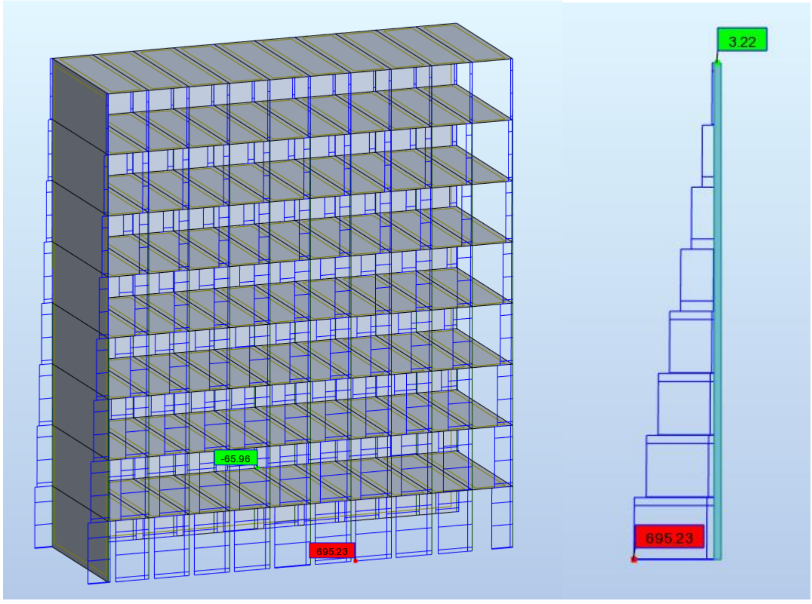


Figure 4.6: Compression forces for reference building with eight stories.

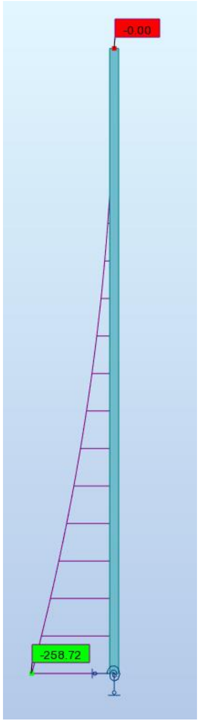


Figure 4.7: Max moment in column for reference building with eight stories before decks are mounted.

Plotting these forces and varying them with different safe bearing capacities in the spreadsheet, the necessary sizes for the foundation for each column is found. The results can be seen in *Table 4.2*.

Soil	Safe bearing capacity (kN/m^2)	Dimension of the foundation (mm x mm x mm)
Fine sand	100	2400 x 4700 x 500
Loose gravel	245	2300 x 2300 x 500
Gravel	440	1950 x 1950 x 500
Rock	3240	1250 x 1250 x 500

Table 4.2: Foundation sizes for eight story building.

As can be seen in *Table 4.2* the sizes vary a lot from what safe bearing capacity is used. The result is that strip foundation is possible for fine sand and loose gravel. Spot foundation is possible for loose gravel, but highly unpractical because of the small gap between the foundations. While spot foundation is possible for both gravel and solid rock. But even if spot foundation is possible for gravel, the gap between the spot foundations will only be 450 millimeters, which might make it more economical to build it as a strip foundation, unless the goal is to minimize CO_2 pollution.

The foundation on fine sand were first found to be 3350x3350x500 [mm], but this had to be changed due to the width between the columns of 2400 millimeters. Since the ground area of the foundation need to be kept constant, the dimension was changed to 2400x4700x500 [mm].

Figure 4.8: Eight story building with strip foundation.

Six story timber building

For the six story building the forces taken by the foundation is:

$$Moment = 146\ kNm \quad Axial\ pressure = 509\ kN \quad Shear = \ 41\ kN$$

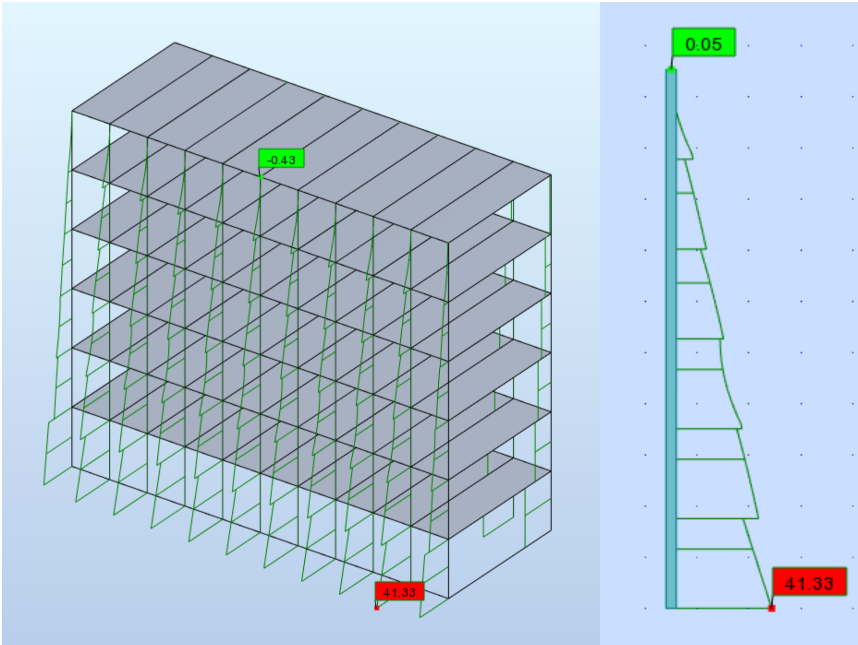


Figure 4.9: Shear forces in reference building with six stories.

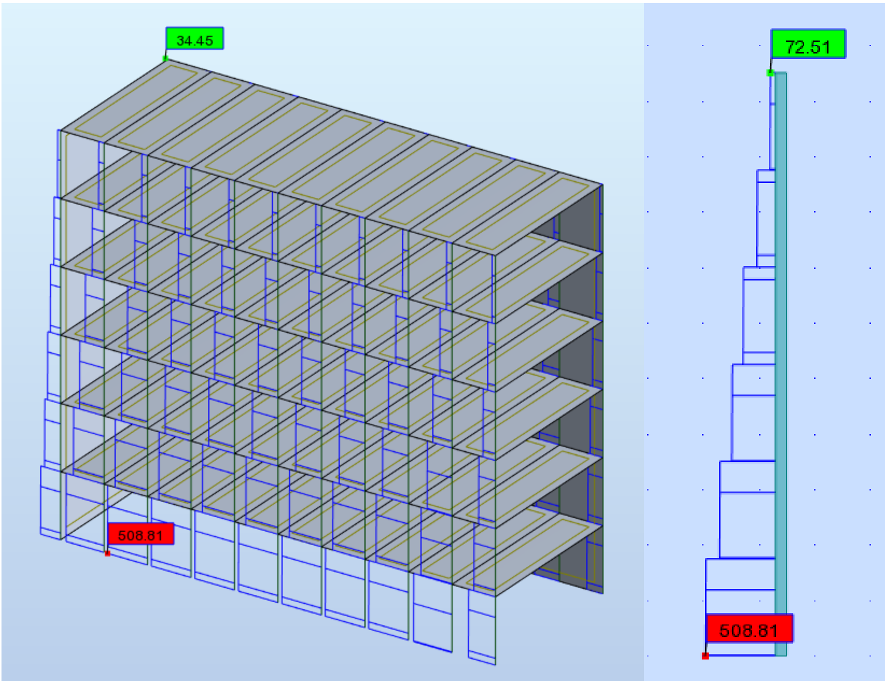


Figure 4.10: Compression forces in reference building with six stories.

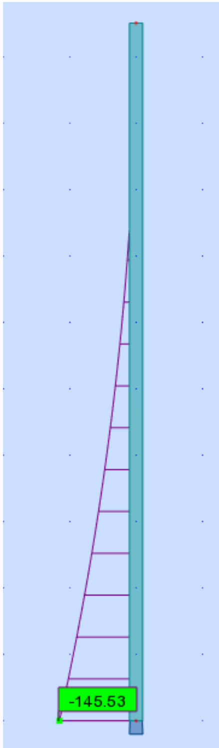


Figure 4.11: Max moment in column for reference building with six stories before decks are mounted.

Plotting these forces and varying them with different safe bearing capacities in the spreadsheet the necessary sizes for the foundation for each column can be found. The results can be seen in *Table 4.3*.

Soil	Safe bearing capacity (kN/m^2)	Dimension of the foundation (mm x mm x mm)
Fine sand	100	2400 x 3400 x 500
Loose gravel	245	1850 x 1850 x 500
Gravel	440	1300 x 1300 x 500
Rock	3240	1000 x 1000 x 500

Table 4.3: Foundations sizes for six story building.

From *Table 4.3* the foundation for every soil type except fine sand can be made as spot foundation. For fine sand it will need to be a strip foundation.

The foundation on fine sand were first found to be 2850x2850x500 [mm], but this had to be changed due to the width between the columns of 2400 millimeters. Since the ground area of the foundation need to be kept constant, the dimension was changed to 2400x3400x500 [mm].

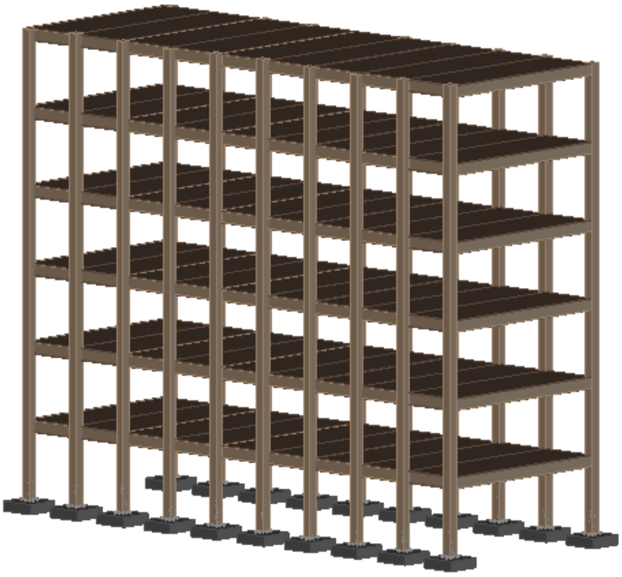


Figure 4.12: Six story timber building with spot foundations.

Four story timber building

For the four story building the forces taken by the foundation is:

$$Moment = 71\ kNm \quad Axial\ pressure = 294\ kN \quad Shear = \ 28\ kN$$

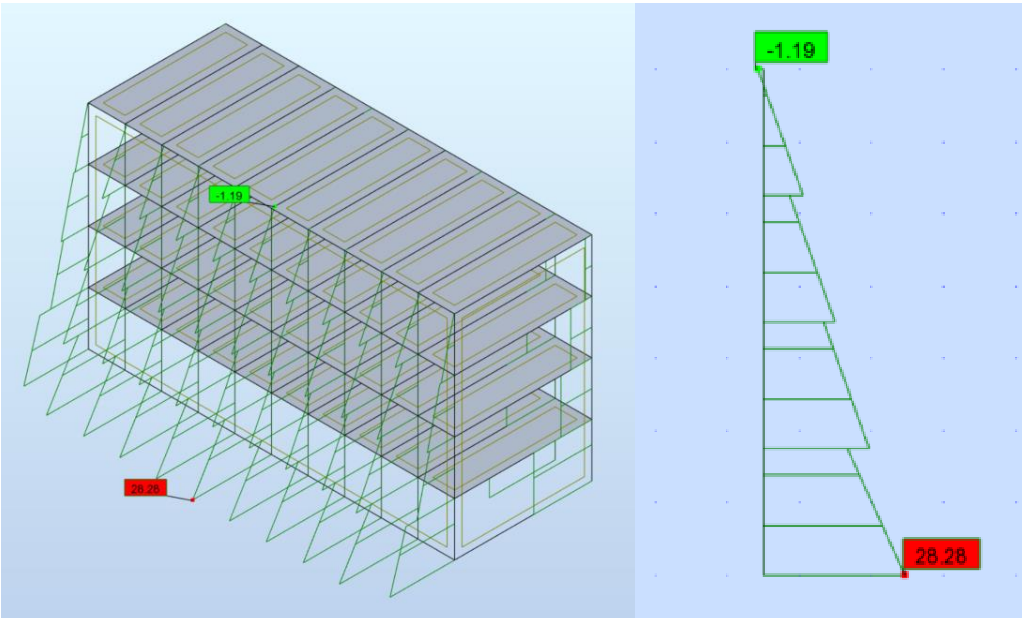


Figure 4.13: Shear forces in reference building with four stories.

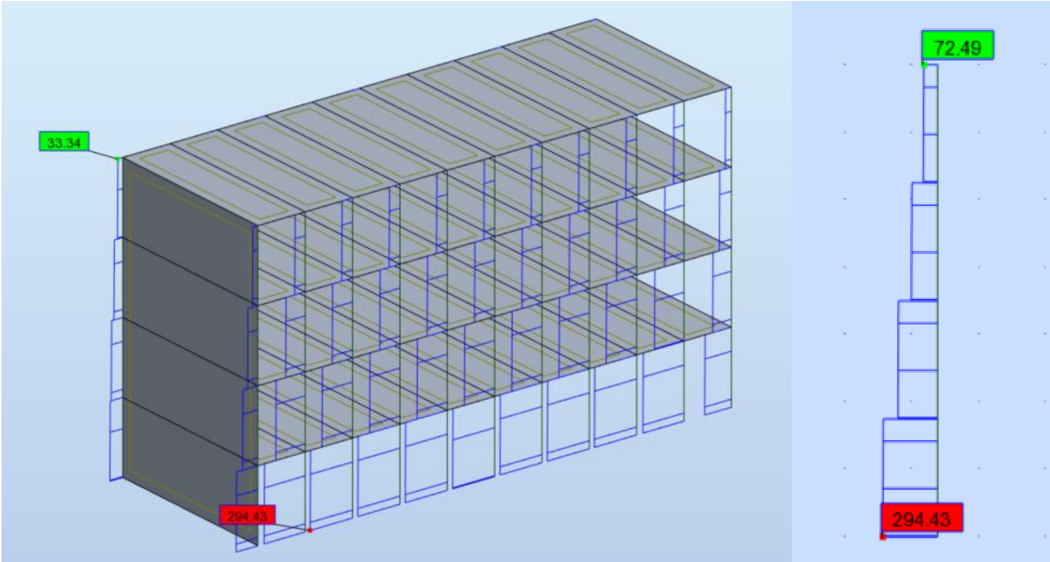


Figure 4.14: Compression forces in reference building with four stories.

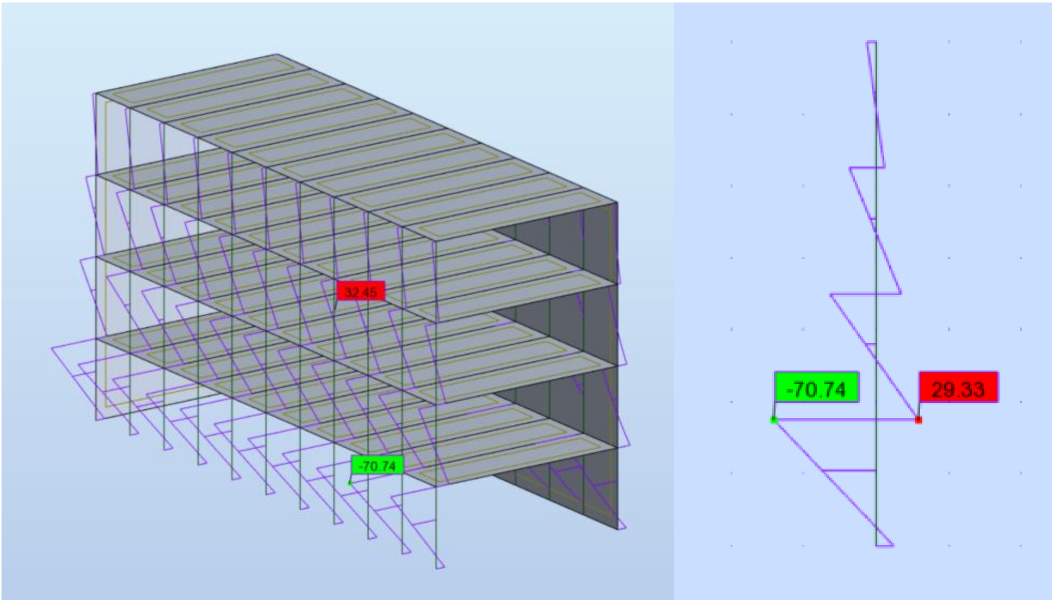


Figure 4.15: Moment in reference building with four stories.

Plotting these forces and varying them with different safe bearing capacities in the spreadsheet the necessary sizes for the foundation for each column can be found. The results can be seen in *Table 4.4*.

Soil	Safe bearing capacity (kN/m^2)	Dimension of the foundation (mm x mm x mm)
Fine sand	100	2200 x 2200 x 500
Loose gravel	245	1600 x 1600 x 500
Gravel	440	1300 x 1300 x 500
Rock	3240	900 x 900 x 500

Table 4.4: Foundation sizes for four story building.

From *Table 4.4* the foundation for every soil type can be made as spot foundations for a four story building, but for fine sand the most practical would be to use strip foundation.

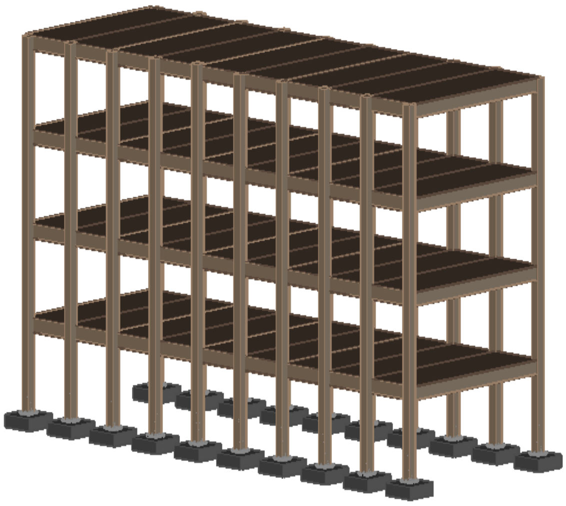


Figure 4.16: Four story building with spot foundation.

Eight story concrete building

For the eight story concrete building the forces taken by the foundation is:

$$Moment = 113\ kNm \quad Axial\ pressure = 1613\ kN \quad Shear = \ 58\ kN$$

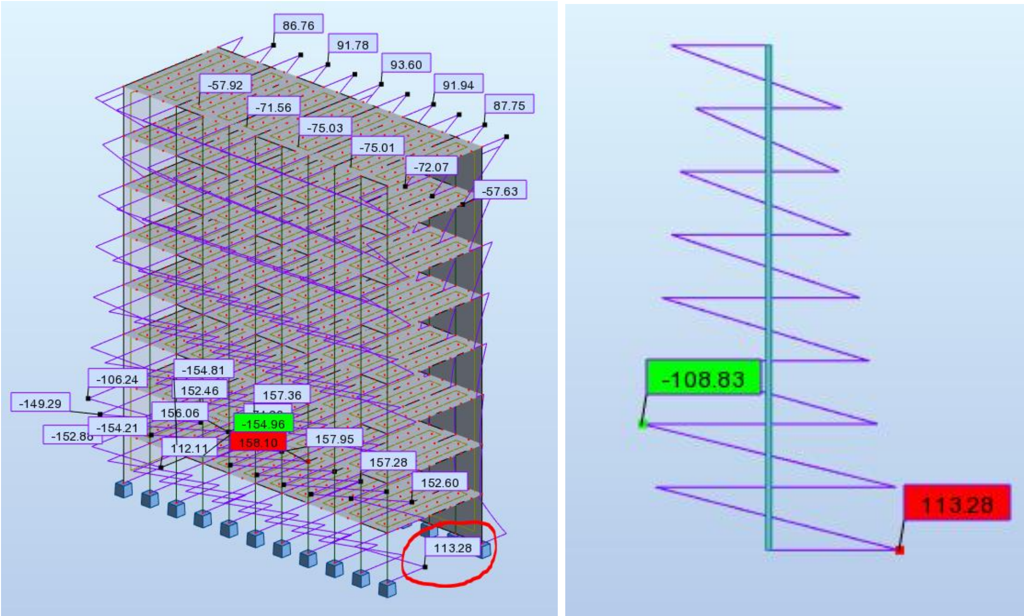


Figure 4.17: Moments in eight story concrete building (all the numbers were needed to show the relevant one, because the worst moments in the columns are not in the foundation).

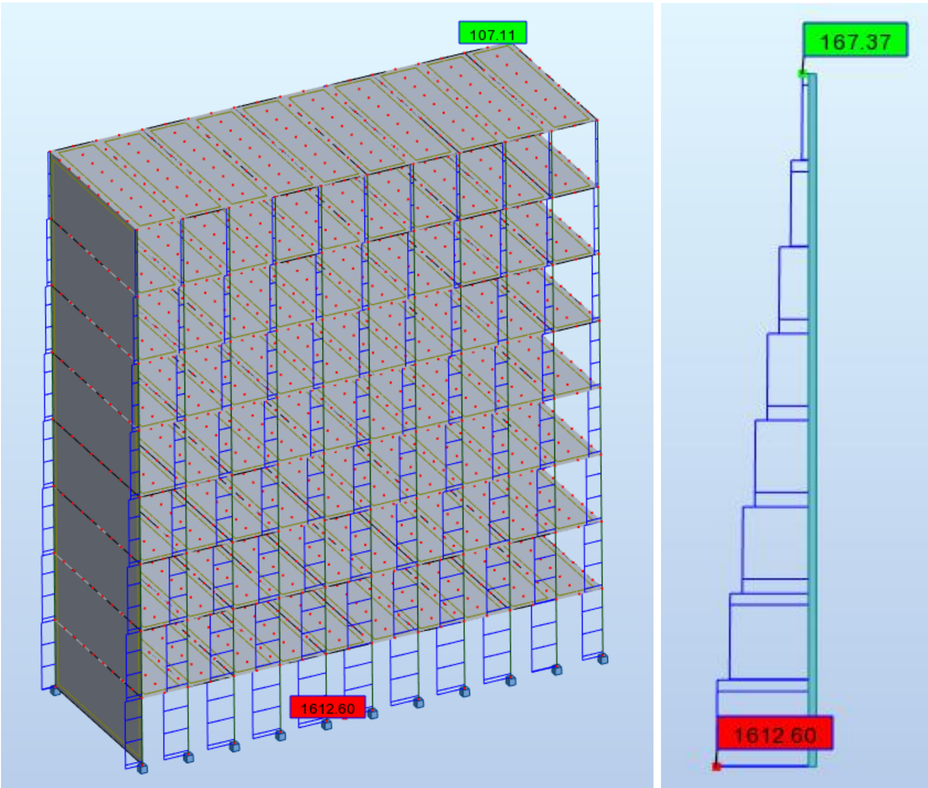


Figure 4.18: Compression forces in eight story concrete building.

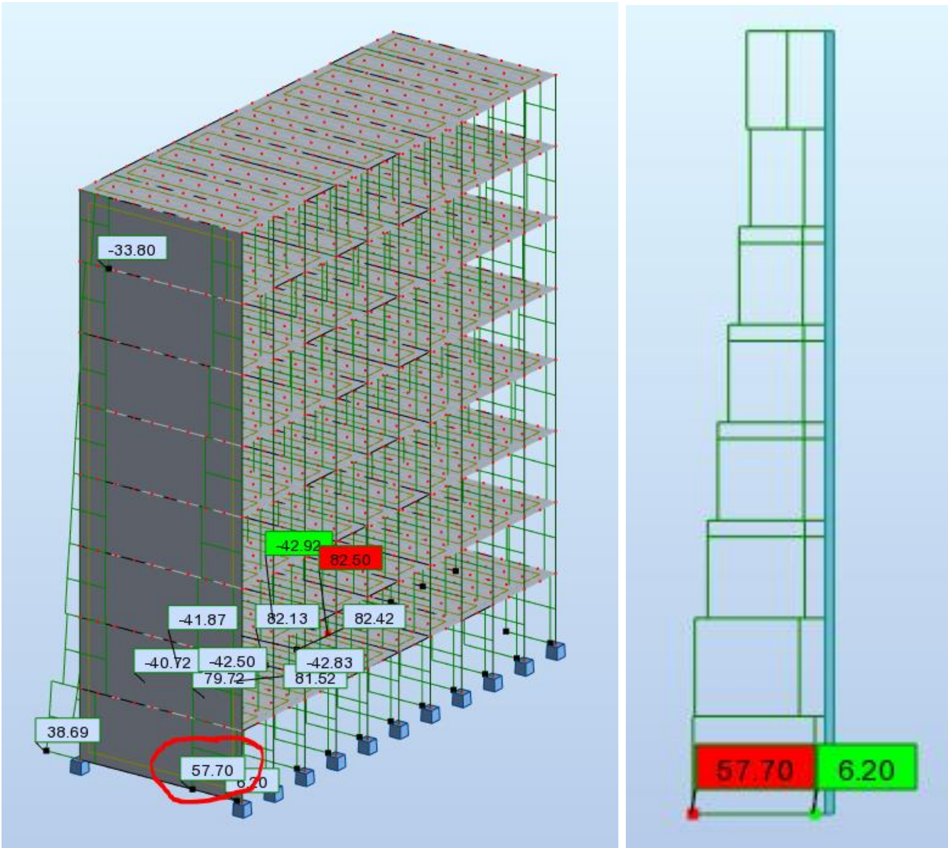


Figure 4.19: Shear forces in eight story concrete building (all the numbers were needed to show the relevant one, because the worst shear forces in the columns are not in the foundation).

Plotting these forces and varying them with different safe bearing capacities in the spreadsheet the necessary sizes for the foundation for each column can be found. The results can be seen in *Table 4.5*.

Soil	Safe bearing capacity (kN/m^2)	Dimension of the foundation (mm x mm x mm)
Fine sand	100	2400 x 8400 x 500
Loose gravel	245	2400 x 3300 x 500
Gravel/Soft rock	440	2100 x 2100 x 500
Rock	3240	1000 x 1000 x 500

Table 4.5: Foundation sizes for eight story building.

As can be seen in *Table 4.5* the sizes vary a lot from what safe bearing capacity is used. The result is that even for the concrete building a strip foundation is useable on fine sand and loose gravel. And spot foundations is usable for gravel/soft rock and solid rock.

The foundation on fine sand were first found to be 4500x4500x500 [mm], but this had to be changed due to the width between the columns of 2400 millimeters. Since the ground area of the foundation need to be kept constant, the dimension was changed to 2400x8400x500 [mm].

Since the distance between the columns in the longitudinal direction is 9 meters and the columns need 8,4 meters wide foundations each, the most practical would be to cast this as a sole foundation.

For the foundation on loose gravel having the foundation size 2800x2800x500 [mm], the same procedure as above was done. This resulted in the dimensions 2400x3300x500 [mm] and thereby a strip foundation can be chosen.

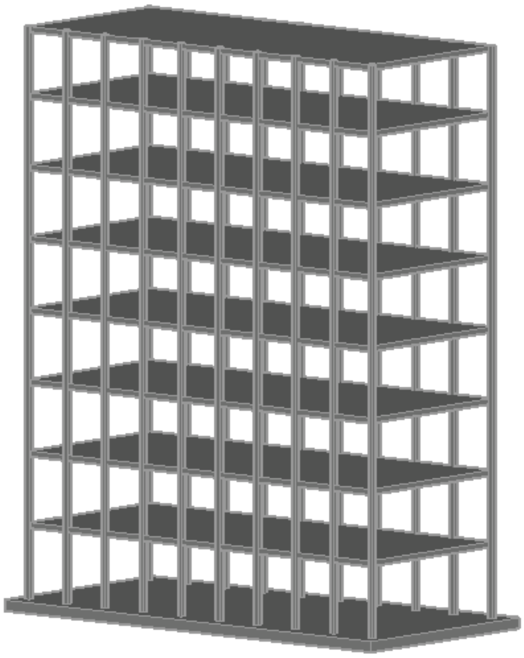


Figure 4.20: Eight story concrete building with sole foundation.

Sources of error

The foundation sizes are calculated from a plot-in spreadsheet made by TumCivil (Tumcivil, 2018) which have used the rules from American concrete institute to dimension the foundations. The results from the spreadsheet have been checked in Appendix A.10, and the spreadsheet gives larger dimensions than calculated in most cases. Compared to the safety factors in the National Annex of Eurocode 0, the spreadsheet gives a conservative result.

Results

It is concluded that a full-scale sole foundation, see chapter 3.4, will not be necessary for any of the WoodSol buildings with eight stories or less. Using a strip foundation will be the most practical for buildings with more than six stories, unless the ground is very stiff. While six stories and below will be able to use spot foundations for the columns unless the ground is very soft.

If the ground is soft it could be beneficial to excavate and use a stiffer soil. This can reduce the dimension size, which can be beneficial for the pollution of CO_2-equivalents and the costs of the foundations.

The use of concrete in the foundation is a lot higher in the concrete building than the timber building for normal/soft soil conditions. For fine sand the timber building uses 42% less concrete than the concrete building in the foundations. While for loose gravel it uses 30 % less.

Another interesting finding is that for building on solid rock the concrete building actually needs a smaller foundation than the timber building, despite the extra compression force due to extra weight. This is because the timber building have a higher moment combined with lesser compression force resulting in smaller stability against overturning. This effect can probably be countered by anchoring the foundation for the timber construction to the rock by steel bars.

The weight of the structure is significantly increased when filling the decks to achieve wanted acoustic properties. This makes the weight differences in the timber building compared to the concrete building less drastic. By finding another solution than adding weight to the decks to achieve the wanted acoustic properties, the need for concrete in the foundation will be drastically reduced. This is a proposal for further work.

4.4. Connection of steel plate to foundation

4.4.1. Rebar steel anchors

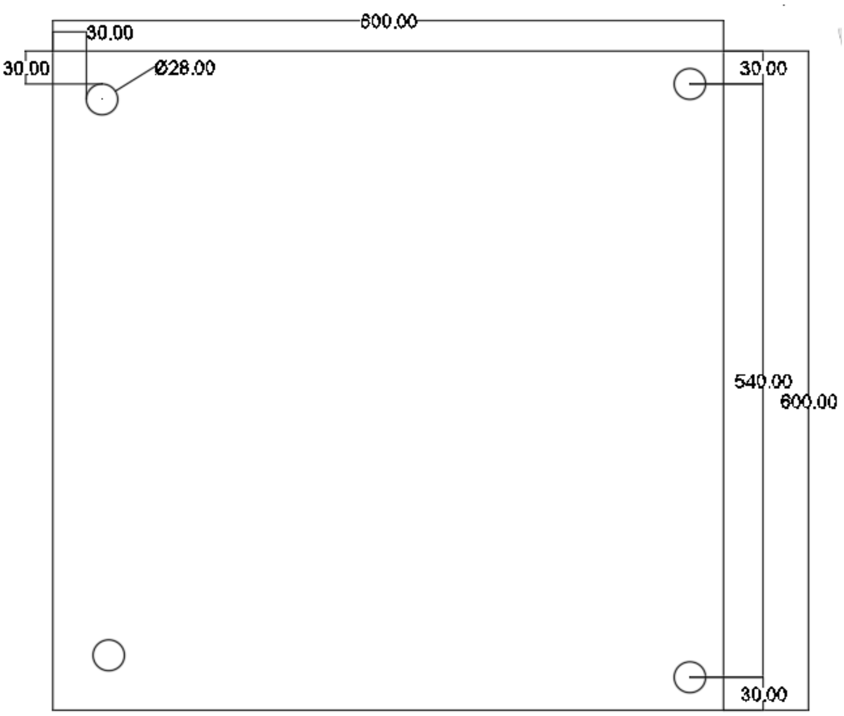


Figure 4.21: Steel plate.

Connecting the steel plates to the foundations can be done by the use of reinforcement like steel anchors without foot as well as anchor bolts with a foot. In Appendix A.2 the necessary diameter of the anchor rods has been calculated, and two solutions that are adequate.

If using rebar steel anchors there will be needed eight anchors with a diameter of 22 millimeters and a length of 480 millimeters. If wanted, four anchors with a diameter of 28 millimeter with an anchoring length of 650 millimeters can be used instead. The two anchors solutions will be bent 90° in the middle of their length, resulting in a depth of about 250 millimeters for the 22 millimeter solution, and 340 millimeters for the 28 millimeter solution. See *Figure 4.22* for illustrations and see Appendix A.2 for calculations.

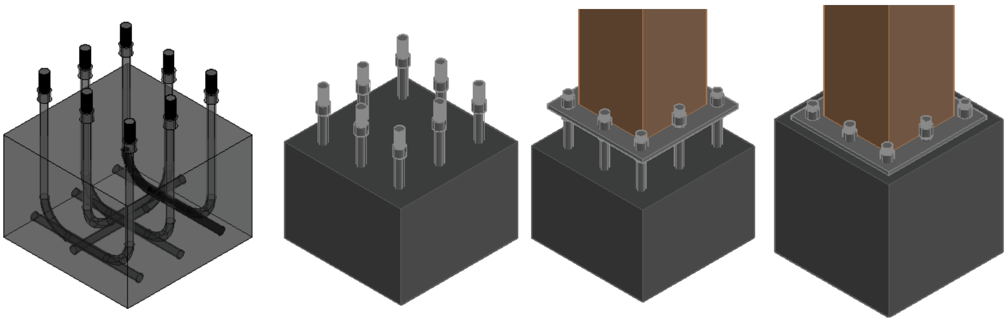


Figure 4.22:

a) Transparent view of steel anchors. b) Casted in steel anchors.

c) Column with steel plate is mounted and adjusted. d) Final grouting up to the underside of the steel plate.

The steel anchors are mounted in the three steps illustrated above. First the anchors are casted in the lower part of the foundation. Then the steel plate connected to the column is mounted on the nuts fastened on the anchors, and the plate is adjusted so that it is level. Last, the connection is grouted up to the underside of the steel plate, resulting in the nuts under the steel plate to be casted into the foundation.

4.4.2. Steel anchors with foot

The usage of anchors with a foot is one of the options when considering the connection of the steel plate to the foundation. In Appendix A.3 the necessary diameter of the anchors and the necessary anchorage depth, as well as the diameter of the foot of the anchor is calculated.

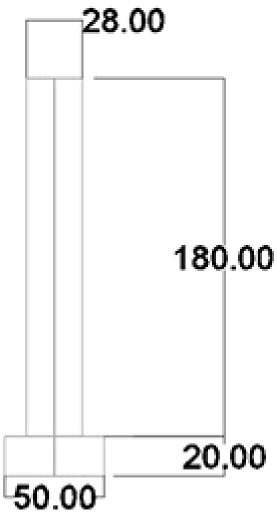


Figure 4.23: Dimensions of the dowel with foot.

The anchors have been calculated to be sufficient for the reference building when using either four anchors with a diameter of 28 millimeters, or eight anchors with a diameter of 22 millimeters. The necessary depth is 200 millimeters and the diameter of the anchor foot needs to be 50 millimeters. The diameter of the anchor of 28 millimeters can withstand the compression and the tensile forces acting in the section, while the dowel foot of 50 millimeters is adequate to prevent pull-out of the anchor. The depth of 200 millimeters will prevent pry-out cracking from happening (Haga and Reiersølmoen, 2012). See annex A.3 for calculations.

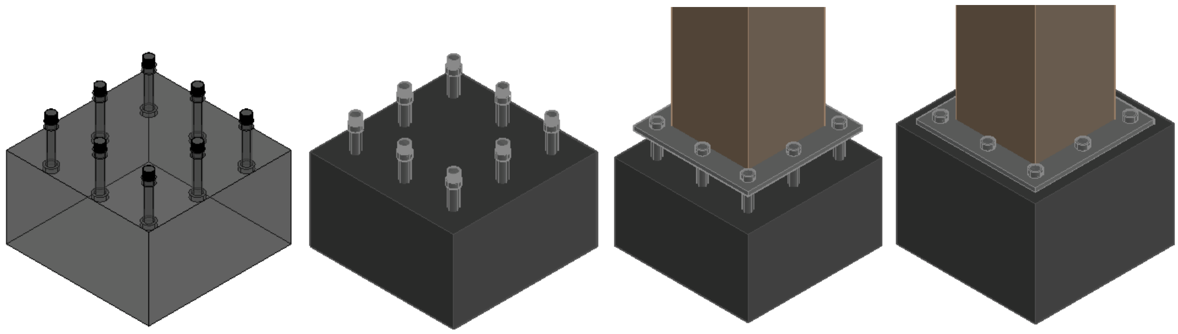


Figure 4.24:

a) Transparent view of dowels. b) Casted in dowels.

c) Column with steel plate is mounted and adjusted. d) Final grouting up to the underside of the steel plate.

The anchors are mounted in three steps as illustrated above. First the anchor bolts are casted in the lower part of the foundation. Then the steel plate with the column on is mounted on the nut fastened on the anchor bolts, and the plate is adjusted so that it is level. Last the connection is grouted up to the underside of the steel plate, resulting in the nuts under the steel plate to be casted into the foundation.

4.4.3. Anchor rods without steel pate

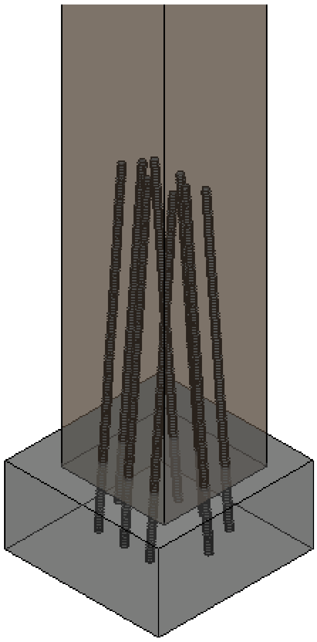


Figure 4.25: Attaching column to foundation by the use of steel rods only.

The last solution for connecting the column to the foundations is by simply removing the steel plate, casting the extended steel rods from the column into the foundations. This solution has not been calculated in this thesis, but this concept could be very beneficial economically. Dimensioning of this concept could be a part of future work.

4.5. Connection of steel plate to column

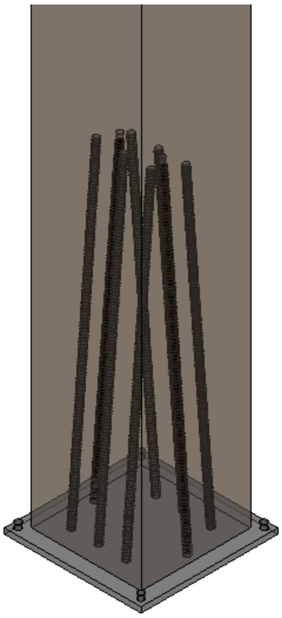


Figure 4.26: Connection of columns to steel plate.

For the connection between the steel plate and the column, there are steel rods welded to the plate, embedded into the column. The number of rods and their diameter will vary considering the moments and forces acting on the column and foundation. For the reference building the worst case scenario is when the columns stand alone, before the decks are mounted. During this time, eight steel rods with diameter of 26 millimeters, and a length of 1200 millimeters is sufficient. The calculations is shown in Appendix A.4. The rods will be installed with an angle of approximately 5-10° to avoid cracking in the timber. The calculations for three rods on two sides were made, but this led to insufficient capacity in the weakest direction. Therefore, the two extra rods were suggested, which eliminates a weak direction. Based on the doctoral thesis by H. Stamatopoulos (Stamatopoulos, 2016) the conclusion were made that the rods will have sufficient withdrawal capacity, due to their large embedment length. Stamatopoulos also gave the advice that the rods' large embedment length and angle ensures that the distances to the edge of the column will not be a problem. 15 millimeters were chosen as safety.

4.6. Rotational stiffness of foundation base

The stiffness of each component of the foundation assembly will have a varying significance on the rotational stiffness for the foundations. The rotational stiffness of the assembly will only be as good as its weakest link, so if the spring stiffness of the ground is too low, it doesn't matter if the rest of the assembly is very stiff. Therefore, investigations were made to find out what rotational stiffness is needed in the foundations. And then how to theoretically obtain this rotational stiffness. The calculations of the different stiffnesses are approximations and simplifications to find realistic ranges for the different stiffness parameters. These parameters were used in the modelling in ABAQUS to acquire values for the rotational stiffness of the foundation base. This will give a reasonable base to move on to the experimental stages to acquire the real rotational stiffness.

In chapter 4.6.1 it is concluded that for the reference building with shafts a high rotational stiffness will not be necessary in the foundations. For other structural solutions, this may not be the case. Therefore different solutions were in the next chapters checked numerically to find out what range of rotational stiffnesses could be achieved.

4.6.1. Necessary stiffness of the foundations

Often in a timber structure the governing criteria will be the serviceability limit state (Malo and Stamatopoulos, 2016). Timber is a very flexible material in comparison to the capacity, and therefore it gives large deformations before the material collapses. Therefore, the serviceability limit state will limit the structure before the ultimate limit state is exceeded. A general rule for buildings like the reference building in this thesis is that the displacement in the top should not exceed H/300. The height of the building is 28 meters, which gives H/300=93,3 millimeters. The displacement in the top of the building is 111,37 millimeters, with a rotational stiffness in the foundation of 3000 [kNm/Rad]. With a rotational stiffness of 11400 [kNm/Rad] in the foundation, the displacement in the top is 98,64 millimeters. For the reasoning behind the rotational stiffnesses in the foundation, see chapter 4.6.2-4.6.7. These displacements are on the reference building with only columns and decks.

Rotational stiffness (kNm/Rad)	3000	11400
Frame direction (mm)	111,37	98,64

Table 4.6: Max displacements for the reference building without shafts for the serviceability limit state, for different rotational stiffnesses in the foundations.

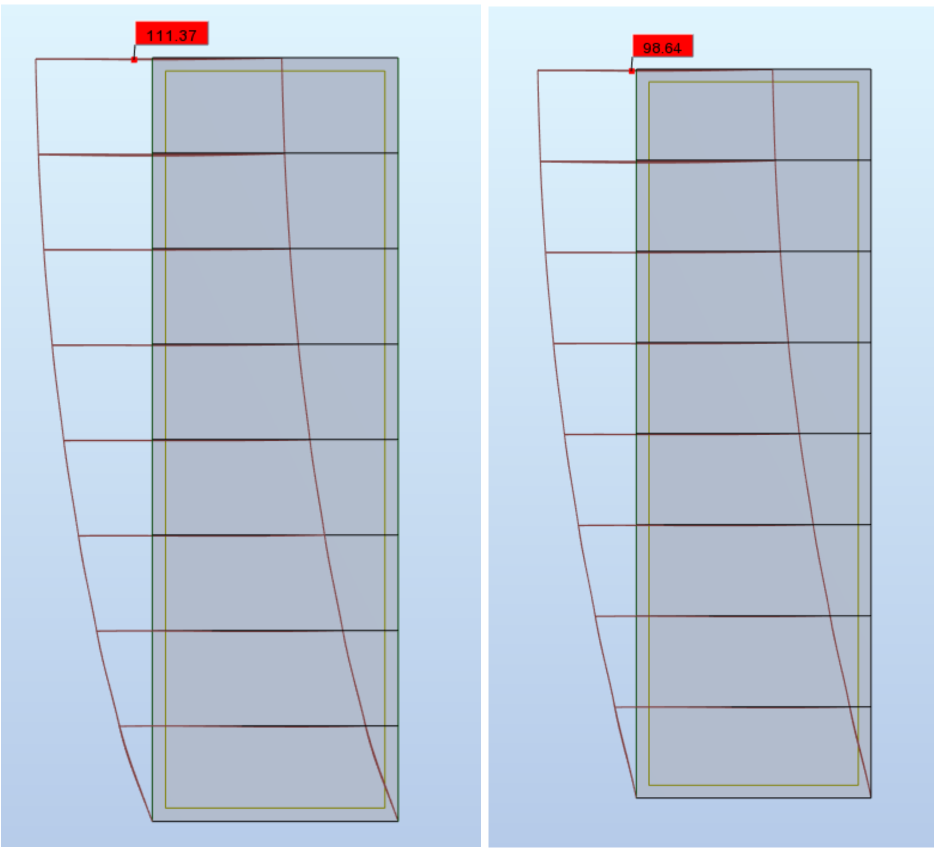


Figure 4.27: Total displacements for the worst-case scenario in the serviceability limit state without shafts

On the left: 3000 kNm/Rad in the foundation. On the right: 11400 kNm/Rad in the foundation.

An elevator shaft is necessary in every type of an urban building for easy access. It is also favorable for handling horizontal forces in the structure, although not able to in every structure.

For a well-planned building it is important to place the shaft in an expedient location, with both accessibility and handling of the forces acting in mind. The elevator shaft in the reference building was modelled with panels with corresponding material properties as a CL28h panel. The thickness was set to the reference thickness of a CLT-panel, which is 150 millimeters. The shaft was made to be 2500x3000 [mm], which is well within the criteria for design with universal accessibility. In addition to the elevator shaft, a staircase is a necessary detail in every building. The shaft for the staircase can, as the elevator shaft, take horizontal forces. With just the elevator shaft, a rotation in the structure will show, due to the difference in stiffness from the shaft to the columns. With a shaft for the staircase located in a different area of the building, the shafts are together able to take the moment caused by the wind loads. They also provide a stiffness for the entire structure, which is crucial for the serviceability of the building.

When an elevator shaft and a shaft for a staircase is connected to the reference building, the displacements are 19,87 millimeters for a rotational stiffness of 11400 [kNm/Rad], and 19,93 millimeters for a rotational stiffness of 3000 [kNm/Rad] (see *Table 4.7*). Directions of the displacements are in accordance to the Robot Structural Analysis-model. As can be seen from *Table 4.7*, the displacements vary very little from 3000 [kNm/Rad] to 11400 [kNm/Rad]. In the longitudinal direction the stiffness remains the same, while it is changed in the frame direction, i.e. along the decks between the two rows of columns. This, because the WoodSol concept is based on moment resisting frames, and the goal is to achieve high stiffness in the frame direction. The displacement in the longitudinal direction of the building will have to be taken by shafts and/or shear walls. This direction is therefore calculated with fixed foundations in Robot Structural Analysis. The displacements in the vertical direction of the model is so small that they are considered negligible.

Rotational stiffness (kNm/Rad)	3000	11400
Longitudinal direction (mm)	7,74	7,73
Frame direction (mm)	19,51	19,45
Total (mm)	19,93	19,87

Table 4.7: Max displacements in the reference building with shafts for the serviceability limit state, for different rotational stiffnesses in the foundation.

The same rotational stiffness as in the bottom of the columns were used for the foundations of the shaft walls. As seen from both *Table 4.6* and *Table 4.7*, the difference in displacements are a lot greater without the shafts, not just by number of millimeters, but also the percentage change. This is because the shafts are a lot stiffer in the stiffest direction than the columns. From the model with shafts, we can see that the displacements are well within the criteria for the serviceability limit state for both 3000 [kNm/Rad] and 11400 [kNm/Rad]. This means that for the reference building, a large rotational stiffness in the foundation of the columns and shaft walls are not necessary. The building is almost within the comfort criteria without shafts or shear walls with a rotational stiffness of 11400 [kNm/Rad] in the foundation. The rotational stiffness of the foundation may be a bigger challenge in other, bigger and more complex structures.

Sources of error

The shafts modelled for the reference building are modelled with simple solutions without detailed calculations. They were mounted on the reference building for comparison, to see how much stiffer a building with shafts will be. The displacements in the reference building with shafts might therefore be inaccurate, but they give an overview of the difference with or without the shafts. A more in depth examination and calculations of the shear walls and shafts in a WoodSol building can be a topic for further work.

4.6.2. Stiffness of soil

As can be found in "Use and abuse of springs to model foundations" (Muccillo, 2014), the elastic foundation modulus of the soil is in the range of 4800 [kN/m^3] for soft sand up to 128000 [kN/m^3] for loose gravel. This is shown in *Table 4.8*.

Type of soil	**K_s (kN/m^3)**
Soft sand	4800-16000
Medium dense sand	9600-80000
Silty medium dense sand	24000-48000
Clayey medium dense sand	32000-80000
Loose gravel	64000-128000

Table 4.8: Elastic foundation modulus for different soils.

The range of the stiffness from soft to dense is very large. The use of 4,8 [MN/m^3] as soil for a building is unrealistic. Either the soil would be changed, or the building would be piled to rock. But for analytical reasons a soil stiffness of 5 [MN/m^3] is used as the lowest value in ABAQUS just to see the effect of denser soil on the rotational stiffness. The limits of the soil elastic foundation modulus will be in the range from 5 [MN/m^3] to 100 [MN/m^3] with medium values of 20 [MN/m^3] representing a medium dense sand and 65 [MN/m^3] representing loose gravel. These four values are used in ABAQUS to see the effect of soil stiffness on the rotational stiffness of the foundation base.

4.6.3. Stiffness of steel anchors

The steel anchors, hereby also referred to as bolts, attach the steel plate to the concrete foundation. These bolts can be assumed theoretically as springs with a calculated stiffness both axial and vertically. In ABAQUS the bolts are modelled as springs between the steel plate and the concrete foundation in order to obtain the final rotational stiffness of the assembly.

Vertical stiffness of bolts

The vertical stiffness of the bolts is calculated by assuming the same displacement as a cantilever beam. The vertical stiffness can then be calculated by:

$$K_v = \frac{12EI}{L^3}$$

Where E is the elasticity modulus for steel and I is the second moment of area for the bolt. The table below shows the vertical stiffness for the same bolts as illustrated in the tables above.

Diameter bolts (mm)	**Area (mm^2)**	**E_{Steel} (MPa)**	**Length of bolt (mm)**	**I (mm^4)**	**K_V (N/mm)**
-	$\Pi \cdot r^2$	-	-	$\frac{\pi * d^4}{64}$	$\frac{12EI}{L^3}$
10	78,5	210000	200	490,9	155
20	314,2	210000	200	7854,0	2470
30	706,9	210000	200	39760,8	12500
40	1256,6	210000	200	125663,7	39600

Table 4.9: Vertical stiffness values for different dowel diameters and 200 millimeter length.

Diameter bolts (mm)	Area (mm²)	E-Steel (MPa)	Length of bolt (mm)	I (mm⁴)	$K_{V.bolt}$ (N/mm)
-	$\Pi \cdot r^2$	-	-	$\frac{\pi * d^4}{64}$	$\frac{12EI}{L^3}$
10	78,5	210000	300	490,9	45,8
20	314,2	210000	300	7854,0	733
30	706,9	210000	300	39760,8	3710
40	1256,6	210000	300	125663,7	11700

Table 4.10: Vertical stiffness values for different dowel diameters and 300 millimeter length.

In *Figure 4.28* the graph shows the vertical stiffness for different lengths and diameters of bolts. The vertical stiffness $K_{v.bolt}$ which is plotted in the Y-direction is the vertical stiffness for each single bolt, and not a group of bolts.

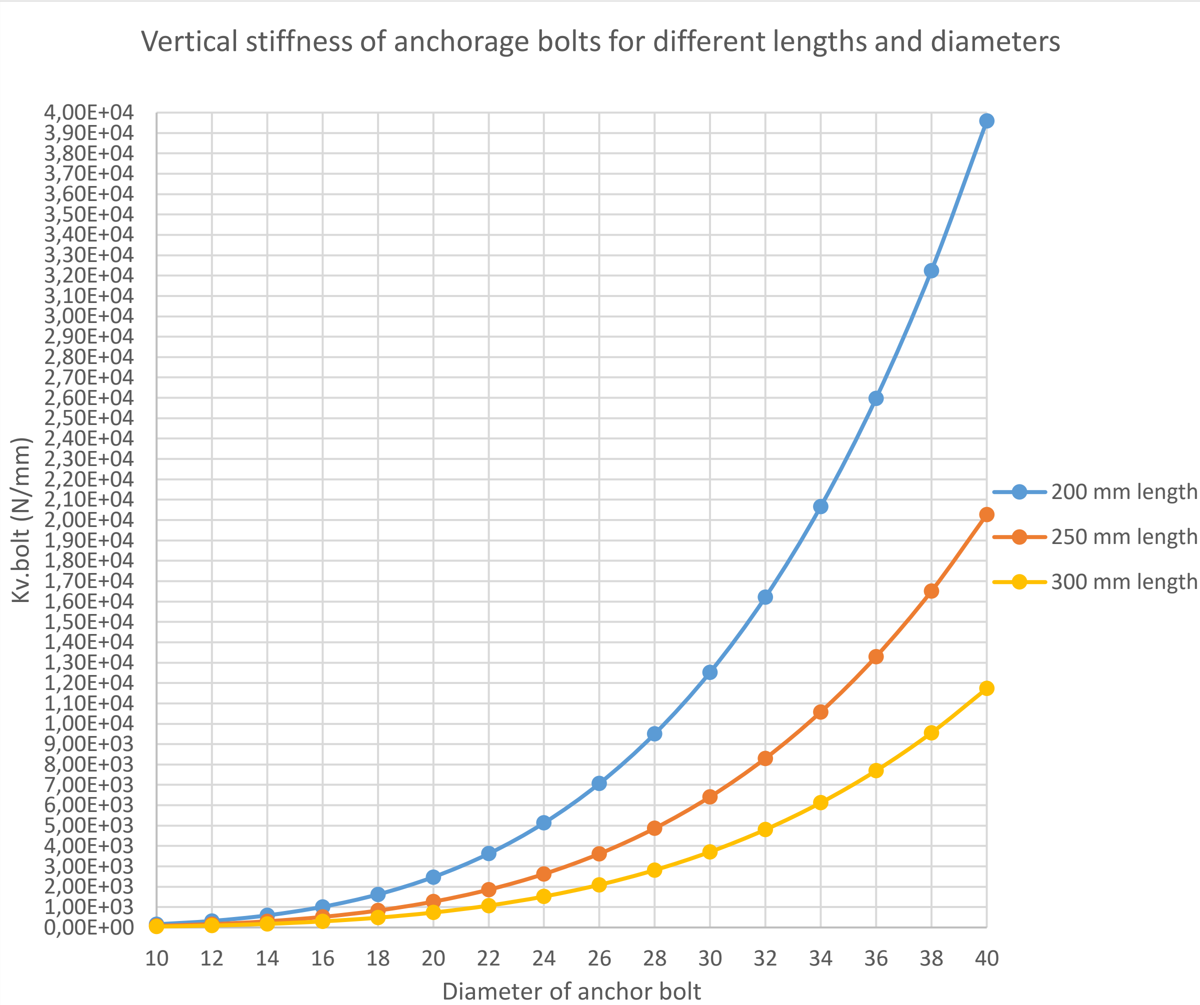


Figure 4.28: Vertical stiffness of anchorage bolts for different lengths and diameters.

The stiffness values for all dimensions from 10 mm to 40 mm can be found exact in Appendix A.5.

Axial stiffness of bolts

The axial spring stiffness $K_{ax.bolt}$ can be calculated as shown in the formula below:

$$K_{ax.bolt} = \frac{A * E}{L_{Bolt}}$$

Where the elasticity modulus is 210 [GPa] for steel and the length of the dowel is in the range of 200 – 300 millimeter. This is a simplification which does not consider the possibility of movement due to creep and shrinkage in the concrete. The shrinkage and creep could have an impact on the stiffness, but the approximations should appropriate.

Diameter steel bolts (mm)	**Area (mm^2)**	**E_{Steel} (Mpa)**	**Length of bolt (mm)**	**$K_{ax.bolt}$ (N/mm)**
-	$\Pi \bullet r^2$	-	-	$\frac{A * E}{L}$
10	78,5	210000	200	82500
20	314,2	210000	200	330000
30	706,9	210000	200	742000
40	1256,6	210000	200	1320000

Table 4.11: Axial stiffness values for different bolt diameters and 200 millimeter length.

When the length of the bolts is extended, the axial stiffness will be reduced, which is because the length is the divider in the formula for axial stiffness. As illustrated in *Table 4.10*, the axial stiffness is lower for the same diameters as shown in *Table 4.9*.

Diameter steel bolts (mm)	**Area (mm^2)**	**E_{Steel} (Mpa)**	**Length of bolt (mm)**	**$K_{ax.bolt}$ (N/mm)**
-	$\Pi \bullet r^2$	-	-	$\frac{A * E}{L}$
10	78,5	210000	300	55000
20	314,2	210000	300	220000
30	706,9	210000	300	495000
40	1256,6	210000	300	880000

Table 4.12: Axial stiffness values for different dowel diameters and 300 millimeter length.

In *Figure 4.29* the graph shows the axial stiffness for different lengths and diameters of bolts. The axial stiffness $K_{ax.bolt}$, which is plotted in the Y-direction is the axial stiffness for each bolt, and not for a group of bolts.

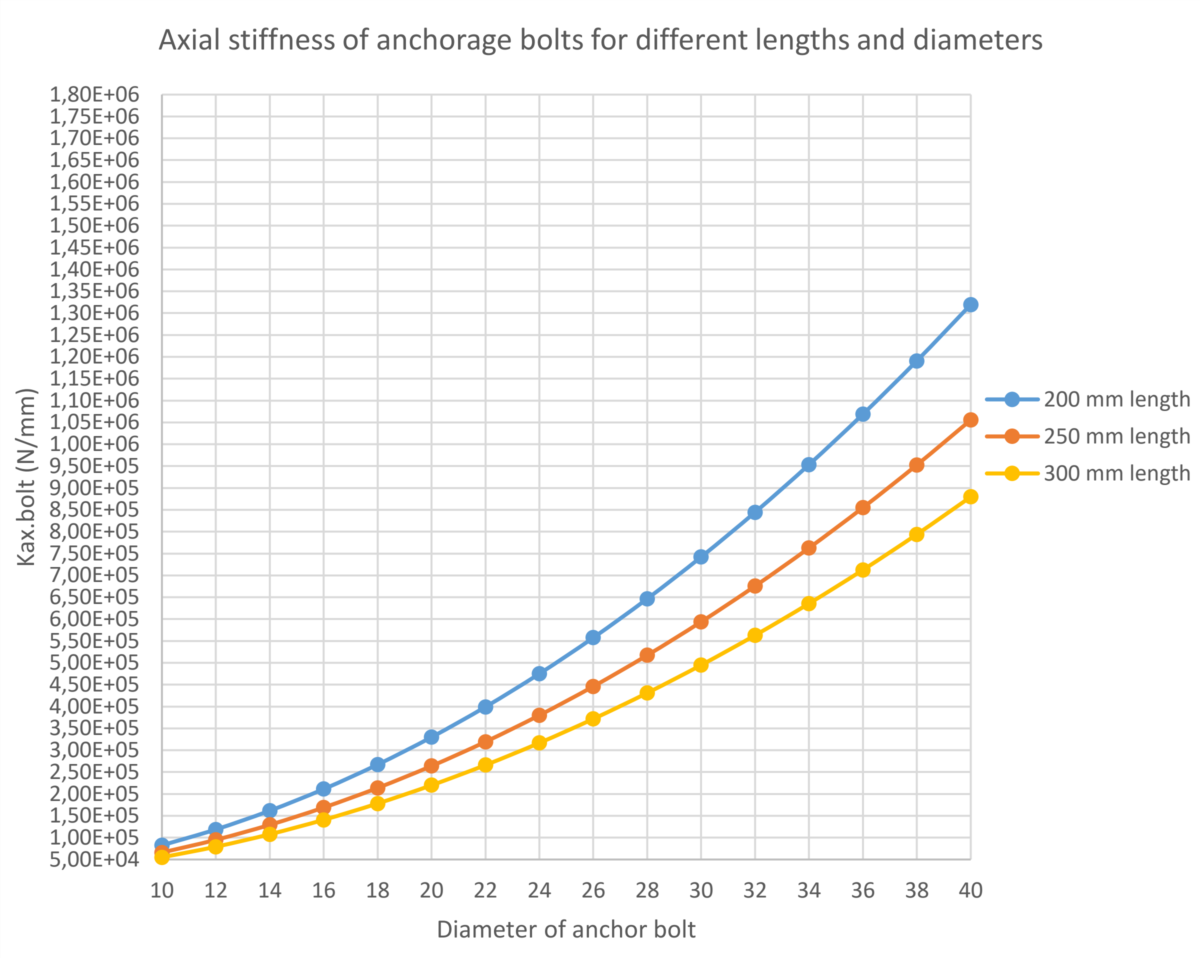


Figure 4.29: Axial stiffness of anchorage bolts for different lengths and diameters.

Sources of error

The bolts could get additional movement both axially and vertically because of the creep and shrinkage in the concrete. This is not accounted for. Additional to creep and shrinkage in the concrete, the nuts used to adjust the steel plate could have movement, mainly axially. These factors would be minimal, but could have an influence on the results.

4.6.4. Rods connecting steel plates to columns

Axial stiffness of rods

The axial stiffness calculated for the steel rods attaching the column to the steel plate $K_{ax.rod}$ is the so-called withdrawal stiffness of the steel rods. This stiffness is calculated by a simplified expression (A.16) found in the doctoral thesis of Haris Stamatopoulos (Stamatopoulos, 2016). The expression used is:

$$K_{ax.rod} = 0.85 * \sqrt{\pi * d * R_e * A * E}$$

Where R_e is a factor taking the angle of the rods into account calculated by the formula:

$$R_e = \frac{9{,}65}{(1{,}5Sin(\alpha)^{2{,}2} + Cos(\alpha)^{2{,}2})}$$

The rods have been decided to be installed inclined with a degree between 5-10° to avoid splitting in the columns. The deriving and basis for these formulas can be found in the thesis by Stamatopoulos (Stamatopoulos, 2016).

Additional to the withdrawal stiffness, the rods also have a free length from the top of the plate to the bottom of the plate. The rods are welded to the bottom of the steel plates as illustrated in *Figure 4.30*. The free length also has a stiffness which behaves like a tensile rod which is fixed in one end and free to move elsewhere. The stiffness is calculated as:

$$K_{ax.0} = \frac{A_{steel} * E_{steel}}{L_0}$$

Where L_0 is the free length illustrated in *Figure 4.30*.

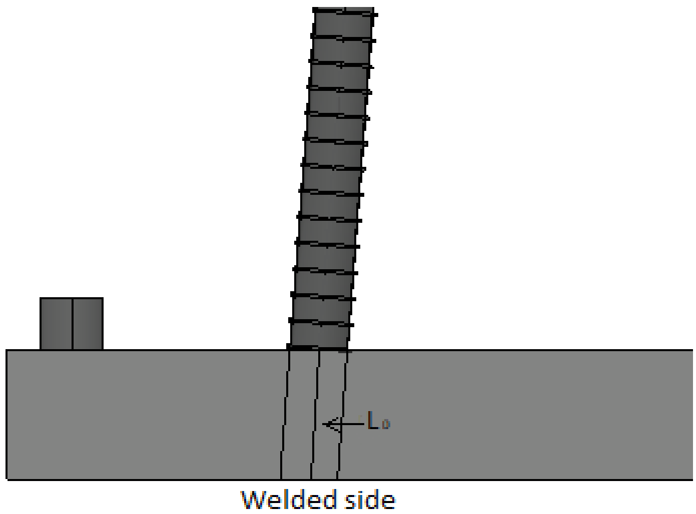


Figure 4.30: Illustration of free length L_0 and welded side.

The stiffness used in the ABAQUS models is the combination of the stiffness of the rod and the free length, combined by the formula:

$$K_{ax.tot} = \frac{K_{ax.rod} * K_{ax.0}}{K_{ax.rod} + K_{ax.0}}$$

This stiffness is a little lower when the stiffness of the free length is not considered. This can be seen by comparing $K_{ax.rod}$ and $K_{ax.tot}$ in *Table 4.13*, where $K_{ax.tot}$ considers the free length and $K_{ax.rod}$ does not.

Diameter rods (mm)	**Area (mm)**	**E_{Steel} (Mpa)**	**L_0 (mm)**	**R_e**	**$K_{ax.rod}$ (N/mm)**	**$K_{ax.0}$ (N/mm)**	**$K_{ax.tot}$ (N/mm)**
-	$\Pi \cdot r^2$	-	$\frac{50}{\sin(80)}$	See formula on previous page	$0{,}85 \cdot \Pi \cdot d \cdot R_e \cdot A \cdot E$	$\frac{A * E}{L}$	$\frac{K_{ax.rod} * K_{ax.0}}{K_{ax.rod} + K_{ax.0}}$
10	78,5	210000	51	9,66	60100	471000	53300
16	201,1	210000	51	9,66	122000	828000	106000
22	380,1	210000	51	9,66	196000	1570000	174000
28	615,8	210000	51	9,66	282000	2540000	254000

Table 4.13: Axial stiffness of steel rods for varying diameter and 50 mm steel plate.

The axial stiffness is calculated with 5° and 10° inclination and have been plotted for different dimensions in *Figure 4.31*. The inclination has no impact on the axial stiffness as can be seen in *Figure 4.31* since the two lines coincide. The stiffness $K_{ax.rod}$ is the stiffness of each single rod, and not of a group of rods and will be correct for rod-lengths from 650 millimeters and above.

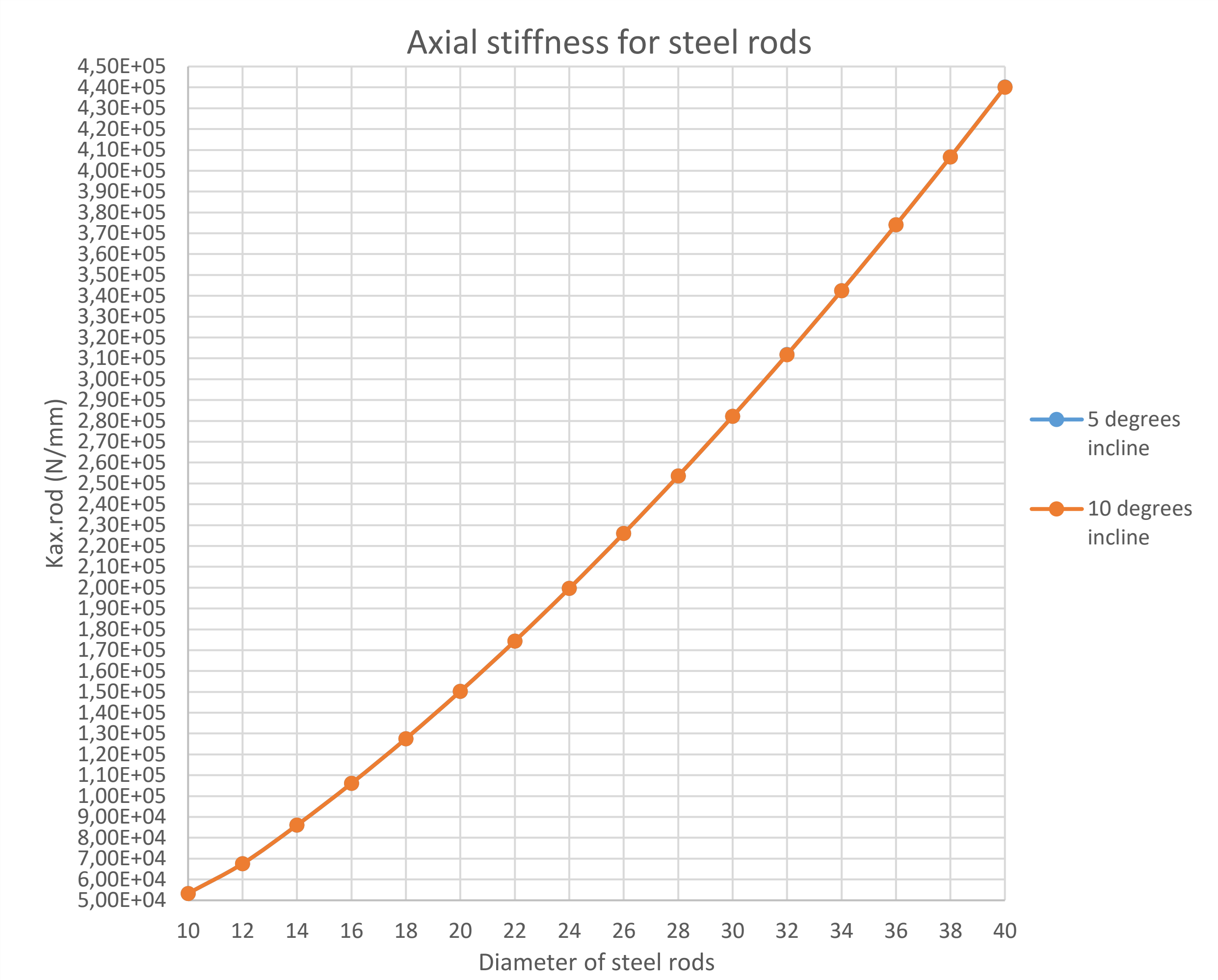


Figure 4.31: Axial stiffness for different diameter of steel rods.

Vertical stiffness of rods

The vertical stiffness is also calculated both for the free length and the steel rod penetrating the wood. The free length is assumed to have the same displacement as a cantilever beam in the vertical direction, having a vertical stiffness of:

$$K_{v.0} = \frac{3EI}{L_0^3}$$

The rods penetrating the wood will act like steel dowels in the vertical direction, therefore the stiffness can be calculated using Formula 7.1 in Eurocode 5 (EC5) (CEN, 2008b).

$$K_{v.rod} = \frac{2 * p_m^{1,5} * d}{23}$$

Where the formula is multiplied by a factor of 2 because of EC5 7.1 (2) (CEN, 2008b) where it is considered a steel to timber connection.

Diameter rods (mm)	Area (mm)	E_{Steel} (Mpa)	L_0 (mm)	$K_{v.rod}$ (N/mm)	$K_{v.0}$ (N/mm)	$K_{v.tot}$ (N/mm)
-	$\Pi \bullet r^2$	-	$\frac{50}{\sin(80)}$	$\frac{2 * p_m^{1,5} * d}{23}$	$\frac{3EI}{L_0^3}$	$\frac{K_{v.rod} * K_{v.0}}{K_{v.rod} + K_{v.0}}$
10	78,5	210000	51	7750	2330	1790
16	201,1	210000	51	12400	15300	6850
22	380,1	210000	51	17100	54600	13000
28	615,8	210000	51	21700	143000	18900

Table 4.14: Vertical stiffness of steel rods with free length.

The vertical stiffness is also calculated with 5° and 10° incline, which gives no impact on the vertical stiffness as can be seen in *Figure 4.32* since they coincide. The stiffness $K_{v.rod}$ is the stiffness each single rod, and not of a group of rods and will be correct for rod-lengths from 650 millimeters and above.

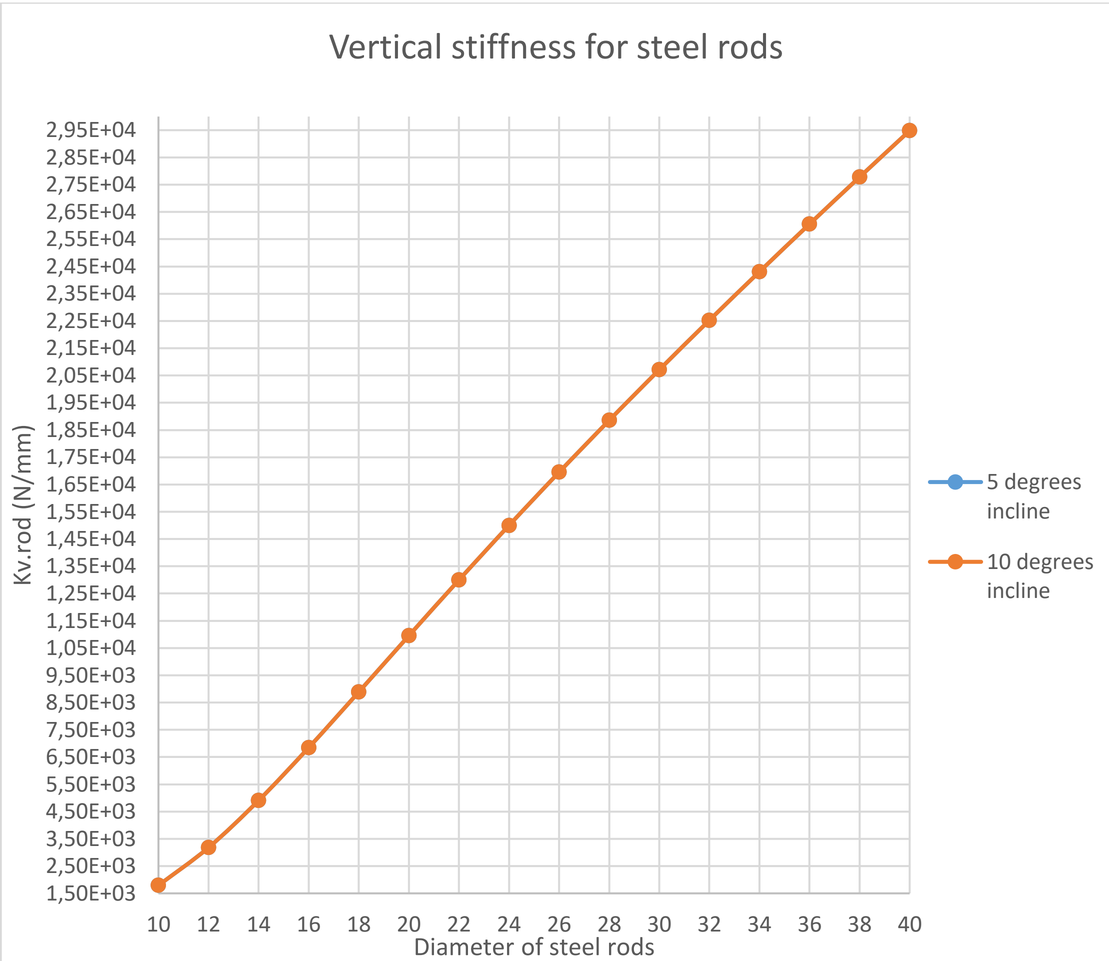


Figure 4.32: Vertical stiffness for steel rods.

Sources of error

The formulas used for calculating the axial stiffness of the rods are simplified calculations. These could be calculated using the full-scale calculations shown in the thesis by Stamatopoulos (Stamatopoulos, 2016) which could give more accurate results.

4.6.5. Modelling of the reference model in ABAQUS

Dimensions of elements

First a reference model was modelled in ABAQUS. This, to have a rotational stiffness value to compare to the values when changes are made to the assembly. This reference model is also the model used to check the results when varying different parameters. In this chapter, the choices of the modelling, simplifications and the sizes and values used will be presented, as well as the results.

The first part of the modelling is choosing the sizes for each of the three parts. The three parts are: foundation, steel plate and column. The foundation is modelled with the dimensions 2000x600x500 [mm]. Then partition cells and datum points are made in specific places, so that the steel plate will be placed correctly and the dowels can be attached at the correct points.

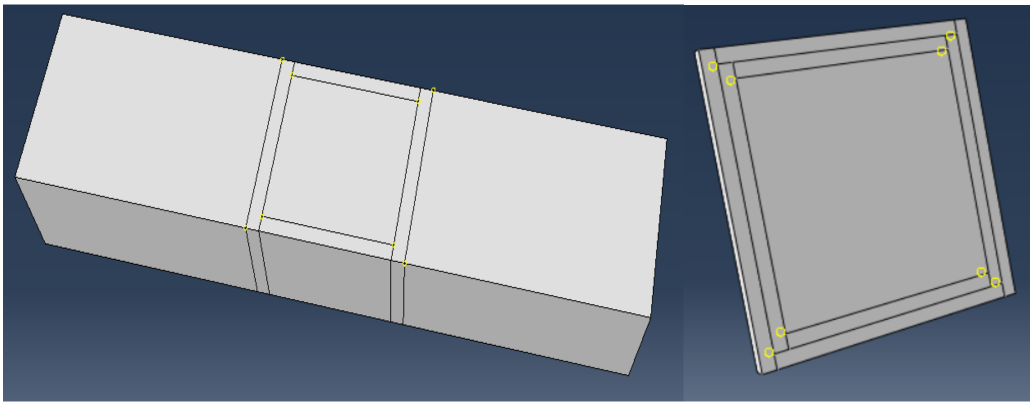


Figure 4.33: Foundation and steel plate modelled in ABAQUS.

The steel plate has the dimensions 600x600x20 [mm]. Because of the demand for edge distances and spacing according to EC3-1-8 table 3.3 (CEN, 2005), the steel plate will need to be about 600x600 [mm] for dowels between 20-40 millimeters. This gives room between the nuts and the column to tighten the nuts.

The column is modelled with the dimensions 400x400x10000 [mm]. It is modelled as rigid since it is only the rotation of the foundation, not the curvature of the column that is relevant.

The steel plate and the concrete foundations is modelled with the elasticity modulus of 210 [GPa] for steel and 30 [GPa] for concrete.

Modelling of springs

The next parameters for the reference model is the bolts and the rods. The bolts and the rods are modelled as four springs each and then assigned connector sections. Since it is only modelled four springs, the stiffness value of the rods and bolts, if for example eight bolts are used, must be divided by four and then assigned to the four modelled springs representing the bolts and rods. For eight bolts with a diameter of 32 millimeters the stiffness value would be 844000 [N/mm] for each bolt. This results in a total stiffness of 8 x 844000 [N/mm] which equals to 6752000 [N/mm] total, which again will be divided by the four springs modelled, resulting in 1688000 [N/mm]. The same is done for the vertical stiffness resulting in a stiffness value of 32400 N/mm. The values are plotted into the Edit Connector Section as illustrated in *Figure 4.34*.

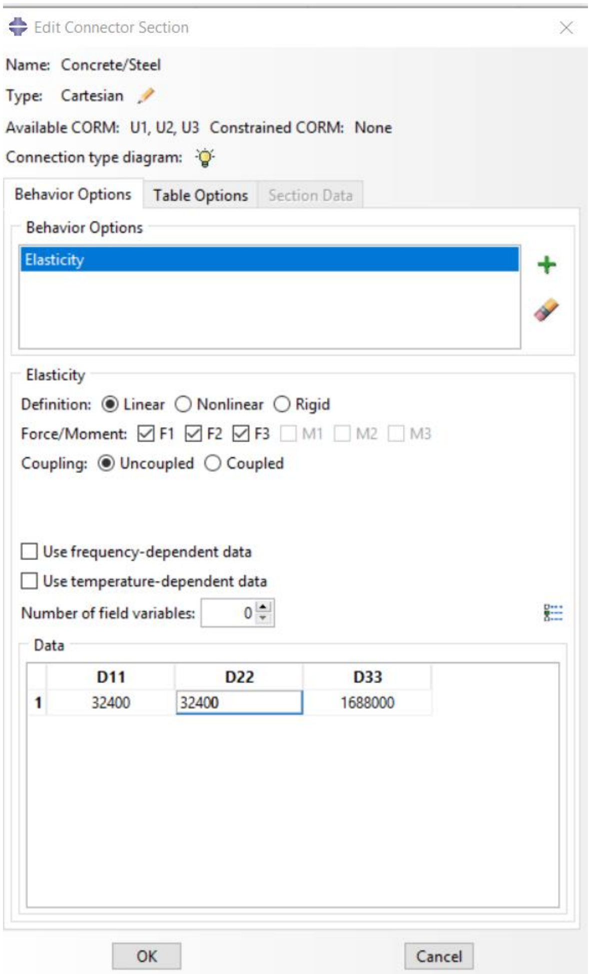


Figure 4.34: Values plotted into the Edit connector section in ABAQUS.

For the steel rods, the same procedure is done as for the bolts. The only difference is that the springs for the rods are modelled with an inclination of 10°. For the reference model six rods with a diameter of 22 millimeters is used, resulting in an axial stiffness of 261000 [N/mm] and a vertical stiffness of 19500 [N/mm] being plotted in the connector section for the springs attaching the column to the steel plate.

Modelling of soil

When modelling the soil as springs in ABAQUS, the value of each spring depends on the mesh size used. In the reference model, the mesh size is cubic with the dimension 50x50x50 [mm]. With this mesh and a soil stiffness for the reference model of 20 [MN/m^3] the spring stiffness for each node-spring will be:

$$K_{soil} = 20 * 10^{-3} \frac{N}{mm^3} * 50\ mm * 50\ mm = 50 \frac{N}{mm}$$

This value is plotted as a spring attaching the nodes beneath the foundation to the ground, as illustrated in *Figure 4.35*.

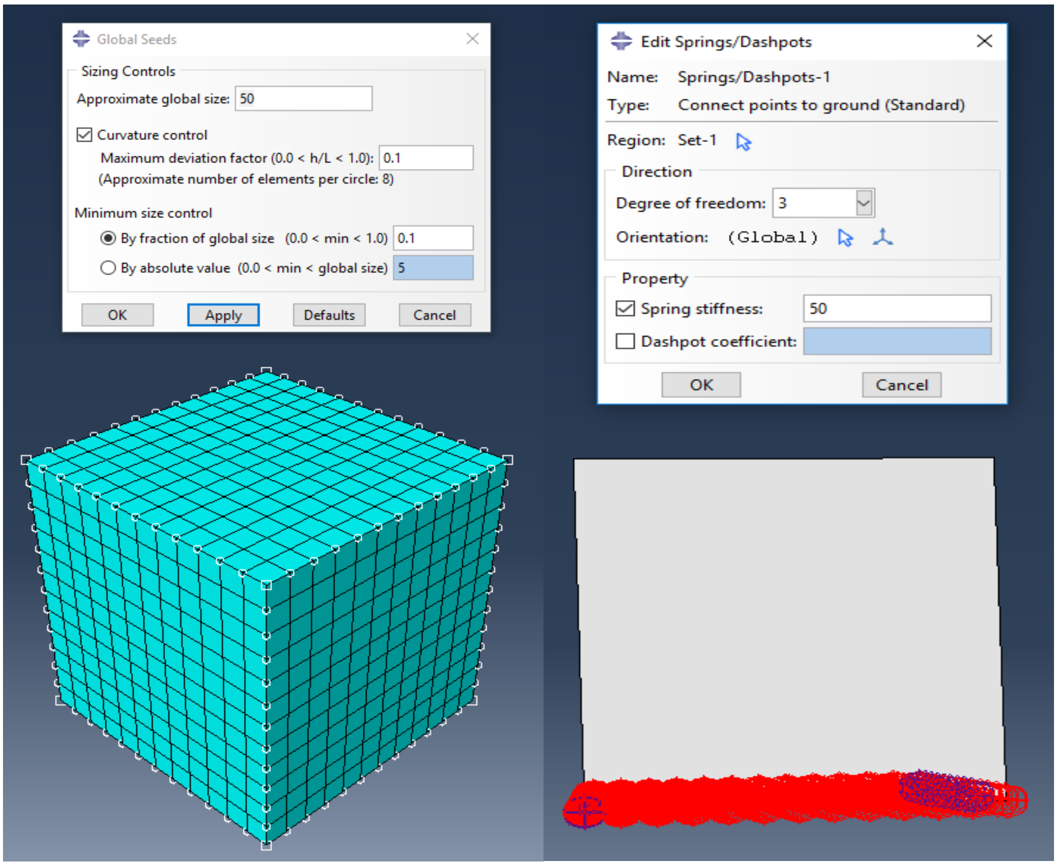


Figure 4.35: Illustration of modelling of mesh size and soil springs in ABAQUS.

Forces and restraints

The model is applied a force of 1 [kN] at the top of the column to achieve a moment which can be used to calculate the rotational stiffness. The foundation is restrained in the X and Y direction, only allowing it to rotate and be pushed down into the springs modelled for the soil. The top of the column is restrained in the Y direction, not allowing it to move out of the X-Z plane. Both the force and the restrains can be seen in *Figure 4.36*.

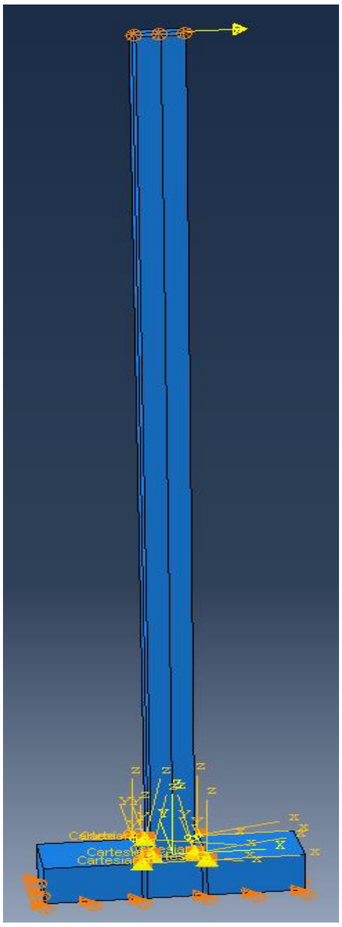


Figure 4.36: ABAQUS model with forces and restrains.

Sources of error

The first source of error in the modelling is the dimension of the foundation. This is just an assumption, which means both the length and width could have a larger necessary size. If the foundation is found to be larger it would result in a larger area connected to the soil, and therefore resulting in a stiffer foundation and a higher rotational stiffness, in other words, the assumption of size is conservative.

The next source of error is the modelling of the springs. It would be more correct to model eight springs if eight rods or bolts are used. The authors of this thesis found that to be too time consuming and therefore chose to divide the stiffnesses of the bolts and rods to four springs thinking it would be an adequate assumption.

4.6.6. Final rotational stiffness of foundations

The final rotational stiffness is calculated by dividing the moment in the foundation by the rotation of the foundation:

$$K_\theta = \frac{Moment\ (kNm)}{Rotation\ (Rad)}$$

The moment is easily calculated to 10,5 [kNm] by multiplying the force of 1 [kN] at the top of the column multiplied by the height of the assembly of 10,5 meters. While the rotation is calculated by formula below, using the displacement in the top of the column:

$$Rotation = Arctan(\frac{Displacement\ (mm)}{10500\ (mm)})$$

Reference model	Sizes and numbers
Soil stiffness (kN/m^3)	20
Plate thickness (mm)	20
Diameter dowel (mm)	32
Length of dowel (mm)	200
Number of dowels	8
Diameter rods (mm)	22
Number of rods	6
Top displacement (mm)	33,24
Rotation (Rad)	0,00317
Rotational stiffness (kNm/Rad)	3315,80

Table 4.15: Parameters and stiffness of reference model.

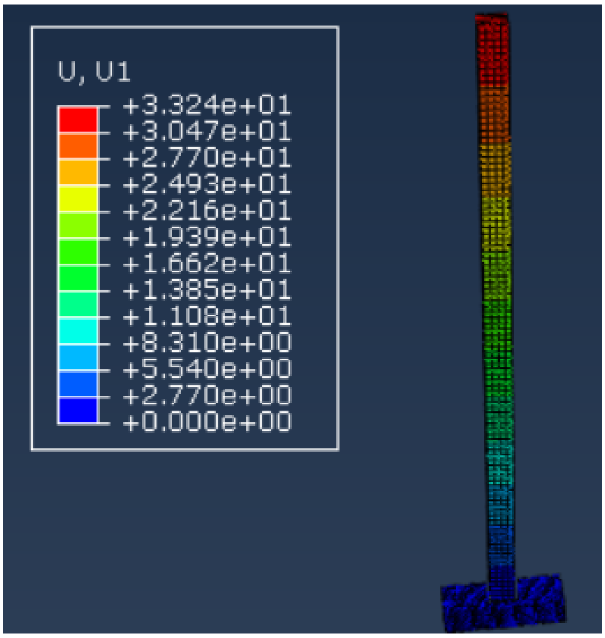


Figure 4.37: Displacement at the top of the column for the reference model.

By having a reference model, each parameter can now be varied, making it possible to figure out which parameters have the biggest influence on the rotational stiffness of the foundation base. Different combinations can also be made trying to achieve the highest possible rotational stiffness. This has been done in chapter 4.6.7 and the results have been plotted into graphs, comparing the results to the stiffness of the reference model.

4.6.7. Effect of different parameters

Effect of the soil stiffness

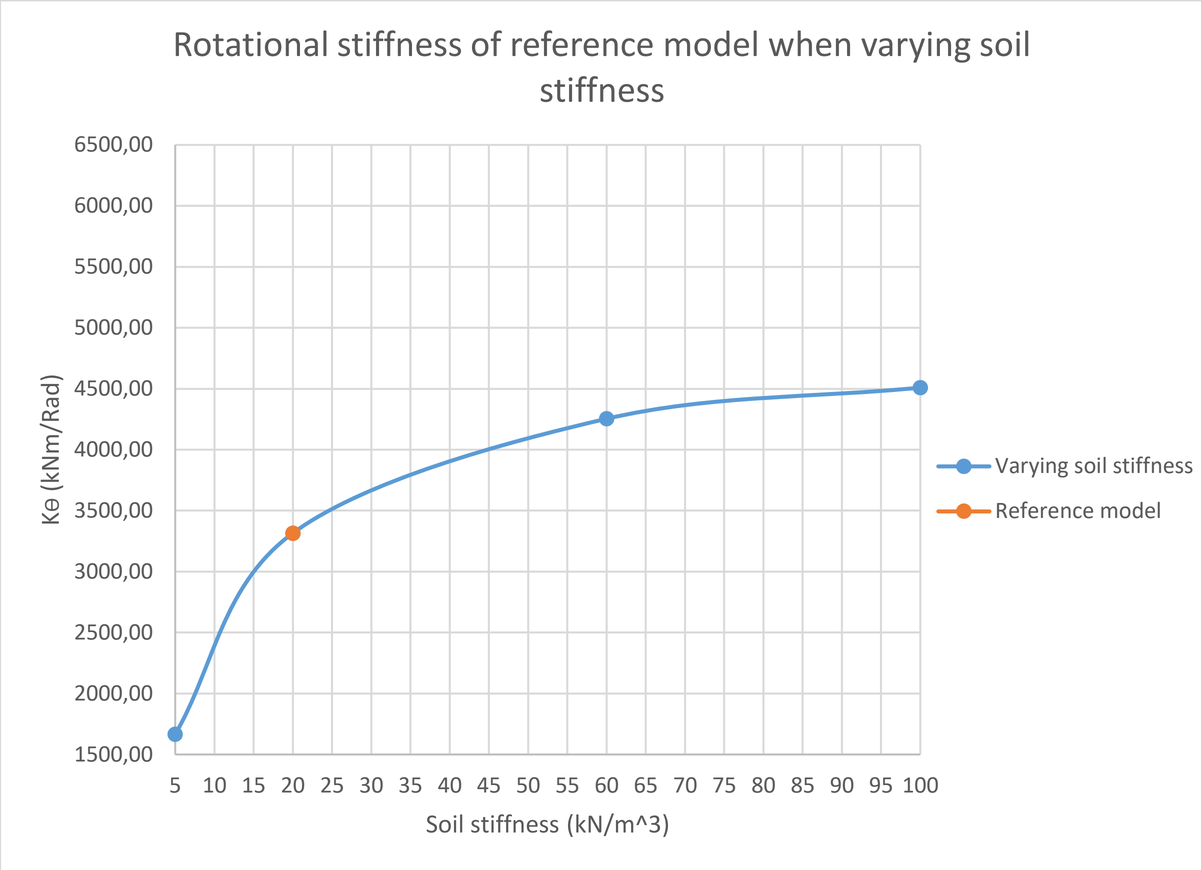


Figure 4.38: Effect of varying soil stiffness.

Figure 4.38 shows the impact of the soil stiffness on the total rotational stiffness for the foundation base. All parameters are as for the reference model in *Table 4.15*, but the soil stiffness is varied from 5 [kN/m^3] to 100 [kN/m^3]. The graph shows that the soil stiffness has a high impact on the total rotational stiffness of the foundation. This implies that to achieve a high rotational stiffness the soil needs to be stiff, even if all other parameters are maximized.

When reaching about 60 [kN/m^3] the graph flattens. By increasing the soil stiffness from 60 [kN/m^3] to 100 [kN/m^3] the change in rotational stiffness is 250 [kN/m^3] and using soil stiffness above 100 [kN/m^3] have almost no impact on the rotational stiffness. By using 200 [kN/m^3] instead of 100 [kN/m^3] the rotational stiffness only increases by about 100 [kNm/Rad].

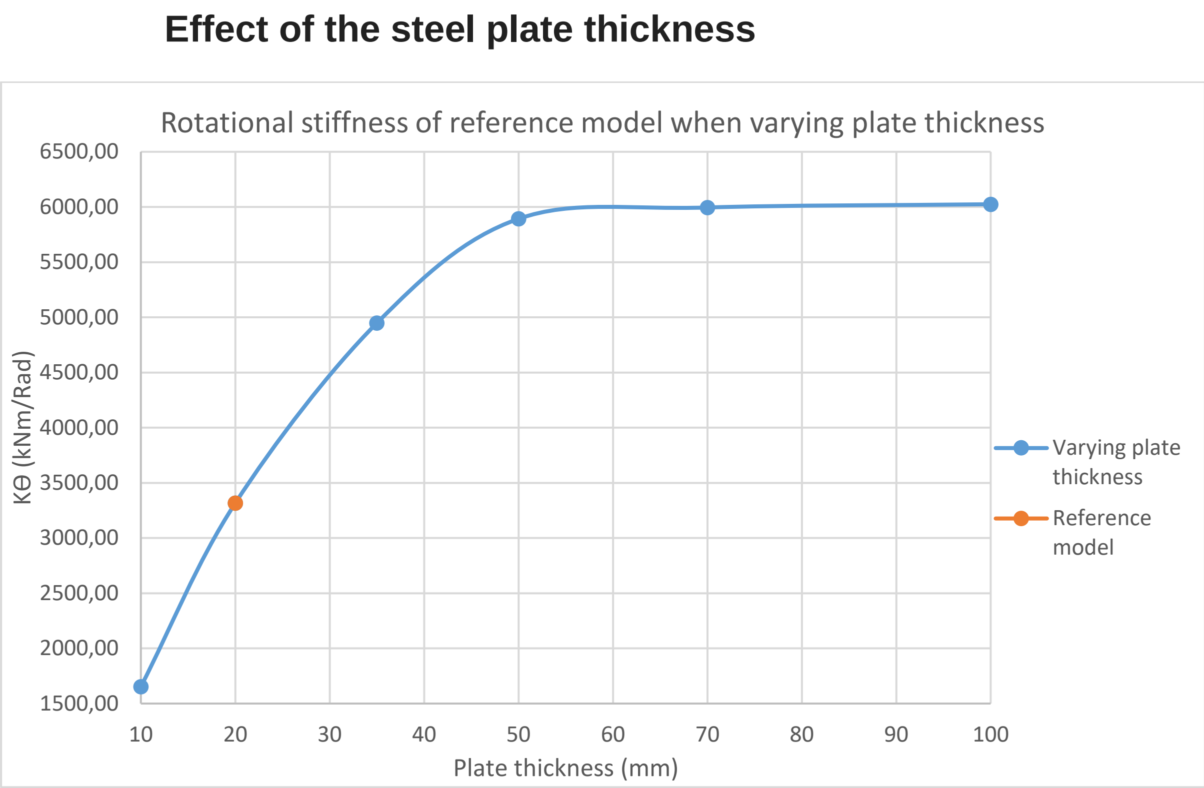


Figure 4.39: Effect of varying plate thickness.

The influence the thickness of the steel plate has on the rotational stiffness is interestingly high. The graph shows high increase of rotational stiffness when increasing the thickness up to about 50 millimeters. Increasing the thickness above 50 millimeters will have almost zero impact on the rotational stiffness.

The thickness of the steel plate has an even greater impact on the rotational stiffness than the impact of the soil stiffness.

Effect of the number and sizes of bolts

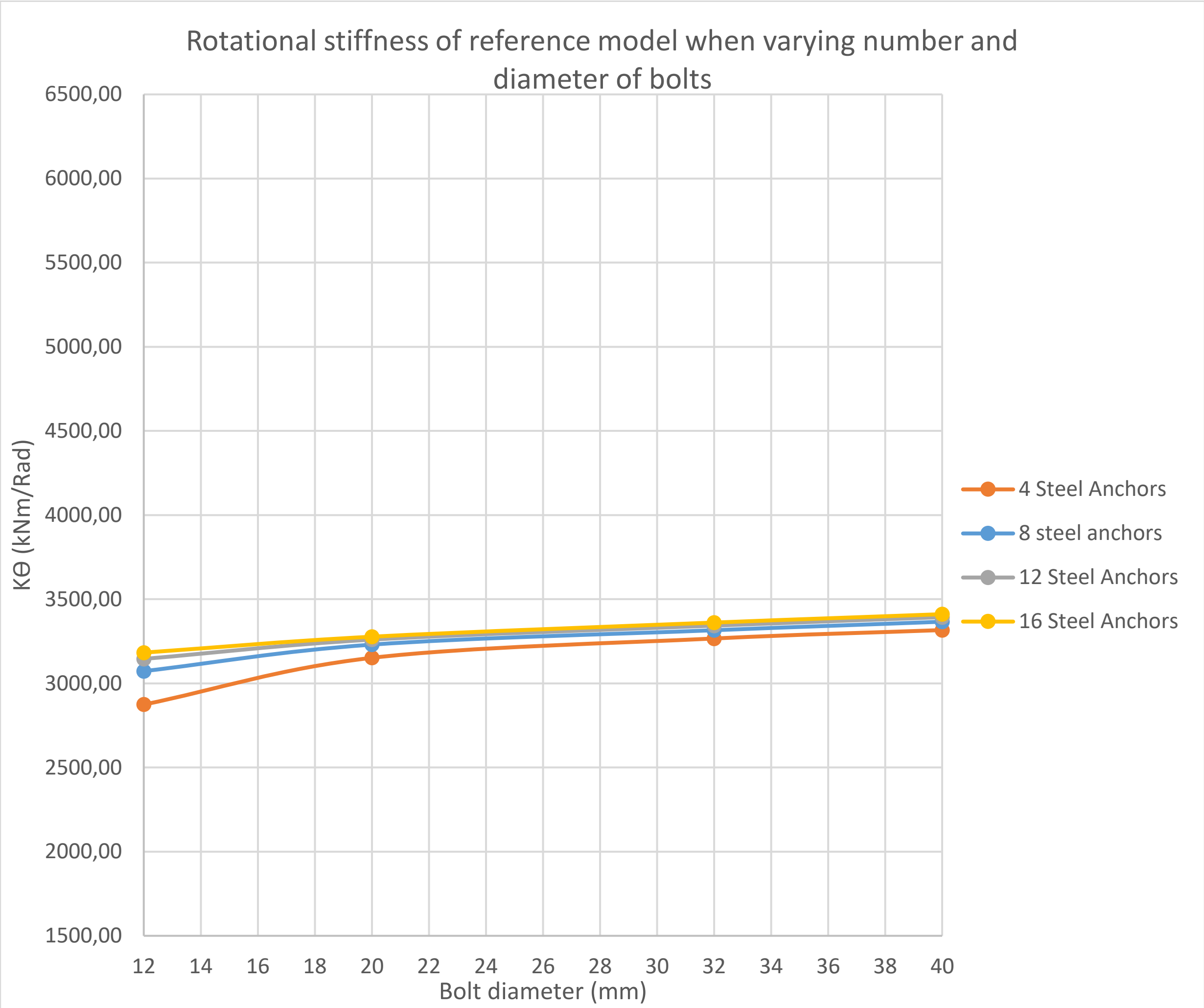


Figure 4.40: Effect of varying diameter and number of bolts.

Figure 4.40 shows the impact on the rotational stiffness of the foundation base by the diameter of the bolts, and the number of bolts used to connect the steel plate to the foundation. As can be seen, both the diameter and the number of bolts have a very small influence on the total rotational stiffness. The difference between using four 12 millimeters bolts and sixteen 40 millimeters bolts is only 340 [kNm/Rad], which is very low.

With this graph it is concluded that the use of eight bolts with a diameter of 20 millimeters, which is relatively small, is stiff enough to be able to contribute to a high rotational stiffness. The bolts will still need to be dimensioned to be able to withstand the forces acting on the connection, but increasing the diameter above 20 millimeters will not contribute significantly to a stiffer foundation.

Effect of the number and diameter of rods

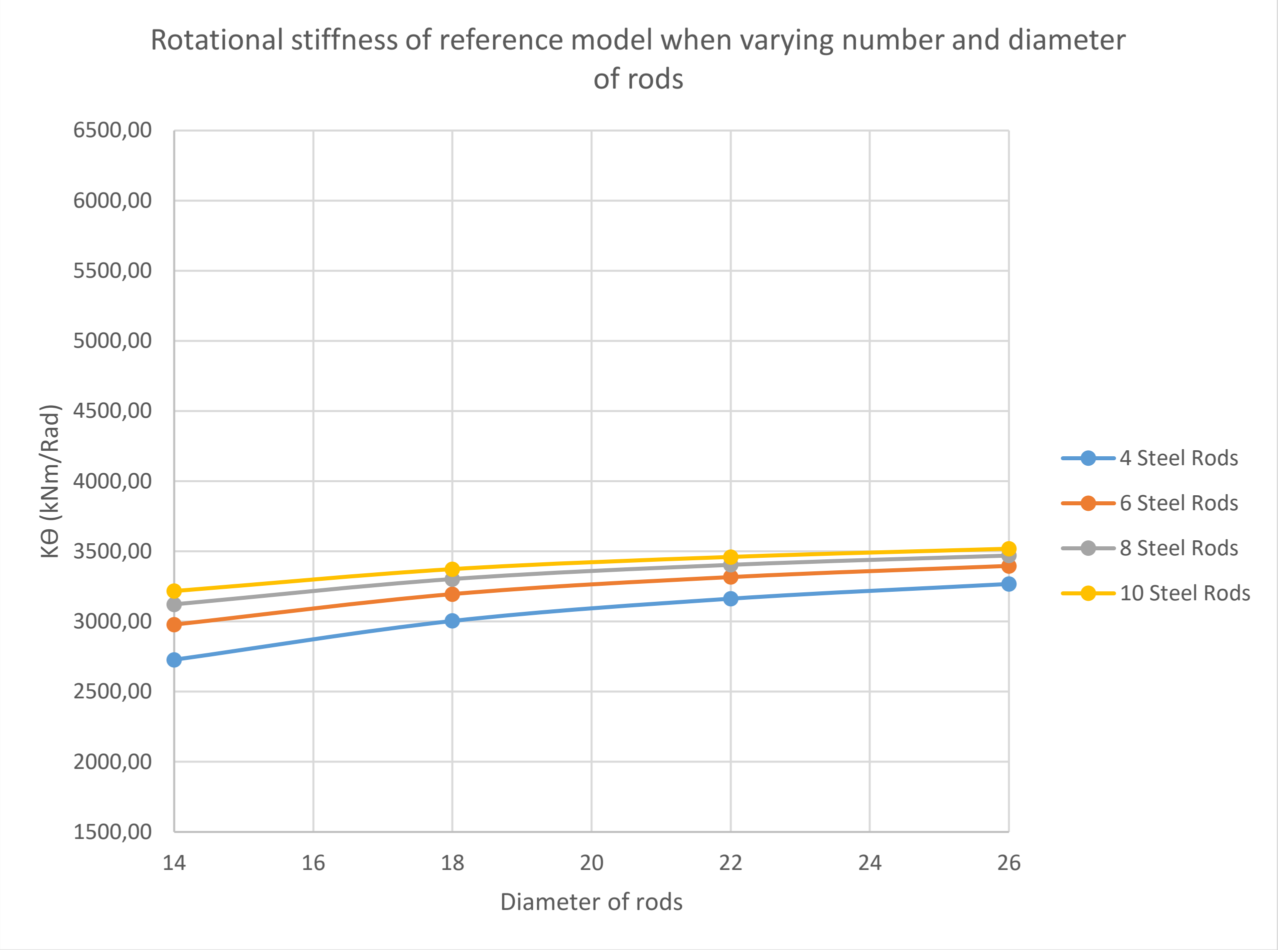


Figure 4.41: Effect of varying the diameter of the rods.

Figure 4.41 shows the effect of varying the diameter and number of steel rods on the rotational stiffness. The graph shows that the difference between using ten 26 millimeter steel rods and four 14 millimeter steel rods is about 800 [kNm/Rad] which is a considerable impact. But the difference between using six or eight rods instead of ten is rather low.

When trying to maximize the rotational stiffness for the foundation base the number and sizes of rods is significant. The diameter should be considered to be between 18 and 26 millimeters and the number of rods should be six or above, if the rotational stiffness is required to be high.

Maximum obtainable rotational stiffness

By analyzing and comparing the influence of the different parameters in this chapter, new numerical tests have been made in ABAQUS. This, to try to find the maximum rotational stiffness achievable using realistic assumptions for the different parameters. The parameters used for the different solutions A, B, C, D and E can be seen in *Table 4.16* and *Figure 4.42* with the final rotational stiffness of each solution.

Maximal expected stiffness	**A**	**B**	**C**	**D**	**E**
Soil stiffness (kN/m^3)	20	20	40	65	100
Plate thickness (mm)	40	40	45	50	100
Diameter bolt (mm)	22	32	32	32	32
Number of bolts	8	8	8	8	8
Length of bolts (mm)	200	200	200	200	200
Diameter rods (mm)	22	24	24	24	24
Number of rods	8	8	8	8	8
Rotational stiffness (kNm/Rad)	6142	6377	9351	11069	11460

Table 4.16: Parameters for final rotational stiffness.

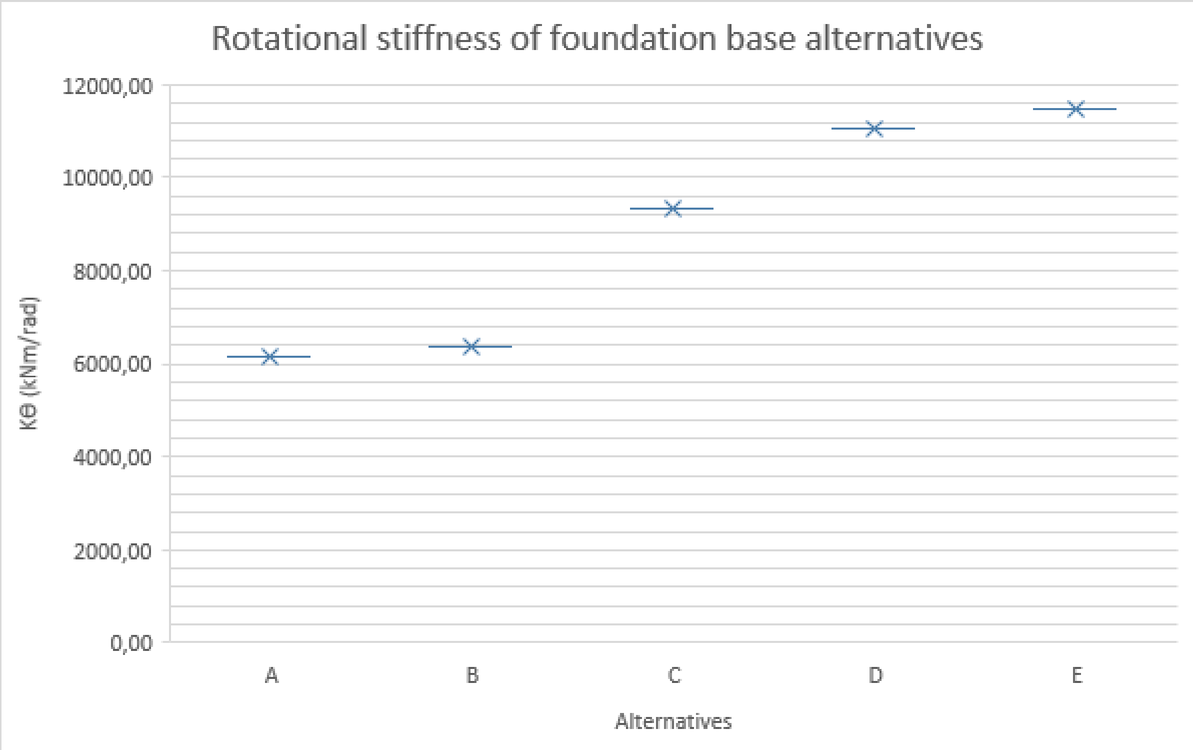


Figure 4.42: Rotational stiffness of foundation base alternatives.

From *Table 4.16* and *Figure 4.42* we can see that the rotational stiffness for the foundation base with realistic assumptions can reach up to 11460 [kNm/Rad].

From solution A to B it is shown that only increasing the diameter of the rods and the bolts have an insignificant impact on the rotational stiffness. From B to C you see the influence of increasing the plate thickness and using denser soil, and as can be seen the influence is very large. From B to C there is a slight increase in both plate thickness and soil stiffness, resulting in an even higher rotational stiffness than for alternative C. While from D to E the plate thickness is increased above 50 millimeters and the soil thickness above 65 [kN/m^3] which have only a small influence on the rotational stiffness.

From this chapter it is concluded that a rotational stiffness of 11460 [kNm/Rad] is obtainable for the foundation base. The factors having the largest impact on the rotational stiffness is the thickness of the steel plate and the soil stiffness. The rods and anchorage bolts have an impact, but the impact is nowhere near the impact of the plate or the soil.

The next step for the rotational stiffness would be to do experiments to check if these numerical results are achievable in practice.

4.7. Modelling of decks

The modelling of the decks is done in Robot as a simplification based on the deck described in the thesis by Klund, Skovdahl and Torp (Klund et al., 2017) and the thesis by Bjørge and Kristoffersen (Bjørge and Kristoffersen, 2017) (henceforth referred to as the reference deck). As concluded by Bjørge and Kristoffersen, the decks will have a self-weight of approximately 200 [kg/m^2], when they are made to meet the requirements for acoustics. This gives the decks a total weight of 4320 kg. The decks are modelled to give a realistic representation of the reactions from wind, snow, self-weight and live loads on the structures in Robot. Therefore, the weight of the decks are important. Also, the weight has a big influence on the acoustic properties of the decks, but this will not be investigated further in this thesis. The decks are modelled as solid decks for simplification, with the same material properties as the reference deck. The elasticity module is 15000 [MPa] in the lateral direction, and 300 [MPa] in the transversal direction.

WoodSol is developed for high-rise buildings in urban areas. A typical use for the buildings can be offices, apartments, schools, gyms etc. With this in mind, the live load that is selected for the structure is 4,0 [kN/m^2]. This load covers the categories of use NS-1991-1-1 (6.1.1) (CEN, 2008b). The snow load chosen for the structure is 3,5 [kN/m^2], which is the decided load for downtown Trondheim found in NS-EN 1991-1-3, NA.4.1 (901) (CEN, 2008b).

The FE simulations made by H. Stamatopoulos and K. A. Malo (Malo and Stamatopoulos, 2016) gave the minimum rotational stiffness of the connections between the decks and the columns of 10 000 - 11 000 [kNm/Rad]. This, to satisfy the criteria of a maximum horizontal displacement in the top of the building of H/300. The height of the building used in the simulations were 30 meters. The decks modelled in Robot for this thesis are therefore given a rotational stiffness in the connections to the columns of 11 000 [kNm/Rad].

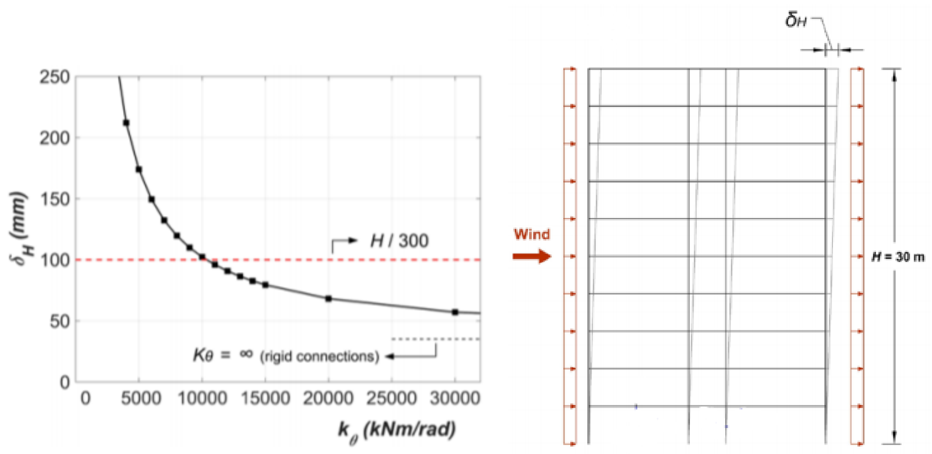


Figure 4.43: Calculations of rotational stiffness in connections to satisfy SLS requirements.

The wind load acting on the structure will vary with regards to were the building is placed. In the simulations by Stamatopoulos and Malo, a structure in an urban environment was evaluated. This will also be the case for WoodSol, when the concept is meant for buildings in an urban environment.

4.8. Stability of columns during erection

The concept of WoodSol is based on moment resisting frames. During the erection of the structure, the columns may have to stand alone for a short period of time, thus not being moment resisting before the structure is complete. This may cause problems, since the stiffness will be considerably lower when the decks are not present. Here the only forces acting is gravity and wind. In this chapter the process of assembly will be looked at. This will reveal whether or not a bracing during the erection is necessary.

According to NS-EN 1991-1-5 table NA.4(901.1) (CEN, 2008c) the dimensioning wind load in Trondheim is 26 [m/s]. This complies with a force of approximately 422 Pa. This multiplied with the exposure factor C_e from NS-EN 1991-1-4 4.5 (4.8) results in a wind speed force q_p equal to 1056 Pa. The calculations are shown in Appendix A.6.

The use of a wind speed of 26 [m/s] is very conservative for the construction phases. Due to the lifting process by crane the wind speed during the construction phase should be considerably lower, due to both effectiveness and safety. The columns are checked with NS-EN 1995-1-1 (CEN, 2008b) and all calculations for stability and design checks are shown in Appendix A.6.

When both the dead loads and wind loads were combined on the columns, the safety factors from Eurocode 0 (CEN, 2008a) is used, which gives a conservative result. The factors used for the combination of the forces are found in NS-EN 1990-1-1 NA.6.4.3.2 (CEN, 2008a). These calculation gives a displacement in the top of the column of over 8 meters. The moment in the bottom of the columns are 259 [kNm], the shear force is 18,5 [kN], and the compression force is 23,1 [kN]. For calculations see Appendix A.6.

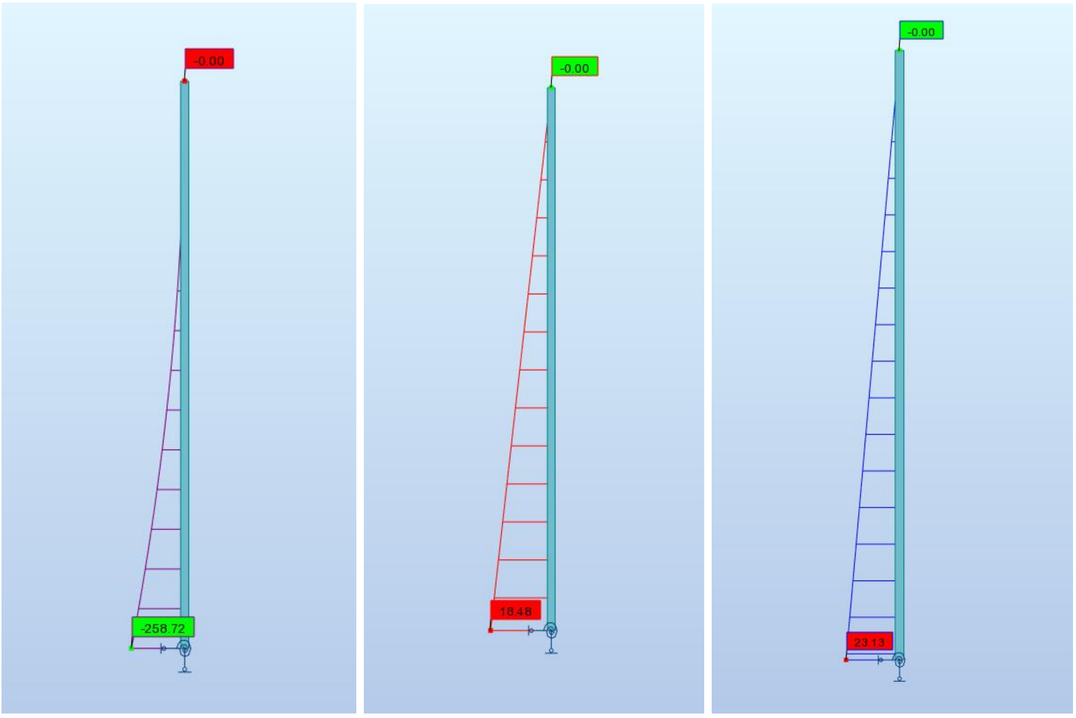


Figure 4.44:

a) Moments from wind load. b) Shear force from wind load. c) Compression force from dead load.

This means that the columns will not be able to stand by them self in the worst-case scenario, which is with wind speeds up to 26 [m/s]. As mentioned earlier, lifting of the columns in wind speeds up to 26 [m/s] is not favorable considering both the preciseness and safety.

For the reference building the columns needed is calculated in Appendix A.6. The dimensions 400x400 [mm] was sufficient for the columns when the wind speed is beneath 26 [m/s] as well as for the building after assembly.

The maximum wind speed calculated to be resisted by the columns standing alone is 25 [m/s], see Appendix A.6. This means that the columns can stand by themselves if the wind is expected to be low until the decks are fastened, but for safety reasons the decks should be fastened as fast as possible after the erection of the columns.

5. Transportational aspects

The transport of elements, be it concrete, steel or timber, is a crucial part of the building process. Transportation of the elements to the building site have an impact on the environmental and the economic aspect of the project. The largest dimensions that can be transported in Norway depends on the roads that needs to be used. The passing of cross roads, roundabouts and the radius of the turns will influence the size of the allowed dimensions.

5.1. Transport of reinforcement and formwork

For the buildings built with the WoodSol concept there will not be need for much reinforcement. But the foundations will need a certain amount depending on the foundation size. For the reference building built on loose gravel, there will be need for about 1,22 m^3 of reinforcement if assuming 2% of the 61 m^3 of concrete is reinforced. The density of steel is 7850 kg/m^3, resulting in a total need of 9577 kg of reinforcement. This can be transported by one delivery with a semi-truck (YNDTransportAS, 2018). If a tower crane is used additional concrete and reinforcement is needed for the foundation of the crane.

The materials for the formwork will also only need one delivery since it will need less than 2 m^3 of formwork panels and other timber materials. This volume is taken from the AutoCAD drawing shown in *Figure 5.1* assuming 20 millimeter thick and 500 millimeter tall formwork panels. The delivery of formwork will be the first delivery of material on site after all the groundworks are finished.

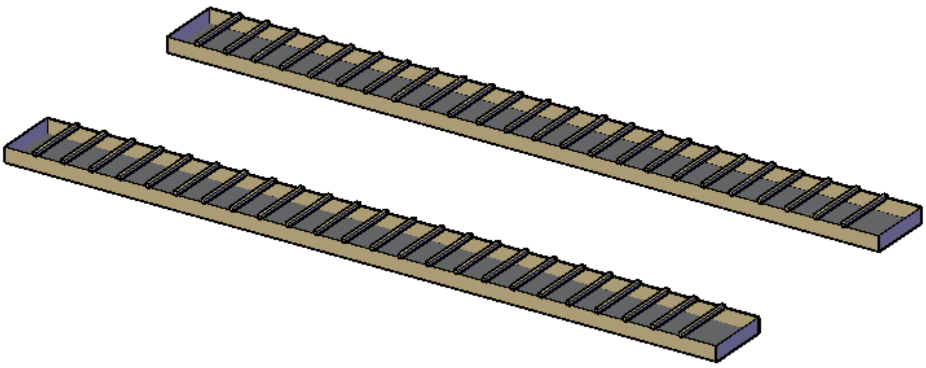


Figure 5.1: Formwork for strip foundations.

If assumed that the building is built in Trondheim the reinforcement can be bought from Celsa, which provides the most CO_2 efficient reinforcement in Europe (Celsa, 2018). Celsa also has a storage of reinforcement in Trondheim providing a short delivery distance for the reinforcement as well. The cost of this delivery by a semi-truck will be 1025 [NOK/hour] by using the prices from YND transport AS in Norway (YNDTransportAS, 2018). While materials can be bought from Optimera, which also provides transportation of the materials.

5.2. Transport of concrete

The concrete for the foundations will be bought from the nearest ready-mixed concrete factory. There is a factory near every major, and a lot of the minor cities in Norway, resulting in a short transportation process for the concrete.

The delivery of concrete on site will come after the formworks are finished with the reinforcement installed.

The amount of concrete for the foundations for the reference building built on loose gravel is 61 m^3. With every concrete truck delivering 7,5 m^3 which is an obtainable delivery volume if there is no overly steep roads to the delivery spot, this will result in 8 deliveries.

Betong Øst, one of the main suppliers of ready-mix concrete in Norway takes 142 [NOK/m^3] for transporting concrete 5 km (BetongØst, 2018).

5.3. Transport of decks

The largest WoodSol deck elements have the dimensions 9000x2400x550 [mm]. Thus, the transportation of these elements is within the rules concerning size for a semi-truck. The rules for transport varies, depending on what kind of vehicle you use, but the dimensions are approximately 18000x2600x4000 [mm] for an extendable semi-truck. This means that the WoodSol-deck elements are within the boundaries (vegvesen, 2017).

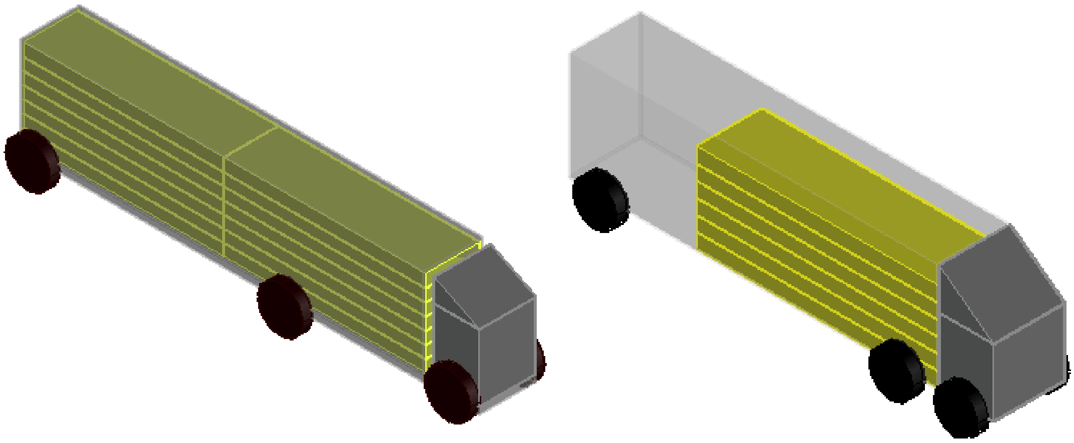


Figure 5.2: Transportation of decks

a) Not allowed. *b) Allowed.*

Unfortunately, the dimensions of 18000x2600x4000 [mm] is only allowed for solid coherent parts using near the full length of the loading plane. Therefore, the decks cannot be transported with two stacks as illustrated in *Figure 5.2 a)*. The decks will have to be transported on normal semi-trucks with a length of the loading plane of 13,6 meters stacking 6 elements on only one stack as illustrated in *Figure 5.2 b)*. The loading is limited by 6 elements, since the maximal weight allowed is 27 tons and each deck weights approximately 4,3 tons resulting in 25,8 tons. This means that for the reference building all the decks could be transported with 14 truck deliveries as concluded by the chief of transportation at Moelven, see Appendix B.1.

The cost of transporting the decks on a semi-truck will be 1025 [NOK/hour] by using YND transport AS in Norway (YNDTransportAS, 2018).

5.4. Transport of columns

To transport elements larger than 18000x2600x4000 [mm] an application have to be sent to Statens Vegvesen in Norway to get special dispensation for the specified cargo. This will need to be done when transporting the columns because of their length of 28000 millimeters. The columns will have to be transported on an extendable semi-truck which is capable of transporting elements above 30000 millimeters with a weight of 27 tons. Hence, an extendable semi-truck is capable of transporting 14 columns at once, resulting in two trucks being necessary for the reference building. Dispensation for additional loading beyond the 27 tons can only be given when the truck is loaded with one single heavy element, and will not be possible for the transportation of the columns.

Since the total length of the transport will be over 30000 millimeters with a loading plane of 28000 millimeters there will be need for two accompanying cars and a police escort. This transport will also need to be done during the night (vegvesen, 2017). The maximal length of a member transported for Moelven Glulam in Norway is 33 meters (RingerriketBlad, 2008).

Figure 5.3: Extendable semi-truck loaded with a 33 meter long glulam element (RingerriketBlad, 2008).

6. Assembly aspects

6.1. Cranes

Due to a high level of prefabrication of elements, there will be need for a crane of some sort at the construction site. In Norway there is three types of cranes that is normally used and easily available. These three are normal tower cranes, self-erecting tower cranes and mobile cranes. The choice for which crane to use will depend on the size and height of the building, building time and the surroundings of the construction site. For very rapid construction the mobile crane may have the upper hand, due to the cost of the erecting of the tower cranes, and the foundations needed. The concrete foundations needed for the tower cranes will also inflict the building time, cost and the CO_2 pollution. The mobile crane may need a greater ground area than the tower cranes, and the lifting process is a little less efficient than the process with the towers.

For the reference building, which has a foot print of 216 m^2, a tall mobile crane LTM 1100-4.1 (Liebherr, 2018) delivered from Roar Wilhelmsen should be sufficient. This mobile crane have a cost of approximately 1450 [NOK/hour] (NorskPrisbok, 2018).

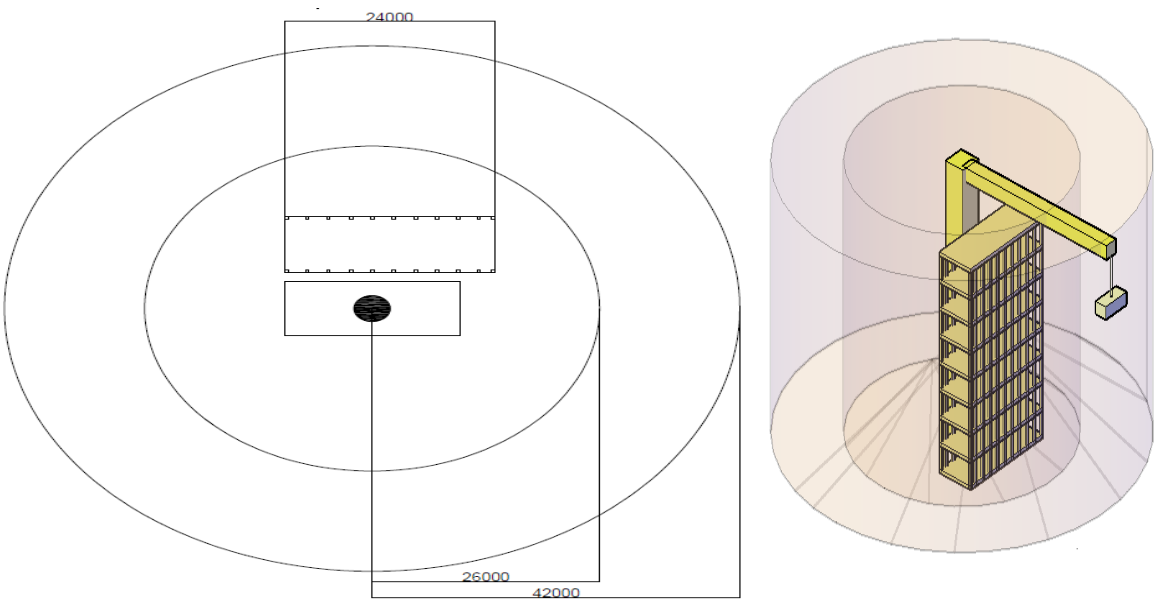


Figure 6.1: Setup of crane.

This mobile crane will give a lifting capacity of 2 tons with a radius of 42 meters and a height of 27 meters and 6,2 tons with a radius of 26 meters and a height of 44 meters. As illustrated above this should be sufficient for the reference building if there is free space around the construction site.

If the construction is to be built in a tight urban area a tower crane might be necessary due to lack of space. For the reference building a Turmdrehkran 132 EC-H tower crane delivered by E.D.Knutsen should be sufficient, having a lifting height of 55 meters and a capability of 3,3 tons at a 40-meter radius. The tower crane needs a foundation of 5000x5000 [mm] while the mobile crane needs 13000x7500 [mm]. This means the tower crane needs ¼ of the ground area compared to the mobile crane. A self-erecting tower crane could also be a good solution. The Potain Igo T 130 delivered by Kranor has almost the same lifting capacity as the Tormdrehkran 132 EC-H, but has a faster and cheaper erecting phase, see chapter 8.2.

6.2. Lifting of columns

The columns need to be checked for stability under the construction phases. The first check is for the self-weight when lifting them from the truck to attach them to the foundations. In this thesis, four different attachment points are calculated, one attachment in the middle of the column, one with an eye bolt screwed in at the top, one with a drilled hole and one with a shackle fastened in the preexisting connectors.

6.2.1. Webbing slings

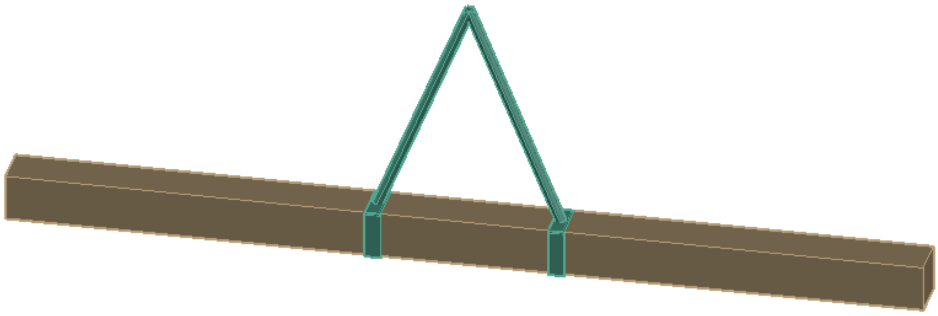


Figure 6.2: Webbing slings attached to the middle of the column.

This solution involves fastening two webbing slings to the middle of the column for a stable and rapid lift to move the columns for temporarily storage if needed until the final lift to connect them to the foundation. The columns will withstand this type of lift easily. The calculations is shown in Appendix A.7. This placement of the slings will not be practical when the columns is to be fastened to the foundations since placing them vertically with these anchor points would be difficult.

6.2.2. Eyebolt

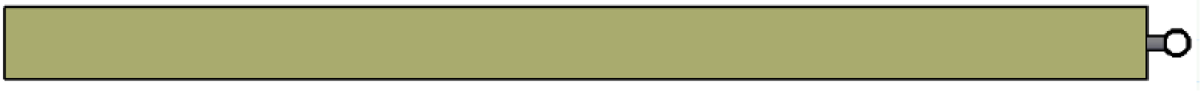


Figure 6.3: Column with screwed in eyebolt.

One idea was connecting the webbing slings to an eyebolt screwed down in the top of the columns. The conclusion was made that the length of the threaded part will probably be unpractically long, and therefore the screw may collide with the screws for the deck connectors. Also, the eyebolt is useless after the lifting process, resulting in an unnecessary usage of expensive, pollutant steel and a more complex prefabrication of the columns.

6.2.3. Drilled hole

Figure 6.4: Column with drilled hole.

The third solution calculated is a solution with a drilled hole with a diameter of 60 millimeters, placed about 500 millimeters from the top of the column depending on the screws from the deck connections. A hole in itself is not optimal for the cross section, but being placed at the top of the column, where the least forces are working, makes it a doable idea. The connection can withstand being lifted by ropes, which is calculated in Appendix A.7, but it's still a suboptimal solution.

6.2.4. Use of shackle in connectors

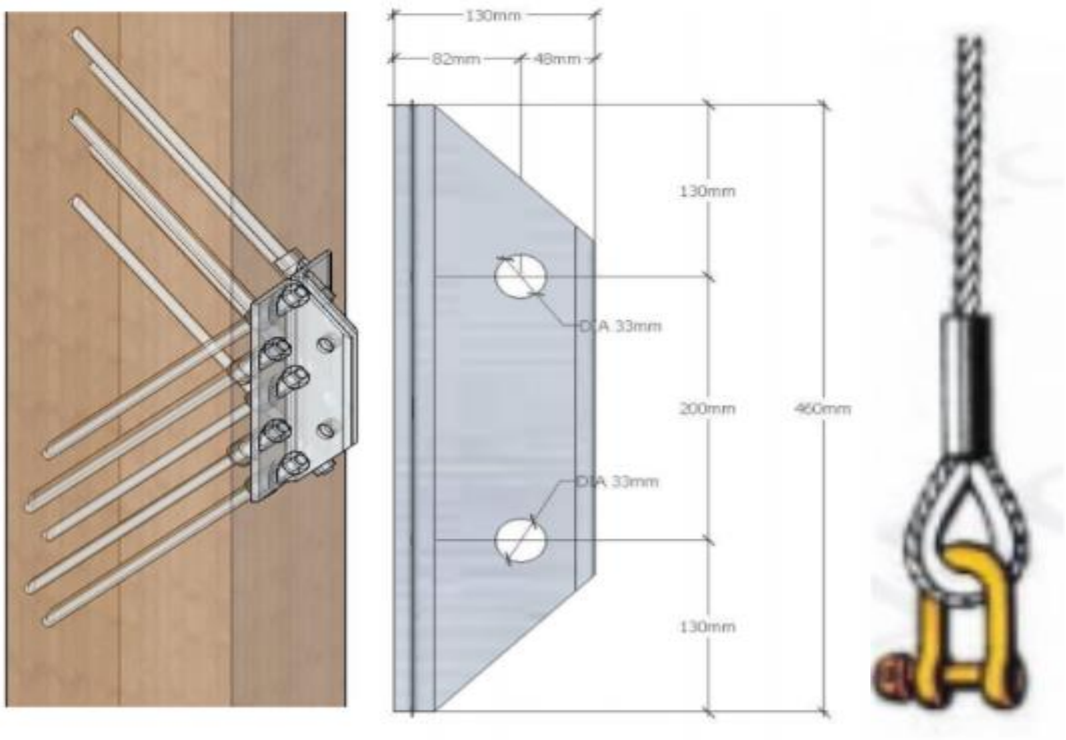


Figure 6.5:

a) Connections in columns (WoodSol, 2016). b) Connector (WoodSol, 2016). c) Shackle (Liftingsafety, 2018).

The best solution when lifting the columns is to use a shackle attached to the holes in the preexisting connectors in the columns. The shackle will need a capacity of about three tons which is easily obtainable while the connector is already dimensioned with a capacity of five tons. This solution will make it possible to lift the columns vertically making the attachment of the columns to the foundation simple.

6.3. Lifting of decks

6.3.1. Two webbing slings around deck

Lifting of decks can be done with two webbing slings attached with a spacing of 2-3 meters centered on the longitudinal side of the decks as illustrated above. This will result in a stable and controlled lifting process enabling the stability needed to lower the connections of the decks straight down into the connections of the columns. However, this may cause problems when the margins are very small. The risk of colliding with, and damaging the columns on the way down to the connections are high. With a perfect lift, a straight lowering should be possible, but this is highly improbable to accomplish. The decks should be lifted at a slight angle, not to collide with the columns on the way down. Therefore, the lifting equipment needs to be able to attach to the deck in a way that allows lifting with an angle.

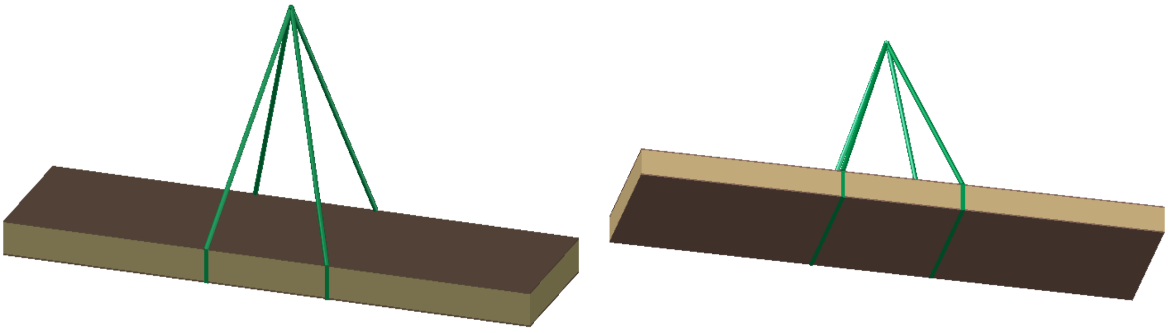


Figure 6.6: Two webbing slings around deck.

6.3.2. Webbing slings attached to eyebolts

One solution to this problem would be to add an additional attachment mechanism to the decks with a hole, which the lifting equipment can be attached to. This mechanism could be an eye bolt or something similar. This allows the deck to safely be lifted at an angle and lowered into the connections on the columns. Here they will be placed on a dowel or some sort of preexisting ledge, so the crane can release the deck and start getting a new one. By doing this, the erection process will be more efficient.

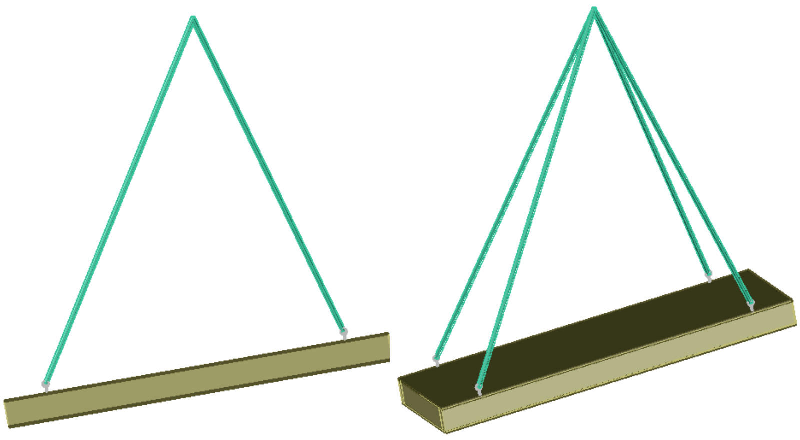


Figure 6.7: Screwed in eyebolts in the deck.

6.4. Erection method

6.4.1. All columns first

A simple way to assemble the structure would be to raise all the columns in their intended places before mounting any decks. For the reference building this would mean that the columns would be placed in two straight lines, with 9 meters between the two lines, and 2,4 meters between the columns in the same line.

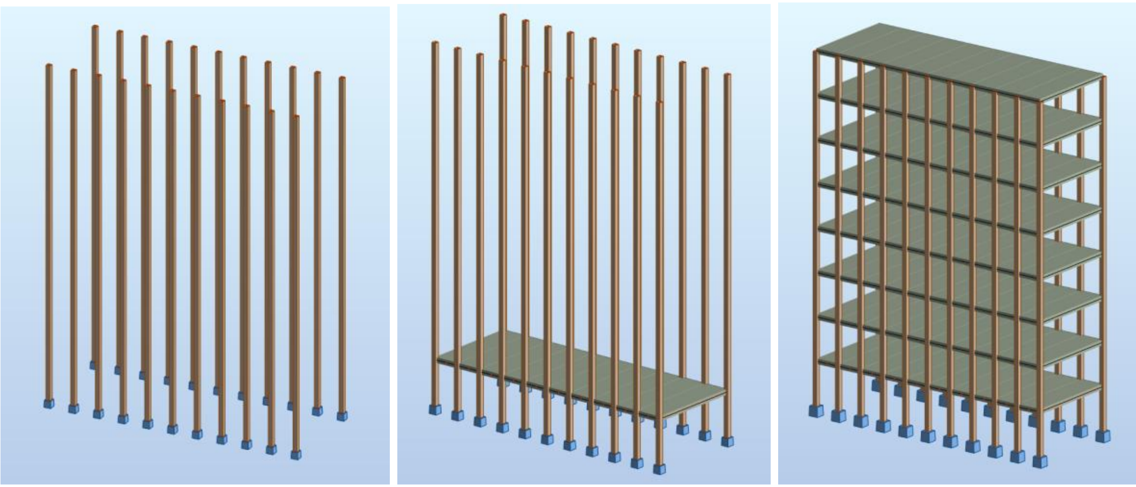


Figure 6.8: Assembly method with all columns raised before mounting decks.

Based on the calculations shown in Appendix A.6, this solution has challenges. The load combination of wind and dead load on the columns results in a moment in the base of the columns bigger than the capacity. The columns are not going to be able to stand by them self (in the worst-case scenario) before at least one story of decks is mounted. Calculations done in Robot Structural Analysis and Mathcad, shown in Appendix A.8, shows that the columns will be able to stand when the first story of decks is mounted. The assembly solution is possible without any further bracing, as long as the decks are connected within a relatively short time. It will be possible to tell how much wind can be expected within a relatively short time frame, and therefore, the assembly can be planned to happen when the workers have time to connect at least one story of decks to the columns, when the wind is acceptable.

6.4.2. Columns section-by-section

Bottom deck first

Considering that the columns are potentially exposed to the largest moment before the decks are mounted, a solution would be to mount a deck for every "section" erected. First, four columns will be risen, and a deck mounted on the first connections to these four columns. Next, two more columns will be risen next to the existing four, and a new deck will be connected, making a new "section", and so on.

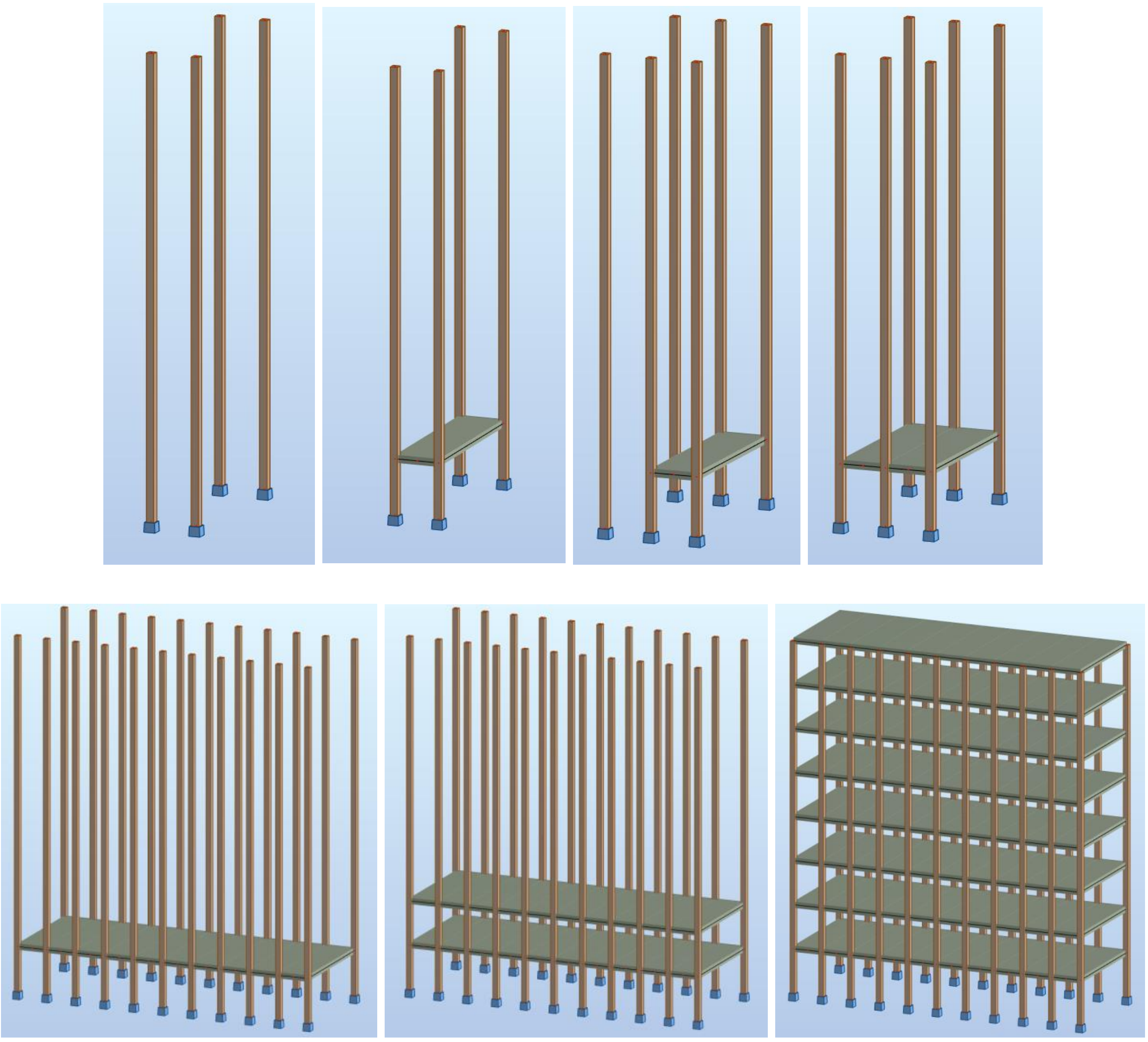


Figure 6.9: Assembly method with only the necessary number of columns raised before mounting decks.

This method will reduce and almost eliminate the time the columns need to stand alone without a deck connected. Based on the calculations shown in Appendix A.8 the columns are within the criteria when one story of decks is mounted. The deformation in the top of the columns are still high, approximately 1,5 meters. This is not necessarily a problem, when this is during erection of the structure. Therefore, the serviceability limit state is not as limiting.

This method of erection can also have some challenges. When the columns and decks are to be mounted more simultaneously, both need to be available on site when the erection process is ongoing. This may cause problems when the storage opportunities in an urban area might be limited.

Top deck first

An alternative method to the above erection method is to place the top deck first, to make the columns able to stand safely (henceforth referred to as top-method). The approach of mounting would be the same as shown in *Figure 6.10*. This method will have both upsides and downsides with regards to the method above, with mounting the bottom decks first. Like described above, this method significantly reduces the time the columns must stand alone, where they are the most vulnerable. In addition, the erection method gives lower loads in the columns during erection, as shown in *Figure 6.10*.

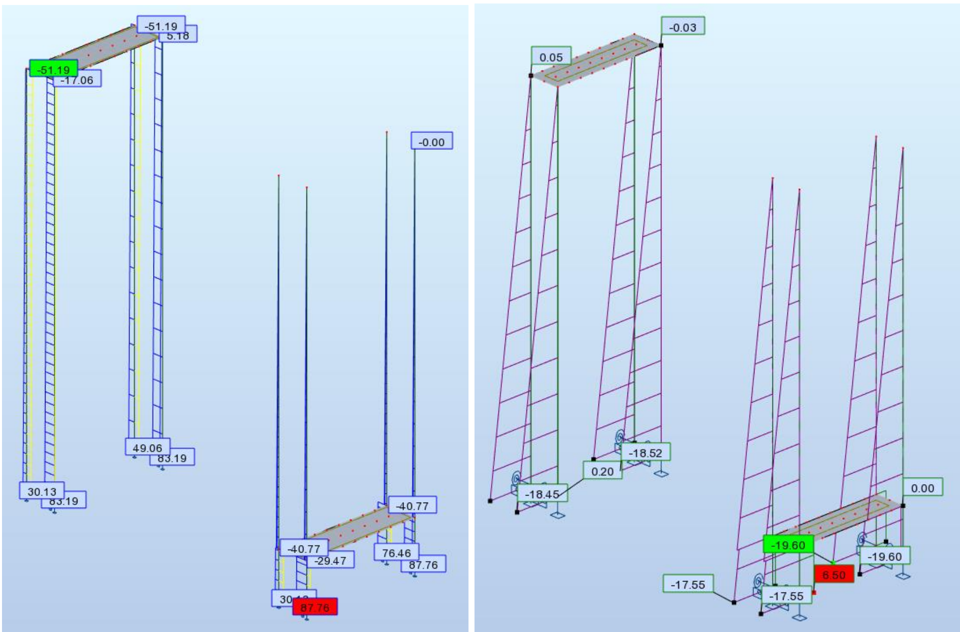
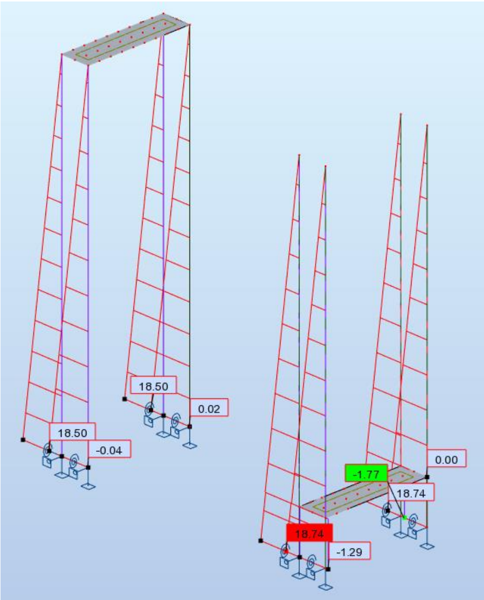
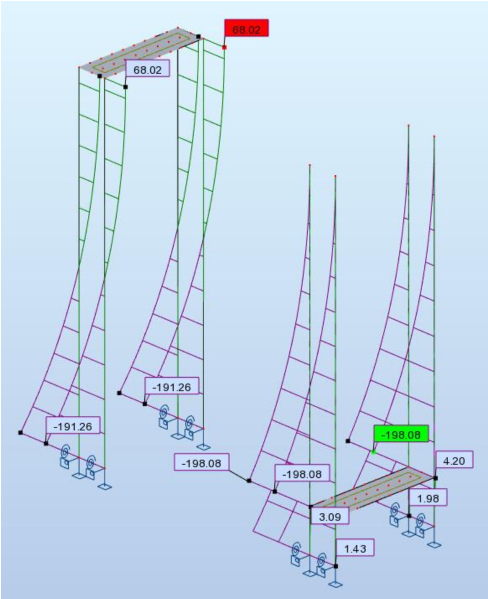


Figure 6.10: Differences in forces acting on the columns with only top deck or only bottom deck attached.

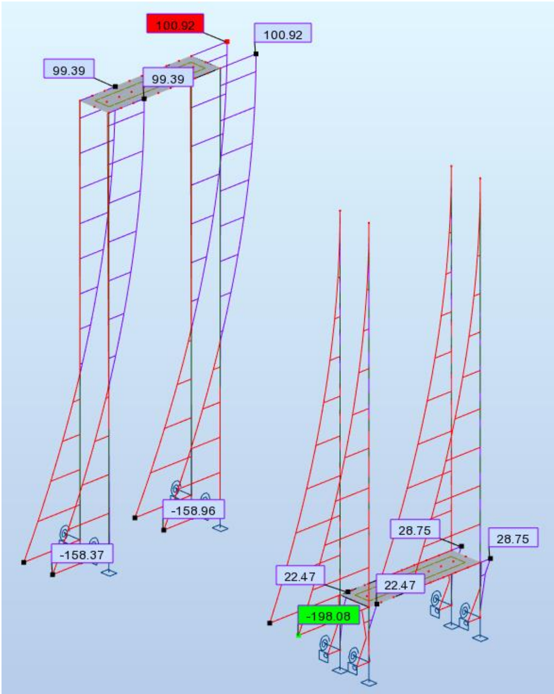
a) Compression. *b) Shear in x-direction.*



c) Shear in y-direction.



d) Moment in x-direction.



e) Moment in y-direction.

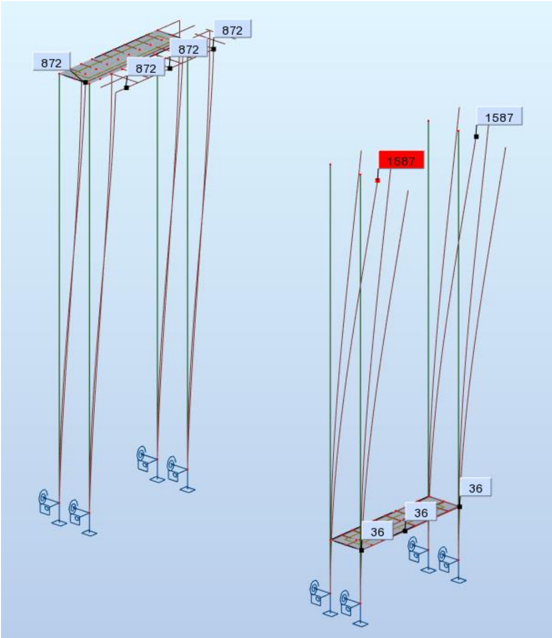


Figure 6.11: Difference in deformations for only top or bottom deck attached.

With these differences in forces and moments in mind, the top-method is a better solution. The top-method also has the advantage that the workers will have a roof over their head as they work inside the construction. This is favorable both for the workers, and for the materials underneath the roof. In a timber structure, moisture is a challenge, and every measure to reduce the risk of this is highly wanted. On the basis of these arguments, the top-method is a solution with many positive sides that the bottom-method does not have. But a challenge the top-method will have is that the erection requires the elements after the top element to be hoisted in from the side. This may cause problems or difficulties with the erection that the bottom-method will not present. Also, with this comes the need for a mobile crane. In an urban area with limited storage space, this can be a challenge, especially if a tower crane is necessary as well.

When the columns stand alone, without decks, they are subjected to the most moment. With this in mind the method with section-by-section seems like the better solution. And the two different possibilities within this solution, the top-method and bottom-method, both have upsides and challenges. But with the forces acting on the columns in mind, the top-method is the best solution. With this method, the columns are subjected to smaller forces and moments than for the bottom-method.

6.5. Decks

6.5.1. Connection of decks to columns

When connecting the decks to the columns, the decks are first hoisted into the connectors on the columns. The connector of the columns have pins that are capable of holding the decks until they are connected. This results in the crane being able to release one deck, then go get another while the deck is being fastened.

The decks are connected to the columns by the use of two bolts for each corner of the decks, as illustrated in *Figure 6.12*. This results in a rapid and uncomplicated procedure when connecting the decks.

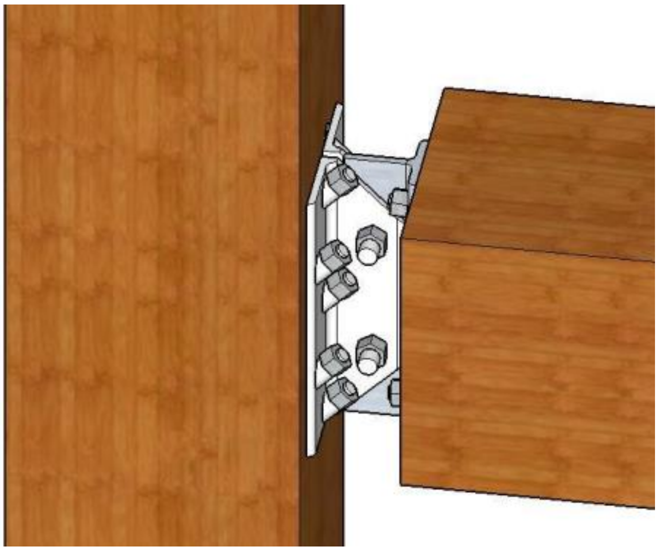


Figure 6.12: Connection between deck and column (WoodSol, 2016).

For the workers to be able to get to the same height as the decks to mount the bolts they will need some kind of scaffolding or a lift. This will be needed early in the assembly process since the first floor of decks are three and a half meter above ground level.

Mobile lift

Mobile lifts are a good solution when only considering the mounting of the bolts for the deck-to-column connection. When considering the whole construction phase, some sort of scaffolding will be needed either to be used as stairs or as safety measures. Therefore the use of mobile lift for connection of decks should only be considered if the plan is to mount some sort of scaffolding after all the decks are connected.

One idea could be to use a mobile lift to connect the first floor of decks so that the columns are stable and stiff, and then mount scaffolding to connect the rest of the decks.

Scaffolding

Scaffolds will probably be necessary regardless when building in these heights. Scaffolds are easy to mount even though it will take up a lot of time to mount them. Since the WoodSol concept is to be applicable in urban areas, the scaffolds will need to be covered in fall safety nets. These will take a lot of time to attach as well.

6.6. Building time

The high degree of prefabrication that the WoodSol concept uses will contribute to a short building process, making it possible to have a building closed within a few weeks after the foundations is done.

Groundworks

The time usage of groundworks will depend a lot on the soil at the construction site and the size of the building. Because of this, and the fact that the authors have very little experience with time consumption of groundworks, this will not be investigated further.

Foundations

If considering the reference building built on loose gravel the foundation sizes needed is 2300x2300x500 [mm], resulting in a 2300x26300x500 [mm] strip foundation under each row of columns. The formwork for these two foundations are estimated to take about 3 days each, using three workers, resulting in 6 days total. By the use of six workers, working three and three on each foundation it will take 3 days total.

After the formwork is done, the reinforcement will need to be installed. The foundations will contain about 2 % reinforcement resulting in 4750 kg of reinforcement each foundation. An approximation of time consumption is about 117 kg/hour resulting in a total time usage of 3 and a half days using three concrete workers (Forsythe, 2017). At the same time as the reinforcement is placed, the steel anchors which the steel plate will be mounted on will also have to be installed. The mounting of steel anchors is included in the three and a half days necessary for the reinforcement to be placed.

When the formwork is finished and the reinforcement is placed, only the pouring of the concrete remains. The two foundations contains a total volume of 61 m^3, this will take 8 truck deliveries. The total casting time will be about three hours, and the forms will only be filled 2/3 to the top, allowing the steel plate to be adjusted later in the process. After the first casting, the foundations will need to harden for about seven days, unless any form of hardener is added.

The total time for the foundations then results in 10 days using three concrete workers, plus seven days of hardening until the columns can be placed.

Column and decks

In chapter 6.4 it was concluded that the building process for the columns and decks would be to mount four columns and then connect them by one deck. This is to obtain necessary strength and stiffness. After all the columns are mounted with one row of decks, the rest of the decks for the other floors and the roof will be installed.

To mount the columns, the steel plate and column will be mounted on the steel anchors after they have been partly casted into the foundations, as illustrated in *Figure 4.24*. The process of lifting the columns on top of the anchors and adjusting the nuts is not very time consuming. A reasonable estimation is that this process will take somewhere around 20 minutes of effective working time per column. After four columns are mounted a deck will need to be connected to the columns. This process is estimated to take about 15 minutes, where 5 minutes is for the lifting process and 10 minutes is for attaching the deck to the four columns.

This results in 1,5 hours to mount four columns connected by one deck, then the next assembly of two columns and one deck can be mounted. After all the columns and decks for the first floor is assembled, the foundations will need to be grouted up to the underside of the steel plates. This step only needs a simple formwork which can be made in about 15 minutes per column, then the foundations are ready for the final grouting. The final grouting have an estimated time of 1,5 hours since only the last third of the formwork will need to be filled.

These assumptions estimate that each assembly of four columns and one deck will take a total of about 2,5 hours of working time. This results in a total time of 14 hours, or two days of effective work for the reference building to get the first floor up and stable.

The next step then is assembly of the rest of the decks. After the first floor is mounted for the reference building 70 decks still remain. Assuming a mounting time of 15 minutes per deck, this step will take about 17,5 hours of effective work.

Adding the time consumption of mounting the columns and decks to the time used for the foundations, the final time usage is 31,5 hours or 4,5 days of effective work.

Figure 6.13: Reference building after final decks are mounted.

Scaffold

If scaffolding is used, which is very likely, the mounting time of these will be significant. Assuming scaffolds around the entire building up to the eight floor it is expected a mounting time of about three days. The mounting of the scaffolds will start immediately after the support system is closed.

Total time

As pointed out earlier the time usage estimated above is effective work time and only includes the construction of the support system for the reference building.

The total time estimated from the formwork for the foundations starts until the support system is closed is 22 days. Only five days is used for mounting of the decks and columns and is dependent on the use of a crane. While seven of the 22 days is waiting for the concrete to harden. After the support system is closed the mounting of scaffolds will take additional three days. Then the mounting of inner and outer walls, ventilation, pipes, electric system and indoor completion remains. The time usage of these phases will not be investigated further.

Source of error

The time usage estimated above is based upon earlier experience of the authors and educated guesses. Work on site is not always as efficient as might be expected or hoped for, but with the right incentives and the right preparations and training the time usage stated earlier should be achievable, maybe even conservative. Especially when knowing that the 10-story building Instacon in India was erected in 48 hours using prefabricated steel and concrete elements (HuffingtonPost, 2012).

7. Environmental aspects

7.1. Transport

Concrete

The emission of CO_2 from transporting concrete is estimated to an average of 11,4 kg/m^3 concrete (Norbetong, 2018). This includes emission of the concrete pump which is used for about 60% of the concrete used on construction sites.

For the reference building, if assumed the building is built on loose gravel, the necessary concrete for the foundations is 61 m^3. The transport of this concrete will pollute 695 kg of CO_2-eq. For a corresponding concrete building, the necessary amount of concrete, including foundation and structure, would be somewhere around 660 m^3. This results in 7524 kg of CO_2-eq for the transportation.

Decks and columns

The decks will be transported by semi-trucks, which have an average emission of CO_2-eq of 0,7 kg/km if transported by environmental friendly trucks (Hagman and Amundsen, 2013). The pollution will in other words depend on where the decks are delivered from. Assuming they are transported from Moelven to Trondheim which is 390 km, the emission for the decks will be 276 kg CO_2-eq per delivery and a total of 3312 kg CO_2-eq for the necessary twelve deliveries.

The authors were not able to find any information regarding the emissions of the transport of the columns, since the transportation is very special for this case. But it is safe to say the emission will be higher per kilometer than the transportation of the decks.

7.2. Materials

In the tables below, the global warming potential (GWP) are listed with the unit kg CO_2-equivalent/m^3 produced. The GWP takes everything into account, from cradle to grave. The numbers are gathered from the EPD (Environmental Product Declaration) (EPD-Norge, 2017) of the products or from Norsk Prisbok (NorskPrisbok, 2018). In Norsk Prisbok, the numbers are sometimes given for an exact example. In these cases the authors of this thesis calculated the corresponding number for the unit given in the tables. The steel in *Table 7.1* are for the steel in the timber structure. Almost all of the steel used in the WoodSol project are used in a new way. This makes it hard to find values of the GWP. Therefore, the GWP has been calculated as an approximation from several other steel GWPs. Hence, the number is general.

Material	kg CO_2-eq/m^3
Standard Glulam beam/column	- 864,8
Kerto-plate	- 653,0
Steel (timber structure)	12000,0
Concrete foundations	508,6
Concrete columns	621,1
Concrete decks	423,3

Table 7.1: GWP for different parts of the reference building made with timber or concrete.

Material	Necessary m^3	Total kg CO_2-eq
Standard Glulam Beam	255,4	- 220 869,9
Kerto-plate	186,6	- 121 849,8
Steel (timber structure)	2,9	34 800,0
Concrete foundations	58,2	29 600,5
Total	-	- 278 319,2

Table 7.2: Necessary volume for parts, and total kg CO2-eq. for timber reference building with eight stories.

As seen in *Table 7.1*, the kg CO_2-eq/m^3 for Standard Glulam beam/column and Kerto-plates has a negative value. This is because wood has the ability to store CO_2 when in use, and can be used as an energy source after the lifetime of the building is over. This can reduce the use of fossil fuels as an energy source. The GWP for Standard Glulam beam/column (EPD-Norge, 2015a), Kerto-plate (EPD-Norge, 2015a) and steel (EPD-Norge, 2015b) were found in the products EDP. The GWP for concrete foundations, columns and decks were found in Norsk Prisbok (NorskPrisbok, 2018). The calculations for the necessary m^3 for each part can be found in Appendix A.9.

Material	Necessary m^3	Total kg CO_2-eq
Concrete foundations	86,2	43 841,3
Concrete columns	55,4	34 408,9
Concrete decks	518,4	219 438,7
Total	-	297 688,9

Table 7.3: Necessary volume for parts, and total kg CO2-eq. for concrete reference building with eight stories.

Table 7.2 and *Table 7.3* show that the GWP for a concrete building with the same lay-out as the reference building in timber, is significantly higher. If the reference building were to be built with concrete, the lay-out would probably be a little different, making the numbers in the tables slightly inaccurate. For example, the number of columns might be reduced. Still, the vastness of differences in the GWP is remarkable. The numbers for the foundations are calculated on the basis of loose gravel in the soil. For the concrete foundations, columns and decks the reinforcement is included in the GWP.

Table 7.4 shows the total kg CO_2-eq for eight, six and four story buildings. The results for six and four stories are calculated in the same way as for eight stories. For more detail see Appendix A.9.

Building	Total kg CO_2-eq
Eight story timber building	- 278 319,2
Eight story concrete building	297 688,9
Six story timber building	- 210 458,1
Six story concrete building	222 629,3
Four story timber building	- 136 767,4
Four story concrete building	149 314,7

Table 7.4: Total GWP for timber or concrete reference building with varying number of stories.

As seen from *Table 7.4,* the GWP difference is larger the taller you build. This confirms the results found by Skullestad (Skullestad, 2016), which were presented in chapter 2.4. As mentioned in chapter 2.4, the negative value of the GWP for timber elements is only valid when assuming sustainable harvest. For the negative GWP to be valid, incineration with heat recovery after destruction of the structure must also be assumed. The timber is then used as a replacement for natural gas as an energy source. For a more detailed analysis with more variation in stories, see *Figure 2.5* (Skullestad, 2016).

Number of stories	Difference in total GWP [kg CO_2-eq]
Four	286 082,1
Six	433 087,4
Eight	576 008,1

Table 7.5: Differences in GWP for timber and concrete reference building.

The difference in GWP increases steadily between each increase in number of stories. This complies with what Skullestad submitted in her thesis and what is shown in *Figure 2.5.* Skullestad takes this further, and looks at structures up to 21 stories. The savings seem to be larger the taller you build after 12 stories. The saving of 576.008 kg CO_2-eq is equal to driving one million new Volvo V90 cars 4,5 kilometers (Marcussen, 2016).

7.3. Sources of error

The different values for the kg CO_2-eq polluted, are not all gathered from the same source. This can lead to errors. The used sources are Norsk Prisbok, EPD's and the provider of the products. Each of the sources should be trustable by itself, but may vary from each other, when the base of the calculations might vary.

8. Economical aspects

For the WoodSol concept to be chosen instead of traditional concrete and steel buildings the concept will have to be comparable economical. This chapter focuses on the costs for the different main parts needed when building with the WoodSol concept. All prices listed is w/o VAT.

8.1. Transport

Concrete

The price for transport of concrete per m^3 transported is listed in the table below. The prices is taken from the list of prices from Betong Øst (BetongØst, 2018).

Km	Price (NOK/m^3)	Km	Price (NOK/m^3)	km	Price (NOK/m^3)
1	135	8	158	15	202
2	135	9	165	16	209
3	135	10	172	17	215
4	135	11	174	18	222
5	142	12	181	19	228
6	142	13	188	20	234
7	149	14	195	21	240

Table 8.1: Prices for transport of concrete per m^3.

Assuming an eight story timber building built on loose gravel, the total volume of concrete needed for the reference building will be about 61 m^3. This will need 8 concrete deliveries. By then assuming a transport distance of 4 km for the concrete, the final cost for the transport of concrete is about 7.965 NOK. This is a small cost compared to other elements in this chapter. If the concrete has to be transported 20 km the final price will be 13.806 NOK.

This price assumes that the emptying of the concrete cars happens effective. With strategic planning by concrete workers this should not be a problem at all.

Decks

The cost of transporting the decks on a semi-truck will be 1.025 NOK/hour by using YND transport AS in Norway (YNDTransportAS, 2018). Where the decks will be delivered from is still not determined. If assumed the decks are delivered from Moelven the cost can be estimated to 6.000 NOK per delivery, and a total of 84.000 NOK for the reference building. By having the decks made by a nearby company, this cost can be reduced by a lot.

Columns

One extendable semi-truck can be loaded with 27 tons. For an eight story building this results in 14 columns at the same time. This will be a special transport, and the price for this transportation from Moelven to Trondheim is calculated to 52.000 NOK. This calculation is done by the chief of transportation at Moelven Limtre, see Appendix B.2. For the reference building, having 22 columns there will be need for two transports. This will have a cost of 104.000 NOK. Dividing the cost by the number of kilometers a price is estimated to 133 NOK/km for this special transport.

The total cost of transportation for the support system of the reference building will be approximately 196.000 NOK.

8.2. Cranes

Tower crane

The price for rigging a 132 EC-H tower crane is approximately 240.000 NOK, and the rent is 40.000 NOK/month, see Appendix B.3. Addition to the renting of the crane, there is need for a concrete foundation to erect the crane upon. This foundation is typical about 5000x5000x800 [mm] (EDKnutsen, 2018) and will have a cost of approximately 210.000 NOK assuming 2 % reinforcement and a formwork price of 10.000 NOK. This results in a starting price of 450.000 NOK for the tower crane before any rent is paid.

Self-erecting tower crane

For a self-erecting tower crane, delivered from KRANOR the price of mounting is 80.000 NOK while the rent per month is 55.000 NOK. While the foundation cost will be the same as for the tower crane, 210.000 NOK. The starting price for this crane is 290.000 NOK before any rent is paid.

Mobile crane

The renting of a mobile crane is approximately 1.450 NOK/hour for the size needed for an eight story building (NorskPrisbok, 2018).

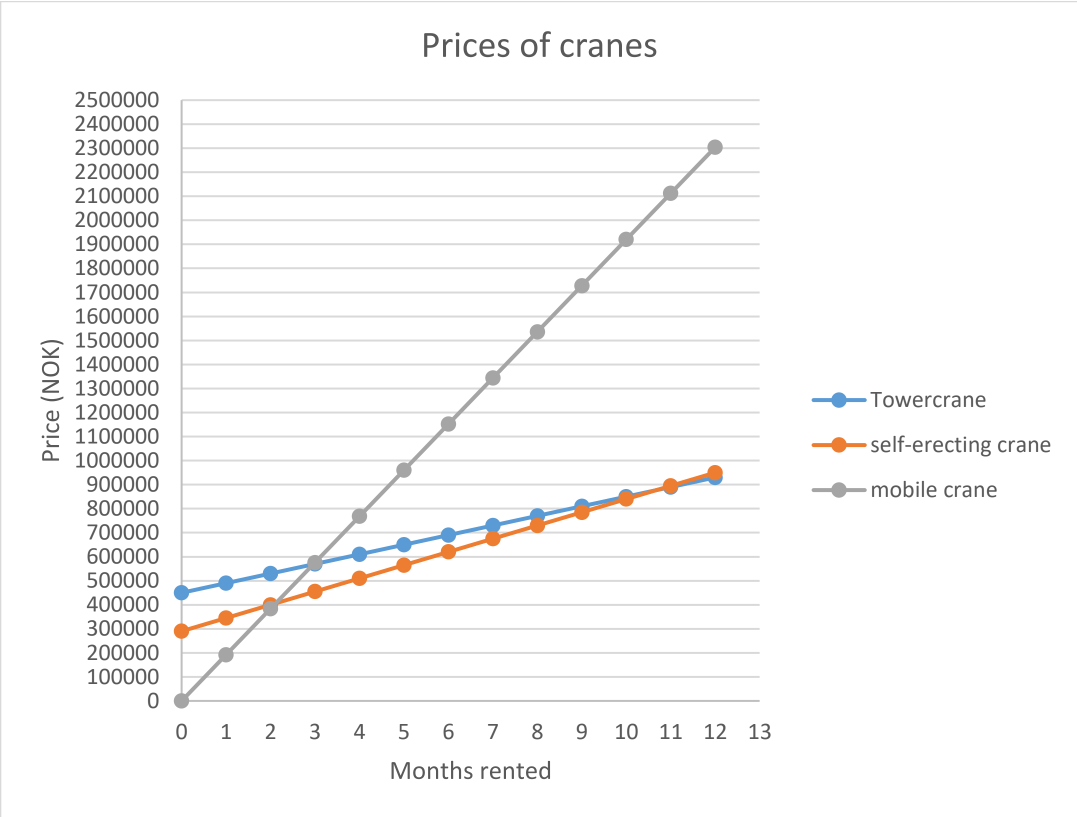


Figure 8.1: Price of cranes.

In figure 8.1 the prices of using a tower crane compared to a mobile crane and a self-erecting crane is plotted depending on the number of months rented. As can be seen, the mobile crane is cheapest up to about 2 months, then the self-erecting crane starts becoming the most economic choice. If building time is over one year, then the tower crane is the cheapest choice. These prices are without the cost of a crane driver, but this cost will be equal for the three cranes. For the building time estimated for the reference building a mobile crane would be the cheapest solution, but this does not consider the phases after the support system is completed.

8.3. Scaffolding

Scaffolding

The price of rigging for the scaffold is 34 NOK/m^2 while the rent is 39 NOK/m^2/month. The down rigging is 13 NOK/m^2.

Additional there will be needed a stair tower which has a cost of 1.213 NOK/m^2 for rigging and a price for down rigging of 778 NOK/m^2. The rent is the same as for normal scaffolding, 39 NOK/m^2/month.

The total area necessary for the reference building will be:

$$A = 24 * 28 + 9 * 28 = 928\ m^2$$

Where 56 m^2 of the 928 m^2 will be consisting of a stair tower. Total price for the scaffolding varying by the renting time is shown in table 8.2.

Months	**Area (Scaffold + Stair tower) (m^2)**	**Price of rigging (NOK)**	**Price of down rigging (NOK)**	**Rent/Month (NOK)**	**Total price (NOK)**
0	872 + 56	97576	54904	36192	152480
2	872 + 56	97576	54904	36192	224864
4	872 + 56	97576	54904	36192	297248
6	872 + 56	97576	54904	36192	369632
8	872 + 56	97576	54904	36192	442016
10	872 + 56	97576	54904	36192	514400
12	872 + 56	97576	54904	36192	586784

Table 8.2: Prices of scaffolds.

The sum of the prices for rigging and down rigging is 152.480 NOK, this is the price before any rent has been paid. For the reference building the rent per month is estimated to 36.192 NOK.

The rent of scaffolding can, depending on the time needed, have a significant impact on the final price of the construction. Having a short building time of about 2 months will reduce the cost of the scaffolding by about 360.000 NOK or 60% compared to renting the scaffolds one year.

8.4. Elements

Steel rods

The price for the steel rods connecting the steel plate to the column have an assumed price of 100 NOK/rod which shouldn't be too far off, see Appendix B.6.

Rods per column base	**Number of columns**	**Price/Rod (NOK)**	**Total price (NOK)**
4	22	100	8800
6	22	100	13200
8	22	100	17600
10	22	100	22000

Table 8.3: Price of steel rods for the column/steel plate connection.

Additional to the rods connecting the column to the steel plate, there will also be rods where the connection from the columns to the decks are. Assuming two rods going into the slabs and four rods entering the column per connector, this results in a total of six rods per connector. With four connectors each deck element, this sums up to 2052 rods being needed for the connectors in the reference building, or an average 96 rods/column including the rods in the slabs for an eight story building.

In the table below it is assumed eight rods in each column base additional to the rods for the deck connectors in the columns and the decks.

Stories	**Number of columns**	**Rods/column**	**Price/rod**	**Total price (NOK)**
4	22	50	100	110000
6	22	73	100	160200
8	22	96	100	210200
10	22	120	100	264000

Table 8.4: Prices of steel rods for the column/deck connection.

The rods will be installed in the column and decks in the prefabrication stage. But the prices in this part chapter is to show the significance the number of rods in the dimensioning phase have on the economical aspect.

Steel plates with anchors

Smith Stål has estimated the cost of a steel plate with the dimensions 600x600x50 [mm] to 7.500 NOK each plate, se Appendix B.4. This results in a price of 53 NOK/kg steel. This price per kg is used to estimate the price for the plate dimensions shown in the table below.

Dimensions (mm x mm x mm)	**Price (NOK)**
600x600x10	1500
600x600x20	3000
600x600x30	4500
600x600x40	6000
600x600x50	7500

Table 8.5: Estimation of prices per steel plate.

Additional to this there is need for about four ϕ28 anchors with a 200 millimeter length, estimated to a price of about 100 NOK each. This results in an additional price of 400 NOK per plate. The table below shows the difference of the total price for steel plates and anchors for the reference building for different plate thicknesses. Choosing to use eight ϕ22 anchors instead of four ϕ28 will add approximately 400 NOK to each steel plate.

Number of steel plates	**Dimension (mm x mm x mm)**	**Price/Steel plate (NOK)**	**Total price (NOK)**
22	600x600x10	1900	31680
22	600x600x20	3400	54560
22	600x600x30	4900	77440
22	600x600x40	6400	100320
22	600x600x50	7900	127600

Table 8.6: Price of different steel plates.

From *Table 8.6* it can be seen that using 10 millimeter steel plates is 95.920 NOK cheaper than using 50 millimeter steel plates. Here it is a consideration of what rotational stiffness is needed in the foundations. If it is necessary with a high rotational stiffness, the 50 millimeter thick plates should be used. If rotational stiffness can be low, thinner plates should be used for economic reasons.

Decks

The material price for the timber-part of the decks is estimated to 1.200 NOK/m^2 (Bjørge and Kristoffersen, 2017). This does not include the production cost or the cost of the steel connectors installed in the decks. The final cost of the decks may be somewhere around 30.100 NOK/element. In this price the four connectors in the deck is assumed a price of 300 NOK/connector and the price of production is estimated to 3.000 NOK/element. The final price per area is then 1.393 NOK/m^2. The price of a HD265 hollow core elements is 839 NOK/m^2 in comparison.

For the reference building this would result in a price of 2.408.000 NOK for the 80 deck elements needed. The use of hollow core elements would result in a price of about 1.449.792 NOK in comparison. The deck elements is estimated to be the most expensive part of the reference building, this is where most could be saved by lower prices.

Columns

The price per cubic of glulam is approximately 11.000 NOK/m^3, see Appendix B.5. This does not include the price of the steel rods for the deck connectors or the foundation base.

For an eight story building, needing columns with dimensions 28000x400x400 [mm] the price is 49.280 NOK per column. For the reference building, needing 22 columns, the total price for the columns is 1.084.160 NOK.

Stories	Length of column (m)	Price/m^3 (NOK)	Price per column (NOK)
10	35	11000	61600
8	28	11000	49280
6	21	11000	36960
4	14	11000	24640

Table 8.7: Price of columns.

Foundations

Price for reinforcement is approximately 20 NOK/kg, while the price of installing the reinforcement is 19 NOK/kg (NorskPrisbok, 2018).

The price for concrete is 1.900 NOK/m^3 which includes the price of transport (YNDTransportAS, 2018). This price may vary a lot from one construction company to another, where some may pay as little as 1.300 NOK/m^3. This is a decrease of about 30 % having a significant impact on the foundation prices.

The price for the finished formwork is 6.000 NOK/formwork for the dimensions 2000x2000x600 [mm]. Assuming 2% of the concrete as reinforced the price can be calculated for different foundations.

Soil	Foundation (mm x mm x mm)	Concrete (m^3)	Steel (Kg)	Price/formwork (NOK)	Price /foundation (NOK)	Total foundation price (NOK)
Fine sand	2400x4700x500	5,64	885	8000	53000	1166000
Loose gravel	2300x2300x500	2,65	416	6300	27560	606320
Gravel	1950x1950x500	1,90	298	6000	21232	467104
Solid rock	1250x1250x500	0,78	123	4500	10800	237600

Table 8.8: Prices of foundations for eight story timber building.

Soil	Foundation (mm x mm x mm)	Concrete (m^3)	Steel (Kg)	Price/formwork (NOK)	Price /foundation (NOK)	Total foundation price (NOK)
Fine sand	2400x8400x500	10,08	1590	10000	91260	2008820
Loose gravel	2400x3300x500	3,92	615	8000	39433	867526
Gravel	2100x2100x500	2,2	345	6000	23635	520000
Solid rock	1000x1000x500	0,5	79	4000	8040	176682

Table 8.9: Prices of foundations for eight story concrete building.

As can be seen from the tables above, the price of the foundation on fine sand is 42% lower for the timber building than for the concrete building, while for loose gravel there can be saved 30%. These prices does not include groundworks where additional savings could be made by choosing a timber building instead of a concrete building, this can be a part of further work.

On the next page the foundation prices for a six and a four story timber building is calculated.

Soil	Foundation (mm x mm x mm)	Concrete (m^3)	Steel (Kg)	Price/formwork (NOK)	Price /foundation (NOK)	Total foundation price (NOK)
Fine sand	2850x2850x500	4,06	637	8000	40557	892254
Loose gravel	1850x1850x500	1,71	269	5500	19240	423280
Gravel/Soft rock	1300x1300x500	0,85	133	4500	11302	248644
Solid rock	1000x1000x500	0,5	79	4000	8031	176682

Table 8.10: Prices of foundations for six story timber building.

Soil	Foundation (mm x mm x mm)	Concrete (m^3)	Steel (Kg)	Price/formwork (NOK)	Price /foundation (NOK)	Total foundation price (NOK)
Fine sand	2200x2200x500	2,42	380	6200	25618	563596
Loose gravel	1600x1600x500	1,28	201	5000	15271	335962
Gravel/Soft rock	1300x1300x500	0,85	133	4500	11302	248644
Solid rock	900 x 900 x 500	0,40	63	4000	7217	158774

Table 8.11: Prices of foundations for four story timber building.

From the tables it can be concluded that building eight instead of six stories increases the costs of the foundations by approximately 60% if built on fine sand, while loose gravel increases the cost of the foundation by 26%.

8.5. Installations and groundworks

Compared to concrete, the installations (el, ventilation etc.) should be easier and less time consuming when mounted in a timber building. This, because it is easier to fasten installations to timber than to concrete, plus drilling of holes is less problematic in timber.

The groundworks should also be of a smaller scale when building in timber instead of concrete, because of the weight. This can partly be seen in chapter 4.3 where it is shown that the reference building in timber need a smaller foundation than when built with concrete.

The cost of installations and groundworks have not been investigated further in this thesis as the authors have very little experience in these fields. To get information about these costs also seemed to be a difficult task. This could be a suggestion for further work.

8.6. Sources of error

The prices of materials, tools and resources may have great variations from project to project, and from the deals done by companies with providers. These calculations are therefore approximations done on the basis of information gathered from different providers and contacts in the industry.

9. Summary

9.1. Conclusive remarks

For this thesis, a reference building is considered. The buildability and assembly aspects are investigated. This is done by, amongst other things, varying the sizes and numbers of components in the foundations, while still maintaining a sufficient stability and capacity of the structure. The capacity of the columns are checked during lifting, when standing without deck elements attached and when the whole bearing structure is complete. Different lifting methods are considered, as well as assembly methods of the bearing structure. Educated guesses are made to estimate the time consumption of the assembly process. The transportational, environmental and economic aspects are also investigated. The reference building is compared to a similar building made in concrete, and the differences in costs and pollution are mapped, varying the number of stories. The components in the foundations are varied in size and number, to see what is decisive for the economical aspect. The transportation of decks, columns, concrete and materials are looked at, with solutions and prices gathered from contacts in the industry.

After the work done in this thesis, the authors see no reasons why the concept should not be buildable. Every dimension needed for the different parts is reasonable when considering the economical aspect, transportational aspect and the buildability aspect. The assembly of the WoodSol concept for the reference building seem to be fast and simple. Considering the investigations done in the environmental aspect, the WoodSol concept seems to be a more eco-friendly solution compared to the traditional steel and concrete buildings.

The main outcome of these investigations are as follows:

- For a six-to-eight story timber building built with the WoodSol concept, a strip foundation for the columns will be the better solution. This, unless the ground is very stiff and bearing, then spot foundations might be considered. For a four story building, assuming the ground is not particularly soft, spot foundations are sufficient.
- The thickness of the steel plate and the stiffness of the soil are the most important contributors to the rotational stiffness of the foundations.
- The obtainable rotational stiffness of the foundation base is in the range of 3315-11460 [kNm/Rad].

- The columns cannot stand by themselves during erection, and one deck element must therefore be mounted as soon as possible to connect the columns together. The best solution for this is to connect the top deck element first.
- It is possible to have the bearing structure for the reference building erected in five days after the foundations are finished.
- The saving of kg CO_2-eq polluted, for the reference building built in timber compared to concrete, are 286.082 kg, 433.087 kg, 576.008 kg, for four, six and eight stories respectively.
- The costs of the foundations are 30-42% less for the reference building in timber compared to concrete, while the amount of concrete is reduced by 33-44%, depending on the stiffness of the ground.
- The most costly parts for the reference building built on fine sand is:

- Deck elements 2.408.000 NOK.
- Foundation 1.166.000 NOK.
- Columns 1.084.160 NOK.

9.2. Suggestions for future work

In this thesis there have been some challenges in areas the authors have not focused on. For further development of the WoodSol concept, some of these areas should be reviewed. On the basis of this thesis, the following recommendations for future work are proposed:

- **Steel rods straight from column to concrete foundation:** As can be seen from chapter 8.4, the steel plates can have a significant impact on the economic aspect of a building. The plates play a big role in the variation of the rotational stiffness in the foundations as well. By eliminating the need for a steel plate, both the economic aspect and rotational stiffness could benefit.
- **Shear walls and shafts in a WoodSol building:** The differences in displacements in the reference building with or without shafts are large. A more detailed calculation and modeling of shafts or shear walls may be important. The placement, production, assembly and capacity of shafts and shear walls in a WoodSol building, may be something the project can benefit from.
- **The economic cost of groundwork and installations:** An in-depth examination of the entirety of groundworks as well as installations in a WoodSol building will be

important. This information can add to and correct some of the results and findings done in this thesis. For every building project, the economic aspect is very important, and the groundwork for the structure, as well as the installations going into the building will have a big impact on the economy.

- **Total building time:** For the calculations on building time done in this thesis, the work on the actual bearing structure are the only thing accounted for. The groundworks and completion, both external and internal, are not calculated. This can be interesting to see, especially in comparison to the completion of a concrete building.
- **Dividing the columns:** In chapter 5.4, the transportation of the columns present a challenge. The length of the transport makes it necessary to use a police escort. By dividing the columns, this may be avoided. Therefore, the possibility for assembly of the two column parts on site could be reviewed, as well as the best way to divide the columns.
- **The decks' added weights influence on the foundation:** The difference in weight of the structure for a timber building and concrete building is reduced when filling the decks to achieve acoustic properties. By finding another solution than adding weight to the decks, the foundations of the building can be drastically reduced. This will be beneficial to both the environmental and economic aspects.

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APPENDICES

A Calculations

A.1 Spreadsheets for foundation calculations

Isolated Foundation Calculation (ACI)		Issue:	Design	Page
		Date:	0	1 of 2
		Revised by:	Test1	
Project:	WoodSol eight story timber building 100 SBC	Checked by:	Test2	

Input Data

Loads (kN)			Mz
	Px	Py	(kN.m)
Dead	57	695	259
Live	0	0	0
Wind	0	0	0
E	0	0	0
Sum	57	695	259

Concrete Strength	
f'c (N/mm2) =	27,6
Steel Strength	
fy (N/mm2) =	414
Allowable Soil Pressure	
qa (kN/m2) =	100
Base Soil angle of internal friction	20

Foundation Properties			
H (mm)	540	h (mm)	0
hw (mm)	0	L/B	1
Pedestal Dimension			
x (mm)	500	y (mm)	500

Ultimate Loads (ACI 9.2.)

Load Case	0.9D+1.3W		1.4D+1.7L		0.75(1.4D+1.7L+1.7W)			0.75(1.4D+1.7L+1.87E)		
Factors	0,9	1,3	1,4	1,7	1,05	1,275	1,275	1,05	1,275	1,4025
Pux (kN) =	51,3		79,8		59,85			59,85		
Puy (kN) =	625,5		973		729,75			729,75		
Muz (kN.m) =	233,1		362,6		271,95			271,95		

<u>CHECKING:</u>

Contact Pressure

q_{max} = 121,5 kN/m2 q_{min} = 28,99 kN/m2

q_{GP} (gross pressure) (kN/m2) = (3qmax + qmin) /4

q_{GP} = 98,36 kN/m2 < qa = 100 <u>YES</u>

Stability against Overturning

Overturning moment	289,78	kN.m
Stabilizing moment	1414,22	kN.m

$$\frac{\text{Stabilizing moment}}{\text{Overturning moment}} = 4,88 > 1,5$$

<u>YES</u>

Stability against Sliding

$$\frac{Fv \times \tan\theta}{F_h} = 5,391 > 1,5$$

<u>YES</u>

Check Wide Beam Shear

Vc (kN)	max Vu (kN)	Load Case
1261,295	440,710	2

max Vu / 0.85Vc = 0,411 < 1,00 <u>YES</u>

Check Punching Shear

Vc (kN)	max Vu (kN)	Load Case
2801,204	898,012	2

max Vu / 0.85Vc = 0,377 < 1,00 <u>YES</u>

<u>Results:</u>

Base Dimensions

B =	3350	mm	d =	430	mm
L =	3350	mm	D =	500	mm

Reinforcement

	x - Direction		z - Direction
	Bottom Reinforcement	Top Reinforcement	Bottom Reinforcement
As (cm2) =	28,75971985	0	25,929
ρ =	0,00199651	0	0,0018

Isolated Foundation Calculation (ACI)		Issue:	Design	Page
		Date:	0	1 of 2
		Revised by:	Test1	
Project:	WoodSol eight story timber building 245 SBC	Checked by:	Test2	

Input Data									
Loads (kN)			Mz	Concrete Strength		Foundation Properties			
	Px	Py	(kN.m)	f'c (N/mm2) =	27,6	H (mm)	540	h (mm)	0
Dead	57	695	259	Steel Strength		hw (mm)	0	L/B	1
Live	0	0	0	fy (N/mm2) =	414	Pedestal Dimension			
Wind	0	0	0	Allowable Soil Pressure		x (mm)	500	y (mm)	500
E	0	0	0	qa (kN/m2) =	245				
Sum	57	695	259	Base Soil angle of internal friction			20		

Ultimate Loads (ACI 9.2.)										
Load Case	0.9D+1.3W		1.4D+1.7L		0.75(1.4D+1.7L+1.7W)			0.75(1.4D+1.7L+1.87E)		
Factors	0,9	1,3	1,4	1,7	1,05	1,275	1,275	1,05	1,275	1,4025
Pux (kN) =	51,3		79,8		59,85			59,85		
Puy (kN) =	625,5		973		729,75			729,75		
Muz (kN.m) =	233,1		362,6		271,95			271,95		

CHECKING:

Contact Pressure qmax = 287,6 kN/m2 qmin = 1,788 kN/m2

q_{GP} (gross pressure) (kN/m2) = (3qmax + qmin) /4

q_{GP} = 216,1 kN/m2 < **qa = 245** **YES**

Stability against Overturning

Overturning moment	289,78	kN.m
Stabilizing moment	880,218	kN.m

$$\frac{\text{Stabilizing moment}}{\text{Overturning moment}} = 3{,}038 > 1{,}5$$ **YES**

Stability against Sliding

$$\frac{Fv \times \tan\theta}{F_h} = 4{,}887 > 1{,}5$$ **YES**

Check Wide Beam Shear

Vc (kN)	max Vu (kN)	Load Case
865,963	370,904	2

max Vu / 0.85Vc = 0,504 < 1,00 YES

Check Punching Shear

Vc (kN)	max Vu (kN)	Load Case
2801,204	813,917	2

max Vu / 0.85Vc = 0,342 < 1,00 YES

Results:

Base Dimensions

B =	2300	mm	d =	430	mm
L =	2300	mm	D =	500	mm

Reinforcment

	x - Direction		z - Direction
	Bottom Reinforcement	Top Reinforcement	Bottom Reinforcement
As (cm2) =	19,63486862	17,802	17,802
ρ =	0,001985325	0,0018	0,0018

Isolated Foundation Calculation (ACI)	Issue:	Design	Page
	Date:	0	1 of 2
Project: WoodSol eight story timber building 440 SBC	Revised by:	Test1	
	Checked by:	Test2	

Input Data

Loads (kN)			Mz	Concrete Strength		Foundation Properties			
	Px	Py	(kN.m)	f'c (N/mm2) =	27,6	H (mm)	540	h (mm)	0
Dead	57	695	259	Steel Strength		hw (mm)	0	L/B	1
Live	0	0	0	fy (N/mm2) =	414	Pedestal Dimension			
Wind	0	0	0	Allowable Soil Pressure		x (mm)	500	y (mm)	500
E	0	0	0	qa (kN/m2) =	440				
Sum	57	695	259	Base Soil angle of internal friction		20			

Ultimate Loads (ACI 9.2.)

Load Case	0.9D+1.3W		1.4D+1.7L		0.75(1.4D+1.7L+1.7W)			0.75(1.4D+1.7L+1.87E)		
Factors	0,9	1,3	1,4	1,7	1,05	1,275	1,275	1,05	1,275	1,4025
Pux (kN) =	51,3		79,8		59,85			59,85		
Puy (kN) =	625,5		973		729,75			729,75		
Muz (kN.m) =	233,1		362,6		271,95			271,95		

CHECKING:

Contact Pressure

qmax = 430,6 kN/m2 qmin = -38,4 kN/m2

q_{GP} (gross pressure) (kN/m2) = 2 Fv / (3 (L/2 - e) B)

q_{GP} = 434,7 kN/m2 < qa = 440 **YES**

Stability against Overturning

Overturning moment	289,78	kN.m
Stabilizing moment	726,983	kN.m

$$\frac{\text{Stabilizing moment}}{\text{Overturning moment}} = 2{,}509 > 1{,}5$$ **YES**

Stability against Sliding

$$\frac{Fv \times \tan\theta}{F_h} = 4{,}761 > 1{,}5$$ **YES**

Check Wide Beam Shear

Vc (kN)	max Vu (kN)	Load Case
734,186	307,472	2

max Vu / 0.85Vc = 0,493 < 1,00 **YES**

Check Punching Shear

Vc (kN)	max Vu (kN)	Load Case
2801,204	751,686	2

max Vu / 0.85Vc = 0,316 < 1,00 **YES**

Results:

Base Dimensions

B =	1950	mm	d =	430	mm
L =	1950	mm	D =	500	mm

Reinforcement

	x - Direction		z - Direction
	Bottom Reinforcement	Top Reinforcement	Bottom Reinforcement
As (cm2) =	16,3645649	15,093	15,093
ρ =	0,001951648	0,0018	0,0018

Isolated Foundation Calculation (ACI)		Issue:	Design	Page
		Date:	0	1 of 2
Project:	WoodSol eight story concrete building 100 SBC	Revised by:	Test1	
		Checked by:	Test2	

Input Data

Loads (kN)			Mz	Concrete Strength		Foundation Properties			
	Px	Py	(kN.m)	f'c (N/mm2) =	27,6	H (mm)	540	h (mm)	0
Dead	58	1613	113	Steel Strength		hw (mm)	0	L/B	1
Live	0	0	0	fy (N/mm2) =	414	Pedestal Dimension			
Wind	0	0	0	Allowable Soil Pressure		x (mm)	500	y (mm)	500
E	0	0	0	qa (kN/m2) =	100				
Sum	58	1613	113	Base Soil angle of internal friction		20			

Ultimate Loads (ACI 9.2.)

Load Case	0.9D+1.3W		1.4D+1.7L		0.75(1.4D+1.7L+1.7W)			0.75(1.4D+1.7L+1.87E)		
Factors	0,9	1,3	1,4	1,7	1,05	1,275	1,275	1,05	1,275	1,4025
Pux (kN) =	52,2		81,2		60,9			60,9		
Puy (kN) =	1451,7		2258,2		1693,65			1693,65		
Muz (kN.m) =	101,7		158,2		118,65			118,65		

CHECKING:

Contact Pressure

qmax = 102,5 kN/m2 qmin = 83,45 kN/m2

q_{GP} (gross pressure) (kN/m2) = (3qmax + qmin) /4

q_{GP} = 97,71 kN/m2 < qa = 100 YES

Stability against Overturning

Overturning moment	144,32	kN.m
Stabilizing moment	4235,34	kN.m

$$\frac{\text{Stabilizing moment}}{\text{Overturning moment}} = 29,35 > 1,5 \quad \text{YES}$$

Stability against Sliding

$$\frac{Fv \times \tan\theta}{F_h} = 11,81 > 1,5 \quad \text{YES}$$

Check Wide Beam Shear

Vc (kN)	max Vu (kN)	Load Case
1694,276	849,059	2

max Vu / 0.85Vc = 0,59 < 1,00 YES

Check Punching Shear

Vc (kN)	max Vu (kN)	Load Case
2801,204	2161,750	2

max Vu / 0.85Vc = 0,908 < 1,00 YES

Results:

Base Dimensions

B =	4500	mm	d =	430	mm
L =	4500	mm	D =	500	mm

Reinforcement

	x - Direction		z - Direction
	Bottom Reinforcement	Top Reinforcement	Bottom Reinforcement
As (cm2) =	70,14490509	0	64,54195404
ρ =	0,00362506	0	0,003335502

Isolated Foundation Calculation (ACI)	Issue:	Design	Page
	Date:	0	1 of 2
Project: WoodSol eight story concrete building 245 SBC	Revised by:	Test1	
	Checked by:	Test2	

Input Data									
Loads (kN)			Mz	Concrete Strength		Foundation Properties			
	Px	Py	(kN.m)	f'c (N/mm2) =	27,6	H (mm)	540	h (mm)	0
Dead	58	1613	113	Steel Strength		hw (mm)	0	L/B	1
Live	0	0	0	fy (N/mm2) =	414	Pedestal Dimension			
Wind	0	0	0	Allowable Soil Pressure		x (mm)	500	y (mm)	500
E	0	0	0	qa (kN/m2) =	440				
Sum	58	1613	113	Base Soil angle of internal friction			20		

Ultimate Loads (ACI 9.2.)										
Load Case	0.9D+1.3W		1.4D+1.7L		0.75(1.4D+1.7L+1.7W)			0.75(1.4D+1.7L+1.87E)		
Factors	0,9	1,3	1,4	1,7	1,05	1,275	1,275	1,05	1,275	1,4025
Pux (kN) =	52,2		81,2		60,9			60,9		
Puy (kN) =	1451,7		2258,2		1693,65			1693,65		
Muz (kN.m) =	101,7		158,2		118,65			118,65		

CHECKING:

Contact Pressure

qmax =	472,6	kN/m2	qmin =	285,6	kN/m2

q_{GP} (gross pressure) (kN/m2) = (3qmax + qmin) /4

q_{GP} = 425,8 kN/m2 < qa = **440** **YES**

Stability against Overturning

Overturning moment	144,32	kN.m
Stabilizing moment	1755,29	kN.m

$$\frac{\text{Stabilizing moment}}{\text{Overturning moment}} = \quad 12{,}16 \quad > \quad 1{,}5 \quad \textbf{YES}$$

Stability against Sliding

$$\frac{Fv \times \tan\theta}{F_h} = \quad 10{,}49 \quad > \quad 1{,}5 \quad \textbf{YES}$$

Check Wide Beam Shear

Vc (kN)	max Vu (kN)	Load Case
790,662	481,664	2

max Vu / 0.85Vc = 0,717 < 1,00 YES

Check Punching Shear

Vc (kN)	max Vu (kN)	Load Case
2801,204	1815,316	2

max Vu / 0.85Vc = 0,762 < 1,00 YES

Results:

Base Dimensions

B =	**2100**	mm	d =	**430**	mm
L =	**2100**	mm	D =	**500**	mm

Reinforcement

	x - Direction		z - Direction
	Bottom Reinforcement	Top Reinforcement	Bottom Reinforcement
As (cm2) =	**26,24657059**	**0**	**21,94811058**
ρ =	**0,002906597**	**0**	**0,002430577**

Isolated Foundation Calculation (ACI)		Issue:	Design	Page
		Date:	0	1 of 2
		Revised by:	Test1	
Project:	WoodSol eight story concrete building . SBC	Checked by:	Test2	

Input Data

Loads (kN)			Mz	Concrete Strength		Foundation Properties			
	Px	Py	(kN.m)	f'c (N/mm2) =	27,6	H (mm)	540	h (mm)	0
Dead	58	1613	113	Steel Strength		hw (mm)	0	L/B	1
Live	0	0	0	fy (N/mm2) =	414	Pedestal Dimension			
Wind	0	0	0	Allowable Soil Pressure		x (mm)	500	y (mm)	500
E	0	0	0	qa (kN/m2) =	440				
Sum	58	1613	113	Base Soil angle of internal friction		20			

Ultimate Loads (ACI 9.2.)

Load Case	0.9D+1.3W		1.4D+1.7L		0.75(1.4D+1.7L+1.7W)			0.75(1.4D+1.7L+1.87E)		
Factors	0,9	1,3	1,4	1,7	1,05	1,275	1,275	1,05	1,275	1,4025
Pux (kN) =	52,2		81,2		60,9			60,9		
Puy (kN) =	1451,7		2258,2		1693,65			1693,65		
Muz (kN.m) =	101,7		158,2		118,65			118,65		

CHECKING:

Contact Pressure

q_{max} = 472,6 kN/m2 q_{min} = 285,6 kN/m2

q_{GP} (gross pressure) (kN/m2) = (3qmax + qmin) /4

q_{GP} = 425,8 kN/m2 < qa = 440 **YES**

Stability against Overturning

Overturning moment	144,32	kN.m
Stabilizing moment	1755,29	kN.m

$$\frac{\text{Stabilizing moment}}{\text{Overturning moment}} = 12{,}16 > 1{,}5$$ **YES**

Stability against Sliding

$$\frac{Fv \times \tan\theta}{F_h} = 10{,}49 > 1{,}5$$ **YES**

Check Wide Beam Shear

Vc (kN)	max Vu (kN)	Load Case
790,662	481,664	2

max Vu / 0.85Vc = 0,717 < 1,00 YES

Check Punching Shear

Vc (kN)	max Vu (kN)	Load Case
2801,204	1815,316	2

max Vu / 0.85Vc = 0,762 < 1,00 YES

Results:

Base Dimensions

B =	2100	mm	d =	430	mm
L =	2100	mm	D =	500	mm

Reinforcement

	x - Direction		z - Direction
	Bottom Reinforcement	Top Reinforcement	Bottom Reinforcement
As (cm2) =	26,24657059	0	21,94811058
ρ =	0,002906597	0	0,002430577

A.2 Dimensioning of anchor rods

Dimensioning of rebar anchors for the steelplate, starting with 8 rods S500 Ø22 before slabs are mounted

$f_y := 500 \dfrac{N}{mm^2}$

$f_{ctk} := 3.0 \dfrac{N}{mm^2}$ Assuming C45/50

$c_d := 50\ mm$ Coverage

$\varnothing := 22\ mm$ Diameter of anchor

$r := \dfrac{\varnothing}{2} = 11\ mm$

$A_{dowel} := \pi \cdot r^2 = \left(3.801 \cdot 10^{-4}\right)\ m^2$

$\gamma_{M0} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M1} := 1.05$ NS-EN 1993-1-1 NA.6.1

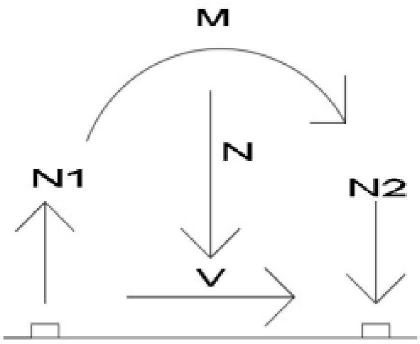
$\gamma_{M2} := 1.25$ NS-EN 1993-1-1 NA.6.1

$\gamma_c := 1.5$ NS-EN 1993-1-1 NA.2.4.2.4

$\gamma_s := 1.15$ NS-EN 1993-1-1 NA.2.4.2.4

Forces acting at in the foundation:

$$M_{ed} := 259\ kN \cdot m$$



$$N_{ed} := \frac{2.3}{8}\ kN = 0.288\ kN$$

$$V_{ed} := \frac{39}{8}\ kN$$

$$e := 490\ mm$$

$$N_{1.tension} := \frac{M_{ed}}{3\ e} + N_{ed} = 176.478\ kN$$

$$N_{2.compression} := \frac{M_{ed}}{3\ e} - N_{ed} = 175.903\ kN$$

NS-EN 1993-1-1 6.2.3 - Tension

$$N_{t.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 181.016\ kN$$

$$\frac{N_{1.tension}}{N_{t.Rd}} \le 1.0$$

$$\frac{N_{1.tension}}{N_{t.Rd}} = 0.975$$ Ok

$$N_{c.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 181.016\ \mathbf{kN}$$

$$\frac{N_{2.compression}}{N_{c.Rd}} \le 1.0$$

$$\frac{N_{2.compression}}{N_{c.Rd}} = 0.972$$ Ok

NS-EN 1993-1-1 6.2.6 - Shear

$$V_{c.Rd} := \frac{\frac{2 \cdot A_{dowel}}{\pi} \cdot \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = 66.533\ \mathbf{kN}$$

$$\frac{V_{ed}}{V_{c.Rd}} \le 1.0$$

$$\frac{V_{ed}}{V_{c.Rd}} = 0.073$$ Ok

Using eight Ø22 anchors will be ok.

NS-EN 1992-1-1 8.4.2 - Ultimate bond stress for finding anchoring dept of the rods

$$\eta_1 := 1.0$$

$$\eta_2 := \frac{(132\ mm - \o)}{100\ mm} = 1.1$$

$$f_{ctk} := 3.0\ \frac{N}{mm^2}$$ Assuming C45/50

$$f_{ctd} := 3.0\ \frac{\frac{N}{mm^2}}{\gamma_c} = 2\ \frac{N}{mm^2}$$

$$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 4.95\ \frac{N}{mm^2}$$ NS-EN 1992-1-1 (8.2)

NS-EN 1992-1-1 8.4.3 - Basic anchoring length

$$\sigma_{sd} := 500\ \frac{\frac{N}{mm^2}}{\gamma_s} = 434.783\ \frac{N}{mm^2}$$

$$l_{b.rqd} := \left(\frac{\o}{4}\right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}}\right) = 483.092\ mm$$ NS-EN 1992-1-1 (8.3)

NS-EN 1992-1-1 8.4.4 - Design anchoring length

Tension

$\alpha_1 := 0.7$ NS-EN 1992-1-1 Table 8.2

$\alpha_2 := 1 - 0.15 \cdot \frac{(c_d - \phi)}{\phi} = 0.809$

$\alpha_3 := 1.0$

$\alpha_4 := 0.7$

$\alpha_5 := 1 - 0.04 \cdot 3.0 = 0.88$

$l_{bd.tension} := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd} = 168.541\ mm$ NS-EN 1992-1-1 (8.3)

Compression

$\alpha_1 := 1.0$ NS-EN 1992-1-1 Table 8.2

$\alpha_2 := 1.0$

$\alpha_3 := 1.0$

$\alpha_4 := 0.7$

$\alpha_5 := 1.0$

$l_{bd.compression} := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd} = 338.164\ mm$ NS-EN 1992-1-1 (8.3)

Using the anchoring length needed for compression in both tension and compression.

$l_{bd\_afterbend} := 10 \cdot \phi = 220\ mm$ NS-EN 1992-1-1 8.5 (2)

Dimensioning of rebar anchors for the steelplate, starting with 4 rods S500 Ø28 before slabs are mounted

$f_y := 500 \frac{N}{mm^2}$

Assuming C45/50

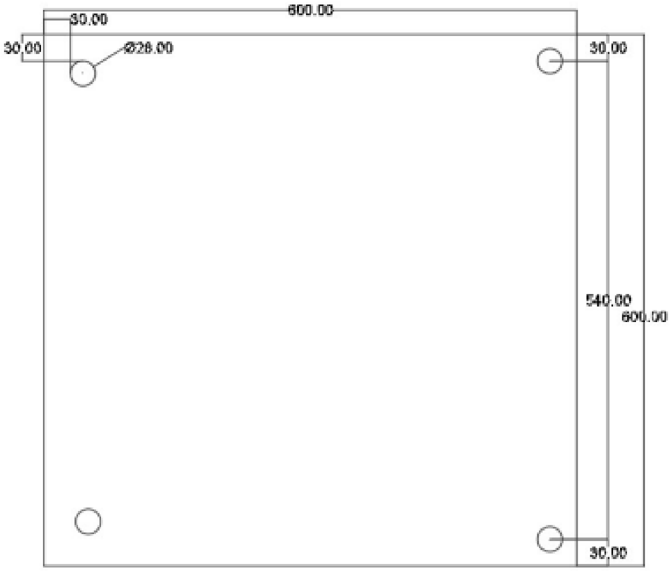
$f_{ctk} := 3.0 \frac{N}{mm^2}$

$c_d := 50\ mm$ Coverage

$\phi := 28\ mm$

$r := \frac{\phi}{2} = 14\ mm$

$A_{dowel} := \pi \cdot r^2 = \left(6.158 \cdot 10^{-4}\right) m^2$



$\gamma_{M0} := 1.05$ NS-EN 1993-1-1 NA.6.1

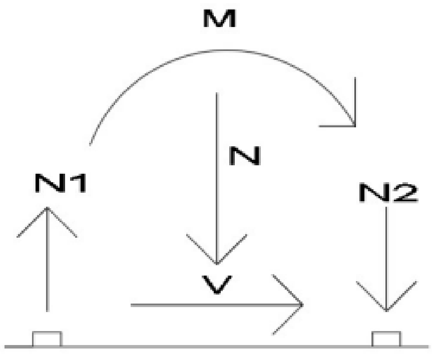
$\gamma_{M1} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M2} := 1.25$ NS-EN 1993-1-1 NA.6.1

$\gamma_c := 1.5$ NS-EN 1993-1-1 NA.2.4.2.4

$\gamma_s := 1.15$ NS-EN 1993-1-1 NA.2.4.2.4

Forces acting at in the foundation:



$$M_{ed} := 259\ kN \cdot m$$

$$N_{ed} := \frac{2.3}{2}\ kN = 1.15\ kN$$

$$V_{ed} := \frac{39}{2}\ kN$$

$$e := 490\ mm$$

$$N_{1.tension} := \frac{M_{ed}}{2\ e} + N_{ed} = 265.436\ kN$$

$$N_{2.compression} := \frac{M_{ed}}{2\ e} - N_{ed} = 263.136\ kN$$

NS-EN 1993-1-1 6.2.3 - Tension

$$N_{t.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 293.215\ kN$$

$$\frac{N_{1.tension}}{N_{t.Rd}} \le 1.0$$

$$\frac{N_{1.tension}}{N_{t.Rd}} = 0.905$$ Ok

NS-EN 1993-1-1 6.2.4 - Compression

$$N_{c.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 293.215\ kN$$

$$\frac{N_{2.compression}}{N_{c.Rd}} \le 1.0$$

$$\frac{N_{2.compression}}{N_{c.Rd}} = 0.897$$ Ok

NS-EN 1993-1-1 6.2.6 - Shear

$$V_{c.Rd} := \frac{\frac{2 \cdot A_{dowel}}{\pi} \cdot \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = 107.772 \ kN$$

$$\frac{V_{ed}}{V_{c.Rd}} \leq 1.0$$

$$\frac{V_{ed}}{V_{c.Rd}} = 0.181$$ Ok

Using four Ø28 anchors will be ok.

NS-EN 1992-1-1 8.4.2 - Ultimate bond stress for finding anchoring dept of the rods

$$\eta_1 := 1.0$$

$$\eta_2 := \frac{(132 \ mm - \phi)}{100 \ mm} = 1.04$$

$$f_{ctk} := 3.0 \ \frac{N}{mm^2}$$ Assuming C55/60

$$f_{ctd} := \frac{3.0 \ \frac{N}{mm^2}}{\gamma_c} = 2 \ \frac{N}{mm^2}$$

$$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 4.68 \ \frac{N}{mm^2}$$ NS-EN 1992-1-1 (8.2)

NS-EN 1992-1-1 8.4.3 - Basic anchoring length

$$\sigma_{sd} := 500 \frac{\frac{N}{mm^2}}{\gamma_s} = 434.783 \frac{N}{mm^2}$$

$$l_{b.rqd} := \left(\frac{\phi}{4}\right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}}\right) = 650.316\ mm$$ NS-EN 1992-1-1 (8.3)

NS-EN 1992-1-1 8.4.4 - Design anchoring length

Tension

$$\alpha_1 := 0.7$$ NS-EN 1992-1-1 Table 8.2

$$\alpha_2 := 1 - 0.15 \cdot \frac{(c_d - \phi)}{\phi} = 0.882$$

$$\alpha_3 := 1.0$$

$$\alpha_4 := 0.7$$

$$\alpha_5 := 1 - 0.04 \cdot 3.0 = 0.88$$

$$l_{bd.tension} := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd} = 247.367\ mm$$ NS-EN 1992-1-1 (8.3)

Compression

$$\alpha_1 := 1.0$$ NS-EN 1992-1-1 Table 8.2

$$\alpha_2 := 1.0$$

$$\alpha_3 := 1.0$$

$$\alpha_4 := 0.7$$

$$\alpha_5 := 1.0$$

$$l_{bd.compression} := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd} = 455.221\ mm$$ NS-EN 1992-1-1 (8.3)

Using the anchoring length needed for compression in both tension and compression.

$$l_{bd\_afterbend} := 10 \cdot \phi = 280\ mm$$ NS-EN 1992-1-1 8.5 (2)

A.3 Dimensioning of anchor bolts

Dimensioning of steel bolts for the steelplate, starting with 8 rods S500 Ø22 before slabs are mounted

$f_y := 500 \frac{N}{mm^2}$

$f_{ctk} := 3.0 \frac{N}{mm^2}$ Assuming C45/50

$f_{ck.cube} := 45 \frac{N}{mm^2}$

$\varnothing := 22\ mm$ Diameter of anchors

$r := \frac{\varnothing}{2}$

$A_{dowel} := \pi \cdot r^2 = \left(3.801 \cdot 10^{-4}\right) m^2$

$n := 8$ Number of anchors

$\gamma_{M0} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M1} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M2} := 1.25$ NS-EN 1993-1-1 NA.6.1

$\gamma_c := 1.5$ NS-EN 1993-1-1 NA.2.4.2.4

$\gamma_s := 1.15$ NS-EN 1993-1-1 NA.2.4.2.4

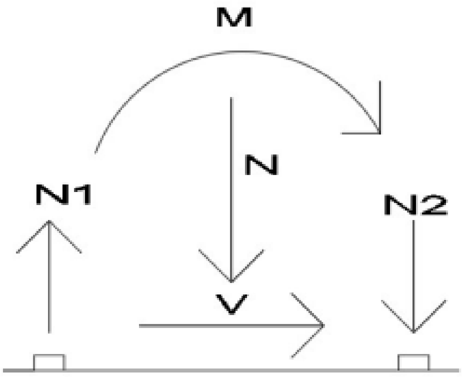
Forces acting at in the foundation before decks are mounted:

$$M_{ed} := 259\ kN \cdot m$$

$$N_{ed} := \frac{2.3}{n}\ kN = 0.288\ kN$$

$$V_{ed} := \frac{39}{n}\ kN$$

$$e := 490\ mm$$



$$N_{1.tension} := \frac{M_{ed}}{3\ e} - N_{ed} = 175.903\ kN$$

$$N_{2.compression} := \frac{M_{ed}}{3\ e} + N_{ed} = 176.478\ kN$$

NS-EN 1993-1-1 6.2.3 - Tension

$$N_{t.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 181.016\ kN$$

$$\frac{N_{1.tension}}{N_{t.Rd}} \le 1.0$$

$$\frac{N_{1.tension}}{N_{t.Rd}} = 0.972$$ Ok

NS-EN 1993-1-1 6.2.4 - Compression

$$N_{c.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 181.016\ kN$$

$$\frac{N_{2.compression}}{N_{c.Rd}} \le 1.0$$

$$\frac{N_{2.compression}}{N_{c.Rd}} = 0.975$$ Ok

NS-EN 1993-1-1 6.2.6 - Shear

$$V_{c.Rd} := \frac{\frac{2 \cdot A_{dowel}}{\pi} \cdot \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = 66.533 \ kN$$

$$\frac{V_{ed}}{V_{c.Rd}} \leq 1.0$$

$$\frac{V_{ed}}{V_{c.Rd}} = 0.073$$ Ok

It is necessary with eight anchor bolts with a diameter of 22 millimeters

Using a depth of 4,5 times the diameter of the dowels Pry out crackings will not occur, using 5 times the diameter should be a somewhat realistic and semi conservative approach with the use of a diameter of 40 mm. [Kapasitet til stålinnstøpningsdetaljer i betong]

$$l_{anchor} := 5 \cdot \phi = 110 \ mm$$

To assure that the dowel wont be pulled out the foot of the size of the dowel foot needs to be calculated. This can be done using CEN/TS 2-4-2 (6.2.4) [Kapasitet til stålinnstøpningsdetaljer i betong]

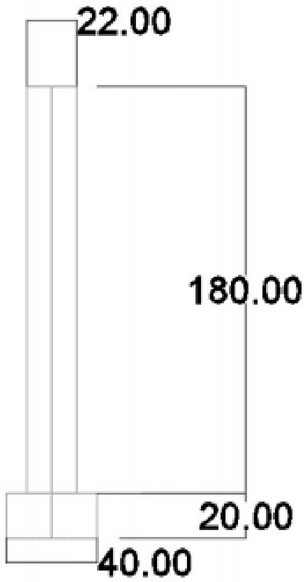
$$d_h := 40 \ mm$$ d_h is the diameter of the dowels foot.

$$A_h := \left(\frac{\pi}{4}\right) \cdot \left(d_h^2 - \phi^2\right) = 876.504 \ mm^2$$

$$\psi_{ucr.N} := 1.4$$

$$N_{Rd.p} := \frac{6}{\gamma_c} \cdot A_h \cdot f_{ck.cube} \cdot \psi_{ucr.N} = 220.879 \ kN$$

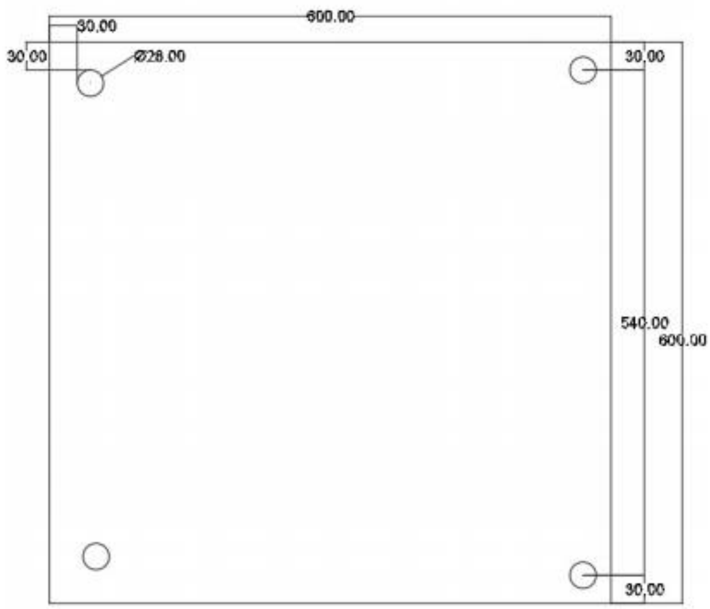
$$\frac{N_{1.tension}}{N_{Rd.p}} = 0.796$$ ok



Using a eight anchor bolts with a diameter of 22 mm, a depth of 200 mm and a foot of 40 mm with a height of the foot of approximatly 20 mm should be sufficient.

Dimensioning of dowels for the steelplate, using 4 rods S500 Ø28

$f_y := 500 \frac{N}{mm^2}$



Assuming C45/50

$f_{ctk} := 3.0 \frac{N}{mm^2}$

$f_{ck.cube} := 45 \frac{N}{mm^2}$

$c_d := 50\ mm$

$\phi := 28\ mm$

$r := \frac{\phi}{2}$

$A_{dowel} := \pi \cdot r^2 = \left(6.158 \cdot 10^{-4}\right)\ m^2$

$\gamma_{M0} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M1} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M2} := 1.25$ NS-EN 1993-1-1 NA.6.1

$\gamma_c := 1.5$ NS-EN 1993-1-1 NA.2.4.2.4

$\gamma_s := 1.15$ NS-EN 1993-1-1 NA.2.4.2.4

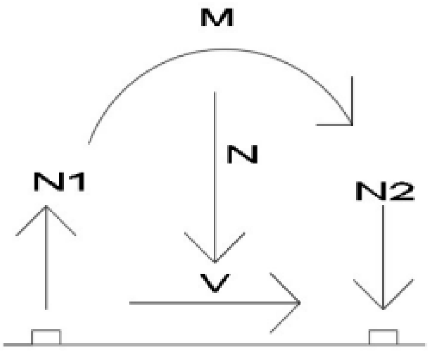
Forces acting at in the foundation before slabs:

$$M_{ed} := 259\ kN \cdot m$$

$$N_{ed} := \frac{2.3}{4}\ kN = 0.575\ kN$$

$$V_{ed} := \frac{39}{4}\ kN$$

$$e := 490\ mm$$



$$N_{1.tension} := \frac{M_{ed}}{2\ e} - N_{ed} = 263.711\ kN$$

$$N_{2.compression} := \frac{M_{ed}}{2\ e} + N_{ed} = 264.861\ kN$$

NS-EN 1993-1-1 6.2.3 - Tension

$$N_{t.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 293.215\ kN$$

$$\frac{N_{1.tension}}{N_{t.Rd}} \leq 1.0$$

$$\frac{N_{1.tension}}{N_{t.Rd}} = 0.899$$ Ok

NS-EN 1993-1-1 6.2.4 - Compression

$$N_{c.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 293.215\ kN$$

$$\frac{N_{2.compression}}{N_{c.Rd}} \leq 1.0$$

$$\frac{N_{2.compression}}{N_{c.Rd}} = 0.903$$ Ok

NS-EN 1993-1-1 6.2.6 - Shear

$$V_{c.Rd} := \frac{\frac{2 \cdot A_{dowel}}{\pi} \cdot \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = 107.772 \ kN$$

$$\frac{V_{ed}}{V_{c.Rd}} \leq 1.0$$

$$\frac{V_{ed}}{V_{c.Rd}} = 0.09$$ Ok

It is necessary with four anchor bolts with a diameter of 28 millimeters

Using a depth of 4,5 times the diameter of the dowels Pry out crackings will not occur, using 5 times the diameter should be a somewhat realistic and semi conservative approach with the use of a diameter of 40 mm. [Kapasitet til stålinnstøpningsdetaljer i betong]

$$l_{anchor} := 5 \cdot ø = 140 \ mm$$

To assure that the dowel wont be pulled out the foot of the size of the dowel foot needs to be calculated. This can be done using CEN/TS 2-4-2 (6.2.4) [Kapasitet til stålinnstøpningsdetaljer i betong]

$$d_h := 50 \ mm$$

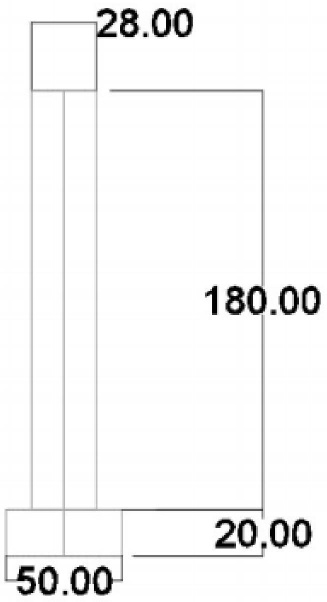
d_h is the diameter of the dowels foot.
ø is the diameter of the dowels rod.

$$A_h := \left(\frac{\pi}{4}\right) \cdot \left(d_h^{\ 2} - ø^2\right) = \left(1.348 \cdot 10^3\right) \ mm^2$$

$$\psi_{ucr.N} := 1.4$$

$$N_{Rd.p} := \frac{6}{\gamma_c} \cdot A_h \cdot f_{ck.cube} \cdot \psi_{ucr.N} = 339.631 \ kN$$

$$\frac{N_{1.tension}}{N_{Rd.p}} = 0.776$$ ok



Using four anchor bolts with a diameter of 28 mm, a depth of 200 mm and a foot of 50 mm with a height of the foot of approximatly 20 mm should be sufficient.

Dimensioning of dowels for the steelplate, starting with 8 rods S500 Ø22 after slabs are mounted

Calculations of dowels will be only a fast approximation to get a length that is somewhat realistic.

$$f_y := 500 \frac{N}{mm^2}$$

$$f_{ctk} := 3.0 \frac{N}{mm^2}$$ Assuming C45/50

$$f_{ck.cube} := 45 \frac{N}{mm^2}$$

$$c_d := 50 \ mm$$

$$\phi := 22 \ mm$$

$$r := \frac{\phi}{2}$$

$$A_{dowel} := \pi \cdot r^2 = \left(3.801 \cdot 10^{-4}\right) m^2$$

$\gamma_{M0} := 1.05$ NS-EN 1993-1-1 NA.6.1

$\gamma_{M1} := 1.05$ NS-EN 1993-1-1 NA.6.1

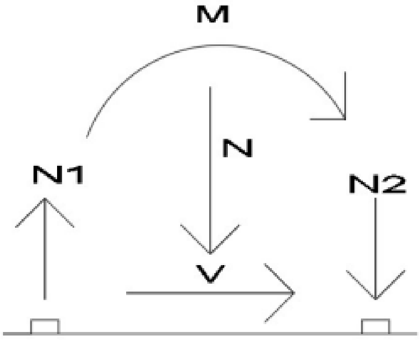
$\gamma_{M2} := 1.25$ NS-EN 1993-1-1 NA.6.1

$\gamma_c := 1.5$ NS-EN 1993-1-1 NA.2.4.2.4

$\gamma_s := 1.15$ NS-EN 1993-1-1 NA.2.4.2.4

Forces acting at in the foundation before slabs:

$$M_{ed} := 102\ kN \cdot m$$



$$N_{ed} := \frac{695}{8}\ kN = 86.875\ kN$$

$$V_{ed} := \frac{58}{8}\ kN$$

$$e := 490\ mm$$

$$N_{1.tension} := \frac{M_{ed}}{3\ e} - N_{ed} = -17.487\ kN$$

$$N_{2.compression} := \frac{M_{ed}}{3\ e} + N_{ed} = 156.263\ kN$$

NS-EN 1993-1-1 6.2.3 - Tension

$$N_{t.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 181.016\ kN$$

$$\frac{N_{1.tension}}{N_{t.Rd}} \leq 1.0$$

$$\frac{N_{1.tension}}{N_{t.Rd}} = -0.097$$ Ok

NS-EN 1993-1-1 6.2.4 - Compression

$$N_{c.Rd} := \frac{A_{dowel} \cdot f_y}{\gamma_{M0}} = 181.016\ kN$$

$$\frac{N_{2.compression}}{N_{c.Rd}} \leq 1.0$$

$$\frac{N_{2.compression}}{N_{c.Rd}} = 0.863$$ Ok

NS-EN 1993-1-1 6.2.6 - Shear

$$V_{c.Rd} := \frac{\frac{2 \cdot A_{dowel}}{\pi} \cdot \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = 66.533\ kN$$

$$\frac{V_{ed}}{V_{c.Rd}} \le 1.0$$

$$\frac{V_{ed}}{V_{c.Rd}} = 0.109$$ Ok

eight anchor bolts with a diameter of 22 millimeters is no problem

Using a depth of 4,5 times the diameter of the dowels Pry out crackings will not occur, using 5 times the diameter should be a somewhat realistic and semi conservative approach with the use of a diameter of 40 mm. [Kapasitet til stålinnstøpningsdetaljer i betong]

$$l_{anchor} := 5 \cdot \phi = 110\ mm$$

To assure that the dowel wont be pulled out the foot of the size of the dowel foot needs to be calculated. This can be done using CEN/TS 2-4-2 (6.2.4) [Kapasitet til stålinnstøpningsdetaljer i betong]

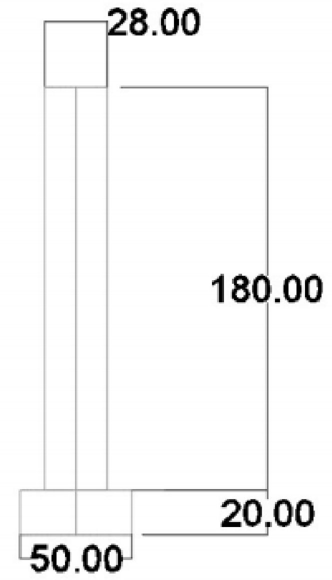
$$d_h := 40\ mm$$ d_h is the diameter of the dowels foot.

$$A_h := \left(\frac{\pi}{4}\right) \cdot \left(d_h^{\,2} - \phi^2\right) = 876.504\ mm^2$$

$$\psi_{ucr.N} := 1.4$$

$$N_{Rd.p} := \frac{6}{\gamma_c} \cdot A_h \cdot f_{ck.cube} \cdot \psi_{ucr.N} = 220.879\ kN$$

$$\frac{N_{1.tension}}{N_{Rd.p}} = -0.079$$ ok



Using a eight anchor bolts with a diameter of 22 mm, a depth of 200 mm and a foot of 40 mm with a height of the foot of approximatly 20 mm should be sufficient.

A.4 Dimensioning of steel rods

Capacity of the rods from foundation into timber columns

$d := 26\ mm$ $l := 1200\ mm$ $f_y := 500\ \frac{N}{mm^2}$ $f_u := 490\ \frac{N}{mm^2}$

$A := \pi \cdot \left(\frac{d}{2}\right)^2 = 530.929\ mm^2$ $\gamma_{M0} := 1.05$ $n := 6$ $n_{ef} := n^{0.9} = 5.016$

$M_{Ed} := 259\ kN \cdot m$

Tension of rods in frame direction:

$$N_{t.Ed} := \frac{M_{Ed}}{0.37\ m} = 700\ kN$$

$$N_{t.Ed.per.rod} := \frac{N_{t.Ed}}{0.5 \cdot n} = 233.333\ kN$$

Ok

$$N_{pl.Rd} := A \cdot \frac{f_y}{\gamma_{M0}} = 252.823\ kN$$ NS-EN 1993-1-1 6.2.3

Tension of rods in other direction:

$z_1 := 370\ mm$ $z_2 := 296\ mm$ $z_3 := 222\ mm$

The fractions of M add up to M

$$N_{1.Ed} := \frac{\frac{M_{Ed}}{2}}{z_1} = 350\ kN$$ Not OK

$$N_{2.Ed} := \frac{\frac{M_{Ed}}{3}}{z_2} = 291.667\ kN$$ Not OK

$$N_{3.Ed} := \frac{\frac{M_{Ed}}{6}}{z_3} = 194.444\ kN$$ OK

$$N_{pl.Rd} := A \cdot \frac{f_y}{\gamma_{M0}} = 252.823\ kN$$ NS-EN 1993-1-1 6.2.3

A.5 Spreadsheet for stiffnesses

Axial stiffness of steel anchors	Area (mm^2)	E-steel (MPa)	Length of dowel (mm)	I (mm^4)	Axial stiffness/anchor Kax.bolt (N/mm)
Diameter of anchors (mm)	PI*r^2			(PI*d^4)/64	AE/L
10	78,5	2,10E+05	200	490,9	8,25E+04
12	113,1	2,10E+05	200	1017,9	1,19E+05
14	153,9	2,10E+05	200	1885,7	1,62E+05
16	201,1	2,10E+05	200	3217,0	2,11E+05
18	254,5	2,10E+05	200	5153,0	2,67E+05
20	314,2	2,10E+05	200	7854,0	3,30E+05
22	380,1	2,10E+05	200	11499,0	3,99E+05
24	452,4	2,10E+05	200	16286,0	4,75E+05
26	530,9	2,10E+05	200	22431,8	5,57E+05
28	615,8	2,10E+05	200	30171,9	6,47E+05
30	706,9	2,10E+05	200	39760,8	7,42E+05
32	804,2	2,10E+05	200	51471,9	8,44E+05
34	907,9	2,10E+05	200	65597,2	9,53E+05
36	1017,9	2,10E+05	200	82448,0	1,07E+06
38	1134,1	2,10E+05	200	102353,9	1,19E+06
40	1256,6	2,10E+05	200	125663,7	1,32E+06

Axial stiffness of steel anchors	Area (mm^2)	E-steel (MPa)	Length of dowel (mm)	I (mm^4)	Axial stiffness/anchor Kax.bolt (N/mm)
Diameter of anchors (mm)	PI*r^2			(PI*d^4)/64	AE/L
10	78,5	2,10E+05	250	490,9	6,60E+04
12	113,1	2,10E+05	250	1017,9	9,50E+04
14	153,9	2,10E+05	250	1885,7	1,29E+05
16	201,1	2,10E+05	250	3217,0	1,69E+05
18	254,5	2,10E+05	250	5153,0	2,14E+05
20	314,2	2,10E+05	250	7854,0	2,64E+05
22	380,1	2,10E+05	250	11499,0	3,19E+05
24	452,4	2,10E+05	250	16286,0	3,80E+05
26	530,9	2,10E+05	250	22431,8	4,46E+05
28	615,8	2,10E+05	250	30171,9	5,17E+05
30	706,9	2,10E+05	250	39760,8	5,94E+05
32	804,2	2,10E+05	250	51471,9	6,76E+05
34	907,9	2,10E+05	250	65597,2	7,63E+05
36	1017,9	2,10E+05	250	82448,0	8,55E+05
38	1134,1	2,10E+05	250	102353,9	9,53E+05
40	1256,6	2,10E+05	250	125663,7	1,06E+06

Axial stiffness of steel anchors	Area (mm^2)	E-steel (MPa)	Length of dowel (mm)	I (mm^4)	Axial stiffness/anchor Kax.bolt (N/mm)
Diameter of anchors (mm)	PI*r^2			(PI*d^4)/64	AE/L
10	78,5	2,10E+05	300	490,9	5,50E+04
12	113,1	2,10E+05	300	1017,9	7,92E+04
14	153,9	2,10E+05	300	1885,7	1,08E+05
16	201,1	2,10E+05	300	3217,0	1,41E+05
18	254,5	2,10E+05	300	5153,0	1,78E+05
20	314,2	2,10E+05	300	7854,0	2,20E+05
22	380,1	2,10E+05	300	11499,0	2,66E+05
24	452,4	2,10E+05	300	16286,0	3,17E+05
26	530,9	2,10E+05	300	22431,8	3,72E+05
28	615,8	2,10E+05	300	30171,9	4,31E+05
30	706,9	2,10E+05	300	39760,8	4,95E+05
32	804,2	2,10E+05	300	51471,9	5,63E+05
34	907,9	2,10E+05	300	65597,2	6,36E+05
36	1017,9	2,10E+05	300	82448,0	7,13E+05
38	1134,1	2,10E+05	300	102353,9	7,94E+05
40	1256,6	2,10E+05	300	125663,7	8,80E+05

Vertical stiffness of steel anchors	Area (mm^2)	E-steel (MPa)	Length of dowel (mm)	I (mm^4)	Vertical stiffness/anchor Kv.anchor (N/mm)
Diameter of anchors (mm)	PI*r^2			(PI*d^4)/64	12EI/L^3
10	78,5	2,10E+05	200	490,9	1,55E+02
12	113,1	2,10E+05	200	1017,9	3,21E+02
14	153,9	2,10E+05	200	1885,7	5,94E+02
16	201,1	2,10E+05	200	3217,0	1,01E+03
18	254,5	2,10E+05	200	5153,0	1,62E+03
20	314,2	2,10E+05	200	7854,0	2,47E+03
22	380,1	2,10E+05	200	11499,0	3,62E+03
24	452,4	2,10E+05	200	16286,0	5,13E+03
26	530,9	2,10E+05	200	22431,8	7,07E+03
28	615,8	2,10E+05	200	30171,9	9,50E+03
30	706,9	2,10E+05	200	39760,8	1,25E+04
32	804,2	2,10E+05	200	51471,9	1,62E+04
34	907,9	2,10E+05	200	65597,2	2,07E+04
36	1017,9	2,10E+05	200	82448,0	2,60E+04
38	1134,1	2,10E+05	200	102353,9	3,22E+04
40	1256,6	2,10E+05	200	125663,7	3,96E+04

Vertical stiffness of steel anchors	Area (mm^2)	E-steel (MPa)	Length of dowel (mm)	I (mm^4)	Vertical stiffness/anchor Kv.anchor (N/mm)
Diameter of anchors (mm)	PI*r^2			(PI*d^4)/64	12EI/L^3
10	78,5	2,10E+05	250	490,9	7,92E+01
12	113,1	2,10E+05	250	1017,9	1,64E+02
14	153,9	2,10E+05	250	1885,7	3,04E+02
16	201,1	2,10E+05	250	3217,0	5,19E+02
18	254,5	2,10E+05	250	5153,0	8,31E+02
20	314,2	2,10E+05	250	7854,0	1,27E+03
22	380,1	2,10E+05	250	11499,0	1,85E+03
24	452,4	2,10E+05	250	16286,0	2,63E+03
26	530,9	2,10E+05	250	22431,8	3,62E+03
28	615,8	2,10E+05	250	30171,9	4,87E+03
30	706,9	2,10E+05	250	39760,8	6,41E+03
32	804,2	2,10E+05	250	51471,9	8,30E+03
34	907,9	2,10E+05	250	65597,2	1,06E+04
36	1017,9	2,10E+05	250	82448,0	1,33E+04
38	1134,1	2,10E+05	250	102353,9	1,65E+04
40	1256,6	2,10E+05	250	125663,7	2,03E+04

Vertical stiffness of steel anchors	Area (mm^2)	E-steel (MPa)	Length of dowel (mm)	I (mm^4)	Vertical stiffness/anchor Kv.anchor (N/mm)
Diameter of anchors (mm)	PI*r^2			(PI*d^4)/64	12EI/L^3
10	78,5	2,10E+05	300	490,9	4,58E+01
12	113,1	2,10E+05	300	1017,9	9,50E+01
14	153,9	2,10E+05	300	1885,7	1,76E+02
16	201,1	2,10E+05	300	3217,0	3,00E+02
18	254,5	2,10E+05	300	5153,0	4,81E+02
20	314,2	2,10E+05	300	7854,0	7,33E+02
22	380,1	2,10E+05	300	11499,0	1,07E+03
24	452,4	2,10E+05	300	16286,0	1,52E+03
26	530,9	2,10E+05	300	22431,8	2,09E+03
28	615,8	2,10E+05	300	30171,9	2,82E+03
30	706,9	2,10E+05	300	39760,8	3,71E+03
32	804,2	2,10E+05	300	51471,9	4,80E+03
34	907,9	2,10E+05	300	65597,2	6,12E+03
36	1017,9	2,10E+05	300	82448,0	7,70E+03

Stiffness without the free length in the steel plate.	Axial stiffness of steel rods	Area (mm^2)	E-steel (MPa)	Length of rod (mm)	I (mm^4)	Re a=10	Axial stiffnes/rod Kax.rod (N/mm)
	Diameter of threaded rods (mm)	PI*r^2			(PI*d^4)/64	9.65/(1,5Sin(a)^2.2+Cos(a)^2.2)	0,85*(Pi()*d*Re*A*E
	10	78,5	2,10E+05	1200	490,9	9,66	6,01E+04
	12	113,1	2,10E+05	1200	1017,9	9,66	7,90E+04
	14	153,9	2,10E+05	1200	1885,7	9,66	9,96E+04
	16	201,1	2,10E+05	1200	3217,0	9,66	1,22E+05
	18	254,5	2,10E+05	1200	5153,0	9,66	1,45E+05
	20	314,2	2,10E+05	1200	7854,0	9,66	1,70E+05
	22	380,1	2,10E+05	1200	11499,0	9,66	1,96E+05
	24	452,4	2,10E+05	1200	16286,0	9,66	2,24E+05
	26	530,9	2,10E+05	1200	22431,8	9,66	2,52E+05
	28	615,8	2,10E+05	1200	30171,9	9,66	2,82E+05
	30	706,9	2,10E+05	1200	39760,8	9,66	3,12E+05
	32	804,2	2,10E+05	1200	51471,9	9,66	3,44E+05
	34	907,9	2,10E+05	1200	65597,2	9,66	3,77E+05
	36	1017,9	2,10E+05	1200	82448,0	9,66	4,11E+05
	38	1134,1	2,10E+05	1200	102353,9	9,66	4,45E+05
	40	1256,6	2,10E+05	1200	125663,7	9,66	4,81E+05

Axial stiffness of steel rods 5 degrees incline	Area (mm^2)	E-steel (MPa)	Length of rod (mm)	I (mm^4)	Re a=5	Axial stiffnes/rod Kax.rod (N/mm)	Axial stiffness free length Kax.0 (N/mm)	Total stiffness (N/mm)
Diameter of threaded rods (mm)	PI*r^2			(PI*d^4)/64	9.65/(1,5Sin(a)^2.2+Cos(a)^2.2)	0,85*(Pi()*d*Re*A*E	A*E/L0	Kax.tot=Kax.rod*Kax.0/Kax.rod+Kax.0
10	78,5	2,10E+05	1200	490,9	9,663	6,01E+04	4,71E+05	5,33E+04
12	113,1	2,10E+05	1200	1017,9	9,663	7,90E+04	4,66E+05	6,76E+04
14	153,9	2,10E+05	1200	1885,7	9,663	9,96E+04	6,34E+05	8,61E+04
16	201,1	2,10E+05	1200	3217,0	9,663	1,22E+05	8,28E+05	1,06E+05
18	254,5	2,10E+05	1200	5153,0	9,663	1,45E+05	1,05E+06	1,28E+05
20	314,2	2,10E+05	1200	7854,0	9,663	1,70E+05	1,29E+06	1,50E+05
22	380,1	2,10E+05	1200	11499,0	9,663	1,96E+05	1,57E+06	1,74E+05
24	452,4	2,10E+05	1200	16286,0	9,663	2,24E+05	1,86E+06	2,00E+05
26	530,9	2,10E+05	1200	22431,8	9,663	2,52E+05	2,19E+06	2,26E+05
28	615,8	2,10E+05	1200	30171,9	9,663	2,82E+05	2,54E+06	2,54E+05
30	706,9	2,10E+05	1200	39760,8	9,663	3,12E+05	2,91E+06	2,82E+05
32	804,2	2,10E+05	1200	51471,9	9,663	3,44E+05	3,31E+06	3,12E+05
34	907,9	2,10E+05	1200	65597,2	9,663	3,77E+05	3,74E+06	3,42E+05
36	1017,9	2,10E+05	1200	82448,0	9,663	4,11E+05	4,19E+06	3,74E+05
38	1134,1	2,10E+05	1200	102353,9	9,663	4,45E+05	4,67E+06	4,07E+05
40	1256,6	2,10E+05	1200	125663,7	9,663	4,81E+05	5,17E+06	4,40E+05

Axial stiffness of steel rods 10 degrees incline	Area (mm^2)	E-steel (MPa)	Length of rod (mm)	I (mm^4)	Re a=10	Axial stiffnes/rod Kax.rod (N/mm)	Axial stiffness free length Kax.0 (N/mm)	Total stiffness (N/mm)
Diameter of threaded rods (mm)	PI*r^2			(PI*d^4)/64	9.65/(1,5Sin(a)^2.2+Cos(a)^2.2)	0,85*(Pi()*d*Re*A*E	A*E/L0	Kax.tot=Kax.rod*Kax.0/Kax.rod+Kax.0
10	78,5	2,10E+05	1200	490,9	9,662	6,01E+04	4,71E+05	5,33E+04
12	113,1	2,10E+05	1200	1017,9	9,662	7,90E+04	4,66E+05	6,76E+04
14	153,9	2,10E+05	1200	1885,7	9,662	9,96E+04	6,34E+05	8,61E+04
16	201,1	2,10E+05	1200	3217,0	9,662	1,22E+05	8,28E+05	1,06E+05
18	254,5	2,10E+05	1200	5153,0	9,662	1,45E+05	1,05E+06	1,28E+05
20	314,2	2,10E+05	1200	7854,0	9,662	1,70E+05	1,29E+06	1,50E+05
22	380,1	2,10E+05	1200	11499,0	9,662	1,96E+05	1,57E+06	1,74E+05
24	452,4	2,10E+05	1200	16286,0	9,662	2,24E+05	1,86E+06	2,00E+05
26	530,9	2,10E+05	1200	22431,8	9,662	2,52E+05	2,19E+06	2,26E+05
28	615,8	2,10E+05	1200	30171,9	9,662	2,82E+05	2,54E+06	2,54E+05
30	706,9	2,10E+05	1200	39760,8	9,662	3,12E+05	2,91E+06	2,82E+05

32	804,2	2,10E+05	1200	51471,9	9,662	3,44E+05	3,31E+06	3,12E+05
34	907,9	2,10E+05	1200	65597,2	9,662	3,77E+05	3,74E+06	3,42E+05
36	1017,9	2,10E+05	1200	82448,0	9,662	4,11E+05	4,19E+06	3,74E+05
38	1134,1	2,10E+05	1200	102353,9	9,662	4,45E+05	4,67E+06	4,07E+05
40	1256,6	2,10E+05	1200	125663,7	9,662	4,81E+05	5,17E+06	4,40E+05

Vertical stiffness of steel rods 5 degrees incline	Area (mm^2)	E-steel (MPa)	Length of rod (mm)	I (mm^4)	Re a=5	Vertical stiffnes/rod Kv.rod (N/mm)	Vertical stiffness free length Kv.0 (N/mm)	Total stiffness (N/mm)
Diameter of threaded rods (mm)	PI*r^2			(PI*d^4)/64	9.65/(1,5Sin(a)^2 .2+Cos(a)^2.2)	2/23*Pm^1.5	3EI/L0^3	Kv.tot=Kv.rod *Kv.0/Kv.rod +Kv.0
10	78,5	2,10E+05	1200	490,9	9,663	7,75E+03	2,33E+03	1,79E+03
12	113,1	2,10E+05	1200	1017,9	9,663	9,30E+03	4,83E+03	3,18E+03
14	153,9	2,10E+05	1200	1885,7	9,663	1,09E+04	8,96E+03	4,91E+03
16	201,1	2,10E+05	1200	3217,0	9,663	1,24E+04	1,53E+04	6,85E+03
18	254,5	2,10E+05	1200	5153,0	9,663	1,40E+04	2,45E+04	8,89E+03
20	314,2	2,10E+05	1200	7854,0	9,663	1,55E+04	3,73E+04	1,10E+04
22	380,1	2,10E+05	1200	11499,0	9,663	1,71E+04	5,46E+04	1,30E+04
24	452,4	2,10E+05	1200	16286,0	9,663	1,86E+04	7,73E+04	1,50E+04
26	530,9	2,10E+05	1200	22431,8	9,663	2,02E+04	1,07E+05	1,70E+04
28	615,8	2,10E+05	1200	30171,9	9,663	2,17E+04	1,43E+05	1,89E+04
30	706,9	2,10E+05	1200	39760,8	9,663	2,33E+04	1,89E+05	2,07E+04
32	804,2	2,10E+05	1200	51471,9	9,663	2,48E+04	2,44E+05	2,25E+04
34	907,9	2,10E+05	1200	65597,2	9,663	2,64E+04	3,12E+05	2,43E+04
36	1017,9	2,10E+05	1200	82448,0	9,663	2,79E+04	3,92E+05	2,61E+04
38	1134,1	2,10E+05	1200	102353,9	9,663	2,95E+04	4,86E+05	2,78E+04
40	1256,6	2,10E+05	1200	125663,7	9,663	3,10E+04	5,97E+05	2,95E+04

Vertical stiffness of steel rods 10 degrees incline	Area (mm^2)	E-steel (MPa)	Length of rod (mm)	I (mm^4)	Re a=10	Vertical stiffnes/rod Kv.rod (N/mm)	Vertical stiffness free length Kv.0 (N/mm)	Total stiffness (N/mm)
Diameter of threaded rods (mm)	PI*r^2			(PI*d^4)/64	9.65/(1,5Sin(a)^2 .2+Cos(a)^2.2)	2/23*Pm^1.5	3EI/L0^3	Kv.tot=Kv.rod *Kv.0/Kv.rod +Kv.0
10	78,5	2,10E+05	1200	490,9	9,662	7,75E+03	2,33E+03	1,79E+03
12	113,1	2,10E+05	1200	1017,9	9,662	9,30E+03	4,83E+03	3,18E+03
14	153,9	2,10E+05	1200	1885,7	9,662	1,09E+04	8,96E+03	4,91E+03
16	201,1	2,10E+05	1200	3217,0	9,662	1,24E+04	1,53E+04	6,85E+03
18	254,5	2,10E+05	1200	5153,0	9,662	1,40E+04	2,45E+04	8,89E+03
20	314,2	2,10E+05	1200	7854,0	9,662	1,55E+04	3,73E+04	1,10E+04
22	380,1	2,10E+05	1200	11499,0	9,662	1,71E+04	5,46E+04	1,30E+04
24	452,4	2,10E+05	1200	16286,0	9,662	1,86E+04	7,73E+04	1,50E+04
26	530,9	2,10E+05	1200	22431,8	9,662	2,02E+04	1,07E+05	1,70E+04
28	615,8	2,10E+05	1200	30171,9	9,662	2,17E+04	1,43E+05	1,89E+04
30	706,9	2,10E+05	1200	39760,8	9,662	2,33E+04	1,89E+05	2,07E+04
32	804,2	2,10E+05	1200	51471,9	9,662	2,48E+04	2,44E+05	2,25E+04
34	907,9	2,10E+05	1200	65597,2	9,662	2,64E+04	3,12E+05	2,43E+04
36	1017,9	2,10E+05	1200	82448,0	9,662	2,79E+04	3,92E+05	2,61E+04
38	1134,1	2,10E+05	1200	102353,9	9,662	2,95E+04	4,86E+05	2,78E+04
40	1256,6	2,10E+05	1200	125663,7	9,662	3,10E+04	5,97E+05	2,95E+04

ABAQUS Referance model sizes	Referance model sizes
Soil stiffness (kN/m^3)	20
Plate thickness (mm)	20
Diameter dowel (mm)	32
Number of dowels	8
Length of dowels (mm)	200
Diameter rods (mm)	22
Number of rods	6
Top deflection (mm)	33,25
Degrees (Rad)	0,00317
Rotational stiffness (kNm/Rad)	3315,80

ABAQUS Varying soil stiffness		Referance model sizes		
Soil stiffness (kN/m^3)	5	20	60	100
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	32	32	32	32
Number of dowels	8	8	8	8
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	22	22	22	22
Number of rods	6	6	6	6
Top deflection (mm)	66,18	33,25	25,92	24,45
Degrees (Rad)	0,00630	0,00317	0,00247	0,00233
Rotational stiffness (kNm/Rad)	1665,93	3315,80	4253,48	4509,21

ABAQUS Varying plate thickness		Referance model sizes				
Soil stiffness (kN/m^3)	20	20	20	20	20	20
Plate thickness (mm)	10	20	35	50	70	100
Diameter dowel (mm)	12	20	32	40	40	40
Number of dowels	8	8	8	8	8	8
Length of dowels (mm)	200	200	200	200	200	200
Diameter rods (mm)	22	22	22	22	22	22
Number of rods	6	6	6	6	6	6
Top deflection (mm)	66,7	33,25	22,28	18,71	18,39	18,3
Degrees (Rad)	0,00635	0,00317	0,00212	0,00178	0,00175	0,00174
Rotational stiffness (kNm/Rad)	1652,95	3315,80	4948,39	5892,58	5995,11	6024,60

ABAQUS Dowel diameter			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	12	20	32	40
Number of dowels	4	4	4	4
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	22	22	22	22
Number of rods	6	6	6	6
Top deflection (mm)	38,36	34,98	33,75	33,24
Degrees (Rad)	0,00365	0,00333	0,00321	0,00317
Rotational stiffness (kNm/Rad)	2874,10	3151,81	3266,68	3316,80

ABAQUS Dowel diameter			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	12	20	32	40
Number of dowels	8	8	8	8
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	22	22	22	22
Number of rods	6	6	6	6
Top deflection (mm)	35,88	34,13	33,25	32,75
Degrees (Rad)	0,00342	0,00325	0,00317	0,00312
Rotational stiffness (kNm/Rad)	3072,75	3230,31	3315,80	3366,42

ABAQUS Dowel diameter			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	12	20	32	40
Number of dowels	12	12	12	12
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	22	22	22	22
Number of rods	6	6	6	6
Top deflection (mm)	35,05	33,82	32,98	32,49
Degrees (Rad)	0,00334	0,00322	0,00314	0,00309
Rotational stiffness (kNm/Rad)	3145,52	3259,92	3342,95	3393,36

ABAQUS Dowel diameter			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	12	20	32	40
Number of dowels	16	16	16	16
Length of dowels (mm)	200	200	200	200

Diameter rods (mm)	22	22	22	22
Number of rods	6	6	6	6
Top deflection (mm)	34,64	33,64	32,8	32,32
Degrees (Rad)	0,00330	0,00320	0,00312	0,00308
Rotational stiffness (kNm/Rad)	3182,75	3277,36	3361,29	3411,21

ABAQUS Varying diameter of rods			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	32	32	32	32
Number of dowels	8	8	8	8
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	14	18	22	26
Number of rods	4	4	4	4
Top deflection (mm)	40,42	36,7	34,86	33,74
Degrees (Rad)	0,00385	0,00350	0,00332	0,00321
Rotational stiffness (kNm/Rad)	2727,62	3004,10	3162,66	3267,65

ABAQUS Varying diameter of rods			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	32	32	32	32
Number of dowels	8	8	8	8
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	14	18	22	26
Number of rods	6	6	6	6
Top deflection (mm)	37,02	34,5	33,24	32,46
Degrees (Rad)	0,00353	0,00329	0,00317	0,00309
Rotational stiffness (kNm/Rad)	2978,13	3195,66	3316,80	3396,50

ABAQUS Varying diameter of rods			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	32	32	32	32
Number of dowels	8	8	8	8
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	14	18	22	26
Number of rods	8	8	8	8
Top deflection (mm)	35,31	33,38	32,39	31,77
Degrees (Rad)	0,00336	0,00318	0,00308	0,00303
Rotational stiffness (kNm/Rad)	3122,36	3302,89	3403,84	3470,27

ABAQUS Varying diameter of rods			Referance model sizes	
Soil stiffness (kN/m^3)	20	20	20	20
Plate thickness (mm)	20	20	20	20
Diameter dowel (mm)	32	32	32	32
Number of dowels	8	8	8	8
Length of dowels (mm)	200	200	200	200
Diameter rods (mm)	14	18	22	26
Number of rods	10	10	10	10
Top deflection (mm)	34,27	32,68	31,86	31,33
Degrees (Rad)	0,00326	0,00311	0,00303	0,00298
Rotational stiffness (kNm/Rad)	3217,11	3373,63	3460,46	3519,00

ABAQUS Maximal expectated stiffness	A	B	C	D	E
Soil stiffness (kN/m^3)	20	20	40	60	100
Plate thickness (mm)	40	40	45	50	100
Diameter dowel (mm)	22	32	32	32	32
Number of dowels	8	8	8	8	8
Length of dowels (mm)	200	200	200	200	200
Diameter rods (mm)	22	24	24	24	24
Number of rods	8	8	8	8	8
Top deflection (mm)	17,95	17,29	11,79	9,96	9,62
Degrees (Rad)	0,00171	0,00165	0,00112	0,00095	0,00092
Rotational stiffness (kNm/Rad)	6142,07	6376,52	9351,15	11069,28	11460,50

A.6 Columns before and after mounting of decks

Calculations of timber columns 400 x 400 GL30c according to NS-EN-1995-1-1 when standing alone

Properties of GL30c

$f_{m.g.k} := 30 \frac{N}{mm^2}$ $f_{c.0.g.k} := 24.5 \frac{N}{mm^2}$ $f_{v.g.k} := 3.5 \frac{N}{mm^2}$

$f_{t.0.g.k} := 19.5 \frac{N}{mm^2}$ $f_{c.90.g.k} := 2.5 \frac{N}{mm^2}$ $E_{0.05} := 10800 \frac{N}{mm^2}$

$f_{t.90.g.k} := 0.5 \frac{N}{mm^2}$ $E_{0.g.mean} := 13000 \frac{N}{mm^2}$ $g := 9.81 \frac{m}{s^2}$

NS-EN-1995-1-1 table NA.2.3

$\gamma_m := 1.15$

NS-EN-1995-1-1 table 3.1

$K_{mod.selfweight} := 0.6$

$K_{mod.windload} := 0.9$

Design properties

$$f_{m.g.d} := f_{m.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 23.478 \frac{N}{mm^2}$$

$$f_{t.0.g.d} := f_{t.0.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 15.261 \frac{N}{mm^2}$$

$$f_{t.90.g.d} := f_{t.90.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 0.391 \frac{N}{mm^2}$$

$$f_{c.0.g.d} := f_{c.0.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 19.174 \frac{N}{mm^2}$$

$$f_{c.90.g.d} := f_{c.90.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 1.957 \frac{N}{mm^2}$$

$$f_{v.g.d} := f_{v.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 2.739 \frac{N}{mm^2}$$

$$E_{0.g.mean} := 13000 \frac{N}{mm^2}$$

Dimensions

$b := 400\ mm$ $\quad h := 400\ mm$ $\quad p_{g.mean} := 430 \frac{kg}{m^3}$ $\quad L := 28\ m$

$A := b \cdot h = 0.16\ m^2$ $\quad m_{selfweight} := p_{g.mean} \cdot L \cdot A = \left(1.926 \cdot 10^3\right)\ kg$

$$I_z := \frac{b \cdot h^3}{12} = \left(2.133 \cdot 10^9\right)\ mm^4$$

Compression according to NS-EN-1995-1-1 6.1.4 (1)

$$\sigma_{c.0.d} \le f_{c.0.g.d}$$

$$\psi_{deadload} := 1.2$$

$$N_{selfweight} := m_{selfweight} \cdot g \cdot \psi_{deadload} = 22.678\ \mathbf{kN}$$

$$\sigma_{c.0.d} := \frac{N_{selfweight}}{A} = 0.142\ \frac{\mathbf{N}}{\mathbf{mm}^2}$$

$$f_{c.0.g.d} = 19.174\ \frac{\mathbf{N}}{\mathbf{mm}^2}$$

$$\frac{\sigma_{c.0.d}}{f_{c.0.g.d}} = 0.007$$ ok

Windspeed in Trondheim according to NS-EN-1991-1-4 is 26 m/s.

$p := 1.25\ \frac{\mathbf{kg}}{\mathbf{m}^3}$ NS-EN 1991-1-4 4.5

$v_b := 26\ \frac{\mathbf{m}}{\mathbf{s}}$ NS-EN 1991-1-4 N.A.4

$q_b := 0.5 \cdot p \cdot {v_b}^2 = 422.5\ \mathbf{Pa}$ NS-EN 1991-1-4 4.5 (4.10)

$c_e := 2.5$ NS-EN 1991-1-4 4.5

$q_p := c_e \cdot q_b = \left(1.056 \cdot 10^3\right)\ \mathbf{Pa}$ NS-EN 1991-1-4 4.5 (4.8)

Bending according to NS-EN-1995-1-1 6.1.6 (1)

$$q_{wind} := q_p \cdot b = 0.423\ \frac{\mathbf{kN}}{\mathbf{m}}$$

$$\psi_{liveload} := 1.5$$

$$M_{ed.wind} := q_{wind} \cdot \frac{L^2}{2} \cdot \psi_{liveload} = 248.43\ \mathbf{kN} \cdot \mathbf{m}$$

$$\frac{\sigma_{m.y.d}}{f_{m.y.d}} \le 1$$

$$W_y := \frac{b \cdot h^2}{6} = \left(1.067 \cdot 10^7\right)\ \boldsymbol{mm^3}$$

$$\sigma_{m.y.d} := \frac{M_{ed.wind}}{W_y} = 23.29\ \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$$

$$f_{m.g.d} = 23.478\ \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$$

$$\frac{\sigma_{m.y.d}}{f_{m.g.d}} = 0.992$$

ok

Deflection at top:

$$\delta_{top.max} := \frac{17 \cdot q_{wind} \cdot L^4}{384 \cdot E_{0.05} \cdot I_z} = 498.993\ \boldsymbol{mm}$$

Shear according to NS-EN 1995-1-1 6.1.7 (1)

$$\tau_d \le f_{v.g.d}$$

$$V_{ed} := q_{wind} \cdot L \cdot \psi_{liveload} = 17.745\ \boldsymbol{kN}$$

$$k_{cr} := 0.67$$

$$b_{ef} := k_{cr} \cdot b = 268\ \boldsymbol{mm}$$

$$\tau_d := \frac{3 \cdot V_{ed}}{2 \cdot b_{ef} \cdot h} = 0.248 \ \frac{N}{mm^2}$$

$$f_{v.g.d} = 2.739 \ \frac{N}{mm^2}$$

$$\frac{\tau_d}{f_{v.g.d}} = 0.091$$ ok

Combined bendind and compression

$$\left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.y.d}}\right) \le 1$$ NS-EN 1995-1-1 6.19

$$\left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) = 0.992$$ ok

Stability - Buckling

$$I_y := \frac{b \cdot h^3}{12} = \left(2.133 \cdot 10^9\right) \ mm^4$$

$$\lambda_y := \frac{L}{\sqrt[2]{\frac{I_y}{A}}} = 242.487$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c.0.g.k}}{E_{0.05}}} = 3.676$$ NS-EN 1995-1-1 6.3.2 (6.21)

$$\beta_c := 0.1$$ NS-EN 1995-1-1 6.3.2 (6.29)

$$k_y := 0.5 \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right) = 7.426$$ NS-EN 1995-1-1 6.3.2 (6.27)

$$k_{c.y} := \frac{1}{k_y + \sqrt[2]{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}} = 0.072$$ NS-EN 1995-1-1 6.3.2 (6.25)

$$\left(\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) \le 1$$ NS-EN 1995-1-1 6.3.2 (6.23)

$$\left(\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) = 1.003$$ Not ok

Lateral torsional buckling

$$l_{ef} := 0.5 \cdot L + 2 \cdot h = 14.8 \ \boldsymbol{m}$$ NS-EN 1995-1-1 6.3.2 table 6.1

For lef, we are choosing to be more conservative. Thereby using the formula beneath instead.

$$l_{ef} := 0.8 \cdot L + 2 \cdot h = 23.2 \ \boldsymbol{m}$$ NS-EN 1995-1-1 6.3.2 table 6.1

$$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05} = 145.241 \ \frac{\boldsymbol{N}}{\boldsymbol{mm}^2}$$ NS-EN 1995-1-1 6.3.2 (6.32)

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{m.g.k}}{\sigma_{m.crit}}} = 0.454$$ NS-EN 1995-1-1 6.3.3 (6.30)

$k_{crit} := 1.0$ NS-EN 1995-1-1 6.3.3 (6.34)

$$\left(\frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.g.d}}\right)^2 + \left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d} \cdot k_{c.y}}\right) \le 1.0$$ NS-EN 1995-1-1 6.3.3 (6.35)

$$\left(\frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.g.d}}\right)^2 + \left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d} \cdot k_{c.y}}\right) = 1.087$$ Not ok

Lateral torsional buckling will occur.

Checking what windspeed can be withstanded:

$p := 1.25 \ \frac{kg}{m^3}$ NS-EN 1991-1-4 4.5

$v_{b.max} := 25 \ \frac{m}{s}$ NS-EN 1991-1-4 N.A.4

$q_b := 0.5 \cdot p \cdot v_{b.max}^{\ 2} = 390.625 \ Pa$ NS-EN 1991-1-4 4.5 (4.10)

$c_e := 2.5$ NS-EN 1991-1-4 4.5

$q_{p.max} := c_e \cdot q_b = 976.563 \ Pa$ NS-EN 1991-1-4 4.5 (4.8)

Bending according to NS-EN-1995-1-1 6.1.6 (1)

$$q_{wind.max} := q_{p.max} \cdot b = 0.391 \ \frac{kN}{m}$$

$$\psi_{liveload} := 1.5$$

$$M_{ed.wind} := q_{wind.max} \cdot \frac{L^2}{2} \cdot \psi_{liveload} = 229.688 \ kN \cdot m$$

$$f_{m.g.d} = 23.478 \ MPa$$

$$\frac{\sigma_{m.y.d}}{f_{m.g.d}} = 0.917$$ ok

Combined bendind and compression

$$\left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.y.d}}\right) \le 1$$ NS-EN 1995-1-1 6.19

$$\left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) = 0.917$$ ok

Stability - Buckling

$$I_y := \frac{b \cdot h^3}{12} = \left(2.133 \cdot 10^9\right) \ mm^4$$

$$\lambda_y := \frac{L}{\sqrt[2]{\frac{I_y}{A}}} = 242.487$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c.0.g.k}}{E_{0.05}}} = 3.676$$ NS-EN 1995-1-1 6.3.2 (6.21)

$$\beta_c := 0.1$$ NS-EN 1995-1-1 6.3.2 (6.29)

$$k_y := 0.5 \ \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 7.426$$ NS-EN 1995-1-1 6.3.2 (6.27)

$$k_{c.y} := \frac{1}{k_y + \sqrt[2]{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.072$$ NS-EN 1995-1-1 6.3.2 (6.25)

$$\left(\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) \le 1$$ NS-EN 1995-1-1 6.3.2 (6.23)

$$\left(\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) = 0.928$$ ok

Lateral torsional buckling

$$l_{ef} := 0.5 \cdot L + 2 \cdot h = 14.8 \; \boldsymbol{m}$$ NS-EN 1995-1-1 6.3.2 table 6.1

For lef, we are choosing to be more conservative. Thereby using the formula beneath instead.

$$l_{ef} := 0.8 \cdot L + 2 \cdot h = 23.2 \; \boldsymbol{m}$$ NS-EN 1995-1-1 6.3.2 table 6.1

$$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05} = 145.241 \; \frac{\boldsymbol{N}}{\boldsymbol{mm}^2}$$ NS-EN 1995-1-1 6.3.2 (6.32)

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{m.g.k}}{\sigma_{m.crit}}} = 0.454$$ NS-EN 1995-1-1 6.3.3 (6.30)

$$k_{crit} := 1.0$$ NS-EN 1995-1-1 6.3.3 (6.34)

$$\left(\frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.g.d}}\right)^2 + \left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d} \cdot k_{c.y}}\right) \le 1.0$$ NS-EN 1995-1-1 6.3.3 (6.35)

$$\left(\frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.g.d}}\right)^2 + \left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d} \cdot k_{c.y}}\right) = 0.944$$ ok

Lateral torsional buckling will not occur.

The columns can withstand 25 m/s windspeed when assuming that the connection to the foundations is ok.

Calculations of timber columns 400 x 400 GL30c according to NS-EN-1995-1-1

Properties of GL30c

$f_{m.g.k} := 30 \ \frac{N}{mm^2}$ $\quad f_{c.0.g.k} := 24.5 \ \frac{N}{mm^2}$ $\quad f_{v.g.k} := 3.5 \ \frac{N}{mm^2}$

$f_{t.0.g.k} := 19.5 \ \frac{N}{mm^2}$ $\quad f_{c.90.g.k} := 2.5 \ \frac{N}{mm^2}$ $\quad E_{0.05} := 10800 \ \frac{N}{mm^2}$

$f_{t.90.g.k} := 0.5 \ \frac{N}{mm^2}$ $\quad E_{0.g.mean} := 13000 \ \frac{N}{mm^2}$ $\quad g := 9.81 \ \frac{m}{s^2}$

NS-EN-1995-1-1 table NA.2.3

$\gamma_m := 1.15$

NS-EN-1995-1-1 table 3.1

$K_{mod.selfweight} := 0.6$

$K_{mod} := 0.9$

Design properties

$f_{m.g.d} := f_{m.g.k} \cdot \frac{K_{mod}}{\gamma_m} = 23.478 \ \frac{N}{mm^2}$ $\quad f_{v.g.d} := f_{v.g.k} \cdot \frac{K_{mod}}{\gamma_m} = 2.739 \ \frac{N}{mm^2}$

$f_{t.0.g.d} := f_{t.0.g.k} \cdot \frac{K_{mod}}{\gamma_m} = 15.261 \ \frac{N}{mm^2}$ $\quad E_{0.g.mean} := 13000 \ \frac{N}{mm^2}$

$f_{t.90.g.d} := f_{t.90.g.k} \cdot \frac{K_{mod}}{\gamma_m} = 0.391 \ \frac{N}{mm^2}$ $\quad f_{c.0.g.d} := f_{c.0.g.k} \cdot \frac{K_{mod}}{\gamma_m} = 19.174 \ \frac{N}{mm^2}$

$f_{c.90.g.d} := f_{c.90.g.k} \cdot \frac{K_{mod}}{\gamma_m} = 1.957 \ \frac{N}{mm^2}$

Dimensions

$b := 400\ mm$ $\quad h := 400\ mm$ $\quad p_{g.mean} := 430\ \frac{kg}{m^3}$ $\quad L := 28\ m$

$A := b \cdot h = 0.16\ m^2$

$I_z := \frac{b \cdot h^3}{12} = \left(2.133 \cdot 10^9\right)\ mm^4$

Compression according to NS-EN-1995-1-1 6.1.4 (1)

$\sigma_{c.0.d} \le f_{c.0.g.d}$

$N_{robot} := 695\ kN$

$\sigma_{c.0.d} := \frac{N_{robot}}{A} = 4.344\ \frac{N}{mm^2}$

$f_{c.0.g.d} = 19.174\ \frac{N}{mm^2}$

$\frac{\sigma_{c.0.d}}{f_{c.0.g.d}} = 0.227$ ok

Bending according to NS-EN-1995-1-1 6.1.6 (1)

$M_{ed.robot} := 102\ kN \cdot m$

$\frac{\sigma_{m.y.d}}{f_{m.y.d}} \le 1$

$W_y := \frac{b \cdot h^2}{6} = \left(1.067 \cdot 10^7\right)\ mm^3$

$\sigma_{m.y.d} := \frac{M_{ed.robot}}{W_y} = 9.563\ \frac{N}{mm^2}$

$f_{m.g.d} = 23.478\ \frac{N}{mm^2}$

$\frac{\sigma_{m.y.d}}{f_{m.g.d}} = 0.407$ ok

Shear according to NS-EN 1995-1-1 6.1.7 (1)

$$\tau_d \leq f_{v.g.d}$$

$$V_{ed.robot} := 58\ kN$$

$$k_{cr} := 0.67$$

$$b_{ef} := k_{cr} \cdot b = 268\ mm$$

$$\tau_d := \frac{3 \cdot V_{ed.robot}}{2 \cdot b_{ef} \cdot h} = 0.812\ \frac{N}{mm^2}$$

$$f_{v.g.d} = 2.739\ \frac{N}{mm^2}$$

$$\frac{\tau_d}{f_{v.g.d}} = 0.296$$ ok

Combined bendind and compression

$$\left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.y.d}}\right) \leq 1$$ NS-EN 1995-1-1 6.19

$$\left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) = 0.459$$ ok

Stability - Buckling

$$L_{buckling} := 3.5\ m$$

$$I_y := \frac{b \cdot h^3}{12} = \left(2.133 \cdot 10^9\right)\ mm^4$$

$$\lambda_y := \frac{L_{buckling}}{\sqrt[2]{\frac{I_y}{A}}} = 30.311$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c.0.g.k}}{E_{0.05}}} = 0.46$$ NS-EN 1995-1-1 6.3.2 (6.21)

$$\beta_c := 0.1$$ NS-EN 1995-1-1 6.3.2 (6.29)

$$k_y := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 0.614$$ NS-EN 1995-1-1 6.3.2 (6.27)

$$k_{c.y} := \frac{1}{k_y + \sqrt[2]{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.98$$ NS-EN 1995-1-1 6.3.2 (6.25)

$$\left(\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) \le 1$$ NS-EN 1995-1-1 6.3.2 (6.23)

$$\left(\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.g.d}}\right)^2 + \left(\frac{\sigma_{m.y.d}}{f_{m.g.d}}\right) = 0.461$$ ok

Lateral torsional buckling

$l_{ef} := 0.5 \cdot L + 2 \cdot h = 14.8\ m$ NS-EN 1995-1-1 6.3.2 table 6.1

For lef, we are choosing to be more conservative. Thereby using the formula beneath instead.

$l_{ef} := 0.8 \cdot L + 2 \cdot h = 23.2\ m$ NS-EN 1995-1-1 6.3.2 table 6.1

$$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05} = 145.241\ \frac{N}{mm^2}$$ NS-EN 1995-1-1 6.3.2 (6.32)

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{m.g.k}}{\sigma_{m.crit}}} = 0.454$$ NS-EN 1995-1-1 6.3.3 (6.30)

$k_{crit} := 1.0$ NS-EN 1995-1-1 6.3.3 (6.34)

$$\left(\frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.g.d}}\right)^2 + \left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d} \cdot k_{c.y}}\right) \le 1.0$$ NS-EN 1995-1-1 6.3.3 (6.35)

$$\left(\frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.g.d}}\right)^2 + \left(\frac{\sigma_{c.0.d}}{f_{c.0.g.d} \cdot k_{c.y}}\right) = 0.397$$ Ok

Lateral torsional buckling will not occur.

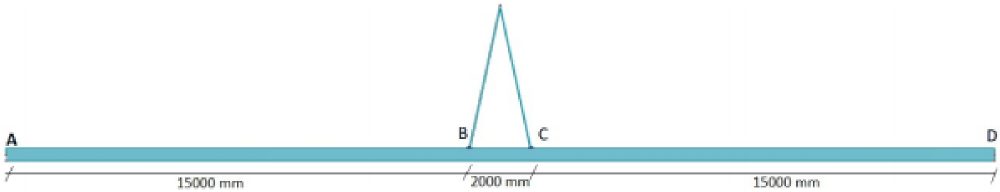
A.7 Lifting of columns

Dimensions

$b := 400\ mm$ $h := 400\ mm$ $p_{g.mean} := 430\ \frac{kg}{m^3}$ $L := 32\ m$

$A := b \cdot h = 0.16\ m^2$ $m_{selfweight} := p_{g.mean} \cdot L \cdot A = (2.202 \cdot 10^3)\ kg$ $g := 9.81\ \frac{m}{s^2}$

Lifting of columns to foundation fastened in the middle



$L_{AB} := 15000\ mm$

$L_{CD} := 15000\ mm$

$L_{BC} := 2000\ mm$

$q_{selfweight} := p_{g.mean} \cdot A \cdot g = 0.675\ \frac{kN}{m}$

Simplified solution to the acting moments: Calculating the beam as a fixed beam from B to A.

$$M_B := q_{selfweight} \cdot \frac{L_{AB}^2}{2} = 75.929\ kN \cdot m$$

$$M_C := q_{selfweight} \cdot \frac{L_{CD}^2}{2} = 75.929\ kN \cdot m$$

$$M_{y.d} := M_B = 75.929\ kN \cdot m$$

$$V_{y.d} := q_{selfweight} \cdot L_{AB} = 10.124\ kN$$

Bending according to NS-EN-1995-1-1 6.1.6 (1)

$$M_{y.d} := M_B = 75.929\ kN \cdot m$$

$$W_y := \frac{b \cdot h^2}{6} = \left(1.067 \cdot 10^7\right)\ mm^3$$

$$\sigma_{m.y.d} := \frac{M_{y.d}}{W_y} = 7.118\ \frac{N}{mm^2}$$

$$f_{m.g.d} = 23.478\ \frac{N}{mm^2}$$

if $\sigma_{m.y.d} \leq f_{m.g.d}$ = "Ok"
"Ok"
else
"Not ok"

Shear according to NS-EN 1995-1-1 6.1.7 (1)

$$V_{y.d} := q_{selfweight} \cdot L_{AB} = 10.124\ kN$$

$$\tau_d \leq f_{v.g.d}$$

$$k_{cr} := 0.67$$

$$b_{ef} := k_{cr} \cdot b = 268\ mm$$

$$\tau_d := \frac{3 \cdot V_{y.d}}{2 \cdot b_{ef} \cdot h} = 0.142\ \frac{N}{mm^2}$$

$$f_{v.g.d} = 2.739\ \frac{N}{mm^2}$$

if $\tau_d \leq f_{v.g.d}$ = "Ok"
"Ok"
else
"Not ok"

Lifting of columns by a hole near the top:

When the column is lifted from the truck the following forces will be present untill the columnt is lifted to standing position:

Assuming it can be considered as a simply supported beam.

Checking the hole for a diameter of 60 mm placed minimum 400 mm from the top of the column.

$$h_d := 60\ \boldsymbol{mm}$$

$$M_{ed} := \frac{q_{selfweight} \cdot L^2}{8} = 86.391\ \boldsymbol{kN \cdot m}$$

$$V_{ed} := \frac{q_{selfweight} \cdot L}{2} = 10.799\ \boldsymbol{kN}$$

Bending according to NS-EN-1995-1-1 6.1.6 (1)

$$\frac{\sigma_{m.y.d}}{f_{m.y.d}} \leq 1$$

$$W_y := \frac{b \cdot h^2}{6} = \left(1.067 \cdot 10^7\right)\ \boldsymbol{mm}^3$$

$$\sigma_{m.y.d} := \frac{M_{ed}}{W_y} = 8.099\ \frac{\boldsymbol{N}}{\boldsymbol{mm}^2}$$

$$f_{m.g.d} = 23.478\ \frac{\boldsymbol{N}}{\boldsymbol{mm}^2}$$

```
if σm.y.d ≤ fm.g.d   = "Ok"
  ‖ "Ok"
else
  ‖ "Not ok"
```

Shear according to NS-EN 1995-1-1 6.1.7 (1)
At the end with the hole, the cross-section will be reduced.

$$\tau_d \le f_{v.g.d}$$

$$k_{cr} := 0.67$$

$$b_{ef} := k_{cr} \cdot b = 268 \ mm$$

$$\tau_d := \frac{3 \cdot V_{ed}}{2 \cdot b_{ef} \cdot (h - 60 \ mm)} = 0.178 \ \frac{N}{mm^2}$$

If the diameter of the hole is 60 mm.

$$f_{v.g.d} = 2.739 \ \frac{N}{mm^2}$$

if $\tau_d \le f_{v.g.d}$ = "Ok"
  "Ok"
else
  "Not ok"

Need to check the hole:
Using formulas from the appendix of Timber Constructions 2.

Requirements for unreinforced holes:

$l_v \ge h$	$l_z \ge 1.5\ h$, not less than 300 mm	$l_A \ge h/2$	$h_{rl(ru)} \ge 0{,}35 \cdot h$	$a \le 0{,}4\ h$	$h_d \le 0{,}15 \cdot h$

$$l_v := 500 \ mm$$

$$l_z := N.A$$

$$l_A := 500 \ mm$$

$$h_{rl} := 150 \ mm$$

$$h_d := 60 \ mm$$

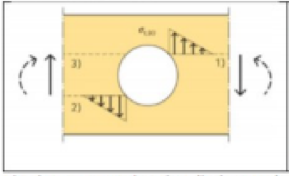
if $l_v \ge h$ = "Ok"
  "Ok"
else
  "Not ok"

if $l_A \ge \frac{h}{2}$ = "ok"
  "Ok"
else
  "Not ok"

if $h_{rl} \ge h \cdot 0.35$ = "Ok"
  "Ok"
else
  "Not ok"

if $h_d \le h \cdot 0.15$ = "ok"
  "Ok"
else
  "Not ok"

Unreinforced holes:



$\sigma_{t,90,d} = \dfrac{F_{t,90,Ed}}{0.5 \cdot l_{t,90} \cdot b} \leq k_{t,90} \cdot f_{t,90,d}$

$l_{t,90} = 0.5 \cdot (h_d + h)$ for rectangular holes

$l_{t,90} = 0.353 \cdot h_d + 0.5 \cdot h$ for round holes

$k_{t,90} = \min\{1; (450/h)^{0.5}\}$

Check net section bending (hole in NA): use parallel axes theorem and calculate $W_{net} = I_{net}/(h/2)$

Forces acting at the hole:

$$F_{t.V.Ed} := \frac{V_{ed} \cdot h_d}{4 \cdot h} \cdot \left(3 - \frac{{h_d}^2}{h^2}\right) = 1.206\ kN$$

$$F_{t.M.Ed} := 0$$

$$F_{t.90.Ed} := F_{t.V.Ed} + F_{t.M.Ed} = 1.206\ kN$$

$$l_{t.90} := 0.353 \cdot h_d + 0.5 \cdot h = 221.18\ mm$$

$$k_{t.90} := \begin{array}{l} \text{if } 1 \geq \left(\dfrac{450\ mm}{h}\right)^{0.5} \\ \quad \| \ 1 \\ \text{else} \\ \quad \| \left(\dfrac{450\ mm}{h}\right)^{0.5} \end{array} = 1.061$$

$$\sigma_{t.90.d} := \frac{F_{t.90.Ed}}{0.5 \cdot l_{t.90} \cdot b} \leq k_{t.90} \cdot f_{t.90.g.d}$$

$$\frac{F_{t.90.Ed}}{0.5 \cdot l_{t.90} \cdot b} = 0.027\ \frac{N}{mm^2}$$

$$k_{t.90} \cdot f_{t.90.g.d} = 0.415\ \frac{N}{mm^2}$$

$$\sigma_{t.90.d} := \begin{array}{l} \text{if } \dfrac{F_{t.90.Ed}}{0.5 \cdot l_{t.90} \cdot b} \leq k_{t.90} \cdot f_{t.90.g.d} \\ \quad \| \ \text{“Ok”} \\ \text{else} \\ \quad \| \ \text{“Not ok”} \end{array} = \text{“Ok”}$$

Need to check the cross-section for tension forces when the column is lifted above the ground.

$$N_{ed.t} := m_{selfweight} \cdot g = 21.598\ kN$$

$$A_{eff} := b \cdot (h - 60\ mm) = \left(1.36 \cdot 10^{5}\right)\ mm^{2}$$

$$\sigma_{t.0.d} \le f_{t.0.g.d}$$ NS-EN 1995-1-1 6.1.2 (6.1)

$$\sigma_{t.0.d} := \frac{N_{ed.t}}{A_{eff}} = 0.159\ \frac{N}{mm^{2}}$$

$$f_{t.0.g.d} = 15.261\ \frac{N}{mm^{2}}$$

```
if σt.0.d ≤ ft.0.g.d    = "Ok"
  ‖ "Ok"
else
  ‖ "Not ok"
```

Calculations of timber columns 400 x 400 GL30c according to NS-EN-1995-1-1

Properties of GL30c

$f_{m.g.k} := 30 \frac{N}{mm^2}$ $f_{c.0.g.k} := 24.5 \frac{N}{mm^2}$ $f_{v.g.k} := 3.5 \frac{N}{mm^2}$

$f_{t.0.g.k} := 19.5 \frac{N}{mm^2}$ $f_{c.90.g.k} := 2.5 \frac{N}{mm^2}$ $E_{0.05} := 10800 \frac{N}{mm^2}$

$f_{t.90.g.k} := 0.5 \frac{N}{mm^2}$ $E_{0.g.mean} := 13000 \frac{N}{mm^2}$ $g := 9.81 \frac{m}{s^2}$

NS-EN-1995-1-1 table NA.2.3

$\gamma_m := 1.15$

NS-EN-1995-1-1 table 3.1

$K_{mod.selfweight} := 0.6$

$K_{mod.windload} := 0.9$

Design properties

$$f_{m.g.d} := f_{m.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 23.478 \frac{N}{mm^2}$$

$$f_{t.0.g.d} := f_{t.0.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 15.261 \frac{N}{mm^2}$$

$$f_{t.90.g.d} := f_{t.90.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 0.391 \frac{N}{mm^2}$$

$$f_{c.0.g.d} := f_{c.0.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 19.174 \frac{N}{mm^2}$$

$$f_{c.90.g.d} := f_{c.90.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 1.957 \frac{N}{mm^2}$$

$$f_{v.g.d} := f_{v.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 2.739 \frac{N}{mm^2}$$

$$E_{0.g.mean} := 13000 \frac{N}{mm^2}$$

Need to check that the rope wont press in the wood:

$$\sigma_{c.0.d} \le f_{c.0.g.d}$$ NS-EN 1995-1-1 6.1.4 (6.2)

$$N_{ed.c} := N_{ed.t}$$

$$\sigma_{c.0.d} := \frac{N_{ed.c}}{A_{eff}} = 0.159 \frac{N}{mm^2}$$

$$\left.\begin{array}{l} \text{if } \sigma_{c.0.d} \le f_{c.0.g.d} \\ \quad \| \text{“Ok”} \\ \text{else} \\ \quad \| \text{“Not ok”} \end{array}\right| = \text{“Ok”}$$

The hole solution with a hole of 60 mm drilled a minimum of 400 mm from the top of the column seems to be applicable.

Lifting of columns by a fastner with hole, fastened on the top:

The cross-section have already been checked that it can withstand the forces.

This solution is probably applicable, but it would result in a unpractical long screw being drilled down at the top of the column. Therefore this solution is disregarded.

A.8 One row of decks mounted

$f_{m.g.k} := 30 \ \frac{N}{mm^2}$ $\quad f_{c.0.g.k} := 24.5 \ \frac{N}{mm^2}$ $\quad f_{v.g.k} := 3.5 \ \frac{N}{mm^2}$

$f_{t.0.g.k} := 19.5 \ \frac{N}{mm^2}$ $\quad f_{c.90.g.k} := 2.5 \ \frac{N}{mm^2}$ $\quad E_{0.05} := 10800 \ \frac{N}{mm^2}$

$f_{t.90.g.k} := 0.5 \ \frac{N}{mm^2}$ $\quad E_{0.g.mean} := 13000 \ \frac{N}{mm^2}$ $\quad g := 9.81 \ \frac{m}{s^2}$

NS-EN-1995-1-1 table NA.2.3

$\gamma_m := 1.15$

NS-EN-1995-1-1 table 3.1

$K_{mod.selfweight} := 0.6$

$K_{mod.windload} := 0.9$

Design properties

$$f_{m.g.d} := f_{m.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 23.478 \ \frac{N}{mm^2}$$

$$f_{t.0.g.d} := f_{t.0.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 15.261 \ \frac{N}{mm^2}$$

$$f_{t.90.g.d} := f_{t.90.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 0.391 \ \frac{N}{mm^2}$$

$$f_{c.0.g.d} := f_{c.0.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 19.174 \ \frac{N}{mm^2}$$

$$f_{c.90.g.d} := f_{c.90.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 1.957 \ \frac{N}{mm^2}$$

$$f_{v.g.d} := f_{v.g.k} \cdot \frac{K_{mod.windload}}{\gamma_m} = 2.739 \ \frac{N}{mm^2}$$

$$E_{0.g.mean} := 13000 \ \frac{N}{mm^2}$$

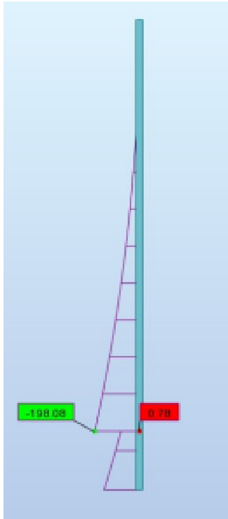
Dimensions

$b := 400 \ mm$ $\quad h := 400 \ mm$ $\quad \rho_{g.mean} := 430 \ \frac{kg}{m^3}$ $\quad L := 28 \ m$

$A := b \cdot h = 0.16 \ m^2$ $\quad m_{selfweight} := \rho_{g.mean} \cdot L \cdot A = \left(1.926 \cdot 10^3\right) \ kg$

$$W_y := \frac{b \cdot h^2}{6} = \left(1.067 \cdot 10^7\right) \ mm^3$$

When the first story of slabs are mounted (forces and moments collected from calculations in Robot Structural Analysis):

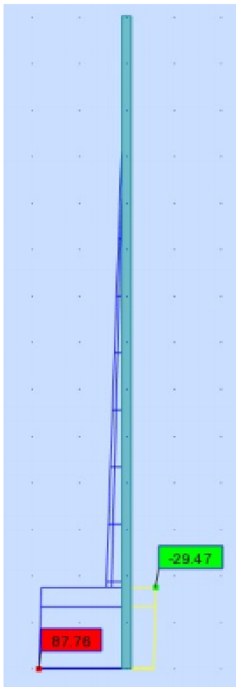


Bending according to NS-EN-1995-1-1 6.1.6 (1)

$$M_{ed.ULS} := 198.1\ \boldsymbol{kN \cdot m}$$

$$\sigma_{m.y.d} := \frac{M_{ed.ULS}}{W_y} = 18.572\ \boldsymbol{MPa}$$

$$f_{m.g.d} = 23.478\ \boldsymbol{MPa}$$ OK!



Tension according to NS-EN-1995-1-1

$$N_{x.ULS} := 29.5\ \boldsymbol{kN}$$

$$\sigma_{t.0.d} := \frac{N_{x.ULS}}{A} = 0.184\ \boldsymbol{MPa}$$

$$f_{t.0.g.d} = 15.261\ \boldsymbol{MPa}$$ *OK!*

Compression according to NS-EN-1995-1-1 6.1.4 (1)

$$N_{x.ULS} := 87.8\ \boldsymbol{kN}$$

$$\sigma_{c.0.d} := \frac{N_{x.ULS}}{A} = 0.549\ \boldsymbol{MPa}$$

$$f_{c.0.g.d} = 19.174\ \boldsymbol{MPa}$$ OK!

A.9 Necessary volume of building parts

Necessary m3 for different parts of the structure

Standard Glulam Beam

Columns:

$$V_{columns} := 0.4\ \mathbf{m} \cdot 0.4\ \mathbf{m} \cdot 28\ \mathbf{m} = 4.48\ \mathbf{m}^3$$

$$n_{columns} := 22$$

$$V_{tot.columns} := V_{columns} \cdot n_{columns} = 98.56\ \mathbf{m}^3$$

Beams in decks:

$$V_{internal.beams} := 0.066\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 8.72\ \mathbf{m} \cdot 3 = 0.699\ \mathbf{m}^3$$

$$V_{end.beams} := 0.14\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 2.12\ \mathbf{m} \cdot 2 = 0.24\ \mathbf{m}^3$$

$$V_{edge.beams} := 0.14\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 9\ \mathbf{m} \cdot 2 = 1.021\ \mathbf{m}^3$$

$$V_{beamsindecks} := V_{internal.beams} + V_{end.beams} + V_{edge.beams} = 1.96\ \mathbf{m}^3$$

$$n_{decks} := 80$$

$$V_{tot.beamsindecks} := V_{beamsindecks} \cdot n_{decks} = 156.821\ \mathbf{m}^3$$

Total

$$V_{tot.columns} + V_{tot.beamsindecks} = 255.381\ \mathbf{m}^3$$

Kerto plate

$$V_{kertoindecks} := (0.063\ \mathbf{m} + 0.045\ \mathbf{m}) \cdot 2.4\ \mathbf{m} \cdot 9\ \mathbf{m} = 2.333\ \mathbf{m}^3$$

$$V_{tot.kertoindecks} := V_{kertoindecks} \cdot n_{decks} = 186.624\ \mathbf{m}^3$$

Steel

per joint

$$A_1 := \left(0.46\ \boldsymbol{m} \cdot 0.13\ \boldsymbol{m} - 0.13\ \boldsymbol{m} \cdot 0.1\ \boldsymbol{m} - 2 \cdot \pi \cdot \left(\frac{0.033}{2}\ \boldsymbol{m}\right)^2\right) = 0.045\ \boldsymbol{m}^2$$

$$A_2 := \left(0.13\ \boldsymbol{m} \cdot 0.46\ \boldsymbol{m} - 5 \cdot \pi \cdot \left(\frac{0.022}{2}\ \boldsymbol{m}\right)^2\right) = 0.058\ \boldsymbol{m}^2$$

$$V_{columns.steelinconnectionplates} := \left(A_1 + A_2\right) \cdot 0.012\ \boldsymbol{m} = \left(1.236 \cdot 10^{-3}\right)\ \boldsymbol{m}^3$$

$$n_{platesoncolumn} := 2$$

$$A_1 := \left(0.12\ \boldsymbol{m} \cdot 0.46\ \boldsymbol{m} + 0.12\ \boldsymbol{m} \cdot 0.46\ \boldsymbol{m}\right) = 0.11\ \boldsymbol{m}^2$$

$$A_2 := \left(2 \cdot 0.045\ \boldsymbol{m} \cdot 0.033\ \boldsymbol{m} + 4 \cdot \pi \cdot \left(\frac{0.022}{2}\ \boldsymbol{m}\right)^2\right) = 0.004\ \boldsymbol{m}^2$$

$$V_{decks.steelinconnectionplates} := \left(A_1 - A_2\right) \cdot 0.012\ \boldsymbol{m} = \left(1.271 \cdot 10^{-3}\right)\ \boldsymbol{m}^3$$

$$n_{platesondeck} := 1$$

$$V_{rodsintowood} := \left(\pi \cdot \left(\frac{0.022}{2}\ \boldsymbol{m}\right)^2 \cdot 0.5\ \boldsymbol{m}\right) = \left(1.901 \cdot 10^{-4}\right)\ \boldsymbol{m}^3$$

$$n_{rodsintowood} := 14$$

$$V_{connectorsbetweenplates} := \pi \cdot \left(\frac{0.033}{2}\ \boldsymbol{m}\right)^2 \cdot 0.05\ \boldsymbol{m} = \left(4.276 \cdot 10^{-5}\right)\ \boldsymbol{m}^3$$

$$n_{connectorrods} := 2$$

$$V_{nuts} := \left(\pi \cdot \left(\frac{0.055}{2}\ \boldsymbol{m}\right)^2 - \pi \cdot \left(\frac{0.033}{2}\ \boldsymbol{m}\right)^2\right) \cdot 0.020\ \boldsymbol{m} = \left(3.041 \cdot 10^{-5}\right)\ \boldsymbol{m}^3$$

$$n_{nuts} := 32$$

Total per joint

$$V_{tot.platesoncolumn} := V_{columns.steelinconnectionplates} \cdot n_{platesoncolumn} = 0.002\ \boldsymbol{m}^3$$

$$V_{tot.platesondeck} := V_{decks.steelinconnectionplates} \cdot n_{platesondeck} = 0.001\ \boldsymbol{m}^3$$

$$V_{tot.connectionplates} := V_{tot.platesoncolumn} + V_{tot.platesondeck} = 0.004\ \boldsymbol{m}^3$$

$$V_{tot.rodsintowood} := V_{rodsintowood} \cdot n_{rodsintowood} = 0.003\ \boldsymbol{m}^3$$

$$V_{tot.connectorrods} := V_{connectorsbetweenplates} \cdot n_{connectorrods} = \left(8.553 \cdot 10^{-5}\right)\ \boldsymbol{m}^3$$

$$V_{tot.nuts} := V_{nuts} \cdot n_{nuts} = \left(9.731 \cdot 10^{-4}\right)\ \boldsymbol{m}^3$$

$$V_{tot.joint} := V_{tot.connectionplates} + V_{tot.rodsintowood} + V_{tot.connectorrods} + V_{tot.nuts} = \left(7.462 \cdot 10^{-3}\right)\ \boldsymbol{m}^3$$

$$n_{joints} := 4 \cdot 80 = 320$$

$$V_{steel.joints} := V_{tot.joint} \cdot n_{joints} = 2.388\ \boldsymbol{m}^3$$

Foundations

$$V_{steelplate} := 0.6\ \boldsymbol{m} \cdot 0.6\ \boldsymbol{m} \cdot 0.05\ \boldsymbol{m} = 0.018\ \boldsymbol{m}^3$$

$$n_{steelplate} := 22$$

$$V_{tot.steelplates} := V_{steelplate} \cdot n_{steelplate} = 0.396\ \boldsymbol{m}^3$$

$$V_{rodsintocolumn} := \pi \cdot \left(\frac{0.022}{2}\ \boldsymbol{m}\right)^2 \cdot 1.2\ \boldsymbol{m} = \left(4.562 \cdot 10^{-4}\right)\ \boldsymbol{m}^3$$

$$n_{rodsintocolumn} := 6 \cdot 22 = 132$$

$$V_{tot.rodsintocolumn} := V_{rodsintocolumn} \cdot n_{rodsintocolumn} = 0.06\ \boldsymbol{m}^3$$

$$V_{dowelsintoconcrete} := \pi \cdot \left(\frac{0.032}{2}\ \boldsymbol{m}\right)^2 \cdot 0.2\ \boldsymbol{m} = \left(1.608 \cdot 10^{-4}\right)\ \boldsymbol{m}^3$$

$$n_{dowelsintoconcrete} := 8 \cdot 22 = 176$$

$$V_{tot.dowelsinconcrete} := V_{dowelsintoconcrete} \cdot n_{dowelsintoconcrete} = 0.028\ \boldsymbol{m}^3$$

$$V_{nuts} := \left(\pi \cdot \left(\frac{0.06}{2}\ \boldsymbol{m}\right)^2 - \pi \cdot \left(\frac{0.032}{2}\ \boldsymbol{m}\right)^2\right) \cdot 0.02\ \boldsymbol{m} = \left(4.046 \cdot 10^{-5}\right)\ \boldsymbol{m}^3$$

$$n_{nuts} := 22 \cdot 12 = 264$$

$$V_{tot.nuts} := V_{nuts} \cdot n_{nuts} = 0.011\ \boldsymbol{m}^3$$

Total steel

$$V_{steel} := V_{steel.joints} + V_{tot.steelplates} + V_{tot.rodsintocolumn} + V_{tot.dowelsinconcrete} + V_{tot.nuts} = 2.883\ m^3$$

Concrete in timber building

Foundation:

$$V_{foundation} := 2.3\ m \cdot 2.3\ m \cdot 0.5\ m = 2.645\ m^3$$

$$n_{foundation} := 22$$

$$V_{tot.foundation} := V_{foundation} \cdot n_{foundation} = 58.19\ m^3$$

Concrete in concrete building:

Foundation

$$V_{foundation} := 2.8\ m \cdot 2.8\ m \cdot 0.5\ m = 3.92\ m^3$$

$$n_{foundatoin} := 22$$

$$V_{tot.foundation} := V_{foundation} \cdot n_{foundation} = 86.24\ m^3$$

Columns

$$V_{column} := 0.3\ m \cdot 0.3\ m \cdot 28\ m = 2.52\ m^3$$

$$n_{column} := 22$$

$$V_{tot.column} := V_{column} \cdot n_{column} = 55.44\ m^3$$

Decks

$$V_{deck} := 2.4\ m \cdot 9\ m \cdot 0.3\ m = 6.48\ m^3$$

$$n_{deck} := 80$$

$$V_{tot.deck} := V_{deck} \cdot n_{deck} = 518.4\ m^3$$

$$M_{concrete} := V_{tot.column} + V_{tot.foundation} + V_{tot.deck} = 660.08\ m^3$$

Six stories

Necessary m3 for different parts of the structure

Standard Glulam Beam

Columns:

$$V_{columns} := 0.4\ \mathbf{m} \cdot 0.4\ \mathbf{m} \cdot 21\ \mathbf{m} = 3.36\ \mathbf{m}^3$$

$$n_{columns} := 22$$

$$V_{tot.columns} := V_{columns} \cdot n_{columns} = 73.92\ \mathbf{m}^3$$

Beams in decks:

$$V_{internal.beams} := 0.066\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 8.72\ \mathbf{m} \cdot 3 = 0.699\ \mathbf{m}^3$$

$$V_{end.beams} := 0.14\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 2.12\ \mathbf{m} \cdot 2 = 0.24\ \mathbf{m}^3$$

$$V_{edge.beams} := 0.14\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 9\ \mathbf{m} \cdot 2 = 1.021\ \mathbf{m}^3$$

$$V_{beamsindecks} := V_{internal.beams} + V_{end.beams} + V_{edge.beams} = 1.96\ \mathbf{m}^3$$

$$n_{decks} := 60$$

$$V_{tot.beamsindecks} := V_{beamsindecks} \cdot n_{decks} = 117.616\ \mathbf{m}^3$$

Total

$$V_{tot.columns} + V_{tot.beamsindecks} = 191.536\ \mathbf{m}^3$$

Kerto plate

$$V_{kertoindecks} := (0.063\ \mathbf{m} + 0.045\ \mathbf{m}) \cdot 2.4\ \mathbf{m} \cdot 9\ \mathbf{m} = 2.333\ \mathbf{m}^3$$

$$V_{tot.kertoindecks} := V_{kertoindecks} \cdot n_{decks} = 139.968\ \mathbf{m}^3$$

Steel

per joint

$$A_1 := \left(0.46\ m \cdot 0.13\ m - 0.13\ m \cdot 0.1\ m - 2 \cdot \pi \cdot \left(\frac{0.033}{2}\ m\right)^2\right) = 0.045\ m^2$$

$$A_2 := \left(0.13\ m \cdot 0.46\ m - 5 \cdot \pi \cdot \left(\frac{0.022}{2}\ m\right)^2\right) = 0.058\ m^2$$

$$V_{columns.steelinconnectionplates} := \left(A_1 + A_2\right) \cdot 0.012\ m = \left(1.236 \cdot 10^{-3}\right)\ m^3$$

$$n_{platesoncolumn} := 2$$

$$A_1 := \left(0.12\ m \cdot 0.46\ m + 0.12\ m \cdot 0.46\ m\right) = 0.11\ m^2$$

$$A_2 := \left(2 \cdot 0.045\ m \cdot 0.033\ m + 4 \cdot \pi \cdot \left(\frac{0.022}{2}\ m\right)^2\right) = 0.004\ m^2$$

$$V_{decks.steelinconnectionplates} := \left(A_1 - A_2\right) \cdot 0.012\ m = \left(1.271 \cdot 10^{-3}\right)\ m^3$$

$$n_{platesondeck} := 1$$

$$V_{rodsintowood} := \left(\pi \cdot \left(\frac{0.022}{2}\ m\right)^2 \cdot 0.5\ m\right) = \left(1.901 \cdot 10^{-4}\right)\ m^3$$

$$n_{rodsintowood} := 14$$

$$V_{connectorsbetweenplates} := \pi \cdot \left(\frac{0.033}{2}\ m\right)^2 \cdot 0.05\ m = \left(4.276 \cdot 10^{-5}\right)\ m^3$$

$$n_{connectorrods} := 2$$

$$V_{nuts} := \left(\pi \cdot \left(\frac{0.055}{2}\ m\right)^2 - \pi \cdot \left(\frac{0.033}{2}\ m\right)^2\right) \cdot 0.020\ m = \left(3.041 \cdot 10^{-5}\right)\ m^3$$

$$n_{nuts} := 32$$

Total per joint

$$V_{tot.platesoncolumn} := V_{columns.steelinconnectionplates} \cdot n_{platesoncolumn} = 0.002\ m^3$$

$$V_{tot.platesondeck} := V_{decks.steelinconnectionplates} \cdot n_{platesondeck} = 0.001\ m^3$$

$$V_{tot.connectionplates} := V_{tot.platesoncolumn} + V_{tot.platesondeck} = 0.004\ m^3$$

$$V_{tot.rodsintowood} := V_{rodsintowood} \cdot n_{rodsintowood} = 0.003\ m^3$$

$$V_{tot.connectorrods} := V_{connectorsbetweenplates} \cdot n_{connectorrods} = \left(8.553 \cdot 10^{-5}\right)\ m^3$$

$$V_{tot.nuts} := V_{nuts} \cdot n_{nuts} = \left(9.731 \cdot 10^{-4}\right)\ m^3$$

$$V_{tot.joint} := V_{tot.connectionplates} + V_{tot.rodsintowood} + V_{tot.connectorrods} + V_{tot.nuts} = \left(7.462 \cdot 10^{-3}\right)\ m^3$$

$$n_{joints} := 4 \cdot 60 = 240$$

$$V_{steel.joints} := V_{tot.joint} \cdot n_{joints} = 1.791\ m^3$$

Foundations

$$V_{steelplate} := 0.6\ m \cdot 0.6\ m \cdot 0.05\ m = 0.018\ m^3$$

$$n_{steelplate} := 22$$

$$V_{tot.steelplates} := V_{steelplate} \cdot n_{steelplate} = 0.396\ m^3$$

$$V_{rodsintocolumn} := \pi \cdot \left(\frac{0.022}{2}\ m\right)^2 \cdot 1.2\ m = \left(4.562 \cdot 10^{-4}\right)\ m^3$$

$$n_{rodsintocolumn} := 6 \cdot 22 = 132$$

$$V_{tot.rodsintocolumn} := V_{rodsintocolumn} \cdot n_{rodsintocolumn} = 0.06\ m^3$$

$$V_{dowelsintoconcrete} := \pi \cdot \left(\frac{0.032}{2}\ m\right)^2 \cdot 0.2\ m = \left(1.608 \cdot 10^{-4}\right)\ m^3$$

$$n_{dowelsintoconcrete} := 8 \cdot 22 = 176$$

$$V_{tot.dowelsinconcrete} := V_{dowelsintoconcrete} \cdot n_{dowelsintoconcrete} = 0.028\ m^3$$

$$V_{nuts} := \left(\pi \cdot \left(\frac{0.06}{2}\ m\right)^2 - \pi \cdot \left(\frac{0.032}{2}\ m\right)^2\right) \cdot 0.02\ m = \left(4.046 \cdot 10^{-5}\right)\ m^3$$

$n_{nuts} := 22 \cdot 12 = 264$

$V_{tot.nuts} := V_{nuts} \cdot n_{nuts} = 0.011\ \boldsymbol{m}^3$

Total steel

$V_{steel} := V_{steel.joints} + V_{tot.steelplates} + V_{tot.rodsintocolumn} + V_{tot.dowelsinconcrete} + V_{tot.nuts} = 2.286\ \boldsymbol{m}^3$

Concrete in timber building

Foundation:

$V_{foundation} := 1.85\ \boldsymbol{m} \cdot 1.85\ \boldsymbol{m} \cdot 0.5\ \boldsymbol{m} = 1.711\ \boldsymbol{m}^3$

$n_{foundation} := 22$

$V_{tot.foundation} := V_{foundation} \cdot n_{foundation} = 37.648\ \boldsymbol{m}^3$

kg CO2-eq/m3 for each part

$k_{glulam} := -864.8\ \frac{\boldsymbol{kg}}{\boldsymbol{m}^3}$ $\qquad k_{steel} := 12000.0\ \frac{\boldsymbol{kg}}{\boldsymbol{m}^3}$

$k_{kerto} := -653.0\ \frac{\boldsymbol{kg}}{\boldsymbol{m}^3}$ $\qquad k_{con.foundation} := 508.6\ \frac{\boldsymbol{kg}}{\boldsymbol{m}^3}$

$k_{con.column} := 621.1\ \frac{\boldsymbol{kg}}{\boldsymbol{m}^3}$ $\qquad k_{con.decks} := 423.3\ \frac{\boldsymbol{kg}}{\boldsymbol{m}^3}$

$M_{glulam} := \left(V_{tot.columns} + V_{tot.beamsindecks}\right) \cdot k_{glulam} = -1.656 \cdot 10^5\ \boldsymbol{kg}$

$M_{kerto} := V_{tot.kertoindecks} \cdot k_{kerto} = -9.14 \cdot 10^4\ \boldsymbol{kg}$

$M_{steel} := V_{steel} \cdot k_{steel} = \left(2.743 \cdot 10^4\right)\ \boldsymbol{kg}$

$M_{foundation} := V_{tot.foundation} \cdot k_{con.foundation} = \left(1.915 \cdot 10^4\right)\ \boldsymbol{kg}$

$M_{tot.six.timber} := M_{glulam} + M_{kerto} + M_{steel} + M_{foundation} = -210458.105\ \boldsymbol{kg}$

Concrete in concrete building:

Foundation

Found by forces in Robot put into spreadsheet for foundations

$$V_{foundation} := 2.40\ \boldsymbol{m} \cdot 2.40\ \boldsymbol{m} \cdot 0.5\ \boldsymbol{m} = 2.88\ \boldsymbol{m}^3$$

$$n_{foundatoin} := 22$$

$$V_{tot.foundation} := V_{foundation} \cdot n_{foundation} = 63.36\ \boldsymbol{m}^3$$

Columns

$$V_{column} := 0.3\ \boldsymbol{m} \cdot 0.3\ \boldsymbol{m} \cdot 21\ \boldsymbol{m} = 1.89\ \boldsymbol{m}^3$$

$$n_{column} := 22$$

$$V_{tot.column} := V_{column} \cdot n_{column} = 41.58\ \boldsymbol{m}^3$$

Decks

$$V_{deck} := 2.4\ \boldsymbol{m} \cdot 9\ \boldsymbol{m} \cdot 0.3\ \boldsymbol{m} = 6.48\ \boldsymbol{m}^3$$

$$n_{deck} := 60$$

$$V_{tot.deck} := V_{deck} \cdot n_{deck} = 388.8\ \boldsymbol{m}^3$$

$$M_{column} := V_{tot.column} \cdot k_{con.column} = \left(2.583 \cdot 10^4\right)\ \boldsymbol{kg}$$

$$M_{foundation} := V_{tot.foundation} \cdot k_{con.foundation} = \left(3.222 \cdot 10^4\right)\ \boldsymbol{kg}$$

$$M_{decks} := V_{tot.deck} \cdot k_{con.decks} = \left(1.646 \cdot 10^5\right)\ \boldsymbol{kg}$$

$$M_{tot.six.concrete} := M_{column} + M_{foundation} + M_{decks} = 222629.274\ \boldsymbol{kg}$$

Four stories

Necessary m3 for different parts of the structure

Standard Glulam Beam

Columns:

$$V_{columns} := 0.4\ \mathbf{m} \cdot 0.4\ \mathbf{m} \cdot 14\ \mathbf{m} = 2.24\ \mathbf{m}^3$$

$$n_{columns} := 22$$

$$V_{tot.columns} := V_{columns} \cdot n_{columns} = 49.28\ \mathbf{m}^3$$

Beams in decks:

$$V_{internal.beams} := 0.066\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 8.72\ \mathbf{m} \cdot 3 = 0.699\ \mathbf{m}^3$$

$$V_{end.beams} := 0.14\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 2.12\ \mathbf{m} \cdot 2 = 0.24\ \mathbf{m}^3$$

$$V_{edge.beams} := 0.14\ \mathbf{m} \cdot 0.405\ \mathbf{m} \cdot 9\ \mathbf{m} \cdot 2 = 1.021\ \mathbf{m}^3$$

$$V_{beamsindecks} := V_{internal.beams} + V_{end.beams} + V_{edge.beams} = 1.96\ \mathbf{m}^3$$

$$n_{decks} := 40$$

$$V_{tot.beamsindecks} := V_{beamsindecks} \cdot n_{decks} = 78.411\ \mathbf{m}^3$$

Total

$$V_{tot.columns} + V_{tot.beamsindecks} = 127.691\ \mathbf{m}^3$$

Kerto plate

$$V_{kertoindecks} := (0.063\ \mathbf{m} + 0.045\ \mathbf{m}) \cdot 2.4\ \mathbf{m} \cdot 9\ \mathbf{m} = 2.333\ \mathbf{m}^3$$

$$V_{tot.kertoindecks} := V_{kertoindecks} \cdot n_{decks} = 93.312\ \mathbf{m}^3$$

Steel

per joint

$$A_1 := \left(0.46\ m \cdot 0.13\ m - 0.13\ m \cdot 0.1\ m - 2 \cdot \pi \cdot \left(\frac{0.033}{2}\ m\right)^2\right) = 0.045\ m^2$$

$$A_2 := \left(0.13\ m \cdot 0.46\ m - 5 \cdot \pi \cdot \left(\frac{0.022}{2}\ m\right)^2\right) = 0.058\ m^2$$

$$V_{columns.steelinconnectionplates} := \left(A_1 + A_2\right) \cdot 0.012\ m = \left(1.236 \cdot 10^{-3}\right)\ m^3$$

$$n_{platesoncolumn} := 2$$

$$A_1 := \left(0.12\ m \cdot 0.46\ m + 0.12\ m \cdot 0.46\ m\right) = 0.11\ m^2$$

$$A_2 := \left(2 \cdot 0.045\ m \cdot 0.033\ m + 4 \cdot \pi \cdot \left(\frac{0.022}{2}\ m\right)^2\right) = 0.004\ m^2$$

$$V_{decks.steelinconnectionplates} := \left(A_1 - A_2\right) \cdot 0.012\ m = \left(1.271 \cdot 10^{-3}\right)\ m^3$$

$$n_{platesondeck} := 1$$

$$V_{rodsintowood} := \left(\pi \cdot \left(\frac{0.022}{2}\ m\right)^2 \cdot 0.5\ m\right) = \left(1.901 \cdot 10^{-4}\right)\ m^3$$

$$n_{rodsintowood} := 14$$

$$V_{connectorsbetweenplates} := \pi \cdot \left(\frac{0.033}{2}\ m\right)^2 \cdot 0.05\ m = \left(4.276 \cdot 10^{-5}\right)\ m^3$$

$$n_{connectorrods} := 2$$

$$V_{nuts} := \left(\pi \cdot \left(\frac{0.055}{2}\ m\right)^2 - \pi \cdot \left(\frac{0.033}{2}\ m\right)^2\right) \cdot 0.020\ m = \left(3.041 \cdot 10^{-5}\right)\ m^3$$

$$n_{nuts} := 32$$

Total per joint

$$V_{tot.platesoncolumn} := V_{columns.steelinconnectionplates} \cdot n_{platesoncolumn} = 0.002\ m^3$$

$$V_{tot.platesondeck} := V_{decks.steelinconnectionplates} \cdot n_{platesondeck} = 0.001\ m^3$$

$$V_{tot.connectionplates} := V_{tot.platesoncolumn} + V_{tot.platesondeck} = 0.004\ m^3$$

$$V_{tot.rodsintowood} := V_{rodsintowood} \cdot n_{rodsintowood} = 0.003\ m^3$$

$$V_{tot.connectorrods} := V_{connectorsbetweenplates} \cdot n_{connectorrods} = \left(8.553 \cdot 10^{-5}\right)\ m^3$$

$$V_{tot.nuts} := V_{nuts} \cdot n_{nuts} = \left(9.731 \cdot 10^{-4}\right)\ m^3$$

$$V_{tot.joint} := V_{tot.connectionplates} + V_{tot.rodsintowood} + V_{tot.connectorrods} + V_{tot.nuts} = \left(7.462 \cdot 10^{-3}\right)\ m^3$$

$$n_{joints} := 4 \cdot 40 = 160$$

$$V_{steel.joints} := V_{tot.joint} \cdot n_{joints} = 1.194\ m^3$$

Foundations

$$V_{steelplate} := 0.6\ m \cdot 0.6\ m \cdot 0.05\ m = 0.018\ m^3$$

$$n_{steelplate} := 22$$

$$V_{tot.steelplates} := V_{steelplate} \cdot n_{steelplate} = 0.396\ m^3$$

$$V_{rodsintocolumn} := \pi \cdot \left(\frac{0.022}{2}\ m\right)^2 \cdot 1.2\ m = \left(4.562 \cdot 10^{-4}\right)\ m^3$$

$$n_{rodsintocolumn} := 6 \cdot 22 = 132$$

$$V_{tot.rodsintocolumn} := V_{rodsintocolumn} \cdot n_{rodsintocolumn} = 0.06\ m^3$$

$$V_{dowelsintoconcrete} := \pi \cdot \left(\frac{0.032}{2}\ m\right)^2 \cdot 0.2\ m = \left(1.608 \cdot 10^{-4}\right)\ m^3$$

$$n_{dowelsintoconcrete} := 8 \cdot 22 = 176$$

$$V_{tot.dowelsinconcrete} := V_{dowelsintoconcrete} \cdot n_{dowelsintoconcrete} = 0.028\ m^3$$

$$V_{nuts} := \left(\pi \cdot \left(\frac{0.06}{2}\ m\right)^2 - \pi \cdot \left(\frac{0.032}{2}\ m\right)^2\right) \cdot 0.02\ m = \left(4.046 \cdot 10^{-5}\right)\ m^3$$

$$n_{nuts} := 22 \cdot 12 = 264$$

$$V_{tot.nuts} := V_{nuts} \cdot n_{nuts} = 0.011 \ m^3$$

Total steel

$$V_{steel} := V_{steel.joints} + V_{tot.steelplates} + V_{tot.rodsintocolumn} + V_{tot.dowelsinconcrete} + V_{tot.nuts} = 1.689 \ m^3$$

Concrete in timber building

Foundation:

$$V_{foundation} := 1.6 \ m \cdot 1.6 \ m \cdot 0.5 \ m = 1.28 \ m^3$$

$$n_{foundation} := 22$$

$$V_{tot.foundation} := V_{foundation} \cdot n_{foundation} = 28.16 \ m^3$$

kg CO2-eq/m3 for each part

$$k_{glulam} := -864.8 \ \frac{kg}{m^3} \qquad k_{steel} := 12000.0 \ \frac{kg}{m^3}$$

$$k_{kerto} := -653.0 \ \frac{kg}{m^3} \qquad k_{con.foundation} := 508.6 \ \frac{kg}{m^3}$$

$$k_{con.column} := 621.1 \ \frac{kg}{m^3} \qquad k_{con.decks} := 423.3 \ \frac{kg}{m^3}$$

$$M_{glulam} := \left(V_{tot.columns} + V_{tot.beamsindecks}\right) \cdot k_{glulam} = -1.104 \cdot 10^5 \ kg$$

$$M_{kerto} := V_{tot.kertoindecks} \cdot k_{kerto} = -6.093 \cdot 10^4 \ kg$$

$$M_{steel} := V_{steel} \cdot k_{steel} = \left(2.027 \cdot 10^4\right) \ kg$$

$$M_{foundation} := V_{tot.foundation} \cdot k_{con.foundation} = \left(1.432 \cdot 10^4\right) \ kg$$

$$M_{tot.four.timber} := M_{glulam} + M_{kerto} + M_{steel} + M_{foundation} = -136767.42 \ kg$$

Concrete in concrete building:

Foundation

Found by forces in Robot put into spreadsheet for foundations

$$V_{foundation} := 2.0\ \boldsymbol{m} \cdot 2.0\ \boldsymbol{m} \cdot 0.5\ \boldsymbol{m} = 2\ \boldsymbol{m}^3$$

$$n_{foundatoin} := 22$$

$$V_{tot.foundation} := V_{foundation} \cdot n_{foundation} = 44\ \boldsymbol{m}^3$$

Columns

$$V_{column} := 0.3\ \boldsymbol{m} \cdot 0.3\ \boldsymbol{m} \cdot 14\ \boldsymbol{m} = 1.26\ \boldsymbol{m}^3$$

$$n_{column} := 22$$

$$V_{tot.column} := V_{column} \cdot n_{column} = 27.72\ \boldsymbol{m}^3$$

Decks

$$V_{deck} := 2.4\ \boldsymbol{m} \cdot 9\ \boldsymbol{m} \cdot 0.3\ \boldsymbol{m} = 6.48\ \boldsymbol{m}^3$$

$$n_{deck} := 40$$

$$V_{tot.deck} := V_{deck} \cdot n_{deck} = 259.2\ \boldsymbol{m}^3$$

$$M_{column} := V_{tot.column} \cdot k_{con.column} = \left(1.722 \cdot 10^4\right)\ \boldsymbol{kg}$$

$$M_{foundation} := V_{tot.foundation} \cdot k_{con.foundation} = \left(2.238 \cdot 10^4\right)\ \boldsymbol{kg}$$

$$M_{decks} := V_{tot.deck} \cdot k_{con.decks} = \left(1.097 \cdot 10^5\right)\ \boldsymbol{kg}$$

$$M_{tot.six.concrete} := M_{column} + M_{foundation} + M_{decks} = 149314.652\ \boldsymbol{kg}$$

A.10 Mathcad foundations

Dimensioning of foundations - www.Civilglobal.com

Eight story timber building:

Foudation for a 400 x 400 [mm] column with axial forces of 650 kN and a moment of 259 kNm.

$D := 400\ mm$ Width of column

Foundation footing

$N_1 := 695\ kN$ Compression force

$M_1 := 259\ kN \cdot m$ Moment

$f_{ck} := 45\ \frac{N}{mm^2}$ C45 concrete

$q_{soil} := 100\ \frac{kN}{m^2}$ Safe bearing capacity medium dense sand.

$A_{req} := \frac{N_1 \cdot 1.5}{q_{soil}} = 10.425\ m^2$ Require ground area

$B := \sqrt{A_{req}} = 3.229\ m$

$L := B = 3.229\ m$ Square footing of 3,2 x 3,2 m

Soil reaction

$$P_u := \frac{1.5 \cdot N_1}{A_{req}} = 100\ \frac{kN}{m^2}$$

$$e := \frac{M_1}{N_1} = 372.662\ mm$$

$$\frac{L}{6} = 538.129\ mm$$

Factor moment

$$M_u := N_1 \cdot B \cdot \left(\frac{L-D}{2}\right)^2 \cdot 0.5$$

$$M := M_u + M_1 = \left(1.391 \cdot 10^3\right)\ kN \cdot m$$

Column depth

$$d := \sqrt{\frac{M}{0.138 \cdot f_{ck} \cdot L}} = 263.431\ mm$$

Foudation for a 400 x 400 [mm] column with axial forces of 650 kN and a moment of 259 kNm.

$D := 400\ mm$ Width of column

Foundation footing

$N_1 := 695\ kN$ Compression force

$M_1 := 259\ kN \cdot m$ Moment

$f_{ck} := 45\ \frac{N}{mm^2}$ C20 concrete

$q_{soil} := 245\ \frac{kN}{m^2}$ Safe bearing capacity medium dense sand.

$A_{req} := \frac{N_1 \cdot 1.5}{q_{soil}} = 4.255\ m^2$ Require ground area

$B := \sqrt{A_{req}} = 2.063\ m$

$L := B = 2.063\ m$ Square footing of 2,1 x 2,1 m

Soil reaction

$P_u := \frac{1.5 \cdot N_1}{A_{req}} = 245\ \frac{kN}{m^2}$

$e := \frac{M_1}{N_1} = 372.662\ mm$

Factor moment

$\frac{L}{6} = 343.798\ mm$

$M_u := N_1 \cdot B \cdot \left(\frac{L-D}{2}\right)^2 \cdot 0.5 = 391.283\ kN \cdot m$

$M := M_u + M_1 = 650.283\ kN \cdot m$

Column depth

$d := \sqrt{\frac{M}{0.138 \cdot f_{ck} \cdot L}} = 225.309\ mm$

Foudation for a 400 x 400 [mm] column with axial forces of 650 kN and a moment of 259 kNm.

$D := 400\ mm$ Width of column

Foundation footing

$N_1 := 695\ kN$ Compression force

$M_1 := 259\ kN \cdot m$ Moment

$f_{ck} := 20\ \frac{N}{mm^2}$ C20 concrete

$q_{soil} := 440\ \frac{kN}{m^2}$ Safe bearing capacity medium dense sand.

$A_{req} := \frac{N_1 \cdot 1.5}{q_{soil}} = 2.369\ m^2$ Require ground area

$B := \sqrt{A_{req}} = 1.539\ m$

$L := B = 1.539\ m$ Square footing of 1,5 x 1,5 m

Soil reaction

$P_u := \frac{1.5 \cdot N_1}{A_{req}} = 440\ \frac{kN}{m^2}$

$e := \frac{M_1}{N_1} = 372.662\ mm$

$\frac{L}{6} = 256.543\ mm$

Factor moment

$M_u := N_1 \cdot B \cdot \left(\frac{L-D}{2}\right)^2 \cdot 0.5$

$M := M_u + M_1 = 442.68\ kN \cdot m$

foundation depth

$d := \sqrt{\frac{M}{0.138 \cdot f_{ck} \cdot L}} = 322.801\ mm$

Dimensioning of foundations - www.Civilglobal.com

Eight story concrete building:

Foudation for a 400 x 400 [mm] column with axial forces of 1613 kN and a moment of 113 kNm.

$D := 400\ mm$ Width of column

Foundation footing

$N_1 := 1613\ kN$ Compression force

$M_1 := 113\ kN \cdot m$ Moment

$f_{ck} := 45\ \frac{N}{mm^2}$ C45 concrete

$q_{soil} := 100\ \frac{kN}{m^2}$ Safe bearing capacity medium dense sand.

$A_{req} := \frac{N_1 \cdot 1.5}{q_{soil}} = 24.195\ m^2$ Require ground area

$B := \sqrt{A_{req}} = 4.919\ m$

$L := B = 4.919\ m$ Square footing of 4,9 x 4,9 m

Soil reaction

$$P_u := \frac{1.5 \cdot N_1}{A_{req}} = 100\ \frac{kN}{m^2}$$

$$e := \frac{M_1}{N_1} = 70.056\ mm$$

$$\frac{L}{6} = 819.807\ mm$$

Factor moment

$$M_u := N_1 \cdot B \cdot \left(\frac{L-D}{2}\right)^2 \cdot 0.5$$

$$M := M_u + M_1 = \left(6.82 \cdot 10^3\right)\ kN \cdot m$$

Column depth

$$d := \sqrt{\frac{M}{0.138 \cdot f_{ck} \cdot L}} = 472.51\ mm$$

Foudation for a 400 x 400 [mm] column with axial forces of 650 kN and a moment of 259 kNm.

$D := 400\ mm$ Width of column

Foundation footing

$f_{ck} := 45\ \frac{N}{mm^2}$ C45 concrete

$q_{soil} := 245\ \frac{kN}{m^2}$ Safe bearing capacity medium dense sand.

$A_{req} := \frac{N_1 \cdot 1.5}{q_{soil}} = 9.876\ m^2$ Require ground area

$B := \sqrt{A_{req}} = 3.143\ m$

$L := B = 3.143\ m$ Square footing of 3,1 x 3,1 m

Soil reaction

$P_u := \frac{1.5 \cdot N_1}{A_{req}} = 245\ \frac{kN}{m^2}$

$e := \frac{M_1}{N_1} = 70.056\ mm$

Factor moment

$\frac{L}{6} = 523.755\ mm$

$M_u := N_1 \cdot B \cdot \left(\frac{L-D}{2}\right)^2 \cdot 0.5$

$M := M_u + M_1 = \left(2.583 \cdot 10^3\right)\ kN \cdot m$

Column depth

$d := \sqrt{\frac{M}{0.138 \cdot f_{ck} \cdot L}} = 363.84\ mm$

Foudation for a 400 x 400 [mm] column with axial forces of 650 kN and a moment of 259 kNm.

$D:=400\ mm$ Width of column

Foundation footing

$f_{ck}:=45\ \frac{N}{mm^2}$ C45 concrete

$q_{soil}:=440\ \frac{kN}{m^2}$ Safe bearing capacity medium dense sand.

$A_{req}:=\frac{N_1\cdot 1.5}{q_{soil}}=5.499\ m^2$ Require ground area

$B:=\sqrt{A_{req}}=2.345\ m$

$L:=B=2.345\ m$ Square footing of 2,4 x 2,4 m

Soil reaction

$P_u:=\frac{1.5\cdot N_1}{A_{req}}=440\ \frac{kN}{m^2}$

$e:=\frac{M_1}{N_1}=70.056\ mm$

$\frac{L}{6}=390.828\ mm$

Factor moment

$M_u:=N_1\cdot B\cdot\left(\frac{L-D}{2}\right)^2\cdot 0.5$

$M:=M_u+M_1=\left(1.355\cdot 10^3\right)\ kN\cdot m$

foundation depth

$d:=\sqrt{\frac{M}{0.138\cdot f_{ck}\cdot L}}=305.093\ mm$

Foudation for a 400 x 400 [mm] column with axial forces of 650 kN and a moment of 259 kNm.

$D := 400\ mm$ Width of column

Foundation footing

C45 concrete

$f_{ck} := 45\ \frac{N}{mm^2}$

Safe bearing capacity medium dense sand.

$q_{soil} := 3240\ \frac{kN}{m^2}$

Require ground area

$A_{req} := \frac{N_1 \cdot 1.5}{q_{soil}} = 0.747\ m^2$

$B := \sqrt{A_{req}} = 0.864\ m$

Square footing of 1 x 1 m

$L := B = 0.864\ m$

Soil reaction

$P_u := \frac{1.5 \cdot N_1}{A_{req}} = \left(3.24 \cdot 10^3\right)\ \frac{kN}{m^2}$

$e := \frac{M_1}{N_1} = 70.056\ mm$

Factor moment

$\frac{L}{6} = 144.025\ mm$

$M_u := N_1 \cdot B \cdot \left(\frac{L-D}{2}\right)^2 \cdot 0.5$

$M := M_u + M_1 = 183.76\ kN \cdot m$

foundation depth

$d := \sqrt{\frac{M}{0.138 \cdot f_{ck} \cdot L}} = 185.048\ mm$

B E-mails

B.1 Transport of decks

Hei !!

27 tonn er grensen på laste vekten.
Det kan bli gitt disp på høyere vekt, men da skal dette være i 1 kolli

Du må være oppmerksom på lengde.

En semi har normalt 12,6 eller 13,6 lengde på lasteplanet.
Dette medfører at du ikke kan legge 2 lengder etter hverandre som gir total lengde utover disse lengdene.

Vis vi for limtre har en lengde på 18 meter som kjøres på en uttrekkbar semi, så kan vi ikke legge 2 lengder på 8 meter etter hverandre på lengden som måler 18 meter.

Vi må holde oss innenfor normal lengden for lasteplanet på øvrige varer på samme lasset…

Med vennlig hilsen
Bernt Grindvoll
Transportleder / Hovedplanlegger

Moelven Limtre AS

Telefon: +4790522811

moelven.no/Limtre

B.2 Transport of columns

Hei !!

Beklager sen tilbakemelding

Største laste vekt du kan legge på en semitrailer med flere kolli er 27 tonn (kan i noen tilfeller søkes disp på større vekt, men da kun for ett kolli)
I dette tilfellet får du på 14 stk. som gir 26,6 tonn

Transportkostnad har jeg beregnet til kr. 52.000,-

Dette er inkl. 2 ledsagerbiler og politi eskorte..

Med lastlengde på 28 meter blir det transportlengde over 30 meter, og dette krever politieskorte og 2 følgebiler for transporten

Her har jeg lagt kjøreruta fra Moelv via Elverum til Trondheim. Dette pga. dårlig fremkommelighet på Dombås.

Med vennlig hilsen
Bernt Grindvoll
Transportleder / Hovedplanlegger

Moelven Limtre AS

Telefon: +4790522811

moelven.no/Limtre

B.3 Price of crane E.D. Knutsen

Hei

Gjennomsnittlig leie kr 40.000,- pr.mnd og rigg opp/ned : kr 240.000,-

Med vennlig hilsen

Kjetil Tettum
Mob.: +47 92851335

E.D.Knutsen | UCO
www.edknutsen.no
Postboks 23, 2027 KJELLER
Besøksadresse: Brånåsparken 9, 2007 Kjeller
Tlf.: +47 63 88 58 00
E-post: kjetil.tettum@edknutsen.no

B.4 Price of steel plate from Smith Stål

Hei,

Det vil bli i underkant av 7500 kr eks. mva ferdig skjært.

NB! Kvalitet NVE 36.

Vennlig hilsen / Best regards

Stian Jakobsen
Salgskonsulent/Kundeansvarlig
E.A. Smith AS avd. Smith Stål Nord Trondheim
Dir. tlf. +4772592340 | www.smith.no

SMITH STÅL

B.5 Price Glulam Moelven

Hei

Blokklimt tverrsnitt (190 +210) x 400 mm GL 30C gran.

Pris pr m3 ab. fabrikk: ca kr 11 000,- pr m3 eks. mva.

28 meter lengde er spesialtransport og pris pr m3 er volumavhengig. Må kalkuleres separat.

Med vennlig hilsen
Kato Sveen
Prosjektsjef

Moelven Limtre AS

Telefon: +4790859468

moelven.no/Limtre

B.6 Price of steel rod

Prisen på dekke-elementer har vi veldig liten info om, må nesten se på råvareprisene og så addere på produksjonskostnader.

For steel rods tror jeg 100 kr pr rod er ok pris.

Kjell A. Malo
Professor, Dr. ing.
Department of Structural Engineering, Norwegian University of Science and Technology, NTNU NO 7491 Trondheim, Norway. Phone: +47 73594784, E-mail: kjell.malo@ntnu.no

B.7 Price of self-erecting tower crane

Hei Ivar.

Ca. pris for leie av Potain IGO T130 er kr. 55.00,- pr. mnd.
Ca. totalpris for montering og demontering av kran i Oslo kr. 80.000,-
I tillegg kommer kost for fundament til kran.

Med vennlig hilsen / Best Regards

Øyvind Frantzen
Kranor AS
Salg/ Utleie øst
Mobil: +47 41 66 44 88
www.kranor.no
Adr: Eternittveien 10, 3470 Slemmestad, Norway
Sentralbord: +47 31 29 78 00