Master's thesis

NTNU
Norwegian University of Science and Technology
Faculty of Engineering
Department of Structural Engineering

Aurora Marie Jørgensen

Design of Community Center in Rural Ghana Using Shipping Containers

In Collaboration with Engineers Without Borders Norway

Master's thesis in Structural Engineering
Supervisor: Arne Aalberg
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Design av samfunnssenter på Ghanas landsbygd ved bruk av shipping containere
I samarbeid med Ingeniører Uten Grenser Norge

BY:

Aurora Marie Jørgensen

SUMMARY:
This thesis concerns the preliminary design of a community center and accommodation on the Trax-Kavli farm in the northern part of Ghana. The community center is intended to be designed using shipping containers on request from Trax Ghana.

The design of the community center has been developed by referring to Ghanaian and Norwegian building regulations. The suggested layout consists of four 40 ft. shipping containers forming the community center and accommodation. Two additional 20 ft. shipping containers hold hygiene facilities for girls and boys. External roofs acting as shading are added to the design to avoid excessive heating of the indoor environment due to solar radiation.

Design of roof supporting structures made of treated wood and foundations made of reinforced concrete are studied. Finite element models for both the roof supporting structures and the foundations are built using Autodesk Robot Structural Analysis Professional 2019. Mat foundations and a roof supporting structure using trusses are considered the best options for the added structures in this project.

RESPONSIBLE TEACHER: Arne Aalberg

SUPERVISOR(S) Arne Aalberg

CARRIED OUT AT: Department of Structural Engineering

NTNU
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MASTEROPPGAVE VÅREN 2021

Aurora Marie Jørgensen

Utforming av samfunnssenter i Ghana ved bruk av shipping containere

Design of community center in rural Ghana using shipping containers

Bakgrunn

Oppgaven er tilknyttet et prosjekt som skal utvikle et samfunnssenter nord i Ghana. Oppgaven utføres i samarbeid med Ingeniører Uten Grenser. Hensikten er å utvikle et skisseprosjekt som skal motivere og legges til grunn ved innhenting av finansiering til prosjektet. Feltarbeid i Ghana i starten av januar var påtenkt, men grunnet Covid-19 restriksjoner ble dette ikke mulig. Forprosjektet i sin helhet vil derfor være utført fra Norge. Deler av konstruksjonen tenkes utviklet med gjenbruk av shipping containere i stål etter ønske fra prosjekteier. Det er en rekke elementer og vurderinger som kan gjøres, bl.a. funksjon, form, tilgjengelighet av materialer, bygningsfysikk (lys, lyd, varme), brannsikring, laster, fundamentering og bærekonstruksjon, samt egnethet for vedlikehold og reparasjon.

Oppgave

Oppgaven er å gjøre en systematisk utgreing av et mulig prosjekt som kan fungere i Ghana, med størrelse som tiltenkt. Prosjektet var ved oppstart av masteroppgaven veldig uklart, og det blir i stor grad opp til kandidaten å knytte kontakter og finne ut det hun trenger om funksjonskrav, ønsker og muligheter.

Oppgaven kan inneholde elementer som:

- Litteraturundersøkelse av containerbygg, byggemetoder og klima i Ghana.
- Skissere rammene for bygget/senteret og velge en passende konstruksjon.
- Se på elementer av bygningskrav, byggemåter, bygningsfysikk og økonomi.
- Vurdere materialenes egnethet.
- Annet.

Oppgaven er relativt åpen. Kandidatene velger fritt hva hun velger å konsentrere seg om, så lenge det er relevant for dette eller lignende prosjekter.

Rapporten
Oppgaven skal skrives som en teknisk rapport og ha gode figurer, tabeller og foto. Rapporten skal inneholde tittelside, forord, oppgavetekst, sammendrag, innholdsfortegnelse, symbolliste (om nødvendig), et fornuftig antall kapitler (med underkapitler), konklusjoner som siste kapittel, referanseliste og vedlegg.
Informasjon om dette er også sendt ut fra instituttet. Det innleveres gjennom Inspera.

Omslag kan med fordel ha en illustrasjon fra oppgaven på framsiden.

Faglærer ønsker en trykket versjon av oppgaven. Faglærer ønsker videre at det lages en pakke med filer fra arbeidet, med rapporten, evt. forsøksresultater, bilder, bakgrunnslitteraturen, elementmodellene, etc. Dette for å lette oppstarten av studentoppgaver som kan tenkes å skulle fortsette undersøkelser på området.

Masteroppgaven skal leveres innen 10. juni 2021

Arne Aalberg

Trondheim, 10. juni 2021
Arne Aalberg, Professor

Preface

This master's thesis concludes the 2 year-long master's degree in civil- and environmental engineering at the Department of Structural Engineering at the Norwegian University of Science and Technology (NTNU).

The thesis is written in collaboration with Engineers Without Borders Norway and concerns the preliminary design of a community center in rural Ghana, aiming to reduce poverty and inequality among genders and different groups of the society. Being fortunate enough myself to grow up as a girl in Norway and choosing my own path in life, I find great motivation in being a part of a project like this providing other not so fortunate girls with a tool to break out of poverty allowing them to take charge of their own future.

By utilizing the knowledge I have obtained during my five years at NTNU, the past 20 weeks have been filled with ups and downs and a decent amount of frustration. However, I can think of no better way of spending my final semester at NTNU than on a project like this.

With that being said, this thesis would never have been a reality had it not been for a handful of people supporting and guiding me through this semester. I would like to thank my supervisor Arne Aalberg for taking on the role as supervisor from NTNU. Without him this thesis would never have been possible to carry out. Secondly, I would like to thank the project team from Engineers Without Borders who have worked hard to define and specify the needs for the project through meetings once a week. A special thank you goes to my mentor Haldor Fosse, for bringing in knowledge and experience from previous development projects, motivating and guiding me at times when the amount of work and the scope of the project have seemed overwhelming. Lastly, I would like to thank Arvid Dalehaug for spending time after his retirement, answering my questions regarding thermal comfort issues of the project.

Trondheim,
June 10, 2021

Aurora Jørgensen

Aurora Marie Jørgensen

Abstract

This thesis concerns the preliminary design of a community center and accommodation on the Trax-Kavli farm in the northern part of Ghana. The farm is owned by the non-governmental organization Trax Ghana and aims to provide the local community with supplementary teaching facilities in addition to the education given by the public schools. Girls, in particular, and other vulnerable groups of the community are the main target groups for this project.

The community center is intended to be designed out of used shipping containers by the request from Trax Ghana. These containers are relatively cheap to purchase and provide waterproof enclosures and a shortened construction time. Use of local labor and locally available materials have been emphasized for the remainder of the project. A literature review concerning shipping container architecture and local building materials in Ghana have been conducted.

The design of the community center has been developed by referring to Ghanaian and Norwegian building regulations. The suggested layout consists of four 40 ft. shipping containers forming a classroom, library, Trax office and accommodation facilities. Two additional 20 ft. shipping containers hold pit latrine toilets and shower facilities for girls and boys. To avoid excessive heating of the indoor environment due to solar radiation, shading in terms of external roofs are added on top of the containers and the outdoor area between them. The roof surfaces also serve as surfaces for rainwater harvesting increasing the water supply on the farm.

Design of roof supporting structures made of treated wood and foundations made of reinforced concrete are investigated. Finite element models for both the roof supporting structures and the foundations are built using Autodesk Robot Structural Analysis Professional 2019. Due to the long span width of the beams supporting the roof over the outdoor area, truss beams have been considered the most favorable solution. Design of the timber truss and other roof members have been done according to EN 1995-1-1. Foundation types such as spread footing foundations and mat foundations are studied. Calculations show that the foundations are subjected to large uplifting forces due to wind loads, thus, mat foundations are considered the most reasonable foundation type for the containers.

Sammendrag

Denne oppgaven omhandler et skisseprosjekt av et samfunnssenter med tilhørende innkvartering på Trax-Kavli gården nord i Ghana. Gården eies av bistandsorganisasjonen Trax Ghana, og har som formål å tilby befolkningen i området et sted med læringsfasiliteter som fungerer som et tillegg til undervisningen som gis ved de offentlige skolene. Særlig jenter og andre sårbare grupper i samfunnet er hovedmålgruppen for dette prosjektet.

Samfunnssenteret er tenkt utformet av brukte shipping containere etter ønske fra Trax Ghana. Slike containere er relativt billige å anskaffe, i tillegg til at de sørger for et vanntett klimaskall og forkorter byggeprosessen. For de resterende delene av prosjektet er bruken av lokal arbeidskraft og lokale materialer vektlagt. I den forbindelse er en litteraturstudie av shipping container-arkitektur og lokale byggematerialer i Ghana gjennomført.

Utformingen av samfunnssenteret er utviklet ved å følge anbefalinger i ghanesiske og norske byggeforskrifter. Det foreslåtte designet av samfunnssenteret består av fire 40-fots shipping containere som til sammen utgjør et klasserom, bibliotek, Trax-kontor og innkvarteringsfasiliteter. I tillegg huser to 20-fots containere latrinetoaletter og dusjfasiliteter for jenter og gutter. For å unngå unødvendig oppvarming av innendørsklimaet på grunn av solstråling, er det tenkt å legge luftede takkonstruksjoner over alle containerne i tillegg til utendørsområdet mellom disse. Takflatene fungerer også som flater for regnvannsoppsamling for å kunne øke vanntilførselen på gården.

Undersøkelser av takkonstruksjoner i behandlet trevirke og fundamenter i armert betong er utført. FEM-modeller for både takkonstruksjoner og fundamenter er modellert i Autodesk Robot Structural Analysis Professional 2019. På grunn av den store spennvidden for bjelkene som understøtter taket over utendørsområdet er fagverksbjelker ansett som den best egnede løsningen. Dimensjonering av fagverksbjelkene i tre og de andre tilhørende takelementene er gjort i henhold til EN 1995-1-1. Direkte fundamentering slik som punktfundamenter og sålefundamenter er vurdert med tanke på fundamentering av containerne. Gjennom beregninger ble det oppdaget at fundamentene er utsatt for store oppløftskrefter på grunn av vindlastene. Sålefundamenter er derfor antatt som den beste løsningen for fundamentering av containerne.

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1 Introduction

This master's thesis is written in collaboration with Engineers Without Borders (EWB) on behalf of Trax Ghana. Trax Ghana is a non-governmental organization which aims at reducing poverty, improving health, and ensuring food security in rural communities in the northern part of Ghana (Trax Ghana, n.d.). Similarly, EWB is a non-governmental organization providing engineering assistance in developing projects (Engineers Without Borders, n.d.).

This thesis concerns the preliminary design of a community center and accommodation on the Trax-Kavli farm owned by Trax Ghana. To apply for funds at the end of this phase an estimation of quantities concerning materials and inventory must be established, thus, being the main focus of this thesis. A focus on solutions using local materials and labor is emphasized. The necessary inventory in terms of furniture for the community center and accommodation, and water and sanitation facilities are determined by other members of the project team. Thus, these aspects will not be covered in this thesis.

Located in the Upper East Region of Ghana, in the Bongo district near the town of Bolgatanga, the Trax-Kavli farm is found. The farm was purchased in 2016 with funds from the Norwegian Kavli Foundation. Currently, a goat farm providing goat scholarships to students are established on the farm. The scholarships involve lending out goats for the students to care for, ensuring an income for their families and enabling them to buy uniforms and books by selling goat offsprings and fertilizer made from goat manure. Solar panels have been installed on the roof of the goat farm by a team from EWB Norway providing electricity to the farm. In addition, there is a bore hole on the farm providing water for farming purposes. An extension of the farm including a greenhouse where food can be grown year-round without worrying about the weather conditions and a community center with accommodation for educational purposes are intended to be a part of the farm in the future. Design of the greenhouse have already been done by another team from EWB and will not be discussed any further in this thesis.

Figure 1-1: Trax goat farm with solar panels on roof.
Copyright: Engineers Without Borders

In order to ensure cost effective and viable solutions in the design a literature study concerning shipping container architecture and local building materials in Ghana have been conducted. Findings from this study are reproduced in chapter 2. Furthermore, considerations regarding the requirements for buildings in accordance with the Norwegian building regulations (TEK17) have been integrated into the design. Implementation of the Norwegian building regulations is done to increase the feasibility of the project and boost the chances of requiting Norwegian sponsors to the project. Passive measures to improve the indoor temperatures have been investigated. The use of mechanical ventilation which depends on frequent maintenance and power supply is not preferable. Lastly, structural considerations for added structures are assessed using Eurocodes.

The end of the preliminary phase should result in a list of materials needed for the project to be used in the application for funding. The contents of this list are discussed in the coming chapters, and the complete list can be found in Appendix A.

1.1 Background

Bordering to the Ivory Coast in the west, Togo in the east and Burkina Faso in the north, the Republic of Ghana is located on the west coast of Africa. The southern part of Ghana holds tropical rain forests in the west and montane forests in the east. In contrast to the southern parts, the northern part of the country is characterized by a much drier climate with large variations in temperature and precipitation during the year (United Nations, 2019).

Figure 1-2: Map of Ghana with location of the Trax-Kavli farm. Source: Google Maps

Figure 1-3: Trax-Kavli scholarship farm. Copyright: Engineers Without Borders.

In line with the United Nation's Sustainability Development Goals, Ghana have actively taken measures to reduce the poverty of its population. However, in 2016, studies showed that the number of people living in poverty were almost four times higher in rural areas than in urban areas. Rural areas in the Northern, Upper East and Upper West regions were the regions with the highest poverty rate (Cooke et al., 2016).

Education have proven to be one of the most effective ways of combating poverty (Rolleston, 2011, Edmond, 2017) . By ensuring that sufficient education is provided, not only to the richest but to all parts of the society, people living in rural communities have a tool to fight poverty enabling them to apply for university scholarships and well-paid jobs. Public education in Ghana is free, however, according to Trax representatives a significant difference in the quality of public and private education exists.

The public educational system in Ghana consists of three levels: primary school (6 years), junior high school (3 years) and senior high school (3 years). The dropout rate for girls is significantly higher than the dropout rate for boys in all stages of education. In 2010, 87,1% of all children enrolled in primary school finished their education. For girls, however, the number of students completing primary school was only 48,7%. Dropout rates for both boys and girls increase with the level of education, with girls having the highest dropout rates of the two genders (Edmond, 2017).

Studies from the Kassena Nankana West District, which is neighboring to the Bongo district, show that 46,8% of all girls in the age of 12-14 years enrolled in junior high school do not finish their education. Among the most common reasons for school dropouts for girls in junior high school are their parents' inability to provide supplies for further education, early marriage and pregnancy (Arko, 2013). A lack of reproductive health education is suggested to be the main reason for unwanted pregnancies and HIV/AIDS infections (Rondini and Krugu, 2009).

To bridge the educational gap between boys and girls, public and private schools, the community center on the Trax-Kavli farm is intended to provide a place for further education and a possibility for students who have dropped out of school to finish their basic education. A focus on reproductive health education, improving the awareness and knowledge about sexually transmitted diseases such as HIV/AIDS infections and contraception use to prevent unwanted pregnancies, are also envisioned to be a part of the agenda at the community center.

In the cases where parents are not able to pay for supplies to allow their kids to continue in school, most of the girls in these families are married off to older men at an early age to be able to provide food and supplies for the rest of their family. These marriages often result in unwanted pregnancies making the girls unable to continue their education. To prevent the girls from being married off against their will, the kids can come and work in the planned greenhouses on the Trax-Kavli farm. Thus, being able to stay in school by providing an income for themselves and their families by growing food and selling some of the crops.

Another vulnerable group of the society in Ghana, apart from young girls, are people with different disabilities. The enhancement of life quality and inclusion of disabled persons in society are covered by both the United Nation's Sustainability Development Goals and the Constitution of Ghana (1992). The community center is envisioned to be accessible for all members of the rural community. Measures to include disabled persons must therefore be considered in the design of the community center.

2 Literature review

2.1 Shipping container architecture

Abandoned shipping containers in seaports have become a common sight in many countries around the world. This is especially a problem in importing countries where the cost of sending them back to its original destination exceeds the costs of manufacturing new ones (Elrayies, 2017). Used and abandoned shipping containers therefore have potential of extending their life span by refurbishing and altering their original layout for housing purposes.

In African countries such as Ghana and Nigeria housing deficit are becoming an increasing problem. In Nigeria studies have been conducted on the possibility of reusing shipping containers as a part of the solution to the housing crisis, offering cheaper houses than using traditional building materials (Oloto and Adebayo, 2015). The Ghanaian government have also recommended to investigate the possibility of using old shipping containers to handle the increasing housing deficit in the country (Agyeman, 2019).

With old containers being relatively cheap to purchase in addition to the shortened construction time and the already provided weatherproofing, the possibility of a budget friendly house may be appealing to upcoming homeowners. In Ghana used containers can be bought via buy-and-sell websites such as Jiji. Through a quick search on this website the price of a used shipping container lies between 5000 and 20000 Ghanaian cedi (GH¢) depending on the size and condition of the container (Jiji, 2021). In Norwegian currency this corresponds to about 7200 and 29000 NOK.

Shipping containers are used for a variety of building types and can be seen both in simple single-container buildings and complex multi-story designs. An example of a school building in the outskirts of Cape Town in South Africa is the Vissershok primary school designed by Tsai Studio shown in Figure 2-1.

Figure 2-1: Vissershok primary school near Cape Town, South Africa.
Source: (Tsai Design Studio, n.d.)

A combination of shipping containers and local materials were used in the construction of Legson Kayira Community Center and Primary school in Malawi, shown in Figure 2-2, where the containers serve as both transportation device for the other materials to the building site and as structural units in the finished building (Architecture for a Change, 2014).

Figure 2-2: Legson Kayira community center and primary school in Malawi.
Source: (Architecture for a Change, 2014)

Examples of larger scale projects made of shipping containers are the colorful Winebox hotel in Valparaíso, Chile, shown in Figure 2-3 and the four-star Quadrum Ski and Yoga Resort located in Gudauri in Georgia pictured in Figure 2-4.

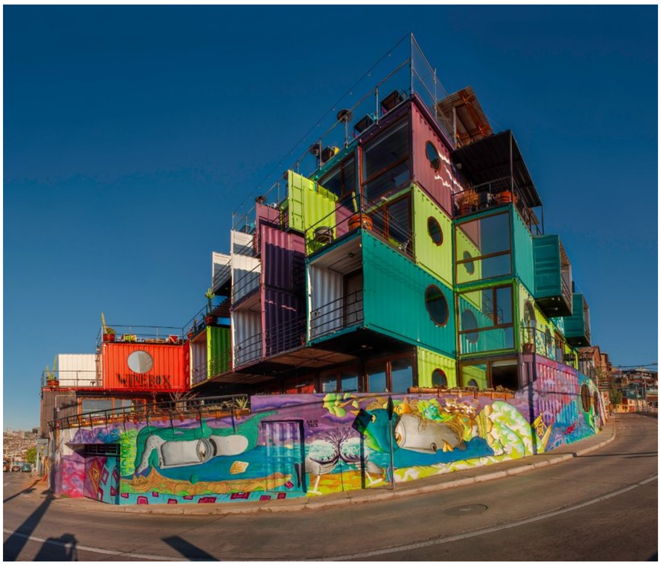

Figure 2-3: The Winebox hotel in Valparaíso, Chile. Source: (Winebox Valparaíso, n.d.)

Figure 2-4: Quadrum ski and yoga resort in Georgia. Source: (Quadrum Ski and Yoga Resort, 2017)

2.1.1 Shipping container properties

Today most containers are designed to meet requirements regarding geometry and load bearing capacity set by the International Organization for Standardization (ISO). A typical shipping container consists of a frame structure with special corner fittings at all corners, doors, trapezoidal steel plates as infill of the frame, and a floor structure consisting of several beams with plywood flooring above (Ling et al., 2020). Figure 2-5 illustrates the different members of a shipping container. Common cross section types for each member described by Ling et al. (2020) and Giriunas et al. (2012) can be found in Appendix B. The location of the doors and the layout of the base structure vary depending on the intended use and size of the container. However, the most common placement of the doors is on one of the short ends of the container.

Shipping containers are made to withstand harsh weather on sea. Most container parts are therefore made of cold formed Corten A steel, which is a weathering steel forming a protective rust layer on its outside to prevent further corrosion of the steel (Rygh, 2019). The weather proofness of shipping containers provides enclosures that are already waterproof and thus shortens the construction time in addition to cost savings. To ensure that all parts of the container are suitable for use on sea, the plywood floors of the container are in most cases treated with pesticides. The type of chemicals used in the treatment of the plywood flooring should be declared on the CSC plate (Convention for Safe Containers), often located at the container doors as shown in Figure 2-7 (Islam et al., 2016).

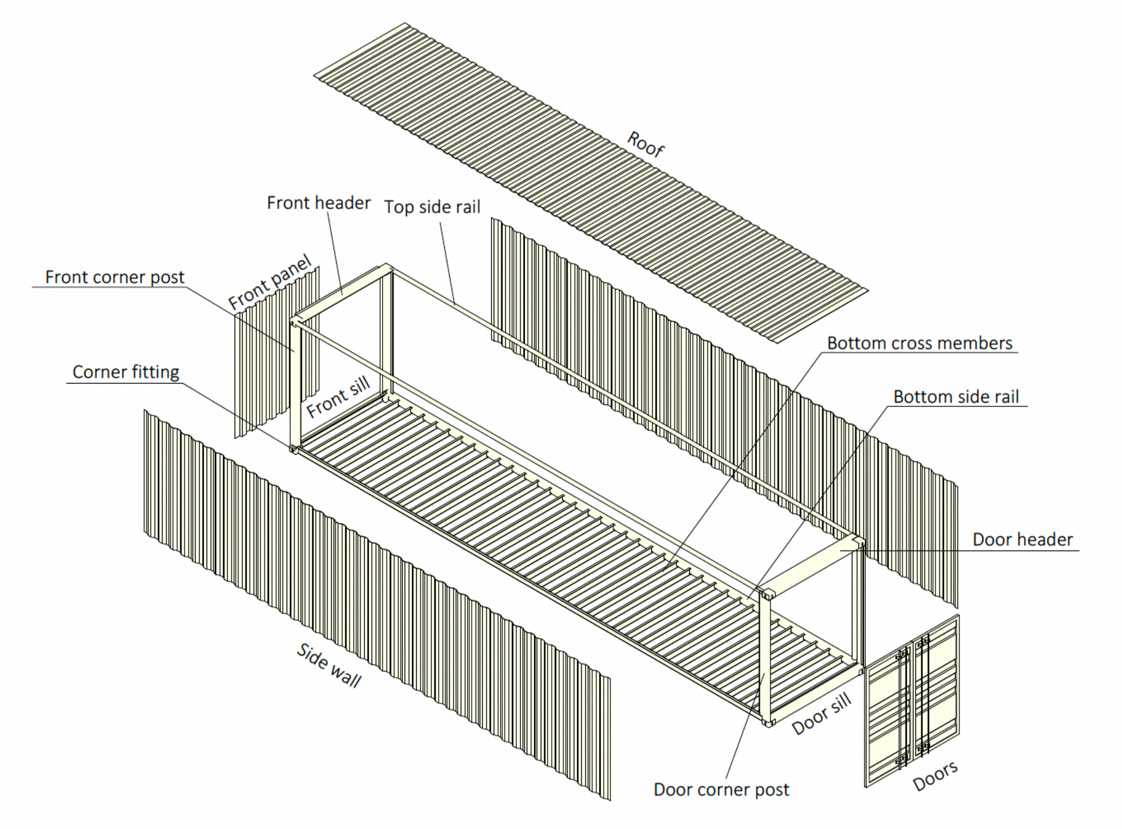

Figure 2-5: Layout of a shipping container

Specifications regarding the geometry and load bearing properties of ISO shipping containers are given in the following ISO standards:

- **ISO 668:2020** Series 1 freight containers – Classification, dimensions and ratings
- **ISO 1496-1:2013** Series 1 freight containers – Specification and testing – Part 1: General cargo containers for general purposes
- **ISO 1161:2016** Series 1 freight containers – Corner and intermediate fittings – Specifications
- **ISO 6346:1995** Freight containers – Coding, identification and marking
- **ISO 830:1999** Freight containers – Vocabulary

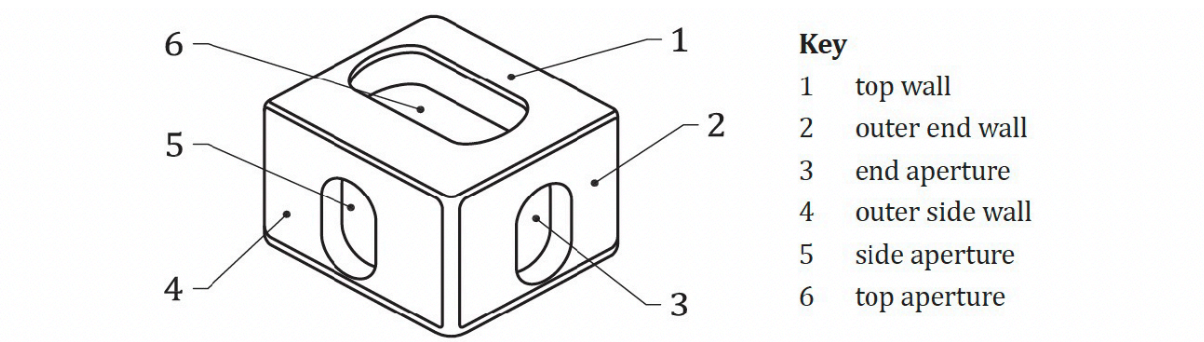

Figure 2-6: Corner fitting.
Source: ISO 1161

Standard widths, lengths and heights are given in ISO 668:2020. For housing purposes 20 feet (20') and 40 feet (40') containers are the most common sizes. The 20' and 40' containers come in different heights, where the containers with the basic height are referred to as standard containers, and the taller ones are called high cube (HC) containers. The HC containers are often preferred for building purposes due to the increased ceiling height. External and minimum internal dimensions for the most common shipping container types used for housing are listed in Table 2-1.

Table 2-1 - External and internal dimensions of typical containers used in housing

Container type	**External dimensions [mm]**			**Minimum internal dimensions [mm]**		
	Length, L_e	Width, W_e	Height, H_e	Length, L_i	Width, W_i	Height, H_i
20 ft standard	6058	2438	2591	5867	2330	2350
40 ft standard	12192	2438	2591	11998	2330	2350
40 ft HC	12192	2438	2896	11998	2330	2655

The gross mass, or rating, of both 20' and 40' containers should not exceed 30480 kg according to ISO 668. The rating is the sum of the tare weight (self-weight of container) and the payload, which is the mass of the stored goods inside the container. The tare weight of a container varies from manufacturer to manufacturer but are always displayed on the front of the container doors as illustrated in Figure 2-7. Nonetheless, Table 2-2 give some examples of mean values for tare weight and payload for 20', 40' and 40'HC containers (Searates, n.d.).

Table 2-2: Rating, tare weight and payload for different container types.

Container type	**Rating [kg]**	**Tare weight [kg]**	**Payload [kg]**
20 ft standard	30 480	2250	28 230
40 ft standard	30 480	3780	26 700
40 ft HC	30 480	4020	26 460

Although the gross mass of one container should not exceed 30480 kg, the load bearing capacity of a shipping container is significantly higher. Containers are designed to carry and transfer loads through their corner fittings and ISO 1161 is devoted to the design and testing of corner fittings. For the container as a whole, capacities according to test procedures given in ISO 1496-1 should be fulfilled. Among the test procedures in ISO 1496-1 a vertical stacking test is included. To ensure sufficient vertical load bearing capacity, a container should be able to carry point loads equal to 942 kN in each of its top corner fittings. This corresponds to 12 fully loaded containers stacked on top of each other, proving that shipping containers are structures with very good load bearing properties as long as load transfer occurs through the corner fittings.

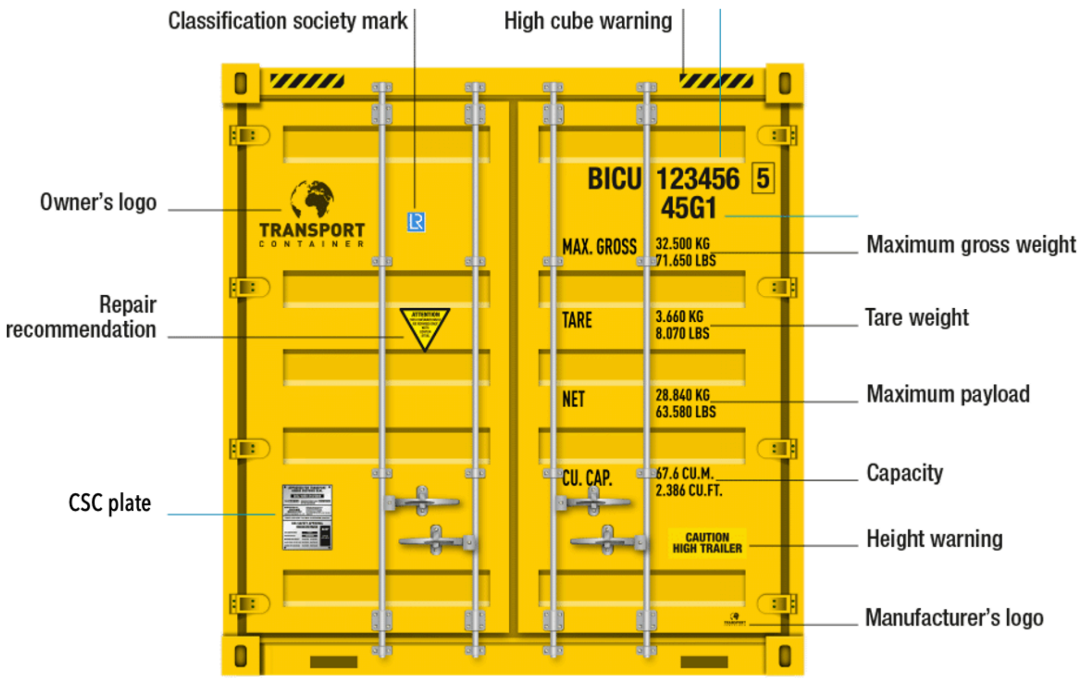

Figure 2-7: Typical markings on shipping container doors according to ISO 6346. Source: (Cargo Master, n.d.)

2.1.2 Thermal comfort in shipping container buildings

When converting a shipping container into a building, the climatic conditions on the location of the building must be taken into consideration. Several studies have been conducted on the thermal performance of shipping containers in different climates. In terms of indoor climate, numerical simulations of shipping container buildings in hot and humid climates, integrating passive cooling strategies, have proven to perform similar to conventional buildings in the same area (Vijayalaxmi, 2010, Elrayies, 2017). In low-income countries mechanical ventilation is often a luxury not available to the majority of the population due to limited financial resources. However, by ensuring natural ventilation and integrating passive measures in the building design, temperatures similar to the ambient temperature may be achieved inside the containers (Vijayalaxmi, 2010). Some measures to improve the indoor climate of shipping container buildings in low-income countries with hot climates are (Elrayies, 2017):

- External shading
- Thermal insulation
- Using windows with a low U-value
- Painting the containers in a light color

Both the use of external shading and thermal insulation reduce the solar radiation on the container. External shade can be provided in terms of separate built structures or by existing trees (Vijayalaxmi, 2010). Structures serving as shading are built with an air gap between the container and shading to remove heat transferred from the shading device to the container as shown in Figure 2-8. Heat is removed by natural convection letting air flow through the gap between these two layers (Biwole et al., 2008). Due to the limited space inside a container, thermal insulation is often applied to the exterior of the containers, not to take up too much of the interior space. A disadvantage of external insulation is that waterproofing of the thermal insulation must be provided, thus, the waterproof containers are not put to their best use (Elrayies, 2017).

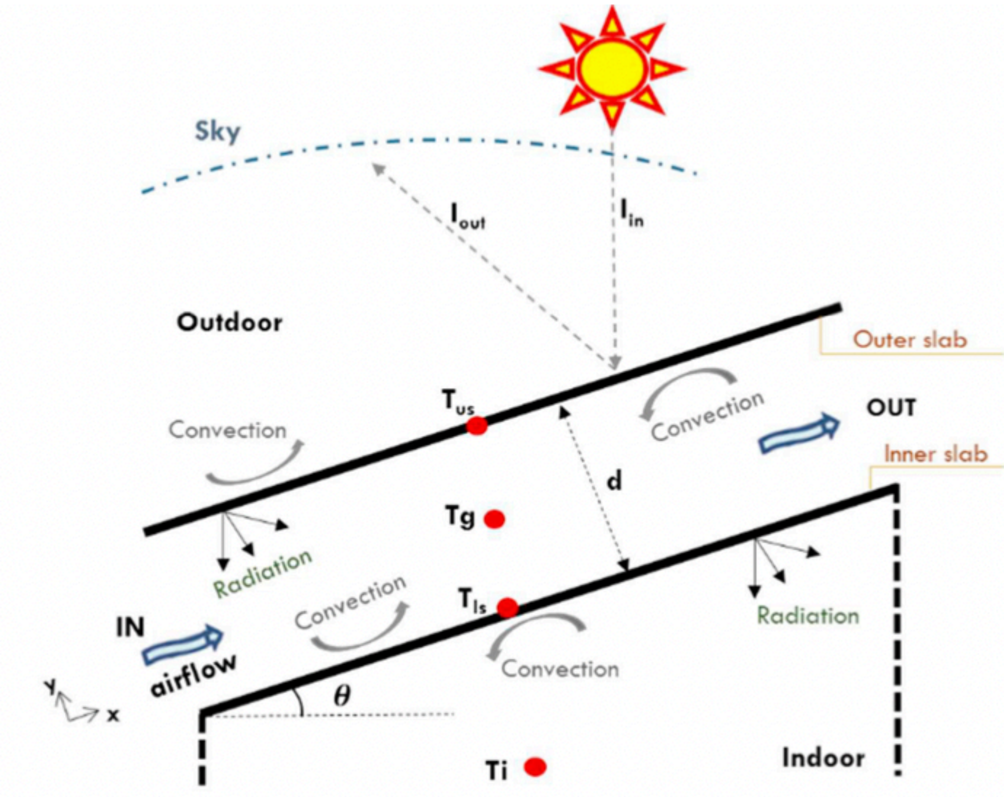

Figure 2-8: Principal of heat reduction due to external shading.
Source: (Biwole et al., 2008)

Sufficient air supply to the indoor climate can be achieved by natural ventilation utilizing wind and buoyancy driven forces to ensure supply of fresh outdoor air. Natural cross ventilation through doors and windows may be the easiest way to provide an air flow through the containers (Compton, 2002). Further measures may include ventilation shafts such as a solar chimney. Solar chimneys remove heated air through a shaft on the outside of the building warmed by solar radiation. Heated air with lower density than the surrounding environment are removed from the chimney by buoyancy driven forces (Khanal and Lei, 2011).

2.1.3 Foundations for shipping container buildings

The most common foundation types for shipping container buildings are shallow foundations such as either spread footings or mat foundations, as shown in Figure 2-9 and 2-10 (Giriunas et al., 2012). Shallow foundations transfer the loads from the structure above directly to the ground through contact pressure between the underside of the foundation and the ground. Spread footings often consists of a short column transferring the loads from the structure above to a base footing with a larger area than the column itself. Mat foundations are slabs placed directly on the ground surface and covers the entire footprint of the building (Larsen, 2008).

Figure 2-9: Shipping container on spread footing foundation Source:(Premier Box, 2017)

Figure 2-10: Shipping container on mat foundation. Source: (Sanchez, 2021)

Spread footings are material efficient and often comes at a reasonable cost. In addition to the economic benefits of spread footings, this foundation type requires minor alterations of the surrounding terrain and environment. When using spread footings, a clear height of minimum 0,5-meter between the ground and the underside of the building is recommended to avoid moisture related problems (Edvardsen and Ramstad, 2014). If the area of spread footings exceeds 50% of the building's footprint, mat foundations are typically used instead. Other reasons for choosing mat foundations are if the ground conditions are very poor or the uplifting forces acting on the foundation become very large (Giriunas et al., 2012).

2.2 Local building materials and traditions in Ghana

Local materials for building purposes in Ghana include sand, stone, grass, thatch, clay, timber, clay bricks and clay blocks (Danso, 2013). Buildings made of local materials with local building techniques such as the wattle and daub, rammed earth construction, timber framing and adobe construction have been in use for many years (Agyekum et al., 2020). However, the tendencies regarding building materials in Ghana today is a reliance on imported materials such as steel and Portland cement, thus, the local materials readily available are often overlooked (Danso, 2013).

Figure 2-11: Adobe building. Source: (Napolitano, 2017)

Figure 2-12: Rammed earth construction. Source: (Hive Earth, 2018)

Figure 2-13: Wattle and daub wall. Source: (MrPanyGoff, 2012)

Earth-based structures such as buildings made using the wattle and daub technique, rammed earth structures and adobe buildings using mud bricks, shown in Figures 2-11, 2-12 and 2-13, provide economical and sustainable housing. Furthermore, earthen structures represent some of the architectural heritage in Ghana, where the Traditional Asante Buildings may be the most famous, listed on UNESCO's World Heritage List. These buildings are made of earth, wood, bamboo and straw and have intricate decorations carved into the mud walls (UNESCO, n.d.). Figure 2-14 show a Traditional Asante Building from the Kumasi region. Earthen structures, however, face a challenge in terms of life expectancy due to the frequent maintenance needed as a result of the mud and clay being washed away during the rainy season (Danso, 2013).

Cement based building blocks, such as sandcrete blocks, are one of the most common materials used for walls in Ghana today. Sandcrete, which is made by mixing water, cement and sand, have good compressive strength and high thermal resistance making them ideal in the hot climate of Ghana. In comparison to mud- and earth-based structures, sandcrete does not degenerate due to the heavy rainfalls in summer. However, a lack of quality control in sandcrete production amongst many manufacturers in Ghana often result in sandcrete blocks with poorer compressive strength than stated, thus making them unsuitable for loadbearing purposes (Baiden and Tuuli, 2004).

Figure 2-14: Asante Traditional Buildings.
Source: (Bosman and Bosman, 2006)

Timber and bamboo are locally grown materials in Ghana. Over 400 different species of wood and about five different species of bamboo can be found in the tropical forests in the middle and southern parts of the country. Despite the vast number of wooden species available in Ghana, timber and bamboo are mostly used for framing of windows, doors and roof supporting structures. Deterioration of these cellulose based species due to fungi and insect attacks, such as termites, are some of the main restraints for using timber and bamboo for structural purposes (Solomon-Ayeh, 2010, Baiden et al., 2005). A lack of treatment to prevent termite and fungi attacks are one of the main reasons for the shortened lifetime of these components. Hence, most of the timber industry in Ghana is based on export of species such as the African Walnut, Mahogany and Teak. However, at the beginning of this millennium, Ghana faced an overexploitation of its forests resulting in deforestation of the most commonly exported timber species. As a result, research regarding the use of lesser-known species for structural purposes have been conducted with promising discoveries showing that there are several lesser-known species providing sufficient strength for load-bearing purposes (Baiden et al., 2005).

For roofing purposes, thatch or corrugated metal sheeting are among the most common roofing materials. Thatched roofs made from bundles of grass are widely used in the northern regions of Ghana. This type of roofing provides good insulation from the sun (Agyekum et al., 2020). The lifespan of thatched roofs, however, are not as long as roofs made of corrugated metal sheeting. The corrugated metal sheeting performs poorer than the thatched roof in terms of insulation value. However, for rainwater collection purposes the corrugated metal roofs have proven to be better than thatched roofs in terms of water quality and the amount of water collected (Efe, 2006).

3 Method

3.1 Building regulations and rules

Ghana received its first building regulations in 2018 called the Ghana Building Code (GBC) . This code is derived from old British Standards, perhaps not surprisingly, as the Republic of Ghana was a British colony until 1957 (United Nations, 2019). Locating a complete copy of the GBC has not been successful. Nevertheless, a copy of Part 5 in the code dealing with actions on structures such as dead loads, live loads and basic wind velocities have been acquired (Ghana Standards Authority, 2018).

In the structural design of members, values for dead loads, live loads and basic wind velocities according to local values given in GBC are used when possible. As attempts to procure a complete copy of the GBC have failed, design procedures according to Eurocodes have been applied in the design of members. Recommended parameter values have been set according to the general recommendations in the Eurocodes and not according to the National Annex, assuming that the nationally determined parameters for Norway are not applicable to Ghanaian conditions. The following Eurocodes will be used in the coming chapters:

- **EN 1990:2002** Eurocode: Basis of structural design
- **EN 1991-1-1:2002** Eurocode 1: Actions on structures. Part 1-1: General actions. Densities, self-weight, imposed loads for buildings
- **EN 1991-1-4:2005** Eurocode 1: Actions on structures. Part 1-4: General actions. Wind actions
- **EN 1995-1-1:2004** Design of timber structures. Part 1-1: General Common rules and rules for buildings

For design of the community center not involving structural design, requirements according to the Norwegian building regulations, TEK17, have been considered (Direktoratet for Byggkvalitet, 2017). However, it should be noted that in this preliminary phase of the project, the most relevant requirements regarding accessibility and safety have been the main focus. Exaggerated slightly, quoting some of the Trax representatives, a room in Ghana is not considered full as long as more people can fit into the space. Considerations regarding the amount of people who can stay in a room at the same time have therefore not been evaluated or checked against any building regulations.

3.2 Structural design

Finite element models for structural design are built in Autodesk Robot Structural Analysis Professional 2019 (Robot), which is available via NTNU software center. The layout of the project, described in chapter 5, includes structural components in addition to the shipping containers. Modelling of these structural components in Robot are described in chapter 8 and 9. Design forces are obtained from the Robot models and used in the design of the associated members.

The design forces obtained from the numerical models are used in manual design checks of the members. Design checks according to the relevant Eurocodes are done using spread sheets in Microsoft Excel. These spread sheets are attached in Appendix E and G. Like the calculation models for each component, the relevant design checks for the additional members included in the layout is given in chapter 8 and 9.

4 Local conditions

The local conditions on the farm influence the design of the project. By ensuring that the climatic conditions are taken into account in terms of material and design choices, durable structures which serve their intended purpose may be accomplished. Weather data for Bolgatanga, which is the town closest to the farm, is used in the design.

The climatic conditions of the Upper East Region in Ghana are classified as savannah climate by Köppen's climate classifications, corresponding to a tropical climate changing from wet to dry over the year (Mamen, 2021). The minimum and maximum temperature, precipitation and relative humidity (RH) for Bolgatanga are given in Table 4-1. From Table 4-1 it can be observed that temperatures in Bolgatanga vary from 20°C in summer to 40°C in winter. Heavy rainfalls are expected in the months of July, August and September, while May, June and October marks the transition from the dry winter to the wet summer and vice versa. Shifts in the relative humidity follow the changes in precipitation through the year (climate-data.org, 2021). According to the Global Facility for Disaster Reduction and Recovery (2015) the northern part of Ghana is prone to drought and flooding due to the drastic changes in rainfall during the year. Furthermore, Trax representatives have reported of frequently occurring thunderstorms during the months with significant precipitation.

Table 4-1: Temperatures, precipitation and relative humidity in Bolgatanga. Source: (climate-data.org, 2021)

	Jan	**Feb**	**Mar**	**Apr**	**May**	**June**	**July**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Min. temp. [°C]	20,5	22,7	25,3	26,9	26,2	24,6	23,2	22,5	22,6	23,2	22,1	20,6
Max. temp. [°C]	34,9	37,3	39,2	38,6	36,5	33,5	30,4	28,9	29,7	32,4	35,3	34,8
Precipitation [mm]	0	2	10	35	57	68	137	218	143	47	4	0
RH [%]	17	15	23	43	57	66	77	84	84	72	40	22

Ghana is located on the Northern Hemisphere just north of the Equator. Being so close to the Equator the sun is set high in the sky during the whole year as can be seen from the sun path diagram for Bolgatanga in Figure 4-1. A sun path diagram describes the path of the sun across the sky during the year, in terms of the sun's vertical position (interior circles) and the horizontal position relative to the north (outer circle).

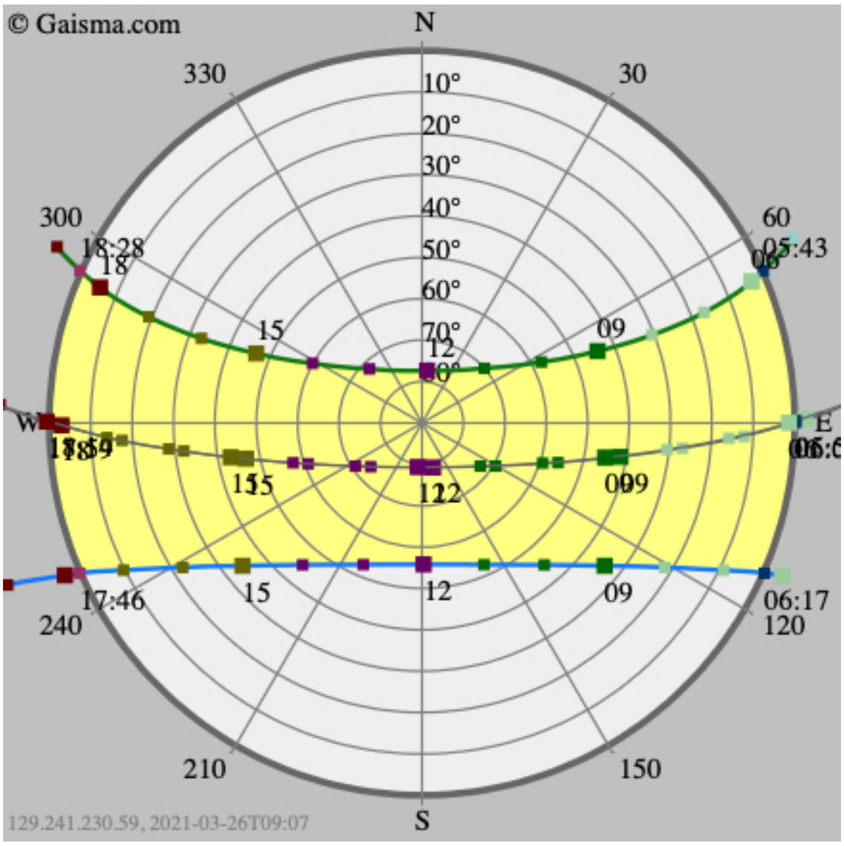

Figure 4-1: Sun path diagram for Bolgatanga.
Source: (Gaisma, 2021)

The average wind velocity in the area near Bolgatanga lies between 4-5 m/s. The prevailing wind direction in Bolgatanga is from the south-west during most of the year, except during the months of December, January and February where wind from the north-east direction is quite common (Weather Spark, 2021).

Seismically active areas can be found in the southern part of Ghana. In the coastal belt west of the capital, Accra, earthquakes have been recorded. In the northern part of Ghana, however, historic earthquake catalogues from 1615-2003 do not report of any earthquake incidents in the area near Bolgatanga. Design for earthquakes is therefore not common in the northern part of the country (Amponsah et al., 2012).

In terms of ground properties, the ground on the farm is assumed to exist of 40% sand, 40% silt and 20% clay based on previous experiences from neighboring buildings in the area.

5 Layout of project

In this preliminary phase of the project, most of the time have been spent on defining the needs that must be satisfied and finding a way to achieve these goals by use of local labor and materials. As the main goal of this phase in the project is to produce a foundation for funding applications there have been a focus on not "over-doing" it but ensuring that the most important features are included in the project and instead allowing for future expansions if possible.

With this in mind a suggested layout of four 40'HC containers, painted in light colors, arranged to form a classroom, a library with server room, accommodation facilities and an office for Trax staff have been established. The accommodation facilities are intended to house six bunk beds for kids, one bunk bed for teachers and two bunk beds for Trax staff. The server room will house a server/laptop and ten tablets donated by Education in a Suitcase (EIAS) for educational purposes. EIAS is a non-profit organization providing electronic equipment to low-income countries (Education in a Suitcase, n.d.). Through these tablets, the students will have access to the tutor-web educational system developed at the University of Iceland in addition to Wikipedia and Khan Academy. This system is developed to work without a constant Wi-Fi connection but syncs up to the server whenever a Wi-Fi connection is established, making it ideal for rural areas like the Trax-Kavli farm (Tutor Web, n.d.).

In addition to the learning and accommodation amenities two 20' containers hold pit latrine toilets and shower facilities A site plan showing the intended layout and suggested placement on the farm can be seen in Figure 5-1 and 5-2 respectively. The terrain slopes slightly in the east-west direction and the toilets are therefore placed as far away from the bore hole as possible. Due to cultural traditions and the potential risk of sexual abuse, particularly for girls, toilet and shower facilities for girls and boys have been placed in separate containers. Furthermore, in addition to individual containers for girls' and boys' facilities, the containers are separated in distance as shown in Figure 5-2. The container holding the girls' facilities is kept in eyesight from the rest of the buildings to ensure that the girls feel safe when using the facilities. Only one dormitory for the kids is incorporated into the suggested layout. Hence, it has been decided, for now, that either girls or boys can stay over at the farm at the same time. However, during daytime the facilities will be open to both genders.

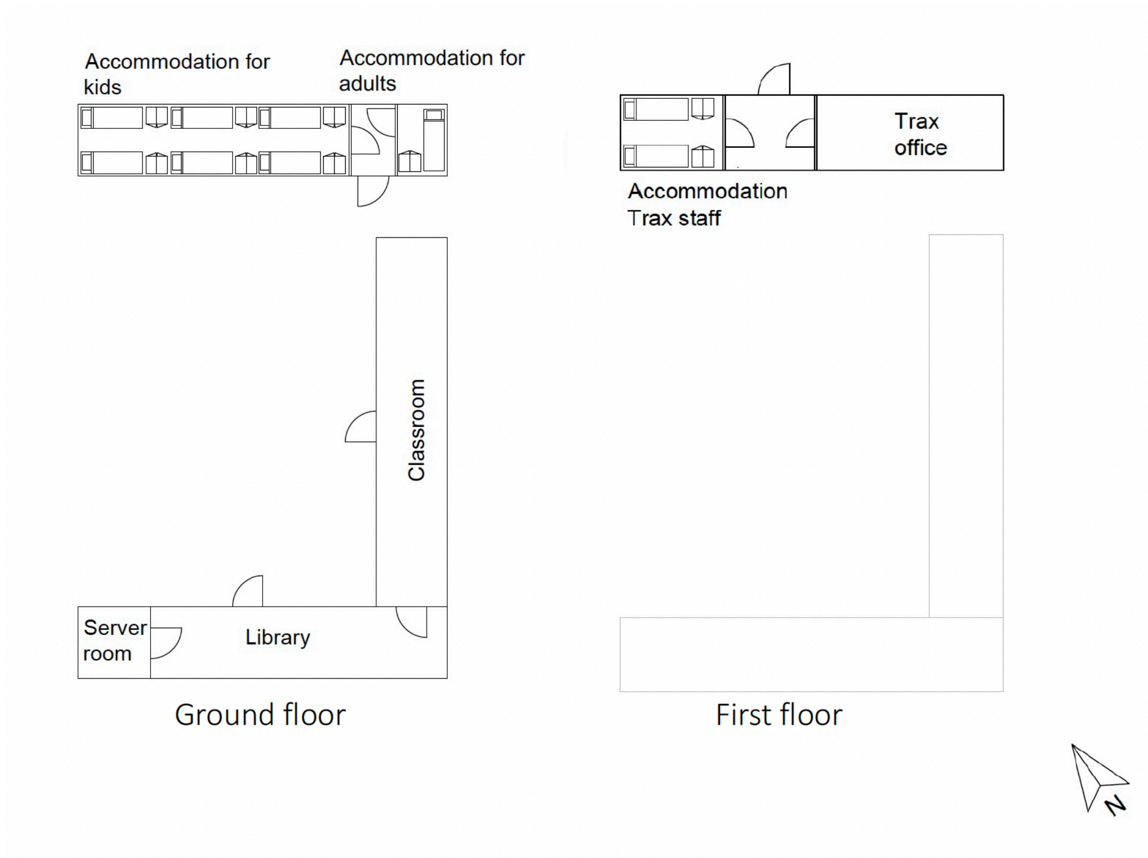

Figure 5-1: Suggested layout

The suggested layout has been developed through conversations with representatives from Trax. Several layouts and placements of the containers in relation to each other were proposed. The final proposed layout for funding applications is shown in Figure 5-1. In this plan the classroom and library are made of two containers forming a L-shaped building. A wish from Trax to keep these two features nearby each other are achieved while keeping the noise transfer between these two units as low as possible. Separated from the classroom and library, the remaining two containers housing accommodation facilities and Trax office are stacked on top of each other, forming a U-shape with the classroom and library building. Thus, utilizing the load bearing capacity of the containers by ensuring load transfer through the corner fittings of the stacked containers.

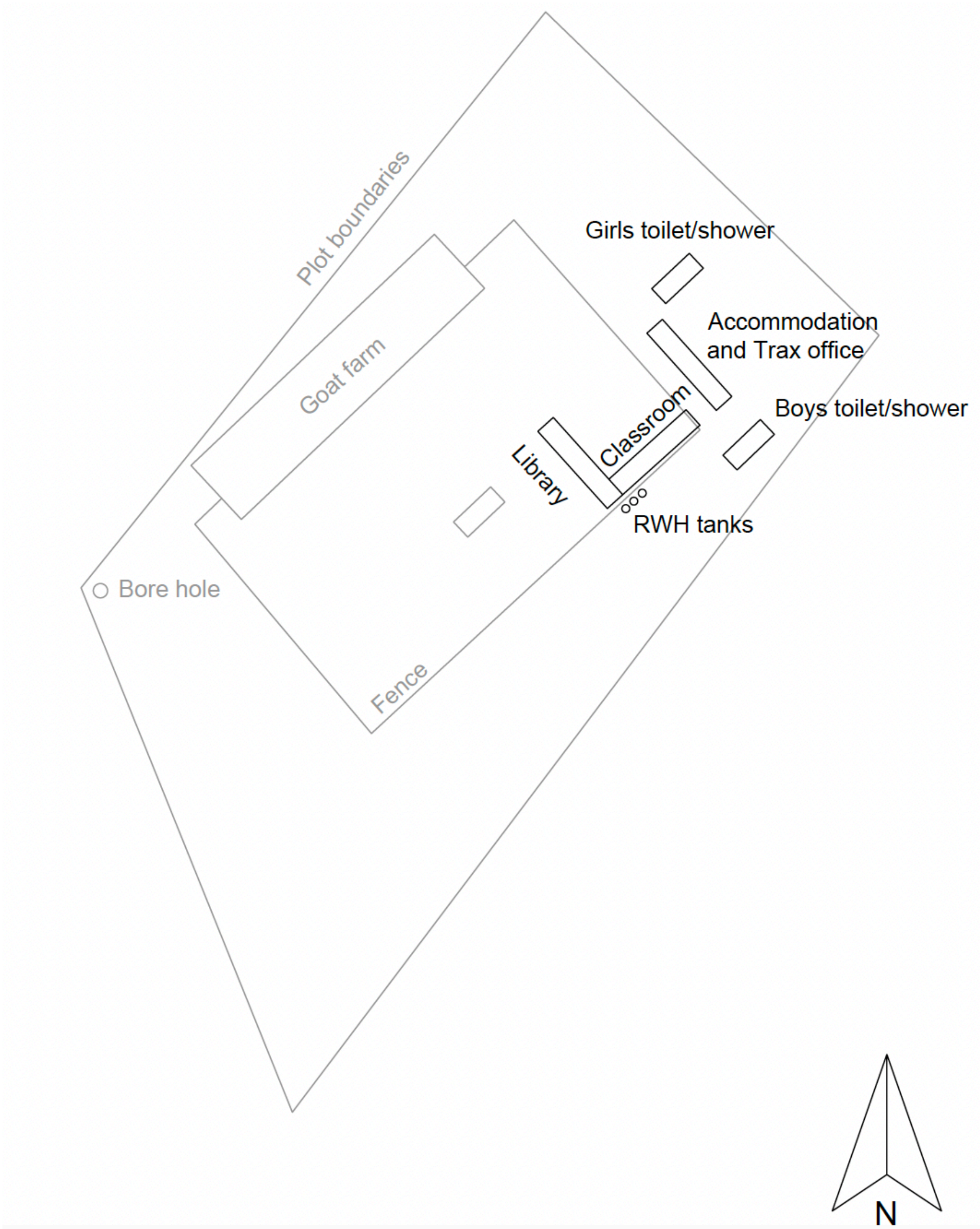

Figure 5-2: Site plan with suggested placement

Placement of doors and windows are not defined in this phase, but the required amount of daylight according to TEK17 have been calculated to give an idea of the necessary window surfaces. The required amount of daylight for rooms intended for permanent stay are given by §13-7(2b) in TEK17:

$$A_g \geq 0{,}07 \cdot \frac{A_{BRA}}{LT}$$

Where:

A_g	Area of glass facing the outside at a min. height 0,8 m from the floor
A_{BRA}	Usable area
LT	Light transmission of the glass

The area of glass required is calculated for a usable area equal to 28 m^2 which corresponds to the internal dimensions of a 40 ft. shipping container. The light transmission is set to 65% which is recommended by Rådgivende Ingeniørers Forening in the early phase of projects (Ulimoen et al., 2020). Hence, for rooms intended for permanent stay, the necessary glass area in a height of minimum 0,8 meters above the floor is 3 m^2. Regardless of whether or not the rooms are intended for permanent stay, A_g is assumed valid for all containers.

The shipping containers are assumed to be supported by spread footing foundations due to their cost effectiveness and limited impact on the surrounding environment compared to other foundation types. To determine if this is an appropriate foundation type for this project, calculations have to be performed. This procedure is described in chapter 9. A 0,5-meter gap between the ground and the underside of the containers must be provided to avoid moisture related problems as described in chapter 2.1.3. However, if spread footings prove not to be an optimal foundation type for this project, mat foundations may be a good alternative due to the fairly flat terrain on the farm.

Should the area inside the classroom and library be too small to teach larger groups, or too hot during the hottest periods, the outdoor area between the containers is envisioned to serve as an extension of the indoor environment. An outdoor kitchen for food preparation and cooking is also intended to be placed in this area. To make the outdoor area user friendly during both the rainy season in summer and the hot and dry season in winter, a roof structure is added. In addition to the roof above the outdoor area, roofs are added on top of each container to enhance the thermal comfort inside them. Hence, one roof covering the outdoor area, classroom and library and one roof on top of the accommodation and office unit are provided as shown in Figure 5-3. Furthermore, roofs over both toilet containers are added. Design and calculations of the roof supporting structures for each roof are discussed in chapter 8.

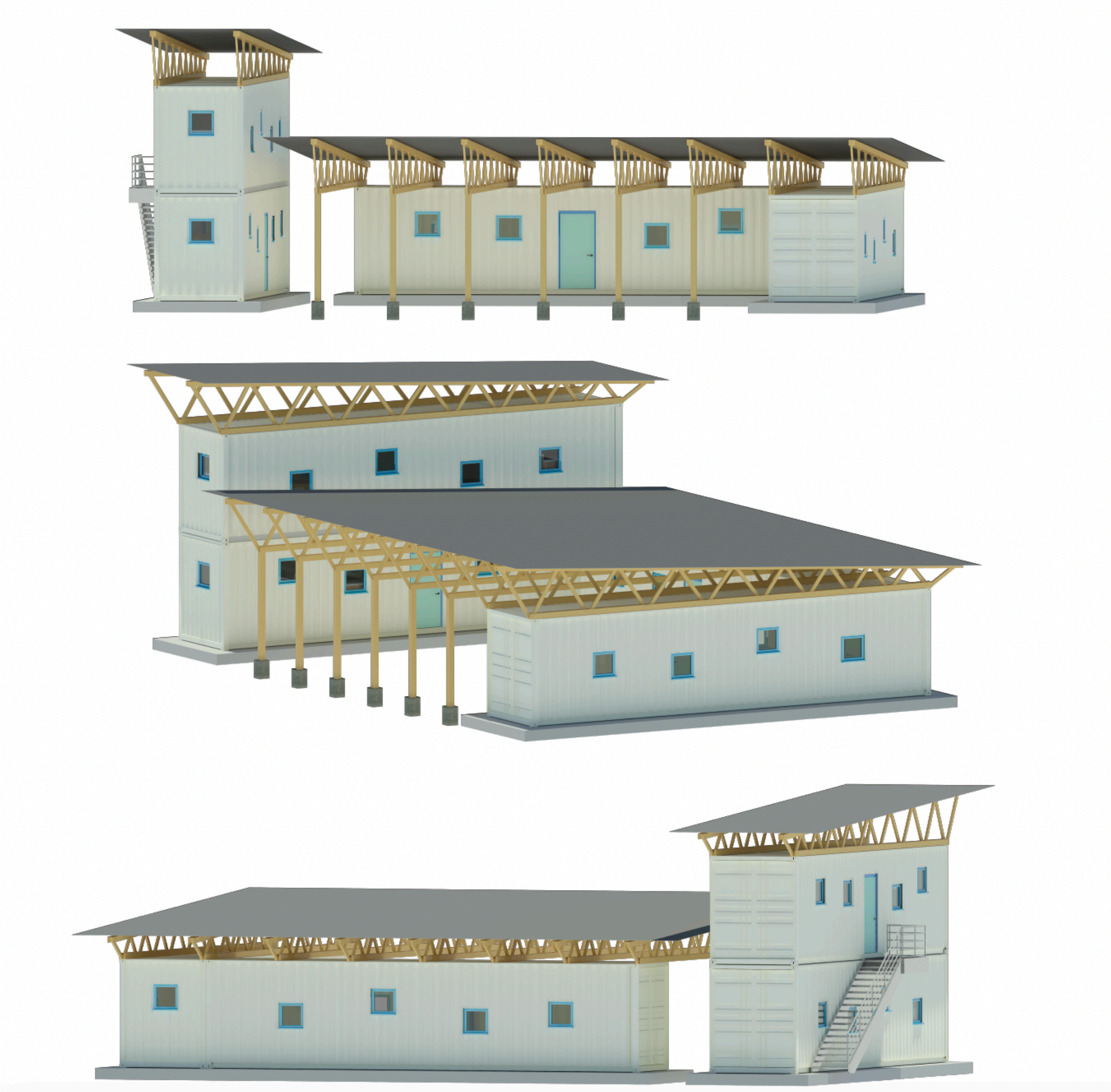

Figure 5-3: Layout with external roofs

To ensure that sufficient shade is provided to the covered outdoor area and all external surfaces of the container, a solar analysis have been conducted in Autodesk Revit 2021. A solar analysis is done by utilizing sun path diagrams as the one shown in Figure 4-1. By doing this it is possible to study how the sun hits the different surfaces of the building during the year. If the external roofs are extended by 1 meter in every direction, shade is provided to almost all surfaces during the mid-day hours where the sun is at its most intense. Numerical simulations regarding the indoor temperature carried out in IDA ICE verify that the indoor temperatures correlate with the outdoor temperatures when the external roofs are applied. Further elaboration regarding the thermal performance of the containers will not be discussed in this thesis but interested readers can find the result from the numerical simulation in Appendix C. Temperatures below ambient are hard to achieve without any form of mechanical cooling, thus, indoor temperatures will reach 40°C during the hottest months.

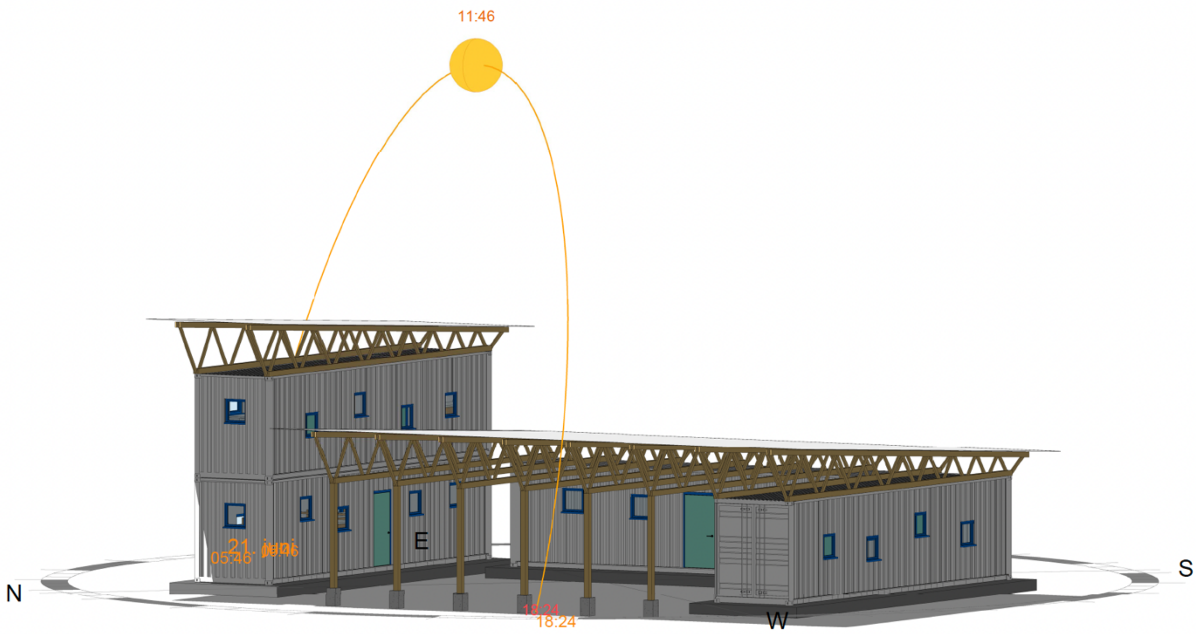

Figure 5-4: Solar analysis in Revit

The added roofs serve a double purpose by providing surfaces suitable for rainwater harvesting (RWH). In areas like the Upper East region of Ghana, where the weather conditions are changing from heavy rainfalls in summer to drought in winter, there is great potential in utilizing RWH. By leading rainwater from the roof in gutters to large barrels, the collected water can be stored and, for instance, be used to water plants in the greenhouse. Due to the open barrels the water is easily contaminated and therefore not recommended for drinking and eating. However, this is a nice way to relieve the bore hole on the farm in some weeks or even months depending on the barrel size and precipitation (Andoh et al., 2018). As mentioned in chapter 2.2, corrugated metal roofs are better in terms of RWH than thatched roofs. Corrugated metal sheeting treated with rust protection is therefore considered the most suitable option in this project. The size and number of barrels for RWH on the farm will depend on the budget. Figure 5-5 shows a typical water barrel for rainwater harvesting from a roof surface in Kenya.

Figure 5-5: Tank for RWH in Kenya.
Source: (African Post Online, 2020)

An evaluation from a health-wise point of view must be done regarding the plywood flooring inside the containers. As mentioned in chapter 2.1 most container floors are treated with pesticides. The toxicity of these pesticides depends on the chemicals that are used. However, a plan to either replace the existing flooring or some sort of encapsuling of the floors should be included in the budget to make sure that these chemicals do not impose a threat to the health of the occupants.

As mentioned in the introduction, this project aims to include all parts of the rural community, therefore considerations have been made with respect to disabled persons. Complete Universal Design as practiced in Norway may not be achieved due to the amount of people and limited space within the containers. However, the classroom, library and toilets can be made accessible for everyone by adding ramps and wide enough doors for a wheelchair to pass. It is assumed that disabled persons will not stay at the farm all by themselves. Thus, temporary ramps which can be moved around and stored away when not in use are both a space saving and economically favorable solution. For external doors §12-13(2a) in TEK17 specifies a free space of 860 mm for a wheelchair to pass through. Therefore, the doors must have a total width including the door frame of approximately 1 meter.

Concerns regarding fire safety design are taken into account by following guidelines given in TEK17. §11-2 and §11-3 in TEK17 categorize buildings into risk classes and fire classes correspondingly. The risk class defines the threat to human safety in case of fire. Risk class 1 implies a low risk for loss of human lives, while risk class 6 denotes a significant danger to human lives in the event of fire. The fire class depends on the number of floors in the building. Consequently, by following the procedure given in §11-2 and §11-3, the classroom and library unit is placed in risk class 3 and fire class 1. The accommodation and office building belongs to risk class 4 and fire class 1, while the toilets are placed in risk class 1 and fire class 1.

At least two escape routes should be provided for any space within a building. These two escape routes should be independent of each other to enhance the possibility of a successful escape during a fire. Windows with a net area of minimum 0,5 m x 0,6 m in width and height correspondingly can be approved as escape routes if the distance between the ground and lower windowsill is less than 5 meters. For disabled persons, escape through windows is not sustainable, thus, doors must be provided to ensure a safe escape of these persons as well (Edvardsen and Ramstad, 2014). As shown in Figure 5-1 both the classroom and the library have external doors providing two separate escape paths for disabled persons as required. According to §11-13(7) in TEK17, doors in escape routes of buildings must have at least a free width of 0,86 meters, which is already required when facilitating for people in a wheelchair.

Thunderstorms are a common sight in the region as described in chapter 4, hence, the need for protection from lightning strikes in terms of lightning rods is necessary. The lightning rod guides the current to the ground, preventing the lightning from damaging the building and electrical appliances (Lotha and Promeet, 2010).

6 Loads

Different values for dead loads, live loads and wind loads are applied to both the calculation models for the roof supporting structures and the calculation models for foundations. The dead loads and live loads applied to the different models are presented in chapter 8.3 for the roof supporting structures and in chapter 9.3 for the foundations. Complete wind load calculations can be found in Appendix D. The procedure used to calculate the wind loads are given in the following sub-chapter, 6.1.

6.1 Wind loads

Calculation of wind loads for containers and external roofs are done according to procedures given in EN 1991-1-4.

6.1.1 Peak velocity pressure

All wind loads calculated according to EN 1991-1-4 are based on the peak velocity pressure, q_p, given at a distance z above the ground. Factors influencing the wind behavior such as terrain characteristics, the effects of turbulence, wind direction and return periods are included in the peak velocity pressure. The basis of the calculation is the fundamental value of the basic wind velocity, $v_{b,0}$, which is specific for a given place and measured at a height 10 meters above ground during a 10-minute period (Larsen, 2008). For this project $v_{b,0}$ is taken from GBC. GBC give fundamental values for the basic wind velocity for 15 major towns in Ghana. Amongst these, Navrongo is closest to Bolgatanga, and $v_{b,0}$ is therefore set to 35 m/s which is valid for Navrongo.

The basic wind velocity, v_b, is found by altering $v_{b,0}$ to be applicable for all wind directions, seasons, altitudes above sea level and return periods. v_b is given by expression 4.1 in EN 1991-1-4:

$$v_b = c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b,0}$$

Where: c_{dir} Directional factor.
Set equal to 1,0 by recommendations in EN 1991-14

c_{season} Seasonal factor.
EN 1991-1-4 recommends a value of 1,0.

c_{alt} Altitude factor

c_{prob} Probability factor.
Equal to 1,0 if return period is set to 50 years.

In GBC the provided $v_{b,0}$-values are already corrected to be valid for a return period of 50 years, thus eliminating c_{prob} from the equation above. Values for the influence of altitude provided by the factor c_{alt} are determined in the National Annex of EN 1991-1-4. However, GBC deals with similar factors as EN 1991-1-4 when determining the basic wind speed although they go by different names and symbols. The altitude factor, or topography factor, as it is called in GBC is set to 1,0 and this value have been used for c_{alt} in the following calculations. The basic wind velocity, v_b, is therefore equal to 35 m/s.

The peak velocity pressure is calculated by equation 4.8 in EN 1991-1-4 and is equal to:

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z)$$

Where: $I_v(z)$ Turbulence intensity at a height z above the ground.

ρ Air density.
A value of 1,25 kg/m^3 is recommended by EN 1991-1-4.

$v_m(z)$ Mean wind velocity at height z above the ground.

The mean wind velocity, v_m, is calculated by:

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b$$

Where: $c_r(z)$ Roughness factor

$c_o(z)$ Orography factor
Set to 1,0 by recommendation in EN 1991-1-4.

Figure 6-1: Surrounding terrain on the Trax-Kavli farm.
Copyright: Engineers Without Borders

The roughness factor, c_r, takes into account the changes in the mean wind velocity due to the characteristics (roughness) of the terrain and the height, z, considered above ground level. Based on conversations with representatives from Trax Ghana, Google maps and pictures from the farm, the terrain category is set to II which applies for sites with low vegetation and isolated buildings and corresponds well to what is seen in Figure 6-1. For a z-value between the minimum height, z_{min}, and the maximum height, z_{max}, the expression for $c_r(z)$ is given by equation 4.4 in EN 1991-1-4 by the logarithmic function:

$$c_r(z) = k_r \cdot ln\left(\frac{z}{z_0}\right)$$

Where: k_r Terrain factor

z_0 Roughness length.
Set to 0,05 m by recommendations in Table 4.1 in EN 1991-1-4.

k_r is given by equation 4.5 in EN 1991-1-4:

$$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}$$

Where: $z_{0,II}$ Roughness length for terrain category II
Obtained from Table 4.1 in EN 1991-1-4.

Values for z_{min} and z_{max} are determined in accordance with EN 1991-1-4 and is set to 2 m and 200 m respectively.

The wind turbulence intensity takes the effect of wind gusts into consideration.
$I_v(z)$ for $z_{min} \leq z \leq z_{max}$ is determined by:

$$I_v(z) = \frac{\sigma_v}{v_m(z)}$$

Where σ_v is the standard deviation of the turbulence defined as:

$$\sigma_v = k_r \cdot v_b \cdot k_l$$

Where: k_l Turbulence factor
Taken as 1,0 according to EN-1991-1-4.

6.1.2 Pressure coefficients for containers

Suction and pressure forces on buildings are calculated by multiplying the peak pressure velocity with external and internal pressure coefficients. The total wind load acting on a surface of the building at a reference height, z, is the difference between the external and internal wind loads on this surface:

$$w = w_e - w_i = q_p(z_e) \cdot c_{pe} - q_p(z_i) \cdot c_{pe}$$

Where: c_{pe} External pressure coefficient

c_{pi} Internal pressure coefficient

External pressure coefficients, c_{pe}, for vertical walls and flat roofs with sharp edges are given in Tables 7.1 and 7.2 in EN 1991-1-4 and depend on the height-to-depth ratio (h/d) of the building. For the external pressure coefficients $c_{pe,10}$-values have been used which is recommended for the overall load-bearing design of buildings. EN 1991-1-4 divides the roof and walls of buildings into zones. These zones receive different intensities of pressure or suction forces, where the corners and edges near the windward side are particularly exposed. Figure 6-2 shows the different zones in a flat roof and vertical walls depending on the wind direction. Depending on the height-to-width ratio (h/b) of the building, the pressure distribution on the windward side of the building may vary over the height.

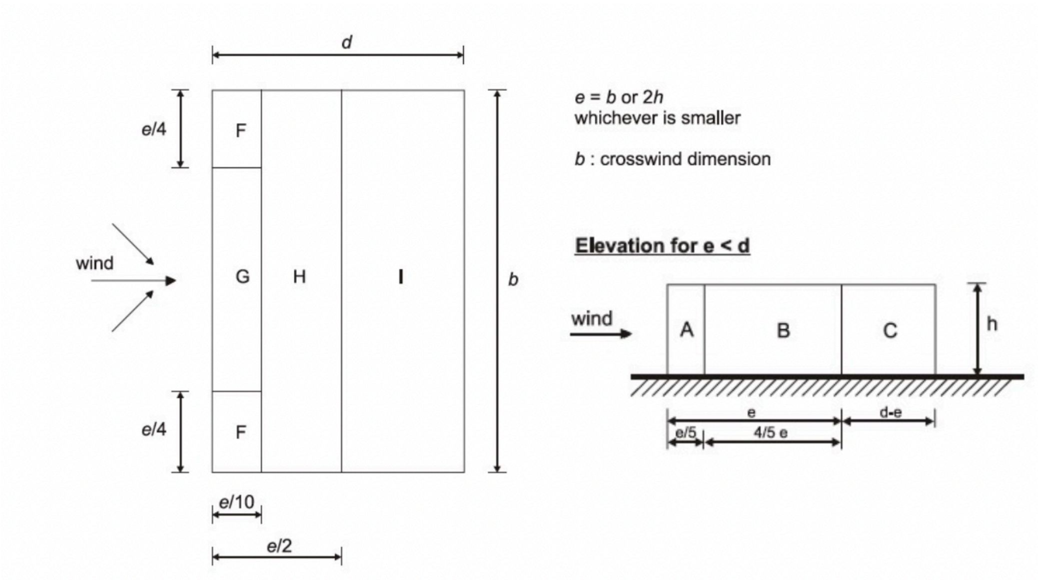

Figure 6-2: Wind distribution zones for wind loads on flat roofs (left) and side walls (right). Source: EN 1991-1-4

Pressure coefficients for wind loads as recommended in EN 1991-1-4 apply for rectangular buildings and are valid for angles ±45° relative to the considered wind direction. A conservative approach for design according to EN 1991-1-4 for buildings without rectangular shape are recommended by Hughes (2014). The recommended approach assumes wind loads equal a width and depth similar to an enclosing rectangle of the building, like illustrated in Figure 6-3. This approach is used in the wind calculations for the L-shaped classroom and library building. The L-shaped classroom and library building, and the accommodation and office building have been studied separately as these are not connected. Detailed figures and descriptions of the calculations can be found in Appendix D.

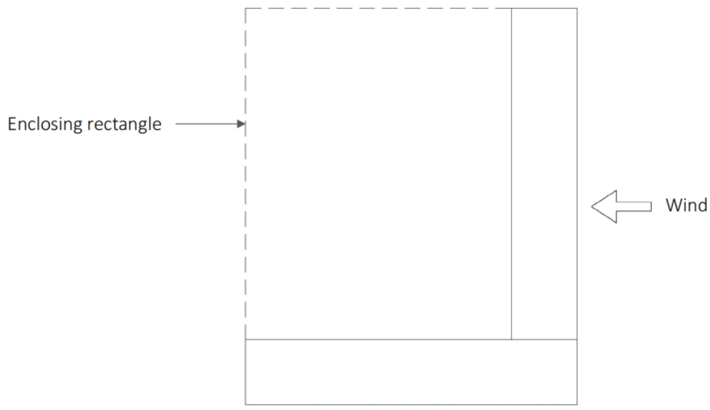

Figure 6-3: Enclosing rectangle for buildings without rectangular shape

Internal pressure coefficients, c_{pi}, depend on the size and placement of openings in the building. c_{pi}-values are determined graphically in EN 1991-1-4 and depends on the h/d-ratio of the building and the factor μ. μ is the ratio between the area of openings on the sides with suction forces (negative c_{pe}) and the total area of openings. Based on the required opening area of 3 m^2, found in chapter 5 to satisfy the daylight requirements in TEK 17, this area is distributed to both side walls and the end wall without doors, for each container. Each side wall is assumed to have four windows with a size 0,6 m x 0,6 m. The end wall is assumed to have one window with similar dimensions as the side wall windows. In addition to windows, entrance doors are placed at one of the side walls as shown earlier in Figure 5-1. Figure 6-4 below show the intended orientation of each shipping container. The end walls marked in blue represent the end wall without windows (the end with container doors) for each container.

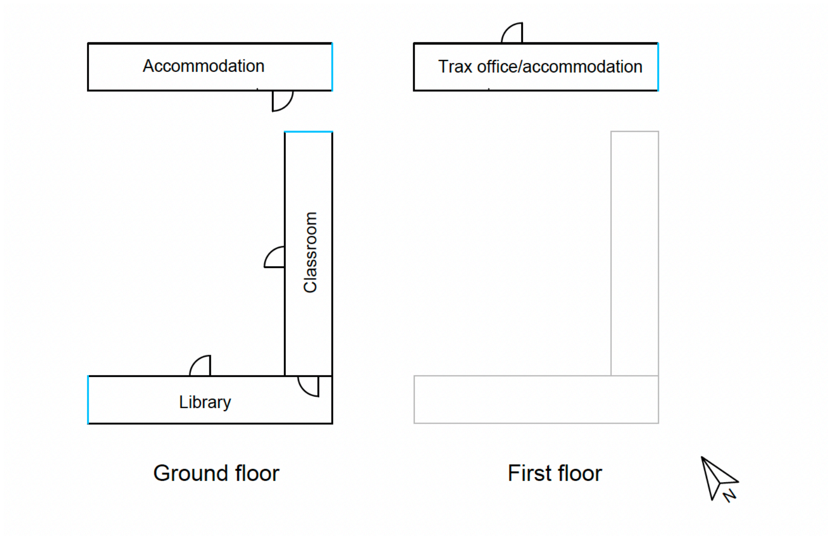

Figure 6-4: Suggested orientation of the containers with container doors marked in blue.

6.1.3 Pressure coefficients for external roofs (canopy roof)

For freestanding canopy roofs, EN 1991-1-4 provide net pressure coefficients, $c_{p,net}$. Wind loads perpendicular to the roof surface are found by multiplying the peak velocity pressure with the net pressure coefficient:

$$w = q_p(z_e) \cdot c_{p,net}$$

Similar to buildings, the roof surface of canopy roofs is divided into zones with higher local suction forces near the corners and edges of the roof, like illustrated in Figure 6-5. The $c_{p,net}$ values obtained for each zone depend on the roof angle and the degree of blockage, φ, underneath the roof. The blockage is the relation between the area of obstacles and the total area under the roof perpendicular to the wind direction.

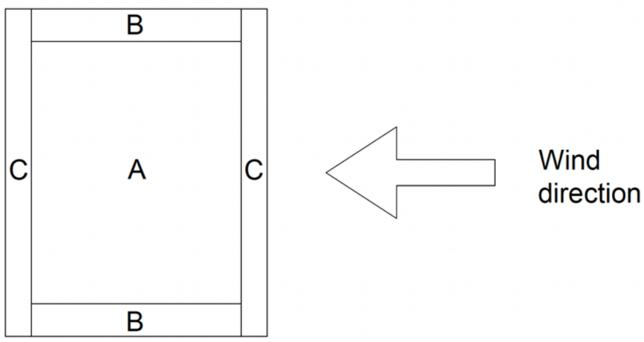

Figure 6-5: Zones for wind load distribution on canopy roofs

6.1.4 Friction forces due to wind

Wind sweeping along the surfaces of the containers and external roofs create drag forces in the direction parallel to the surfaces. The size of the friction forces depends on the frictional coefficient, c_{fr}, the peak velocity pressure and the reference area for friction, A_{fr}:

$$F_{fr} = c_{fr} \cdot q_p(z_e) \cdot A_{fr}$$

The frictional coefficient describes the texture of the surfaces and can be found in Table 7.10 in EN 1991-1-4. Logically, a smooth surface has a lower frictional coefficient than a rougher surface. In accordance with Table 7.10, a smooth surface is assumed for wind parallel to the longitudinal direction of the corrugations, while a very rough surface is presumed for wind parallel to the transverse direction of the corrugations.

The reference area for friction is determined in accordance with §7.2(3) and Figure 7.22 in EN 1991-1-4, reproduced in Figure 6-6. Reference areas for buildings are the surface area parallel to the wind beyond a distance equal to 2b or 4h from the windward side, where b is the width of the building and h is the building height. In cases where the depth of the building is small compared to the width and height, the reference area for frictional forces may end up outside the footprint of the building. Thus, it is assumed that the surface area of the building parallel to the wind are too small for frictional forces to develop. For canopy roofs, the frictional area is equal to the area of the top and bottom surface, corresponding to friction forces developing along the entire length of the roof.

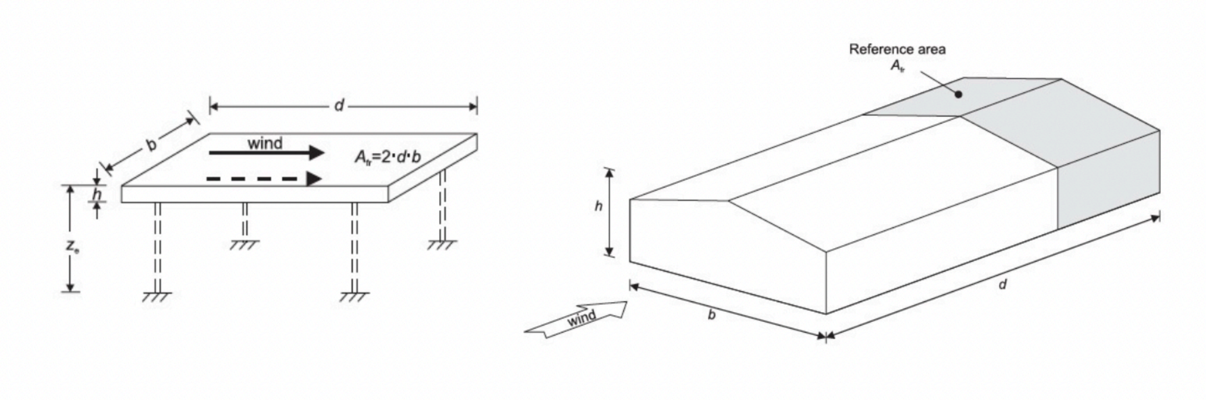

Figure 6-6: Reference areas for friction according to EN 1991-1-4.
Source: EN 1991-1-4

7 Load combination factors

In this phase of the project rough estimations of the roof and foundation elements are done in the ultimate limit state (ULS) to ensure sufficient load bearing capacity. Serviceability limit state design (SLS) is not considered as this limit state concerns the behavior of the structure in terms of tolerances for equipment and user comfort regarding deformations and dynamic responses. Thus, design in SLS is not necessary to determine the structural safety of the structure. Load combinations for ULS are created in accordance with equation 6.10a and 6.10b in EN 1990, recited below.

$$q_d = \gamma_G G_k + \gamma_{Q,1} \Psi_{0,1} Q_{k,1} + \textstyle\sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \qquad (6.10a)$$

$$q_d = \xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1} + \textstyle\sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \qquad (6.10b)$$

Where:
- γ_G Partial safety factor for permanent actions
- γ_Q Partial safety factor for variable actions
- Ψ_0 Combination factor for variable actions
- ξ Reduction factor

Partial factors for the load types described in chapter 6 are given in Table 7-1 in agreement with EN 1990.

Table 7-1: Recommended partial factors for design in ULS according to EN 1990.

Load type	Partial factors		
	Symbol	Unfavorable	Favorable
Dead load	γ_G	1,35	1,0
	ξ	0,89	-
Live load (floor)	γ_Q	1,5	0
	Ψ_0	0,7	-
Live load (roof)	γ_Q	1,5	0
	Ψ_0	0	-
Wind load	γ_Q	1,5	0
	Ψ_0	0,6	-

8 Roof design

To support the external roofs covering the accommodation and office building and the classroom, library and outdoor area, supporting structures need to be designed. Materials such as steel and timber are common materials for such structures (Larsen, 2008). As mentioned in chapter 2.2 steel is mostly an imported material in Ghana, while timber for structural purposes is grown in the southern part of the country and can therefore be derived from local resources (Baiden et al., 2005). Due to this a wooden roof supporting structure is considered more reliable from an economic point of view as variations in exchange rates for different currencies do not affect the prices of timber. Bamboo is also a common material for roof supporting structures in countries where this is grown locally. However, as the author of this thesis is not familiar with the design procedure of bamboo structures, a suggested solution for the roof supporting structure made of wood will be investigated.

Termites and deforestation problems, as explained in chapter 2.2, are the two main disadvantages of using timber in the roof supporting structure. To deal with these problems, measures have to be taken in order to ensure a normal lifespan of the structure and avoid further contribution to the deforestation. Termites live and move in dark nests and channels shielded from the outdoor environment and feed upon cellulose which is found in wooden species. Timber members in direct contact with the ground are therefore more prone to termite attacks than timber structures raised above the ground. However, termites can create mud tunnels which enable them to climb structures without being in contact with the outdoor environment. Figure 8-1 shows an example of mud tubes created by termites on a foundation. Thus, making sure that the timber members do not touch the ground is not a complete solution to the problem. Nevertheless, ensuring that none of the timber members are in direct contact with the ground will allow for the discovery of mud tubes created by termites if regular inspection of foundations and container walls are carried out. Furthermore, using pressure-treated wood is also recommended when building a wooden structure in termite-exposed areas (Krishna et al., 2020). In terms of deforestation, new trees should be planted on the farm to make sure that the use of wood in this project does not contribute to a further deforestation in Ghana. A nice side-effect of this act is the extra shade these trees will provide with time.

Figure 8-1: Mud tubes created by termites.
Source: (Terminix, n.d.)

A layout of the roof supporting structure for the roof covering the outdoor area, classroom and library is shown in Figure 8-2. To not limit the use of the outdoor area due to obstacles such as columns and other supporting structures in the middle of the covered space, relatively large-spanning timber beams are required. These beams are supported by columns in one end and the top of the classroom building in the other end. Above the library container the columns supporting the beams in one end is replaced by the corner fittings of the container.

The beams supported by columns are assumed to be evenly distributed. A total of eight beams, two on top of the library container and six supported by columns and the classroom container, are chosen in this design to give a fairly similar center distance between each beam. As can be seen from Figure 8-2 the center distance between the two beams on top of the library container (between axis 2 and 3) is 2,4 meters. This corresponds to the width of the container and allow the beams to be aligned with the corner fittings of the container. The remaining six beams are evenly distributed over the length of the classroom container which is assumed to be 12,2 meters. This corresponds to a center distance of 2,033 meters between each beam.

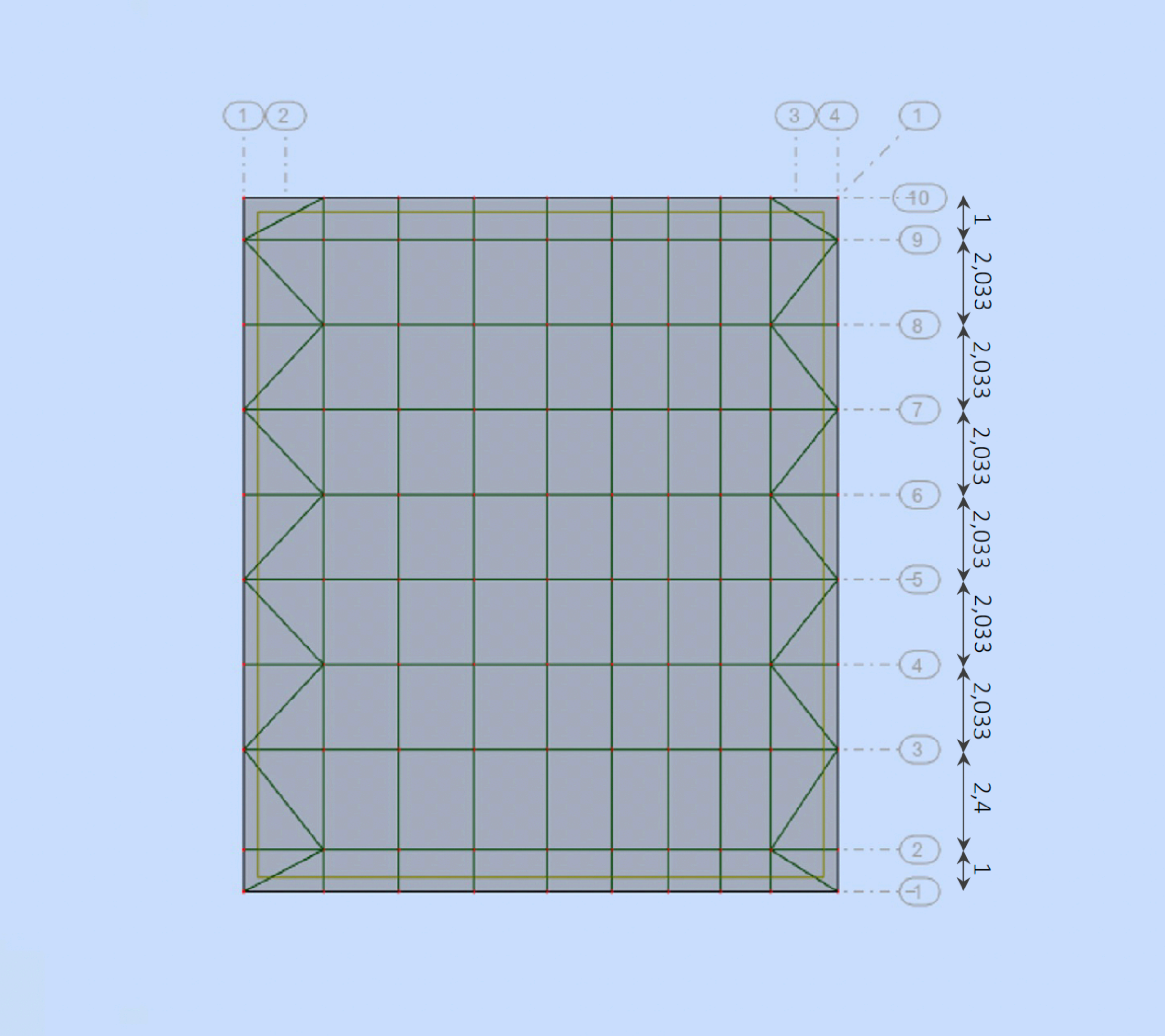

Figure 8-2: Center distances of truss beams supporting the roof above the outdoor area, classroom and library

A rule of thumb regarding the beam height necessary to withstand the imposed loading for a continuous spanning glulam beam is given by Crocetti et al. (2015). For a continuous spanning glulam beam a required beam height equal to L/14 is anticipated. L corresponds to the span length, which in this case is 9,8 meters for the largest span. Hence, the required beam height is 700 mm. Transportation of such long and tall beams may be challenging depending on the condition of the roads and the vehicle length needed to transport these beams to the farm. Moreover, it is not known whether glulam is possible to procure in Ghana or not. Glulam has significantly better strength properties than structural timber and an even taller beam would be required if these beams were to be made of structural timber (Crocetti et al., 2015). Due to the large diameter and length of the lumber that would be needed to make these beams, a roof supporting structure with solid beams is considered unrealistic.

Large-spanning timber beams often require large cross-sections as shown in the previous paragraph due to large moment forces in the mid-span. By reducing the moments, material efficient large-spanning structures can be obtained. Structures such as trusses carry the imposed loads mostly through tensile and compressive forces in its members and is therefore an ideal solution when it comes to large-spanning structures (Crocetti, 2016).

Stresses due to axial forces utilize the cross-section in a more efficient way than bending stresses, as the axial stresses are evenly distributed over the entire cross-section. By comparison, bending stresses are linearly distributed over the cross-sectional height. Thus, the outermost fibers of the cross-section receive larger stresses than the internal fibers. Nonetheless, the cross-section must be designed to carry the largest stresses, and the utilization of the cross-section is therefore not as good as for axial stresses (Crocetti et al., 2015). In addition, trusses are comprised of smaller elements joined together to form series of triangles, which makes transportation of materials and the total weight of the structure more manageable than long solid beams. Materials can be bought locally and transported to the farm in a suitable vehicle where cutting and assembly of members can be completed by local workers and/or volunteers from the rural community using simple tools and techniques.

Load transfer and geometrical relations between diagonals and chord members are some key points to keep in mind when designing a timber truss. In an ideal truss the members are only subjected to axial forces, resulting in either tension or compression in each member. To achieve this, load transfer should occur in the joints of the truss. By placing purlins above the connections in each truss an ideal load transfer is secured. However, it should be noted that in the real-world additional forces will be present due for example self-weight and wind loads. Wood is an orthotropic material with different properties in the direction parallel and perpendicular to the grain. Due to this, the angle between the diagonals and the upper and lower chord should be within the range 45°±15° to better utilize the properties of the timber (Crocetti et al., 2015).

Figure 8-3 illustrates the proposed solution for the truss design taking geometrical and practical considerations into account. All angles between diagonals and upper and lower chords are between 30° - 60°. To allow for both rainwater harvesting from the roof and ensuring that the gap between container roof and the external roof is large enough for the wind to pass through, a minimum gap of 0,6 meters and a roof angle of 4° is recommended. This configuration is also compatible with the truss design, ensuring that critical angles are not surpassed. The height difference between the two sides of the sloping roof is approximately 0,95 meters which should be sufficient for rainwater collection at one side of the roof even with small foundation settlements. It should be noted that Figure 8-3 shows the intended centerlines of each component.

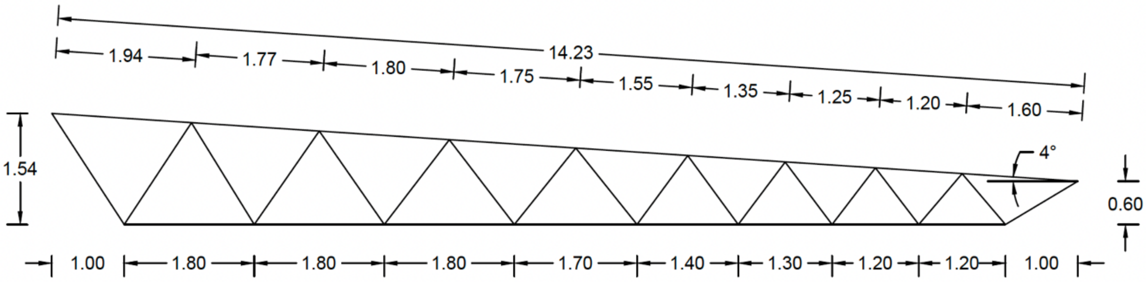

Figure 8-3: Figure of proposed truss design with member lengths in meters

The same truss configuration is assumed to be suitable for the roof supporting structure over the accommodation and office building. For the roof supporting structures above the two toilet containers, a total structure similar to the structure above the accommodation and office unit, in terms of material quantities, are assumed. This assumption has been made due to a lack of time. Hence, no calculations for the toilet containers are carried out.

8.1 Material properties

The wooden structure supporting the roofs are assumed to be made of local solid timber corresponding to a C24 strength class. Material properties for C24 timber from EN 338, which applies for solid timber, is used in the member design and can be found in Table 8-1 . In the future, these strength properties should be checked against the properties of the locally available C24 timber to make sure that they correspond, if not new design checks have to be carried out. As mentioned in the introduction to this chapter, timber has different properties if loaded in the direction parallel to grain or perpendicular to grain. In Table 8-1 the properties valid for actions parallel to grain and perpendicular to grain are denoted "0" and "90" respectively.

Table 8-1: Characteristic material properties for C24 solid timber (Standard Norge, 2016)

Material properties	Symbol	Characteristic value [N/mm^2]
Bending strength	$f_{m,k}$	24
Tensile strength	$f_{t,0,k}$	14,5
	$f_{t,90,k}$	0,4
Compressive strength	$f_{c,0,k}$	21
	$f_{c,90,k}$	2,5
Shear strength	$f_{v,k}$	4,0
5-percentile modulus of elasticity	$E_{0,05}$	7400

The design strength properties for wood depend on the load duration and moisture content of the timber which is regulated by the relative humidity (RH). A modification factor, k_{mod}, takes these effects into account and should be included in the determination of the design strengths. k_{mod} for the shortest load duration in the load combination resulting in the largest design forces is used in calculation of design strengths. As mentioned in chapter 4, Bolgatanga has values of RH ranging between 17% - 84% during the year, corresponding to service class 2 which applies for a RH in the range 65% - 85% (Eie, 2016). Table 8-2 gives k_{mod} values obtained from EN 1995-1-1.

Table 8-2: k_{mod} for different load types according to EN 1995-1-1

Load type	Load duration	k_{mod}
Dead load	Permanent	0,60
Live load	Long-term	0,70
Wind load	Short-term	0,90

The partial safety factor for the material properties of solid timber in ULS, γ_M, is set to 1,3 according to the general recommendation in Table 2.3 in EN-1995-1-1.

8.2 Calculation model

The roof supporting structure for both the covered outdoor area and the external roof of the accommodation and office building have been modelled as two separate models in Autodesk Robot using 3D frame structures.

8.2.1 Classroom/ Library/ Covered outdoor area

Trusses supported by columns and roofs of the classroom and library containers have been modelled with the center-to-center measurements as shown in Figure 8-2. Figure 8-5 shows the different elements of the roof structure. The truss members are modelled using bar elements. The diagonals of the truss are modelled with pinned-pinned releases (pinned connections in each end) to avoid any moment transfer to these elements. Robot refers to the local axis of the bar when determining releases. Pinned-pinned connections are therefore modelled by restraining all translations (UX, UY, UZ), in addition to rotation about the longitudinal axis of the bar (RX). Rotations about the local y- and z-axis are released for both ends. Figure 8-4 shows the local axis system for bar elements.

At least one of the intersecting members in a connection cannot be modelled with pinned releases, otherwise a mechanism is created in this joint and instability warnings will display when trying to run the model. Due to this, the upper and lower chords are modelled as beam elements with releases set to default at their ends. The default release in Robot corresponds to a fixed-fixed release where all translations (UX, UY, UZ) and rotations (RX, RY, RZ) are restrained.

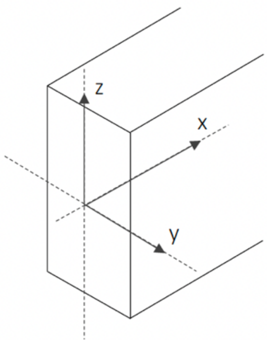

Figure 8-4: Illustration of local axis system for bar elements

Ten purlins are added on top of the trusses to transfer the forces from the corrugated roof sheeting to the trusses. The center distance between these purlins varies depending on the distance between each joint in the upper chord. Each purlin is extended 1 meter beyond the first and last truss to carry the loads from the roof extension needed to provide shade as described in chapter 5. As the upper chords of the trusses have an inclination due to the sloping roof, the cross-sections of each purlin have been rotated to match the sloped upper chord. Rotation of the cross-sectional axis of the purlins are done by changing the gamma-angle to -4 degrees. Each purlin is modelled with pinned-pinned releases and is anchored in sideboards on each side of the roof as shown in Figure 8-5. The sideboards, however, serve no structural purposes but are necessary to avoid instabilities.

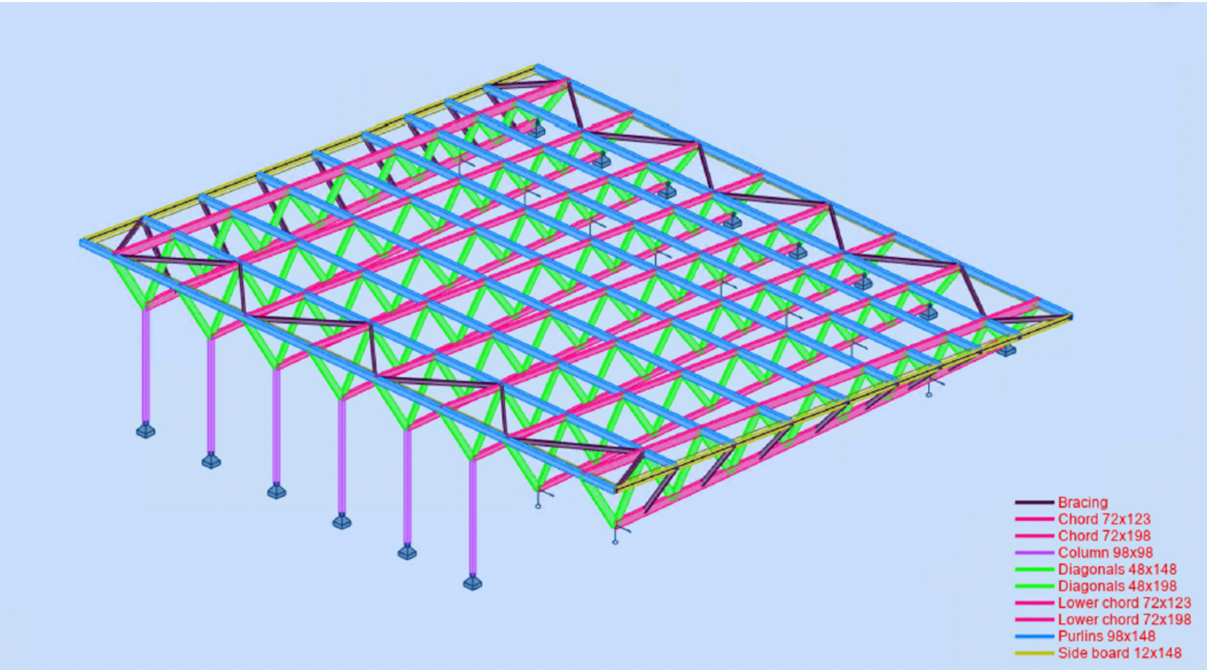

Figure 8-5: Roof supporting structure over outdoor area, classroom and library showing the different members

Pinned supports are added to the base of the six columns. To avoid moment transfer from truss to column, releases for the column are modelled as pinned-fixed releases. The pinned release connects the top of the column to the truss and thus makes sure that no moment is transferred through this connection. Furthermore, the column is considered “fixed” into the pinned support as releases on this end are already provided by the support. The column length is set to 2,9 meters, which is the same as the container heights. Thus, ensuring that the timber columns are not in direct contact with the ground and reducing the risk of termite attacks.

The remaining supports anchored on top of the containers are modelled to allow for expansion in the global x-direction but is restrained in the global y- and z-direction. The global x-direction corresponds to the longitudinal direction of the trusses as shown in Figure 8-6. The supports furthest away from the columns are modelled as pinned supports with restrained translations in all directions (UX, UY, UZ). The remaining supports are modelled as pinned supports with UX released to eliminate unwanted coercive forces and allow for expansion of the structure due to swelling and shrinkage triggered by changes in moisture content (Eie, 2016). Figure 8-6 shows the supports for the trusses supported by columns and the trusses supported by the containers only.

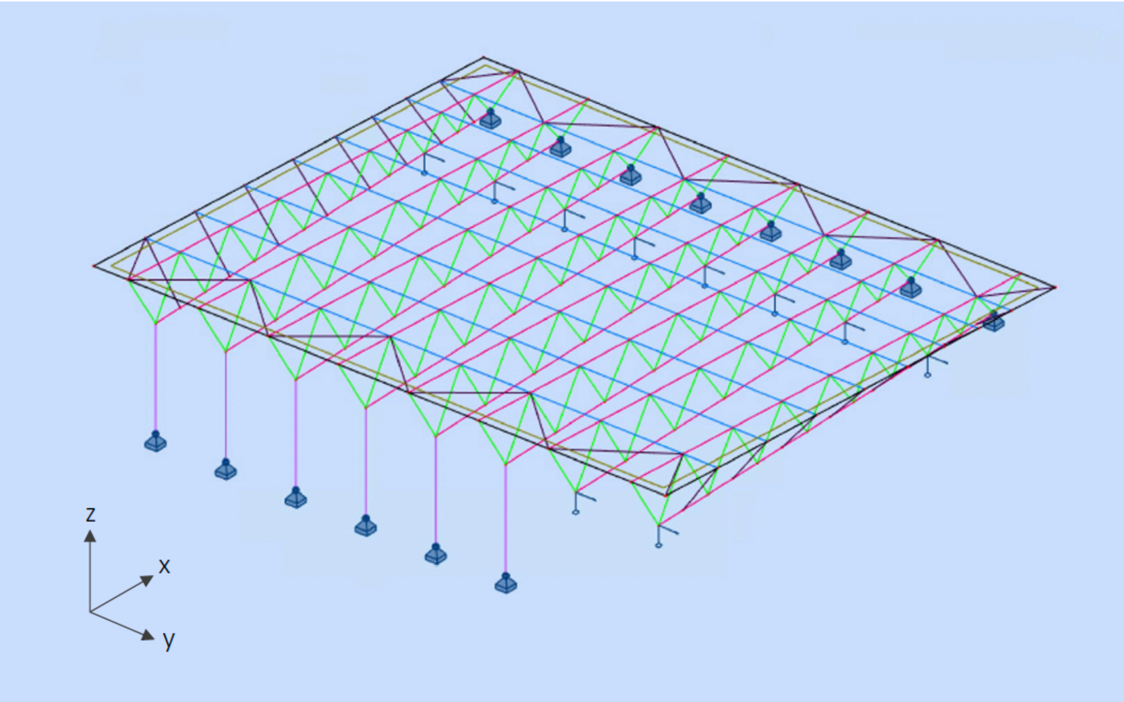

Figure 8-6: Modelled supports in Robot showing the restrained translations for the roof above outdoor area, classroom and library

Claddings are added to disperse the evenly distributed loads on the roof to the purlins. Due to uncertainties in the available corrugated sheeting materials for the roof cover and their load bearing properties, a cladding whose only purpose is load distribution is chosen. Hence, the stiffness and structural properties of the roofing are neglected. The load distribution of the cladding is set to one-way x-direction, which transfer the loads to the purlins and not to the upper chords of the trusses as these elements have their structural axes in the same plane in the Robot model. The sideboards and bracing elements in the same plane as the roof have been ignored from the cladding load distribution.

Bracing elements have been included in the model but have not been designed in this phase. Due to lateral forces, bracing elements must be added to be able to run the model without instability warnings. These instabilities are shown by very large and unrealistic displacements in the global y-direction, due to insufficient support of the purlins and trusses in this direction. To be able to run the model with no error messages bracing is added by anchoring the purlins to the lower chord of the outer trusses on each side. Additional bracing is added between the two outer purlins on each side as shown in Figure 8-5. All bracing elements are modelled with pinned-pinned releases to prevent moment transferring connections. However, by introducing these anchoring elements, bending moments and shear forces will develop in the lower chords of the outer trusses due to the axial forces from the bracing.

8.2.2 Accommodation and office

All components of the structure supporting the external roof of the accommodation and office building are modelled the same way as the roof covering the classroom, library and outdoor area. The only difference is the number of supports. To reduce the design forces in each member, the two trusses on top of the accommodation and office building are supported in every second joint of the lower chord as shown in Figure 8-7. The supports at one of the ends of each truss are modelled as pinned supports restrained from translation in all directions (UX, UY, UZ). The remaining supports are modelled as pinned supports with y- and z-translations restrained to allow the trusses to expand due to swelling and shrinking besides reducing the coercive forces.

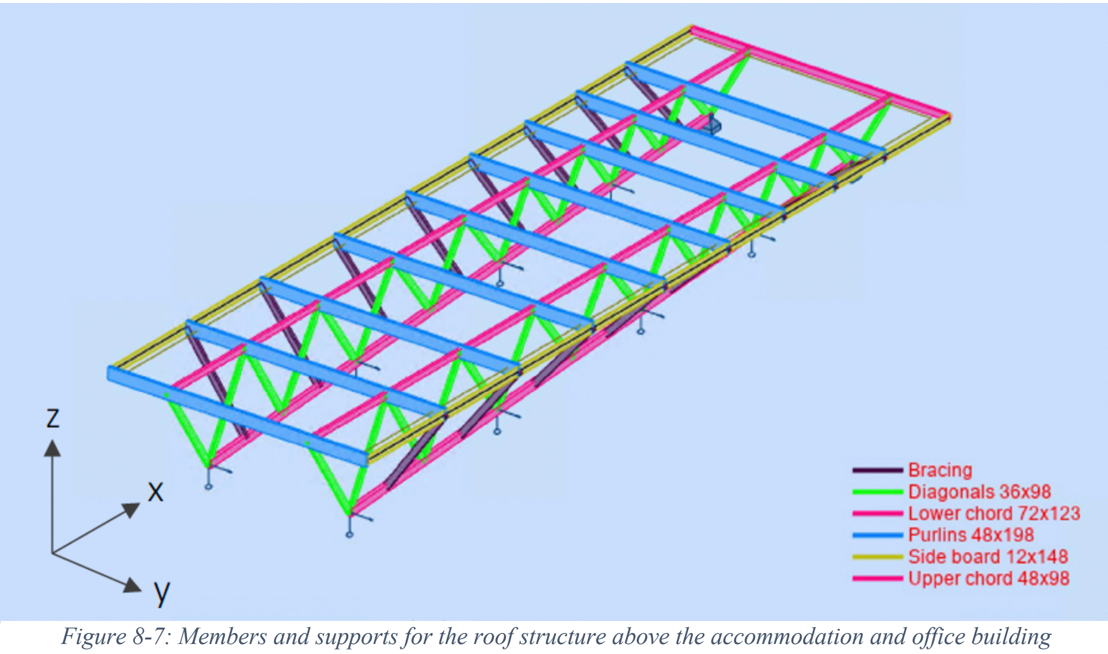

Figure 8-7: Members and supports for the roof structure above the accommodation and office building

8.3 Loads

Separate load cases for dead loads, live loads and wind loads are created. Dead loads corresponding to the self-weight of the corrugated roofing have been applied as an evenly distributed surface load to the cladding. The corrugated sheeting is assumed to have a self-weight of 10 kg/m^2 per mm thickness (Porteous and Kermani, 2007). A thickness equal to 2 mm have been assumed, thus resulting in a self-weight of 20 kg/m^2 which is approximately equal to 0,20 kN/m^2. The self-weight of the timber members is added automatically in Robot.

Table 8-3 give live loads for roofs according to both GBC and EN 1991-1-1. For comparison recommended values according to both codes are given. However, as mentioned in chapter 3.1 live loads according to GBC is used in the calculations.

Table 8-3: Live loads according to GBC and EN 1991-1-1 for roofs

Zone	Category according to EN 1991-1-1	GBC [kN/m^2]	EN 1991-1-1 [kN/m^2]
Roof	H	0,25	0,75

Two load cases for live loads acting on the roof above the classroom, library and outdoor area, and two load cases for live loads acting on the roof above the accommodation and office are created. One of the load cases (LL1) for each roof has the live load applied to the entire roof. The second load case for the live load (LL2) is added based on separate analyses of the purlins resulting in the largest reaction forces transferred from the purlins to the trusses. Figure 8-8 shows the application of LL2 for both roofs.

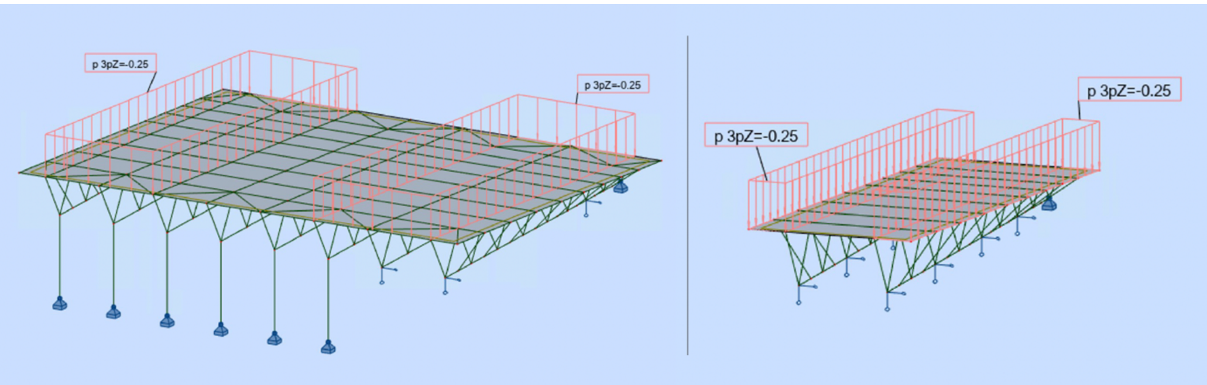

Figure 8-8: Application of LL2 for roof over outdoor area, classroom and library (left) and roof over accommodation and office (right)

Wind loads (suction and friction) are applied to load cases for each wind direction. Wind loads from the north/south are assumed to be equal, and three load cases have therefore been created: WL east, WL west and WL north/south. Suction forces act perpendicular to the cladding and are therefore applied in the local z-direction of the cladding. However, this is not shown visually in Robot for this type of load application (uniform loads on contours) but is verified to be correct as the support reactions display forces in both the vertical and horizontal direction.

As described in chapter 6.1, wind loads on roofs are separated into different zones with different intensities. All wind loads and their corresponding zones used in the load application can be found in Appendix D. Wind loads on the roof supporting structure, i.e., columns, trusses and purlins, are neglected due to the unknown dimensions of these members. A reference height equal to the greatest height difference between roof and ground is used for the calculation of suction forces. For the friction forces, a reference height equal to the thickness of the roof is used. A thickness of 0,1 m is assumed as the distance between the top and bottom point of two neighboring corrugations. Friction forces are applied as line loads at the rear edge of the roof relative to the wind direction.

8.4 Load combinations

Six manual load combinations have been created for each roof model. The loads included in each combination and their respective partial safety factors according to EN 1990 can be found in Table 8-4.

As the dead loads are smaller than the live loads, only load combinations according to equation 6.10b as described in chapter 7 are created for the combination of dead loads and live loads. The wind loads and live loads act in the opposite direction of each other. Hence, these two load types are never included in the same load combination as either of them will be equal to zero due to their favorable influence on each other.

Wind loads and dead loads also act in the opposite direction of each other. Thus, load combinations according to equation 6.10b in EN 1990, as mentioned above, including dead loads acting favorable of the wind loads are created for the combined effect of these two load cases.

Table 8-4: Manual load combinations (LC) created in Robot

LC	DL	LL1	LL2	WL east	WL west	WL north/south
1	1,20	1,50				
2	1,20		1,50			
3	1,35					
4	1,0			1,50		
5	1,0				1,50	
6	1,0					1,50

8.5 Design forces

The largest design forces for each member of the roof supporting structure, except the bracing, are found for both roofs and are displayed in Tables 8-5 to 8-9. Further design checks are carried out for the member of each element type receiving the biggest forces. All design forces obtained for each element type in Tables 8-5 to 8-9 are due to load combinations including dead load and wind load, with some exceptions. The tensile forces in the upper and lower chord of the trusses over the accommodation and office facilities and the compressive forces in the columns, correspond to a load combination including dead loads and live loads. Tensile forces in the purlins above the accommodation and office building is also a result of a dead load and live load combination.

Design forces for the trusses supporting the roof above the classroom, library and outdoor area are obtained for both the outer trusses on each side of the roof and the internal trusses in-between these as shown in Figure 8-9. A difference of approximately 10 kN in the axial forces acting in the two exterior trusses compared to the interior trusses is observed from Table 8-5 and Table 8-6. This increase in design forces for the outer trusses are expected due to high local suction forces on the corners and edges of the roof. The lower chord of the two outer trusses is subjected to larger forces than the internal ones due to the bracing elements between purlins and lower chord. Separate design checks for the exterior and interior trusses are therefore performed.

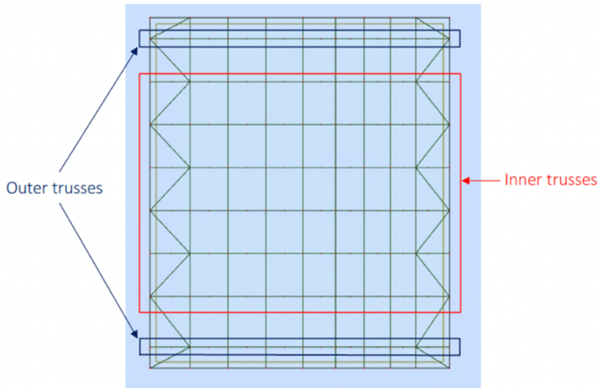

Figure 8-9: Definition of outer and inner trusses

The design forces for each element given in Robot corresponds to the local coordinate system of the member. Consequently, F_x is the axial force, F_y and F_z are shear forces acting in y- and z-direction respectively. In the following tables F_x, F_y and F_z have been replaced with $N_{t,Ed}$ and $N_{c,Ed}$ for axial forces acting in tension and compression correspondingly. $V_{y,Ed}$ and $V_{z,Ed}$ corresponds to F_y and F_z. The subscripts "s" and "f" for the moments refer to support and field moments specifically. Very small torsional moments in the range 0,01-0,02 kNm are observed for some members, and these are therefore neglected.

Table 8-5: Design forces for outer trusses supporting the roof above classroom, library and outdoor area

	$N_{t,Ed}$ [kN]	$N_{c,Ed}$ [kN]	$V_{y,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$M_{y,f,Ed}$ [kNm]	$M_{y,s,Ed}$ [kNm]	$M_{z,f,Ed}$ [kNm]	$M_{z,s,Ed}$ [kNm]
Diagonals	55,34	38,53	-	-	-	-	-	-
Upper chord	51,66	58,99	0,47	0,34	0,38	0,38	-	0,32
Lower chord	32,96	61,74	2,36	2,48	0,52	1,83	0,40	1,22

Table 8-6: Design forces for inner trusses supporting the roof above the classroom, library and outdoor area

	$N_{t,Ed}$ [kN]	$N_{c,Ed}$ [kN]	$V_{y,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$M_{y,f,Ed}$ [kNm]	$M_{y,s,Ed}$ [kNm]	$M_{z,f,Ed}$ [kNm]	$M_{z,s,Ed}$ [kNm]
Diagonals	45,96	30,79	-	-	-	-	-	-
Upper chord	41,98	47,73	0,38	0,38	0,20	0,39	-	0,26
Lower chord	27,87	46,34	-	0,44	0,13	0,42	-	-

Table 8-7: Design forces for trusses supporting the roof above the accommodation and office building

	$N_{t,Ed}$ [kN]	$N_{c,Ed}$ [kN]	$V_{y,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$M_{y,f,Ed}$ [kNm]	$M_{y,s,Ed}$ [kNm]	$M_{z,f,Ed}$ [kNm]	$M_{z,s,Ed}$ [kNm]
Diagonals	17,20	3,31	-	-	-	-	-	-
Upper chord	1,72	12,54	0,06	0,03	-	-	-	0,07
Lower chord	0,60	5,65	0,78	0,55	-	0,26	0,22	0,46

Table 8-8: Design forces for purlins

	$N_{t,Ed}$ [kN]	$N_{c,Ed}$ [kN]	$V_{y,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$M_{y,f,Ed}$ [kNm]	$M_{y,s,Ed}$ [kNm]	$M_{z,f,Ed}$ [kNm]	$M_{z,s,Ed}$ [kNm]
Outdoor area	1,67	2,10	0,54	8,30	1,63	4,08	0,07	0,51
Accommodation and office	0,25	0,76	0,11	7,78	1,03	2,27	0,02	0,09

Table 8-9: Design forces for columns

$N_{t,Ed}$ [kN]	$N_{c,Ed}$ [kN]
45,31	8,67

8.6 Design of roof members

Design of cross sections and member stability have been done according to EN 1995-1-1. To apply for funds in this phase of the project a rough estimation of the necessary running meters and cross-sections of timber is needed. Design of the connections are therefore not required at this stage. Most design checks in EN 1995-1-1 require the design stresses to not exceed the design strengths. In other words, the utilization of either cross-section properties or member capacity should be below 100%. Except from cases where EN 1995-1-1 specifically recommends a lower utilization ratio, cross-sections and members are designed to not exceed a utilization ratio of approximately 85%. Thus, a safety margin is included in the design in addition to the partial safety factors used in the calculations. k_{mod} is determined in agreement with the load combination providing the design forces for each member.

8.6.1 Design of cross-sections

Tables 8-5 to 8-9 show that all members are subjected to both tensile and compressive axial forces. Design check for the utilization of the tensile capacity is given by equation 6.1 in EN 1995-1-1 and should be in agreement with:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} \leq 1$$

Where: $\sigma_{t,0,d}$ Tensile design stresses acting on the net cross sectional area A_{net}

$f_{t,0,d}$ Design tensile strength parallel to grain

A_{net} is the reduced cross-sectional area due to cut outs and holes created by connectors for instance. As no connections are designed in this phase of the project, A_{net} is assumed to be equal to the gross cross-sectional area, A, and adjustments for the reduced cross-section will have to be done in the next phase.

Equation 6.2 in EN 1995-1-1 gives the design check for compressive axial stresses:

$$\frac{\sigma_{c,0,d}}{f_{c,0,d}} \leq 1$$

Where: $\sigma_{c,0,d}$ Compressive design stresses acting on the net cross-sectional area A_{net}

$f_{c,0,d}$ Design compressive strength parallel to grain

The reduction in the cross-sectional area A_{net} can according to §5.2(3) in EN 1995-1-1 be neglected for members in compression if the holes are filled with materials of higher stiffness than the timber. Assuming that the connectors have a higher stiffness than the timber, A_{net} is set equal to A. §9.2.1(4) in EN 1995-1-1 propose a 10% increase in the design compressive forces for truss members in compression. This increase in compressive stresses is applied to the design of all truss elements, namely: diagonals, upper and lower chords.

§9.2.1(5) in EN 1995-1-1 limits the utilization of truss-elements acting in either tension or compression to 70%. The diagonals, upper and lower chords of all trusses are thus covered by this regulation.

Elements exposed to bending stresses about either one or two principal axes must fulfill the following requirement:

$$\frac{\sigma_{m,y,d}}{f_{m,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,d}} \leq 1 \quad \text{and} \quad k_m \frac{\sigma_{m,y,d}}{f_{m,d}} + \frac{\sigma_{m,z,d}}{f_{m,d}} \leq 1$$

Where: $\sigma_{m,y,d}$ Design bending stresses for moment about y-axis

$\sigma_{m,z,d}$ Design bending stresses for moment about z-axis

$f_{m,d}$ Design bending strength

k_m Factor taking redistribution of stresses into account. EN 1995-1-1 recommends a value of 0,7 for rectangular cross-sections made of solid timber.

The combined effect of normal stresses due to axial forces and bending moments should be checked for elements in tension and compression. The utilization of members both in tension and bending are restricted by equation 6.17 and 6.18 in EN 1995-1-1 and should not exceed:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad \text{and} \quad \frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

Design checks for elements subjected to both compression and bending are given by equations 6.19 and 6.20 in EN 1995-1-1:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad \text{and} \quad \left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

For truss-members the utilization of elements in compression and bending by equation 6.19 and 6.20 should be limited to 90% according to §9.2.1(1) in EN 1995-1-1. However, as mentioned earlier in this chapter the elements will be designed to not exceed a utilization ratio of 85%. Consequently, the latter condition applies to this design check.

Shear stresses in the local z- and y-direction of the elements should be verified to not exceed the design shear strength as given in equation 6.13 in EN 1995-1-1:

$$\frac{\tau_{v,y,d}}{f_{v,d}} \leq 1 \quad \text{and} \quad \frac{\tau_{v,z,d}}{f_{v,d}} \leq 1$$

Where: $\tau_{v,y,d}$ Design shear stresses in y-direction

$\tau_{v,z,d}$ Design shear stresses in z-direction

$f_{v,d}$ Design shear strength

The design shear stresses are calculated by using an effective width which takes the effect of moisture induced cracks into account resulting in a reduced cross-section for the shear stress distribution (Blass and Sandhaas, 2017). For a rectangular cross-section the maximum shear stresses are calculated as:

$$\tau_{v,Ed} = \frac{3}{2} \cdot \frac{V_{Ed}}{b_{ef} \cdot h} = \frac{3}{2} \cdot \frac{V_{Ed}}{k_{cr} \cdot b \cdot h}$$

Where: V_{Ed} Design shear force

b_{ef} Effective width

k_{cr} Reduction factor taking moisture induced cracks into consideration
For solid timber a value of 0,67 is recommended by EN 1995-1-1.

h Height of cross section

8.6.2 Member stability – Axial buckling

Members subjected to compressive forces or a combination of compression and bending moments in the span may be prone to axial buckling. To avoid axial bucking, design procedures according to chapter 6.3.2 in EN 1995-1-1 are followed. Equation 6.23 and 6.24 give the design checks for buckling about the y-axis of the element (in-plane buckling) and buckling about the z-axis of the element (out-of-plane buckling) respectively. In-plane buckling does not occur if the following requirement is satisfied:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

Out-of-plane buckling is prevented if the subsequent condition is not exceeded:

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

Where: $k_{c,y}$ Instability factor for buckling about y-axis

$k_{c,z}$ Instability factor for buckling about z-axis

The instability factors $k_{c,y}$ and $k_{c,z}$ reduce the design compressive strengths, as can be seen from the equations above, and are given by equations 6.25 an 6.26 in EN 1995-1-1:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \quad \text{and} \quad k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$$

Where: k_y Instability factor for buckling about y-axis

k_z Instability factor for buckling about z-axis

$\lambda_{rel,y}$ Relative slenderness for bending about y-axis

$\lambda_{rel,z}$ Relative slenderness for bending about z-axis

k_y and k_z are functions of the relative slenderness and the straightness of the element. Equations 6.27 and 6.28 in EN 1995-1-1 provide expressions for k_y and k_z:

$$k_y = 0{,}5(1 + \beta_c(\lambda_{rel,y} - 0{,}3) + \lambda_{rel,y}^2 \quad \text{and} \quad k_z = 0{,}5(1 + \beta_c(\lambda_{rel,z} - 0{,}3) + \lambda_{rel,z}^2$$

Where: β_c Straightness factor
For solid timber a value of 0,2 is recommended by EN 1995-1-1.

The recommended value for β_c applies to structural timber with a deviation in straightness less than 1/300 of the element length as determined in §10.2(1) in EN 1995-1-1. This criterion is assumed to be satisfied and should be kept in mind when procuring materials in the construction phase.

The relative slenderness for bending about y-axis and z-axis according to equations 6.21 and 6.22 in EN 1995-1-1 are:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad \text{and} \quad \lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

Where:
- λ_y Slenderness ratio for bending about y-axis
- λ_z Slenderness ratio for bending about z-axis
- $f_{c,0,k}$ Characteristic strength for compression parallel to grain
- $E_{0,05}$ 5-percentile value of modulus of elasticity

The slenderness of an element describes the relation between the effective length of the member and the radius of gyration (Blass and Sandhaas, 2017). The radius of gyration describes the distribution of cross-sectional area about its centroid axis (Engineering ToolBox, 2008):

$$\lambda_y = \frac{l_{k,y}}{i_y} = \frac{l_{k,y}}{\sqrt{\frac{I_y}{A}}} \quad \text{and} \quad \lambda_z = \frac{l_{k,z}}{i_z} = \frac{l_{k,z}}{\sqrt{\frac{I_z}{A}}}$$

Where:	$l_{k,y}$	Effective buckling length for buckling about y-axis
	$l_{k,z}$	Effective buckling length for buckling about z-axis
	i_y	Radius of gyration about y-axis
	i_z	Radius of gyration about z-axis
	I_y	Second moment of area about y-axis
	I_z	Second moment of area about z-axis

The effective buckling lengths $l_{k,y}$ and $l_{k,z}$ depend on the support conditions for each element. For truss-members §9.2.1(4) in EN 1995-1-1 give recommended effective buckling lengths. The diagonals in the truss components are assumed to have hinged connections at both their ends and the buckling length therefore corresponds to the length of each member, L. Only the diagonal receiving the largest compressive design force are checked against axial buckling.

Buckling lengths for the upper and lower chord are found assuming that these elements are continuous spanning. According to §9.2.1(4) the buckling length for end spans of continuous spanning truss-members should be taken as 0,8 of the span length, L_{span}, while the buckling length for internal spans is set to 0,6 times the span length. Buckling of the members are in this phase of the project not calculated for every span and member. To be on the safe side a buckling length equal to 0,8 times the longest span for both upper and lower chord is assumed.

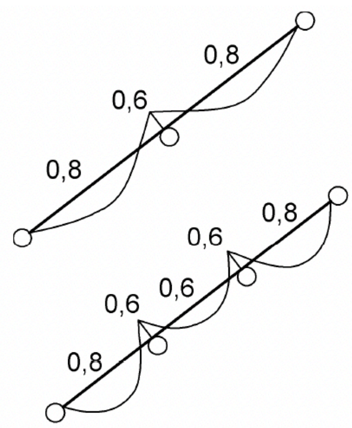

Figure 8-10: Buckling lengths for truss members without moment resisting connections. Source: EN 1995-1-1

For simplicity, buckling lengths for the columns and purlins are assumed to be based on the same conditions as the truss members. A buckling length equal to the member length is assumed for the simply supported columns. Moreover, the buckling lengths for the upper and lower chords of the trusses are assumed to be applicable to the purlins. Thus, a buckling length equal to 0,8 times the longest span of the purlins is used.

8.6.3 Member stability - Lateral torsional buckling

Elements subjected to bending moments about their strong axis may be prone to lateral torsional buckling (LTB), if the height-to-width ratio (h/b) of the cross-section becomes large. Bending moment about the element's strong axis, sometimes in combination with compressive axial forces, may result in a lateral deflection and twisting of the cross-sectional axis of the member (Blass and Sandhaas, 2017). Chapter 6.3.3 in EN 1995-1-1 concerns the design of members where LTB may occur. To prevent LTB from happening, the following design check given by equation 6.35 in EN 1995-1-1 must be satisfied:

$$\left(\frac{\sigma_{m,y,d}}{k_{crit} \cdot f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \leq 1$$

Where: k_{crit} Strength reduction factor for lateral buckling

The value of k_{crit} depends on the relative slenderness for bending, $\lambda_{rel,m}$. If $\lambda_{rel,m}$ is increased the value of k_{crit} decreases resulting in a lower design bending strength in the design check for LTB above. Depending on the value of $\lambda_{rel,m}$, EN 1995-1-1 recommends k_{crit} to be:

For $\lambda_{rel,m} \leq 0{,}75$: $k_{crit} = 1{,}0$

For $0{,}75 < \lambda_{rel,m} \leq 1{,}4$: $k_{crit} = 1{,}56 - 0{,}75\lambda_{rel,m}$

For $1{,}4 < \lambda_{rel,m}$: $k_{crit} = 1/(\lambda_{rel,m}^2)$

The relative slenderness is given by the square-root of the relation between the characteristic bending strength and the critical bending stress and can be found in equation 6.30 in EN 1995-1-1:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}}$$

Where: $f_{m,k}$ Characteristic bending strength

$\sigma_{m,crit}$ Critical bending stress

The critical bending stresses for rectangular cross sections made of softwood, which includes C24 structural timber, is determined from equation 6.32 in EN 1995-1-1:

$$\sigma_{m,crit} = \frac{0{,}78 \cdot b^2 \cdot E_{0,05}}{h \cdot l_{ef}}$$

Where: l_{ef} Effective length of member

The effective length of a member depends on the load application, load type and the support conditions. Load application at the compressive side of the element is considered unfavorable in terms of LTB, as this increases the instability and risk of twisting. The upper and lower chord of the trusses, in addition to the purlins, are subjected to bending moments, hence, these members have to be checked against LTB. The stiffness of the corrugated roofing is neglected as the available sheeting options in Bolgatanga are not known. Consequently, the purlins are not assumed to be sufficiently supported for out-of-plane deformations in the spans and these members must be included in the LTB-checks.

Table 6.1 in EN 1995-1-1 give recommended values for l_{ef} for simply supported beams and cantilever beams. The purlins are assumed to be supported out-of-plane by the trusses and vice versa. An effective length equal to 0,9 times the length of the longest span in each member plus two times the cross-sectional member height is obtained from EN 1995-1-1. This value corresponds to simply supported beams with evenly distributed loading adjusted for load application on the compressive edge.

8.7 Required member sizes

The required cross-sections to satisfy all design checks described in chapter 8.6 are found in Table 8-10 for the members supporting the roof above the classroom, library and outdoor area. Complete design checks with their respective utilization ratios can be found in Appendix E. In the determination of necessary widths and heights for each cross-section, dimensions for pressure-treated structural timber available from Bergene Holm AS have been used. These dimensions may differ from the ones obtainable in Ghana. If this is the case an increase of the cross-sectional values should be on the safe side.

Table 8-10: Required cross-sections for members supporting the roof above classroom, library and outdoor area

Member		**Cross section**
Diagonals	Outer trusses	48 mm x 198 mm
	Inner trusses	48 mm x 148 mm
Upper chord	Outer trusses	2 x 36 mm x 198 mm
	Inner trusses	2 x 36 mm x 123 mm
Lower chord	Outer trusses	2 x 36 mm x 198 mm
	Inner trusses	2 x 36 mm x 123 mm
Purlins		98 mm x 148 mm
Columns		98 mm x 98 mm

As can be seen from Table 8-10 the cross-sections for the outer trusses require taller cross-section heights than the inner trusses. Based on the design forces in Tables 8-5 and 8-6 this result is as expected, as the forces in the outer trusses are larger than in the inner trusses. Both the inner and outer trusses are designed to have double chords with the diagonals in-between the two chord elements. Figure 8-11 shows the intended truss design for the inner and outer trusses.

To avoid axial buckling in the y-direction of a single member of the upper chord, support should be provided in the mid-span reducing the buckling length by half. This applies to upper chords in both the outer and inner trusses. Likewise, the same support must be provided for the lower chords of the inner trusses. The lower chords of the outer trusses, however, must be supported at a distance 1/3 and 2/3 in the longest spans to prevent axial buckling in the y-direction of a single chord. A suggested solution for this can be seen in Figure 8-11 by filling in the gap between the chords at the given distances. The gaps can be filled with cut offs produced in the manufacture of the trusses for instance.

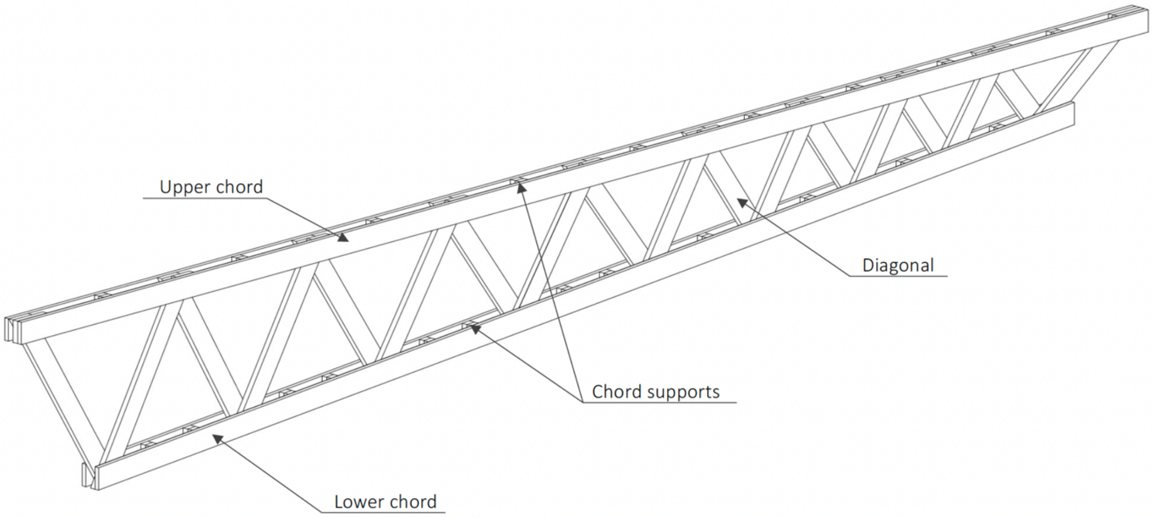

Figure 8-11: Proposed truss design with support in chords

Calculations for axial buckling with an area equal to the total cross-section of both chord elements in the upper and lower chords have been verified to have sufficient capacity to prevent axial buckling. However, by introducing the spacers in-between the chord elements, interaction between the two separate chord members is established. The chord members should therefore be considered as mechanically jointed columns in terms of axial buckling. The axial buckling capacity should be checked in accordance with Annex C in EN 1995-1-1 which applies for built-up columns. For now, the axial buckling capacity of the upper and lower chords are considered sufficient but further investigations should be carried out later in the project. The cross-sectional capacity is not influenced by the interaction of chord members and, thus, remains unchanged.

The required cross-sectional dimensions for the roof supporting structure above the accommodation and office are given in Table 8-11. The design forces acting on each member from this roof is smaller than the design forces acting on the members supporting the roof above classroom, library and outdoor area and therefore result in smaller cross-sectional areas. As a result, there is no need for double chords as a cross-sectional width equal to 48 mm is sufficient to pass the relevant design checks.

Table 8-11: Required cross-sections for members supporting the roof above the accommodation and office

Member	Cross section
Diagonal	36 mm x 98 mm
Upper chord	48 mm x 98 mm
Lower chord	48 mm x 123 mm
Purlins	48 mm x 198 mm

The purlins supporting both roofs require relatively large cross-sections when considering the values in Tables 8-10 and 8-11. The height-to-width ratios, especially for the purlins above the accommodation and office building, may lead to overturning of the purlins due to the sloping surface provided by the trusses. Additional support, in terms of brackets, may be needed to anchor the purlins to the trusses. Depending on the necessary amounts and dimensions, this solution may prove to be rather expensive compared to just using nails and screws.

An alternative solution to reduce the imposed loading on each member, may be to double the number of purlins and evenly distribute them in each span. However, although this may reduce the necessary cross-sectional dimensions for the purlins, the load application provided by the purlins in the mid-span of the upper chords is not ideal with respect to truss design. If increasing the number of purlins, the design forces and design checks for the truss components have to be re-calculated to retrieve new design values. Further examinations regarding purlin design are not carried out.

9 Foundation design

All loads acting on a structure are transported through the foundations to the ground. The design of foundations is therefore closely linked to the load bearing properties of the soil and its composition (Larsen, 2008). Downward acting compressive forces from the structure above and the foundation itself have to be distributed over a surface area at the bottom of the foundation to make sure that the pressure imposed on the ground from the foundation does not exceed the safe bearing pressure of the soil, σ_{gd} (Sørensen, 2013). In addition to the downward acting vertical forces, the upward acting vertical forces must also be anchored in the foundation. The foundation itself and the soil above must therefore be heavy enough to resist overturning or uplift. Ultimately, the foundation must be controlled to satisfy the capacity of the material properties which the foundation is made of.

In this phase of the project, an estimation of the necessary amounts of materials for the foundations with concerns to the load bearing properties of the soil and the necessary anchoring of the foundations will be investigated. No further geotechnical assessment has been done due to uncertainties regarding the ground properties. The foundations are assumed to be made of reinforced concrete as the strength properties of locally made sandcrete blocks are not predictable as described in chapter 2.2. No design of the concrete members has been carried out. A simple layout of the foundations with columns on spread footings – one in each corner of each container – have been assumed.

9.1 Material properties

9.1.1 Safe ground bearing pressure, σ_{gd}

The soil on the farm is assumed to roughly consist of 40% sand, 40% silt and 20% clay. Of these three constituents, sand is the coarsest one with grain sizes between 0,06-2 mm in diameter, clay has grain sizes smaller than 0,002 mm and silt lies somewhere between clay and fine sand (Aarhaug, 1984).

The properties of the soil depend amongst others upon the grain size of its constituents and the porosity. A larger grain size often results in higher load bearing strength. Furthermore, the porosity of the soil – the volume of pores containing water or air – describes how densely packed the constituents of the soil are. As water and air does not contribute much to the overall load bearing strength, a densely packed soil with low pore volume has higher load bearing properties than a less compact soil with larger pore volume (Aarhaug, 1984). To demonstrate this, Edvardsen and Ramstad (2014) give examples of design values for safe load bearing pressure of different soil types. Fine sand to coarse silt has σ_{gd} = 100-150 kN/m^2, loosely packed sand and silt has σ_{gd} = 50-150 kN/m^2, firm clay has σ_{gd} = 150-200 kN/m^2 and medium firm clay has σ_{gd} = 70-150 kN/m^2.

Based on the presumed composition of the soil and photos of the farm, the ground is assumed to be relatively compact throughout the year. During the drought period little water is assumed to be present in the upper layers of the soil. The water content in the soil will increase during the rainy season due to infiltration in the ground. However, the increased water content is not assumed to affect the load bearing properties greatly. Based on these assumptions, a design ground pressure equal to 150 kN/m^2 is chosen.

9.2 Calculation model

The design forces acting on the foundations are found by modelling a simplified model of each of the shipping container buildings in Robot. Three models as shown in Figure 9-1 have been created: one for the accommodation and office building, one for the classroom container and one for the library container. Each container has a simplified width x length x height equal to 2,4 m x 12,2 m x 2,9 m. Like the roof supporting structures, the foundations for the toilet containers are not calculated but will be assigned estimated amounts of material at the end of this chapter.

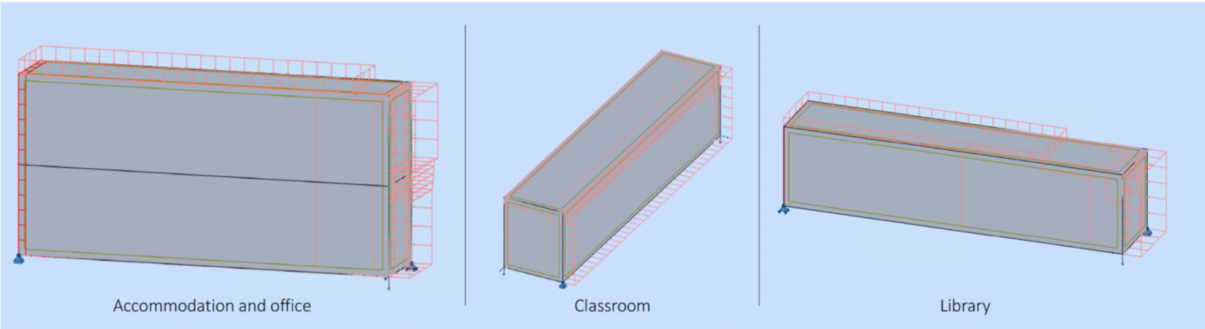

Figure 9-1: Models for support calculations showing application of wind loads from the east

The modelling of each container is done by using beam and column elements for the horizontal and vertical elements respectively. The frame of the shipping container, in addition to floor beams, are modelled to make sure that horizontal loads are distributed to each side of the bottom part of the container through the floor joists. Small steel cross-sections are chosen for the beams and columns to minimize their self-weight, as the self-weight of the container will be added as a separate load case. Claddings are added to distribute the surface loads acting on the floor, walls and roof of the real containers to the foundations. Releases for the beam and column elements have been set to default for all members. It is assumed that these elements are jointed together forming rigid connections which prevents all types of translation (UX, UY, UZ) and rotation (RX, RY, RZ).

Supports are modelled to allow the containers to expand in the horizontal plane (xy-plane) due to thermal changes and to avoid unnecessary coercive forces. All supports are modelled as pinned supports. Translations in x-, y- and z-direction have been modified to make sure that the containers are secured to the foundations but are also free to expand and shrink. Translation in z-direction have been restrained for all supports. Figure 9-2 shows the numbering of each support and the restrained translations for each container.

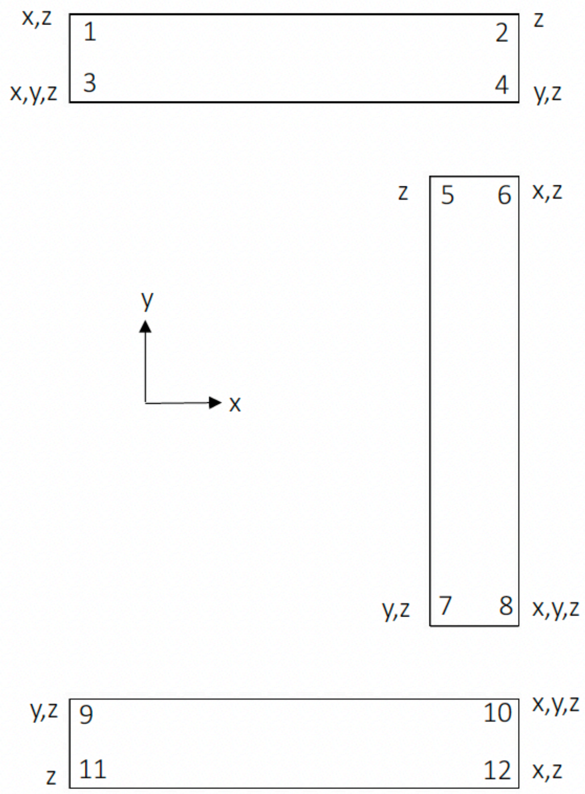

Figure 9-2: Foundation/support numbers and their restrained translations

9.3 Loads

9.3.1 Loads on Robot models

Separate load cases for dead loads, live loads and wind loads are applied to each model. In addition to these loads, the support reactions from the roof supporting structure above the containers are applied. The dead loads applied are equal to the self-weight of the containers, which is 39,4 kN per container, or 1,35 kN/m^2 if considered evenly distributed over the footprint of one container. Live loads differ depending on the use of each container. Values for each room in the planned layout are given in Table 9-1. As for the roof structure, values according to both GBC and EN 1991-1-1 are given to compare the values recommended in each code. However, to be consistent only the loads from GBC are used in the Robot models of each building.

Table 9-1: Live load values according to GBC and EN 1991-1-1.

Zone	Category according to EN 1991-1-1	GBC [kN/m^2]	EN 1991-1-1 [kN/m^2]
Classroom	C1	3,0	3,0
Library	C1	4,0	3,0
Server room	C1	3,5	3,0
Trax office	B	5,0	3,0
Accommodation	A	1,5	2,0

As the layout described in chapter 5 is only a sketch, no set division of each container into areas for each zone have been made. Therefore, the zone with the highest live load value have been used for each container. This results in a live load value equal to 3,0 kN/m^2 for the classroom container. A value of 4,0 kN/m^2 is assumed for the library, while the accommodation and office building is assigned values of 1,5 kN/m^2 and 5,0 kN/m^2 for the ground floor and first floor respectively. No live loads have been added to the roofs of the containers as these are not considered the actual roofs of the structure and have been added to the external roofs already as described in chapter 8.3.

Wind loads for each wind direction acting on each container have been calculated by the procedure given in chapter 6.1. A detailed overview of the wind load calculations, including pressure, suction and friction forces, areas of each zone and the load distribution regions can be found in Appendix D. Wind loads for both east and west have been calculated for all buildings. For wind in the south-north direction, wind from the south is considered critical for the classroom and library, with the accommodation and office building shielding for wind from the north. The opposite is assumed for the accommodation and office building, where wind from the north is considered critical, due to the classroom, library and the external roof over the outdoor area partially shielding the building for wind from the south. Reference heights equal to the building height, plus a 0,5-meter foundation height above ground is assumed for all buildings.

The support reactions from the external roof structure acting on the containers can be found in Appendix F. Several load cases for loads from the roof to the containers have been added. This is done to include the worst-case scenario for both downward acting and upward acting support forces. Two worst-case scenarios for the downward acting forces have been added to each model and corresponds to a load combination with dead loads and live loads acting on the roof structure. These load cases are named LL1 roof and LL2 roof. The upward acting forces mainly corresponds to a load combination with minimum dead load and 1,5 times the wind load. Support reactions corresponding to each wind direction have been added to separate load cases. These load cases are called WL east roof, WL west roof and WL south/north roof.

The dead loads and live loads are added as evenly distributed surface loads to the floor claddings. Wind loads are applied to the wall and roof claddings and divided into zones as previously described in chapter 6.1. The support reactions from the external roof above the containers are added as point loads acting on the beam elements on the upper part of the container frame.

9.3.2 Self-weight of foundation and soil

The self-weight of the foundation and the soil above the footing must be included in the foundation design. Hence, these loads are added to the support reactions found in Robot for each container. The foundations are assumed to be made of normal weight concrete which has a density of 24 kN/m^3 according to Appendix A in EN 1991-1-1.

An assumption has been made regarding the density of the soil. Sandy soil typically has a density of 1800 kg/m^3 while the density of silt is around 2100 kg/m^3. For clay soil the density is approximately 1900 kg/m^3 (Anupoju, n.d.). Based on these density values and the fractions of each constituent (40% sand, 40% silt, 20% clay), a mean density value between 1900-1950 kg/m^3 is expected. Thus, a density equal to 1900 kg/m^3 is assumed to be on the safe side. By multiplying the density with the gravitational acceleration (9,81 m/s^2) the density in kN/m^3 is obtained and corresponds to 18,6 kN/m^3.

9.4 Load combinations

Manual ULS load combinations are created in Robot to better control which loads are combined and which partial safety factors to be used. Partial safety factors are determined according to EN 1990 as described in chapter 7. For the loads acting from the external roof to the containers, partial factors are already included in the resulting reactions for the different load cases of the roof. Hence, a combination factor of 1,0 is chosen for all load cases with loads from the external roofs.

14 load combinations have been created for each container model. The included load cases for each combination, plus their partial factors are shown in Table 9-2. Support reactions due to wind loads on the external roof and wind loads on the containers are always included in the same combination, as the wind does not act separately on either the containers or the roof. Due to this, a partial factor equal to 1,50 is used for the wind loads on the containers for each load combination containing wind. This is thus the same partial factor used in the load combination for wind on the roof structure, as described in chapter 8.4.

For combinations according to equation 6.10a in EN 1990 the support reactions from the external roof due to live loads are equal to zero because Ψ_0 is zero. In addition, most of the vertical wind loads on the containers act in the opposite direction of the dead loads and live loads. Any of these can therefore be considered as favorable loads. This results in either the live loads or the wind loads being equal to zero, as can be seen in Table 7-1 in chapter 7. The lateral wind loads on the containers may, however, act unfavorable in combination with dead loads and/or live loads. Nevertheless, the suction forces on the roof of the containers will act favorable with respect to the foundation loads. It is therefore assumed that a combination including both dead loads, live loads and wind loads, with a partial factor of 1,35 for the dead load, is not critical. Thus, it is not included in the combinations.

Table 9-2: Manual load combinations created in Autodesk Robot for each model

LC	DL	LL	LL1 roof	LL2 roof	WL east	WL east roof	WL west	WL west roof	WL south/north	WL south/north roof
1	1,2									
2	1,2	1,5								
3	1,2		1							
4	1,2			1						
5	1,2	1,5	1							
6	1,2	1,5		1						
7	1,2	1,05			1,5	1				
8	1,2	1,05					1,5	1		
9	1,2	1,05							1,5	1
10	1,35									
11	1,35	1,05								
12	1				1,5	1				
13	1						1,5	1		
14	1								1,5	1

Depending on the load combinations providing the largest downward acting and upward acting support forces, the same partial safety factors used for dead loads in these combinations are applied to the self-weight of the concrete foundation and the soil above.

9.5 Design forces

The support reactions from the analysis done in Robot are shown in Table 9-3. F_x and F_y are lateral forces acting in the global x- and y-direction respectively. The vertical forces acting on the foundation are F_z^+ which acts in compression and F_z^- which acts as an uplifting force.

Table 9-3: Support reactions for container supports and their corresponding load combinations (LC)

Building	**Foundation**	**F_x [kN]**	**LC**	**F_y [kN]**	**LC**	**F_z^+ [kN]**	**LC**	**F_z^- [kN]**	**LC**
Accommodation and office	1	26,9	13			106,3	5	200,2	14
	2					106,3	5	190,9	14
	3	26,9	13	114,7	14	140,8	5	37,6	13
	4			114,6	14	149,8	5	28,4	12
Classroom	5					91,8	5	159,0	13
	6	71,2	12			45,4	2	69,0	12
	7			18,2	14	78,3	5	114,7	13
	8	61,7	12	10,9	14	45,4	2	58,6	12
Library	9			45,4	14	68,1	6	37,9	12
	10	16,3	13	45,2	14	68,2	6	40,8	12
	11					68,1	5	83,6	14
	12	16,0	13			67,8	5	80,5	14

From Table 9-3 it can be seen that it is the load combinations including dead loads and live loads that create critical F_z^+ forces, while it is the load combinations containing dead loads and wind loads that give the largest F_z^- forces.

For F_z^- forces, the load combinations containing wind perpendicular to the side walls of the containers produce the biggest uplifting forces on the foundations. The large surface of the side walls results in larger resultant forces compared to when the wind acts perpendicular to the end walls of the container. These large lateral resultant forces act in the same direction, trying to tip the container. To avoid overturning of the container the support reactions act as a force pair to resist the moment created by the wind, as there are no moment resisting connections between the containers and foundations to deal with this. The force pair is created by having two forces acting in opposite directions with a distance between them, acting as a lever arm. If the lever arm is long, the forces that make up the force pair will have to be smaller than if the lever arm is shorter. The relatively short lever arm which is the case in this situation is what makes the uplifting forces in the supports quite large.

The horizontal F_x an F_y forces are a result of the lateral wind loads due to suction and pressure on the walls in combination with the friction forces due to wind. When considering a foundation like spread footings, the horizontal forces from the container acting on the columns supporting it, result in moments at the base of the columns. These moments have to be transferred to the foundation footing.

The columns are assumed to be casted into the footings and a moment resisting connection between these two elements can therefore be assumed. The column is not assumed to be restrained against lateral translations at its top and the moment transferred at the base of the column to the footing is therefore equal to P·L. Where P is the horizontal point load, i.e., F_x or F_y, and L is the length of the column. For simplicity, the soil surrounding the column is neglected in the calculation of design moments at the column base. Figure 9-3 shows the static model of the foundation.

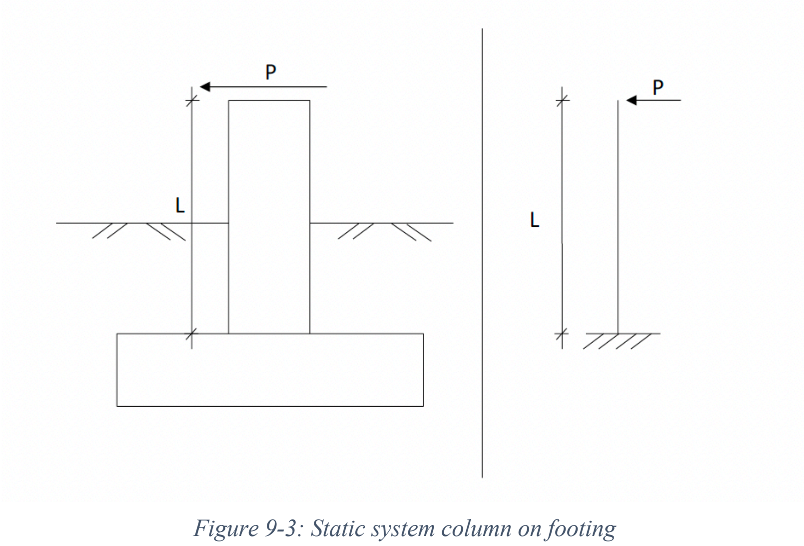

Figure 9-3: Static system column on footing

The design values in Table 9-3 for the support reactions due to loads on the containers are used in combination with the self-weight of the foundation and soil to determine the necessary foundation depths and areas. The moments at the column base depend on the length of the column. Furthermore, the self-weight of the foundation and soil above the footing depend on the size and depth of the foundation. Hence, the design loads change with the foundation design. To determine the necessary foundation depth and size, a spread sheet of the design calculations have been created in Excel. By doing this, the column moments and weight of foundation and soil are re-calculated automatically as the geometrical foundation parameters are changed. Partial safety factors equal to 1,20 for the downward acting forces and 1,0 for the upward acting forces, have been applied to the self-weight of the concrete and soil to obtain the design values of these contributions.

9.6 Design of foundations

The necessary foundation surface to distribute the compressive forces to the ground without exceeding the safe ground bearing pressure is found by using methods described by Sørensen (2013). Depending on whether the foundation is exposed to vertical forces only, or if lateral forces act in combination with the vertical forces, the necessary load distribution area must be determined. Foundations exposed to vertical forces only, are typically made as symmetrical foundations. Due to economic and environmental considerations, foundations exposed to both vertical forces and moments usually are made as asymmetrical foundations (Sørensen, 2013). Figure 9-4 shows typical layouts for foundations with either vertical forces only, or vertical forces and moments and the corresponding distribution of forces to the ground.

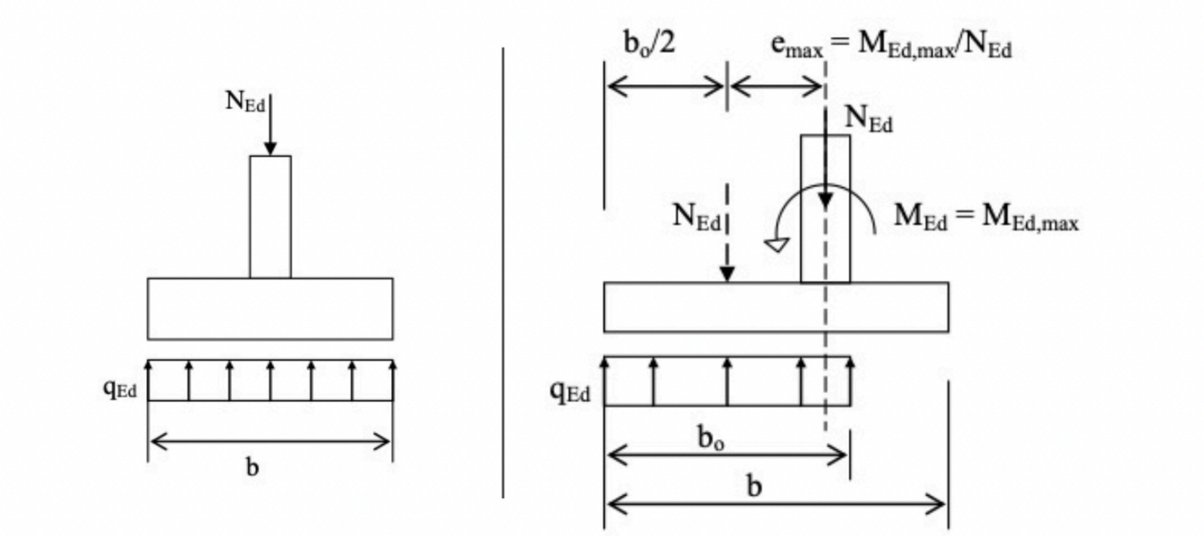

Figure 9-4: Spread footings subjected to vertical forces only (left) and both vertical forces and moment (right). Source: (Sørensen, 2013)

The necessary width of a foundation exposed to vertical forces only requires a minimum effective foundation width, b_0, equal to:

$$b_0 = \sqrt{\frac{N_{Ed1}}{\sigma_{gd}}}$$

Where:

b_0 Effective foundation width

N_{Ed1} Design force acting on the foundation in the vertical direction

Foundation widths for foundations exposed to moments in one or two directions are calculated by assuming that the vertical load acts with an eccentricity in either x- or y-direction.

$$e_{x,max} = \frac{M_{Edx,max}}{N_{Ed1}} \quad \text{and} \quad e_{y,max} = \frac{M_{Edy,max}}{N_{Ed1}}$$

Where:

$e_{x,max}$	Eccentricity in x-direction
$M_{Edx,max}$	Maximum moment acting in the x-direction
$e_{y,max}$	Eccentricity in y-direction
$M_{Edy,max}$	Maximum moment acting in the y-direction

This eccentricity is then added to the effective width, b_0, to obtain the necessary width in x- and y-direction:

$$b_x = b_0 + e_{x,max} \quad \text{and} \quad b_y = b_0 + e_{y,max}$$

Where:

b_x	Necessary foundation width in x-direction
b_y	Necessary foundation width in y-direction

Figure 9-5 shows the different load distribution areas depending on the loads acting on the foundation.

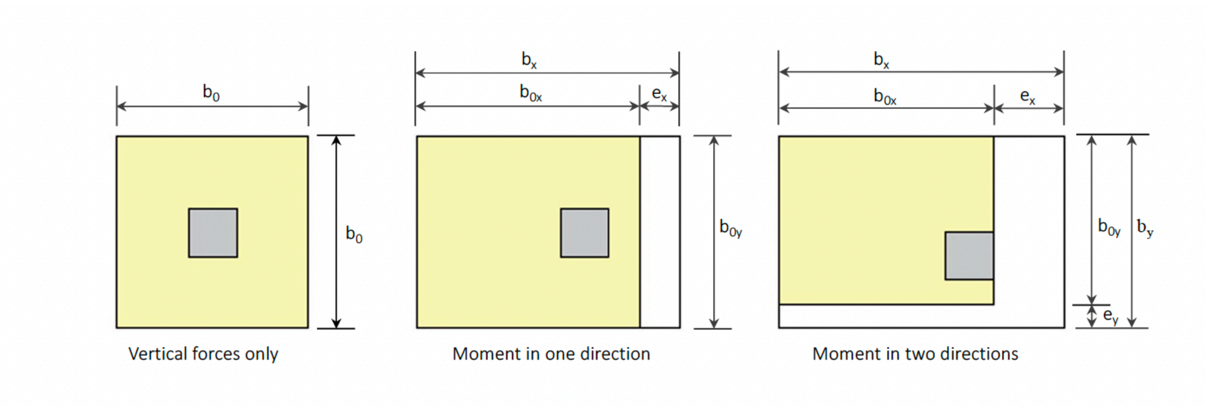

Figure 9-5: Load areas (yellow) for foundations subjected to either vertical forces, or a combination of vertical forces and moments

In the determination of b_0 the weight of the foundation and the soil above is not included. This means that the foundation width must be bigger than b_0, as this value is the minimum necessary width for the foundation to distribute the forces acting from the container to the foundation. Hence, b_0 does not include the added forces due to the self-weight of the foundation and soil. In the calculations carried out, b_0 is therefore called $b_{0,min.}$ Values for b_0 in the x- and y-directions, b_{0x} and b_{0y} respectively, are made bigger than $b_{0,min}$ to make sure that the load distribution from the forces acting on the foundation and the weight of the foundation and soil does not exceed σ_{gd}. Thus, the design check for the load bearing properties of the soil becomes:

$$q_{Ed} = \frac{N_{Ed1}}{A_0} + \frac{N_{Ed2}}{A} \leq \sigma_{gd}$$

Where:

A_0	Effective area for load distribution for forces acting on the foundation
A	Total area of the foundation
N_{Ed2}	Self-weight of foundation and soil above foundation

In the final design check above, N_{Ed1} is assumed to act on a load surface, A_0, equal to $b_{0x} \cdot b_{0y}$. Furthermore, N_{Ed2} is assumed to be evenly distributed over the entire bottom surface of the foundation footing, A, equal to $b_x \cdot b_y$.

The connection between shipping container and foundation must be anchored into the foundation to make sure that the uplifting forces from the support reactions does not cause tipping of the container. As a result of this, the foundation must in turn be held down by its self-weight and the weight of the soil above its base plate to prevent uplifting of the foundation as a whole.

The connection between container and foundation must withstand the uplifting load acting from the container on the foundation. Some kind of anchorage of the connection into the column of the foundation must therefore be ensured. To avoid the container from tipping, due to the lateral forces acting on the container, the weight of the foundation in combination with the weight of the soil above the foundation must be bigger than the uplifting force. The necessary volumes of concrete and soil to prevent uplift of the foundation have been calculated by the following relation:

$$\gamma_{concrete} \cdot V_{concrete} + \gamma_{soil} \cdot V_{soil} \geq F_z^-$$

Where:

$\gamma_{concrete}$	Density of concrete
γ_{soil}	Density of soil
$V_{concrete}$	Volume of concrete (footing + column)
V_{soil}	Volume of soil above footing

9.7 Required foundation sizes

Table 9-4 gives the required foundation sizes for distribution of compressive forces to the ground when using spread footings. The necessary volume of concrete to anchor the foundations are also given in the table. A column size of 0,35 m x 0,35 m is assumed for all foundations. A foundation thickness, h_f, equal to 0,5 m and a total foundation depth, L_f, equal to 1 meter applies to all foundations. Complete calculations can be found in Appendix G.

Table 9-4: Necessary foundation areas and volumes to distribute forces to the ground and preventing uplift.

Building	Foundation	Compression				Uplifting
		b_x [m]	b_y [m]	A [m^2]	V [m^3]	V [m^3]
Accommodation and office	1	1,20	0,95	1,14	0,69	8,34
	2	0,95	0,95	0,90	0,57	7,95
	3	1,30	1,85	2,41	1,33	1,56
	4	1,15	1,85	2,13	1,19	3,82
Classroom	5	0,90	0,90	0,81	0,53	6,63
	6	2,20	0,65	1,43	0,84	2,87
	7	0,85	1,00	0,85	0,55	4,78
	8	2,00	0,85	1,70	0,97	2,44
Library	9	0,80	1,40	1,12	0,68	1,58
	10	1,00	1,40	1,40	0,82	1,70
	11	0,80	0,80	0,64	0,44	3,48
	12	1,00	0,75	0,75	0,50	3,35

As described in chapter 2.1.3, spread footings are a suitable foundation type if the total foundation area is less than 50% of the building's footprint. With a building footprint of 29,3 m^2 per container, Table 9-4 shows that the necessary foundation area to distribute the compressive forces to the ground without exceeding σ_{gd} is well below those 50%. However, when comparing the volumes of concrete needed for the anchoring of each foundation to the volume of concrete needed for compressive load distribution, quite large differences can be observed. If assuming a foundation thickness equal to 1 meter with no soil above, the total necessary areas to withstand the uplifting forces exceeds 50% of the building footprint for both the accommodation and office building and the classroom.

The original plan of supporting the containers with simple spread footing foundations may have to be reconsidered. Adding more supports pointwise along each sidewall is not assumed to reduce the large uplifting forces or the horizontal support reactions significantly. Most of the lateral wind loads acting on the sidewalls are assumed to be resisted by the end walls. Thus, the support reactions for each corner foundation will be large regardless of whether more supports are added to the sidewalls or not. This assumption is based on research done by Giriunas et al. (2012), which found that the roof of a shipping container contributes little to the lateral resistance of the container for lateral loads on either side walls or end walls. Moreover, for the accommodation and office building forces are transferred from the first-floor container to the ground-floor container through their corner fittings. These forces are therefore assumed to be allocated to the corner foundations only, and not to any of the extra supports along the side walls.

As mentioned in chapter 2.1.3, the most common foundation type for container buildings, apart from the spread footing foundation, is mat foundations. If assuming that all the containers are supported by mat foundations which extends 0,5-meters beyond the building footprint in all directions, a concrete slab with a 0,5-meter thickness has a self-weight of approximately 538 kN. This is enough to resist the total uplifting forces from the accommodation and office building which comes at about 460 kN for the relevant load combinations. The total uplifting forces from the classroom container are approximately 401 kN, while for the library container the uplifting forces are roughly 243 kN. A shared mat foundation for the classroom and library containers with a thickness of 0,35 meter has a self-weight of approximately 725 kN which is enough to prevent the foundation from lifting.

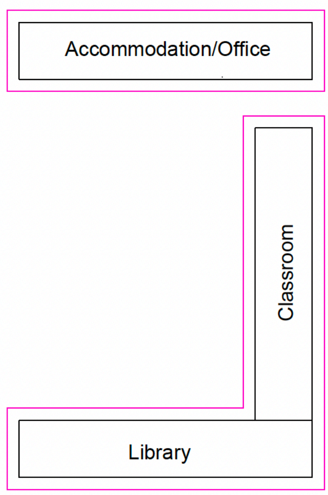

Figure 9-6: Proposed layout for mat foundations marked in pink

The alternative solution with mat foundations requires a larger volume of concrete than using spread footings. However, for the accommodation building a mat foundation as described above requires 22,44 m^3 concrete, while the total amount of concrete needed to anchor the spread footings is 21,67 m^3, which is less than 1 m^3 difference. For the classroom and library, a mat foundation requires 30,23 m^3 concrete, while the total amount of concrete needed for spread footings for both containers is 26,83 m^3, which gives a bigger difference than for the accommodation and office building. Nonetheless, a mat foundation will require less deep digging and may therefore save time in the construction phase. Another desirable feature of the mat foundations is the reduced height between the ground and door sill, making the need for steps up to the entrance doors unnecessary. Ramps for wheelchairs will also need to be shorter as the height of the gap between the ground and door sill decreases.

The 20' toilet containers are shorter in length than the other containers, thus smaller compressive and uplifting forces are expected to develop at their supports. However, in line with the rest of the containers, mat foundations are assumed for the toilet containers. A necessary thickness of 0,3 m is assumed to provide sufficient height for concrete cover and reinforcement.

10 Discussion

During the design process some thoughts have been made concerning the design choices of the proposed layout which should be further evaluated in the next phase of the project.

Most of the design forces for both the roof supporting structures and the foundations are a result of large wind loads. The large wind loads are a result of the high basic wind velocity used in the determination of the peak velocity pressure. The deviation in wind velocities between the basic wind velocity (35 m/s) and the average wind speed (4-5 m/s) may give reason to assume that a reduction of the basic wind velocity is possible. Thus, smaller design forces may be obtained for both the roof supporting structures and the foundations. For comparison, the basic wind velocity for the most weather exposed places in Norway are 31 m/s according to the National Annex in EN 1991-1-4.

However, due to the open landscape and the relative proximity to the Sahara Desert, the local conditions may be as extreme as the basic wind velocity suggests. Due to travel restrictions caused by Covid-19, no field work has been possible to carry out which would have been useful in terms of a better understanding of the local conditions on site. To check whether or not the basic wind velocity may be reduced, wind measurements on site according to standardized procedures can be conducted. For now, however, the design loads and necessary quantities of materials, particularly for foundations, are determined in accordance with the recommendations in GBC and should be on the safe side.

If a reduction of wind loads is proven to be acceptable for the design, spread footing foundations may be a good option for foundations after all. Like the results in chapter 9.7 suggests, the load bearing properties of the soil are satisfactory in terms of compressive forces. If the reduction in uplifting forces is large enough, anchorage of the foundations due to a reasonable foundation depth and size may be achieved.

Not included in this thesis, but an important factor to mention, are considerations regarding the load bearing properties of the shipping containers when altered from their original design. As described in chapter 2.1.1, shipping containers have sufficient load bearing capacity to stack 12 fully loaded containers on top of each other. However, when cutting out holes for windows and doors the load bearing properties are assumed to be reduced. Some reinforcement of the containers may therefore be necessary. An investigation regarding the load bearing properties of the container walls and optimal placement and size of openings to reduce the amount of reinforcement would be beneficial from a cost-wise point of view. This has not been done in this thesis as more urgent design calculations regarding the external roofs and foundations were needed for this phase of the project. In addition, the load bearing properties of the containers supporting the external roofs should be considered as these loads act outside the containers' corner fittings. If expensive reinforcement of the containers is needed to support the roof structure, columns can be placed on each side of the classroom and library containers, altering the layout of the roof supporting structure slightly. If mat foundations are used, foundations for these added columns will already be provided.

Depending on the connections used in the timber elements of the roof supporting structures, the net area, A_{net}, may be smaller than assumed in the calculations carried out. As a consequence, some of the elements where tensile forces determine the necessary cross-sectional dimensions, may not satisfy the design requirements for the recommended cross-sections found in this thesis. An increase of either width or height is therefore necessary. Evaluations with respect to member stability, including geometrical and practical concerns regarding the design should be done to determine whether the width or heigh should be increased.

11 Conclusion

The suggested layout presented in this thesis covers the most basic needs of the project. Teaching facilities in terms of a classroom and library provide a space for educational purposes, offering a place to finish the basic education for dropout students or to facilitate for complimentary education in addition to the basic education. Separated hygiene facilities for girls and boys, in addition to accommodation housing either girls or boys at the same time, have been prioritized to ensure a feeling of security when staying at the Trax-Kavli farm.

Using shipping containers as a basis for this project have proven to provide several advantages. Firstly, a fast construction process is ensured when using shipping containers. Secondly, waterproofing of the outer surfaces of the buildings are already provided by the container walls. Furthermore, considering the local conditions on the farm, the walls of a steel container are not washed away like earthen structures are during the rainy season. Thus, little maintenance is required. In addition, termite related problems are not a big concern when using shipping containers.

Despite the advantages shipping container buildings provide, with respect to durability, special considerations must be made in terms of the thermal comfort inside shipping container houses. Passive measures to avoid excessive heating inside the containers have therefore been included in the design by providing shade and painting the containers in a light color. External roofs provide shade and surfaces for rainwater harvesting, increasing the water supply on the farm. Furthermore, the external shading provides an outdoor area shielded from the sun where larger groups can gather. The indoor temperatures of the containers have been verified to correspond to the ambient temperature, exceeding 40ºC during the hottest months.

Locally procured timber has been considered an appropriate material for the roof supporting structures. In the design of the roof supporting structures, truss beams supported by columns and the containers are considered the most reasonable solution due to the long span. The recommended element sizes given in chapter 8.7 and Appendix A are assumed sufficient to support the roofs. However, design using updated A_{net} values and design checks for mechanically jointed columns should be carried out in the next phase of the project.

Due to the high basic wind velocities in the area, mat foundations are considered the best solution for foundations by findings presented in chapter 9.7. Thus, changing the original plan of having spread footing foundations. Investigations in terms of the actual wind conditions on the Trax-Kavli farm may give reduced support reactions, making it possible to consider spread footing foundations later on. However, until such investigations are carried out, the suggested mat foundations in this thesis are recommended.

12 Further work

A list of bullet points describing some aspects of the proposed design which can be considered in the next phase is given below:

- Design checks of shipping containers with cut outs and support reactions from external roofs.
- Investigation regarding the basic wind velocity.
- Design of concrete foundations.
- Design of connections.
- Updated design of timber members taking connections into account.
- Further enhancement of the indoor climate.
- Noise-related investigations inside the containers.
- Individual design of toilet containers not based on assumptions like have been done in this thesis.

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Appendix

Appendix A: List of materials

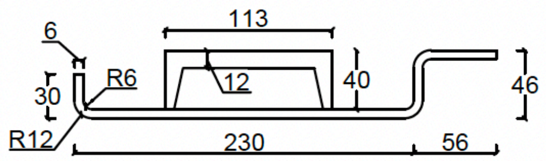
List of materials - buildings

Component		Material	Specification	Amount
Container	40 ft. HC	Steel		4
	20 ft. HC/standard			2
Roof	Outdoor area	Corrugated metal sheet	14,2 m x 16,6 m	239 m^2
	Accommodation/Office		4,4 m x 14,2 m	63 m^2
	Toilets		2 x 4,4 m x 7,1 m	63 m^2
Gutters along roof edge	Outdoor area			16,6 m
	Accommodation/Office			4,4 m
	Toilets			14 m
Roof support	Diagonal	Treated wood, C24	36 mm x 98 mm	62 m
	Upper/lower chord		36 mm x 123 mm	327 m
	Upper/lower chord		36 mm x 198 mm	109 m
	Upper chord		48 mm x 98 mm	30 m
	Lower chord		48 mm x 123 mm	26 m
	Diagonal		48 mm x 148 mm	185 m
	Diagonal/Purlin		48 mm x 198 mm	106 m
	Column		98 mm x 98 mm	18 m
	Purlin		98 mm x 148 mm	166 m
	Bracing		Not specified	150 m
	Connections	Galvanized steel	19/truss	228
Foundations	Classroom/library	Reinforced concrete	86,36 m^2 x 0,35 m	31 m^3
	Accommodation/Office		44,88 m^2 x 0,5 m	23 m^3
	Toilets		2 x 24 m^2 x 0,3 m	15 m^3
New flooring/ encapsuling of existing floors	Classroom	Not specified	2,4 m x 12,2 m	29 m^2
	Library		2,4 m x 12,2 m	29 m^2
	Accommodation/Office		2 x 2,4 m x 12,2 m	58 m^2
	Toilets		2 x 2,4 m x 6,1 m	29 m^2
Interior framing	Classroom	Treated wood/Metal	Framing	200 m
		Gypsum board/Other	Paneling	102 m^2
	Library	Treated wood/Metal	Framing	200 m
		Gypsum board/Other	Paneling	102 m^2
	Accommodation/Office	Treated wood/Metal	Framing	400 m
		Gypsum board/Other	Paneling	204 m^2
	Toilets	Treated wood/Metal	Framing	179 m
		Gypsum board/Other	Paneling	114 m^2
White paint	Classroom	Outdoor paint container	Surface area	114 m^2
	Library			114 m^2
	Accommodation/Office			199 m^2
	Toilets			128 m^2

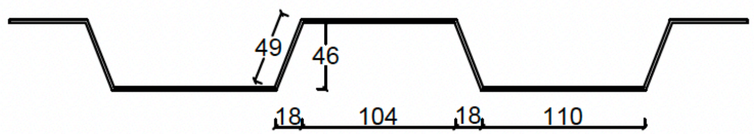
Doors	External	Classroom/Library	0,86 m free space	2
		Accommodation/Office	Not specified	2
	Internal	Classroom/Library	0,86 m free space	2
		Accommodation/Office	Not specified	4
Windows	Classroom	Metal or treated timber frame. Low U-value	0,6 m x 0,6 m	8
	Library		0,6 m x 0,6 m	9
	Accommodation/Office		0,6 m x 0,6 m	18
Staircase	Accommodation/Office	Weather resistant		1
Lightning rods			Consult with local agent	
Connections	Foundation/container			20
Fasteners				RS

Appendix B: Shipping container cross sections

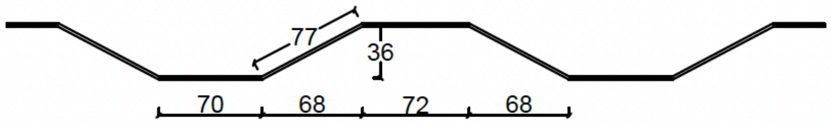
Door corner post



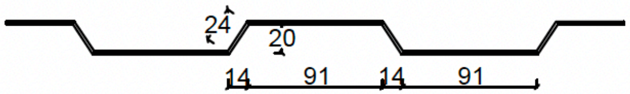
Front wall

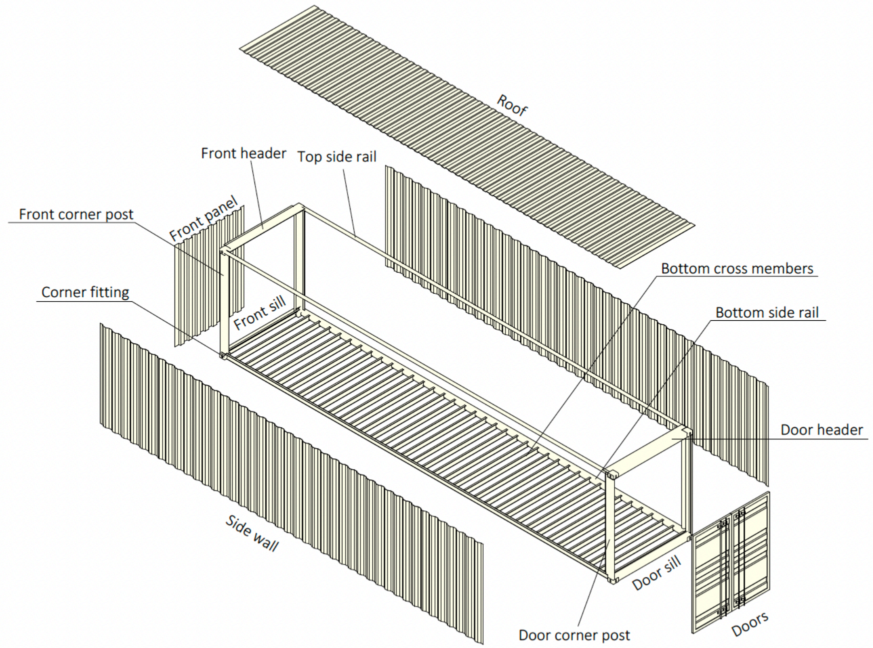


Side wall

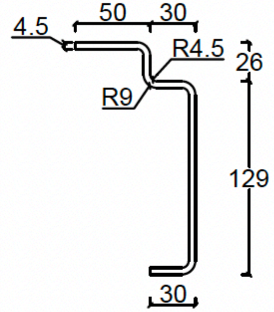


Roof

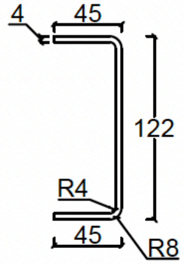




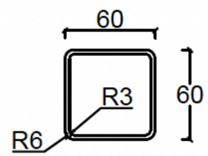
Bottom cross member



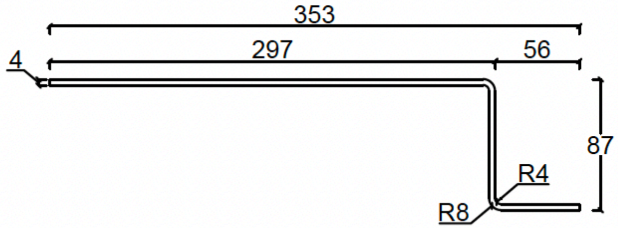
Bottom side rail



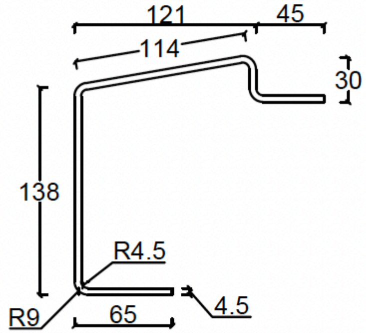
Front sill & top side rail



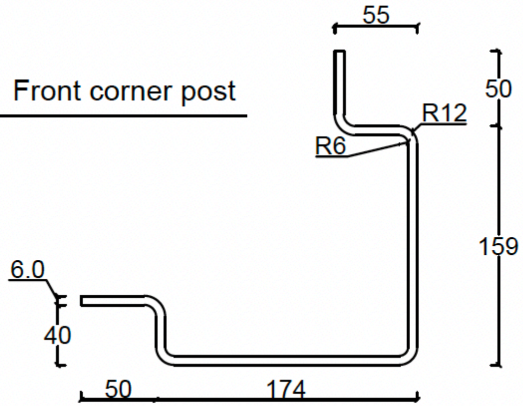
Front header



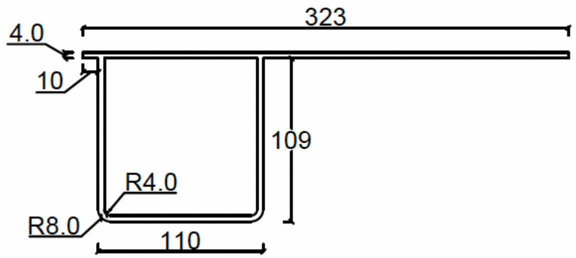
Door sill



Front corner post



Door header



Appendix C: Numerical simulation of indoor temperature

Numerical simulation of indoor temperature in IDA ICE

Input

Climate file: Ouagadougou in Burkina Faso.
Climate file for the town closest to Bolgatanga with the most similar climate conditions

Wind profile: Open country

Simulation model: 1 container with windows, doors, roof and air gap between roof and container. No insulation.
Windows: 2 pane glazing windows
Windows and doors are set to always open
Thermal conductivity for air gap: $\lambda = 0{,}025$ W/mK

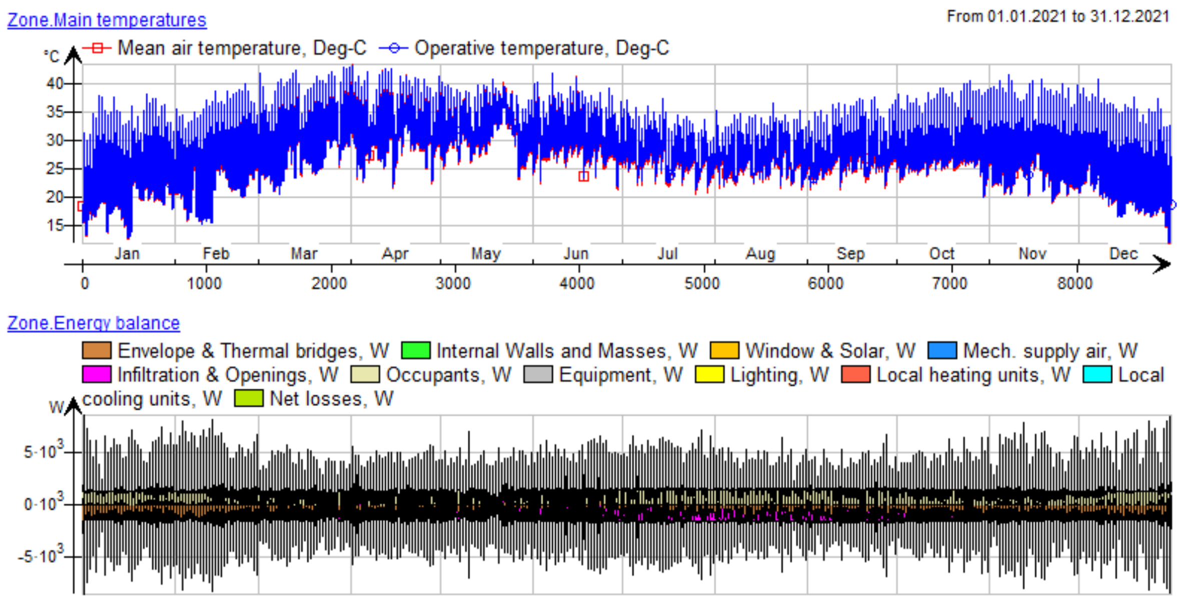


Figure 1: Indoor temperatures throughout the year for the simulation described above.

Results

Maximum operative temp: 43,08°C

Minimum operative temp: 12,13°C

RH maximum: 100%

RH minimum: 5%

Simulations with interior insulation were carried out as well. With reasonable thicknesses to not take up too much of the interior space the maximum operative temperatures were reduced with 2°C, while the minimum operative temperatures were increased with 2°C.

Appendix D: Wind load calculations

Wind loads - external roof over outdoor area

Common parameters for all wind directions:

Forces perpendicular to roof surface

Reference height, z_e:	5	m
Peak velocity pressure, q_p:	1,48	kN/m^2

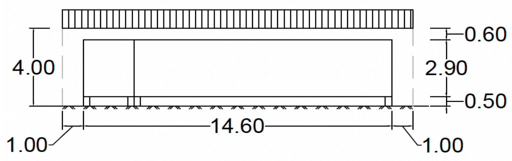
Forces parallel to roof surface

Reference height, ze:	0,10	m
Peak velocity pressure, q_p:	0,15	kN/m^2
Reference area friction, A_{fr}:	471,40	m^2

Wind from east

Roof angle, α:	4,0	°
Blockage, φ:	0,69	

	Net pressure coefficient		Wind pressure [kN/m^2]	
Zone A	$c_{p,net,A}$	-1,40	$w_{k,A}$	-2,07
Zone B	$c_{p,net,B}$	-1,97	$w_{k,B}$	-2,90
Zone C	$c_{p,net,C}$	-2,22	$w_{k,C}$	-3,27

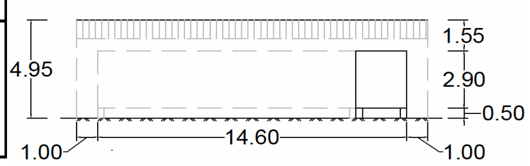


Friction coefficient, c_{fr}:	0,01	
Perimeter length, l_{fr}:	16,60	m
Friction forces, w_{fr}:	0,04	kN/m

Wind from west

Roof angle, α:	4,0	°
Blockage, φ:	0,08	

	Net pressure coefficient		Wind pressure [kN/m^2]	
Zone A	$c_{p,net,A}$	-1,05	$w_{k,A}$	-1,55
Zone B	$c_{p,net,B}$	-1,66	$w_{k,B}$	-2,45
Zone C	$c_{p,net,C}$	-1,78	$w_{k,C}$	-2,63

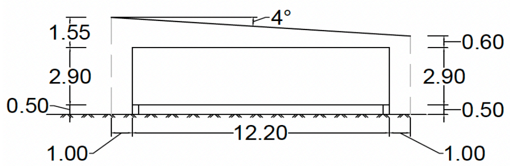


Friction coefficient, c_{fr}:	0,01	
Perimeter length, l_{fr}:	16,60	m
Friction forces, w_{fr}:	0,04	kN/m

Wind from south

Roof angle, α:	0,0	°
Blockage, φ:	0,56	

	Net pressure coefficient		Wind pressure [kN/m^2]	
Zone A	$c_{p,net,A}$	-1,10	$w_{k,A}$	-1,65
Zone B	$c_{p,net,B}$	-1,58	$w_{k,B}$	-2,38
Zone C	$c_{p,net,C}$	-1,85	$w_{k,C}$	-2,78



Friction coefficient, c_{fr}:	0,04	
Perimeter length, l_{fr}:	14,20	m
Friction forces, w_{fr}:	0,20	kN/m

Wind loads - external roof over accommodation and office building

Common parameters for all wind directions:

Forces perpendicular to roof surface

Reference height, z_e:	7,85	m
Peak velocity pressure, q_p:	1,68	kN/m^2

Forces parallel to roof surface

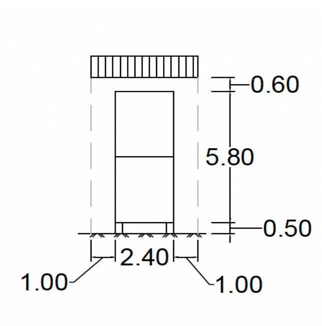
Reference height, ze:	0,10	m
Peak velocity pressure, q_p:	0,15	kN/m^2
Reference area friction, A_{fr}:	124,96	m^2

Wind from east

Roof angle, α: 4,0 °

Blockage, φ: 0,46

	Net pressure coefficient		Wind pressure [kN/m^2]	
Zone A	$c_{p,net,A}$	-1,27	$w_{k,A}$	-2,14
Zone B	$c_{p,net,B}$	-1,85	$w_{k,B}$	-3,12
Zone C	$c_{p,net,C}$	-2,05	$w_{k,C}$	-3,45



Friction coefficient, c_{fr}: 0,01

Perimeter length, l_{fr}: 4,40 m

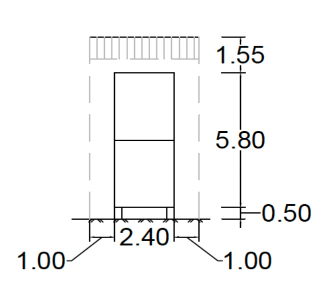
Friction forces, w_{fr}: 0,04 kN/m

Wind from west

Roof angle, α: 4,0 °

Blockage, φ: 0,40

	Net pressure coefficient		Wind pressure [kN/m^2]	
Zone A	$c_{p,net,A}$	-1,23	$w_{k,A}$	-2,07
Zone B	$c_{p,net,B}$	-1,82	$w_{k,B}$	-3,07
Zone C	$c_{p,net,C}$	-2,01	$w_{k,C}$	-3,39



Friction coefficient, c_{fr}: 0,01

Perimeter length, l_{fr}: 4,40 m

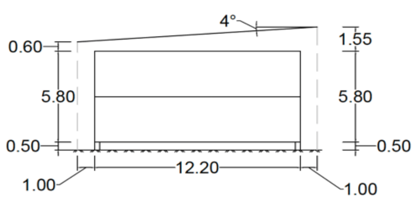
Friction forces, w_{fr}: 0,04 kN/m

Wind from south/north

Roof angle, α: 0,0 °

Blockage, φ: 0,68

	Net pressure coefficient		Wind pressure [kN/m^2]	
Zone A	$c_{p,net,A}$	-1,21	$w_{k,A}$	-2,04
Zone B	$c_{p,net,B}$	-1,64	$w_{k,B}$	-2,76
Zone C	$c_{p,net,C}$	-1,94	$w_{k,C}$	-3,27



Friction coefficient, c_{fr}: 0,04

Perimeter length, l_{fr}: 4,40 m

Friction forces, w_{fr}: 0,05 kN/m

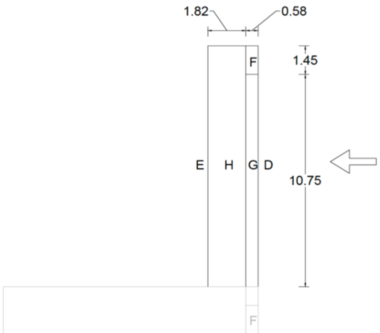
Wind loads - classroom container

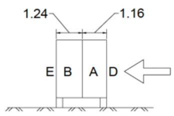
Common parameters for all wind directions:

Reference height, z_e:	3,40	m
Peak velocity pressure, q_p:	1,31	kN/m^2
Building height, h_b:	2,90	m

Wind from east

Depth of building, d:	2,40	m
Breadth of building, b:	14,60	m
Height-to-depth ratio, h_b/d:	1,21	
Edge distance, e:	5,80	m





Forces perpendicular to roof surface

Opening ratio, μ: 0,70

Internal pressure coefficient, c_{pi}: -0,15

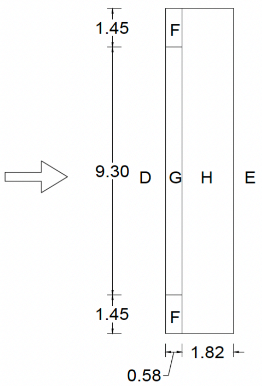
Zone	A	B	D	E	F	G	H
Wind pressure [kN/m^2]	-1,37	-0,85	1,24	-0,47	-2,16	-1,37	-0,72

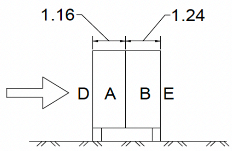
Forces parallel to roof surface

Reference area friction, A_{fr}: N.A.

Wind from west

Depth of building, d:	2,40	m
Breadth of building, b:	12,20	m
Height-to-depth ratio, h_b/d:	1,21	
Edge distance, e:	5,80	m





Forces perpendicular to roof surface

Opening ratio, μ: 0,30

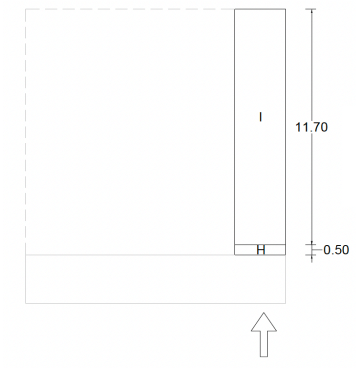
Internal pressure coefficient, c_{pi}: 0,35

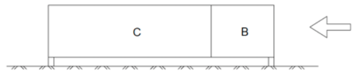
Zone	A	B	D	E	F	G	H
Wind pressure [kN/m^2]	-2,03	-1,50	0,59	-1,13	-2,81	-2,03	-1,37

Forces parallel to roof surface

Reference area friction, A_{fr}: N.A.

Wind from south





Depth of building, d:	14,60	m
Breadth of building, b:	12,20	m
Height-to-depth ratio, h_b/d:	0,20	
Edge distance, e:	5,80	m

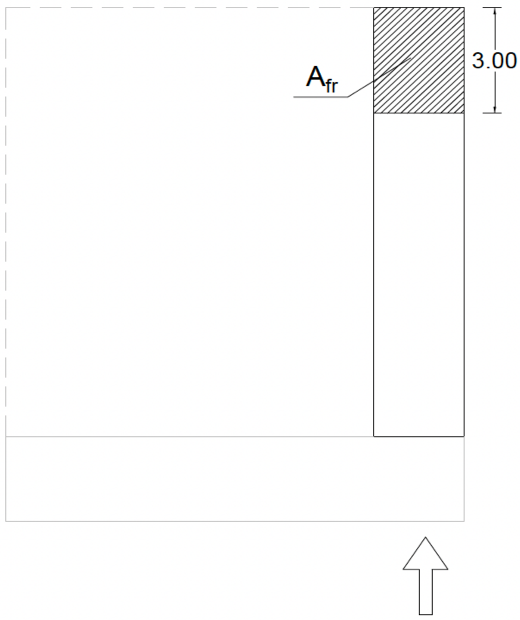
<u>Forces perpendicular to roof surface</u>

Opening ratio, μ:	1,00	
Internal pressure coefficient, c_{pi}:	-0,30	

Zone	B	C	E	H	I
Wind pressure [kN/m^2]	-0,65	-0,26	0,00	-0,52	0,65

<u>Forces parallel to roof surface</u>

Reference area friction, A_{fr}:	24,60	m^2
Friction coefficient, c_{fr}:	0,04	
Perimeter length, l_{fr}:	8,20	m
Friction forces, w_{fr}:	0,16	kN/m



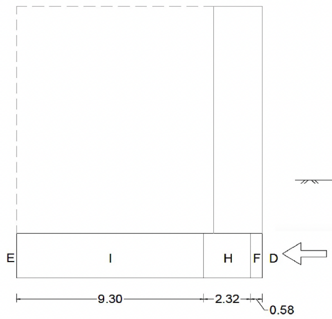
Wind loads - library container

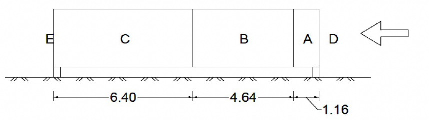
Common parameters for all wind directions:

Reference height, z_e:	3,40	m
Peak velocity pressure, q_p:	1,31	kN/m^2
Building height, h_b:	2,90	m

Wind from east

Depth of building, d:	12,20	m
Breadth of building, b:	14,60	m
Height-to-depth ratio, h_b/d:	0,24	
Edge distance, e:	5,80	m





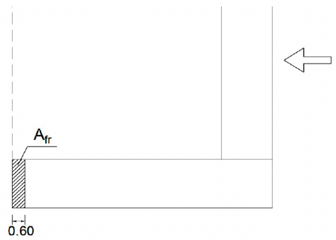
Forces perpendicular to roof surface

Opening ratio, μ:	0,93	
Internal pressure coefficient, c_{pi}:	-0,30	

Zone	A	B	C	D	E	F	G	H	I
Wind pressure [kN/m^2]	-1,18	-0,65	-0,26	1,31	0,00	-1,96	-1,18	-0,52	0,65

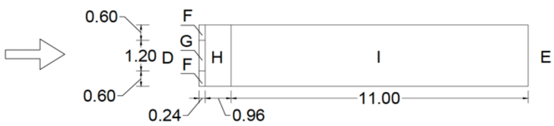
Forces parallel to roof surface

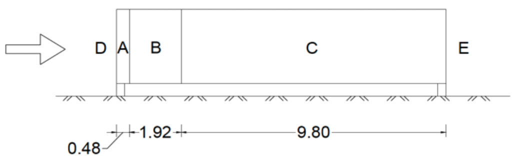
Reference area friction, A_{fr}:	4,92	m^2
Friction coefficient, c_{fr}:	0,04	
Perimeter length, l_{fr}:	8,20	m
Friction forces, w_{fr}:	0,03	kN/m



Wind from west

Depth of building, d:	12,20	m
Breadth of building, b:	2,40	m
Height-to-depth ratio, h_b/d:	0,24	
Edge distance, e:	2,40	m





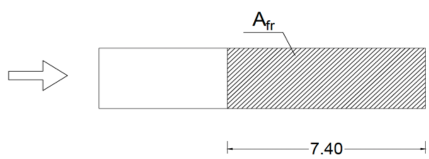
Forces perpendicular to roof surface

Opening ratio, μ:	1,00	
Internal pressure coefficient, c_{pi}:	-0,30	

Zone	A	B	C	D1	D2	E	F	G	H	I
Wind pressure [kN/m^2]	-1,18	-0,65	-0,26	1,24	1,31	0,00	-1,96	-1,18	-0,52	0,65

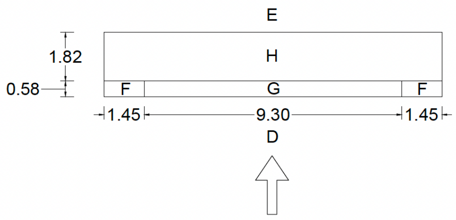
Forces parallel to roof surface

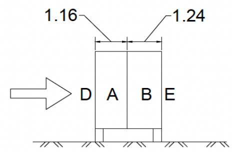
Reference area friction, A_{fr}:	60,68	m^2
Friction coefficient, c_{fr}:	0,04	
Perimeter length, l_{fr}:	8,20	m
Friction forces, w_{fr}:	0,39	kN/m



Wind from south

Depth of building, d:	2,40	m
Breadth of building, b:	12,20	m
Height-to-depth ratio, h_b/d:	1,21	
Edge distance, e:	5,80	m





<u>Forces perpendicular to roof surface</u>

Opening ratio, μ:	0,73
Internal pressure coefficient, c_{pi}:	-0,20

Zone	A	B	D	E	F	G	H
Wind pressure [kN/m^2]	-1,31	-0,79	1,31	-0,41	-2,09	-1,31	-0,65

<u>Forces parallel to roof surface</u>

Reference area friction, A_{fr}:	N.A.

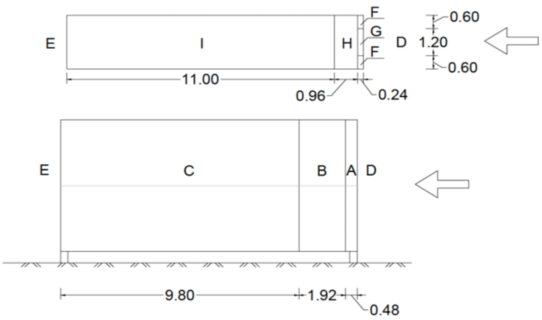
Wind loads - accommodation and office containers

Common parameters for all wind directions:

Reference height, z_e:	6,30 m
Peak velocity pressure, q_p:	1,58 kN/m^2
Building height, h_b:	5,80 m

Wind from east

Depth of building, d:	12,20 m
Breadth of building, b:	2,40 m
Height-to-depth ratio, h_b/d:	0,48
Edge distance, e:	2,40 m



Forces perpendicular to roof surface

Opening ratio, μ:	1,00
Internal pressure coefficient, c_{pi}:	-0,40

Zone	A	B	C	E	F	G	H	I
Wind pressure [kN/m^2]	-1,27	-0,63	-0,16	0,06	-2,22	-1,27	-0,47	0,95

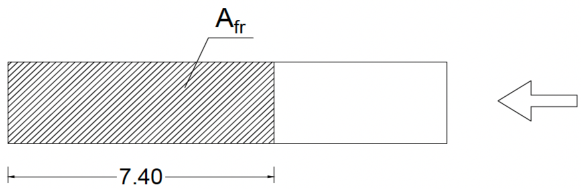
Variation in wind pressure on the windward side:

Zone	D1	D2	D3	D4	D5	D6
Wind pressure [kN/m^2]	1,40	1,44	1,48	1,51	1,55	1,79

Strip height
h_{strip}: 0,25 m

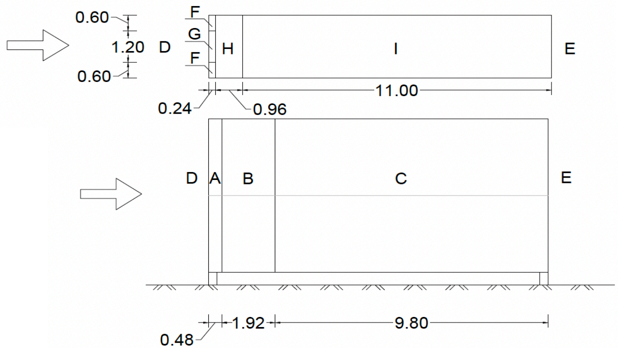
Forces parallel to roof surface

Reference area friction, A_{fr}:	103,6 m^2
Friction coefficient, c_{fr}:	0,04
Perimeter length, l_{fr}:	14,0 m
Friction forces, w_{fr}:	0,47 kN/m



Wind from west

Depth of building, d:	12,20 m
Breadth of building, b:	2,40 m
Height-to-depth ratio, h_b/d:	0,24
Edge distance, e:	2,40 m



Forces perpendicular to roof surface

Opening ratio, μ:	0,93
Internal pressure coefficient, c_{pi}:	-0,37

Zone	A	B	C	E	F	G	H	I
Wind pressure [kN/m^2]	-1,31	-0,68	-0,21	0,02	-2,26	-1,31	-0,52	0,90

Variation in wind pressure on the windward side:

Zone	D1	D2	D3	D4	D5	D6
Wind pressure [kN/m^2]	0,91	1,40	1,44	1,47	1,50	1,74

Strip height
h_{strip}: 0,25 m

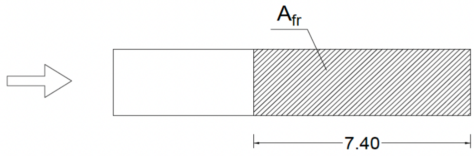
Forces parallel to roof surface

Reference area friction, A_{fr}: 103,6 m^2

Friction coefficient, c_{fr}: 0,04

Perimeter length, l_{fr}: 14,0 m

Friction forces, w_{fr}: 0,47 kN/m



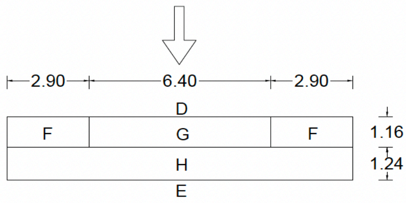
Wind from north

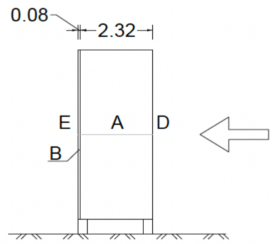
Depth of building, d: 2,40 m

Breadth of building, b: 12,20 m

Height-to-depth ratio, h_b/d: 2,42

Edge distance, e: 11,60 m





Forces perpendicular to roof surface

Opening ratio, μ: 0,53

Internal pressure coefficient, c_{pi}: 0,08

Zone	A	B	D	E	F	G	H
Wind pressure [kN/m^2]	-2,03	-1,39	1,14	-1,03	-2,97	-2,03	-1,23

Forces parallel to roof surface

Reference area friction, A_{fr}: N.A.

Appendix E: Design of roof supporting structure

Roof design - Outer Truss Members (Classroom/ Library/ Outdoor area)

Common parameters:	γ_M:	1,3		Definitions
	k_{mod}:	0,9		(s): support moment
	k_{cr}:	0,67		(f): field moment
	k_m:	0,7		
	β_c:	0,2		

Material properties, C24

	Characteristic strength		Design strength	
Bending strength	$f_{m,k}$:	24 N/mm^2	$f_{m,d}$:	16,6 N/mm^2
Tensile strength	$f_{t,0,k}$:	14,5 N/mm^2	$f_{t,0,d}$:	10,0 N/mm^2
	$f_{t,90,k}$:	0,4 N/mm^2	$f_{t,90,d}$:	0,3 N/mm^2
Compressive strength	$f_{c,0,k}$:	21 N/mm^2	$f_{c,0,d}$:	14,5 N/mm^2
	$f_{c,90,k}$:	2,5 N/mm^2	$f_{c,90,d}$:	1,7 N/mm^2
Shear strength:	$f_{v,k}$:	4 N/mm^2	$f_{v,d}$:	2,8 N/mm^2
5-percentile Modulus of elasticity	$E_{0,05}$:	7400		N/mm^2

Diagonal

Member properties:	b:	48	mm	
	h:	198	mm	
	A:	9504	mm^2	
	A_{net}:	9504	mm^2	
	I_y:	3,1E+07	mm^4	
	I_z:	1824768	mm^4	
	L:	1080	mm	
Design forces:	$N_{t,Ed}$:	55,34	kN	
	$N_{c,Ed}$:	42,38	kN	(10% increase)

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	5,82	N/mm^2	
	$\sigma_{c,0,d}$:	4,46	N/mm^2	
Design checks:	Tension:	0,58		OK
	Compression:	0,31		OK

Axial buckling

Effective lengths:	$l_{k,y}$:	1080	mm
	$l_{k,z}$:	1080	mm
Slenderness:	λ_y:	18,90	
	λ_z:	77,94	
Relative slenderness:	$\lambda_{rel,y}$:	0,32	
	$\lambda_{rel,z}$:	1,32	
Instability factors:	k_y:	0,55	
	$k_{c,y}$:	1,00	
	k_z:	1,48	
	$k_{c,z}$:	0,47	

Utilization - buckling about y-axis:		0,31	OK	
Utilization - buckling about z-axis:		0,65	OK	

Upper chord

Member properties:	b:	72	mm	(2 x 36 mm)
	h:	198	mm	
	A:	14256	mm^2	
	A_{net}:	14256	mm^2	
	I_y:	4,7E+07	mm^4	
	I_z:	6158592	mm^4	
	L_{span}:	1940	mm	
Design forces:	$N_{t,Ed}$:	51,66	kN	
	$N_{c,Ed}$:	64,89	kN	(10% increase)
	$V_{y,Ed}$:	0,47	kN	
	$V_{z,Ed}$:	0,34	kN	
	$M_{y,f,Ed}$:	0,38	kNm	
	$M_{y,s,Ed}$:	0,38	kNm	
	$M_{z,f,Ed}$:	0,00	kNm	
	$M_{z,s,Ed}$:	0,32	kNm	

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	3,62	N/mm^2	
	$\sigma_{c,0,d}$:	4,55	N/mm^2	
	$\tau_{v,z,d}$:	0,05	N/mm^2	
	$\tau_{v,y,d}$:	0,07	N/mm^2	
	$\sigma_{m,y,d}$:	0,81	N/mm^2	(s)
		0,81	N/mm^2	(f)
	$\sigma_{m,z,d}$:	1,87	N/mm^2	(s)
		0,00	N/mm^2	(f)

Design checks:	Tension:	0,36	OK	
	Compression:	0,31	OK	
	Shear (y):	0,03	OK	
	Shear(z):	0,02	OK	
	Bending (y-axis):	0,13	OK	(s)
	Bending (z-axis):	0,15	OK	(s)
	Bending (y-axis):	0,05	OK	(f)
	Bending (z-axis):	0,03	OK	(f)
	Tension + bending (y):	0,49	OK	(s)
	Tension + bending (z):	0,51	OK	(s)
	Compression + bending (y):	0,23	OK	(s)
	Compression + bending (z):	0,24	OK	(s)

Axial buckling - Double chord

Effective lengths:	$l_{k,y}$:	1552	mm		
	$l_{k,z}$:	1552	mm		
Slenderness:	λ_y:	27,15			
	λ_z:	74,67			
Relative slenderness:	$\lambda_{rel,y}$:	0,46			
	$\lambda_{rel,z}$:	1,27			
Instability factors:	k_y:	0,62			
	$k_{c,y}$:	0,96			
	k_z:	1,40			
	$k_{c,z}$:	0,50			
Utilization - buckling about y-axis:		0,37		OK	(f)
Utilization - buckling about z-axis:		0,66		OK	(f)

Axial buckling - Single chord

Design stresses:	$\sigma_{c,0,d}$:	4,55	N/mm^2		
	$\sigma_{m,y,d}$:	0,81	N/mm^2		
	$\sigma_{m,z,d}$:	0,00	N/mm^2		
Effective lengths:	$l_{k,y}$:	1552	mm		
	$l_{k,z}$:	776	mm		
Slenderness:	λ_y:	27,15			
	λ_z:	74,67			
Relative slenderness:	$\lambda_{rel,y}$:	0,46			
	$\lambda_{rel,z}$:	1,27			
Instability factors:	k_y:	0,62			
	$k_{c,y}$:	0,96			
	k_z:	1,40			
	$k_{c,z}$:	0,50			
Utilization - buckling about y-axis:		0,37		OK	(f)
Utilization - buckling about z-axis:		0,66		OK	(f)

Lateral Torsional Buckling - Double chord

Effective length:	l_{ef}:	2142	mm	
	$\sigma_{m,crit}$:	70,6	N/mm^2	
	$\lambda_{rel,m}$:	0,55		
	k_{crit}:	1		
Utilization - LTB:		0,63		OK

Lateral Torsional Buckling - Single chord

Effective length:	l_{ef}:	1890	mm	
	$\sigma_{m,crit}$:	20,0	N/mm^2	
	$\lambda_{rel,m}$:	1,02		
	k_{crit}:	0,79		
Utilization - LTB:		0,63		OK

Lower chord

Member properties:	b:	72	mm	(2 x 36 mm)
	h:	198	mm	
	A:	14256	mm^2	
	A_{net}:	14256	mm^2	
	I_y:	4,7E+07	mm^4	
	I_z:	6158592	mm^4	
	L_{span}:	1800	mm	
Design forces:	$N_{t,Ed}$:	32,96	kN	
	$N_{c,Ed}$:	67,91	kN	(10% increase)
	$V_{y,Ed}$:	2,36	kN	
	$V_{z,Ed}$:	2,48	kN	
	$M_{y,f,Ed}$:	0,52	kNm	
	$M_{y,s,Ed}$:	1,83	kNm	
	$M_{z,f,Ed}$:	0,40	kNm	
	$M_{z,s,Ed}$:	1,22	kNm	

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	2,31	N/mm^2	
	$\sigma_{c,0,d}$:	4,76	N/mm^2	
	$\tau_{v,z,d}$:	0,39	N/mm^2	
	$\tau_{v,y,d}$:	0,37	N/mm^2	
	$\sigma_{m,y,d}$:	3,89	N/mm^2	(s)
		1,11	N/mm^2	(f)
	$\sigma_{m,z,d}$:	7,13	N/mm^2	(s)
		2,34	N/mm^2	(f)

Design checks:	Tension:	0,23	OK	
	Compression:	0,33	OK	
	Shear (y):	0,13	OK	
	Shear(z):	0,14	OK	
	Bending (y-axis):	0,53	OK	(s)
	Bending (z-axis):	0,59	OK	(s)
	Bending (y-axis):	0,17	OK	(f)
	Bending (z-axis):	0,19	OK	(f)
	Tension + bending (y):	0,76	OK	(s)
	Tension + bending (z):	0,82	OK	(s)
	Compression + bending (y):	0,64	OK	(s)
	Compression + bending (z):	0,70	OK	(s)

Axial buckling - Double chord

Effective lengths:	$l_{k,y}$:	1440	mm		
	$l_{k,z}$:	1440	mm		
Slenderness:	λ_y:	25,19			
	λ_z:	69,28			
Relative slenderness:	$\lambda_{rel,y}$:	0,43			
	$\lambda_{rel,z}$:	1,17			
Instability factors:	k_y:	0,60			
	$k_{c,y}$:	0,97			
	k_z:	1,28			
	$k_{c,z}$:	0,56			
Utilization - buckling about y-axis:		0,50		OK	(f)
Utilization - buckling about z-axis:		0,77		OK	(f)

Axial buckling - Single chord

Design stresses:	$\sigma_{c,0,d}$:	4,76	N/mm^2		
	$\sigma_{m,y,d}$:	1,11	N/mm^2		(f)
	$\sigma_{m,z,d}$:	4,68	N/mm^2		(f)
Effective lengths:	$l_{k,y}$:	1440	mm		
	$l_{k,z}$:	480	mm		
Slenderness:	λ_y:	25,19			
	λ_z:	46,19			
Relative slenderness:	$\lambda_{rel,y}$:	0,43			
	$\lambda_{rel,z}$:	0,78			
Instability factors:	k_y:	0,60			
	$k_{c,y}$:	0,97			
	k_z:	0,86			
	$k_{c,z}$:	0,83			
Utilization - buckling about y-axis:		0,60		OK	
Utilization - buckling about z-axis:		0,72		OK	

Lateral Torsional Buckling - Double chord

Effective length:	l_{ef}:	2016	mm		
	$\sigma_{m,crit}$:	75,0	N/mm^2		
	$\lambda_{rel,m}$:	0,53			
	k_{crit}:	1			
Utilization - LTB:		0,59		OK	(f)

Lateral Torsional Buckling - Single chord

Effective length:	l_{ef}:	1764	mm		
	$\sigma_{m,crit}$:	21,4	N/mm^2		
	$\lambda_{rel,m}$:	0,99			
	k_{crit}:	0,82			
Utilization - LTB:		0,40		OK	(f)

Roof design - Inner Truss Members (Classroom/ Library/ Outdoor area)

Common parameters:	γ_M:	1,3	**Definitions**
	k_{mod}:	0,9	(s): support moment
	k_{cr}:	0,67	(f): field moment
	k_m:	0,7	
	β_c:	0,2	

Material properties, C24

		Characteristic strength			Design strength	
Bending strength	$f_{m,k}$:	24	N/mm^2	$f_{m,d}$:	16,6	N/mm^2
Tensile strength	$f_{t,0,k}$:	14,5	N/mm^2	$f_{t,0,d}$:	10,0	N/mm^2
	$f_{t,90,k}$:	0,4	N/mm^2	$f_{t,90,d}$:	0,3	N/mm^2
Compressive strength	$f_{c,0,k}$:	21	N/mm^2	$f_{c,0,d}$:	14,5	N/mm^2
	$f_{c,90,k}$:	2,5	N/mm^2	$f_{c,90,d}$:	1,7	N/mm^2
Shear strength:	$f_{v,k}$:	4	N/mm^2	$f_{v,d}$:	2,8	N/mm^2
5-percentile Modulus of elasticity	$E_{0,05}$:	7400				N/mm^2

Diagonal

Member properties:	b:	48	mm	
	h:	148	mm	
	A:	7104	mm^2	
	A_{net}:	7104	mm^2	
	I_y:	1,3E+07	mm^4	
	I_z:	1363968	mm^4	
	L:	1080	mm	
Design forces:	$N_{t,Ed}$:	45,96	kN	
	$N_{c,Ed}$:	33,869	kN	(10% increase)

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	6,47	N/mm^2	
	$\sigma_{c,0,d}$:	4,77	N/mm^2	
Design checks:	Tension:	0,64		OK
	Compression:	0,33		OK

Axial buckling

Effective lengths:	$l_{k,y}$:	1080	mm
	$l_{k,z}$:	1080	mm
Slenderness:	λ_y:	25,28	
	λ_z:	77,94	
Relative slenderness:	$\lambda_{rel,y}$:	0,43	
	$\lambda_{rel,z}$:	1,32	
Instability factors:	k_y:	0,60	
	$k_{c,y}$:	0,97	
	k_z:	1,48	
	$k_{c,z}$:	0,47	

Utilization - buckling about y-axis:		0,34	OK	
Utilization - buckling about z-axis:		0,70	OK	

Upper chord

Member properties:	b:	72	mm	(2 x 36 mm)
	h:	123	mm	
	A:	8856	mm^2	
	A_{net}:	8856	mm^2	
	I_y:	1,1E+07	mm^4	
	I_z:	3825792	mm^4	
	L_{span}:	1940	mm	
Design forces:	$N_{t,Ed}$:	41,98	kN	
	$N_{c,Ed}$:	52,50	kN	(10% increase)
	$V_{y,Ed}$:	0,38	kN	
	$V_{z,Ed}$:	0,38	kN	
	$M_{y,f,Ed}$:	0,20	kNm	
	$M_{y,s,Ed}$:	0,39	kNm	
	$M_{z,f,Ed}$:	0,00	kNm	
	$M_{z,s,Ed}$:	0,26	kNm	

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	4,74	N/mm^2	
	$\sigma_{c,0,d}$:	5,93	N/mm^2	
	$\tau_{v,z,d}$:	0,10	N/mm^2	
	$\tau_{v,y,d}$:	0,10	N/mm^2	
	$\sigma_{m,y,d}$:	2,15	N/mm^2	(s)
		1,10	N/mm^2	(f)
	$\sigma_{m,z,d}$:	2,45	N/mm^2	(s)
		0,00	N/mm^2	(f)

Design checks:	Tension:	0,47	OK	
	Compression:	0,41	OK	
	Shear (y):	0,03	OK	
	Shear(z):	0,03	OK	
	Bending (y-axis):	0,23	OK	(s)
	Bending (z-axis):	0,24	OK	(s)
	Bending (y-axis):	0,07	OK	(f)
	Bending (z-axis):	0,05	OK	(f)
	Tension + bending (y):	0,70	OK	(s)
	Tension + bending (z):	0,71	OK	(s)
	Compression + bending (y):	0,40	OK	(s)
	Compression + bending (z):	0,40	OK	(s)

Axial buckling - Double chord

Effective lengths:	$l_{k,y}$:	1552	mm		
	$l_{k,z}$:	1552	mm		
Slenderness:	λ_y:	43,71			
	λ_z:	74,67			
Relative slenderness:	$\lambda_{rel,y}$:	0,74			
	$\lambda_{rel,z}$:	1,27			
Instability factors:	k_y:	0,82			
	$k_{c,y}$:	0,86			
	k_z:	1,40			
	$k_{c,z}$:	0,50			
Utilization - buckling about y-axis:		0,54		OK	(f)
Utilization - buckling about z-axis:		0,86		OK	(f)

Axial buckling - Single chord

Design stresses:	$\sigma_{c,0,d}$:	5,93	N/mm^2		
	$\sigma_{m,y,d}$:	1,10	N/mm^2		
	$\sigma_{m,z,d}$:	0,00	N/mm^2		
Effective lengths:	$l_{k,y}$:	1552	mm		
	$l_{k,z}$:	776	mm		
Slenderness:	λ_y:	43,71			
	λ_z:	74,67			
Relative slenderness:	$\lambda_{rel,y}$:	0,74			
	$\lambda_{rel,z}$:	1,27			
Instability factors:	k_y:	0,82			
	$k_{c,y}$:	0,86			
	k_z:	1,40			
	$k_{c,z}$:	0,50			
Utilization - buckling about y-axis:		0,54		OK	(f)
Utilization - buckling about z-axis:		0,86		OK	(f)

Lateral Torsional Buckling - Double chord

Effective length:	l_{ef}:	1992	mm	
	$\sigma_{m,crit}$:	122,1	N/mm^2	
	$\lambda_{rel,m}$:	0,41		
	k_{crit}:	1		
Utilization - LTB:		0,82		OK

Lateral Torsional Buckling - Single chord

Effective length:	l_{ef}:	1890	mm	
	$\sigma_{m,crit}$:	32,2	N/mm^2	
	$\lambda_{rel,m}$:	0,81		
	k_{crit}:	0,95		
Utilization - LTB:		0,82		OK

Lower chord

Member properties:	b:	72	mm	(2 x 36 mm)
	h:	123	mm	
	A:	8856	mm^2	
	A_{net}:	8856	mm^2	
	I_y:	1,1E+07	mm^4	
	I_z:	3825792	mm^4	
	L_{span}:	1800	mm	
Design forces:	$N_{t,Ed}$:	27,87	kN	
	$N_{c,Ed}$:	50,97	kN	(10% increase)
	$V_{y,Ed}$:	0,00	kN	
	$V_{z,Ed}$:	0,44	kN	
	$M_{y,f,Ed}$:	0,13	kNm	
	$M_{y,s,Ed}$:	0,42	kNm	
	$M_{z,f,Ed}$:	0,00	kNm	
	$M_{z,s,Ed}$:	0,00	kNm	

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	3,15	N/mm^2	
	$\sigma_{c,0,d}$:	5,76	N/mm^2	
	$\tau_{v,z,d}$:	0,11	N/mm^2	
	$\tau_{v,y,d}$:	0,00	N/mm^2	
	$\sigma_{m,y,d}$:	2,31	N/mm^2	(s)
		0,72	N/mm^2	(f)
	$\sigma_{m,z,d}$:	0,00	N/mm^2	(s)
		0,00	N/mm^2	(f)

Design checks:	Tension:	0,31	OK	
	Compression:	0,40	OK	
	Shear (y):	0,00	OK	
	Shear(z):	0,04	OK	
	Bending (y-axis):	0,14	OK	(s)
	Bending (z-axis):	0,10	OK	(s)
	Bending (y-axis):	0,04	OK	(f)
	Bending (z-axis):	0,03	OK	(f)
	Tension + bending (y):	0,45	OK	(s)
	Tension + bending (z):	0,41	OK	(s)
	Compression + bending (y):	0,30	OK	(s)
	Compression + bending (z):	0,25	OK	(s)

Axial buckling - Double chord

Effective lengths:	$l_{k,y}$:	1440	mm
	$l_{k,z}$:	1440	mm
Slenderness:	λ_y:	40,56	
	λ_z:	69,28	
Relative slenderness:	$\lambda_{rel,y}$:	0,69	
	$\lambda_{rel,z}$:	1,17	

Instability factors:	k_y:	0,78		
	$k_{c,y}$:	0,88		
	k_z:	1,28		
	$k_{c,z}$:	0,56		
Utilization - buckling about y-axis:		0,49	OK	(f)
Utilization - buckling about z-axis:		0,73	OK	(f)

Axial buckling - Single chord

Design stresses:	$\sigma_{c,0,d}$:	5,76 N/mm^2		
	$\sigma_{m,y,d}$:	0,72 N/mm^2		(f)
	$\sigma_{m,z,d}$:	0,00 N/mm^2		(f)
Effective lengths:	$l_{k,y}$:	1440 mm		
	$l_{k,z}$:	720 mm		
Slenderness:	λ_y:	40,56		
	λ_z:	69,28		
Relative slenderness:	$\lambda_{rel,y}$:	0,69		
	$\lambda_{rel,z}$:	1,17		
Instability factors:	k_y:	0,78		
	$k_{c,y}$:	0,88		
	k_z:	1,28		
	$k_{c,z}$:	0,56		
Utilization - buckling about y-axis:		0,49	OK	
Utilization - buckling about z-axis:		0,73	OK	

Lateral Torsional Buckling - Double chord

Effective length:	l_{ef}:	1866 mm		
	$\sigma_{m,crit}$:	130,4 N/mm^2		
	$\lambda_{rel,m}$:	0,40		
	k_{crit}:	1		
Utilization - LTB:		0,71	OK	(f)

Lateral Torsional Buckling - Single chord

Effective length:	l_{ef}:	1764 mm		
	$\sigma_{m,crit}$:	34,5 N/mm^2		
	$\lambda_{rel,m}$:	0,78		
	k_{crit}:	0,97		
Utilization - LTB:		0,71	OK	(f)

Roof design - Columns (Classroom/ Library/ Outdoor area)

Parameters:	γ_M:	1,3
	k_{mod}:	0,9
	k_{mod}:	0,7
	β_c:	0,2

Material properties, C24

	Characteristic strength		Design strength		
				$k_{mod} = 0,9$	$k_{mod} = 0,7$
Bending strength	$f_{m,k}$:	24 N/mm^2	$f_{m,d}$:	16,6 N/mm^2	12,9 N/mm^2
Tensile strength	$f_{t,0,k}$:	14,5 N/mm^2	$f_{t,0,d}$:	10,0 N/mm^2	7,8 N/mm^2
	$f_{t,90,k}$:	0,4 N/mm^2	$f_{t,90,d}$:	0,3 N/mm^2	0,2 N/mm^2
Compressive strength	$f_{c,0,k}$:	21 N/mm^2	$f_{c,0,d}$:	14,5 N/mm^2	11,3 N/mm^2
	$f_{c,90,k}$:	2,5 N/mm^2	$f_{c,90,d}$:	1,7 N/mm^2	1,3 N/mm^2
Shear strength:	$f_{v,k}$:	4 N/mm^2	$f_{v,d}$:	2,8 N/mm^2	2,2 N/mm^2
5-percentile Modulus of elasticity	$E_{0,05}$:	7400			N/mm^2

Member properties:	b:	98	mm
	h:	98	mm
	A:	9604	mm^2
	A_{net}:	9604	mm^2
	I_y:	7686401	mm^4
	I_z:	7686401	mm^4
	L:	2900	mm
Design forces:	$N_{t,Ed}$:	45,31	kN
	$N_{c,Ed}$:	8,67	kN

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	4,72 N/mm^2		
	$\sigma_{c,0,d}$:	0,90 N/mm^2		
Design checks:	Tension:	0,47	OK	($k_{mod} = 0,9$)
	Compression:	0,08	OK	($k_{mod} = 0,7$)

Axial buckling

Effective lengths:	$l_{k,y}$:	2900 mm	
	$l_{k,z}$:	2900 mm	
Slenderness:	λ_y:	102,51	
	λ_z:	102,51	
Relative slenderness:	$\lambda_{rel,y}$:	1,74	
	$\lambda_{rel,z}$:	1,74	
Instability factors:	k_y:	2,15	
	$k_{c,y}$:	0,29	
	k_z:	2,15	
	$k_{c,z}$:	0,29	
Utilization - buckling about y-axis:		0,27	OK
Utilization - buckling about z-axis:		0,27	OK

Roof design - Purlins (Classroom/ Library/ Outdoor area)

Common parameters:	γ_M:	1,3		Definitions
	k_{mod}:	0,9		(s): support moment
	k_{cr}:	0,67		(f): field moment
	k_m:	0,7		
	β_c:	0,2		

Material properties, C24

	Characteristic strength		Design strength	
Bending strength	$f_{m,k}$:	24 N/mm^2	$f_{m,d}$:	16,6 N/mm^2
Tensile strength	$f_{t,0,k}$:	14,5 N/mm^2	$f_{t,0,d}$:	10,0 N/mm^2
	$f_{t,90,k}$:	0,4 N/mm^2	$f_{t,90,d}$:	0,3 N/mm^2
Compressive strength	$f_{c,0,k}$:	21 N/mm^2	$f_{c,0,d}$:	14,5 N/mm^2
	$f_{c,90,k}$:	2,5 N/mm^2	$f_{c,90,d}$:	1,7 N/mm^2
Shear strength:	$f_{v,k}$:	4 N/mm^2	$f_{v,d}$:	2,8 N/mm^2
5-percentile Modulus of elasticity	$E_{0,05}$:	7400		N/mm^2

Member properties:	b:	98	mm
	h:	148	mm
	A:	14504	mm^2
	A_{net}:	14504	mm^2
	I_y:	2,6E+07	mm^4
	I_z:	1,2E+07	mm^4
	L_{span}:	2400	mm
Design forces:	$N_{t,Ed}$:	1,67	kN
	$N_{c,Ed}$:	2,10	kN
	$V_{y,Ed}$:	0,54	kN
	$V_{z,Ed}$:	8,30	kN
	$M_{y,f,Ed}$:	1,63	kNm
	$M_{y,s,Ed}$:	4,08	kNm
	$M_{z,f,Ed}$:	0,07	kNm
	$M_{z,s,Ed}$:	0,51	kNm

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	0,12	N/mm^2	
	$\sigma_{c,0,d}$:	0,14	N/mm^2	
	$\tau_{v,z,d}$:	1,28	N/mm^2	
	$\tau_{v,y,d}$:	0,08	N/mm^2	
	$\sigma_{m,y,d}$:	11,40	N/mm^2	(s)
		4,56	N/mm^2	(f)
	$\sigma_{m,z,d}$:	2,15	N/mm^2	(s)
		0,30	N/mm^2	(f)

Design checks:	Tension:	0,01	OK	
	Compression:	0,01	OK	
	Shear (y):	0,03	OK	
	Shear(z):	0,46	OK	
	Bending (y-axis):	0,78	OK	(s)
	Bending (z-axis):	0,61	OK	(s)
	Bending (y-axis):	0,29	OK	(f)
	Bending (z-axis):	0,21	OK	(f)
	Tension + bending (y):	0,79	OK	(s)
	Tension + bending (z):	0,62	OK	(s)
	Compression + bending (y):	0,78	OK	(s)
	Compression + bending (z):	0,61	OK	(s)

Axial buckling

Effective lengths:	$l_{k,y}$:	1920 mm		
	$l_{k,z}$:	1920 mm		
Slenderness:	λ_y:	44,94		
	λ_z:	67,87		
Relative slenderness:	$\lambda_{rel,y}$:	0,76		
	$\lambda_{rel,z}$:	1,15		
Instability factors:	k_y:	0,84		
	$k_{c,y}$:	0,85		
	k_z:	1,25		
	$k_{c,z}$:	0,58		
Utilization - buckling about y-axis:		0,30	OK	(f)
Utilization - buckling about z-axis:		0,23	OK	(f)

Lateral Torsional Buckling

Effective length:	l_{ef}:	2456 mm	
	$\sigma_{m,crit}$:	152,5 N/mm^2	
	$\lambda_{rel,m}$:	0,37	
	k_{crit}:	1	
Utilization - LTB:		0,09	OK

Roof design - Truss (Accommodation & Office)

Common parameters:	γ_M:	1,3		<u>Definitions</u>
	k_{mod}:	0,9		(s): support moment
	k_{mod}:	0,7		(f): field moment
	k_{cr}:	0,67		
	k_m:	0,7		
	β_c:	0,2		

Material properties, C24

	Characteristic strength		Design strength		
				$k_{mod} = 0,9$	$k_{mod} = 0,7$
Bending strength	$f_{m,k}$:	24 N/mm^2	$f_{m,d}$:	16,6 N/mm^2	12,9 N/mm^2
Tensile strength	$f_{t,0,k}$:	14,5 N/mm^2	$f_{t,0,d}$:	10,0 N/mm^2	7,8 N/mm^2
	$f_{t,90,k}$:	0,4 N/mm^2	$f_{t,90,d}$:	0,3 N/mm^2	0,2 N/mm^2
Compressive strength	$f_{c,0,k}$:	21 N/mm^2	$f_{c,0,d}$:	14,5 N/mm^2	11,3 N/mm^2
	$f_{c,90,k}$:	2,5 N/mm^2	$f_{c,90,d}$:	1,7 N/mm^2	1,3 N/mm^2
Shear strength:	$f_{v,k}$:	4 N/mm^2	$f_{v,d}$:	2,8 N/mm^2	2,2 N/mm^2
5-percentile Modulus of elasticity	$E_{0,05}$:	7400			N/mm^2

Diagonal

Member properties:	b:	36	mm	
	h:	98	mm	
	A:	3528	mm^2	
	A_{net}:	3528	mm^2	
	I_y:	2823576	mm^4	
	I_z:	381024	mm^4	
	L:	1580	mm	
Design forces:	$N_{t,Ed}$:	17,2	kN	
	$N_{c,Ed}$:	3,64	kN	(10% increase)

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	4,88	N/mm^2	
	$\sigma_{c,0,d}$:	1,03	N/mm^2	
Design checks:	Tension:	0,49		OK
	Compression:	0,07		OK

Axial buckling

Effective lengths:	$l_{k,y}$:	1580	mm
	$l_{k,z}$:	1580	mm
Slenderness:	λ_y:	55,85	
	λ_z:	152,04	
Relative slenderness:	$\lambda_{rel,y}$:	0,95	
	$\lambda_{rel,z}$:	2,58	
Instability factors:	k_y:	1,01	
	$k_{c,y}$:	0,73	

	k_z:	4,05		
	$k_{c,z}$:	0,14		

Utilization - buckling about y-axis:	0,10	OK	
Utilization - buckling about z-axis:	0,51	OK	

Upper chord

Member properties:	b:	48	mm	
	h:	98	mm	
	A:	4704	mm^2	
	A_{net}:	4704	mm^2	
	I_y:	3764768	mm^4	
	I_z:	903168	mm^4	
	L_{span}:	1940	mm	
Design forces:	$N_{t,Ed}$:	1,72	kN	
	$N_{c,Ed}$:	13,79	kN	(10% increase)
	$V_{y,Ed}$:	0,06	kN	
	$V_{z,Ed}$:	0,03	kN	
	$M_{y,f,Ed}$:	0,00	kNm	
	$M_{y,s,Ed}$:	0,00	kNm	
	$M_{z,f,Ed}$:	0,00	kNm	
	$M_{z,s,Ed}$:	0,07	kNm	

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	0,37	N/mm^2	
	$\sigma_{c,0,d}$:	2,93	N/mm^2	
	$\tau_{v,z,d}$:	0,01	N/mm^2	
	$\tau_{v,y,d}$:	0,03	N/mm^2	
	$\sigma_{m,y,d}$:	0,00	N/mm^2	(s)
		0,00	N/mm^2	(f)
	$\sigma_{m,z,d}$:	1,86	N/mm^2	(s)
		0,00	N/mm^2	(f)

Design checks:	Tension:	0,05	OK	(k_{mod} = 0,7)
	Compression:	0,20	OK	
	Shear (y):	0,01	OK	
	Shear(z):	0,01	OK	
	Bending (y-axis):	0,08	OK	(s)
	Bending (z-axis):	0,11	OK	(s)
	Bending (y-axis):	0,00	OK	(f)
	Bending (z-axis):	0,00	OK	(f)
	Tension + bending (y):	0,13	OK	(s)
	Tension + bending (z):	0,16	OK	(s)
	Compression + bending (y):	0,12	OK	(s)
	Compression + bending (z):	0,15	OK	(s)

Axial buckling

Effective lengths:	$l_{k,y}$:	1552	mm		
	$l_{k,z}$:	1552	mm		
Slenderness:	λ_y:	54,86			
	λ_z:	112,01			
Relative slenderness:	$\lambda_{rel,y}$:	0,93			
	$\lambda_{rel,z}$:	1,90			
Instability factors:	k_y:	1,00			
	$k_{c,y}$:	0,74			
	k_z:	2,46			
	$k_{c,z}$:	0,25			
Utilization - buckling about y-axis:		0,27		OK	(f)
Utilization - buckling about z-axis:		0,81		OK	(f)

Lateral Torsional Buckling - Double chord

Effective length:	l_{ef}:	1942	mm	
	$\sigma_{m,crit}$:	69,9	N/mm^2	
	$\lambda_{rel,m}$:	0,55		
	k_{crit}:	1		
Utilization - LTB:		0,81		OK

Lower chord

Member properties:	b:	48	mm	
	h:	123	mm	
	A:	5904	mm^2	
	A_{net}:	5904	mm^2	
	I_y:	7443468	mm^4	
	I_z:	1133568	mm^4	
	L_{span}:	1800	mm	
Design forces:	$N_{t,Ed}$:	0,60	kN	
	$N_{c,Ed}$:	6,22	kN	(10% increase)
	$V_{y,Ed}$:	0,78	kN	
	$V_{z,Ed}$:	0,55	kN	
	$M_{y,f,Ed}$:	0,00	kNm	
	$M_{y,s,Ed}$:	0,26	kNm	
	$M_{z,f,Ed}$:	0,22	kNm	
	$M_{z,s,Ed}$:	0,46	kNm	

Cross-sectional design

Design stresses:		$\sigma_{t,0,d}$:	0,10 N/mm^2		
		$\sigma_{c,0,d}$:	1,05 N/mm^2		
		$\tau_{v,z,d}$:	0,21 N/mm^2		
		$\tau_{v,y,d}$:	0,30 N/mm^2		
		$\sigma_{m,y,d}$:	2,15 N/mm^2		(s)
			0,00 N/mm^2		(f)
		$\sigma_{m,z,d}$:	9,74 N/mm^2		(s)
			4,66 N/mm^2		(f)
Design checks:	Tension:		0,01	OK	(k_{mod} = 0,7)
	Compression:		0,07	OK	
	Shear (y):		0,11	OK	
	Shear(z):		0,08	OK	
	Bending (y-axis):		0,54	OK	(s)
	Bending (z-axis):		0,68	OK	(s)
	Bending (y-axis):		0,20	OK	(f)
	Bending (z-axis):		0,28	OK	(f)
	Tension + bending (y):		0,55	OK	(s)
	Tension + bending (z):		0,69	OK	(s)
	Compression + bending (y):		0,54	OK	(s)
	Compression + bending (z):		0,68	OK	(s)

Axial buckling

Effective lengths:	$l_{k,y}$:	1440 mm		
	$l_{k,z}$:	1440 mm		
Slenderness:	λ_y:	40,56		
	λ_z:	103,92		
Relative slenderness:	$\lambda_{rel,y}$:	0,69		
	$\lambda_{rel,z}$:	1,76		
Instability factors:	k_y:	0,78		
	$k_{c,y}$:	0,88		
	k_z:	2,20		
	$k_{c,z}$:	0,28		
Utilization - buckling about y-axis:		0,28	OK	(f)
Utilization - buckling about z-axis:		0,53	OK	(f)

Lateral Torsional Buckling

Effective length:	l_{ef}:	1866 mm		
	$\sigma_{m,crit}$:	57,9 N/mm^2		
	$\lambda_{rel,m}$:	0,60		
	k_{crit}:	1		
Utilization - LTB:		0,25	OK	(f)

Roof design - Purlins (Accommodation & Office)

Common parameters:	γ_M:	1,3		Definitions
	k_{mod}:	0,9		(s): support moment
	k_{mod}:	0,7		(f): field moment
	k_{cr}:	0,67		
	k_m:	0,7		
	β_c:	0,2		

Material properties, C24

	Characteristic strength		Design strength		
				$k_{mod} = 0,9$	$k_{mod} = 0,7$
Bending strength	$f_{m,k}$:	24,0 N/mm^2	$f_{m,d}$:	16,6 N/mm^2	12,9 N/mm^2
Tensile strength	$f_{t,0,k}$:	14,5 N/mm^2	$f_{t,0,d}$:	10,0 N/mm^2	7,8 N/mm^2
	$f_{t,90,k}$:	0,4 N/mm^2	$f_{t,90,d}$:	0,3 N/mm^2	0,2 N/mm^2
Compressive strength	$f_{c,0,k}$:	21,0 N/mm^2	$f_{c,0,d}$:	14,5 N/mm^2	11,3 N/mm^2
	$f_{c,90,k}$:	2,5 N/mm^2	$f_{c,90,d}$:	1,7 N/mm^2	1,3 N/mm^2
Shear strength:	$f_{v,k}$:	4,0 N/mm^2	$f_{v,d}$:	2,8 N/mm^2	2,2 N/mm^2
5-percentile Modulus	$E_{0,05}$:	7400			N/mm^2

Member properties:	b:	48	mm	
	h:	198	mm	
	A:	9504	mm^2	
	A_{net}:	9504	mm^2	
	I_y:	3,1E+07	mm^4	
	I_z:	1824768	mm^4	
	L_{span}:	2400	mm	
Design forces:	$N_{t,Ed}$:	0,25	kN	
	$N_{c,Ed}$:	0,76	kN	
	$V_{y,Ed}$:	0,11	kN	
	$V_{z,Ed}$:	7,78	kN	
	$M_{y,f,Ed}$:	1,03	kNm	
	$M_{y,s,Ed}$:	2,27	kNm	
	$M_{z,f,Ed}$:	0,02	kNm	
	$M_{z,s,Ed}$:	0,09	kNm	

Cross-sectional design

Design stresses:	$\sigma_{t,0,d}$:	0,03	N/mm^2	
	$\sigma_{c,0,d}$:	0,08	N/mm^2	
	$\tau_{v,z,d}$:	1,83	N/mm^2	
	$\tau_{v,y,d}$:	0,03	N/mm^2	
	$\sigma_{m,y,d}$:	7,24	N/mm^2	(s)
		3,28	N/mm^2	(f)
	$\sigma_{m,z,d}$:	1,18	N/mm^2	(s)
		0,26	N/mm^2	(f)

Design checks:	Tension:	0,00	OK	(k_{mod} = 0,7)
	Compression:	0,01	OK	
	Shear (y):	0,01	OK	
	Shear(z):	0,66	OK	
	Bending (y-axis):	0,49	OK	(s)
	Bending (z-axis):	0,38	OK	(s)
	Bending (y-axis):	0,21	OK	(f)
	Bending (z-axis):	0,15	OK	(f)
	Tension + bending (y):	0,49	OK	(s)
	Tension + bending (z):	0,38	OK	(s)
	Compression + bending (y):	0,49	OK	(s)
	Compression + bending (z):	0,38	OK	(s)

Axial buckling

Effective lengths:	$l_{k,y}$:	1920 mm		
	$l_{k,z}$:	1920 mm		
Slenderness:	λ_y:	33,59		
	λ_z:	138,56		
Relative slenderness:	$\lambda_{rel,y}$:	0,57		
	$\lambda_{rel,z}$:	2,35		
Instability factors:	k_y:	0,69		
	$k_{c,y}$:	0,93		
	k_z:	3,47		
	$k_{c,z}$:	0,17		
Utilization - buckling about y-axis:		0,21	OK	(f)
Utilization - buckling about z-axis:		0,19	OK	(f)

Lateral Torsional Buckling

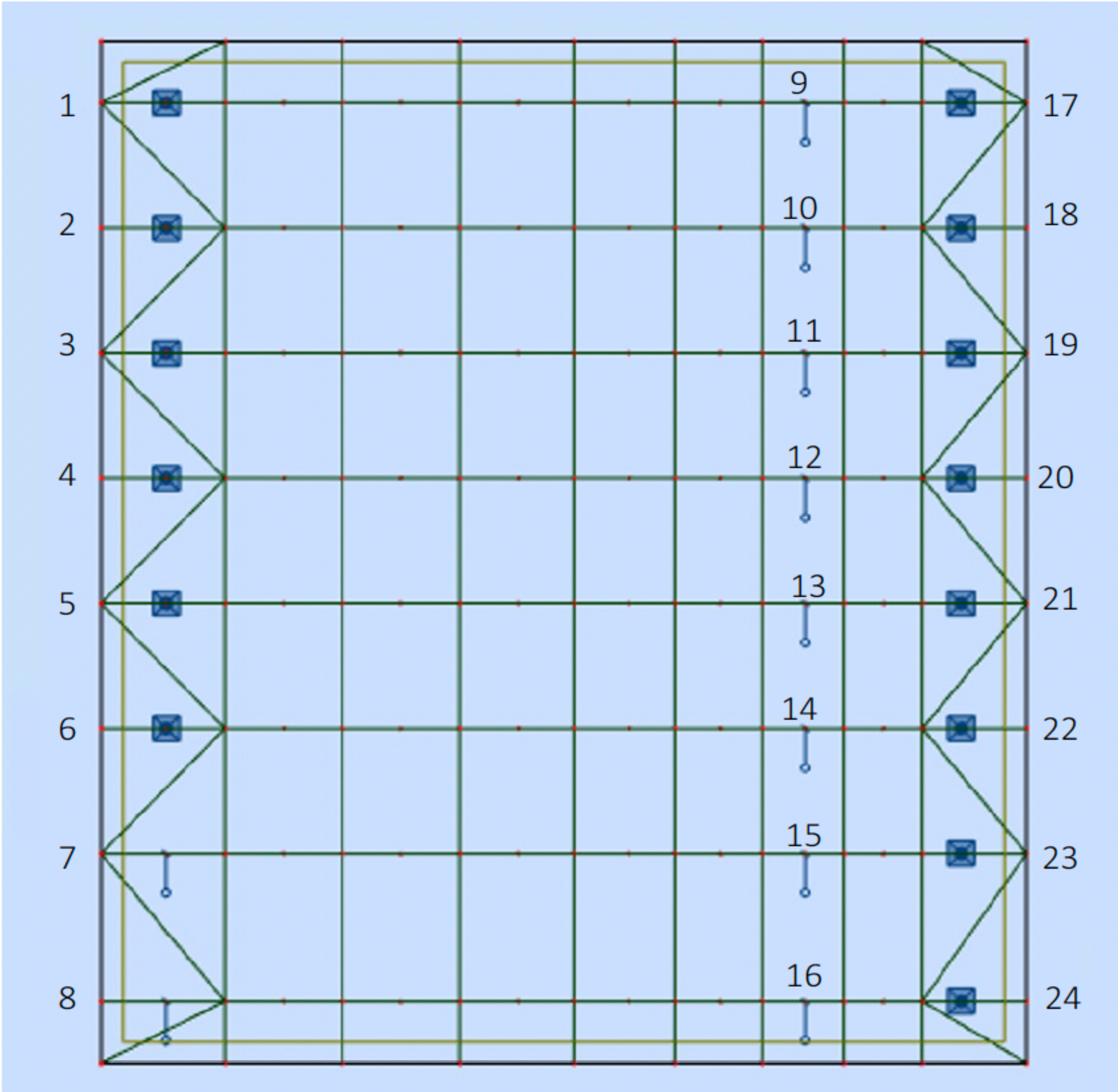
Effective length:	l_{ef}:	2556 mm	
	$\sigma_{m,crit}$:	26,3 N/mm^2	
	$\lambda_{rel,m}$:	0,89	
	k_{crit}:	0,89	
Utilization - LTB:		0,08	OK

Appendix F: Support reactions from external roofs

Support reactions - Roof over classroom/library/outdoor area

Support	Member support	Load combination														
		DL + LL1			DL + LL2			DL + WL east			DL + WL west			DL + WL south		
		Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz
1	Column support	0,00	0,00	8,67	0,00	0,00	6,13	0,00	0,00	-45,21	0,00	0,00	-36,34	0,00	0,00	-39,95
2	Column support	0,00	0,00	7,51	0,00	0,00	8,20	0,00	0,00	-34,59	0,00	0,00	-26,02	0,00	0,00	-25,83
3	Column support	0,00	0,00	8,07	0,00	0,00	5,97	0,00	0,00	-36,59	0,00	0,00	-27,40	0,00	0,00	-27,54
4	Column support	0,00	0,00	7,97	0,00	0,00	3,76	0,00	0,00	-36,13	0,00	0,00	-27,06	0,00	0,00	-27,13
5	Column support	0,00	0,00	8,04	0,00	0,00	3,97	0,00	0,00	-36,51	0,00	0,00	-27,34	0,00	0,00	-27,39
6	Column support	0,00	0,00	7,76	0,00	0,00	5,61	0,00	0,00	-35,01	0,00	0,00	-26,22	0,00	0,00	-26,50
7	Truss/on library		0,00	8,55		0,00	9,04		0,00	-40,56		0,00	-30,58		0,00	-30,16
8	Truss/on library		-0,03	8,98		0,02	6,58		0,23	-47,61		0,20	-38,13		-0,28	-41,84
15	Truss/on library		0,00	14,11		0,00	14,54		0,00	-57,91		0,00	-42,46		0,00	-45,28
16	Truss/on library		-0,33	14,65		0,10	10,98		2,33	-71,26		2,09	-57,28		0,86	-65,26
23	Truss/on library	0,20	0,00	0,07	0,73	0,00	0,09	-8,14	0,00	-4,36	-6,34	0,00	-4,06	-5,88	0,00	-1,97
24	Truss/on library	-0,05	-0,08	0,08	-0,26	0,02	0,04	-9,40	0,49	-1,93	-8,07	0,41	-1,26	-8,00	0,13	-0,31
9	Truss/on classroom		0,36	14,08		-0,11	10,20		-2,55	-69,23		-2,28	-55,98		-4,42	-62,72
10	Truss/on classroom		0,00	12,50		0,00	13,24		0,00	-50,51		0,00	-36,99		0,00	-41,24
11	Truss/on classroom		0,00	13,30		0,00	9,86		0,00	-52,23		0,00	-37,67		0,00	-40,37
12	Truss/on classroom		0,00	13,12		0,00	6,10		0,00	-51,31		0,00	-37,01		0,00	-40,08
13	Truss/on classroom		0,00	13,22		0,00	6,35		0,00	-52,01		0,00	-37,56		0,00	-40,44
14	Truss/on classroom		0,00	12,92		0,00	9,44		0,00	-50,30		0,00	-36,24		0,00	-39,43
17	Truss/on classroom	0,17	0,08	-0,12	-0,18	-0,03	-0,10	-10,00	-0,49	-0,45	-8,56	-0,42	-0,02	-10,23	-0,88	1,59
18	Truss/on classroom	-0,28	0,00	0,07	0,37	0,00	0,09	-5,14	0,00	-4,01	-4,03	0,00	-3,72	-3,90	0,00	-2,14
19	Truss/on classroom	0,22	0,00	-0,13	0,19	0,00	-0,14	-7,80	0,00	-3,38	-6,12	0,00	-3,35	-6,07	0,00	-1,63
20	Truss/on classroom	-0,14	0,00	-0,03	-0,43	0,00	0,05	-6,18	0,00	-3,87	-4,95	0,00	-3,71	-4,99	0,00	-2,00
21	Truss/on classroom	0,14	0,00	-0,10	-0,23	0,00	0,05	-7,66	0,00	-3,50	-6,08	0,00	-3,42	-6,03	0,00	-1,71
22	Truss/on classroom	-0,25	0,00	-0,06	-0,19	0,00	-0,08	-5,42	0,00	-3,65	-4,30	0,00	-3,54	-4,31	0,00	-1,93

Note: When added to the top of shipping containers the sign of the forces changes. (Negative = downward, Positive = upward)
Grey areas = no support for translation in this direction



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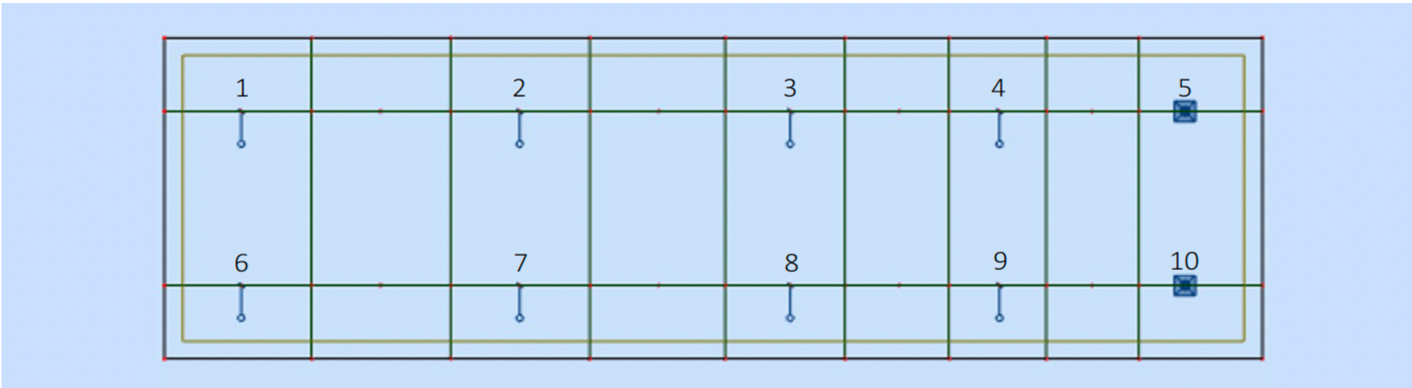
Support reactions - Roof over accommodation and office

Support	Load combination														
	DL + LL1			DL + LL2			DL + WL east			DL + WL west			DL + WL south		
	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz
1		0,03	3,91		0,07	2,68		-0,20	-25,68		-0,20	-25,13		-0,29	-22,41
2		0,14	5,33		0,38	3,67		-1,07	-26,04		-1,06	-25,22		-1,36	-25,46
3		0,13	4,47		0,37	3,05		-0,76	-21,74		-0,77	-21,08		-1,01	-21,21
4		0,17	3,61		0,48	2,51		-1,15	-18,71		-1,15	-18,12		-1,44	-18,41
5	0	0,03	3,18	0	0,08	2,17	-8,28	-0,22	-19,73	-8,06	-0,22	-19,35	-7,96	-0,29	-16,64
6		-0,03	3,91		-0,07	2,68		0,20	-25,65		0,19	-25,09		0,15	-22,55
7		-0,14	5,33		-0,38	3,67		1,04	-26,30		1,03	-25,47		0,99	-26,04
8		-0,13	4,47		-0,37	3,06		0,83	-22,39		0,83	-21,71		0,84	-22,07
9		-0,17	3,61		-0,48	2,51		1,14	-19,12		1,14	-18,52		1,17	-18,94
10	0	-0,03	3,18	0	-0,08	2,17	-8,34	0,20	-19,72	-8,12	0,19	-19,34	-7,88	0,17	-16,74

Note: When added to the top of shipping containers the sign of the forces changes. (Negative = downward, Positive = upward)
Grey areas = no support for translation in this direction



Appendix G: Foundation design

Foundation design - Classroom

Common properties for all foundations

Safe bearing pressure:	σ_{gd}:	150	kN/m^2
Density of soil:	γ_{soil}:	18,6	kN/m^3
Density of concrete:	$\gamma_{concrete}$:	24,0	kN/m^3
Partial factor compression:	γ_G:	1,2	
Partial factor uplift:	γ_G:	1,0	
Column properties:	b:	0,35	m
	h:	0,35	m
	A_{column}:	0,12	m^2
	L_{column}:	1,0	m
Foundation properties:	h_f:	0,5	m
	L_f:	1,0	m

Foundation 5

Design forces:	F_z^+:	91,8	kN
	F_z^-:	159,0	kN
Footing properties:	$b_{0,min}$:	0,78	m
	b_{0x}:	0,90	m
	b_{0y}:	0,90	m
	b_x:	0,90	m
	b_y:	0,90	m
Design check ground pressure:	N_{Ed1}:	91,8	kN
	A_0:	0,81	m^2
	q_{Ed1}:	113,3	kN/m^2
	Self-weight foundation:	15,2	kN
	Self-weight soil:	7,7	kN
	N_{Ed2}:	22,9	kN
	A:	0,81	m^2
	q_{Ed2}:	28,2	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	141,5	kN/m^2
	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	6,63	m^3

Foundation 6

Design forces:	F_x:	71,2	kN
	$M_{Edx,max}$:	71,2	kNm
	F_z^+:	45,4	kN
	F_z^-:	69,0	kN
Footing properties:	$b_{0,min}$:	0,55	m
	b_{0x}:	0,63	m
	b_{0y}:	0,65	m
	$e_{x,max}$:	1,57	m
	b_x:	2,20	m
	b_y:	0,65	m
Design check ground pressure:	N_{Ed1}:	45,4	kN
	A_0:	0,41	m^2
	q_{Ed1}:	110,8	kN/m^2
	Self-weight foundation:	24,1	kN
	Self-weight soil:	14,6	kN
	N_{Ed2}:	38,7	kN
	A:	1,43	m^2
	q_{Ed2}:	27,1	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	137,9	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	
Uplift of foundation			
Necessary volume:	V:	2,87	m^3

Foundation 7

Design forces:	F_y:	18,2	kN
	$M_{Edy,max}$:	18,2	kNm
	F_z^+:	78,3	kN
	F_z^-:	114,7	kN
Footing properties:	$b_{0,min}$:	0,72	m
	b_{0x}:	0,85	m
	b_{0y}:	0,77	m
	$e_{y,max}$:	0,23	m
	b_x:	0,85	m
	b_y:	1,00	m
Design check ground pressure:	N_{Ed1}:	78,3	kN
	A_0:	0,65	m^2
	q_{Ed1}:	119,6	kN/m^2

Foundation design - Library

Common properties for all foundations

Safe bearing pressure:	σ_{gd}:	150	kN/m^2
Density of soil:	γ_{soil}:	18,6	kN/m^3
Density of concrete:	$\gamma_{concrete}$:	24,0	kN/m^3
Partial factor compression:	γ_G:	1,2	
Partial factor uplift:	γ_G:	1,0	
Column properties:	b:	0,35	m
	h:	0,35	m
	A_{column}:	0,12	m^2
	L_{column}:	1,0	m
Foundation properties:	h_f:	0,5	m
	L_f:	1,0	m

Foundation 9

Design forces:	F_y:	45,4	kN
	$M_{Edy,max}$:	45,4	kNm
	F_z^+:	68,1	kN
	F_z^-:	37,9	kN
Footing properties:	$b_{0,min}$:	0,67	m
	b_{0x}:	0,80	m
	b_{0y}:	0,73	m
	$e_{y,max}$:	0,67	m
	b_x:	0,80	m
	b_y:	1,40	m
Design check ground pressure:	N_{Ed1}:	68,1	kN
	A_0:	0,58	m^2
	q_{Ed1}:	116,6	kN/m^2
	Self-weight foundation:	19,6	kN
	Self-weight soil:	11,1	kN
	N_{Ed2}:	30,7	kN
	A:	1,12	m^2
	q_{Ed2}:	27,5	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	144,1	kN/m^2
	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	1,58	m^3

Foundation 10

Design forces:	F_x:	16,3	kN
	F_y:	45,2	kN
	$M_{Edx,max}$:	16,3	kNm
	$M_{Edy,max}$:	45,2	kNm
	F_z^+:	68,2	kN
	F_z^-:	40,8	kN
Footing properties:	$b_{0,min}$:	0,67	m
	b_{0x}:	0,76	m
	b_{0y}:	0,74	m
	$e_{x,max}$:	0,24	m
	$e_{y,max}$:	0,66	m
	b_x:	1,00	m
	b_y:	1,40	m
Design check ground pressure:	N_{Ed1}:	68,2	kN
	A_0:	0,56	m^2
	q_{Ed1}:	121,3	kN/m^2
	Self-weight foundation:	23,7	kN
	Self-weight soil:	14,3	kN
	N_{Ed2}:	38,0	kN
	A:	1,40	m^2
	q_{Ed2}:	27,1	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	148,4	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	
Uplift of foundation			
Necessary volume:	V:	1,70	m^3

Foundation 11

Design forces:	F_z^+:	68,1	kN
	F_z^-:	83,6	kN
Footing properties:	$b_{0,min}$:	0,67	m
	b_{0x}:	0,80	m
	b_{0y}:	0,80	m
	b_x:	0,80	m
	b_y:	0,80	m
Design check ground pressure:	N_{Ed1}:	68,1	kN
	A_0:	0,64	m^2
	q_{Ed1}:	106,4	kN/m^2

	Self-weight foundation:	12,7	kN
	Self-weight soil:	5,8	kN
	N_{Ed2}:	18,5	kN
	A:	0,64	m^2
	q_{Ed2}:	28,9	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	135,3	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	3,48	m^3

Foundation 12

Design forces:	F_x:	16,0	kN
	$M_{Edx,max}$:	16,0	kNm
	F_z^+:	67,8	kN
	F_z^-:	80,5	kN
Footing properties:	$b_{0,min}$:	0,67	m
	b_{0x}:	0,76	m
	b_{0y}:	0,75	m
	$e_{x,max}$:	0,24	m
	b_x:	1,00	m
	b_y:	0,75	m
Design check ground pressure:	N_{Ed1}:	67,8	kN
	A_0:	0,57	m^2
	q_{Ed1}:	118,9	kN/m^2
	Self-weight foundation:	14,3	kN
	Self-weight soil:	7,0	kN
	N_{Ed2}:	21,3	kN
	A:	0,75	m^2
	q_{Ed2}:	28,5	kN/m^2
Ground pressure, $q_{Ed1}+q_{Ed2}$:		147,4	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	3,35	m^3

Foundation design - Accommodation and office

Common properties for all foundations

Safe bearing pressure:	σ_{gd}:	150	kN/m^2
Density of soil:	γ_{soil}:	18,6	kN/m^3
Density of concrete:	$\gamma_{concrete}$:	24,0	kN/m^3
Partial factor compression:	γ_G:	1,2	
Partial factor uplift:	γ_G:	1,0	
Column properties:	b:	0,35	m
	h:	0,35	m
	A_{column}:	0,12	m^2
	L_{column}:	1,0	m
Foundation properties:	h_f:	0,5	m
	L_f:	1,0	m

Foundation 1

Design forces:	F_x:	26,9	kN
	$M_{Edx,max}$:	26,9	kNm
	F_z^+:	106,3	kN
	F_z^-:	200,2	kN
Footing properties:	$b_{0,min}$:	0,84	m
	b_{0x}:	0,95	m
	b_{0y}:	0,95	m
	$e_{x,max}$:	0,25	m
	b_x:	1,20	m
	b_y:	0,95	m
Design check ground pressure:	N_{Ed1}:	106,3	kN
	A_0:	0,90	m^2
	q_{Ed1}:	117,8	kN/m^2
	Self-weight foundation:	20,0	kN
	Self-weight soil:	11,4	kN
	N_{Ed2}:	31,4	kN
	A:	1,14	m^2
	q_{Ed2}:	27,5	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	145,3	kN/m^2
	$\sigma_{gd} > q_{Ed}$:	OK	

Uplift of foundation

Necessary volume:	V:	8,34	m^3

Foundation 2

Design forces:	F_z^+:	106,3	kN
	F_z^-:	190,9	kN
Footing properties:	$b_{0,min}$:	0,84	m
	b_{0x}:	0,95	m
	b_{0y}:	0,95	m
	b_x:	0,95	m
	b_y:	0,95	m
Design check ground pressure:	N_{Ed1}:	106,3	kN
	A_0:	0,90	m^2
	q_{Ed1}:	117,8	kN/m^2
	Self-weight foundation:	16,5	kN
	Self-weight soil:	8,7	kN
	N_{Ed2}:	25,2	kN
	A:	0,90	m^2
	q_{Ed2}:	28,0	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	145,8	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	
Uplift of foundation			
Necessary volume:	V:	7,95	m^3

Foundation 3

Design forces:	F_x:	26,9	kN
	F_y:	114,7	kN
	$M_{Edx,max}$:	26,9	kNm
	$M_{Edy,max}$:	114,7	kNm
	F_z^+:	140,8	kN
	F_z^-:	37,6	kN
Footing properties:	$b_{0,min}$:	0,97	m
	b_{0x}:	1,11	m
	b_{0y}:	1,04	m
	$e_{x,max}$:	0,19	m
	$e_{y,max}$:	0,81	m
	b_x:	1,30	m
	b_y:	1,85	m

Design check ground pressure:	N_{Ed1}:	140,8	kN
	A_0:	1,15	m^2
	q_{Ed1}:	122,0	kN/m^2
	Self-weight foundation:	38,3	kN
	Self-weight soil:	25,6	kN
	N_{Ed2}:	63,8	kN
	A:	2,41	m^2
	q_{Ed2}:	26,5	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	148,4	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	1,56	m^3

Foundation 4

Design forces:	F_y:	114,6	kN
	$M_{Edy,max}$:	114,6	kNm
	F_z^+:	149,8	kN
	F_z^-:	91,8	kN
Footing properties:	$b_{0,min}$:	1,00	m
	b_{0x}:	1,15	m
	b_{0y}:	1,09	m
	$e_{y,max}$:	0,76	m
	b_x:	1,15	m
	b_y:	1,85	m
Design check ground pressure:	N_{Ed1}:	149,8	kN
	A_0:	1,25	m^2
	q_{Ed1}:	119,5	kN/m^2
	Self-weight foundation:	34,2	kN
	Self-weight soil:	22,4	kN
	N_{Ed2}:	56,7	kN
	A:	2,13	m^2
	q_{Ed2}:	26,6	kN/m^2
Ground pressure, $q_{Ed1}+q_{Ed2}$:		146,1	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	3,82	m^3

	Self-weight foundation:	15,8	kN
	Self-weight soil:	8,1	kN
	N_{Ed2}:	23,9	kN
	A:	0,85	m^2
	q_{Ed2}:	28,1	kN/m^2
	$q_{Ed1}+q_{Ed2}$:	147,7	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	4,78	m^3

Foundation 8

Design forces:	F_x:	61,7	kN
	F_y:	10,9	kN
	$M_{Edx,max}$:	61,7	kNm
	$M_{Edy,max}$:	10,9	kNm
	F_z^+:	45,4	kN
	F_z^-:	58,6	kN
Footing properties:	$b_{0,min}$:	0,55	m
	b_{0x}:	0,64	m
	b_{0y}:	0,61	m
	$e_{x,max}$:	1,36	m
	$e_{y,max}$:	0,24	m
	b_x:	2,00	m
	b_y:	0,85	m
Design check ground pressure:	N_{Ed1}:	45,4	kN
	A_0:	0,39	m^2
	q_{Ed1}:	116,3	kN/m^2
	Self-weight foundation:	28,0	kN
	Self-weight soil:	17,6	kN
	N_{Ed2}:	45,7	kN
	A:	1,70	m^2
	q_{Ed2}:	26,8	kN/m^2
Ground pressure, $q_{Ed1}+q_{Ed2}$:		143,1	kN/m^2
Design check:	$\sigma_{gd} > q_{Ed}$:	OK	

<u>Uplift of foundation</u>

Necessary volume:	V:	2,44	m^3

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