



Temporary Steel Frame Warehouses

A Conceptual Design for a Modulus System

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

FREDRIK ECKERWALL & DAVID GLANS

Department of Civil and Environmental Engineering Division of Structural Engineering CHALMERS UNIVERSITY OF TECHNOLOGY Göteborg, Sweden 2012 Master's Thesis 2012: 149

MASTER'S THESIS 2012:149

Temporary Steel Frame Warehouses

A Conceptual Design for a Modulus System

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

FREDRIK ECKERWALL & DAVID GLANS

Department of Civil and Environmental Engineering Division of Structural Engineering Steel and Timber Structures CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2012

Temporary Steel Frame Warehouses A Conceptual Design for a Modulus System

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design FREDRIK ECKERWALL & DAVID GLANS

© FREDRIK ECKERWALL & DAVID GLANS, 2012

Examensarbete / Institutionen för bygg- och miljöteknik, Chalmers tekniska högskola 2012: 149

Department of Civil and Environmental Engineering Division of Structural Engineering Chalmers University of Technology SE-412 96 Göteborg Sweden Telephone: + 46 (0)31-772 1000

Cover:

Overview of the modulus concepts for temporary steel frame warehouses. More about the different modulus concepts can be found in chapter 6.

Name of the printers / Department of Civil and Environmental Engineering Göteborg, Sweden 2012

Temporary Steel Frame Warehouses A Conceptual Design for a Modulus System

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design FREDRIK ECKERWALL & DAVID GLANS Department of Civil and Environmental Engineering Division of Structural Engineering Chalmers University of Technology

ABSTRACT

Permanent and temporary steel frame buildings are commonly used as sport facilities, exhibition halls, leisure facilities and warehouses. The advantages of a temporary steel frame structure are the quick erection time, the standardized manufacturing and its flexibility with regards to location and area of use. While there has been great advances in the environmental aspects of the concept the basic structural system has virtually remained the same. The need for a simple and cost effective system has continued to be a dominant factor and the usage of steel provides the possibility of both flexibility in the design and a standardized manufacturing. The temporary building supplier MIT AB and the steel supplier Hallmek AB are now about to develop a new concept for temporary warehouses. The aim is to create a modular system which is easy to assemble/disassemble, capable to withstand repeated handling and contain as few different components as possible. MIT AB also wants the modular system to be able to cover spans of 10-20 meters using the same components to reduce inventory cost and time of delivery.

A study of the temporary warehouse market, with main focus on Sweden but also countries with similar climate conditions, was performed to get an overview of existing solutions. This led to further research of portal frames and the roof shapes normally used for these types of structures. Furthermore, other areas with similar requirements for temporary structures were investigated e.g. deployable structures in the military and the entertainment industry.

Several different concepts and solutions which correspond to the requirements from MIT AB and Hallmek AB were developed. This resulted in four promising concepts which were more thoroughly analyzed in the 2D- software *Frame Analysis*. The loads used in the analysis were calculated according to *Eurocode 1 – Actions on structures*. To assess the four alternatives an evaluation-matrix, containing a number of criteria, were developed.

The results of the evaluation show that the concept with an arched frame with columns is the most suitable solution. The results from the calculations show that the portal frame should consist of truss elements to reduce the weight. Furthermore, the connections used in the winning concept should be adjustable to minimize the number of different parts in the structure.

Key words: Temporary structure, steel frame, conceptual design, portal frame, deployable structures, connections.

Temporära lagerhallar med stålstomme Konceptuell design för ett modulkoncept Examensarbete inom Structural Engineering and Building Performance Design FREDRIK ECKERWALL & DAVID GLANS Institutionen för bygg- och miljöteknik Avdelningen för Konstruktionsteknik Stål- och träbyggnad Chalmers tekniska högskola

SAMMANFATTNING

Permanenta och tillfälliga rambyggnader i stål används ofta som sport- och fritidsanläggningar, utställningshallar och lagerhallar. Fördelarna med en tillfällig stålramsbyggnad är snabbt uppförande, standardiserad tillverkning och flexibilitet när det gäller placering och användningsområde. Även om det har skett stora framsteg inom möjligheterna att påverka inomhusmiljön, så har stomsystemet i stort sett varit oförändrat. Behovet av ett enkelt och kostnadseffektivt system har fortsatt att vara en dominerande faktor och användningen av stål ger möjligheten till både flexibilitet med hänsyn till utformning och standardiserad tillverkning. Lagerhallsleverantören MIT AB och stålleverantören Hallmek AB försöker nu att utveckla ett nytt koncept för tillfälliga lagerhallar. Syftet är att skapa ett modulsystem som är lätt att montera upp/ner, kunna motstå upprepad hantering samt bestå av så få olika komponenter som möjligt. MIT AB vill även att komponenterna i modulsystemet skall kunna täcka spännvidder på 10-20 meter för att minska lagerkostnad och leveranstid.

En studie över marknaden för tillfälliga lagerhallar, med fokus på Sverige men även för länder med liknande klimatförhållanden, genomfördes för att få en överblick över redan befintliga lösningar. Studien ledde till ytterligare fördjupning inom portalramar och dess takformer. Vidare studerades andra marknader med liknande krav på tillfälliga anläggningar så som militär- och nöjesindustrin.

Ett flertal olika koncept och lösningar utvecklades i linje med önskemål och krav från MIT AB och Hallmek AB, vilket resulterade i fyra lovande koncept. Koncepten analyserades mer ingående i 2D-programmet *Frame Analysis* där de införda lasterna beräknades enligt *Eurocode 1 - Laster på bärverk*. För att bedöma de fyra alternativen skapades en utvärderingsmatris innehållande ett antal kriterier.

Resultaten av utvärderingen visar att konceptet med en bågformad ram med ben är den bäst lämpade lösningen. Resultaten från beräkningarna visar att elementen ska konstrueras som ett fackverk för att minska vikten. Vidare så ska en justerbar infästning användas i det vinnande konceptet för att minimera antalet olika delar i byggnaden.

Nyckelord: Temporära byggnader, stålramar, konceptuell design, portalramar, utvecklingsbara konstruktioner, infästningar.

Contents

ABST	RACT	Ι	
SAMM	IANFATTNING	II	
CONTENTS			
PREFA	ACE	V	
NOTA	TIONS	VI	
1 IN	TRODUCTION	1	
1.1	Purpose	2	
1.2	Limitations	2	
1.3	Method	2	
2 ST	FRUCTURAL SYSTEMS – PORTAL FRAMES	3	
2.1	Roof types for portal frames	4	
2.	1.1 Pitched roof structures	4	
2.	1.2 Curved roof structures	5	
2.2	Types of rafters (truss, rolled or welded)	5	
2.1	2.1 Trusses	5	
2.2	2.2 Hot-rolled sections	/	
2	2.5 Cold-formed profiles 2.4 Welded girders	o 8	
2		8	
2.3	Transportation	8	
3 TI	EMPORARY- AND DEPLOYABLE STRUCTURES	9	
3.1	Entertainment industry	9	
3.2	Military applications	11	
3.	2.1 DRASH (Deployable Rapid Assembly Shelter)	11	
3.	2.2 MCP (Modular Command Post)	12	
3.3	Space industry	12	
3.4	Temporary bridges	13	
4 D	ESIGN CRITERIA	16	
4.1	Requests from MIT AB and Hallmek AB	16	
4.2	Design and evaluation criteria	18	
4.	2.1 Material cost	18	
4.	18		
4.	2.3 Number of connections	18	
4.	2.4 Elements length	18	
4. /	2.3 weight 2.6 Element types	18	
4.	2.0 Liement types	19	

	4.2.7 4.2.8 4.2.9	Requested volume Adjustable height Utilization	19 19 19
	4.3 Re	sult of the weighing process	19
5	THE CO	DNCEPTUAL DESIGN	21
5.1 Out 5.1.1 5.1.2 5.1.3 5.1.4 5.1.5 5.2 Res 5.2.1 5.2.2 5.2.3 5.2.4 5.2.5 5.2.6 5.2.7 5.3 Eva		tline of the concepts Duo-pitched roof Mansard roof Arced roof Arced roof with columns Summary of the four concepts	21 21 23 23 25 26
		sult from the analysis Loads acting on the structure Calculations for the weight and utilization factor Duo-pitched roof Mansard roof Arced roof Arced roof with columns Summary of the results aluation of the concepts	27 28 29 29 32 35 38 42 43
6	CONNE	ECTION EXAMPLES	45
	6.1 Bo	lted connections	45
	6.2 Sna 6.2.1 6.2.2	ap- and adjustable connections Snap connections Adjustable connections	46 46 48
7	DISCUS	SSION AND CONCLUSIONS	50
Rł	EFERENC	ES	52
AI	PPENDIX	I - THE PROBLEMATIC WINTER OF 2009/10	

APPENDIX II- LOAD CALCULATIONS

APPENDIX III – DIMENSIONING OF ELEMENTS

Preface

In this Master's thesis a conceptual design of a temporary steel frame warehouse have been carried out. The project has been done on behalf of MIT AB and Hallmek AB who are about to develop a modular system for temporary warehouses.

The thesis has been carried out from January to June 2012 as the final step in our education at the civil engineering program at Chalmers University of Technology.

We would like to thank our supervisors Mohammad Al- Emrani and Urban Svensson who have shared their knowledge and kept us motivated throughout the entire project. We would also like to thank Peter Gullbrandsson at Tyréns who gave us the opportunity to work in a stimulating environment and Håkan Landebring who took his time to answer our questions regarding *Frame Analysis*.

A special thanks to Bengt-Åke Flöner, Christian Petersson, Åke Lundh and Bertil Smidfelt for sharing their knowledge and keeping our spirits high with entertaining anecdotes during the project. We also appreciate the support from Bengt Lundin and his involvement in this project.

Finally we would like to thank our opponents Hanna Jansson and Isak Svensson for their good advice and inspiring attitude.

Göteborg June 2012 Fredrik Eckerwall David Glans

Notations

Roman upper case letters

Α	Area of the cross section	$[m^2]$
A_{e}	Axial force	[kN]
C_{ce}	Length of column element	[m]
C_{e}	Topography coefficient	[-]
$C_{{\it pe},i}$	External pressure coefficient	[-]
C_t	Thermal coefficient	[-]
F_i	Design load for wind	[kN/m]
G_k	Self-weight	[kN/m]
Ι	Moment of inertia	[m ⁴]
L_{e}	Length of roof element	[m]
L_F	The distance between the frames	[m]
М	Bending moment	[kNm]
$Q_{k,1}$	Primary variable load	[kN/m]
$Q_{k,2}$	Secondary variable load	[kN/m]
$S_{1,i}$	Design load for snow, case 1	$[kN/m^2]$
$S_{2,i}$	Design load for snow, case 2	$[kN/m^2]$
W	Weight of an element	[kg]
W_{tot}	Weight of one frame	[kg]
Ζ	Distance to neutral layer	[m]

Roman lower case letters

b	Width of the flange	[m]
d	Thickness of the web	[m]
d_i	The width of the structure	[m]
f_i	The height of the arced part of the structure	[m]
f_{yd}	Yield strength	[MPa]
h	Height of the beam	[m]
h_i	Total height	[m]
h_1	The height of the columns	[m]
n	Number of elements in one frame	[-]
$q_{p,i}$	Peak velocity pressure	$[kN/m^2]$

s _k	Characteristic snow load	$[kN/m^2]$
t	Thickness of the flange	[m]
$Z_{e,i}$	Reference height	[m]

Greek letters

γ_g	Safety factor for permanent action	[-]
γ_q	Safety factor for variable action	[-]
μ_1	Snow load shape coefficient	[-]
μ_2	Snow load shape coefficient	[-]
μ_3	Snow load shape coefficient	[-]
μ_{e}	Utilization ratio	[-]
ρ	Density	$[kg/m^3]$
$\sigma_{_e}$	Stresses	[MPa]
$\psi_{0,i}$	Form factor	[-]

1 Introduction

The industrial steel building is a common sight all over the world. The concept has been around for many years and it seems like it is here to stay. The name industrial building might be a bit misleading since the concept is as commonly used in sports facilities, exhibition halls, leisure facilities and supermarkets as it is in the industry. A more suitable term for this type of structure is therefore to simply call it single story buildings. Many companies, regardless of their businesses, have peaks in their production or have seasonal products. These production peaks can be hard to plan and a new temporary building which is simple to erect and easy to dismantle was developed.

The increased use of industrial buildings in a more commercial area has led to a greater focus on the environment that the building provides such as aesthetics, insulation, energy consumption and air tightness.

While there has been great advances in the environmental aspects of the concept the basic structural system has virtually remained the same. The need for a simple and cost effective system has continued to be a dominant factor and the usage of steel provides the possibility of both flexibility in the design and a standardized manufacturing (AccessSteel, 2012b).

In Sweden approximately 300 temporary warehouses are erected each year. The market is dominated by O.B.Wiik, PMH and Hallbyggarna Jonsered, but there are about six other companies trying to increase their influence within the field. Since this type of warehouse is an economically beneficial solution compared to a permanent building, the warehouse market is extensive and there is a competitive situation amongst the manufacturers.

There is a wide range of applications for temporary buildings such as different types of sport halls, warehouses, aircraft hangars, showrooms, machine halls and recycling centers. The difference between these halls are the enhancements provided by almost all manufactures such as sliding wall panels, changing rooms, staff entrances and insulating sheets. There are many possibilities to change the halls to meet the different requirements from the customer. As an example they can easily be shortened, extended, moved or with accessories completely change the character of the hall. The vast majority of the halls can be purchased, leased or rented on both short and long term basis.

The temporary building manufacturer MIT AB has for many years a close relationship with the steel supplier Hallmek AB. They are now about to develop a new concept for temporary warehouses together with our supervisor Urban Svensson. The aim is to create a modular system which is easy to assemble/disassemble, capable to withstand repeated handling and contain as few different components as possible. MIT AB and Hallmek AB also wants the modular system to be able to cover span widths of 10 to 20 meters using the same components to reduce inventory costs the time of delivery and to get a standardized production.

The new concept is an overall business optimization where also the manufacturing cost and the weight of the components are essential. The demand for structures with spans larger than 20 meters is widespread. Therefore, if the new concept becomes successful, these larger structures will be optimized in a similar way.

1.1 Purpose

The aim of this Master's thesis is to develop a new concept for fabric-covered warehouse buildings in the span range of 10-20 meters and an eave height of 4-8 meters. The main focus is to develop the supporting structure with regard to the manufacturing, ease of the assembling and a robust design in order to withstand repeated handling. Focus on the simplicity of the structural member will also be considered since the shape affects the fabrication, galvanization, transportation and the assembly/disassembly. Furthermore, a modular system which can be used regardless of the width, within the span range, should be sought. The task is to perform an overall optimization where the aspects of the manufacturing cost and weight are essential.

1.2 Limitations

The structure is design by a 2D-analysis which means that vertical and longitudinal bracings have been neglected. No second order effects have been taken into account since the deformations have not been considered in the preliminary design. Furthermore, the frame has been assumed to be restrained from buckling.

1.3 Method

The first part of the report is a literature study of portal frames for permanent warehouses. The study comprises different types of support conditions, roof types and designs of structural elements. Furthermore, the advantages, disadvantages and important criteria for the temporary structure are described. The purpose with this part is to give the reader an understanding of the portal frame as a structural system and what factors which is of importance in the design of a temporary structure.

The second part of the report is a conceptual design of a modulus system for temporary steel frame warehouses. Promising solutions are analyzed and evaluated based on the criteria developed with MIT AB and Hallmek AB. The structural response have been analyzed in the 2D-software *Frame Analysis*, which is a tool for calculating bending moments, axial forces and shear forces in first- and second order theory. The loads used in the calculations are according to *Eurocode 1 – Actions on structures*.

2 Structural systems – portal frames

The portal frame is a frequently used structural system for single story buildings. The system is suitable for low-rise structures and consists of columns and horizontal or pitched rafters which are joined together by moment-resistant connections. These rigid connections makes it is possible for the frame to not only take care of vertical loads but also horizontal loads. Another advantage with the moment-resistant joints is that the support moment can be utilized and the field moment can be reduced. This means that the section of the rafters can be decreased but also that the section of the columns will have to be increased due to higher moments (Reichel, 2007, Salter et al., 2004).

The portal frame system can be designed in many different ways with fixed columnbase connections or hinged connections as can be seen in Figure 2.1. In order to achieve a fully fixed connection, extensive groundwork is required. This is both expensive and time consuming and is therefore not suitable for temporary structures where the speed of erection is an important factor. The deformations of the frame will increase with an increasing number of hinges but at the same time, the internal stresses will decrease which makes the frame more resistant against stresses due to settlement and temperature changes (Reichel, 2007, SCI, 2008).



Figure 2.1 Different types of frames, from left to right; two-hinge frame, three hinged frame and fully fixed frame.

The two-hinged system is the most commonly used alternative due to its convenience when designing and building the foundation. Although it is the most commonly used option it does not mean that it is the most economical choice due to the fact that even modest base stiffness can result in significant increases of the frame stability (Salter et al., 2004).

In order to decrease the dimensions of the frame, without affecting its stability, a tie can be added in the roof structure, see Figure 2.2. The introduction of the tie will lead to a reduction of the moments in the columns and the horizontal movements at the eaves of the frame. The tie will reach its peak efficiency when placed at the level of the eaves. This can be a big disadvantage with regards to the functionality of the building since it will have a significant effect of the available height. When designing a tie for long spans it might be necessary to add hangers, connect the tie to the rafters, to keep the stability of the tie. If the slope of the roof is less than 15 degrees, large forces will develop in both the rafters and the tie. These large forces will reduce the overall stability in the frame and will have to be carefully analyzed during the design (Salter et al., 2004).



Figure 2.2 A duo-pitched roof with a tie placed at the height of the eave to reduce the bending moment in the most efficient way.

2.1 Roof types for portal frames

There are three types of roof shapes; pitched, curved and mansard. The different types, which can be seen in Figure 2.3, are suitable for different concepts and when choosing a shape factors like aesthetics, span length, required volume and height must be considered. The properties of each roof type will be explained in the following section.



Figure 2.3 a) A duo-pitched roof, b) A curved roof, c) A mansard roof, d) A curved beam with cut-out holes to reduce the weight.

2.1.1 Pitched roof structures

The pitched roof can be built for spans of 15-60 meters but are most efficient for spans between 20-30 meters. They are most structurally efficient at an eaves height of 5-6 meters but are, due to the supposed activity in the building, often constructed as high as 10 meters. The roof pitch is normally selected to be between 6-10 degrees. A lower angle then six degrees is not recommended because of the deformations. The deformation itself reduces the actual roof angle and could lead to accumulation of water or snow on the roof. Haunches are usually placed at the eaves and the notch in

order to reduce the dimensions of the rafters and to create an efficient moment connection at these points. The advantages with the pitched roof is that it is a simple and well proved concept with straight components which makes it a good option in regards of transportation, manufacturing and assembling (AccessSteel, 2012a, AccessSteel, 2012c).

2.1.2 Curved roof structures

The curved shape is another common roof type used for portal frames. The curvature of the rafter results in a redirection of the transverse load, meaning that the load will be carried via axial force instead of bending. Hence the member will only be subjected to compression which makes the curved roof very material efficient. This only applies for a perfect arc subjected to a symmetrical load and prevented from movement in the horizontal direction. The compression forces will increase around the supports and the rafters can be optimized for this by changing the sections of the members along the span. The curved roof can be constructed with trusses or structural sections but in either case hollow sections should be used due to their good buckling behaviour. For small spans the members can be manufactured to follow the curve of the roof, see Figure 2.3b, but for longer spans this can be hard to accomplish and it might be better to use straight elements which will form a polygon shaped roof, also known as a mansard roof which can be seen in Figure 2.3c. The mansard roof shape is preferred due its straight components, compared to the problematic and more expansive manufacturing of curved components¹. Another variation of the curved roof is the cellular roof structure with cut-out holes in the web which lead to a reduction in the self-weight, see Figure 2.3d. This solution is suitable for span range of 40-55 meters but larger spans are possible (SCI, 2008).

2.2 Types of rafters (truss, rolled or welded)

A portal frame can be constructed from truss structures, rolled members, welded members or a combination of these. The following section will describe the three different types of members and their advantages or disadvantages.

2.2.1 Trusses

A truss can be described as a beam consisting of a web of struts most commonly in a triangular pattern. The struts are designed either in a W-formation with only diagonal members or in an N-formation with both diagonal and vertical members, see Figure 2.4.

¹ Åke Lundh, Hallmek AB, meeting 2/3 2012.



W-formation

Figure 2.4 Example of the most common triangular patterns for trusses.

The triangular shape enables the structural components to solely handle axial stresses and also prevent the upper chord to buckle. Furthermore, the risk of buckling will reduce when the distance between the nodes are decreasing (SCI, 2009).

There are numerous types of trusses which can be categorized by the outline of the structure, see Figure 2.5. The *parallel truss* is suitable for flat roofs but can also be used as mono-pitched roofs with a connection between the truss and the support providing the inclination. The *pitched truss* is solely made for duo-pitched type of roofs while the *tapered truss* is suited for mono-pitched roofs. Note that in order to optimize the use of the pitched truss, it should be turned upside down compared to the one shown in Figure 2.5. By doing so, the shape of the truss will relate to the moment distribution in a better way. When using the *pitched-, tapered-* or the *curved truss*, free space near the ceiling is occupied by the framework. If the indoor height in the structure is of great importance, the *parallel pitched truss* or the *curved beam* is a better alternative (SCI, 2009).



Figure 2.5 Examples of truss shapes for different roof types and for different needs.

Trusses can either be statically determinate or statically indeterminate over one or several spans. Statically determinate truss mean that it is possible to calculate the forces and moments at the supports and in each member by equilibrium equations. If the truss is statically indeterminate its deformations need to be considered to be able to calculate the forces in each member (Bengtsson and Pettersson, 1972).

Statically determined trusses are most common and characterized by as few members and connections as possible and are preferable when the structure is exposed to large temperature movement or settlements. The downside with a statically determinate truss is that the structure is vulnerable e.g. it is not possible to change a member in the truss without causing a mechanism and the whole structure will be unstable, see Figure 2.6.



Figure 2.6 A shows a statically determinate system. It is a vulnerable system that will cause a mechanism if a member is removed. B shows a statically indeterminate system where it is possible to remove a member without causing a mechanism.

The major determinant to decide whether the roof is going to comprise of trusses-, rolled- or welded sections is the economical aspect. Other determinates are the length of the span and the estimated vertical load. If the structure doesn't have any height limitations a truss could be a possible option instead of a solid cross section. The truss has a different moment distribution than the solid sections and therefore needs a higher web to be able to carry the load. A well designed truss is rigid and has a high load carrying capacity with less usage of material than a solid section. The production cost for the truss is higher than for the welded- and rolled sections, but since the truss requires less material, the advantage for the truss model is increasing with longer spans. The profit to use trusses instead of solid sections starts with spans over 15 meters. To design and construct large trusses it is of importance to consider the selfweight to prevent failure during the assembly. Some other advantages for truss structures are the possibility to build in large spans, in some buildings over 100 meters. Truss simplifies for installations often hidden in the ceiling, for example ventilation and electricity compared to solid sections The truss structure use less material compared to a beam structure and hence results in a lighter building which is preferable in temporary structures (AccessSteel, 2012a, SCI, 2008).

2.2.2 Hot-rolled sections

Beams like IPE or profiles like VKR are generally hot-rolled i.e. formed in the incandescent state. When hot-rolling, the incandescent steel will pass through special rolling grooves and are formed to the right dimensions and shapes. Because of different thickness of the cross-section, the parts are not cooled down in the same rate which is causing residual stresses which in turn affects the buckling curve. Hot-rolled sections are pre-dimensioned but are possible to customize if needed (Stålbyggnadsinstitutet, 2008).

2.2.3 Cold-formed profiles

By bending of a steel plate it is possible to produce cold-formed profiles like KKR. The modern technique provides KKR-profiles with a thickness of up to 15 millimetres. The profiles, mainly the U-profile, are used increasingly in welded steel trusses in single storey buildings because it is possible to adjust the shape and the function of the profile. When the steel is bent at room temperature, it causes deformation hardening containing residual stresses (Stålbyggnadsinstitutet, 2008, SBI, 2012).

2.2.4 Welded girders

A welded girder gives the opportunity to customize the girder for the intended function. The advantage to be able to produce any type of profile shape generates a high flexibility. But to fabricate a large variety of welded beams are expensive which makes the welded beam frequently used in simpler models. Due to the fact that welded girders are customized implies that the girders are not a stock item but are produced on request with a short time of delivery. One of the most common welded girders is the I-beam which is usually used at large span constructions as e.g. in bridges. For roof constructions is it more common to use a truss member or an I-beam with cut out holes to reduce the weight. The truss member is often welded together by a hot-rolled rod or VKR- or KKR profiles (Stålbyggnadsinstitutet, 2008).

2.3 Transportation

The way to transport a portal frame is depending on the economic aspects as well as the location of the factory and the new building. Truck transportation is widely used because it is the best way to avoid reloading of the cargo. However, this alternative might not be an environmental friendly approach but often the time- and money consuming reloads between e.g. trains and trucks must be avoided. A length of 20 meters and a height of 3.5 meters is the maximum size of portal frame transportable on most European roads (AccessSteel, 2012a, Stålbyggnadsinstitutet, 2008).

3 Temporary- and deployable structures

Temporary structures are common within many different areas. The main qualities that are sought from a temporary structure are transportability, simplicity in assembly/ disassembly, light-weight materials and functionality. Industries that have shown a special interest in temporary structures are the military, aerospace, entertainment and to a certain extent the construction industry. The military and the space industry have shown a special interest in a concept called deployable structures and spends a lot of time and money on both research and development within this field.

A deployable structure is all kinds of structures where the structural members are linked together in a compact form for easier transportation. Their assembly results from unfolding the compact form by rapidly deploy a large volume structure. The concept of a deployable structure is shown in Figure 3.1. The potential of compact storage, transportability and simplicity to erect and retrieve sometimes outweigh the complex design which a deployable structure results in. A complex design which includes the disadvantages such as an advanced production, complex connections and difficulties in changing components, and these drawbacks generates a slower rate of production which also is more expensive.



Figure 3.1 The concept behind a deployable structure where all members are linked together. The red line shows one specific member during the assembly of the structure. Picture modified from (Gantes, 2001).

The purpose when designing a deployable structure is to design the structural members for the regular service load, and if possible, get the deployable ability as an extra feature without compromising with the weight or the load carrying capacity of the structure. As already mentioned, applications for deployable structures are found in the military and aerospace industry. But there are other potential and promising applications of deployable structures e.g. emergency shelters or bridges after earthquakes, re-locatable warehouse- or concert-structures and temporary protective covers for outdoor activities such as road construction. It is the speed and ease of erection, ease of transportation and the minimal skill required for erection, dismantling and relocation among many advantages that make the deployable structures an interesting possibility (Gantes, 2001). The following chapter will give some examples of where temporary and deployable structures are used.

3.1 Entertainment industry

The show business frequently arranges large events which have the need to host thousands of people in outdoor areas. This calls for a variety of different temporary structures such as concert roof systems, spot towers, press risers, lighting trusses, sound bays, video bays and solid stages. The entire setup needs to be mobile enough to move from city to city and the assembly time must be kept short to keep up with tight tour schedules. The high mobility requirement creates the need for light weight constructions containing small parts for simple transportation on trailers and trucks. However the construction must be robust enough to withstand relatively high wind loads and to support the weight of large TV screens and speakers. This might sound quite simple compared to the design of permanent structures but it calls for both inventiveness and qualitative engineering. Due to the fact that the system is to be used at a number of different locations make it a quite complex system. The construction has to be able to be adapted according to the current terrain type, ground conditions, weather conditions and requirements from the costumers. Consequently the key to a successful system is flexibility.

Another important criterion to take into account is safety. These types of events are often hosting thousands of people, packed tightly together. A falling speaker, screen or tower would be devastating and must be avoided at any cost. The design also has to provide a safe way of assembly. Normally a lot of workers are involved in the construction and due to tight schedules this has to be quick. Hence the site will be full of activity and working on ladders and such can be a dangerous thing. A good solution in terms of safety and speed is the height-adjustable roof that is used by many manufactures, see Figure 3.2.



Figure 3.2 Shows a typical solution for a concert stage roof. The construction is made of aluminium trusses due to the importance of a lightweight construction and has a height-adjustable roof. Picture modified from (Alustage, 2012).

This solution provides the possibility to assemble the roof structure and install the lights and speakers without using high-rising ladders or lifts. The most common structural system for concert stages is the truss system. The truss system has the advantages of being light, material efficient and provides good possibilities to anchor lights, screens and speakers (Gorlin, 2009).

3.2 Military applications

Military command centers need to be capable of relocating and rapidly be mounted or dismounted. Time is the most important factor when erecting command structures and easy deployment is needed in order to maintain communication with the troops, escape enemy attacks and provide the commander with high-tech multimedia like unit tracking programs. However, it is of no use if the structure that comprises these items is incapable to keep up with the operational tempo of modern warfare.

The functional requirements of the structural systems are:

- The system must be transportable by existing military vehicles.
- The erection time must be three hours or less.
- The system must protect computing systems from detrimental heat, cold and humidity.

A few years ago, the erection time for a commander post was about 30 minutes. Today the erection time is approximately three hours and the main reason is that the different components are designed separately. To facilitate the construction time, improvement for the most time-consuming stages e.g. internal wiring and external camouflage netting has to be combined in the building process. Examples of two building systems for Command Posts are The DRASH-system and the MCP-system.

3.2.1 DRASH (Deployable Rapid Assembly Shelter)

The DRASH tent folds and unfolds like a spider when assembled and disassembled, as can be seen in Figure 3.3. It is a complex design and is prone to accidental stresses during the assembly and disassembly. These stresses will result in plastic deformations in the structural members and might cause blockage which can result in fracture of the supports. Replacing a structural member can be time consuming and the reparation is a liability of the system.



Figure 3.3 An example of a military tent during the spider-like assembly. Picture modified from (Gantes, 2001).

3.2.2 MCP (Modular Command Post)

The MCP is an older design and does not include any system features as climate control like the DRASH concepts. The MCP is a lightweight construction and its simplicity makes it the most robust design currently used as a Commander Post. Nonetheless, the MCP has moving joints that are prone to failure and due to the lack of integrations of e.g. power, heating and cooling, which means that the system takes longer time to erect.

There have been difficulties when constructing the Command Post, regardless of the building system used. The planning of the fabric covering expansion and recovery during the different assembling- and disassembling-phases was a major issue, but also the stresses in the folding and unfolding mechanisms triggered by compressed fabric caused problems. The stresses from the fabric covering were far greater than the load carrying stressed from e.g. self-weight, snow or wind which resulted in time consuming reparations.

3.3 Space industry

The need for deployable structures has always been great in the aerospace industry due to its unique conditions. The fact that the structures need to be launched into space creates a great number of challenges for the engineers. The structures must for example be able to withstand the launch loads which are considerably higher than the orbital loads, be stowed in a small spacecraft and be deployed without risking the life of the astronauts (Tibert, 2002).

Due to the unique conditions the main design criteria for deployable space structures are compactness, dimensional, tolerance, rigidity, ease of deployment, durability, endurance and cost. Furthermore, light-weight structures are of great importance due to the fact that it has to be launched up into space (Kiper and Söylemez, 2009).

Common deployable structures used in the aerospace industry are antennas, solar panels and masts. An example of a mast is *The Folding Articulated Square Truss, FAST*, created by a company called *ATK Aerospace Systems*. The FAST-mast was developed for the International space station, ISS, in order to support its solar arrays (Tibert, 2002). The mast is built up from a number of single bays which includes six main components: upper and lower longerons, rigid battens, flex battens, corner joints, elbow joints and diagonal cables (Knight et al., 2012). The principle of the mast and the configuration of a single bay can be seen in Figure 3.4 and in Figure 3.5.



Figure 3.4 Principle of the FAST-mast (Tibert, 2002).



Figure 3.5 A single bay of the FAST-mast and its six main components (Knight et al., 2012)

3.4 Temporary bridges

An early example of a temporary bridge is the design invented by Leonardo Da Vinci. This design, which can be seen in Figure 3.6, includes a self-supporting system which has no need for connections, consists of straight elements and can be erected without any tools. Although there are no real proof that the bridge was actually built during the life of Leonardo Da Vinci the design can be seen, in a more complex state, in a number of bridges in China (Bernardoni et al., 2005).



Figure 3.6 Sketch of Leonardo Da Vinci's portable bridge design (Bernardoni et al., 2005).

During the Second World War Donald Coleman Bailey developed a portable bridge called the Bailey Bridge. The purpose of the design was to create a bridge which could span various lengths, be made from available materials, be manufactured by standard engineering companies, contain parts that could be carried by no more than six men and be simple to construct on site. As can be seen in Figure 3.7, the result was a modular system made from truss girders joined together with pin-connections and cross beams in various layers forming the deck (ThinkDefence, 2012).



Figure 3.7 Sketch showing the modulus concept of the Bailey Bridge. Picture modified from (ThinkDefence, 2012).

The concept of the Bailey Bridge has lived on and the French company Matière has developed their own design called the Unibridge. The design is carried out according to *Eurocode 3* and can be used as either a temporary or a permanent structure (Unibridge, 2009). The design differs from the Bailey Bridge due to the fact that it consists of less components and has a more rapid deployment (Shepherd, 2005). The Unibridge design is shown in Figure 3.8.



Figure 3.8 Showing the modulus design of the Matière Unibridge system (Unibridge, 2009).

4 Design criteria

The following chapter is describing the design criteria used to evaluate and compare different designs to each other. The first section describes the requests for the new concept from MIT AB and Hallmek AB. Furthermore, a description of the identified design criteria is presented followed by a weighing showing their importance.

4.1 Requests from MIT AB and Hallmek AB

As mentioned earlier in Chapter 1, the Swedish market produces roughly 300 temporary storage buildings each year, by approximately nine manufacturers. MIT AB is among the top five of the manufacturers but is trying to increase their share of the market. MIT AB has an average production of 20 - 30 temporary structures per year, mostly to industries where they are generally used for storage purpose but also as a weather shelter in the production. Their clientele is mainly based in the centraland southern parts of Sweden, partly because of the smaller market up north but also due to the long distance for maintenances and repairs. MIT AB has an agreement with their clients to solve urgent problems within 24 hour and to be able to fulfill that agreement MIT AB has focused on the central- and southern part of Sweden. MIT AB has today a variety of temporary structures with spans between 10–40 meters where the 20 meter span structure is the most requested. The 40 meters span structure is produced on demand and is usually for buying unlike the smaller structures of 10-20 meters which is more suited for rental. Hence, the smaller span structures will not be as price-sensitive as the larger ones because the structure is rentable several times. Consequently it does not matter if the smaller span width is structurally redundant, as long as the 20 meter span structure has a good degree of utilization.²

The height of the eave can be dimensioned from four meters up to eight meters with an interval of one meter. The angle for a duo-pitched roof is in general between 18 and 20 degrees and for a mansard roof somewhere between 20 and 30 degrees. For the span of 20 meter the eave height is generally between 6 and 8 meters. To prevent a mechanism due to large bending moment in the intersections between the roof and the columns, a tie can be used in order to reduce the moments. However, the tie must not be positioned at the eave height due to demands from clients to avoid hitting the tie while loading. By moving the tie upwards in the roof construction, the tie reduce some of its ability and the bending moment will increase rapidly as the placement of the tie moves upwards.²

As also mentioned in Chapter 1, MIT AB and Hallmek AB are aiming to develop a new modular concept for their temporary warehouses. The concept should be adaptable for spans in the range of 10-20 meters and an eave height of 4-8 meters depending on the size of the structure. The aim is to develop a system containing as few parts as possible in order to reduce the storage cost, simplify the assembly and to get a standardized production. Since the same elements are to be used for all structures within the span range the smaller structures will be redundant but as mentioned earlier this is thought to be irrelevant due to the fact that these structures are less price-sensitive. This also goes for the connections between the elements and the number of roof angles should therefore also be kept to a minimum between the

² Christian Peterson, MIT AB, telephone conference 3/4 2012.

different span widths. To speed up the assembly/disassembly of the structure simple connections are essential.²

The assembly of the structure should be possible to perform by a team of 2-4 people with access to one mobile crane. To let the workers be able to carry the elements by hand the total weight of one element should not exceed 50 kilograms. The columns should be pinned to the ground to avoid bending moments between the columns and the foundation which requires large anchorage which is both time consuming and difficult to accomplish. Furthermore, the structure should be a solid construction that is capable of repeated handling. A summary of MIT AB's and Hallmek AB's requests is presented in Table 4.1.²

Table 4.1The requests from MIT AB and Hallmek AB for the developing of a new
concept

Requests;	Company			
Span widths between 10 and 20 meters	MIT AB			
Main focus should be on the 20 meter structure	MIT AB			
Create a concept containing as few different element types as possible	MIT AB & Hallmek AB			
The elements should be dimensioned to withstand the stress of all span widths	MIT AB			
Minimize the number of roof angles by finding mutual roof inclinations for different span widths	MIT AB & Hallmek AB			
The height of the eave should be able to vary between 4 and 8 meters	MIT AB			
The length of the elements should not exceed 9 meters	Hallmek AB			
One element should not exceed the weight of 50 kilograms	MIT AB			
Simple connections	MIT AB & Hallmek AB			
The columns should be pinned to the ground	MIT AB			
The structure must be a solid construction that is capable of repeated handling	MIT AB			

² Christian Peterson, MIT AB, telephone conference 3/4 2012.

4.2 Design and evaluation criteria

The requirements from section 4.1 together with the knowledge gained from the literature study presented in Chapter 3 and Chapter 4 led to the identification of four main criteria; economy, assembly, flexibility and forces. These criteria apply to temporary and deployable structures in general. To specify the criteria for temporary warehouses, the main criteria were divided into 9 sub-criteria which were weighted against each other in order to rank their importance. The sub-criteria are described in detail in sections 5.2.1-5.2.12.

In the weighing process, the grade is varying from 1-3 where (1) is less important, (2) is equally important and (3) is more important. The results of this weighing process are presented in Table 4.2 in Section 4.3.

4.2.1 Material cost

The material cost refers to the actual quantity of steel in the construction. Due to the low price of steel, this criterion is considered to be of less importance than all other criteria.

4.2.2 Production cost

The production cost refers to the actual cost of manufacturing of the structure. The most important factor is the possibility of a standardized and quick production. Another factor is which type of connection that is used, e.g. a fixed or an adjustable connection.

4.2.3 Number of connections

The number of connections that are needed in the structure will affect the time it takes to assemble/disassemble the structure. The most time consuming step in the assembly is when two or more elements are linked together, and by developing a concept that involves as few connections as possible, a time-efficient system is obtained.

4.2.4 Elements length

The elements length also affects the weight of the elements and with lighter elements, an option to not necessarily use a crane for every step during the assembly. The fact that the elements can be lifted by hand, the time to connect the different elements to each other will be shortened. In this way, a more efficient construction of the structure is gained when the crane solely can be focusing on raising the already assembled steel frames.

4.2.5 Weight

The weight is an important factor for each element as mentioned in Section 4.2.4, and is also of great importance for the whole structure. A light weight structure will be easier to handle during the loading/unloading and assemble/disassemble. Furthermore, light weight elements are preferable at transportation because of less impact of the environment. A noticeable weight loss of the structure can lead to fewer trucks during transportation resulting in reduced transport costs.

4.2.6 Element types

The number of element types will affect the possibility to get a standardized production. Few element types lead to a more standardized production which provides a lower manufacturing cost. Few element types could result in a reduced storage quantity, but at the same time, be able to deliver warehouses of different widths and lengths with short notice. Furthermore, few number of element types will simplify the assembly/disassembly, the risk of human error will decrease and theoretically the erection should be quicker.

4.2.7 Requested volume

The volume at requested height is of importance since the warehouse should not take more space than what is necessary. If a client order a warehouse with a work space of a certain width and height, it is important to meet their requirements without for example significantly increase the width to satisfy the demanded height. This is the case for warehouses with inclined columns, which may result in a reduced demand of the product.

4.2.8 Adjustable height

The adjustable height is to meet costumers various needs of indoor height and to make the concept more flexible to customize the product.

4.2.9 Utilization

The utilization is important with regards to the transportation costs and economical and environmental aspects. The criterion considers the average utilization for the elements in one frame of the 20 meter structures.

4.3 **Result of the weighing process**

As can be seen in Table 4.2 the most important criteria according to the authors are the number of element types followed by the volume at requested height. The values for the rest of the criteria are quite similar except for the adjustable height and the material cost which receives a low impact in the evaluation matrix shown in Table 5.7.

			1	2	3	4	5	6	7	8	9
Economy	1	Material costs		3	3	3	3	3	3	3	3
	2	Production costs	1		1	2	3	3	3	2	1
Assembly	3	Number of connections	1	3		2	2	3	3	2	1
	4	Elements length	1	2	2		2	3	3	1	1
	5	Weight	1	1	2	2		3	2	1	1
Flexibility	6	Element types	1	1	1	1	1		1	1	1
	7	Requested volume	1	1	1	1	2	3		1	1
	8	Adjustable height	1	2	2	3	3	3	3		2
Forces	9	Utilization	1	3	3	3	3	3	3	2	
		Percent	5,6	11,1	10,4	11,8	13,2	16,7	14,6	9,0	7,6

Table 4.2Matrix for weighing the criteria in order to rank their importance.

5 The conceptual design

This chapter begins with a description of the most promising concepts according to the criteria in Section 4.2. In Section 5.2 is the results of the analysis are given, comprising load calculations, stress calculations and the utilization ratio for the elements. Furthermore, a summary of these calculations can be seen in Table 5.1, which is followed by a section of the evaluation of the concepts in order to obtain the best solutions.

5.1 Outline of the concepts

Several different types of two-hinged portal frames were developed during the first design phase. By changing the roof angles, the shape of the structure or the length of the element four final concepts were studied further with regard to the requests presented earlier in Table 4.1.

5.1.1 Duo-pitched roof

The duo-pitched roof consists of two different elements. With these elements it is possible to cover a span of 10, 15 or 20 meters with the same roof inclination. It is also possible to use only one type of element but this will lead to more connections and therefore increase the assembly time and the costs. The roof angle has been set to 20 degrees in the preliminary design but can be changed in order to optimize the functionality. The fact that the roof angle remains the same for all spans simplifies both manufacturing and assembly since no adjustable connections are required. There are two solutions for the columns; separate columns for each span length or 2 and 4 meter elements that can be used for all spans. This will be decided in a later state of the project. The duo-pitched concept can be seen in Figure 5.1.



Figure 5.1 Sketch of the duo-pitched roof concept which shows how the elements can be combined for different span lengths.

The concept can also be adapted on 12 and 18 meter span by adding a notch element, see Figure 5.2. In order to use the same notch element and to keep the same roof inclination the span lengths will alter slightly from the original requirements. This is a simple solution and gives the concept the advantage of covering five different spans with five different element types including the column elements.



Figure 5.2 Sketch of how a notch element can be used to cover 12-meter and 18meter spans.

The duo-pitched roof consists of three different fixed angles through the entire system. As can be seen in Figure 5.1, there is one angle where the column connect to the inclined roof, another angle at the notch and finally one angle of 180 degrees to connect both the columns and the roof elements to each other.

5.1.2 Mansard roof

The mansard roof consists of two different elements, but can also consist of just one element which can be seen in Figure 5.3. With these element lengths it is possible to cover spans of 10, 15 or 20 meter. The roof angles are constant through the spans and are set to 39 and 13 degrees in the first phase but can be altered in order to improve the functionality. The fact that this concept has two different angles generates a choice regarding whether to have one adjustable or two stiff connections. The columns will be designed in the same way as for the duo-pitched roof explained in section 5.1.1.

The mansard roof consists of four different fixed angles through the concept. As can be seen in Figure 5.3, there is one angle where the column connect to the inclined roof, one angle at the first breaking point, another angle at the notch and finally one angle of 180 degrees to connect both the columns and the roof elements to each other.



Figure 5.3 Sketch of the mansard roof concept for the different span widths.

5.1.3 Arced roof

The arced roof without columns consists of just one type of element to cover the spans of 10, 15 and 20 meter. It is possible to design 12- and 18-meter spans in this concept as well and due to the fact that each span width comprises two different angles it is necessary to create a connection that is adjustable to multiple angles. This makes it possible to keep both the number of members and the production cost to a minimum.

The aim with this concept was that the outline should resemble a perfect arch and this led to the absence of columns, see Figure 5.4.

The arced roof with columns also consists of two different angles for every span width, which generates a total of six different angles for the different span widths as can be seen in Figure 5.4. All the angles within one span width, e.g. 20 meters, are equal apart from the angle closest to the foundation.



Figure 5.4 Sketch of the arced roof without any column element concept for the different spans.

Without columns the height of the structure is limited which is a significant problem. A solution to the requirements of the width and height is to simply expand the structure until the requirements are reached. Consequently the structure gets far more volume that does not serve any purpose, see Figure 5.5.


Figure 5.5 Shows a height-width comparison and a possible solution for the indoor height for the arced without column concept.

5.1.4 Arced roof with columns

The arced roof with column is the same concept as mention in section 5.1.3 with the exception that columns were added to increase the available indoor height, as can be seen in Figure 5.6.

The arced roof with columns also consists of two different angles for every span width, which generates a total of six different angles for the different span widths as can be seen in Figure 5.6. All the angles within one span width, e.g. 20 meters, are equal apart from the angle closest to the column.



Figure 5.6 Sketch of the arced roof with column element concept for the different spans.

5.1.5 Summary of the four concepts

A short summary of the concepts are shown in Table 5.1 below. The advantages and disadvantages that are presented illustrate the differences between the concepts. The features that are shared by all concepts, e.g. straight elements, have been left out since they will have no influence in further assessments.

	Duo- pitched	Mansard	Arced	Arced with columns		
Advantages						
- Adaptable for 12- and 18-meter spans	X*	X*	Х	X		
- Few different angles within the concept	Х	Х				
- Significant effects by adding a tie in the 20 meter structure	X	X		x		
- Straight columns to maximize the usage of the width of the structure	X	X		X**		
Reduced wind loads due to the curved shape			X	X		
Disadvantages						
- Adjustable connections required			Х	X		
- Increased width to obtain required height			x	X**		
Different element types for roof and columns	X	X				
High horizontal load due to the straight columns	X	X		x		
* - Is possible by adding an additional element						
** - Small inclination on the second element from the bottom generates a minimum loss of width						

Table 5.1Summary of the concepts advantages and disadvantages.

5.2 Result from the analysis

The different concepts were analyzed in a 2D frame software called *Frame Analysis*. The software is a computer program for calculations of portal frames where both firstand second order effects are taken into account. The aim of the analysis was to determine the bending moments, shear forces and axial forces to calculate the steel area needed for the different concepts and to be able to perform a final sizing of the structural elements. Furthermore, an average utilization ratio for the combined effect of the bending moments and the axial force was calculated. The reason why only the first order effects are presented in this thesis is that since no steel profile was chosen, no second order effects could be calculated. In Frame Analysis a profile could be chosen, but for a preliminary sizing, the second order effect is not relevant. The loads used in the analysis can be seen in Appendix II and the calculations for the weight and the utilization factor in Appendix III. In the following sections the loads and the results from the analysis are presented and discussed for the four concepts.

5.2.1 Loads acting on the structure

The design loads in this project are chosen in accordance to *Eurocode 1*. Since the structure is a one story building and the floor structure will be detached from the load bearing system no imposed loads are taken into account. Since no profile has been chosen the self-weight have been neglected. This means that the structure will be dimensioned according to snow load and wind load. Due to the fact that the structure should be able to function in a wide range of terrains the snow zone is chosen to 3.5kN/m^2 and the terrain type to 0, which is the worst possible terrain according to *Eurocode 1*. The reason to the high value for the snow zone is to be able to provide structures in large parts of Sweden and have the ability to expand the production in northern Europe. Another reason is the problematic winters of 2009/10 and 2010/11 where several roof collapsed partly because of the significant snowfall. More information about these rough winters and reasons for the roof collapses is found in Appendix I. The design will be performed in the ultimate limit state and the serviceability limit state and the load combinations for these can be seen in equation 5.1 and 5.2.

The load combination for ULS:

$$\gamma_{g} * G_{k} + \gamma_{q} * Q_{k,1} + \gamma_{q} * \psi_{0,i} * Q_{k,2}$$
(5.1)

The load combination for SLS:

$$G_k + Q_{k,1} + \psi_{0,i} * Q_{k,2} \tag{5.2}$$

Where:

γ_g	=	Safety factor of 1,35
γ_q	=	Safety factor of 1,5
G_k	=	Selfweight
$Q_{k,1}$	=	Primary variable load
$Q_{k,2}$	=	Secondary variable load
$\psi_{0,i}$	=	Form factor

The worst load cases for the preliminary design have been calculated according to *Eurocode 1* and are as follow:

- *Symmetric snow load* as primary variable load, *wind pressure* as secondary variable load with the form factor 0.3
- *Non-symmetric snow load* as primary variable load, *wind pressure* as secondary variable load with form factor 0.3
- *Wind pressure* as primary variable load, *symmetric snow load* as secondary variable load with form factor 0.8
- *Wind pressure* as primary variable load, *non-symmetric snow load* as secondary variable load with form factor 0.7

• *Non-symmetrical snow load* as primary variable load, *no* secondary variable load due to no wind load

5.2.2 Calculations for the weight and utilization factor

In order to get a comparable weight for the four concepts the cross-sectional area required to keep the maximal stress below the yield strength of the material was calculated. The stresses were calculated for a combination of the axial force and the bending moment for each element according to equation 5.3.

$$\sigma_e = \frac{A_e}{A} + \frac{M_e}{I} * Z \tag{5.3}$$

Where:

σ_e	=	Stress in the element
A_e	=	Axial force in the element
Α	=	Area of the cross section
M_e	=	Bending moment in the element
Ĩ	=	Moment of inertia
Ζ	=	Distance to the natural layer

When calculating the required steel area an I-profile was used. This is not necessarily the best cross-section with regards to e.g. buckling, but will give a comparable result between the concepts. The utilization ratio for each element was calculated by dividing the stress in the element with the maximum stress and finally this was added up in order to find the average utilization ratio for the whole frame. The calculations for the weight and the utilization ratios can be seen in Appendix III.

5.2.3 Duo-pitched roof

The loads will mostly be carried via bending. This means that the moments will be relatively large and the dimensions of the cross-sections will be increased compared to an arch. The moment distribution for the duo-pitched roof can be seen in Figure 5.7. By looking at the moment distribution it is clear that the moments are largest at the corners and the elements closer to the notch will therefore be oversized with regards to the bending moment. The maximum moment, 544 kNm, appears in the load case with symmetric snow load as the primary variable load and wind pressure as the secondary variable load. For the 20 meter structure the utility ration between the right top corner and the left part of the notch is 49.6 %. Furthermore, to use the same elements through the concept the 10 and 15-meter structures, in particular the 10-meter structure will be highly redundant according to the bending moment. Consequently, the material efficiency in the shorter spans will be heavily decreased. By comparing the right top shear force for the 20 meter structure with the left part of the notch shear force for the 10 meter structure, the utility ratio is 12.4 %. For the columns for the same structures, the bending moment utilization is 24%.



Figure 5.7 Moment distribution for the duo-pitched roof concept. The maximum moment, 544 kNm, appears in the load case with symmetric snow load as the primary variable load and wind pressure as the secondary variable load.

The shear force distribution for the duo-pitched roof can be seen in Figure 5.8. The maximum shear force, 160.3 kN, appears in the load case with symmetric snow load as the primary variable load and wind pressure as the secondary variable load. The largest shear force will appear in the corners which make the elements close to the notch redundant. For the 20 meter structure the utility ration between the right top corner and the left part of the notch is 21.3%. Furthermore, to use the same elements through the concept the 10 and 15 meter structures, in particular the 10 meter structure will be highly redundant according to the shear force. By comparing the right top shear force for the 20 meter structure with the left part of the notch shear force for the 10 meter structures, the utility ratio is 9.6%. For the columns for the same structures, the utility ratio is 47.6%.



Figure 5.8 Shear force distribution for the duo-pitched roof concept. The maximum shear force, 160.3 kN, appears in the load case with symmetric snow load as the primary variable load and wind pressure as the secondary variable load.

The axial force distribution for the duo-pitched roof can be seen in Figure 5.8. The maximum axial force, 192.8 kN, appears in the load case with symmetric snow load as the primary variable load and wind pressure as the secondary variable load. The largest axial force will appear in the columns. For the 20 meter structure the utility ratio between the right top corner and the left part of the notch is 51.0%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures will be redundant according to the axial force. By comparing the right top axial force for the 20 meter structure with the left part of the notch shear force for the 10 meter structure, the utility ratio is 24.0%. For the columns for the same structures is the utility ratio 49.3%.



Figure 5.9 Axial force distribution for the duo-pitched roof concept. The maximum axial force, 192.8 kN, appears in the load case with symmetric snow load as the primary variable load and wind pressure as the secondary variable load.

The reaction forces for the duo-pitched roof can be seen in Table 5.2, where the largest horizontal force is -74.8 kN and the largest vertical force is 192.9 kN and they appear in the load case with symmetrical snow load as primary variable load and wind pressure as the secondary variable load.

	Direction	Reaction forces [kN]			
Support	Direction	10 m	15 m	20 m	
Left support	Horizontal	16.5	23.1	30	
	Vertical	88.9	133.3	178.9	
Dight cupport	Horizontal	-35.6	-55	-74.8	
Right support	Vertical	95.0	143.3	192.8	

Table 5.2Horizontal and vertical reaction forces for the duo-pitched roof

The total weight of one frame, when using an I-profile, is 3079 kg and the average utilization ratio when combining the effect of the bending moment and the axial force is 66%.

5.2.4 Mansard roof

The loads will both be carried via bending and axial force. This result in a reduction of the bending moment and an increase of the axial force compared to the duo-pitched roof concept. The moment distribution for the mansard roof can be seen in Figure 5.10. The maximum moment, 429.9 kNm, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load. For the 20 meter structure the utility ratio between the moment at the eave and the moment in the first break point are 20.5%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures, in particular the 10-meter structure will be highly redundant according to the bending moment. Consequently, the material efficiency in the shorter spans will be heavily decreased. By comparing the moment at the left eave for the 20 meter structure with the first breaking point for the 10 meter structure, the utility ratio is 2.0 %. For the columns for the same structures, the bending moment utility ratio is 21.7%.



Figure 5.10 Moment diagram for the mansard roof concept the maximum moment, 429.9 kNm, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load.

The shear force distribution for the mansard roof can be seen in Figure 5.11. The maximum shear force, 130.8 kN, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load. The largest shear force will appear at the left eave which make the elements close to the notch redundant. For the 20 meter structure the utility ration between the left eave and the left part of the notch is 29.7%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures, in particular the 10-meter structure will be redundant according to the shear force. By comparing the shear force at the left eave for the 20 meter structure with the left part of the notch shear force for the 10 meter structure, the utility ratio is 14.8%. For the columns for the same structures, the utility ration is 50.1%.



Figure 5.11 Shear force diagram for the mansard roof concept. The maximum shear force, 130.8 kN, appears in the load case with unsymmetrical snow load as the primary variable load and no secondary variable load.

The axial force distribution for the mansard roof can be seen in Figure 5.12. The maximum axial force, 208.2 kN, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load. The largest axial force will appear in the columns. For the 20 meter structure the utility ration between the left eave and the right part of the notch is 32.2%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures will be redundant according to the axial force. By comparing the left eave axial force for the 20 meter structure with the right part of the notch shear force for the 10 meter structure, the utility ratio is 16.1%. For the columns for the same structures, the utility ratio is 50%.





The reaction forces for the mansard roof can be seen in Table 5.3, where the largest horizontal force is -49.3 kN and the largest vertical force is 208.2 kN and they appear in the load case with unsymmetrical snow load as primary variable load acting as the only load.

	Direction	Reaction forces [kN]			
Support	Direction	10 m	15 m	20 m	
Left support	Horizontal	10.8	16.4	49.3	
	Vertical	84.2	109.1	208.2	
Dight cupport	Horizontal	-26.5	-41.1	-49.3	
Right support	Vertical	60.6	121.9	146.8	

The total weight of one frame, when using an I-profile, is 2538 kg and the average utilization ratio when combining the effect of the bending moment and the axial force is 69%.

5.2.5 Arced roof

The loads will mostly be carried via axial force when the load is symmetrical. When subjected to unsymmetrical snow load or wind, bending moment will occur. The moment distribution for the arced roof can be seen in Figure 5.13. The maximum

moment, 186.6 kNm, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load. For the 20 meter structure the utility ratio between the moment at the first break point and the notch is 13.3%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures, in particular the 10-meter structure will be highly redundant according to the bending moment. Consequently, the material efficiency in the shorter spans will be heavily decreased. By comparing the moment at the first break points for the 20 meter and at the notch for the 15 meter structure is 10.4%.



Figure 5.13 Moment diagram for the arced roof without column concept. The maximum moment, 186.6 kNm, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load.

The shear force distribution for the arced roof can be seen in Figure 5.14. The maximum shear force, 41.9 kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load. Noticeable for this concept is that the maximum shear force not is found for the 20 meter but for the 15 meter structure. The largest shear force will appear at the first break point which makes the elements close to the notch redundant. For the 15 meter structure the utility ratio between the first break point and the right support is 57.8%. By comparing the shear force at the first break point for the 15 meter structure with the right support shear force for the 10 meter structure, the utility ratio is 23.9%.



Figure 5.14 Shear force diagram for the arced roof without column concept. The maximum shear force, 41.9 kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load.

The axial force distribution for the arced roof can be seen in Figure 5.15. The maximum axial force, 197.7 kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load. The largest axial force will appear in the elements at the supports. For the 20 meter structure the utility ration between the right support and the notch is 30.7%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures will be redundant according to the axial force. By comparing the right support axial force for the 20 meter structure with the notch axial force for the 10 meter structure, the utility ratio is 12.4%.



Figure 5.15 Axial force diagram for the arced roof without column concept. The maximum axial force, 197,7kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load.

The reaction forces for the arced roof can be seen in Table 5.4, where the largest horizontal force is -74.8 kN and the largest vertical force is 175.5 kN and they appear in the load case with unsymmetrical snow load as primary variable load and wind pressure as the secondary variable load.

	Direction	Reaction forces [kN] 10 m 15 m 20		N]
Support	Direction			20 m
Loft cupport	Horizontal	21.9	36.5	48.8
Left support	Vertical	60.4	120.8	175.7
Dight support	Horizontal	-32.9	-53.3	-74.8
Right support	Vertical	45.5	82.6	116.5

Table 5.4Horizontal and vertical reaction forces for the arced roof

The total weight of one frame, when using an I-profile, is 1439 kg and the average utilization ratio when combining the effect of the bending moment and the axial force is 74%.

5.2.6 Arced roof with columns

The loads will both be carried via bending and axial force. The moment distribution for the arced roof with columns can be seen in Figure 5.16. The maximum moment, 334.9 kNm, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load. For the 20 meter

structure, the utility ration between the moment at the second and fourth break point to the right is 13.8%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures, in particular the 10-meter structure will be highly redundant according to the bending moment. Consequently, the material efficiency in the shorter spans will be heavily decreased. By comparing the moment at the second break point for the 20 meter and the moment at the notch for the 10 meter structure, the utilization ratio will be 12.7%.



Figure 5.16 Moment diagram for the arced roof with column concept. The maximum moment, 334.9 kNm, appears in the load case with unsymmetrical snow load as the primary variable load and wind pressure as the secondary variable load.

The shear force distribution for the arced roof with columns can be seen in Figure 5.17. The maximum shear force, 62.6 kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load. The largest shear force will appear at the right support which makes the elements close to the left support redundant. For the 20 meter structure the utility ratio between the shear force at the right support and at the notch is 31.9%. By comparing the shear force at the right support for the 20 meter structure with the shear force at the notch for the 10 meter structure, the utility ratio is 25.7%.



Figure 5.17 Shear force diagram for the arced roof with column concept. The maximum shear force, 62.6 kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load.

The axial force distribution for the arced roof with columns can be seen in Figure 5.18. The maximum axial force, 176.8 kN, appears in the load case with symmetrical snow load as the primary variable load and wind pressure as the secondary variable load. The largest axial force will appear in the elements at the supports. For the 20 meter structure the utility ration between the right support and the notch is 18.7%. Furthermore, to use the same elements through the concept the 10 and 15-meter structures will be redundant according to the axial force. By comparing the right support axial force for the 20 meter structure with the notch axial force for the 10 meter structure, the utility ratio is 7.0%.





The reaction forces for the arced roof with column can be seen in Table 5.5, where the largest horizontal force is 162.6 kN and the largest vertical force is 168.2 kN and they appear in the load case with symmetrical snow load as primary variable load and wind pressure as the secondary variable load.

C .	Disseties	Reaction forces [kN]			
Support	Direction	10 m	15 m	20 m	
Left support	Horizontal	-9.1	1.7	12.4	
	Vertical	38.1	100.4	153.7	
Right support	Horizontal	27.3	-43.6	162.6	
	Vertical	59.1	116.4	168.2	

Table 5.5Horizontal and vertical reaction forces for the arced roof with columns

The total weight of one frame, when using an I-profile, is 2356 kg and the average utilization ratio when combining the effect of the bending moment and the axial force is 63%.

5.2.7 Summary of the results

A summary of the results from the analysis of the four concepts can be seen in Table 5.6 below.

Table 5.6Summary of the results from the analysi	s of the four concepts.
--	-------------------------

		Duo- pitched	Mansard	Arced	Arced with columns
Maximum bending moment	kNm	544.0	429.9	186.6	334.9
Maximum shear force	kN	160.3	130.8	41.9	62.6
Maximum axial force	kN	192.8	208.2	197.7	176.8
Maximum horizontal reaction force	kN	74.8	49.3	74.8	162.6
Maximum vertical reaction force	kN	192.9	208.2	175.5	168.2
Lowest utilization for the 10 meter structure with regards to bending moment	%	12.4	2.0	10.4	12.7
Lowest utilization for the 10 meter structure with regards to shear force	%	9.6	14.8	57.8	31.9
Lowest utilization for the 10 meter structure with regards to axial force	%	24.0	16.1	12.4	7.0
Average utilization in one frame for the 20 meter structure due to combination of axial force and bending moment	%	66	69	74	63
Total weight of one frame	kg	3079	2538	1439	2356

5.3 Evaluation of the concepts

The second step in the evaluation process is to assess the concepts with regards to the criteria. Due to the fact that the duo-pitched and the mansard concept may consist of either one or two roof elements they have to be evaluated separately. Each concept are then graded on the scale from 1-4, were (1) is not good, (2) is decent, (3) is good and (4) is excellent. The grades are then multiplied with the weighing factor from Table 4.2, and summed together. As can be seen in Table 5.7 the concept with the *arced roof with columns* is the most promising followed by the *arced roof*.

		Duo- pitched, short	Duo- pitched, long	Mansard, short	Mansard, long	Arced	Arced with columns
Element types	16,7%	2	1	2	1	4	4
Requested volume	14,6%	4	4	4	4	1	4
Weight	13,2%	1	1	2	2	4	3
Element length	11,8%	3	2	3	2	3	3
Production costs	11,1%	2	1	2	1	4	4
Number of connections	10,4%	1	4	1	4	3	3
Adjustable height	9,0%	4	4	4	4	1	1
Utilization	7,6%	2	2	3	3	4	2
Material costs	5,6%	2	2	2	2	4	3
Percent		58,9%	56,8%	64,1%	62,0%	76,8%	79,2%

Table 5.7Evaluating matrix for the concepts showing the percentage from the
weighing matrix and the grade from 1-4, where 1 is not good and 4 is
excellent, for the different concepts

The duo-pithed concepts receive the lowest score both when using long and short elements. The main reason for this is that the concepts consist of to many different element types due to the fact that different elements are used for the roof and the columns. The design with short roof elements will consist of 3 element types and the design with long and short elements will consist of 4 element types. Compared to the arced concepts this is considered to be a high number which explains the low grades. The number of elements is closely related to the production cost and the concepts therefore receive low scores in this criterion as well. Another reason is that the total weight of one frame will be high which will affect both the transportation and the

assembly. The main advantages with the duo-pitched designs are the requested volume and the adjustable height which will increase the flexibility of the system.

The mansard concepts receive almost as low scores as the duo-pitched. The reason that the score is slightly higher is the fact that the total weight of the frame is lower and that the utilization factor is slightly higher. As for the duo-pitched concept the mansard concept with short elements is more promising. The use of short elements will increase the number of connections but at the same time give a more standardized production and lighter elements which will be easier to carry and assemble.

The arced concepts obtain high scores in most categories. The two different designs are very similar which explains the resemblance in the results from the evaluation. The most important factor for the high total score is the fact that the concepts only consists of one element type which is considered to be the most important design criterion. As mentioned before the number of element types are closely related to the production costs which explains the high grade in this category compared to the other concepts. The main factor contributing to lowering the overall score is the lack of the possibility to adjust the height of the structure which will lower the flexibility of the structures significantly. The arced concept without columns surpasses the arced with columns with regards to weight, utilization and material costs. The utilization and the material costs are the two least important design criteria and due to the fact that the arced with columns concept has the ability to deliver the requested volume, which is considered to be very important, gives this concept the slight advantage and should be considered the most promising solution.

6 Connection examples

Two of the most important factors for temporary warehouses are the erection time and the ease of assembly/disassembly. Hence, the number of connections and in particular, the type of connections is of great importance since they represent a great part of the assembly. Connections that involve large number of steps e.g. fastening of nuts or poorly positioned bolts should be avoided as they take longer time to execute. An appropriate connection can be designed in numerous ways and in the following sections a few example are illustrated.

6.1 Bolted connections

For permanent warehouses, the most common connection is the bolted connection due to the simple design and the low manufacturing cost. Two examples of bolted connections can be seen in see Figure 6.1. The erection time for the permanent warehouse with bolted connections is longer compared to other type of connections, but since a permanent structure is assembled only one time, the extra cost for the longer erection time is recovered by the low production cost of the connection. For a temporary structure, which could be assembled up to 30 times during its lifetime, this type of connection may not the most economical regarding the costs of the assembly. The assembly cost bould be multiplyed up to 30 times the original cost, and that is why more complex and expensive solutions could be a better alternative.



Figure 6.1 An example of a beam to beam connection (CivilEA, 2011) and a column to beam connection (Construction53, 2011) with bolts which is frequently used in structures today.

Another example of a bolted connection is the Pin-Fuse Joint, see Figure 6.2, which makes it possible for structures to roll during an earthquake and then return to its original stage when the aftershocks are finished. Since there is a low risk of earthquakes in northern Europe this type of beam will not be necessary, especially not for temporary warehouse buildings. But the idea with rounded corners could make the beam adjustable for different roof angles which is an advantage compared to the bolted connection in Figure 6.1. There are still several bolts in this connection, but the structure gets more flexible and the number of different elements will be reduced if a single connection is able to adjust to multiple angles.



Figure 6.2 A Pin-Fuse Joint used to counteract earthquakes by letting the building roll. After the aftershocks the connection is returning to its original state (ARCHITECT, 2009).

6.2 Snap- and adjustable connections

For temporary structures, and in particularly for modulus concepts, a more complex connection may be a more cost-effective solution. This is due to the fact that these types of buildings should be erected several times during its lifetime. The production costs for the connections are not as important since the connections are reusable for as many times as the temporary structure is erected. Examples of adjustable- and more complex connections than the bolted connections are presented below.

6.2.1 Snap connections

A snap connection is the fastest solution to link two or more elements together. The theory is to precision cut the elements so they by force can be joined together as in Figure 6.3. The gap from the precision cuts will reduce during the assembly and the clasp snaps back like a spring to the original position if mounted correctly. If the assembly is not perfectly preformed, the clasp will not fit and only the friction will hold the elements together, which is not enough for these types of connections.



Figure 6.3 Precision cut elements joined together by force to create an effective connection (Houseofmods, 2011).

To manufacture a snap connection like in Figure 6.3, an advanced machinery is need to be able to precision cut every part exactly the same. A similar type of connection, but with a simpler design is visualized in Figure 6.4 where the elements are connected by wedges in the upper right- and lower left corners. When comparing the snap-fit connections, it is noticeable that the connection in Figure 6.4 is easier to separate, which is an important aspect of temporary buildings. The dimensions must be decided from case to case, but the hocking arm should not be smaller than 15 mm (Aluminiumdesign, 2012).



Figure 6.4 Example of a snap-fit connection where the two elements are joined together by wedges in the corners (Aluminiumdesign, 2012)

An alternative solution to the snap connection is the friction connection. One of the most well-known friction connections is LEGO, see Figure 6.5, where the different pieces fit together regardless of size due to the standardized manufacturing process. The idea to erect a temporary warehouse with the principle of LEGO with its fast assembly/disassembly, simple connections and standardized manufacturing is appealing.



Figure 6.5 A well-known friction connection is LEGO where the friction keeps the pieces in to place (3DDeSiGn, 2011).

6.2.2 Adjustable connections

Similar to the solution of the Pin-Fuse Joint in Figure 6.2 but without the bolts, the angled connection in Figure 6.6 allows various angles up to 180 degrees. The connection can also be used as heavy-duty hinges, rigid angles element and pinned element (BearingEngineers, 2012a). Adjustable connections are flexible which is favorable for the temporary warehouse concepts with many different angles in the roof structure.



Figure 6.6 An angled connection for various angles up to 180 degrees (BearingEngineers, 2012a).

Another example of an adjustable connection more suitable for larger structures is shown in Figure 6.7, where the steel profile is joined together with the connection by a rotating axis which provides this type of connection can be used for a wide range of angles.



Figure 6.7 The connection is slid onto the I-beam and is kept in place by the two nuts. The steel profile is allowed to rotate freely around the axis (WallaceCranes, 2009).

Furthermore, another type of adjustable connection is parallel profile shaped like a truss which can be seen in Figure 6.8. The web diagonals slide into position in the tracks in both upper and lower flange. The diagonals are kept in place by the fasteners and are clamped by tightening of an internal screw. If it is possible to connect the parallel profile to other profiles in different angles e.g. between column and roofs or in different part of the roof, this would be a very competitive choice for all temporary structures.



Figure 6.8 An example of parallel profile connection as a truss member (BearingEngineers, 2012b).

7 Discussion and conclusions

The purpose of this Master's Thesis was to design a modular system for temporary warehouses with various span widths. A set of requests were given by MIT AB and Hallmek AB and together with the design criteria we created four promising solutions which were investigated further. The main boundary conditions were that the concept should be developed for structures with a span width of 10, 15 and 20 meters and an eave height of 4-8 meters. We have assumed that the given span widths are the maximum ground area that the structures are allowed to use. Another approach would have been to see the given span widths and eave heights as a required working space. Hence, the arced concept would have fulfilled the requirement for the requested volume but would also occupy a much larger ground area and be subjected to larger loads. We believe that our approach is reasonable since a temporary structure should not occupy more ground area than necessary in order to keep its flexibility with regards to location.

If allowing the dimensions to slightly differ from the given boundary conditions, all concepts could be further optimized to better fit the design criteria. An example is the duo-pitched roof which could contain the same element type for both the columns and the roof by using the roof element as a column element instead. This would have a positive impact for the number of element types, but also increase the number of connections and reduce the flexibility with regards to the adjustable height.

The weighing of the design criteria used in the evaluation were based on our own thoughts and information given from representatives from MIT AB and Hallmek AB. No actual investigation with regards to economic aspects and the time of assembly/disassembly has been made. The grades used in the evaluation varied from 1-4 but for some criteria it might have been better to use a larger grading scale to get a fairer result. On the other hand the grading scale used allowed us to really think about the pros and cons for each concept.

The results from the evaluation clearly show that the two concepts with the arc shaped roof are the best solutions. The final result for these two was very close which was expected since they basically are the same. The fact that the arced roof with columns can deliver the requested volume makes it the winning concept. The fact that the winning concept is quite high in the notch makes it ideal for storing bulk material such as salt, pellets or grain because large machinery can move freely within the structure. Furthermore, if the arced roof without columns is more suitable for the client the straight columns can simply be removed which is great with regards to flexibility.

The winning concept only includes one element type which is ideal for a standardized production. The element weight calculated in the Thesis for an I-beam clearly shows that the amount of material needs to be heavily reduced in order to reach the requirement of 25 kilograms per worker. Hence, the element should be constructed as a truss which will reduce the weight. A suitable profile, according to Åke Lundh at Hallmek AB, for the truss members is a squared KKR-profile which has a great resistance to buckling.

To keep the number of different parts to a minimum within the concept an adjustable connection should be used. The connection must be adjustable for at least the six angles in the winning concept but even more angles are preferable to increase the flexibility in the system. Furthermore, the connection should include as few bolts as possible in order to speed up the assembly/disassembly.

The snow and wind loads used in the design might be unnecessarily large for a temporary structure but the fact that this structure should be used for many years makes it more like a permanent structure with the exception that it is re-locatable. The distance between the frames was set to 4 meters in our design but this can of course be changed. It is important to note that if it is chosen to use different spacing between the frames for different snow zones it will require multiple sets of longitudinal struts or one set that has adjustable struts.

An interesting option to a modulus system would be to make the structure deployable which would decrease the erection time significantly. The disadvantage with a deployable structure is the complexity in the design due to the numerous amount of moving members and joints. For this project a deployable system would be even more complex due to the fact that it has to be adaptable for multiple span widths. It would be an interesting solution but with regards to the difficult design we do not think it would be a profitable investment.

The main purpose with developing a modulus system for temporary warehouses is to reduce the cost by reducing the inventory costs, the time of delivery and to get a standardized production. The main disadvantage with the modulus system is that the utilization ratio for the 10 and 15 meter structures will be very low. This could be solved by adding a tie in the 20 meter structure which will reduce the dimensions of the elements heavily, hence improving the utilization ratio significantly. The solution with a tie has been neglected in the analysis since the clients are against it because of the large risks of collisions which will cause failure of the structure.

However, a thorough investigation regarding the profitability of a modulus system compared to non-modulus systems should be performed. The investigation should comprise the economic benefits gained from reducing the inventory costs, the time of delivery and to get a standardized production compared to the loss of time and money due to the additional weight.

References

- 3DDESIGN. 2011. *Major Lego project* [Online]. Available: <u>http://3ddesign-gmccone.blogspot.se/2011/05/major-lego-project.html#!/2011/05/major-lego-project.html</u> [Accessed June 5 2012].
- ACCESSSTEEL 2012a. Conceptual design of truss and column solutions.
- ACCESSSTEEL 2012b. Design of portal frames using fabricated welded sections.
- ACCESSSTEEL 2012c. Overview of structural systems for single-storey buildings.
- ALUMINIUMDESIGN. 2012. *Joining aluminium* [Online]. Available: <u>http://www.aluminiumdesign.net/design-support/joining-aluminium/#toc-latitudinal-joining</u> [Accessed June 5 2012].
- ALUSTAGE. 2012. Arch roof AR [Online]. Available: <u>http://alustage.eu/alustage,www,strona,produkty,dzial,3,produkt,24,lang,en.ht</u> <u>ml</u> [Accessed June 4 2012].
- ARCHITECT. 2009. *Pin-Fuse Joint* [Online]. Available: <u>http://www.architectmagazine.com/technology/pin-fuse-joint.aspx</u> [Accessed June 5 2012].
- BEARINGENGINEERS. 2012a. Angled Connections [Online]. Available: <u>http://www.bearingengineers.com/products/product.php?id=266</u> [Accessed June 5 2012].
- BEARINGENGINEERS. 2012b. *Parallel Profile Connections* [Online]. Available: <u>http://www.bearingengineers.com/products/product.php?id=272</u> [Accessed June 5 2012].
- BENGTSSON, Å. & PETTERSSON, O. 1972. Statiskt obestämda fackverk, balkar, ramar. AB Byggmästarens Förlag: Stockholm, Sweden.
- BERNARDONI, A., TADDEI, M. & ZANON, E. 2005. I ponti di Leonardo -Leonardo 's bridges, Italy.
- CIVILEA. 2011. *Beam to beam connection* [Online]. Available: <u>http://forum.civilea.com/thread-28620.html</u> [Accessed June 5 2012].
- CONSTRUCTION53. 2011. *Details of Steel Framing* [Online]. Available: <u>http://www.construction53.com/2011/04/details-of-steel-framing/</u> [Accessed June 5 2012].
- GANTES, C. J. 2001. *Deployable structures: analysis and design*, WIT Press, , Southampton: UK.
- GORLIN, W. B. 2009. *Temporary Structures Need Wind-Load Standards. In:* STRUCTURE (ed.). C³ ink: USA.
- HOUSEOFMODS. 2011. *Snap connection* [Online]. Available: houseofmods.com [Accessed June 5 2012].
- KIPER, G. & SÖYLEMEZ, E. 2009. *Deployable Space Structures*. Middle East Technical University: Ankara, Turkey.

- KNIGHT, N. F., JR, ELLIOTT, K. B., TEMPLETON, J. D., SONG, K. & RAYBURN, J. T. 2012. FAST Mast Structural Response to Axial Loading: Modeling and Verification. Langley Research Center, Marshall Space Flight Center, USA.
- REICHEL, A. 2007. *Building with steel : details, principles, examples, Birkhäuser, Basel, Switzerland.*
- SALTER, P. R., MALIK, A. S. & KING, C. M. 2004. *Design of Single-Span Steel Portal Frames to BS 5950-1:2000.* The Steel Construction Institute: Berkshire, UK.
- SBI. 2012. *Cold-formed profiles* [Online]. Available: <u>http://www.sbi.se/omraden/o_kat_dokument.asp?mId=3&kId=82&subKId=88</u> <u>&mgrp=0</u> [Accessed May 15 2012].
- SCI 2008. Best Practice in Steel Construction Industrial Buildings Guidance for Architects, Designers & Constructors. The Steel Construction Institute.
- SCI. 2009. *The European Steel Design Education Programme* [Online]. Available: <u>http://www.fgg.uni-lj.si/kmk/esdep/master/wg07/l1200.htm#SEC_1</u> [Accessed March 21 2012].
- SHEPHERD, B. 2005. Bridges to go Temporary Bridge Structures. Australia.
- STÅLBYGGNADSINSTITUTET 2008. *Stålbyggnad, 6:e upplagan (Steel construction, Vol 6. In Swedish),* Stålbyggnadsinstitutet (The Swedish Institute of Steel Construction), Stockholm, Sweden.
- THINKDEFENCE. 2012. UK Military Bridging Equipment (The Bailey Bridge) [Online]. Available: <u>http://www.thinkdefence.co.uk/2012/01/uk-military-bridging-equipment-the-bailey-bridge/</u> [Accessed June 4 2012].
- TIBERT, G. 2002. *Deployable Tensegrity Structures for Space Applications*. Royal Institute of Technology: Stockholm, Sweden.
- UNIBRIDGE. 2009. UNIBRIDGE Modular steel bridge uses [Online]. Available: <u>http://www.unibridge.net.au/en/about/unibridge_modular_steel_bridge_uses/</u> [Accessed June 4 2012].
- WALLACECRANES. 2009. *Design Elegance* [Online]. Available: <u>http://www.wallacecranes.com/elegance.htm</u> [Accessed June 5 2012].

Appendix I - The problematic winter of 2009/10

During the winter of 2009/10 a large amount of snow fell all over Sweden. In some parts of the country the snow depth was the highest ever measured. In the same period of time the temperature was below zero without any thaw and the wind was mainly blowing in a north-east direction. The combination of the low density snow, due to the low temperature, and the constant wind direction caused large snow drifts to form.

Calculations by the Swedish Institute of Metrology and Hydrology was performed due to this problematic winter and are based on the maximum snow depth from the area of interest and a snow density of 280 kg/m². These calculations are considered to be on the safe side due to the heavy density of the snow which usually only occurs in the end of the snow season. Therefore SMHI stated that the values for the snow load on the ground in different parts of Sweden are an overestimation of the actual snow load, as can be seen in Table A.1.

The snowfall of 2009/10 and 2010/11was significant but not extreme, and compared to BFS 2006:21 there was only the city of Lysekil which had a higher value than the norm value. The snow was only a contributing factor to the approximately 200 roof collapses that occurred during these winters. About 75% of these collapses depended on errors in the planning and execution of the buildings. However, the snow became the catalyst that revealed the errors.

For example, before the introduction of EKS in January 2011, the unsymmetrical loads on roofs with a slope less than 15 degrees was neglected. During these winters several cases were found with large amount of snow on the leeward side and almost no snow at all on the windward side, even on roofs with slopes less than 15 degrees.

The question is whether the number of roof collapses had been the same if the unsymmetrical load cases had been noted earlier, even though the unbalanced snow load was greater than the *Eurocode* states.

Even shoveling of the roof can be a contributing factor where ten percent of the 200 collapses caved in during the time of shoveling. This also depends on the unbalanced snow load formed if not the shoveling is done properly (Boverket, 2011, Johansson et al., 2011).

Table A.1Snow load on the ground based on SMHI's measurements of the snow
depth and with a snow density of 280kg/m². This is compared with, for
this winter the current standards and earlier norm values (Johansson et
al.,2011).

Station	Maximum snow depth 2010	Calculated snow load on the ground according to SMHI		Snow load according to BFS 2006:21	Snow load according to BFS 1998:39
	(cm)	(k	N/m²)	(kN/m^2)	(kN/m^2)
Lund	29		0,81	1,5	1
Osby	52		1,25	1,5 -2,0	1,5
Ljungby	64	*	1,54	2,0 - 2,5	1,5
Karlshamn	38		1,06	1,5 - 2,0	1,5
Ronneby (Bredåkra)	50		1,4	2	1,5
Kalmar	45		1,26	2,0 - 2,5	1,5
Varberg	45	*	1,26	1,5 - 2,0	1,0 - 1,5
Borås	67	*	1,61	2,0 - 2,5	1,5
Jönköping (Flahult)	94	*	2,26	2,5 - 3,0	1,5 - 2,0
Oskarshamn	82	*	2,30	2,5	2
Lysekil d	81	**	2,27	1,5	1,0 - 1,5
Vänersborg	57		1,37	2	1,5 - 2,0
Linköping (Malmslätt)	68		1,63	2	2
Kristinehamn	58		1,39	2,5	2
Örebro	85	*	2,04	2,5	2
Stockholm	50		1,2	2	2
Härnösand	86		2,06	3,5	3,5
Piteå	79		1,82	3,0 - 3,5	3
Luleå flygplats	77		1,77	3	3
* - The snow load** - The snow load	I was exceeded according to the second	ording t ording t	o BFS 2006 o BFS 1998	:21 :39	

Reference

JOHANSSON, C.-J., LINDGREN, C., NILSSON, C. & CROCETTI, R. 2011. Takras vintrarna 2009/2010 och 2010/2011 Orsaker och förslag till åtgärder

Appendix II- Load calculations

1. Duo-pitched

Geometry

The distance between the frames have been set to 4 meters



Snow load

The case with exceptional snow loads does not occur in Sweden has therefore been disregarded in the calculations.

The snow zone has been set to 3.5 since this covers most parts of Sweden.

Characteristic snow load, sk: $s_k := 3.5 \frac{kN}{m^2}$ The topography is normal $C_e := 1$ Thermal coefficient, C_t : $C_t := 1$



 $\mu_{1.20} := 0.8$

$$\underline{Load \ case \ 1}$$

$$S_{1.20} \coloneqq \mu_{1.20} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}$$

$$S_{2.1.20} \coloneqq \mu_{1.20} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}$$

$$S_{2.1.20} \coloneqq \mu_{1.20} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}$$

$$S_{2.2.20} := 0.5 \mu_{1.20} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 5.6 m \cdot \frac{kN}{m^2}$$

Width 15 m:

 $\mu_{1.15} := 0.8$

<u>Load case 1</u>

 $\mathbf{S}_{1.15} \coloneqq \boldsymbol{\mu}_{1.15} \cdot \mathbf{C}_{e} \cdot \mathbf{C}_{t} \cdot \mathbf{s}_{k} \cdot \mathbf{L}_{F} = 11.2 \mathbf{m} \cdot \frac{\mathbf{kN}}{\mathbf{m}^{2}}$

$$S_{2.1.15} := \mu_{1.15} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}$$
$$S_{2.2.15} := 0.5\mu_{1.15} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 5.6m \cdot \frac{kN}{m^2}$$

 $\begin{array}{l} \underline{Width \ 10 \ m:} \\ \mu_{1.10} \coloneqq 0.8 \\ \underline{Load \ case \ 1} \\ s_{1.10} \coloneqq \mu_{1.10} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2} \end{array}$

Load case 2

$$S_{2.1.10} \coloneqq \mu_{1.10} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}$$
$$S_{2.2.10} \coloneqq 0.5\mu_{1.10} \cdot C_e \cdot C_t \cdot s_k \cdot L_F = 5.6m \cdot \frac{kN}{m^2}$$

Wind load

Terrain type: Zone 0

For dimensioning the load bearing structure the Cpe.10 values should be used



Width 20 m:

Characteristic wind pressure for a wind speed of 26 m/s

 $h < d \rightarrow z_{e,20} := h_{20} = 11.64 \text{m} \rightarrow Zone \ 0 \rightarrow q_{p0.20} := 1.22 \cdot \frac{\text{kN}}{\text{m}^2}$

Pressure coefficients on the external walls

$$\frac{h_{20}}{d_{20}} = 0.556 \quad --> \quad C_{pe.D.20} := 0.74$$
$$C_{pe.E.20} := -0.39$$

Wind load on the columns

 $F_{\text{wind}.\text{D.20}} \coloneqq C_{\text{pe}.\text{D.20}} \cdot q_{\text{p0.20}} \cdot L_{\text{F}} = 3.611 \cdot \frac{\text{kN}}{\text{m}}$ $F_{\text{wind}.\text{E.20}} \coloneqq C_{\text{pe}.\text{E.20}} \cdot q_{\text{p0.20}} \cdot L_{\text{F}} = -1.903 \cdot \frac{\text{kN}}{\text{m}}$

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.
<u>Width 15 m:</u> Characteristic wind pressure for a wind speed of 26 m/s

 $h < d \longrightarrow z_{e.15} := h_{15} = 8.729 m \longrightarrow Zon 0 \longrightarrow q_{p0.15} := 1.18 \cdot \frac{kN}{m^2}$

Pressure coefficients on the external walls

$$\frac{h_{15}}{d_{15}} = 0.582 \quad --> \quad C_{pe,D,15} := 0.740$$
$$C_{pe,E,15} := -0.390$$

Wind load on the columns

$$F_{\text{wind.D.15}} := C_{\text{pe.D.15}} \cdot q_{\text{p0.15}} \cdot L_{\text{F}} = 3.493 \cdot \frac{\text{kN}}{\text{m}}$$
$$F_{\text{wind.E.15}} := C_{\text{pe.E.15}} \cdot q_{\text{p0.15}} \cdot L_{\text{F}} = -1.841 \cdot \frac{\text{kN}}{\text{m}}$$

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.

$$e_{15} := 2 \cdot h_{15} = 17.458 \text{m}$$
 $L_{15} := 7.98 \text{m}$

$$\frac{Case \text{ one - The wind is pressing}}{C_{pe.F.15.1} := 0.325}$$

$$C_{pe.G.15.1} := 0.325$$

$$C_{pe.H.15.1} := 0.25$$

$$C_{pe.J.15.1} := 0$$

$$F_{wind.F.15.1} := C_{pe.F.15.1} \cdot q_{p0.15} \cdot L_{F} = 1.534 \cdot \frac{kN}{m}$$

$$F_{wind.H.15.1} := C_{pe.H.15.1} \cdot q_{p0.15} \cdot L_{F} = 1.18 \cdot \frac{kN}{m}$$

$$F_{wind.J.15.1} := C_{pe.J.15.1} \cdot q_{p0.15} \cdot L_{F} = 0 \cdot \frac{kN}{m}$$

$$F_{wind.J.15.1} := C_{pe.J.15.1} \cdot q_{p0.15} \cdot L_{F} = 0 \cdot \frac{kN}{m}$$

Case two - The wind is lifting $C_{pe.F.15.2} := -0.63$ $C_{pe.G.15.2} := -0.67$ $C_{pe.J.15.2} := -0.67$ $C_{pe.H.15.2} := -0.23$ $C_{pe.I.15.2} := -0.4$ $F_{wind.F.15.2} := C_{pe.F.15.2} \cdot q_{p0.15} \cdot L_{F} = -2.974 \cdot \frac{kN}{m}$ $F_{wind.H.15.2} := C_{pe.H.15.2} \cdot q_{p0.15} \cdot L_{F} = -1.086 \cdot \frac{kN}{m}$

$$F_{wind.J.15.2} := C_{pe.J.15.2} \cdot q_{p0.15} \cdot L_F = -3.162 \cdot \frac{kN}{m}$$

kN

$$F_{\text{wind.I.15.2}} := C_{\text{pe.I.15.2}} \cdot q_{\text{p0.15}} \cdot L_F = -1.888 \cdot \frac{KN}{m}$$

<u>Width 10 m:</u> <u>Characteristic wind pressure for a wind speed of 26 m/s</u>

$$h < d \longrightarrow z_{e.10} := h_{10} = 5.82 \text{ m} \longrightarrow Zon 0 \longrightarrow q_{p0.10} := 1.06 \frac{\text{kN}}{\text{m}^2}$$

. . .

Pressure coefficients on the external walls

$$\frac{h_{10}}{d_{10}} = 0.582 \quad --> \quad C_{pe.D.10} := 0.74$$
$$C_{pe.E.10} := -0.39$$

Wind load on the columns

$$F_{\text{wind.D.10}} \coloneqq C_{\text{pe.D.10}} \cdot q_{\text{p0.10}} \cdot L_{\text{F}} = 3.138 \cdot \frac{\text{kN}}{\text{m}}$$
$$F_{\text{wind.E.10}} \coloneqq C_{\text{pe.E.10}} \cdot q_{\text{p0.10}} \cdot L_{\text{F}} = -1.654 \cdot \frac{\text{kN}}{\text{m}}$$

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.

-->
$$e_{10} := 2 \cdot h_{10} = 11.64m$$
 $L_{10} := 5.32m$

Case one - The wind is pressing

 $C_{pe.F.10.1} := 0.325$ $C_{pe.G.10.1} := 0.325$

 $C_{pe.H.10.1} := 0.25$

 $C_{pe,J,10,1} := 0$

 $C_{pe.I.10.1} := 0$

 $F_{wind.F.10.1} := C_{pe.F.10.1} \cdot q_{p0.10} \cdot L_{F} = 1.378 \cdot \frac{kN}{m}$ $F_{wind.H.10.1} := C_{pe.H.10.1} \cdot q_{p0.10} \cdot L_{F} = 1.06 \cdot \frac{kN}{m}$ $F_{wind.J.10.1} := C_{pe.J.10.1} \cdot q_{p0.10} \cdot L_{F} = 0 \cdot \frac{kN}{m}$ $F_{wind.I.10.1} := C_{pe.I.10.1} \cdot q_{p0.10} \cdot L_{F} = 0 \cdot \frac{kN}{m}$

Case two - The wind is lifting

$$\begin{split} & C_{pe.F.10.2} := -0.63 \\ & C_{pe.G.10.2} := -0.60 \\ & C_{pe.J.10.2} := -0.67 \\ & C_{pe.H.10.2} := -0.23 \\ & C_{pe.I.10.2} := -0.40 \\ & F_{wind.F10.2} := C_{pe.F.10.2} \cdot q_{p0.10} \cdot L_{F} = -2.671 \cdot \frac{kN}{m} \\ & F_{wind.H.10.2} := C_{pe.H.10.2} \cdot q_{p0.10} \cdot L_{F} = -0.975 \cdot \frac{kN}{m} \\ & F_{wind.J.10.2} := C_{pe.J.10.2} \cdot q_{p0.10} \cdot L_{F} = -2.841 \cdot \frac{kN}{m} \\ & F_{wind.I.10.2} := C_{pe.I.10.2} \cdot q_{p0.10} \cdot L_{F} = -1.696 \frac{kN}{m} \end{split}$$

4. Mansard

Geometry

The distance between the frames have been set to 4 meters



 $\begin{array}{cccc} \hline wtain 15 \ m. \\ f_{15} \coloneqq 3.08m & d_{15} \coloneqq 15.54m & h_{1.15} \coloneqq 6m & h_{15} \coloneqq 9.08m & L_{\rm F} = 4m \\ \hline \hline Width \ 10 \ m. \\ f_{10} \coloneqq 2.44m & d_{10} \coloneqq 9.98m & h_{1.10} \coloneqq 4m & h_{10} \coloneqq 6.44m & L_{\rm F} = 4m \end{array}$

Snow load

The case with exceptional snow loads does not occur in Sweden has therefore been disregarded in the calculations.

The snow zone has been set to 3.5 since this covers most parts of Sweden.

Characteristic snow load, sk:	$s_k := 3.5 \frac{kN}{m^2}$
The topography is normal->	$C_e := 1$
Thermal coefficient, C _t :	C _t := 1

Load cases according to Eurocode



Recommendation according to Eurocode is that my3 shouldn't be larger than 2.0

<u>Width 20 m:</u> $\mu_{1.20} \coloneqq 0.8$ $\mu_{3.20} \coloneqq 0.2 \pm 10 \frac{f_{20}}{d_{20}} = 2.639$ $\mu_{3.20} \coloneqq 2.0$

 $\frac{Load \ case \ 1}{s_{1.20} \coloneqq \mu_{1.20} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}} \qquad \frac{Load \ case \ 2}{s_{2.1.20} \coloneqq \mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 28m \cdot \frac{kN}{m^2}}$ $s_{2.2.20} \coloneqq 0.5\mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 14m \cdot \frac{kN}{m^2}$

<u>Width 15 m:</u> $\mu_{1.15} := 0.8$ $\mu_{3.15} := 0.2 + 10 \frac{f_{15}}{d_{15}} = 2.182$

µ3.1.5, = 2.€

Load case 1

$$S_{1.15} := \mu_{1.15} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2 \text{m} \cdot \frac{\text{kN}}{\text{m}^2}$$

$$S_{2.1.15} := \mu_{3.15} C_e \cdot C_t \cdot s_k \cdot L_F = 28 \text{m} \cdot \frac{\text{kN}}{\text{m}^2}$$

$$S_{2.2.15} := 0.5 \mu_{3.15} C_e \cdot C_t \cdot s_k \cdot L_F = 14 \text{m} \cdot \frac{\text{kN}}{\text{m}^2}$$

Load case 2

Width 10 m:

<u>Wind load</u>

Terrain type: Zone 0

For dimensioning the load bearing structure the Cpe.10 values should be used

<u>Width 20 m:</u> <u>Characteristic wind pressure for a wind speed of 26 m/s</u>

 $h < d \longrightarrow z_{e.20} := h_{20} = 12.87m$ \longrightarrow Zone 0 $\longrightarrow q_{p0.20} := 1.22 \cdot \frac{kN}{m^2}$

Pressure coefficients on the external walls

 $\frac{h_{20}}{d_{20}} = 0.644 \qquad --> \qquad C_{pe.D.20} := 0.75$ $C_{pe.E.20} := -0.40$

Wind load on the columns

 $F_{wind.D.20} := C_{pe.D.20} q_{p0.20} L_F = 3.66 \frac{kN}{m}$ $F_{wind.E.20} := C_{pe.E.20} q_{p0.20} L_F = -1.952 \frac{kN}{m}$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.20}}{d_{20}} = 0.401$$

$$-->$$

$$C_{pe.10.A.20} = 0.20$$

$$C_{pe.10.B.20} = -1.10$$

$$C_{pe.10.C.20} = -0.40$$

Wind load on the roof

 $F_{wind.A.20} \coloneqq C_{pe.10.A.20} q_{p0.20} L_F = 0.976 \frac{kN}{m}$ $F_{wind.B.20} \coloneqq C_{pe.10.B.20} q_{p0.20} L_F = -5.368 \frac{kN}{m}$ $F_{wind.C.20} \coloneqq C_{pe.10.C.20} q_{p0.20} L_F = -1.952 \frac{kN}{m}$

Width 15 m:

 $h < d \rightarrow z_{e.15} := h_{15} = 9.08m$ $\rightarrow Zone 0$ $\rightarrow q_{p0.15} := 1.18 \frac{kN}{m^2}$ <u>Pressure coefficients on the external walls</u>

$$\frac{h_{15}}{d_{15}} = 0.584 \qquad --> \qquad C_{pe.D.15} := 0.74$$
$$C_{pe.E.15} := -0.39$$

Wind load on the columns

 $F_{wind.D.15} := C_{pe.D.15} \cdot q_{p0.15} \cdot L_F = 3.493 \frac{kN}{m}$

 $F_{wind.E.15} := C_{pe.E.15} q_{p0.15} L_F = -1.841 \cdot \frac{kN}{m}$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.15}}{d_{15}} = 0.386 \qquad C_{pe.10.A.15} := -0.85$$

$$\frac{f_{15}}{d_{15}} = 0.198 \qquad C_{pe.10.B.15} := -1.05$$

$$C_{pe.10.C.15} := -0.40$$

Wind load on the roof

$$F_{\text{wind.A.15}} := C_{\text{pe.10.A.15}} q_{\text{p0.15}} L_{\text{F}} = -4.012 \frac{\text{kN}}{\text{m}}$$
$$F_{\text{wind.B.15}} := C_{\text{pe.10.B.15}} q_{\text{p0.15}} L_{\text{F}} = -4.956 \frac{\text{kN}}{\text{m}}$$

$$F_{\text{wind.C.15}} := C_{\text{pe.10.C.15}} q_{\text{p0.15}} L_{\text{F}} = -1.888 \frac{\text{kN}}{\text{m}}$$

Width 10 m:

 $h < d \longrightarrow z_{e.10} := h_{10} = 6.44m$ \longrightarrow Zone 0 $\longrightarrow q_{p0.10} := 1.06 \frac{kN}{m^2}$

Pressure coefficients on the external walls

$$\frac{h_{10}}{d_{10}} = 0.645 \qquad --> \qquad C_{pe.D.10} := 0.75$$
$$C_{pe.E.10} := -0.41$$

Wind load on the columns

$$F_{\text{wind.D.10}} := C_{\text{pe.D.10}} q_{\text{p0.10}} L_{\text{F}} = 3.18 \frac{\text{kN}}{\text{m}}$$
$$F_{\text{wind.E.10}} := C_{\text{pe.E.10}} q_{\text{p0.10}} L_{\text{F}} = -1.738 \frac{\text{kN}}{\text{m}}$$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.10}}{d_{10}} = 0.401 \qquad C_{pe.10.A.10} := -0.70 \\ --> \qquad C_{pe.10.B.10} := -1.10 \\ \frac{f_{10}}{d_{10}} = 0.244 \qquad C_{pe.10.C.10} := -0.40 \\ --> \qquad C_{pe.10$$

Wind load on the roof

$$F_{wind.A.10} \coloneqq C_{pe.10.A.10}q_{p0.10}L_{F} = -2.968 \frac{kN}{m}$$
$$F_{wind.B.10} \coloneqq C_{pe.10.B.10}q_{p0.10}L_{F} = -4.664 \frac{kN}{m}$$
$$F_{wind.C.10} \coloneqq C_{pe.10.C.10}q_{p0.10}L_{F} = -1.696 \frac{kN}{m}$$

3. Arced

Geometry

The distance between the frames have been set to 4 meters



 $f_{10} := 5.226m$ $d_{10} := 10.453m$ $h_{1.10} := 0m$ $h_{10} := 5.226m$ $L_F = 4m$

Snow load

The case with exceptional snow loads does not occur in Sweden has therefore been disregarded in the calculations.

The snow zone has been set to 3.5 since this covers most parts of Sweden.

Characteristic snow load, sk:	$s_k := 3.5 \frac{kN}{m^2}$
The topography is normal>	$C_e := 1$
Thermal coefficient, Ct:	C _t := 1

Load cases according to Eurocode



Recommendation according to Eurocode is that my3 should not be larger than 2.0

 Width 20 m:

 $\mu_{1,20} \coloneqq 0.8$ $\mu_{3,20} \coloneqq 0.2 + 10 \frac{f_{20}}{d_{20}} = 5.028$
 $\mu_{3,20} \coloneqq 2.0$

 $\frac{Load \ case \ 1}{S_{1.20} \coloneqq \mu_{1.20} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}} \qquad \frac{Load \ case \ 2}{S_{2.1.20} \coloneqq \mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 28m \cdot \frac{kN}{m^2}}$ $S_{2.2.20} \coloneqq 0.5\mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 14m \cdot \frac{kN}{m^2}$

 $\begin{array}{ll} \underline{Width\ 15\ m:} \\ \mu_{1.15} \coloneqq 0.8 \\ \mu_{3.15} \coloneqq 0.2 \pm 10 \frac{f_{15}}{d_{15}} = 5.2 \\ \underline{W_{3ad5}} \coloneqq 2.0 \\ \underline{Load\ case\ 1} \\ S_{1.15} \coloneqq \mu_{1.15}C_{e}\cdot C_{t}\cdot s_{k}\cdot L_{F} = 11.2m \cdot \frac{kN}{m^{2}} \\ S_{2.1.15} \coloneqq \mu_{3.15}C_{e}\cdot C_{t}\cdot s_{k}\cdot L_{F} = 28m \cdot \frac{kN}{m^{2}} \\ S_{2.2.15} \coloneqq 0.5\mu_{3.15}C_{e}\cdot C_{t}\cdot s_{k}\cdot L_{F} = 14m \cdot \frac{kN}{m^{2}} \\ \end{array}$

Width 10 m:

 $\mu_{1.10} \coloneqq 0.8 \qquad \mu_{3.10} \coloneqq 0.2 \pm 10 \frac{f_{10}}{d_{10}} = 5.2$ $\underline{\text{M3AMO}} \coloneqq 2.0$ $\underline{\text{Load case 1}} \qquad \underline{\text{Load case 2}}$ $s_{1.10} \coloneqq \mu_{1.10}C_{e} \cdot C_{t} \cdot s_{k} \cdot L_{F} = 11.2 \text{m} \cdot \frac{\text{kN}}{\text{m}^{2}} \qquad s_{2.1.10} \coloneqq \mu_{3.10}C_{e} \cdot C_{t} \cdot s_{k} \cdot L_{F} = 28 \text{m} \cdot \frac{\text{kN}}{\text{m}^{2}}$ $s_{2.2.10} \coloneqq 0.5 \mu_{3.10}C_{e} \cdot C_{t} \cdot s_{k} \cdot L_{F} = 14 \text{m} \cdot \frac{\text{kN}}{\text{m}^{2}}$

Wind load

Terrain type: Zone 0

For dimensioning the load bearing structure the Cpe.10 values should be used



Width 20 m:

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.20}}{d_{20}} = 0$$

$$\frac{h_{1.20}}{d_{20}} = 0$$

$$\frac{h_{1.20}}{d_{20}} = 0.483$$

$$C_{pe.10.A.20} = 0.78$$

$$C_{pe.10.B.20} = -1.19$$

$$C_{pe.10.C.20} = 0$$

Wind load on the roof

 $F_{wind.A.20} \coloneqq C_{pe.10.A.20} q_{p0.20} L_F = 3.682 \frac{kN}{m}$ $F_{wind.B.20} \coloneqq C_{pe.10.B.20} q_{p0.20} L_F = -5.617 \frac{kN}{m}$ $F_{wind.C.20} \coloneqq C_{pe.10.C.20} q_{p0.20} L_F = 0 \frac{kN}{m}$

Width 15 m:

$$\begin{array}{l} \underline{Characteristic \ wind \ pressure \ for \ a \ wind \ speed \ of \ 26 \ m/s} \\ h < d \ --> \qquad z_{e.15} \coloneqq h_{15} = 7.727 m \quad --> \qquad Zone \ 0 \quad --> \qquad q_{p0.15} \coloneqq 1.13 \frac{kN}{m^2} \end{array}$$

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.15}}{d_{15}} = 0$$

$$\frac{f_{15}}{d_{15}} = 0.5$$

$$C_{pe.10.A.15} \coloneqq 0.78$$

$$C_{pe.10.B.15} \coloneqq -1.19$$

$$C_{pe.10.C.15} \coloneqq 0$$

Wind load on the roof

$$F_{wind.A.15} := C_{pe.10.A.15} q_{p0.15} L_F = 3.526 \frac{kN}{m}$$

 $F_{wind.B.15} := C_{pe.10.B.15} q_{p0.15} L_F = -5.379 \frac{kN}{m}$

$$F_{\text{wind.C.15}} := C_{\text{pe.10.C.15}} q_{\text{p0.15}} L_{\text{F}} = 0 \cdot \frac{kN}{m}$$

Width 10 m:

Characteristic wind pressure for a wind speed of 26 m/s

$$h < d \rightarrow z_{e.10} := h_{10} = 5.226 m \rightarrow Zone 0 \rightarrow q_{p0.10} := 1.06 \frac{kN}{m^2}$$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.10}}{d_{10}} = 0$$

$$\frac{h_{1.10}}{d_{10}} = 0.5$$

$$C_{pe.10.A.10} = 0.78$$

$$C_{pe.10.B.10} = -1.19$$

$$C_{pe.10.C.10} = 0$$

Wind load on the roof

 $F_{\text{wind.A.10}} := C_{\text{pe.10.A.10}} q_{\text{p0.10}} L_{\text{F}} = 3.307 \frac{\text{kN}}{\text{m}}$

 $F_{wind.B.10} := C_{pe.10.B.10} q_{p0.10} L_F = -5.046 \frac{kN}{m}$

 $F_{wind.C.10} := C_{pe.10.C.10} q_{p0.10} L_F = 0 \cdot \frac{kN}{m}$

4. Arced with columns

Geometry

The distance between the frames have been set to 4 meters



Width 20 m:

$f_{20} := 10.116 \mathrm{m}$	$d_{20} := 20.951 \mathrm{m}$	$h_{1.20} := 4m$	$h_{20} := 14.12m$	$L_F := 4m$
Width 15 m:				
$f_{15} := 7.727 m$	$d_{15} := 15.455 \mathrm{m}$	$h_{1.15} := 4m$	$h_{15} := 11.73 m$	$L_F = 4 m$
Width 10 m:				
$f_{10} := 5.226m$	$d_{10} := 10.453m$	$h_{1.10} := 4m$	$h_{10} := 9.226m$	$L_F = 4 m$

Snow load

The case with exceptional snow loads does not occur in Sweden and has therefore been disregarded in the calculations.

The snow zone has been set to 3.5 since this covers most parts of Sweden.

Characteristic snow load, sk:
$$s_k := 3.5 \frac{kN}{m^2}$$
The topography is normal $C_e := 1$ Thermal coefficient, C_t : $C_t := 1$

-->

Load cases according to Eurocode



Recommendation according to Eurocode is that my3 should not be larger than 2.0

 Width 20 m:

 $\mu_{1.20} \coloneqq 0.8$ $\mu_{3.20} \coloneqq 0.2 + 10 \cdot \frac{f_{20}}{d_{20}} = 5.028$
 $\mu_{3.20} \coloneqq 2.0$

 $\frac{Load \ case \ 1}{s_{1.20} \coloneqq \mu_{1.20} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}} \qquad \frac{Load \ case \ 2}{s_{2.1.20} \coloneqq \mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 28m \cdot \frac{kN}{m^2}}$ $s_{2.2.20} \coloneqq 0.5\mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 14m \cdot \frac{kN}{m^2}$

$$\begin{array}{lll} \underline{Width \ 15 \ m:} \\ \mu_{1.15} \coloneqq 0.8 & \mu_{3.15} \coloneqq 0.2 \pm 10 \cdot \frac{f_{15}}{d_{15}} = 5.2 \\ \underline{\mu_{3\lambda a \lambda 5v}} \coloneqq 2.0 & \\ \underline{Load \ case \ 1} & \underline{Load \ case \ 2} \\ s_{1.15} \coloneqq \mu_{1.15} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2 m \cdot \frac{kN}{m^2} & s_{2.1.15} \coloneqq \mu_{3.15} C_e \cdot C_t \cdot s_k \cdot L_F = 28 m \cdot \frac{kN}{m^2} \\ s_{2.2.15} \coloneqq 0.5 \mu_{3.15} C_e \cdot C_t \cdot s_k \cdot L_F = 14 m \cdot \frac{kN}{m^2} \end{array}$$

Width 10 m:

 $\mu_{1.10} \coloneqq 0.8 \qquad \mu_{3.10} \coloneqq 0.2 \pm 10 \frac{f_{10}}{d_{10}} = 5.2$ $\underline{\text{M3AMO}} \coloneqq 2.0$ $\underline{\text{Load case 1}} \qquad \underline{\text{Load case 2}}$ $s_{1.10} \coloneqq \mu_{1.10}C_{e} \cdot C_{t} \cdot s_{k} \cdot L_{F} = 11.2 \text{m} \cdot \frac{\text{kN}}{\text{m}^{2}} \qquad s_{2.1.10} \coloneqq \mu_{3.10}C_{e} \cdot C_{t} \cdot s_{k} \cdot L_{F} = 28 \text{m} \cdot \frac{\text{kN}}{\text{m}^{2}}$ $s_{2.2.10} \coloneqq 0.5 \mu_{3.10}C_{e} \cdot C_{t} \cdot s_{k} \cdot L_{F} = 14 \text{m} \cdot \frac{\text{kN}}{\text{m}^{2}}$

Wind load

Terrain type: Zone 0

For dimensioning the load bearing structure the Cpe.10 values should be used



Width 20 m:

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.20}}{d_{20}} = 0$$

$$\frac{h_{1.20}}{d_{20}} = 0$$

$$\frac{h_{1.20}}{d_{20}} = 0.483$$

$$C_{pe.10.A.20} = 0.78$$

$$C_{pe.10.B.20} = -1.19$$

$$C_{pe.10.C.20} = 0$$

Wind load on the roof

 $F_{wind.A.20} \coloneqq C_{pe.10.A.20} q_{p0.20} L_F = 3.682 \frac{kN}{m}$ $F_{wind.B.20} \coloneqq C_{pe.10.B.20} q_{p0.20} L_F = -5.617 \frac{kN}{m}$ $F_{wind.C.20} \coloneqq C_{pe.10.C.20} q_{p0.20} L_F = 0 \frac{kN}{m}$

Width 15 m:

$$\begin{array}{l} \underline{Characteristic \ wind \ pressure \ for \ a \ wind \ speed \ of \ 26 \ m/s} \\ h < d \ --> \qquad z_{e.15} \coloneqq h_{15} = 7.727 m \quad --> \qquad Zone \ 0 \quad --> \qquad q_{p0.15} \coloneqq 1.13 \frac{kN}{m^2} \end{array}$$

Pressure coefficients on the roof

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.15}}{d_{15}} = 0$$

$$\frac{c_{pe.10.A.15} = 0.78}{c_{pe.10.B.15} = -1.19}$$

$$\frac{f_{15}}{d_{15}} = 0.5$$

$$c_{pe.10.C.15} = 0$$

Wind load on the roof

$$F_{wind.A.15} := C_{pe.10.A.15} q_{p0.15} L_F = 3.526 \frac{kN}{m}$$

 $F_{wind.B.15} := C_{pe.10.B.15} q_{p0.15} L_F = -5.379 \frac{kN}{m}$

$$F_{\text{wind.C.15}} := C_{\text{pe.10.C.15}} q_{\text{p0.15}} L_{\text{F}} = 0 \cdot \frac{kN}{m}$$

Width 10 m:

Characteristic wind pressure for a wind speed of 26 m/s

$$h < d \longrightarrow z_{e.10} := h_{10} = 5.226m \longrightarrow Zone 0 \longrightarrow q_{p0.10} := 1.06 \frac{kN}{m^2}$$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.10}}{d_{10}} = 0$$

$$\frac{h_{1.10}}{d_{10}} = 0.5$$

$$C_{pe.10.A.10} = 0.78$$

$$C_{pe.10.B.10} = -1.19$$

$$C_{pe.10.C.10} = 0$$

Wind load on the roof

 $F_{\text{wind.A.10}} := C_{\text{pe.10.A.10}} q_{\text{p0.10}} L_{\text{F}} = 3.307 \frac{\text{kN}}{\text{m}}$

 $F_{wind.B.10} := C_{pe.10.B.10} q_{p0.10} L_F = -5.046 \frac{kN}{m}$

 $F_{wind.C.10} := C_{pe.10.C.10} q_{p0.10} L_F = 0 \cdot \frac{kN}{m}$

4. Arced with columns

Geometry

The distance between the frames have been set to 4 meters



Width 20 m:

$f_{20} := 10.116 \mathrm{m}$	$d_{20} := 20.951 \mathrm{m}$	$h_{1.20} := 4m$	$h_{20} := 14.12m$	$L_F := 4m$
Width 15 m:				
$f_{15} := 7.727 m$	$d_{15} := 15.455 \mathrm{m}$	$h_{1.15} := 4m$	$h_{15} := 11.73 m$	$L_F = 4 m$
Width 10 m:				
$f_{10} := 5.226m$	$d_{10} := 10.453 m$	$h_{1.10} := 4m$	$h_{10} := 9.226m$	$L_F = 4 m$

<u>Snow load</u>

The case with exceptional snow loads does not occur in Sweden and has therefore been disregarded in the calculations.

The snow zone has been set to 3.5 since this covers most parts of Sweden.

Characteristic snow load, sk:
$$s_k := 3.5 \frac{kN}{m^2}$$
The topography is normal $C_e := 1$ Thermal coefficient, C_t : $C_t := 1$

-->

Load cases according to Eurocode



Recommendation according to Eurocode is that my3 should not be larger than 2.0

 Width 20 m:

 $\mu_{1.20} \coloneqq 0.8$ $\mu_{3.20} \coloneqq 0.2 + 10 \cdot \frac{f_{20}}{d_{20}} = 5.028$
 $\mu_{3.20} \coloneqq 2.0$

 $\frac{Load \ case \ 1}{s_{1.20} \coloneqq \mu_{1.20} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2m \cdot \frac{kN}{m^2}} \qquad \frac{Load \ case \ 2}{s_{2.1.20} \coloneqq \mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 28m \cdot \frac{kN}{m^2}}$ $s_{2.2.20} \coloneqq 0.5\mu_{3.20} C_e \cdot C_t \cdot s_k \cdot L_F = 14m \cdot \frac{kN}{m^2}$

Width 15 m:

$$\mu_{1.15} \coloneqq 0.8$$
 $\mu_{3.15} \coloneqq 0.2 + 10 \cdot \frac{f_{15}}{d_{15}} = 5.2$
 $\mu_{3.15} \coloneqq 2.0$
 $\mu_{3.15} \coloneqq 2.0$

 Load case 1

 $S_{1.15} \coloneqq \mu_{1.15} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2 \text{m} \cdot \frac{\text{kN}}{\text{m}^2}$
 $S_{2.1.15} \coloneqq \mu_{3.15} C_e \cdot C_t \cdot s_k \cdot L_F = 28 \text{m} \cdot \frac{\text{kN}}{\text{m}^2}$

Width 10 m:

 $\mu_{1.10} := 0.8$ $\mu_{3.10} := 0.2 + 10 \cdot \frac{f_{10}}{d_{10}} = 5.2$ $\mu_{3.10} := 2.0$

 $\underline{Load \ case \ 1} \qquad \underline{Load \ case \ 2} \\
 S_{1.10} := \mu_{1.10} C_e \cdot C_t \cdot s_k \cdot L_F = 11.2 m \frac{kN}{m^2} \qquad S_{2.1.10} := \mu_{3.10} C_e \cdot C_t \cdot s_k \cdot L_F = 28 m \cdot \frac{kN}{m^2} \\
 S_{2.2.10} := 0.5 \mu_{3.10} C_e \cdot C_t \cdot s_k \cdot L_F = 14 m \cdot \frac{kN}{m^2}$

Wind load

Terrain type: Zone 0

For dimensioning the load bearing structure the Cpe.10 values should be used

Width 20 m:

Characteristic wind pressure for a wind speed of 26 m/s

 $h < d \longrightarrow z_{e.20} := h_{20} = 14.12m \longrightarrow Zone 0 \longrightarrow q_{p0.20} := 1.26 \frac{kN}{m^2}$

Pressure coefficients on the external walls

$$\frac{h_{1.20}}{d_{20}} = 0.191 \quad --> \quad C_{pe.D.20} := 0.7$$
$$C_{pe.E.20} := -0.3$$

Wind load on the columns

 $F_{wind.D.20} := C_{pe.D.20} q_{p0.20} L_F = 3.528 \frac{kN}{m}$ $F_{wind.E.20} := C_{pe.E.20} q_{p0.20} L_F = -1.512 \frac{kN}{m}$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.20}}{d_{20}} = 0.191$$

$$-->$$

$$C_{pe.10.A.20} := 0.7$$

$$C_{pe.10.B.20} := -1.3$$

$$\frac{f_{20}}{d_{20}} = 0.483$$

$$C_{pe.10.C.20} := -0.25$$

Wind load on the roof

 $F_{\text{wind.A.20}} \coloneqq C_{\text{pe.10.A.20}} q_{\text{p0.20}} L_{\text{F}} = 3.528 \frac{\text{kN}}{\text{m}}$ $F_{\text{wind.B.20}} \coloneqq C_{\text{pe.10.B.20}} q_{\text{p0.20}} L_{\text{F}} = -6.552 \frac{\text{kN}}{\text{m}}$ $F_{\text{wind.C.20}} \coloneqq C_{\text{pe.10.C.20}} q_{\text{p0.20}} L_{\text{F}} = -1.26 \frac{\text{kN}}{\text{m}}$

Width 15 m:

$$\frac{Characteristic wind pressure for a wind speed of 26 m/s}{h < d -->} \qquad z_{e.15} := h_{15} = 11.73m \qquad --> \qquad Zone \ 0 \qquad --> \qquad q_{p0.15} := 1.22 \frac{kN}{m^2}$$

Pressure coefficients on the external walls

$$\frac{h_{1.15}}{d_{15}} = 0.259 \quad --> \quad C_{pe.D.15} := 0.7$$
$$C_{pe.E.15} := -0.3$$

Wind load on the columns

 $F_{wind.D.15} := C_{pe.D.15} \cdot q_{p0.15} \cdot L_F = 3.416 \frac{kN}{m}$

 $F_{wind.E.15} := C_{pe.E.15} q_{p0.15} L_F = -1.464 \frac{kN}{m}$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.15}}{d_{15}} = 0.259 \qquad C_{pe.10.A.15} \coloneqq 0.8$$

-->
$$C_{pe.10.B.15} \coloneqq -1.25$$

$$\frac{f_{15}}{d_{15}} = 0.5 \qquad C_{pe.10.C.15} \coloneqq -0.25$$

Wind load on the roof

$$F_{wind.A.15} \coloneqq C_{pe.10.A.15} q_{p0.15} L_F = 3.904 \frac{kN}{m}$$

$$F_{wind.B.15} \coloneqq C_{pe.10.B.15} q_{p0.15} L_F = -6.1 \cdot \frac{kN}{m}$$

$$F_{wind.C.15} \coloneqq C_{pe.10.C.15} q_{p0.15} L_F = -1.22 \cdot \frac{kN}{m}$$

Width 10 m:

<u>Characteris</u>	<u>stic wind pressure for a</u>	wind sp	peed of 26 m	<u>/s</u>	
h < d>	$z_{e.10} := h_{10} = 9.226m$	>	Zone 0	>	$q_{p0.10} := 1.18 \frac{kN}{m^2}$
<u>Pressure co</u>	efficients on the extern	al walls	5		111

$$\frac{h_{1.10}}{d_{10}} = 0.383 \quad --> \qquad C_{pe.D.10} := 0.72$$
$$C_{pe.E.10} := -0.33$$

Wind load on the columns

$$F_{\text{wind.D.10}} \coloneqq C_{\text{pe.D.10}} q_{\text{p0.10}} L_{\text{F}} = 3.398 \frac{\text{kN}}{\text{m}}$$
$$F_{\text{wind.E.10}} \coloneqq C_{\text{pe.E.10}} q_{\text{p0.10}} L_{\text{F}} = -1.558 \frac{\text{kN}}{\text{m}}$$

Assume that the length of the building always will be more than 24 meters.

$$\frac{h_{1.10}}{d_{10}} = 0.383 \qquad C_{pe.10.A.10} \coloneqq 0.8$$

$$\frac{f_{10}}{d_{10}} = 0.5 \qquad C_{pe.10.B.10} \coloneqq -1.27$$

Wind load on the roof

$$F_{wind.A.10} := C_{pe.10.A.10} q_{p0.10} L_{F} = 3.776 \frac{kN}{m}$$

$$F_{wind.B.10} := C_{pe.10.B.10} q_{p0.10} L_{F} = -5.994 \frac{kN}{m}$$

$$F_{wind.C.10} := C_{pe.10.C.10} q_{p0.10} L_{F} = -1.794 \frac{kN}{m}$$

Appendix III – Dimensioning of elements

1. Dup-Pitched roof concept, 20 meter

Forces and moments

Axial forces and bending moments for the load case with symmetrical snow load + wind load.

$$\mathbf{A}_{\text{sym}} \coloneqq \begin{pmatrix} 178.9 \\ 113.8 \\ 98.5 \\ 67.9 \\ 77.5 \\ 108.1 \\ 123.4 \\ 192.8 \end{pmatrix} \mathbf{kN} \qquad \mathbf{M}_{\text{sym}} \coloneqq \begin{pmatrix} 343.9 \\ 343.9 \\ 297.5 \\ 297.5 \\ 267.7 \\ 232.4 \\ 544 \\ 544 \\ 544 \end{pmatrix} \mathbf{kN} \cdot \mathbf{m}$$

Yield strength

f_{yd} := 355MPa

<u>Steel area</u>

b := 195mm

h := 460mm

t := 16.0mm

d := 10.0mm

A := 2·b·t + (h - 2·t)·d = 0.011m²
I :=
$$2\left[\frac{b \cdot t^3}{12} + b \cdot t \cdot \left(\frac{h}{2} - \frac{t}{2}\right)^2\right] + \frac{d \cdot (h - 2 \cdot t)^3}{12} = 3.73 \times 10^8 \cdot \text{mm}^4$$

Z := $\frac{h}{2} = 0.23\text{m}$

Stresses for all elements

The maximum bending moment and the maximum axial force will occur in the load case with symmetrical snow load + wind.

$$\sigma_{sym} := \frac{A_{sym}}{A} + \frac{M_{sym} \cdot Z}{I} = \begin{pmatrix} 229.061 \\ 222.873 \\ 192.808 \\ 189.899 \\ 172.436 \\ 153.578 \\ 347.172 \\ 353.769 \end{pmatrix} \cdot MPa$$

Maximum stress

 $\sigma_{Max} \coloneqq max(\sigma_{sym}) = 353.769 \text{MPa}$

$$\frac{\sigma_{\text{Max}}}{f_{\text{yd}}} = 0.997$$

Utilization ratio for the elements based on moments and axial forces

$$\mu_{\text{sym}} := \frac{\sigma_{\text{sym}}}{\sigma_{\text{Max}}} = \begin{pmatrix} 0.647\\ 0.63\\ 0.545\\ 0.537\\ 0.487\\ 0.434\\ 0.981\\ 1 \end{pmatrix}$$

 $i := \text{length}(\sigma_{\text{sym}}) = 8$

Utilization ratio for the total frame based on moments and axial forces

$$\mu_{\text{average.sym}} \coloneqq \frac{\sum \sigma_{\text{sym}}}{i \cdot \max(\sigma_{\text{sym}})} = 0.658$$

<u>Total weight</u>

$$\rho := 7850 \frac{\text{kg}}{\text{m}^3}$$

L := 5.320m

C := 4m

- $W := A \cdot \rho \cdot L = 439.336 \text{kg}$
- $W_{tot} := A \cdot \rho \cdot (4 \cdot C + 4 \cdot L) = 3.079 \times 10^3 \text{kg}$

2. Mansard roof concept, 20 meter

Forces and moments

Axial forces and bending moments for the load case with unsymmetrical snow load + wind load.

$$A_{unsym} := \begin{pmatrix} 171.2 \\ 136.7 \\ 53.1 \\ 57.6 \\ 115.7 \\ 126.2 \end{pmatrix} kN \qquad M_{unsym} := \begin{pmatrix} 192.6 \\ 261.1 \\ 261.1 \\ 178.7 \\ 429.9 \\ 429.9 \end{pmatrix} kN \cdot m$$

Axial forces and bending moments for the load case with unsymmetrical snow as the only load.



<u>Yield strength</u>

 $f_{yd} := 355MPa$

<u>Steel area</u>

b := 190mm

h := 450mm

t := 13.0mm

d := 8.0mm

A :=
$$2 \cdot b \cdot t + (h - 2 \cdot t) \cdot d = 8.332 \times 10^{-3} \text{ m}^2$$

$$I := 2\left[\frac{b \cdot t^{3}}{12} + b \cdot t \cdot \left(\frac{h}{2} - \frac{t}{2}\right)^{2}\right] + \frac{d \cdot (h - 2 \cdot t)^{3}}{12} = 2.867 \times 10^{8} \cdot \text{mm}^{4}$$
$$Z := \frac{h}{2} = 0.225 \text{m}$$

Stresses for all elements

The maximum bending moment will occur in the load case with unsymmetrical snow + wind.

$$\sigma_{\text{unsym}} := \frac{A_{\text{unsym}}}{A} + \frac{M_{\text{unsym}} \cdot Z}{I} = \begin{pmatrix} 171.681 \\ 221.292 \\ 211.259 \\ 147.139 \\ 351.23 \\ 352.49 \end{pmatrix} \cdot \text{MPa}$$

The maximum axial force will occur in the symmetrical load case

$$\sigma_{\text{snow}} := \frac{A_{\text{snow}}}{A} + \frac{M_{\text{snow}} \cdot Z}{I} = \begin{pmatrix} 334.788\\ 330.132\\ 176.227\\ 179.527\\ 325.487\\ 327.419 \end{pmatrix} \cdot \text{MPa}$$

Maximal stress for the maximum bending moment and the corresponding axial force

 $\sigma_{Max.M} := max(\sigma_{unsym}) = 352.49 \text{ MPa}$

$$\frac{\sigma_{\text{Max.M}}}{f_{\text{yd}}} = 0.993$$

Maximal stress for the maximal axial force and the corresponding bending moment

$$\sigma_{\text{Max},A} := \max(\sigma_{\text{snow}}) = 334.788 \text{ MPa}$$

$$\frac{\sigma_{\text{Max.A}}}{f_{\text{yd}}} = 0.943$$



Utilization ratio for the total frame based on moments and axial forces

$$i := \operatorname{length}(\sigma_{\operatorname{unsym}}) = 6$$

$$\mu_{\operatorname{average.unsym}} := \frac{\sum \sigma_{\operatorname{unsym}}}{i \cdot \max(\sigma_{\operatorname{unsym}})} = 0.688 \qquad \mu_{\operatorname{average.snow}} := \frac{\sum \sigma_{\operatorname{snow}}}{i \cdot \max(\sigma_{\operatorname{snow}})} = 0.833$$

$$\frac{Total \ weight}{\rho} := 7850 \frac{\operatorname{kg}}{\operatorname{m}^3}$$

$$L := 2.85m$$

$$C := 4m$$

$$W := A \cdot \rho \cdot C = 261.625 \text{kg}$$

$$W_{tot} := \rho \cdot (8 \cdot L + 4 \cdot C) \cdot A = 2.538 \times 10^3 \text{ kg}$$

3. Arced concept, 20 meter

Forces and moments

Axial forces and bending moments for the load case with unsymmetrical snow + wind.

$$A_{unsym} := \begin{pmatrix} 182.1 \\ 177.3 \\ 130.2 \\ 63.7 \\ 71 \\ 115.2 \\ 138.4 \\ 130.4 \end{pmatrix} \cdot kN \qquad M_{unsym} := \begin{pmatrix} 61.1 \\ 68.5 \\ 166.6 \\ 166.6 \\ 74.2 \\ 179.7 \\ 186.6 \\ 186.6 \\ 186.6 \end{pmatrix} \cdot kN \cdot m$$

Yield strength

f_{yd} := 355MPa

<u>Steel area</u>

b := 150mm

h := 290mm

t := 12mm

d := 8mn

A := $2 \cdot b \cdot t + (h - 2 \cdot t) \cdot d = 5.728 \times 10^{-3} \text{ m}^2$

$$I := 2\left[\frac{b \cdot t^{3}}{12} + b \cdot t \cdot \left(\frac{h}{2} - \frac{t}{2}\right)^{2}\right] + \frac{d \cdot (h - 2 \cdot t)^{3}}{12} = 8.215 \times 10^{7} \cdot \text{mm}^{4}$$
$$Z := \frac{h}{2} = 0.145\text{m}$$

Stresses for all elements

The maximum bending moment and maximum axial force will occur in the unsymmetrical load case

$$\sigma := \frac{A_{unsym}}{A} + \frac{M_{unsym} \cdot Z}{I} = \begin{pmatrix} 139.642\\ 151.866\\ 316.804\\ 305.194\\ 143.369\\ 337.308\\ 353.538\\ 352.142 \end{pmatrix} \cdot MPa$$

Maximum stress

 $\sigma_{\text{Max.M}} \coloneqq \max(\sigma) = 353.538 \text{MPa}$

$$\frac{\sigma_{\text{Max.M}}}{f_{\text{yd}}} = 0.996$$

Utilization ratio for the elements based on moments and axial forces

$$\mu := \frac{\sigma}{\sigma_{\text{Max.M}}} = \begin{pmatrix} 0.395\\ 0.43\\ 0.896\\ 0.863\\ 0.406\\ 0.954\\ 1\\ 0.996 \end{pmatrix}$$

Utilization ratio for the total frame based on moments and axial forces

$$\frac{\sum \sigma}{8 \cdot \sigma_6} = 0.742$$

<u>Total weight</u>

$$\rho := 7850 \frac{\text{kg}}{\text{m}^3}$$

L := 4m

$$W := A \cdot \rho \cdot L = 179.859 \text{kg}$$

n := 8

 $W_{tot} := W \cdot n = 1.439 \times 10^3 \text{ kg}$

4. Arced with columns concept, 20 meter

Forces and moments

Axial forces and bending moments for the load case with unsymmetrical snow +wind.

$$A_{unsym} := \begin{pmatrix} 161.8\\ 163\\ 151.2\\ 100.2\\ 33.5\\ 44.2\\ 95\\ 126.9\\ 127.6\\ 18.1 \end{pmatrix} kN \qquad M_{unsym} := \begin{pmatrix} 69.9\\ 69.9\\ 143.8\\ 263.3\\ 263.3\\ 104.4\\ 235.1\\ 334.9\\ 334.9\\ 233.7 \end{pmatrix} kN \cdot m$$

Axial forces and bending moments for the load case with symmetrical snow + wind.

$$A_{sym} := \begin{pmatrix} 153.7\\ 155.5\\ 145.4\\ 82.3\\ 40.5\\ 45.9\\ 97.9\\ 168.8\\ 176.8\\ 168.2 \end{pmatrix} kN \qquad M_{sym} := \begin{pmatrix} 75.6\\ 75.6\\ 62.3\\ 140.7\\ 144.5\\ 144.5\\ 144.5\\ 152.5\\ 301\\ 301\\ 239.4 \end{pmatrix} kN \cdot m$$

<u>Yield strength</u>

f_{yd} := 355MPa
Steel area

b := 175mr h := 395mn t := 13.0mn d := 8.0mn A := 2·b·t + (h - 2·t)·d = 7.502× 10⁻³ m² I := 2 $\left[\frac{b \cdot t^{3}}{12} + b \cdot t \cdot \left(\frac{h}{2} - \frac{t}{2}\right)^{2}\right] + \frac{d \cdot (h - 2 \cdot t)^{3}}{12} = 1.995 \times 10^{8} \cdot \text{mm}^{4}$ Z := $\frac{h}{2} = 0.198\text{m}$

Stresses for all elements

The maximum bending moment will occur in the load case with unsymmetrical snow + wind.

$$\sigma_{unsym} := \frac{A_{unsym}}{A} + \frac{M_{unsym} \cdot Z}{I} = \begin{pmatrix} 90.75\\ 90.91\\ 162.479\\ 273.954\\ 265.063\\ 109.22\\ 245.35\\ 348.378\\ 348.471\\ 247.044 \end{pmatrix} \cdot MPa$$

The maximum axial force will occur in the load case with symmetrical snow + wind.

$$\sigma_{sym} := \frac{A_{sym}}{A} + \frac{M_{sym} \cdot Z}{I} = \begin{pmatrix} 95.312\\ 95.552\\ 81.042\\ 150.226\\ 148.415\\ 149.135\\ 163.985\\ 320.411\\ 321.477\\ 259.363 \end{pmatrix} \cdot MPa$$

Maximum stress for the maximum bending moment with the corresponding axial force.

 $\sigma_{\text{Max.M}} := \max(\sigma_{\text{unsym}}) = 348.471 \text{ MPa}$

$$\frac{\sigma_{\text{Max.M}}}{f_{\text{yd}}} = 0.982$$

Maximum stress for the maximum axial force and the corresponding bending moment

$$\sigma_{\text{Max.A}} := \max(\sigma_{\text{sym}}) = 321.477 \text{MPa}$$

 $\frac{\sigma_{Max.A}}{f_{yd}} = 0.906$

Utilization ratio for the elements based on moments and axial forces

$$\mu_{\text{unsym}} := \frac{\sigma_{\text{unsym}}}{\sigma_{\text{Max},\text{M}}} = \begin{pmatrix} 0.26\\ 0.261\\ 0.466\\ 0.786\\ 0.786\\ 0.761\\ 0.313\\ 0.704\\ 1\\ 1\\ 0.709 \end{pmatrix} \qquad \mu_{\text{sym}} := \frac{\sigma_{\text{sym}}}{\sigma_{\text{Max},\text{A}}} = \begin{pmatrix} 0.296\\ 0.297\\ 0.252\\ 0.467\\ 0.462\\ 0.464\\ 0.51\\ 0.997\\ 1\\ 0.807 \end{pmatrix}$$

Utilization ratio for the total frame based on moments and axial forces

$$\mu_{average.unsym} := \frac{\sum \sigma_{unsym}}{10 \cdot \sigma_{unsym_8}} = 0.626 \qquad \mu_{average.sym} := \frac{\sum \sigma_{sym}}{10 \cdot \sigma_{sym_8}} = 0.555$$

<u>Total weight</u>

$$\rho := 7850 \frac{\text{kg}}{\text{m}^3}$$

 $W := A \cdot \rho \cdot L = 235.563 \text{kg}$

 $W_{tot} := W \cdot n = 2.356 \times 10^3 \text{ kg}$