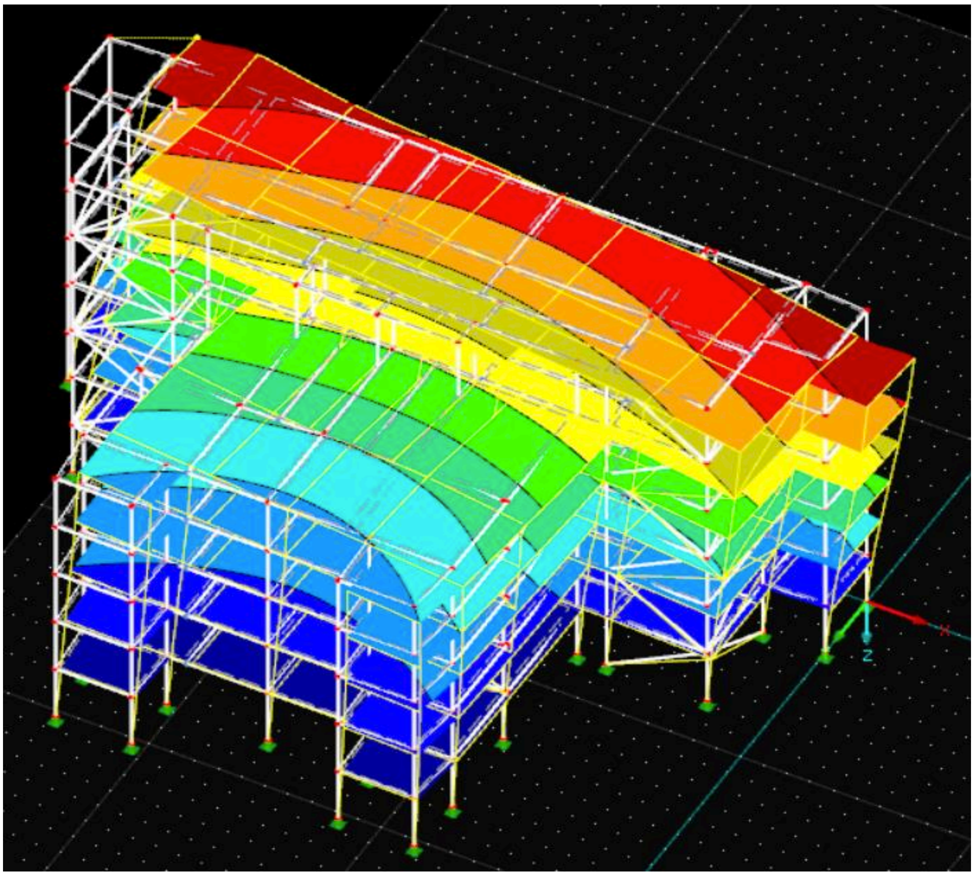

Horizontal stabilization of a trä8 building using glulam trusses

Finite element analysis of a multiple storey building in timber

Master's thesis in the Master's Programme Structural Engineering and Building Technology

STINA ÅKESSON

Department of Architecture and Civil Engineering
Division of Structural engineering
Steel and timber structures
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Master's Thesis ACEX30-18-54
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Chalmers tekniska högskola, 2018

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Cover:
The eigenmode of the building modelled with six storeys and floor stiffness of 80 %. Additional trusses A, B and C are added to the structure. See Appendix B, page B87.

Department of Architecture and Civil Engineering
Göteborg, Sweden, 2018

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ABSTRACT

Due to an increasing environmental awareness along with a need of building high-rise buildings the demand of multiple storey timber buildings has reached its highest yet. Trä8 is a construction method for constructing multiple storey timber buildings developed by Moelven Töreboda. As timber is a light-weight construction material stability is often a concern for high-rise timber buildings.

Frostaliden 3 is a newly built residential building in Skövde which was constructed using the trä8 method. This thesis refers to investigate the horizontal stability of this building modelled with four and six storeys. For the purpose of the thesis the glulam trusses were designed, and the complete structure of the building was modelled in a finite element software, RFEM, in order to investigate its static and dynamic behaviour. The stiffness of the floor is not yet established. The stiffness has been estimated to between 35 and 80 % of the stiffness of one floor element. The modelling in this thesis was therefore done including two cases of floor stiffness, within which range the actual stiffness will be.

For the static behaviour of the structure the horizontal displacements were examined and compared to the limit set at L/500 for the four- and the six-storey model. The results showed to be within limits for both heights and both modelled stiffnesses. For the dynamic analysis guidelines are provided in ISO 10137 in terms of fundamental eigenfrequency and wind peak acceleration. For the six-storey model the results exceeded the guidelines except for the most favourable case with stiffness of 80 % and wind acting on the short side. For the four-storey building all cases showed to be within the limits except for the worst case, 35 % stiffness and wind on the long side. The analysis further showed that the importance of additional trusses and floor stiffnesses decreases with increasing building height. For tall buildings it is not enough to add trusses or stiffer floor elements, other measures need to be taken in order to increase the fundamental frequency, such as increasing the structures stiffness or adding dampers.

Key words: timber engineering, trä8, tall timber buildings

Horisontell stabilisering av en trä8-byggnad med hjälp av limträfackverk

FE-analys på ett flervåningshus i trä

Examensarbete inom masterprogrammet Structural Engineering and Building Technology

STINA ÅKESSON

Institutionen för arkitektur och samhällsbyggnadsteknik
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Chalmers tekniska högskola

SAMMANFATTNING

På grund av dagens miljömedvetenhet samt det ökande behovet av att bygga på höjden har efterfrågan på flervåningshus i trä nått sin högsta nivå hittills. Trä8 är en byggnadsmetod, för att konstruera flervåningshus i trä, som är framtagen av Moelven Töreboda. Eftersom trä har en låg egenvikt kan stabiliteten utgöra problem för höga träbyggnader.

Frostaliden 3 är ett nybyggt lägenhetshus i Skövde som är konstruerat med trä8-metoden. Projektet avser att undersöka den horisontella stabiliteten i denna byggnad modellerad med fyra respektive sex våningar. I detta syfte har limträfackverken designats och den sammansatta stommen har vidare modelleras i RFEM, ett program baserat på finita elementmetoden, där dess statiska och dynamiska egenskaper undersökts. Styvheten på golvet är ännu inte fastställd. Styvheten har estimerats till mellan 35 och 80 % av styvheten för ett golvelement. Modelleringen i detta projekt kommer därför göras för två fall som representerar lägsta och högsta möjliga styvheten.

För den statiska analysen undersöktes byggnadens horisontella förskjutningar vilka jämfördes med gränsvärdet på L/500 för fyra- och sexvåningsmodellen. Resultaten visade sig vara inom gränsvärdena för bägge byggnadshöjderna och för bägge modellerade styvheter. För den dynamiska analysen ges riktlinjer i ISO 10137 som involverar första egenfrekvensen samt maximala vind accelerationen. För sexvåningsmodellen överstiger resultaten riktlinjerna för alla fall utom det mest fördelaktiga med styvhet på 80 % och vind mot kortsidan. För modellen med fyra våningar var samtliga fall inom riktlinjerna förutom det mest kritiska med styvhet på 35 % och vind mot långsidan. Analysen visade vidare att effekten från ytterligare fackverk samt styvhet i golvelementen minskar då byggnadshöjden ökar. För höga byggnader räcker det inte med extra fackverk eller styvare golvelement, för att uppnå riktlinjerna krävs åtgärder för att öka egenfrekvensen genom att göra stommen styvare eller lägga in dämpare.

Nyckelord: träkonstruktioner, trä8, höga trähus

Contents

Preface

This master thesis was carried out as the concluding part of the master programme Structural Engineering and Building Technology which is part of the five-year civil engineering programme at Chalmers University of Technology. The thesis was carried out at Moelven Töreboda from January 2018 to June 2018.

I want to thank my colleagues at Moelven Töreboda for all help and support throughout this thesis, especially Fredrik Morell for initiating the project and Thomas Johansson who has been the supervisor. Further thanks to my supervisor at Chalmers Rasoul Atashipour and examiner Mohammad Al-Emrani, and lastly opponents Hanna Närhi and Viktor Wiberg.

Töreboda, June 2018

Stina Åkesson

1 Introduction

A challenge when constructing high-rise timber buildings is to ensure the stability. Compared to its load carrying capacity timber has a light self-weight. This is regarded as one of the many advantages of timber as a construction material as it facilitates erection. However, regarding dynamic performance a light self-weight is not favourable and as a result the stability, with regard to dynamic motions, can become an issue for high-rise timber buildings. The focus of this project is to investigate how horizontal stability against wind forces can be achieved in trä8 buildings using glulam trusses.

1.1 Background

A reason behind the increasing popularity of timber constructions, is that timber in many ways can be regarded as an environmentally friendly construction material. In the *Report of the World Commission on Environment and Development: Our Common Future* the term sustainable development is explained as a development in order to meet today's needs without compromising future generations ability to meet theirs (NGO Committee on Education, 1987). Consequently, with timber being a renewable resource (Svenskt trä, 2003), usage of timber goes in line with what is stated as a sustainable development. Furthermore, a big threat to our environment is the increasing global warming (Naturvårdsverket, 2017). The reason for the rising temperatures is a number of greenhouse gases which has increased in quantity due to human impact. The foremost greenhouse gas is carbon dioxide and with timber being part of the natural carbon cycle, it is not considered as a contributor to the increasing carbon dioxide in our atmosphere. Further proving timber to be a good choice of material with regard to the environment.

As the populations continues to rise in the big cities of Sweden (Statistiska centralbyrån, 2018) so does the demand of constructing high-rise buildings. High-rise buildings have been constructed for centuries (Guinness World Records, 2018) although construction of high-rise buildings made of timber is comparatively new. In fact, constructing timber buildings of more than two storeys were prohibited in Sweden until the mid 90's (Näringsdepartementet, 2004). Today due to the increasing environmental awareness the demand of high-rise buildings constructed in timber has reached its highest yet. However, construction of high-rise timber buildings is not a matter of course, but by overcoming the challenges of the material a step towards sustainable building is ensured.

Trä8 is a construction method used for construction of high-rise timber buildings developed by Moelven Töreboda (Moelven Töreboda AB, n.d.). Trä8 refers to a post beam structure that was originally stabilized by shear walls and rigid floor elements made of timber. It has been found that the capacity of the shear walls was not adequate (Tlustochowicz, Johnsson, & Girhammar, 2010) and these walls are now commonly replaced by glulam trusses. The trusses have shown to perform better than the shear walls, but further research of the trusses has not currently been made[1].

[1] Thomas Johansson, Structural engineer at Moelven Töreboda. Interview 2018-01-22.

1.2 Aim

This project aims to investigate the horizontal stability, with regard to wind forces, of a trä8 building stabilized by glulam trusses. Investigations are carried out for a six- and a four-storey model of the building Frostaliden 3 in Skövde.

For the purpose of the thesis the glulam trusses are designed regarding sizing and connections to withstand the vertical and horizontal forces they are exposed to. By modelling the complete structure of the building, the response is examined through a finite element analysis regarding horizontal static displacements and further the dynamic performance involving eigenfrequency and peak acceleration.

1.3 Research questions

Following questions are to be answered in the thesis.

- Does the four- and six-storey building fulfil the demands regarding static behaviour?
- Does the four- and six-storey building fulfil the demands regarding dynamic behaviour?
- How does the difference in building height affect the results?

1.4 Limitations and assumptions

Calculations of dimensions and connections of this building are limited to the trusses. Other components such as beams, columns and floor- and roof elements and also the connections between them is not considered in design and when modelling the complete structure, the dimension of these members are taken from the original design performed by the responsible engineer.

As the actual stiffness of the floor construction is yet to be determined the precise behaviour of the building cannot be established. The rigidity of each floor element is provided by a Kerto Q board and for this thesis the rigidity of the floor has been modelled by 35 % and 80 % of the stiffness of the board, within which range the actual stiffness is approximated to according to engineers involved with the particular floor elements. As a result, the actual behaviour of the building cannot be determined, the findings will rather circle the range of which the performance vary with respect to the stiffness of the floor construction.

Furthermore, also the stiffness of the connections between the glulam members is unknown. The connections, consisting of steel plates and dowels, will in reality have some resistance regarding moment but for this thesis all joints have been modelled as moment free as the actual stiffness is not determined. This applies not only for the trusses, but for all members of the structure.

The performance of the building model is only analysed with regard to wind induced motions. Motions and vibrations due to other sources such as footsteps, earthquakes and accidents is not considered. Moreover, other aspects such as durability, fatigue, manufacturing, foundation work or economic concerns is not covered.

1.5 Disposition

Following of this introduction is a theory chapter that describes some background and theory on which this thesis is based. Thereafter comes a chapter describing the considered object and the aids of the project and later a chapter describing the methods used for achieving the aims. Further the results are presented and discussed and in the last chapter the conclusions are drawn. In the following appendixes drawings, output data and calculations are gathered.

2 Theoretical background

This chapter consists of gathered information that describes the theory on which this project is based. The knowledge of this chapter was obtained as a result of an initial literature study.

2.1 Horizontal stabilization of structures

In order to make a structure stable it needs to be able to resist horizontal forces. A regular frame with moment free joints is not stable in its plane, see Figure 2.1A. In order to obtain stability there are a few possible alternatives: either by adding a diagonal, Figure 2.1B, adding stiff connections , Figure 2.1C and Figure 2.1D, or by adding a rigid plate, Figure 2.1E (Swedish wood, 2015).

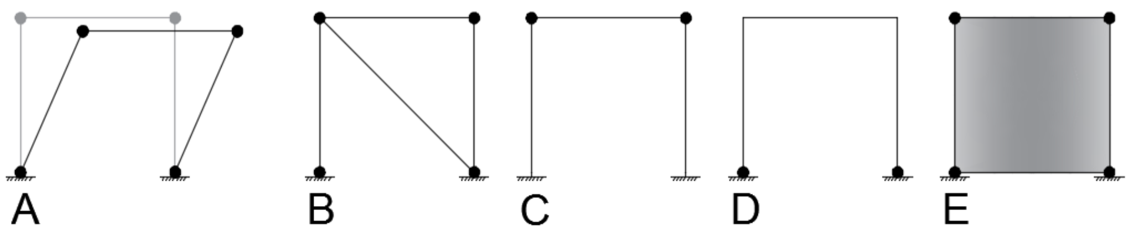

Figure 2.1. Methods on creating a stable frame construction.

To obtain horizontal stability for a building, one of these methods, see Figure 2.1, or combinations of them must be included in the structure (Swedish wood, 2015). Dependent on how the stabilizing frames are placed with regard to each other, the response of the building will differ. Considering the roof as rigid in its plane stability can be obtained as long as there are at least three stabilizing elements that are not parallel and does not meet in one common point (Svenskt trä, 2016b). In Figure 2.2 this is illustrated. Figure 2.2A shows the case where all elements are parallel, this only gives stability in one direction which is not sufficient. In Figure 2.2B all the elements have a common meeting point, this will cause rotation around this point which makes the construction unstable. By using only two elements, the elements will either be parallel or have a common meeting point, which means that two elements are too few. Figure 2.2C and Figure 2.2D fulfils all the criteria for a horizontally stable construction. However, the arrangement in Figure 2.2C cause torsional forces (Swedish wood, 2015). The size of the torsional forces depends on the size and shape of the building. If these forces become too large problems with regard to deformation might occur. A way to overcome the torsional forces is to place the stabilizing elements symmetrically in the structure as seen in Figure 2.2D.

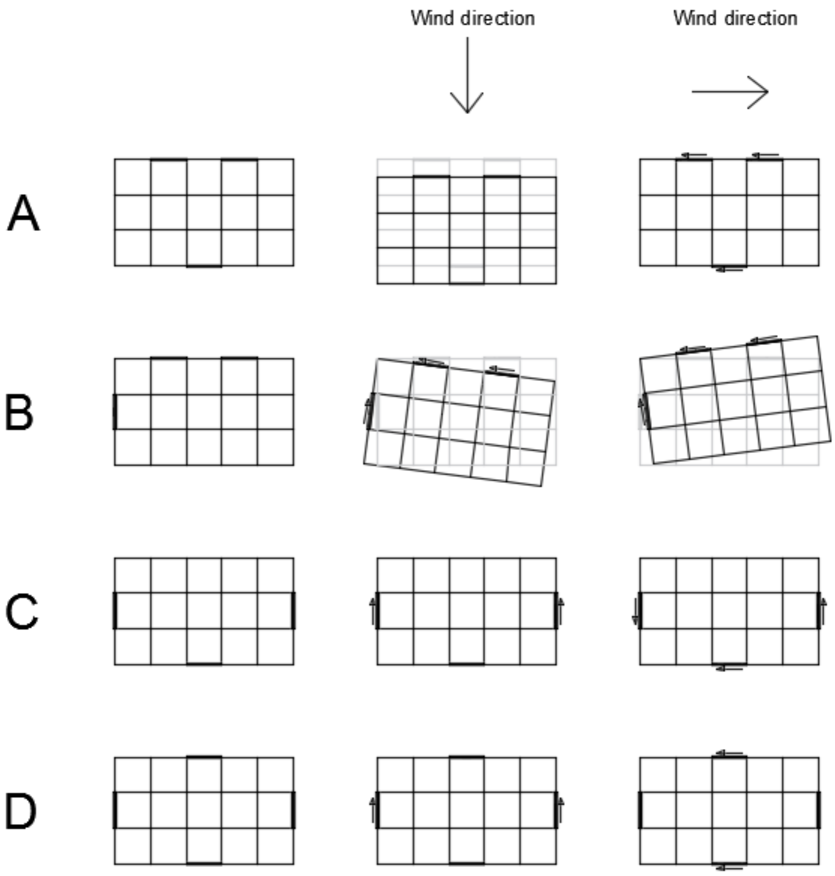

Figure 2.2 How the location of the stabilizing elements will affect the buildings behaviour.

For large buildings, requiring several stabilizing elements in the same direction, the force distribution will depend on the rigidity of the elements and the roof (Swedish wood, 2015). In Figure 2.3 models of a simple rectangular building with four equally spaced stabilizing elements is illustrated. In Figure 2.3A the model consists of a rigid roof and four, comparatively less rigid, stabilizing units. As the stabilizing units are of equal stiffness the same force will be acting in each of these units. In Figure 2.3B the roof is identical as in Figure 2.3A but the rigidity of the two middle units is decreased, due to less rigidity these will uptake less load linearly dependent of their rigidity. In Figure 2.3C the roof is flexible compared to the stabilizing units. This gives that the load resisted by each unit is dependent on the span length between them and not their rigidity. So in Figure 2.3A and Figure 2.3B the roof is considered as infinitely rigid compared to the stabilizing elements and in Figure 2.3C they are considered as infinitely rigid compared to the flexible roof. In reality there will obviously occur cases that are between these two extremes.

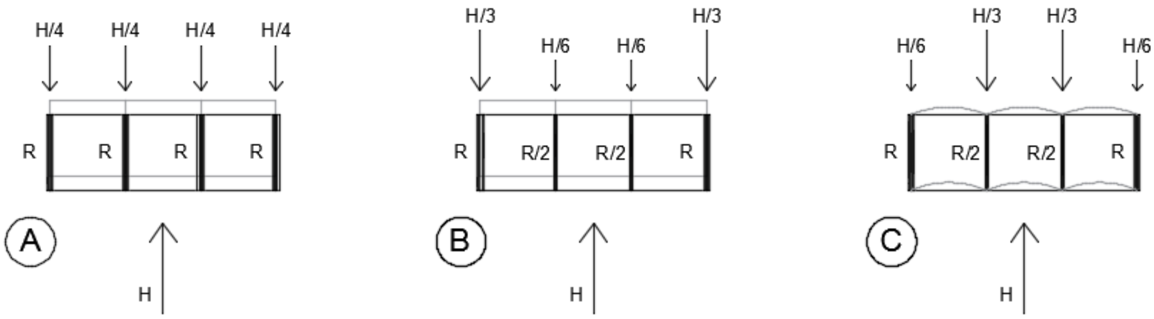

Figure 2.3 Distribution of reaction forces between stabilizing elements.

2.2 Dynamic behaviour of structures

A mass-spring system can be used to illustrate a dynamic behaviour (Strømmen, 2014). A mass attached to a spring set in motion will start to oscillate, however this oscillation will in reality eventually die out leaving the system at rest once again. This damping that occurs in reality is modelled by adding a damper to the system, see Figure 2.4.

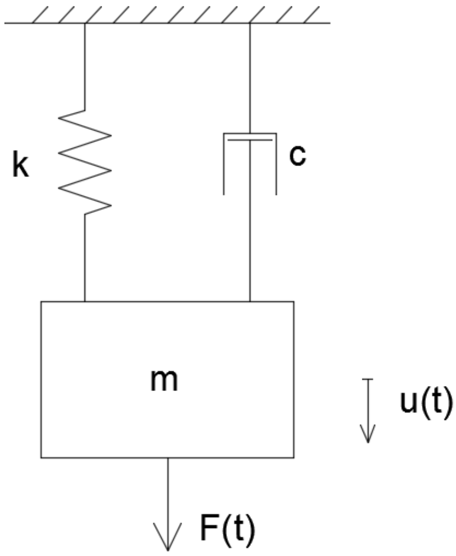

Figure 2.4 Simple mass-spring system with damper and an external time dependent force.

The dynamic equilibrium for the system presented in Figure 2.4 can be described by the equation of motion, see Equation (2.1) (Strømmen, 2014). For the equation of motion m denotes the mass in kg, c is the damping coefficient in Ns/m and k is the elastic spring constant in N/m. $F(t)$ is an external force in N and $\ddot{u}(t)$, $\dot{u}(t)$ and $u(t)$ describes acceleration, velocity and displacement in m/s^2, m/s and m respectively.

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = F(t) \tag{2.1}$$

The eigenfrequency, or the natural frequency, of a system is a frequency for which the system will oscillate without any driving forces. This means that very small forces applied with this frequency can cause vibrations of great amplitude. For an undamped system the eigenfrequency can be determined by using Equation (2.2) (Strømmen, 2014).

$$\omega_n = \sqrt{k/m} \tag{2.2}$$

Using the eigenfrequency for the undamped case, the damping ratio can be determined for damped systems by Equation (2.3) (Strømmen, 2014).

$$\zeta_n = \frac{c}{2m\omega_n} \tag{2.3}$$

If the value of the damping ratio, ζ_n, is equal to or greater than one this means that no oscillations will occur (Strømmen, 2014). Therefore, no eigenfrequency can be obtained for such systems. When ζ_n<1 the damped eigenfrequency can be calculated using Equation (2.4).

$$\omega_d = \omega_n\sqrt{1 - \zeta_n{}^2} \tag{2.4}$$

Once the eigenfrequency of a given system is obtained the corresponding eigenmode, φ, can be determined from Equation (2.5) (Strømmen, 2014). The eigenmode is also called the shape mode and describes the shape of the oscillating system.

$$(k - \omega^2 m)\varphi = 0 \tag{2.5}$$

In this chapter the equation of motion is described for a system with only one degree of freedom, it is however applicable for all kind of systems (Strømmen, 2014). For multiple degree of freedom systems, the factors for mass, damping and stiffness will be arranged in matrices with number of columns and rows corresponding to the degrees of freedom and $\ddot{u}(t)$, $\dot{u}(t)$, u(t) and $F(t)$ becomes vectors of equal length. Consequently, this results in multiple eigenfrequencies and corresponding eigenmodes, and the resulting shape of motion is usually a combination of all or some of the eigenmodes. The lowest eigenfrequency is the most critical and is referred to as the fundamental frequency.

From these equations it can be read that the eigenfrequencies are dependent on the stiffness, the mass and the damping of the structure. Measures to be taken in order to increase the eigenfrequency is to increase the stiffness or decrease the mass. However, decreasing the mass will cause increased accelerations according to Newton's second law, see Equation (2.6), which is not preferable for a building structure.

$$F = ma \tag{2.6}$$

2.2.1 Recommended guidelines

According to Eurocode (European Commitee for Standardization, 2002) the comfort criteria should be evaluated if the fundamental frequency, i.e. the lowest eigenfrequency, is lower than 5 Hz in vertical vibration and lower than 2,5 Hz for horizontal and torsional vibrations. Further the comfort criteria can be stated individually for given projects however a recommended maximum value is stated at 0,7 m/s^2 for vertical vibrations and 0,2 m/s^2 for horizontal vibrations.

In ISO 10137 guidelines are provided with regard to wind-induced motions (Swedish Standards Institute, 2008) with respect to peak accelerations and the fundamental frequency of the building. The guidelines are based upon humans’ perception of the vibrations; however, it is stated that humans’ perception vary with individuals, activity and context. An article by Kwok, Hitchcock and Burton reviews tests done on humans’ interpretation of vibration and conclusions are drawn that both psychological and physiological factors affect the response (Kwok, Hitchcock, & Burton, 2009). Due to difference in activity ISO 10137 provide guidelines of different magnitude dependent on if the facility is used for offices or residences as seen in Figure 2.5. The guidelines for residences constitutes of 2/3 of what is accepted for offices.

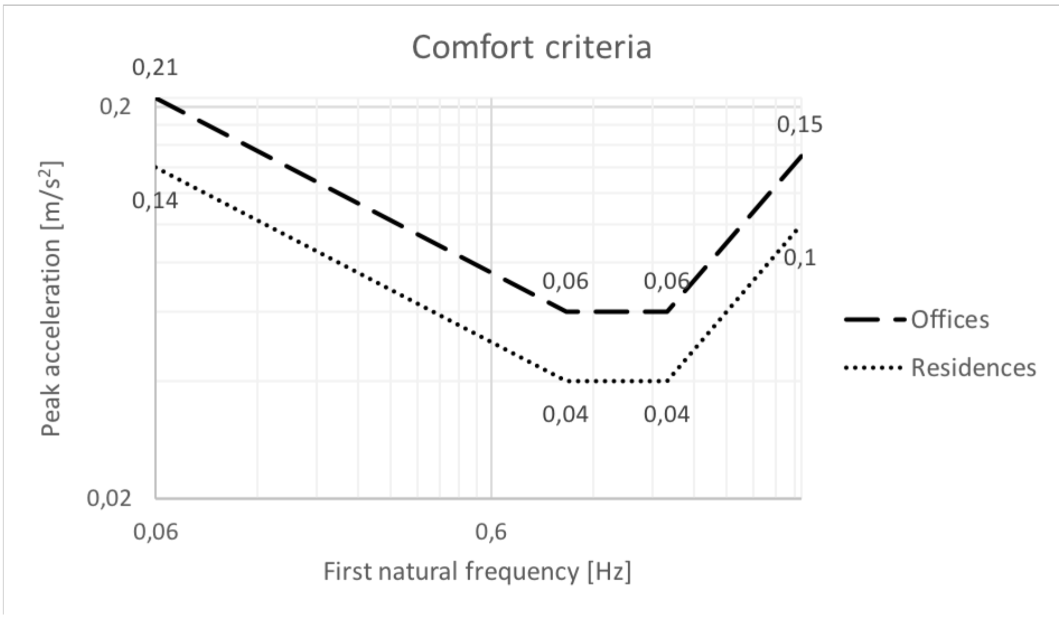

Figure 2.5 Guidelines of vibrations from ISO 10137 (Swedish Standards Institute, 2008).

2.3 Finite element modelling

The finite element method is a calculation method used to solve complicated problems in a simplified manner. Differential equations are widely used to describe the behaviour of structures and materials and when the analytical solution of these equations become too complicated the finite element method provides an approximate numerical solution (Ottosen & Petersson, 1992).

A finite element analysis consists of dividing the considered structure or member into small regions, finite elements, and then perform a numerical approximation over each of these elements (Ottosen & Petersson, 1992). The division in elements is called the finite element mesh, and the size of the mesh determines the accuracy of the approximation. Boundary conditions are set at points where the output is known, and the behaviour of adjacent elements follows from given conditions.

An example of how to use finite element approximation is shown in Figure 2.6. In this example the temperature varies non-linearly over a rod. In the finite element analysis, the temperature is approximated to vary linearly over the finite elements, and as seen in Figure 2.6 the accuracy of the approximation rises with the number of elements used.

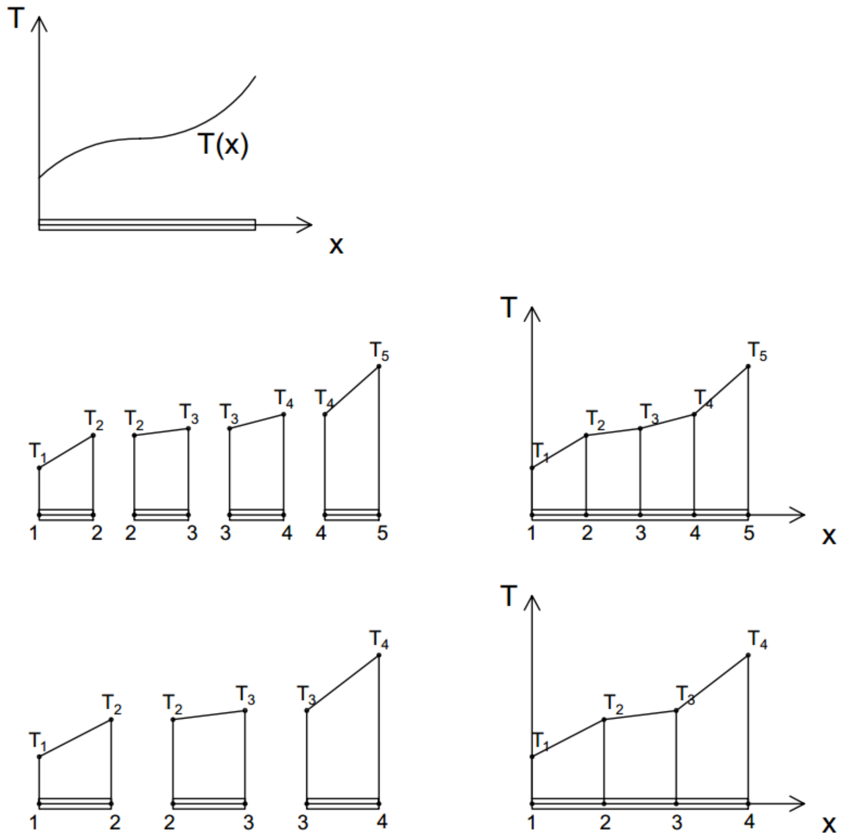

Figure 2.6 Example of how to use finite element modelling. Approximation of the temperature distribution over a rod divided in three respectively four elements.

2.4 Timber as a construction material

Many devastating city fires occurred in Sweden during the 1800s and as a result of that it was stated in 1874 that no more than two storeys were allowed for timber constructions (Näringsdepartementet, 2004). When Sweden later joined EU the regulations changed. In 1995 Boverket (National Board of Housing, Building and Planning) gave restrictions with regard to functionality of the construction rather than choice of material. This allowed for timber to become an alternative construction material for multiple storey buildings.

Usage of timber in construction has many advantages, especially regarding environmental issues as timber is renewable and carbon neutral but also regarding its self-weight putting less pressure on the foundation than a corresponding concrete structure would. However, ordinary timber logs have some limitations. Since timber is a natural material it is obvious that variations in quality will occur from log to log. The timber logs also have limitations regarding size and shape (Svenskt trä, 2016a). To overcome these limitations engineered wood products have become a solution (Swedish wood, 2015). Engineered wood products, EWP, consists of wooden parts that are put together with adhesives.

2.4.1 Glulam

Glulam, which refers to glued laminated timber, was the first EWP to be developed (Swedish wood, 2015). It was the German Otto Hetzer who was the first to introduce glulam as a feasible construction material (Svenskt trä, 2016a). By gluing wooden lamellas together, he was able to achieve large cross-sections which ensured enough capacity for long span structures. Hetzer also created curved beams, which he got the patent for in 1906. Economically glulam showed to be advantageous compared to steel and reinforced concrete regarding great span structures, and as a result glulam became a popular construction material for railway buildings and hangars. In Sweden the first glulam factory was established in 1919 in Töreboda, and it provided glulam for the constructions of the central stations in Stockholm, Gothenburg and Malmö, which was all constructed during the 1920s (Svenskt trä, 2016b) and are still standing today, see Figure 2.7 and Figure 2.8.

Figure 2.7 Stockholm central station with glulam frames manufactured during the 1920s.

Figure 2.8 Malmö central station (left) and Gothenburg central station (right) with glulam manufactured during the 1920s.

Glulam consists of lamellas of timber that are finger jointed and glued together to the dimension and length of choice. The lamellas are usually sawn from fir but also pine can be used (Svenskt trä, 2016a). Before the different lamellas are joint they are graded with regard to strength (Swedish wood, 2015). Depending on the strength of the lamellas the strength of the composed glulam beam can differ, the standard strength class of glulam beams manufactured for the Swedish market is GL30 (Svenskt trä, n.d.). The number 30 corresponds to the characteristic bending strength in N/mm^2 (Svenskt trä, 2016a). The strength class is usually followed by a letter *c* for combined or *h* for homogeneous. The combined beams have lamellas of higher strength in the upper and lower parts of the cross-section while lamellas placed in the middle are of less strength. The reason for this arrangement is that the maximum stresses from bending will occur in the top and bottom parts while the stresses in the middle part will be of less magnitude (Svenskt trä, n.d.). By limiting the high strength lamellas to where the maximum stress occurs an efficient use of material is achieved. The homogeneous beams have same strength lamellas throughout the cross-section, which is suitable for columns where the stress is more evenly distributed (Svenskt trä, 2016a).

Comparing the strength of glulam with ordinary solid timber beams of the same size, the difference is neglectable (Swedish wood, 2015). What is obvious is though that the variation in strength is smaller for glulam. An explanation for this is that since the wood is cut in pieces the imperfections will be spread throughout the beam. One bigger defect localised to one area of a solid timber beam is likely to cause early failure in that point, however for glulam each imperfection has less effect on the strength since they are smaller and evenly distributed over the whole beam. Considering the effects of moisture in wood, glulam will expand and shrink with differing moisture content in the same way as a solid timber beam. Though, since glulam consists of lamellas originating from different parts of the log, the glulam beam is less likely to bend and turn like an ordinary timber beam might.

Glulam can be made in straight and curved beams (Svenskt trä, 2016a). The straight beams have a lamella thickness of 45 mm and curved beams at 33 mm. The available heights of glulam beams are generally multiples of the lamella thickness. The maximum height is dependent of the machinery at manufacturing and is often around 2 m. The width is dependent on the available width of lamellas and the maximum is usually at 215 mm. If a wider beam is desired the lamellas can be glued together which will give widths up to 430 mm. Sizing is also limited with regard to transportation, and the maximum length of beams is normally restricted to 30 m and for curved beams also the width and height must be taken into account.

2.4.1.1 Material properties

The building investigated in this project will have beams, columns and trusses made of glulam of class GL30c. The material properties of GL30c can be found in Table 2.1.

Table 2.1 Material properties of glulam GL30c.

GL30c		
Modulus of elasticity	E	13 000 MPa
Shear modulus	G	650 MPa
Poisson's ratio	ν	9
Specific weight	ϒ	4,3 kN/m^3

2.4.2 LVL

LVL stands for laminated veneer lumber and is a material made from thin veneer sheets, 2-4 mm, glued together (Swedish wood, 2015). LVL can be made into beams or boards depending on the direction of the fibres (Moelven Töreboda AB, n.d.). Kerto S is the name of the beam material with all fibres in the same direction and Kerto Q has an 80/20 distribution of fibres which makes it suitable for boards.

2.4.2.1 Material properties

In this project Kerto Q boards will be used in the floor and roof. The material properties of Kerto Q can be found in Table 2.2.

Table 2.2 Material properties of Kerto Q.

Kerto Q		
Modulus of elasticity, x	E_x	10 500 MPa
Modulus of elasticity, y	E_y	2 000 MPa
Shear modulus, yz	G_{yz}	22,0 MPa
Shear modulus, xz	G_{xz}	120 MPa
Shear modulus, xy	G_{xy}	600 MPa
Poisson's ratio	ν_{xy}	0
Poisson's ratio	ν_{yx}	0

2.5 The construction method trä8

Trä8 was the name of a construction method in timber that was launched in 2009 by Moelven Töreboda (Moelven Töreboda AB, n.d.). It was a system for buildings up to four storeys consisting of columns and beams, and with floor elements that could span up to eight metres. The columns and beams of the system was made of glulam and for floor, roof and shear walls prefabricated elements were used consisting of LVL and glulam. The roof and floor elements consisted of beams and boards both in LVL. The stabilization elements where made in shapes of L and T which made them stable on their own. The stabilization elements consisted of LVL boards mounted on glulam frames.

The stabilizing system developed for the trä8 system was only used in two projects[2], this was due to insufficient capacity. A report by Tlustochowicz, Johnsson and Girhammar (2010) shows that the failing mechanism originates from the glued in rods at the foundation. To solve the problems the stabilizing elements were replaced by

[2] Thomas Johansson, Structural engineer at Moelven Töreboda. Interview 2018-01-22

glulam trusses. The construction of the floor elements has also changed in construction since the launch of the trä8 system[1]. Today the LVL beams of the floors has been replaced by glulam, and the reason for that is mainly in order to favour the factory in Töreboda as the Kerto beams are imported.

Even though the construction methods used today have developed from the original idea of trä8 the term is still used today, but today the term is referring to Moelven Töreboda's gathered methods of building high timber buildings rather than the system developed in 2009 (Moelven, n.d.). These methods include some standardizations as the continuous tall glulam columns with standardized footings as well as standardized joist hangers on which the beams are fastened to the columns[1]. Furthermore, the rigid floor elements are common for all trä8 houses built today and the horizontal stability is usually achieved by trusses and is sometimes complemented by a concrete core for stairwells.

Another common way of stabilization of just high-rise timber buildings is usage of cross laminated timber, CLT (Mills, 2017), which is rigid wall elements made of offcuts from sawmills that are glued together (Svenskt trä, 2017). The advantage of these components is that they are very rigid and also made from waste products. However, the use of material is inefficient compared to glulam trusses and further since it requires a sawmill nearby the factory it is not an option for Moelven Töreboda's way of construction[3].

[3] Thomas Johansson, Structural engineer at Moelven Töreboda. Interview 2018-01-22.

3 Basis of analysis

This chapter refers to describing the building to be analysed and also the aides used for analysis.

3.1 Frostaliden 3

The building to be examined in this thesis is Frostaliden 3 located in Skövde. The building is part of the residential area Frostaliden which will be the biggest residential area consisting of high-rise timber buildings in Sweden once it is finalised (Skövde Kommun, 2017). The area is part of a national project called Trästad 2012, whose purpose is to develop the commonly used construction methods of today and promote usage of timber as a construction material (Länsstyrelsen Dalarnas län, n.d.). The Frostaliden area is aimed to serve as inspiration regarding its environmentally friendly attributes, encouraging sustainable construction (Skövde Kommun, 2017).

Figure 3.1 The considered building, Frostaliden 3.

Frostaliden 3, see Figure 3.1, was finalised around the turn of the year 2017-2018 and is a six-storey apartment building by the developer Götenehus (Götenehus, n.d.). The floors consist of two, three and four room apartments of 64-91 m^2 with accompanying balconies and the top floor apartments also have roof terraces. The height between each floor is approximately 3 m. The architect for the building was Staffan Morud from CH Arkitekter (Götenehus, 2016) and the building structure was developed by Moelven Töreboda AB. The structure follows the trä8 concept and is further stabilized by a concrete core for the staircase, see plan in Figure 3.2.

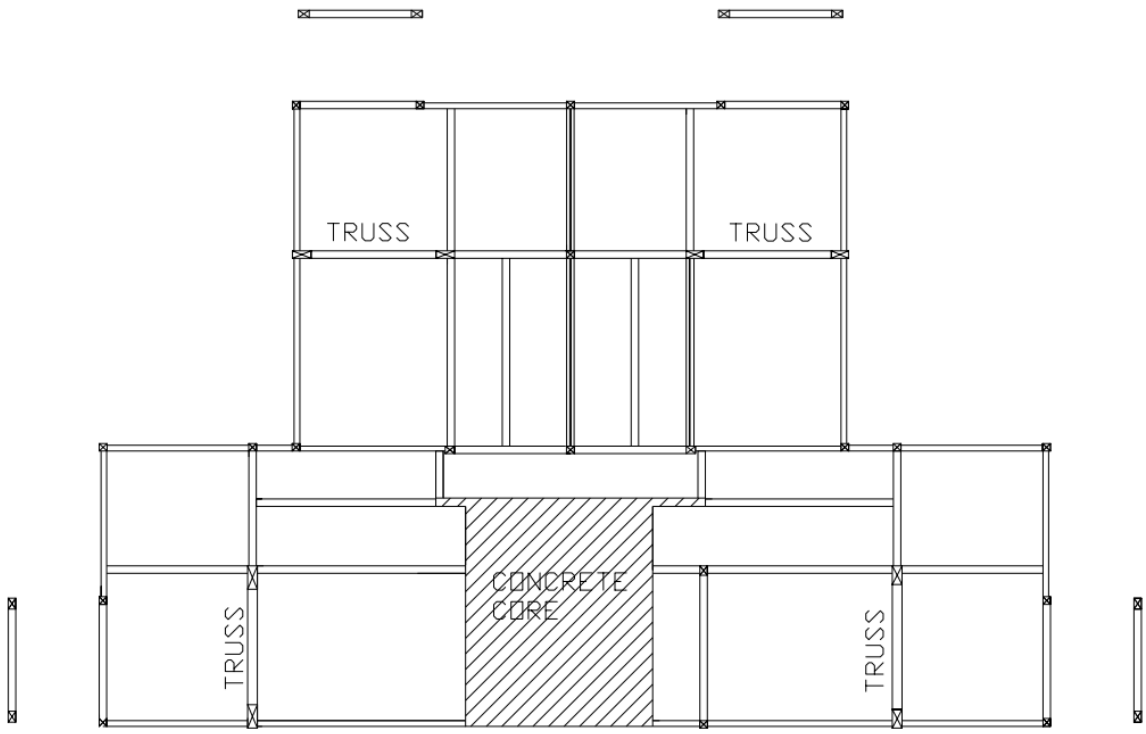

Figure 3.2 Original layout of Frostaliden.

3.1.1 Modifications in the building model

This thesis refers to examine the wind induced motions of a four- and a six-storey building. For the four-storey building the two bottom floors of Frostaliden 3 was removed but otherwise the building was modelled the same way. Changes were also made regarding the stabilizing system. In order to obtain a building with timber as the only construction material the concrete core was removed and replaced by additional trusses. The layout of the additional trusses was established in agreement with a qualified engineer regarding number and placement. The layout was modelled as shown in Figure 3.3 and Figure 3.4, describing the placement of trusses as well as orientation of floor elements and measurements.

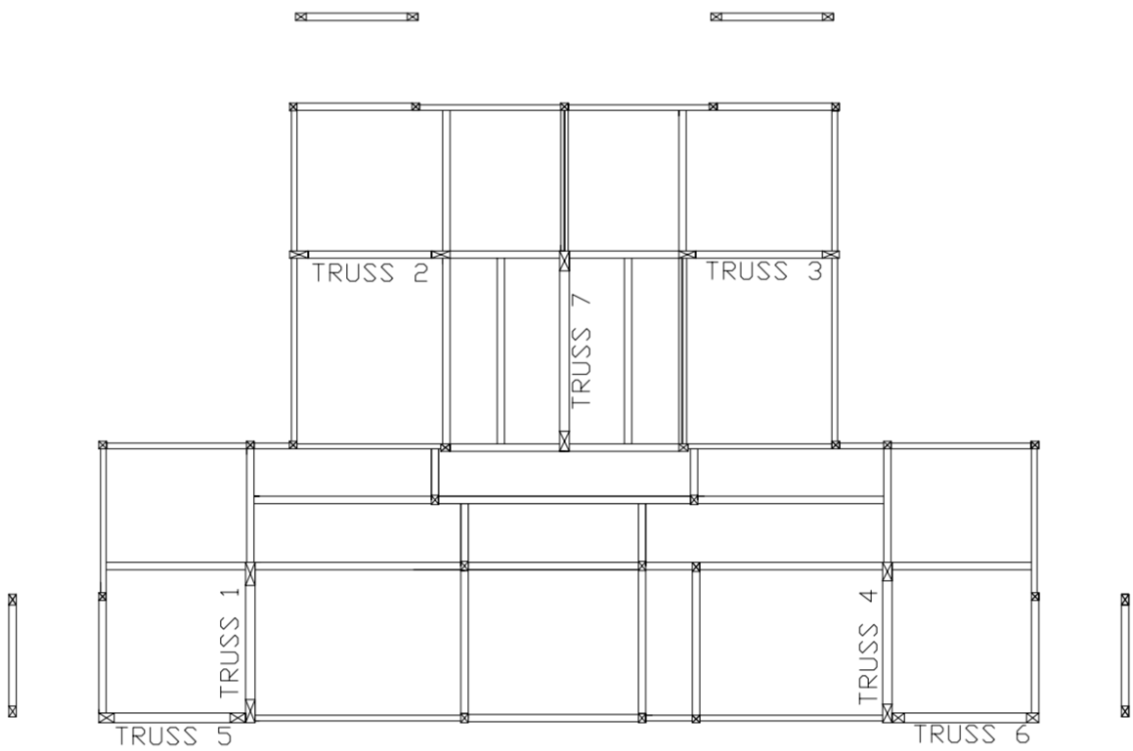

Figure 3.3 Numbering and position of original trusses and the added ones compensating for the concrete core.

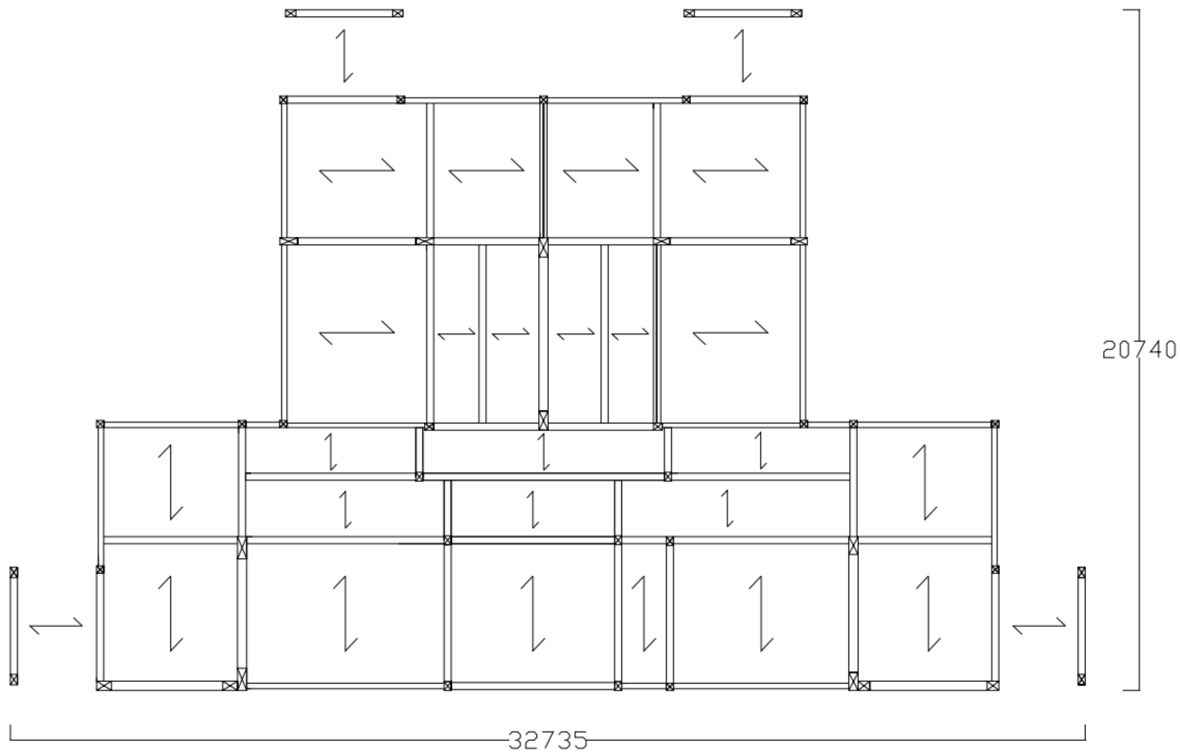

Figure 3.4 Outer measurements and orientation of floor elements.

3.2 Computational aids

For the purpose of this thesis, trusses should be designed, and the structure should be modelled and analysed with regard to its performance regarding horizontal wind forces through a finite element analysis. This was done using computational softwares which are described below.

3.2.1 RFEM

Dlubal RFEM is a structural analysis program which allows the user to model complete 3D structures of plates, shells, walls and members (Dlubal, 2018). The output on the analysed structure involves deformations, internal forces, stresses and support reactions. For this thesis RFEM was used both in order to get the appropriate dimensions of the glulam trusses and later when analysing the complete structure with regard to statics and dynamics. When designing the trusses, the add-on module RF-TIMBER Pro was used, which provides the utility of the timber components. For the dynamic analysis the add-on module RF-DYNAM Pro was used to get the natural frequencies.

3.2.2 Statcon

Statcon is a computational software developed by Elecosoft (Elecosoft, 2017). For this thesis the version Statcon connections was used for designing the connections between the glulam members of the trusses. By using the forces of the members of the truss obtained from RFEM, the connections were designed with regard to the capacity of the connectors as well as the capacity of the glulam members.

4 Method

The methodology for this thesis can be divided in three parts; the initial literature study, designing of trusses and lastly, modelling and analysis of the complete structure, see further below.

4.1 Literature study

The project was initiated by a study of literature in order to gain knowledge about the area to be analysed. This was done by studying released books, searching information through online databases and also by discussing with professionals in the area. By initiating the project with gathering of information a deepened understanding for the theory behind the calculations and analysis was gained which increased efficiency and reduced the risk of mistakes during calculation and analysis.

4.2 Design of glulam trusses

Before any calculations were initiated the arrangement of trusses was set with regard to the floor plan. This was done by identifying sections in the building where the trusses could be placed without interfering with the floor plan or the outlook of the facades. The width of the trusses was set around 4-6 m wide.

4.2.1 Calculation of forces

In order to design the trusses, the horizontal forces acting on the building must be determined. In this case the horizontal forces were determined from wind and initial imperfections. Calculations of wind was done according to Eurocode 1 Part 1-4 (European Committee for Standardization, 2010). Calculations of the initial imperfections include vertical loads regarding self-weight, snow and imposed loads. The snow load was taken from Swedish Eurocode 1 Part 1-3 (Swedish Standards Institute, 2015) and the values for imposed loads was taken from the Swedish Eurocode 1 Part 1-1 (Swedish Standards Institute, 2009), see calculations in Appendix C.1. When the total horizontal force, acting on each storey of the building, was determined, the force was distributed on the trusses resisting force in that direction. This was done by modelling a continuous beam of same length as the building in that direction and with supports acting at the same distances as the trusses were placed. By comparing the reaction forces the distribution factors could be determined, see Appendix C.2.

Since the trusses, additionally to the diagonal struts, consists of beams and columns they are also responsible for vertical load carrying. Load carrying beams are mounted on the columns of the truss and in some cases the horizontal members of the truss have floor and roof elements resting on them. Therefore, also vertical forces must be taken into account while designing. Calculations of all the loads acting on the trusses can be found in Appendix C.3.

4.2.2 Dimensioning of glulam components

The design of the trusses was carried out using the software RFEM, based on SS-EN 14080:2013-08. 2D models of all the trusses was made and the previously calculated forces, both horizontal and vertical, was applied to the structures. The add-on module RF-TIMBER Pro was used in order to examine the utility of the members and the consequence class was set to three. The connections consist of slotted steel plates fastened with dowels, and since parts of the glulam needs to be cut out in order to make this kind of connection the capacity of the glulam is slightly lowered. To account for this the maximum utility was set at 70 % in ULS. Moreover, the displacements of the truss must be considered, and limits were set at L/500 for horizontal displacements and L/300 for vertical deformations. By ensuring that these criteria were fulfilled the preliminary dimensions of the glulam components could be determined, see Appendix B.1.

4.2.3 Design of dowelled connections

The connections were designed using the computer software Statcon, based on EKS10. This was done by modelling each connection for all of the trusses and adding the loads in each member obtained from the RFEM models. Two load cases were applied to each connection representing forces acting from the left and right side of the truss, see Figure 4.1.

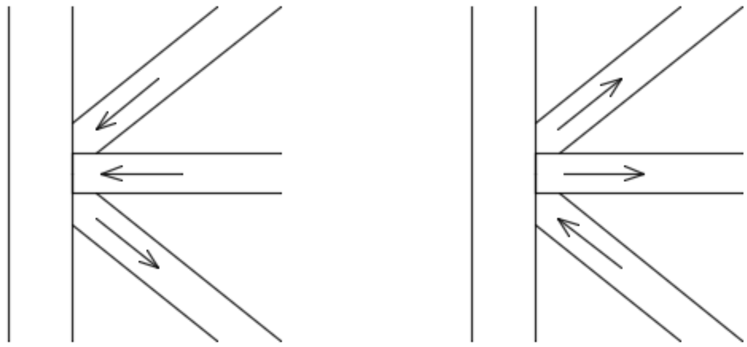

Figure 4.1 Load cases for design of connections.

The connections consist of 8 mm thick steel plates and dowels of 12 mm in diameter both of quality S355. The spacing of the dowels was set according to Eurocode 5 (Swedish Standards Institute, 2004) which gave limits at 48 mm perpendicular to the fibres and 80 mm parallel to the fibre direction. At the edges the limits were 84 mm parallel to the fibres and 48 mm perpendicular, see Figure 4.2.

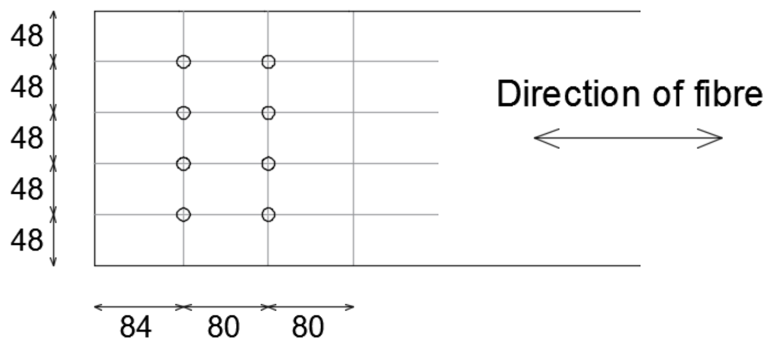

Figure 4.2 Arrangement of dowels.

The design of the connections was made with the following criteria:

- As few steel plates as possible, and maximum three
- Maximum five dowels in a row parallel to the fibres
- Minimum four dowels in each member
- Maximum 70 % utility

In cases were all these criteria could not be met the dimensions of the glulam was enlarged in order to fit more dowels. The result of the design of connections can be seen in Appendix B.2, and drawings of the final design of trusses can be seen in Appendix A.2 and Appendix A.3.

4.3 Analysis of the complete structure

For analysis of the static and dynamic behaviour of the building, models of the entire structure of the building were made, incorporating the trusses that previously was designed.

4.3.1 Modelling of building structure

The structures of the two buildings were modelled in RFEM. The floor elements were modelled only by the Kerto Q board which constitutes the rigidity of the elements. Two cases were modelled for each building, one with 35 % stiffness of the Kerto Q board and one case with 80 % stiffness. As the floors and roof only where modelled with regard to rigidity and not load carrying, the loads acting on the elements were distributed directly on the beams on which they rest. Calculations of loads acting on the FE model can be found in Appendix C.4. Load combinations of the applied loads were generated by RFEM for SLS.

The modelled structures consist of nodes, lines and surfaces meaning that all components are modelled centre to centre. This cause problems at places where the columns have large cross sections and the centre of the column doesn't line up with where the beam should attach. This was the case especially for the columns of the trusses and was solved by making rigid couplings from the node of the column to a node where the beam should be fastened.

A picture of the modelled structure for Frostaliden 3 with six storeys can be seen in Figure 4.3. The pink and blue colours of the floors correspond to the direction of the floor elements as the Kerto Q board have different material properties in x- and y-direction. In Figure 4.4 the finite element mesh is shown for the same model, the length was set to 0,5 m.

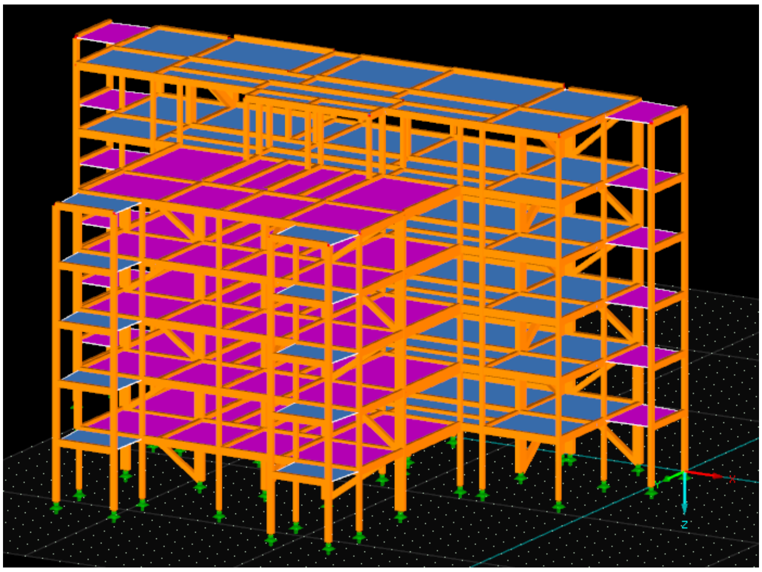

Figure 4.3 Structural model of Frostaliden.

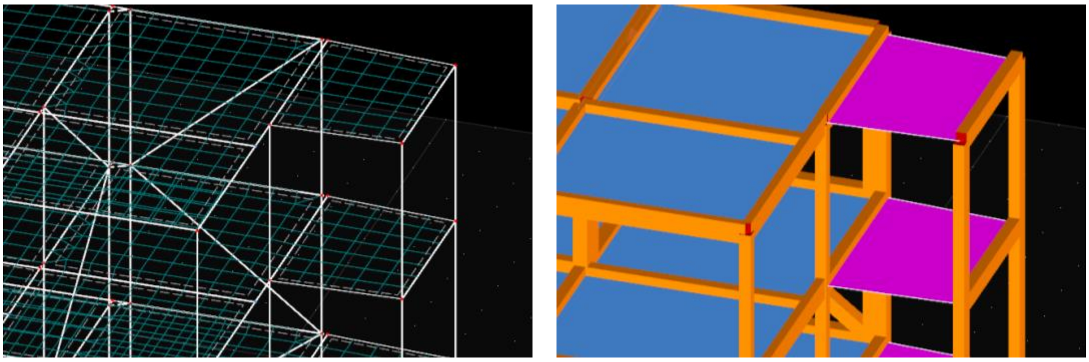

Figure 4.4 Illustration of the FE-mesh for Frostaliden.

4.3.2 Static analysis

The static behaviour of the structure was examined with regard to horizontal displacements at SLS. The displacements were obtained from RFEM and was compared to the limit of L/500, see outputs in Appendix B.3.

4.3.3 Dynamic analysis

The dynamic response of the building consists of determining eigenfrequencies and peak accelerations in SLS and comparing them to the recommended guidelines set in ISO 10137, see Figure 2.5. The eigenfrequencies were obtained in RFEM using the add-on module RF-DYNAM Pro and the peak accelerations were determined by calculations following Eurocode (European Committee for Standardization, 2010), EKS 10 (Boverket, 2016) and BSV 97 (Boverket, 1997). See full calculations in Appendix C.5.

After determining eigenfrequencies and peak accelerations for the original design idea additional trusses was added in order to investigate the importance of each truss, see layout in Figure 4.5. Placements of these trusses were set by studying the fundamental eigenmode of the building and placing trusses to prevent the particular movement, see Appendix B.4. All of the additional trusses were set to the same dimensions, see Table 4.1, and no calculations were carried out regarding design. As these trusses are mainly to describe the approximate behaviour additional calculations were assumed to be unnecessary. Some of these trusses also interfere with the placements of windows in the façade design, which makes them unfeasible in reality.

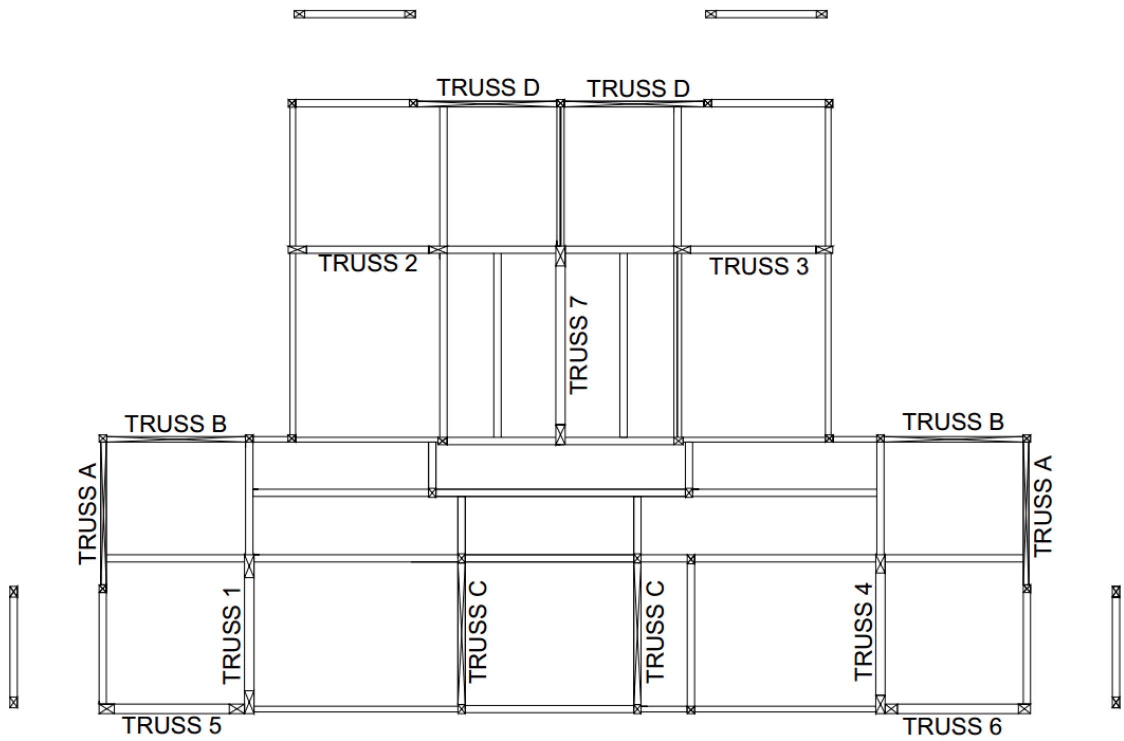

Figure 4.5 Layout of Frostaliden along with additional trusses A-D.

Table 4.1 Dimensions of the components of the additional trusses.

Additional trusses	Six-storey model	Four-storey model
Vertical members	280x540	215x360
Horizontal members	280x270	215x225
Diagonal members	280x360	215x225

Further analysis was made in order to investigate the impact of the height of the building. For this purpose, models were made with five, three and two floors as well and the obtained fundamental eigenfrequencies was compared. As also this was done just to analyse the approximate behaviour, and therefore no further calculations were carried out for the matter and this was only performed regarding the original design of truss layout.

5 Results

The results of the analysis for both buildings regarding statics and dynamics is presented below. The resulting design of trusses can be found in Appendix A.2 and Appendix A.3.

5.1 Static analysis

Below the results from the static analysis will be presented. The defined coordinate system can be read in Figure 5.1.

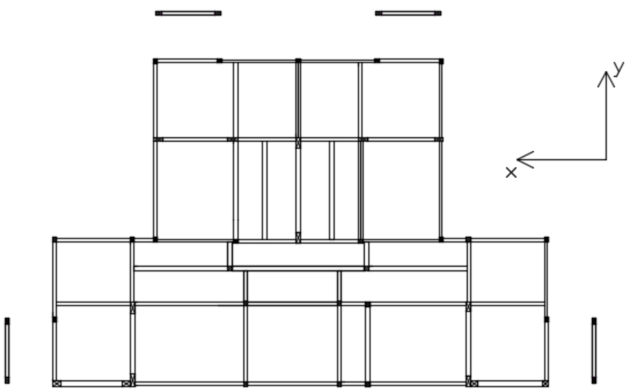

Figure 5.1 Description of the defined axes for the model of Frostaliden.

The limit of L/500 for the six-storey model gives a maximum displacement of 36,8 mm. The horizontal displacements of the six-storey model are shown in Table 5.1 and further in Appendix B.3.

Table 5.1 Horizontal displacements of Frostaliden, six floors, at static analysis.

Six-storey model	k=0,8	k=0,35
$u_{x,max}$	5,9 mm	8,3 mm
$u_{x,min}$	-6,1 mm	-8,6 mm
$u_{y,max}$	13,7 mm	16,2 mm
$u_{y,min}$	-8,6 mm	-10,6 mm

For the four-storey model the limit of L/500 gives maximum displacement at 24,8 mm. Results from the model are shown Table 5.2 and further in Appendix B.3.

Table 5.2 Horizontal displacements of Frostaliden, four floors, at static analysis.

Four-storey model	k=0,8	k=0,35
$u_{x,max}$	2,7 mm	6,1 mm
$u_{x,min}$	-2,8 mm	-6,3 mm
$u_{y,max}$	5,8 mm	11,6 mm
$u_{y,min}$	-3,6 mm	-6,8 mm

The results are further illustrated and compared to the limit in Figure 5.2 and Figure 5.3.

Figure 5.2 Comparison of the static response to the limit for the six-storey model.

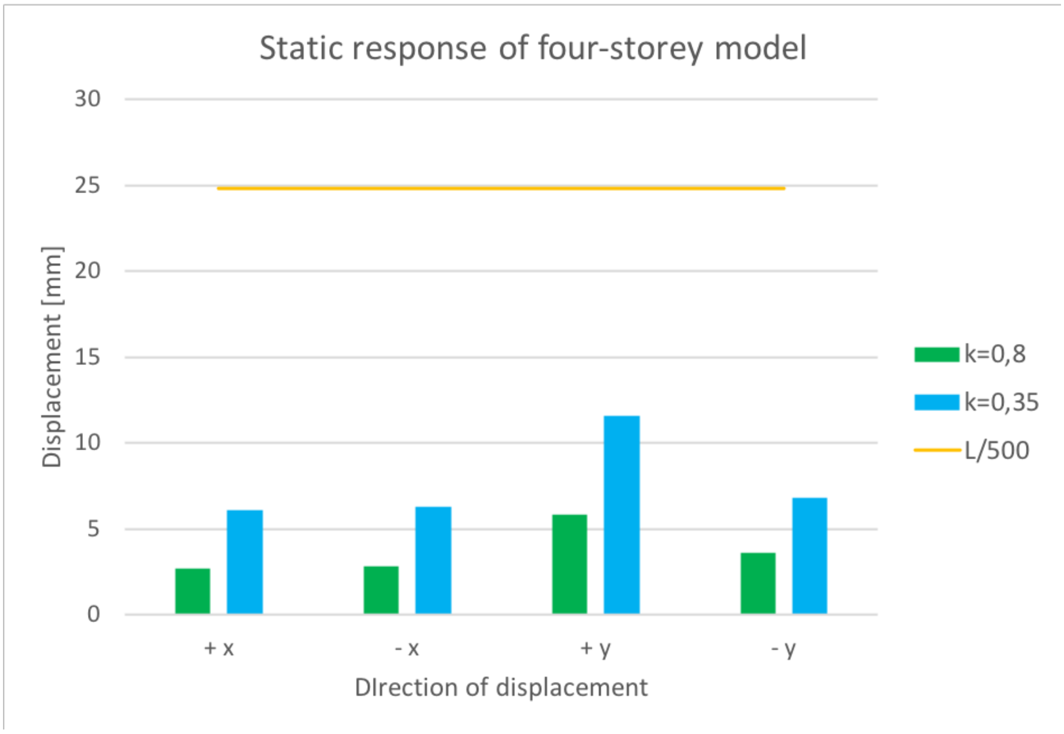

Figure 5.3 Comparison of the static response to the limit for the four-storey model.

5.2 Dynamic analysis

The results from the dynamic analysis for the six-storey model can be read in Table 5.3 and the results from the four-storey model in Table 5.4. The different mode shapes are found in Appendix B.4.

Table 5.3 Eigenfrequency and peak acceleration for different truss plans of Frostaliden, six floors.

Six-storey model	Wind on short side		Wind on long side	
	k=0,8	k=0,35	k=0,8	k=0,35
Original design				
Natural frequency	1,566 Hz	1,315 Hz	1,566 Hz	1,315 Hz
Peak wind acceleration	0,036 m/s^2	0,043 m/s^2	0,058 m/s^2	0,070 m/s^2
Additional trusses: A, B				
Natural frequency	1,922 Hz	1,697 Hz	1,922 Hz	1,697 Hz
Peak wind acceleration	0,028 m/s^2	0,033 m/s^2	0,047 m/s^2	0,053 m/s^2
Additional trusses: A, B, C				
Natural frequency	1,951 Hz	1,866 Hz	1,951 Hz	1,866 Hz
Peak wind acceleration	0,028 m/s^2	0,029 m/s^2	0,046 m/s^2	0,048 m/s^2
Additional trusses: A, B, C, D				
Natural frequency	1,972 Hz	1,880 Hz	1,972 Hz	1,880 Hz
Peak wind acceleration	0,028 m/s^2	0,029 m/s^2	0,045 m/s^2	0,048 m/s^2

Table 5.4 Eigenfrequency and peak acceleration for different truss plans of Frostaliden, four floors.

Four-storey model	Wind on short side		Wind on long side	
	k=0,8	k=0,35	k=0,8	k=0,35
Original design				
Natural frequency	2,575 Hz	1,543 Hz	2,575 Hz	1,543 Hz
Peak wind acceleration	0,022 m/s^2	0,038 m/s^2	0,035 m/s^2	0,062 m/s^2
Additional trusses: A, B				
Natural frequency	3,031 Hz	2,159 Hz	3,031 Hz	2,159 Hz
Peak wind acceleration	0,018 m/s^2	0,026 m/s^2	0,029 m/s^2	0,043 m/s^2
Additional trusses: A, B, C				
Natural frequency	3,042 Hz	2,315 Hz	3,042 Hz	2,315 Hz
Peak wind acceleration	0,018 m/s^2	0,024 m/s^2	0,029 m/s^2	0,040 m/s^2

The results of the original design idea compared to the guidelines from ISO 10137 can be seen in Figure 5.4 for the six-storey model and Figure 5.5 for the four-storey model.

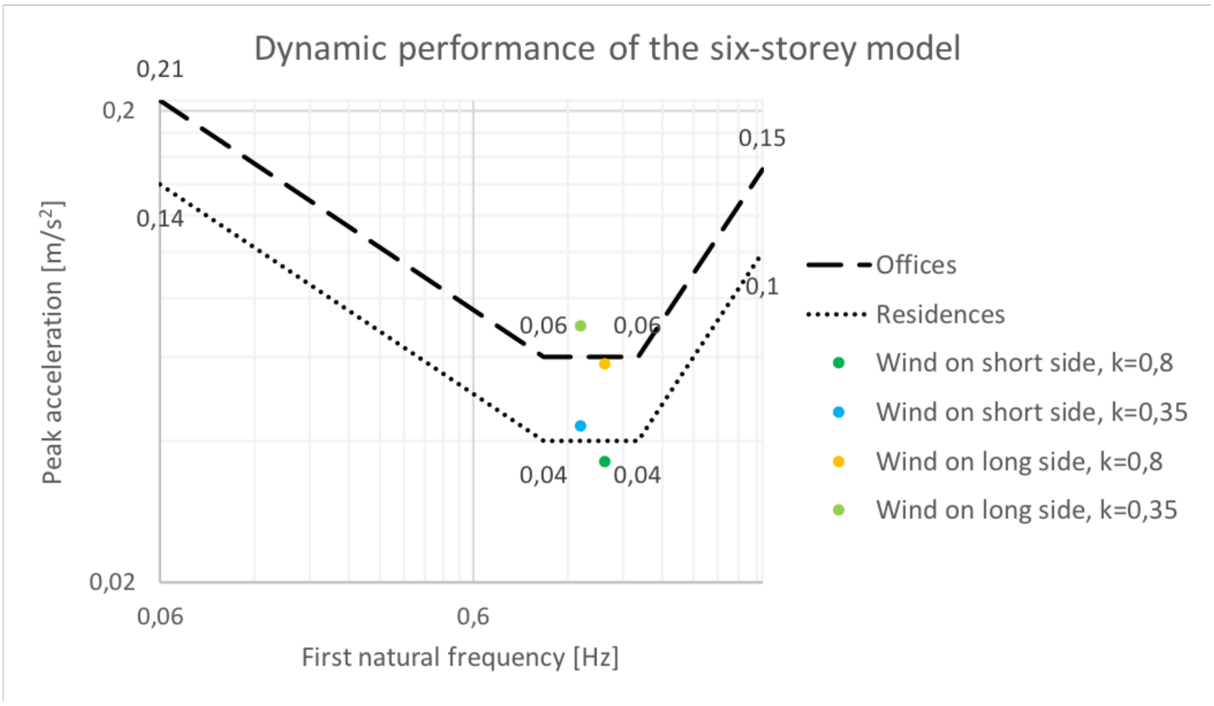

Figure 5.4 The results of the original design idea for the six-storey model compared to the guidelines defined in ISO 10137.

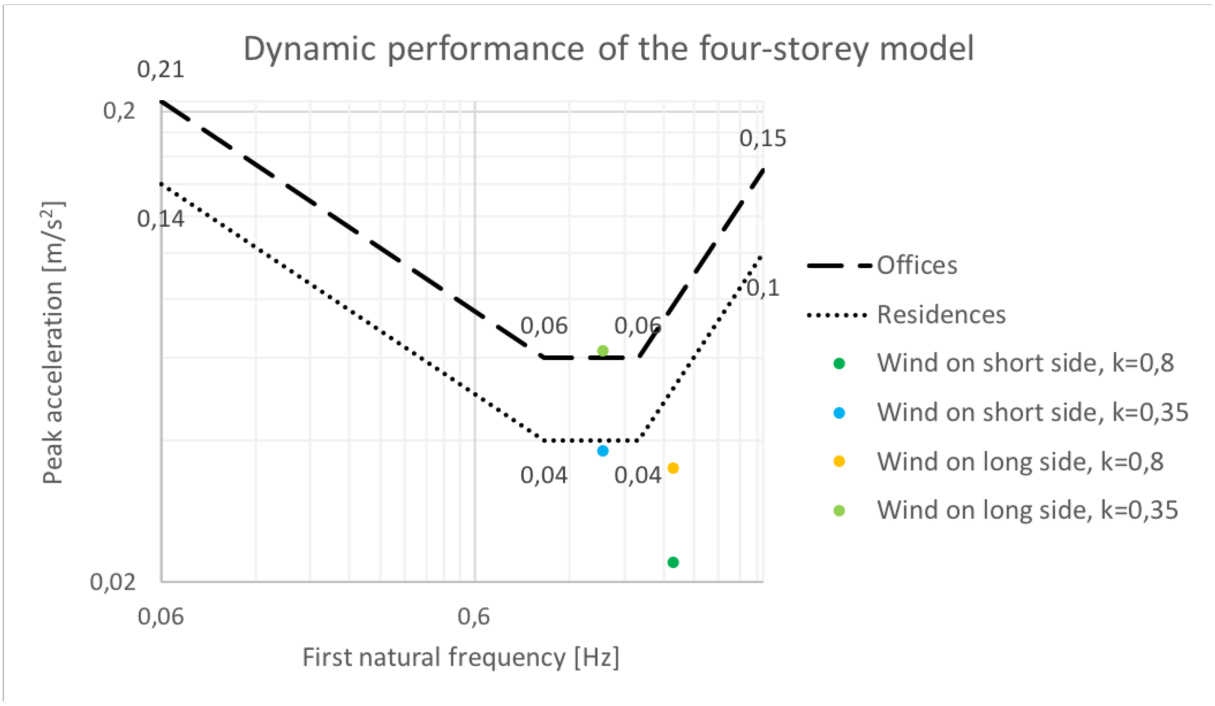

Figure 5.5 The results of the original design idea for the four-storey model compared to the guidelines defined in ISO 10137.

The critical wind direction is wind acting on the long side. Comparison between the different truss layouts can be found in Figure 5.6 and Figure 5.7 regarding stiffnesses of 35-80 % with wind acting in the critical direction for the six- and the four-storey model.

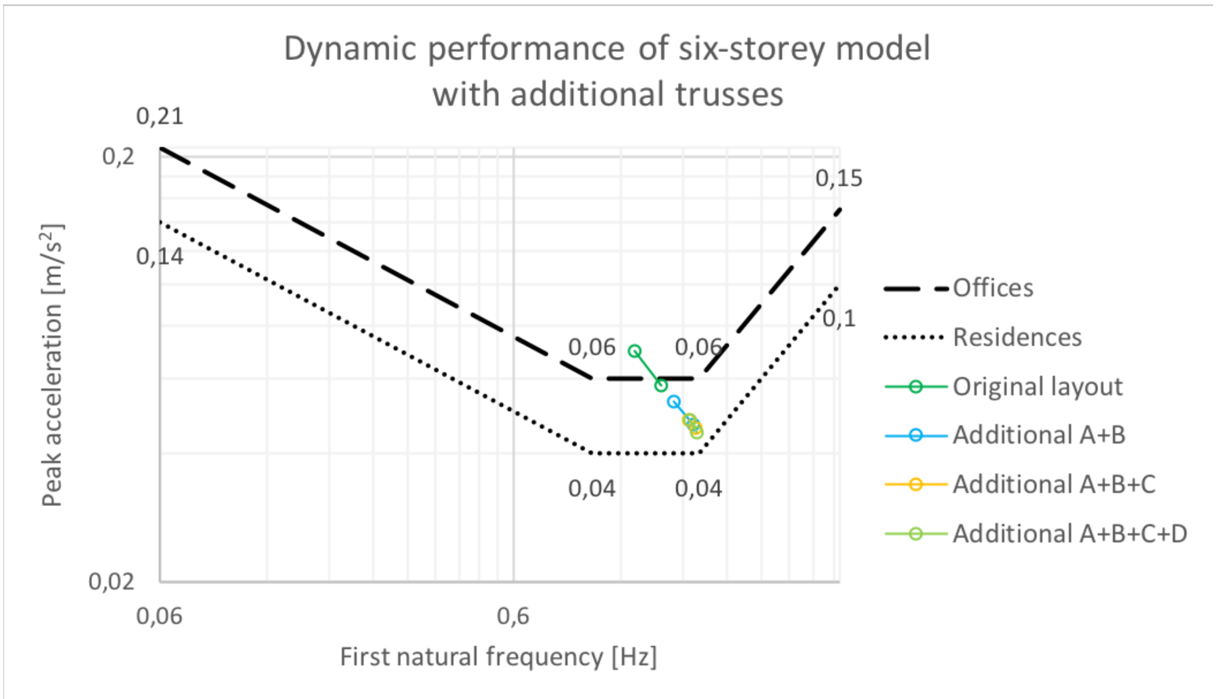

Figure 5.6 The result of the different layouts of trusses for the six-storey model compared to the guidelines. Only response to wind in the critical direction is displayed.

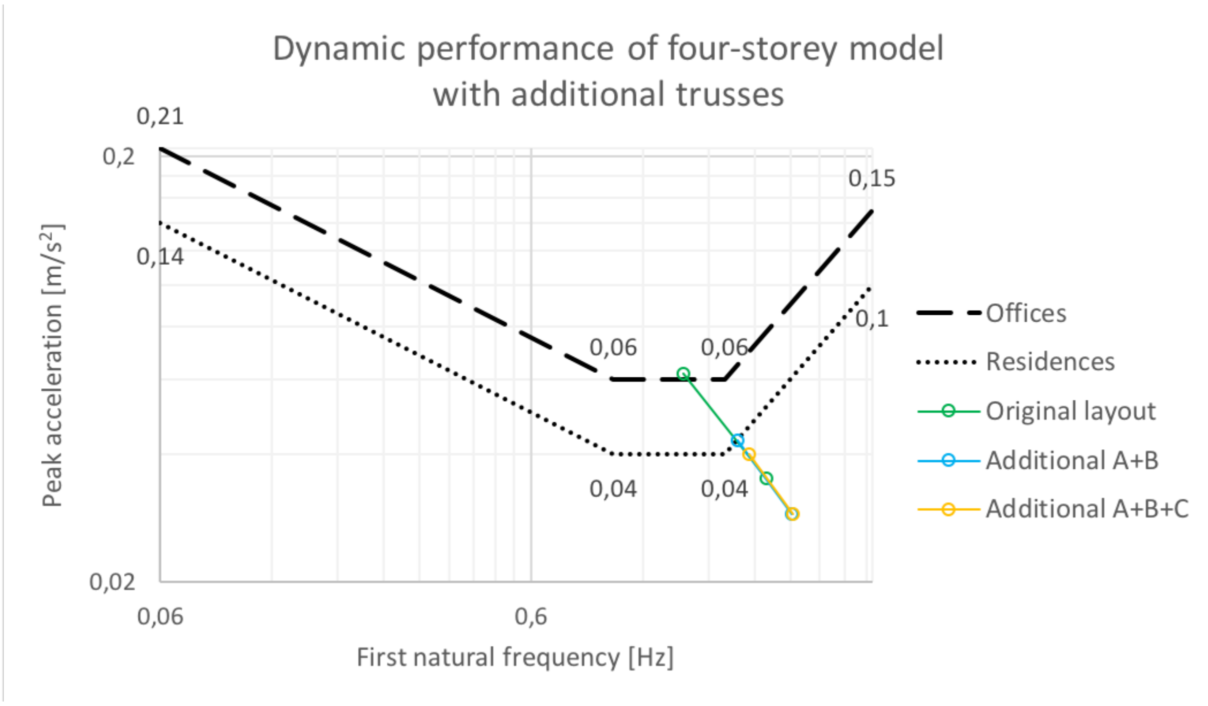

Figure 5.7 The result of the different layouts of trusses for the four-storey model compared to the guidelines. Only response to wind in the critical direction is displayed.

The change in fundamental eigenfrequency due to different truss designs are shown in Figure 5.8.

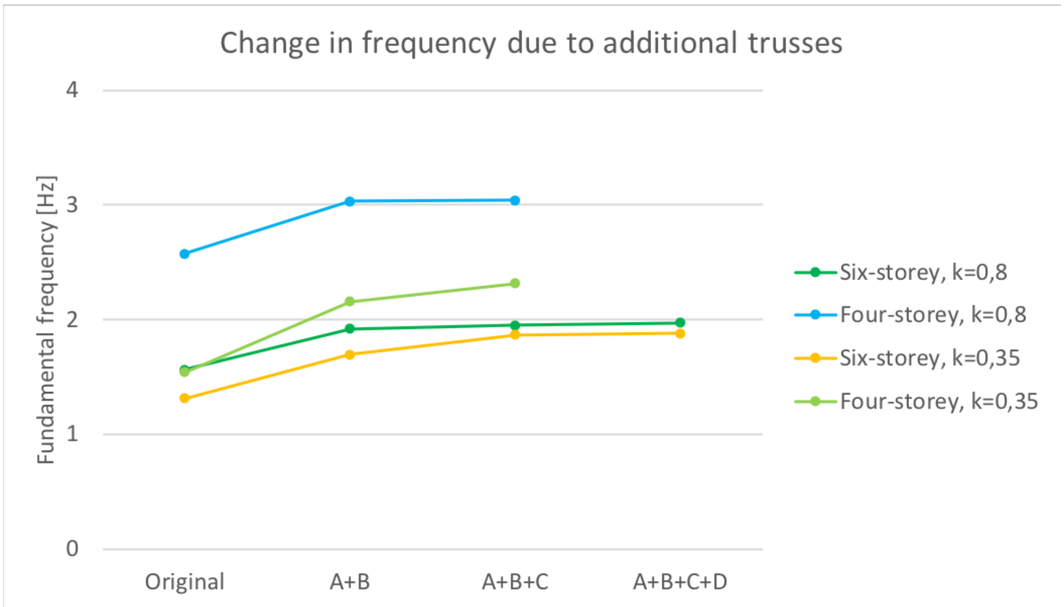

Figure 5.8 The influence on the fundamental eigenfrequency when the additional trusses are added to the model.

In Figure 5.9 the fundamental frequency is plotted towards the number of storeys for stiffnesses of 35 and 80 % of the Kerto Q board for the floor elements.

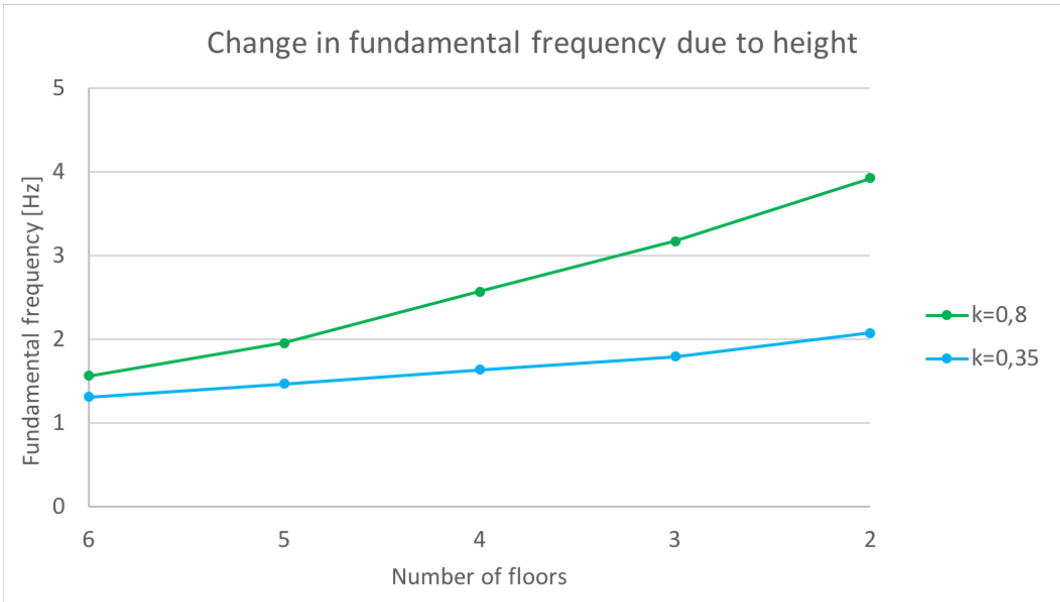

Figure 5.9 Illustration on how the fundamental eigenfrequency changes depending on the number of storeys.

6 Discussion

This chapter handles evaluation on the results and methods used to reach them. The first sections involve analysis of the obtained results and further the used methods and assumptions are discussed.

6.1 Analysis of static results

Considering horizontal displacements both building models fulfil the demands regarding all of the modelled cases, see Figure 5.2 and Figure 5.3. Each truss was designed in order to withstand the wind forces it was exposed to, hence each truss will keep within the limits of L/500 also when modelled in the complete structure. If the static displacement of the complete structure were to exceed the limits it should be due to insufficient stiffness of the floor elements, giving displacements similar to the ones illustrated in Figure 2.3C. Since the displacements were within limits for all modelled cases it seems that the floor elements provide sufficient rigidity.

The case with wind acting in the y-direction, on the long side, gives larger displacements than the cases with wind acting in the other direction. In the y-direction there are three trusses withstanding the forces, see Figure 3.3, truss 1, 4 and 7. However, due to the shape of the building only truss 1 and 4 will cover the top floor. This gives that the distance between the trusses are larger in the top floor putting pressure on the elements of the roof to transfer the forces further. In this case the displacements are visibly larger at the roof level than the floor below, especially regarding wind in the positive y-direction, see Appendix B.3.

When comparing the influence of the stiffness of floor elements, on the horizontal displacements it is seen that the difference is greater for the four-storey model than the six-storey. This is easily seen by comparing Figure 5.2 and Figure 5.3. This indicates that the importance of the stiffness decreases as the building height increases. Possibly since with a higher building comes additional instability due to more connections and taller columns.

6.2 Analysis of dynamic results

The results from the dynamic analysis are less adequate compared to the static analysis. This means that even if the static response is satisfactory a dynamic analysis should be carried out before constructing high-rise buildings like the building considered in this thesis.

In Figure 5.4 it can be read that the original design idea regarding placements and number of trusses do not perform well when modelled in the six-storey model. The only case that fulfils the demands stated in ISO 10137 is the most favourable case with wind on the short side and 80 % stiffness. In Figure 5.5 the corresponding results for the four-storey model can be read. In this case the demands are fulfilled for all cases except one, the most critical with wind acting on the long side and stiffness modelled at 35 %. Consequently, the dynamic performance differs quite a lot for the six- and the four-storey model.

In order to improve the dynamic behaviour additional trusses were added to the structure and the contribution can be analysed from Figure 5.6 and Figure 5.7. For the six-storey model the demands for residential buildings are not fulfilled even when in total eight additional trusses are added to the structure. This indicates that the influence of the trusses is saturated, which is further illustrated in Figure 5.8 where the inclination of the curves markedly decreases. This implies that the given structure will not be able to fulfil the demands regardless of how many trusses are added, and this is the case regarding both modelled stiffnesses for the six-storey model. On the other hand, considering the four-storey model the additional trusses actually do help the dynamic performance to such length that the demands are fulfilled for all modelled cases for the four-storey model.

It can be read from Figure 5.8 that the stiffness of the floor has higher influence on the eigenfrequency for the four-storey model compared to the six-storey. This is also confirmed in Figure 5.9, where the difference in fundamental frequency increases as the number of floor decreases. This indicates that the stiffness of the floor elements is of less importance when constructing high buildings. Similar findings were also obtained in the static analysis, as discussed in previous section.

Consequently, both the number of trusses and the stiffnesses of the floor elements become irrelevant at certain building heights. This implies that in order to fulfil the dynamic demands other measures needs to be taken. The eigenfrequency depends on the mass, stiffness and damping following from Chapter 2.2. The peak accelerations depend on many factors such as fundamental eigenfrequency, wind speed, building's geometry, see further Appendix C.5. The wind speed will increase with height and is something all high-rise buildings will be exposed to, however by altering the geometry the peak acceleration can be changed. A wide building will catch more wind than a narrow building structure. However, changes in geometry will also affect the eigenfrequency, a narrow building will be less rigid since the lever arm is decreased and also the weight of the building is decreased.

Considering the eigenmodes, see Appendix B.4, it's clear that the stiffness of the floor elements has an impact on the resulting fundamental eigenfrequency. From the figures in Appendix B.4 the floor modelled with 35 % stiffness appear flexible compared to the post beam structure and the floor with 80 % stiffness behaves rigidly in comparison. In this case it also appears that the eigenmodes are similar for the six- and the four-storey model regarding the different truss layouts modelled. This suggests that the eigenmode will look the same regardless of building height and is only dependent on the building's geometry and layout of stabilization. The fundamental eigenfrequency is however lowered with increasing height.

6.3 Method and assumptions

The disadvantages regarding the method lies in the uncertainties of the modelled structure. First and foremost, the stiffness of the floor elements is unknown, however by modelling the floor elements at the extremes of their assumed stiffness conclusions can be drawn within which span the actual results lie and also how the stiffness of the elements affects the total stability of the structure. Moreover, the stiffness of the connections in between glulam members is unknown. When modelling, all connections were assumed moment free in order to model the worst case as with semi-rigid couplings the structure would be stiffer. Nevertheless, even though the structure would be stiffer with semi-rigid couplings, the rigidity might cause other problems. As the members are not free to move, stresses will occur within the members which might cause problems regarding fatigue among others.

The dynamic response of the structure is compared to guidelines set in ISO 10137. However, these guidelines are not compelled to be met as there are no definite rules regarding vibrations and motions of building structures. As the perception of vibrations and motions varies with individuals it is possible that some people might feel uncomfortable in a building even though these guidelines have been met or maybe none of the residents would have noticed if these guidelines weren't fulfilled. Meaning that a building which doesn't fulfil the guidelines might still be serviceable in practice.

The modelling was carried out for wind acting on the sides of the building. In reality the wind might blow at an angle on the building structure. However, wind acting on the sides of the building shows the worst case since if wind is blowing at an angle trusses in both directions will help take the wind forces down to the foundation, and with wind blowing straight at the building only the trusses oriented in that direction will be able to resist the forces.

Further, as the analysis is only carried out for one particular building, no definite conclusions can be drawn regarding four- and six-storey buildings in general. In order to fully understand the importance of trusses and stiffness of floor elements analysis must be carried out for multiple geometries. Also, as the building was originally designed as a six-storey building and the dimensions of the four-storey model will be over-sized regarding columns as they carry less load and the wind forces decrease with decreasing height.

7 Conclusions

The static response of the building models is satisfactory regarding both modelled building heights and floor stiffnesses.

Considering the dynamic response of the six-storey model, the guidelines set in ISO 10137 cannot be fulfilled, independently on the number of trusses and floor stiffness within the limits of 35-80 % of the stiffness of the Kerto Q board. In order to fulfil the guidelines for this specific building model measures needs to be taken in order to increase the eigenfrequency by increasing the stiffness or by adding damping mechanisms to decrease the amplitudes.

The four-storey model shows better results regarding dynamics and fulfils the guidelines for a stiffness of the floor elements modelled to 80 % however not for the 35 %. In order to ensure the stability of this building model the stiffness of the floor elements needs to be established. If the established stiffness is not sufficient additional trusses can ensure that the guidelines are fulfilled.

By comparing the behaviour of the four- and the six-storey model it is seen that the influence regarding number of trusses and stiffness of floor elements is larger for buildings of less height. Consequently, the remaining instability found in higher structures originates from other factors.

7.1 Suggestions on further research

For further analysis of the trusses the stiffness of the floor elements needs to be determined. Furthermore, analysis regarding stiffness of the connections would give a more exact result.

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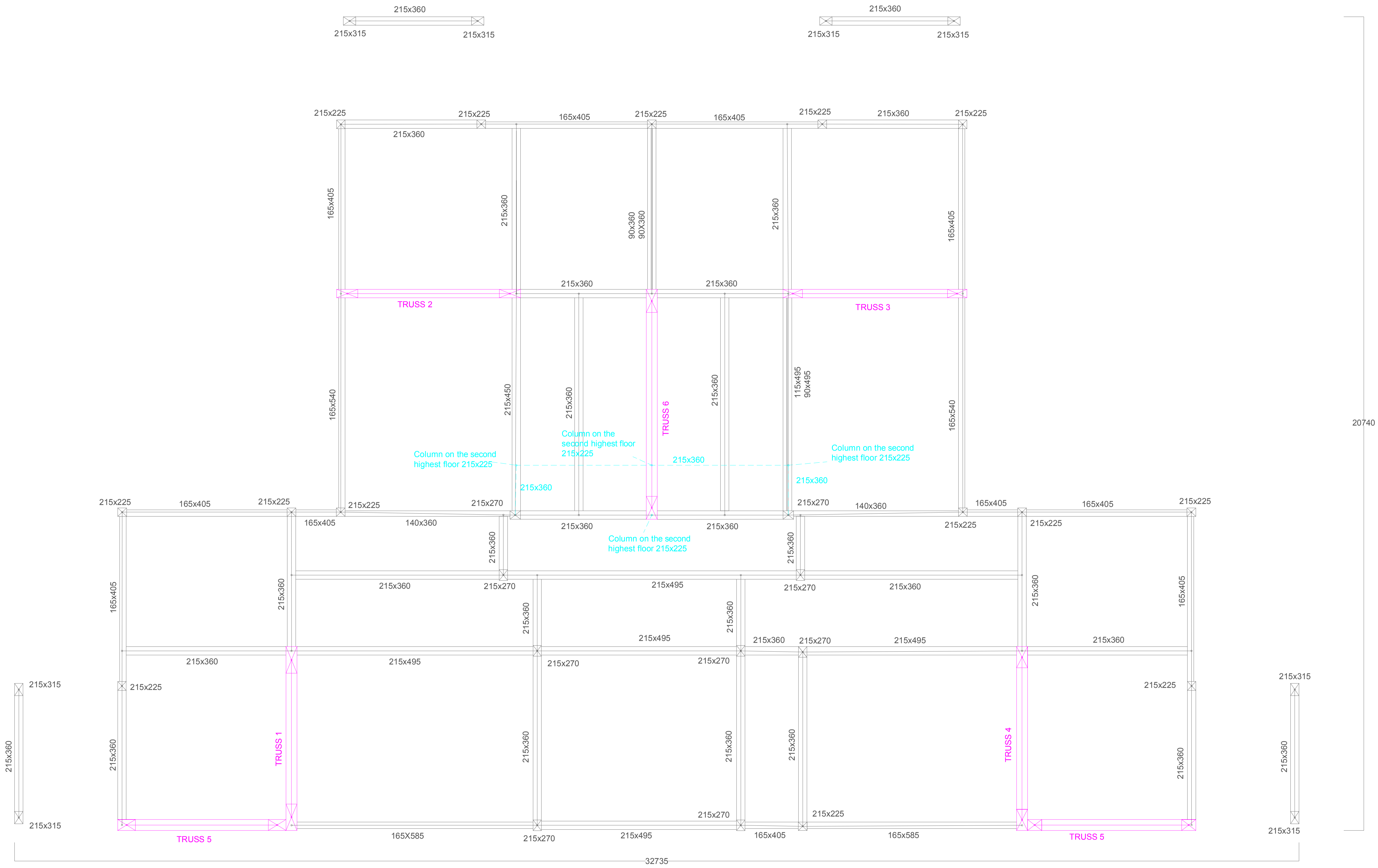
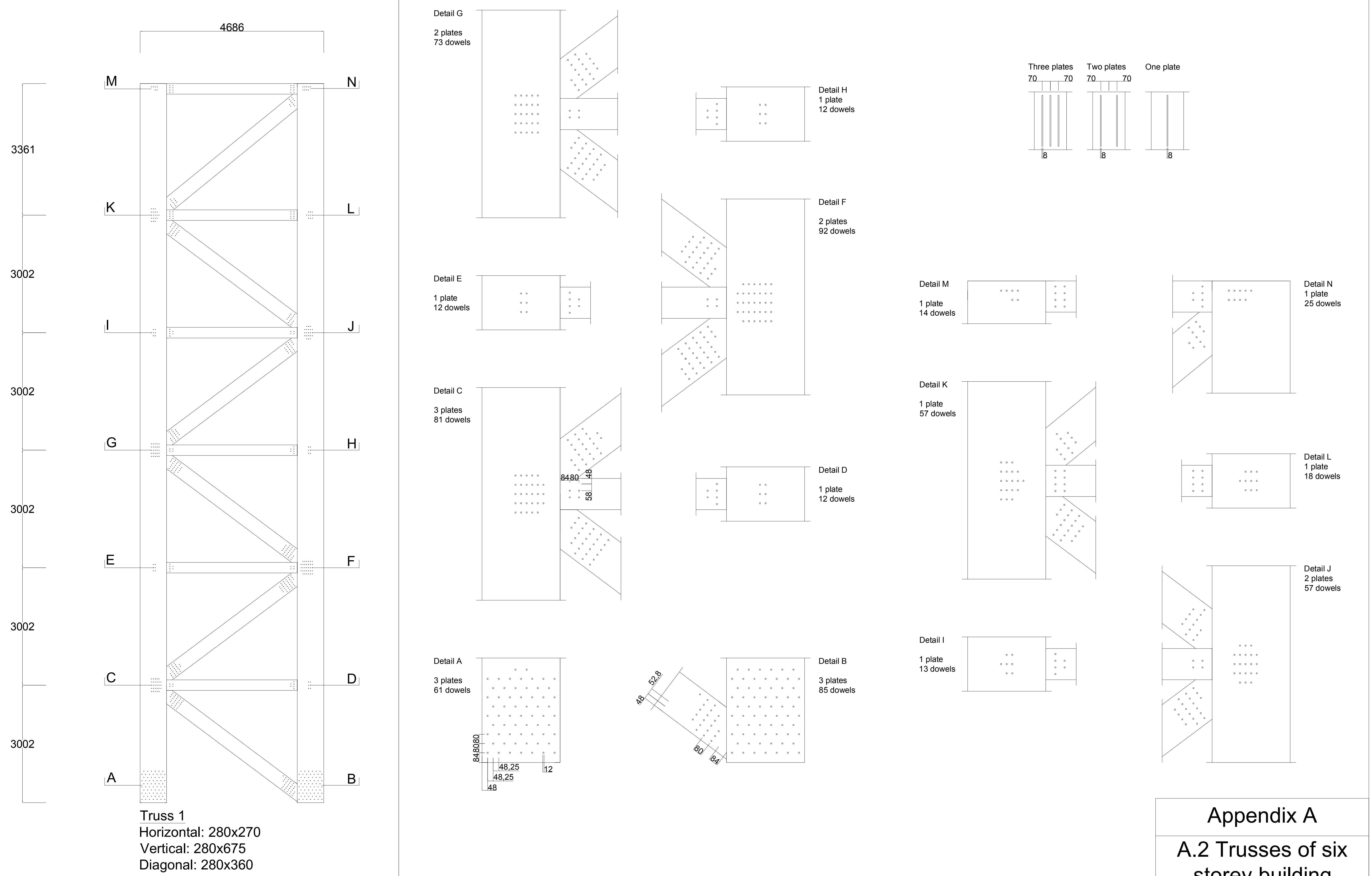
Appendix A - Drawings



Appendix A

A.1 Plan

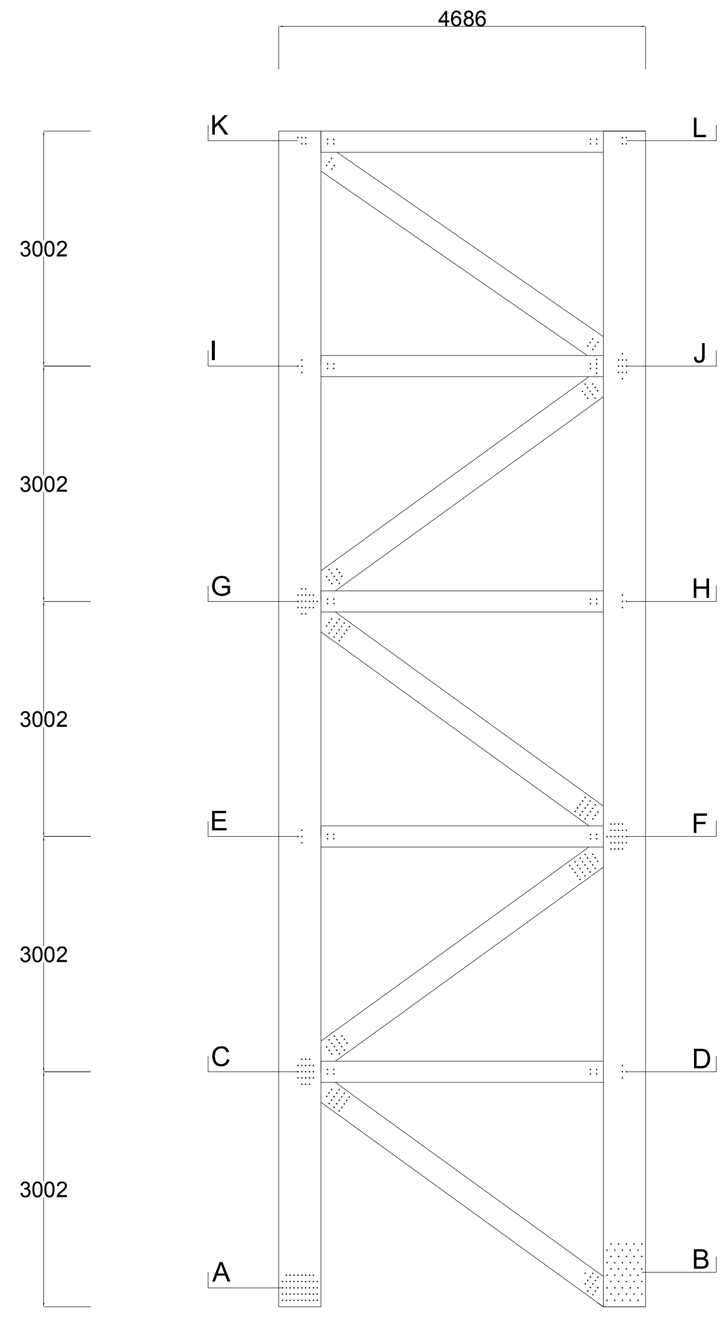
A1



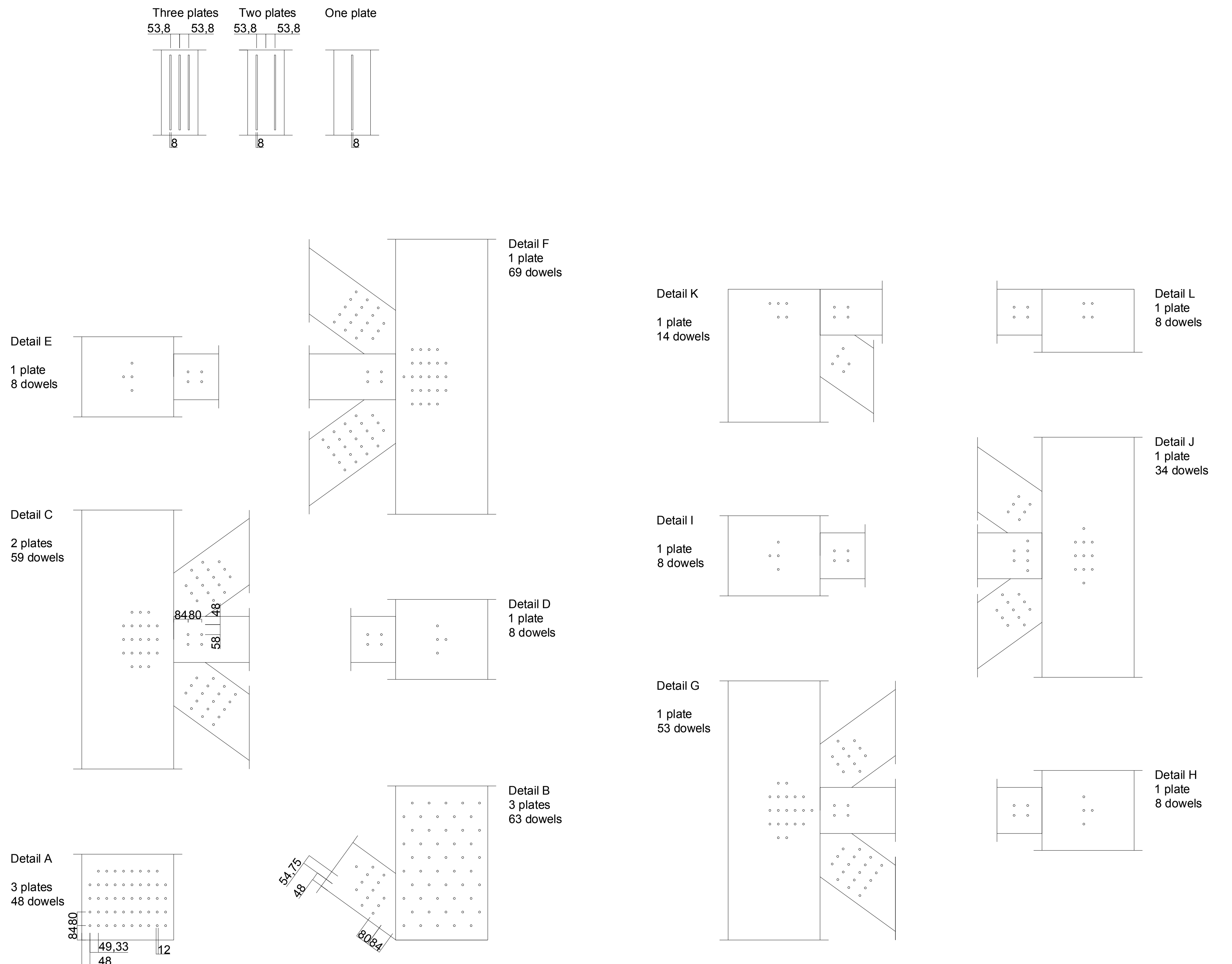
Appendix A

A.2 Trusses of six storey building

A2



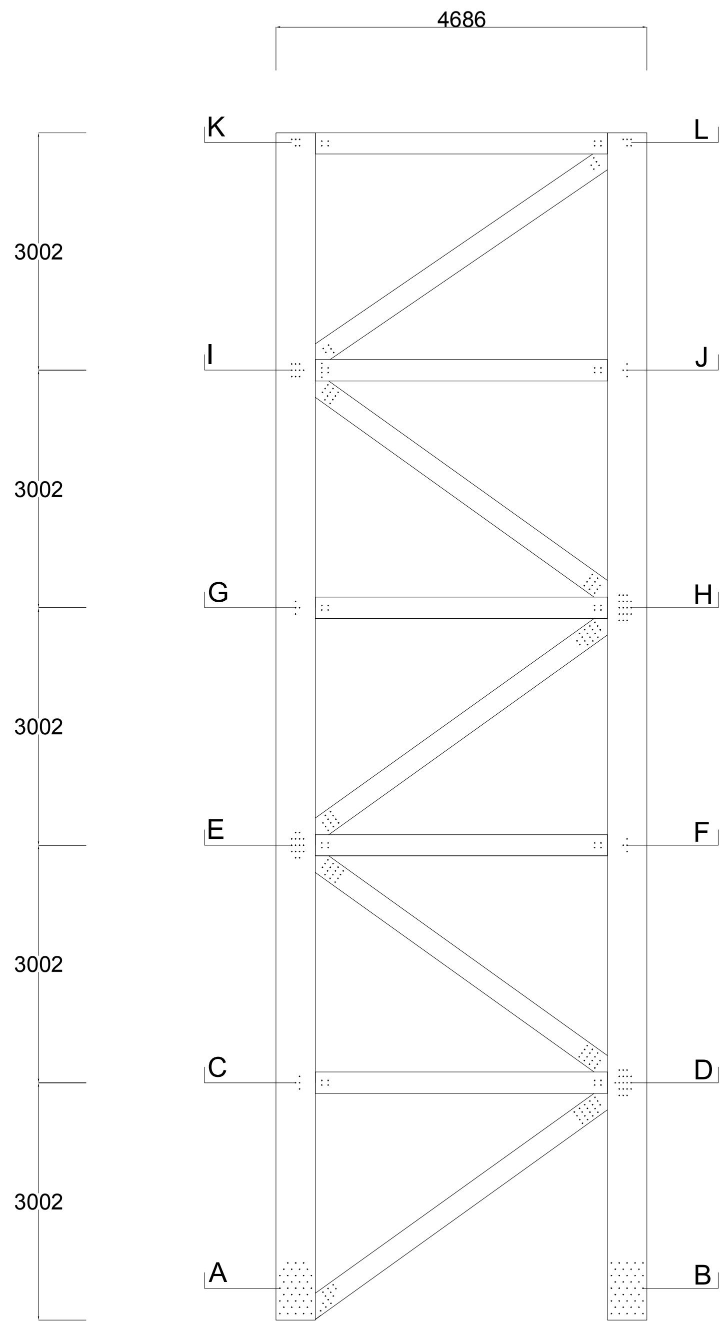
Truss 2
Horizontal: 215x270
Vertical: 215x540
Diagonal: 215x315



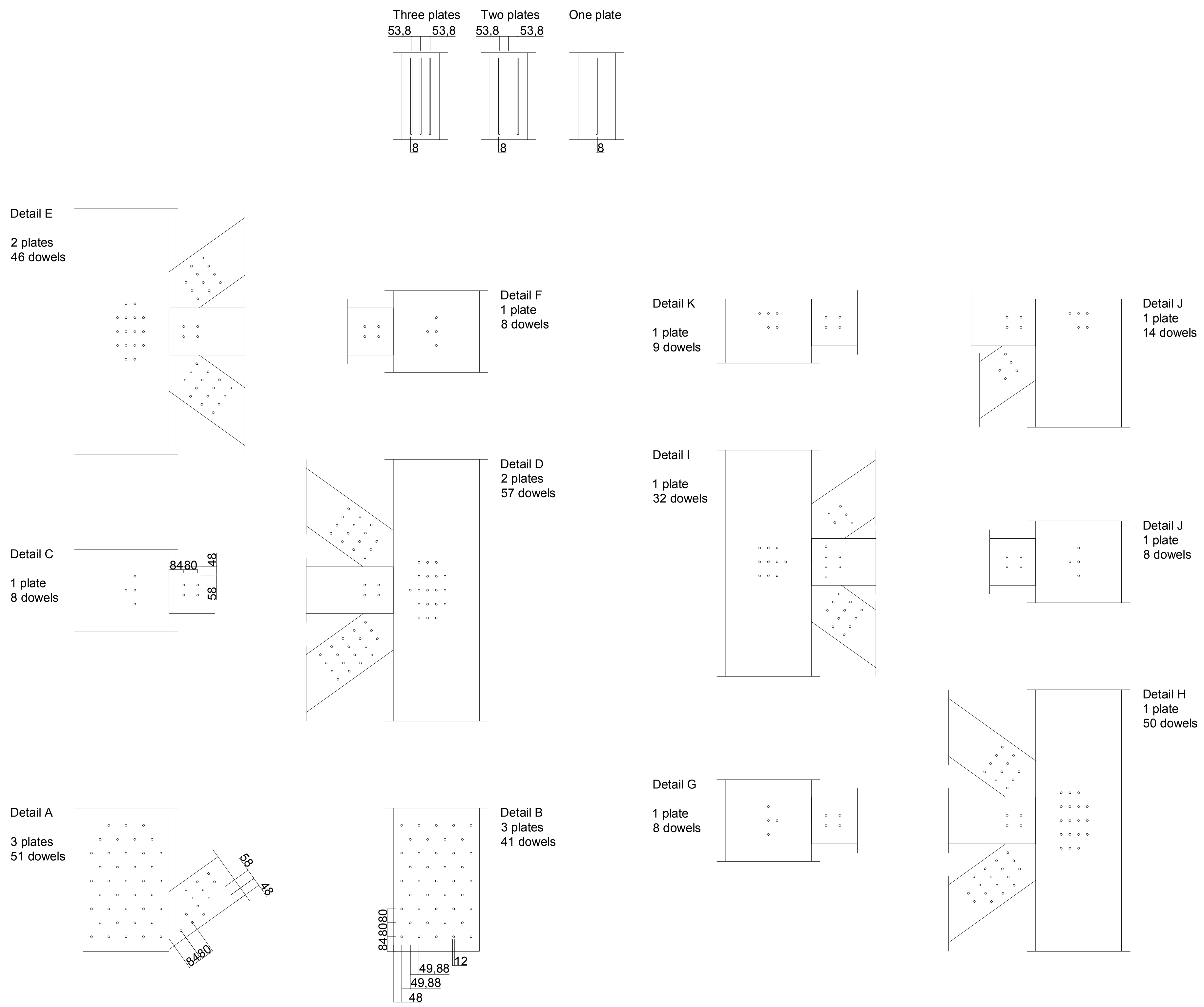
Appendix A

A.2 Trusses of six storey building

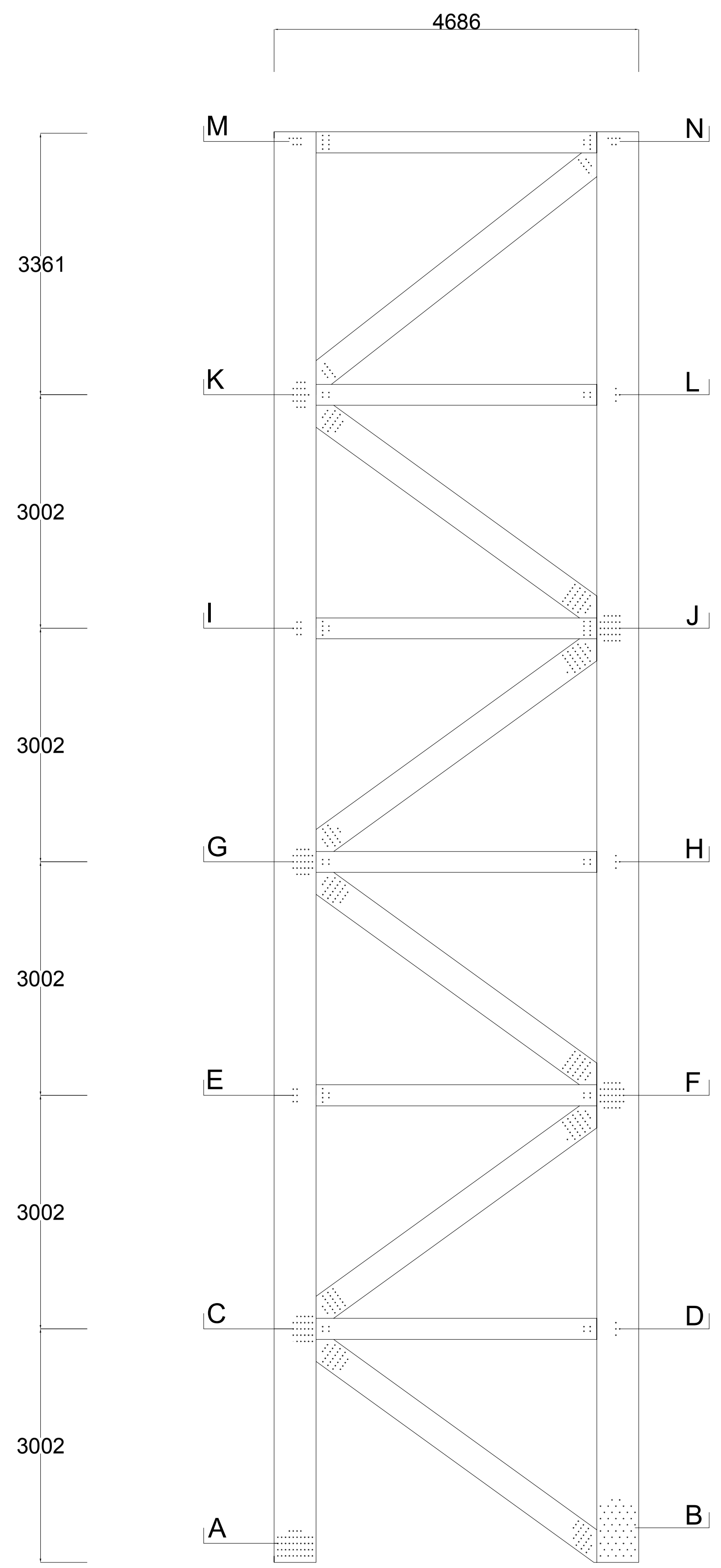
A3



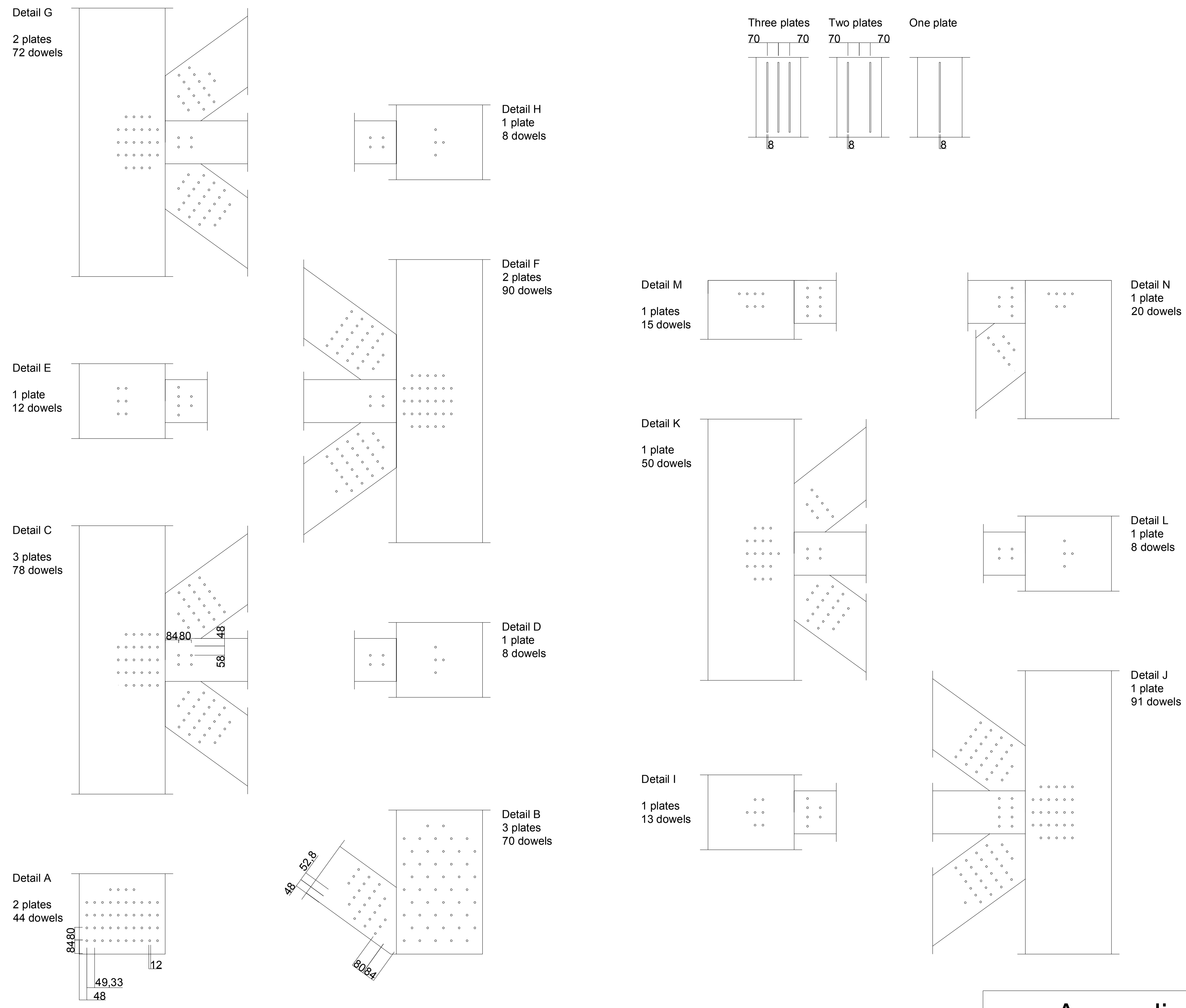
Truss 3
Horizontal: 215x270
Vertical: 215x495
Diagonal: 215x270



Appendix A
A.2 Trusses of six storey building
A4



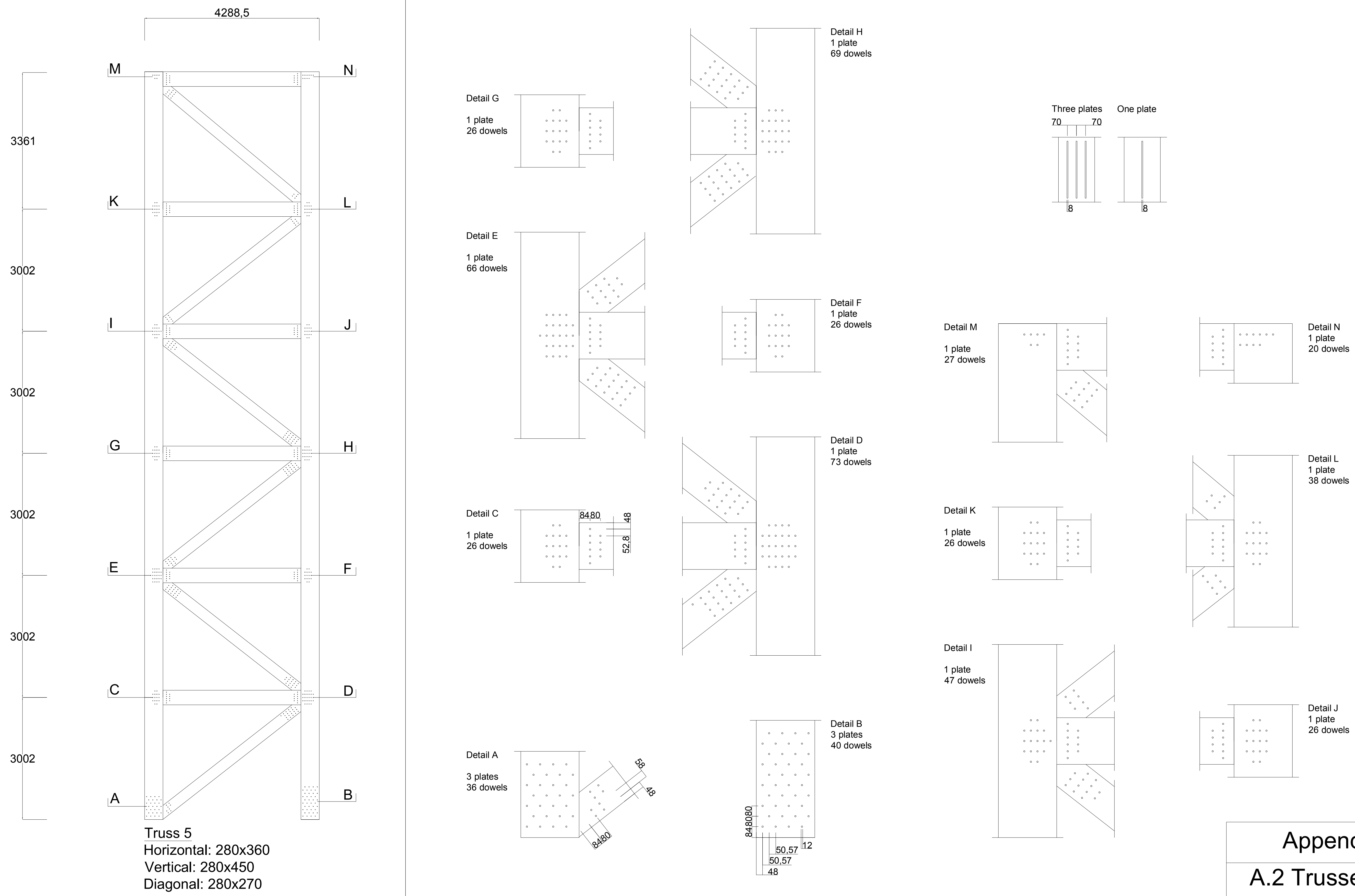
Truss 4
Horizontal: 280x270
Vertical: 280x540
Diagonal: 280x360



Appendix A

A.2 Trusses of six storey building

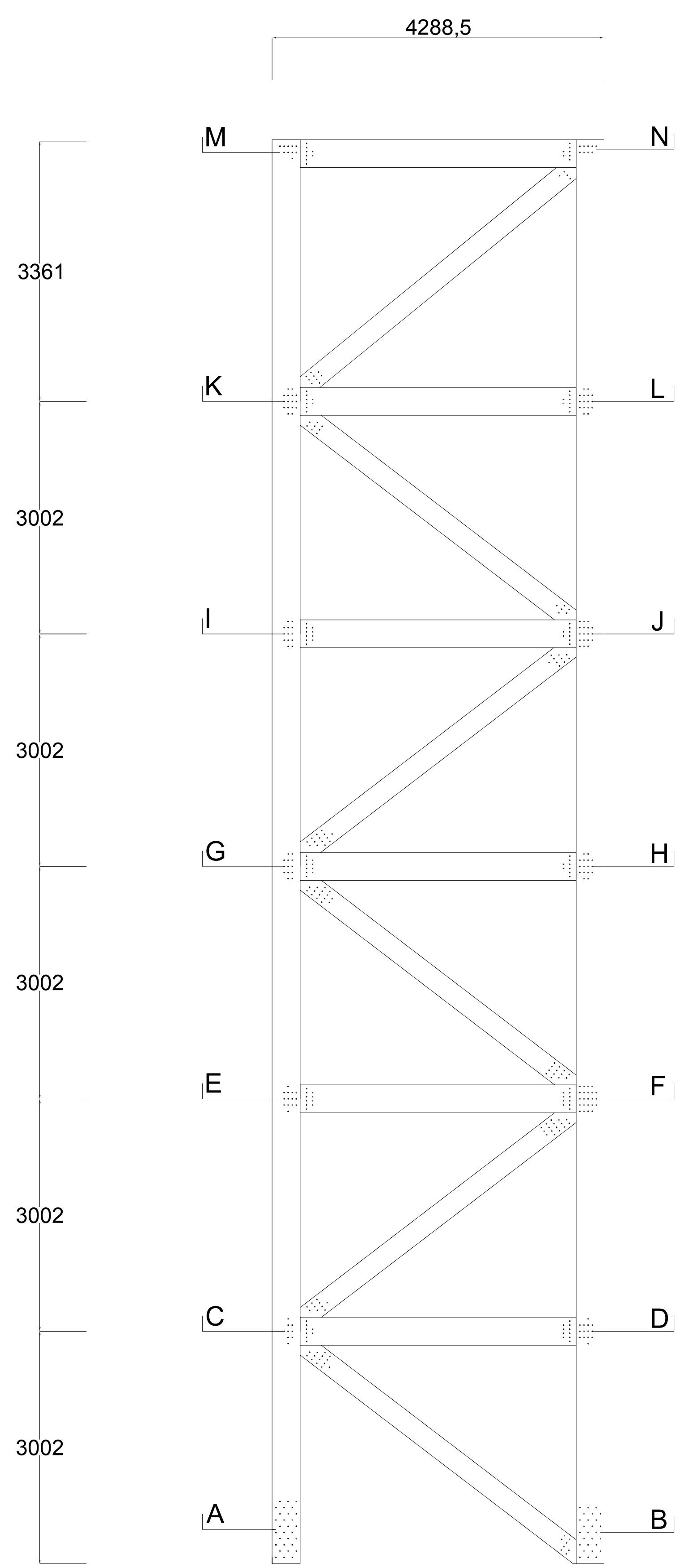
A5



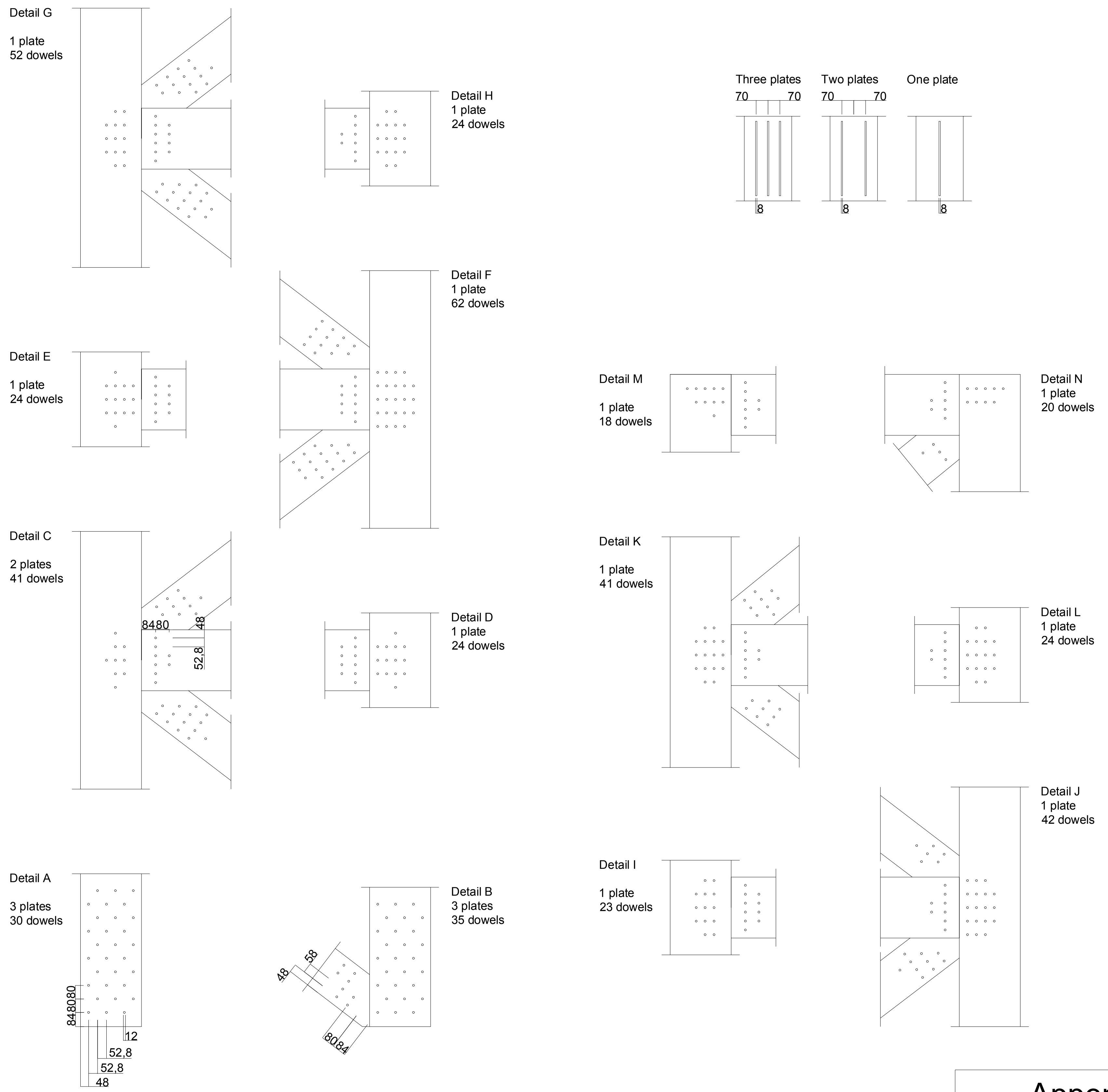
Appendix A

A.2 Trusses of six storey building

A6



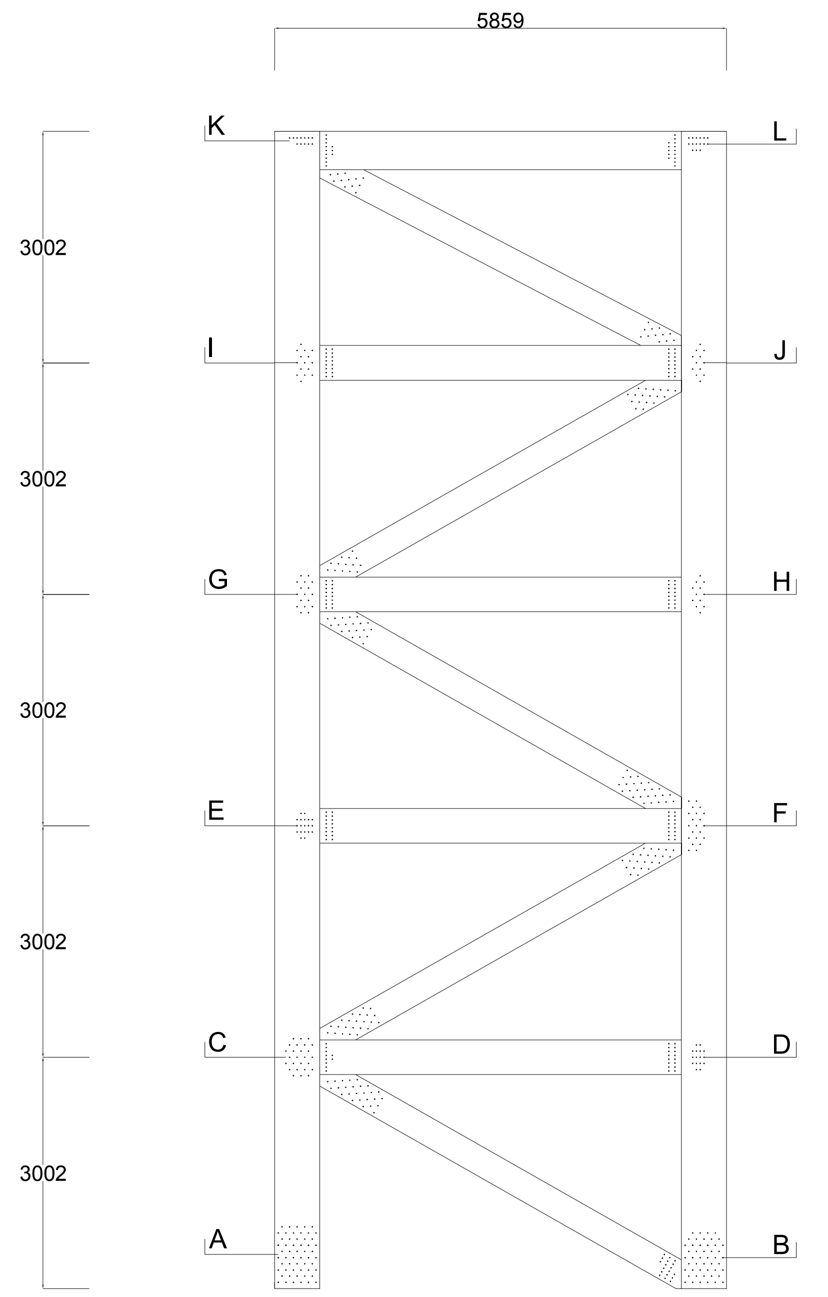
Truss 6
Horizontal: 280x360
Vertical: 280x360
Diagonal: 280x270



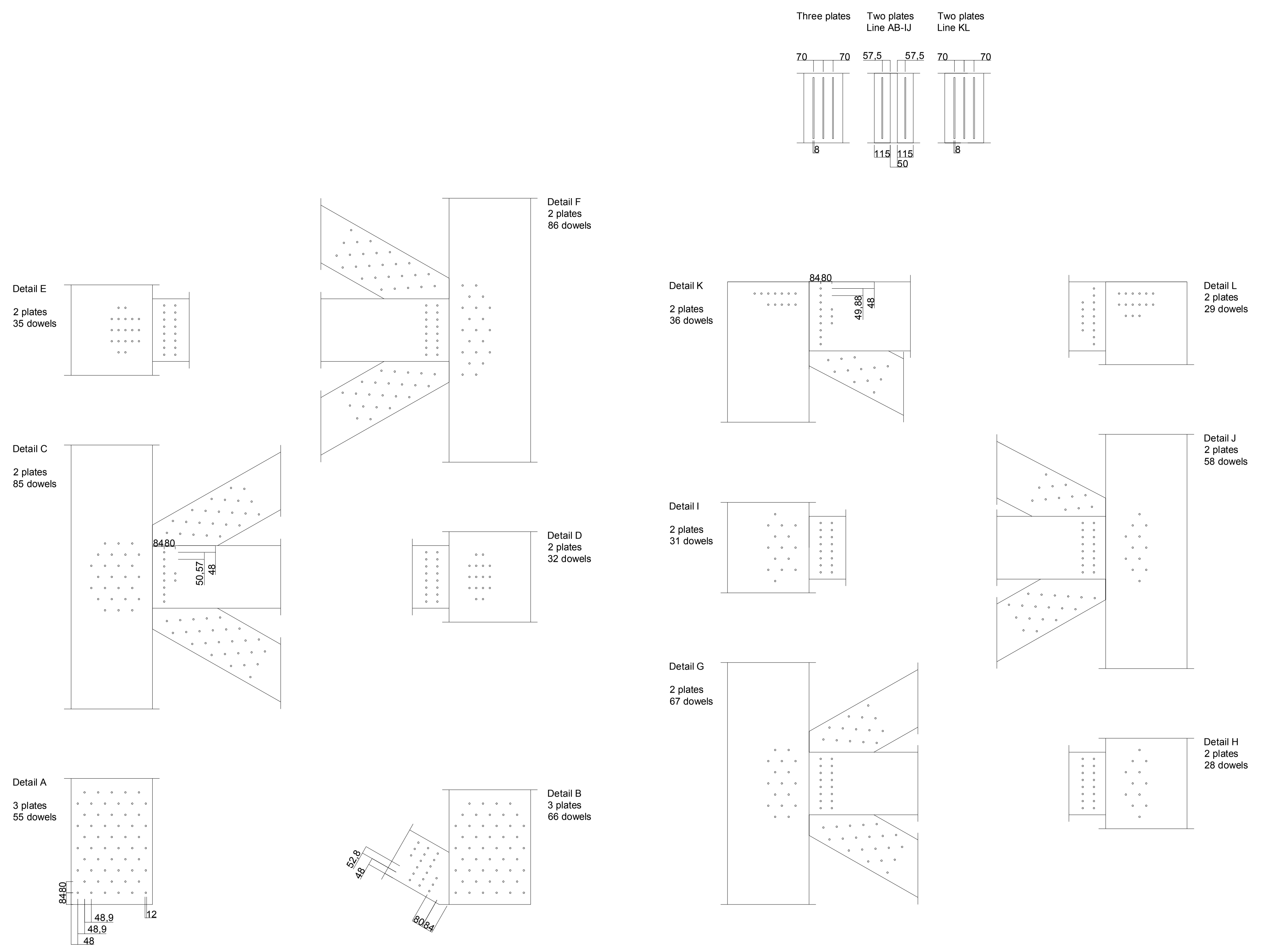
Appendix A

A.2 Trusses of six storey building

A7



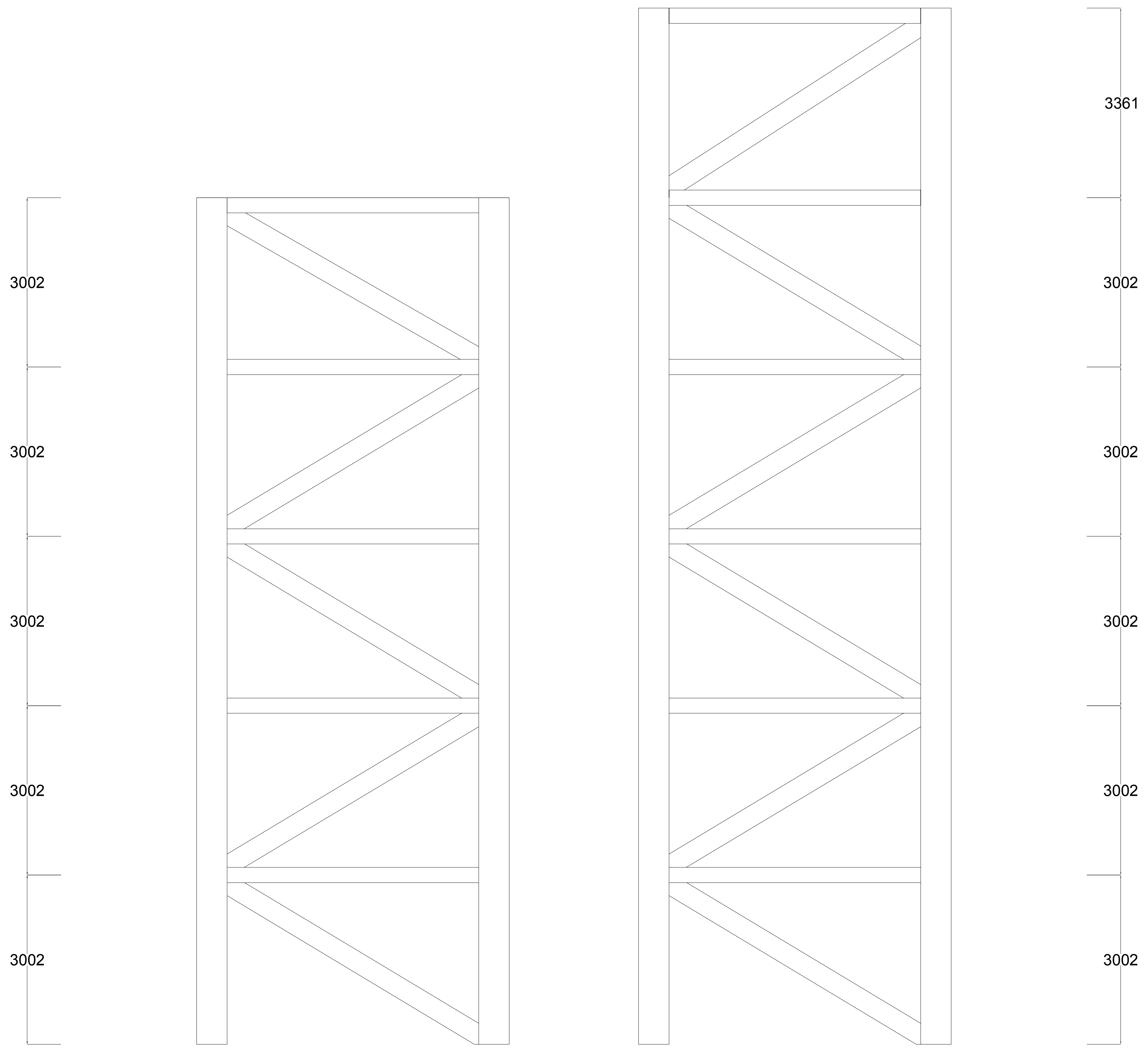
Truss 7
Horizontal: 115x450 x2
Horizontal at roof level: 280x495
Vertical: 280x585
Diagonal: 280x360



Appendix A

A.2 Trusses of six storey building

A8

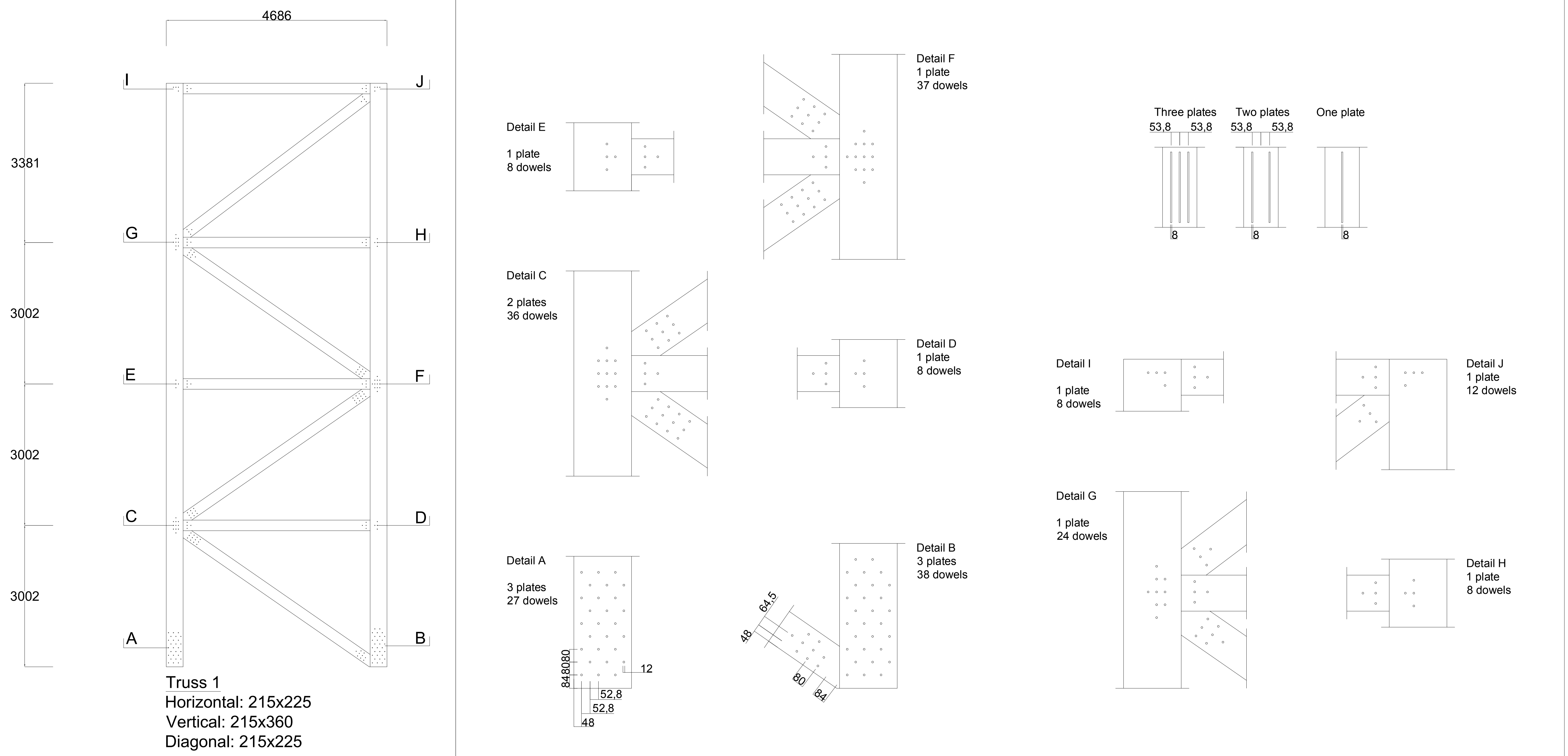


Additional trusses
Horizontal: 280x270
Vertical: 280x540
Diagonal: 280x360

Appendix A

A.2 Trusses of six storey building

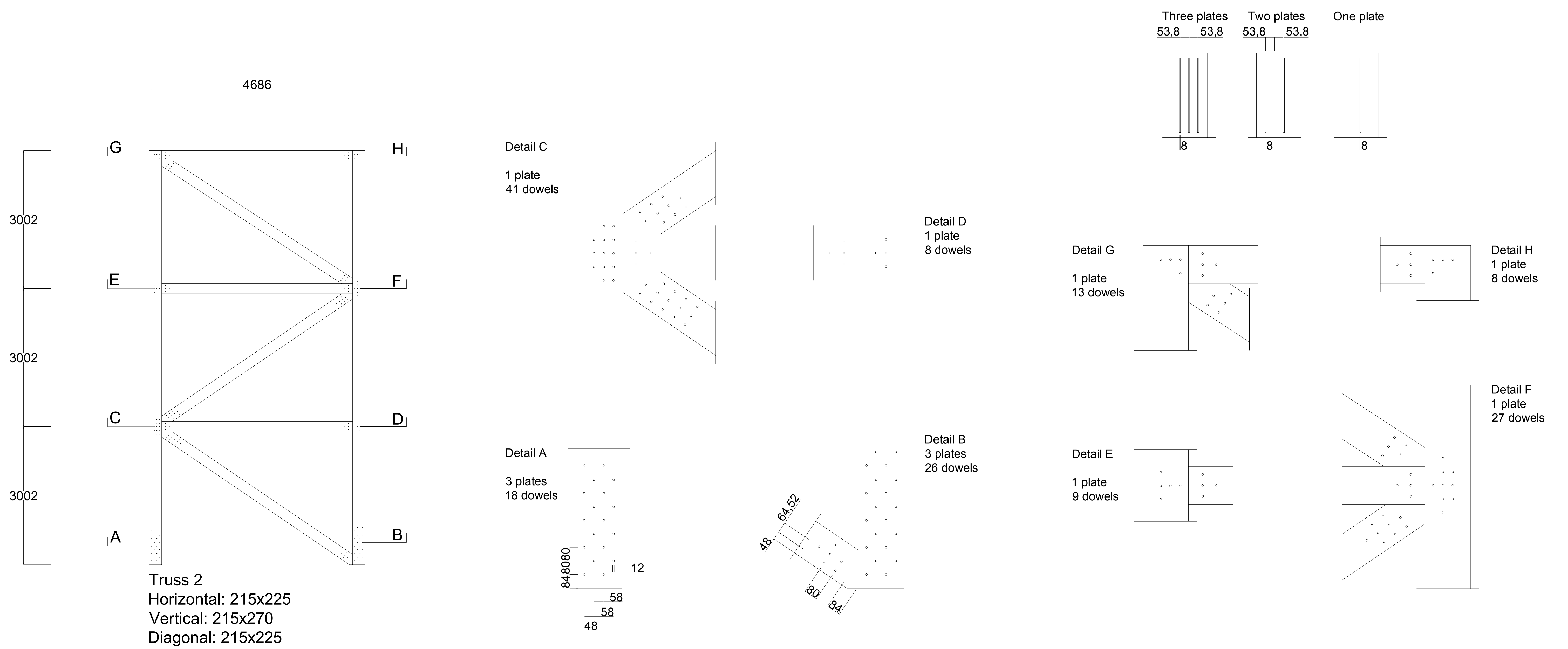
A9



Appendix A

A.3 Trusses of four storey building

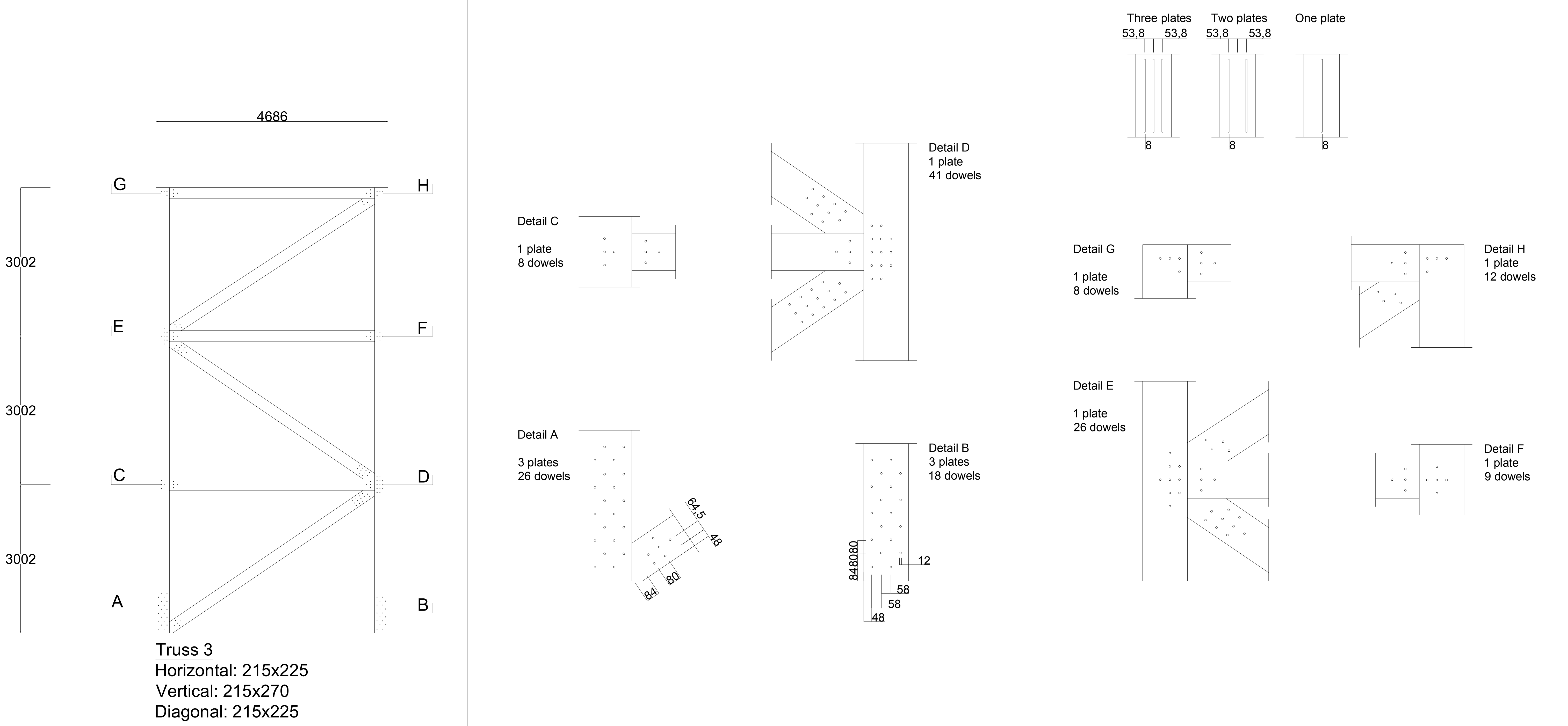
A10



Appendix A

A.3 Trusses of four storey building

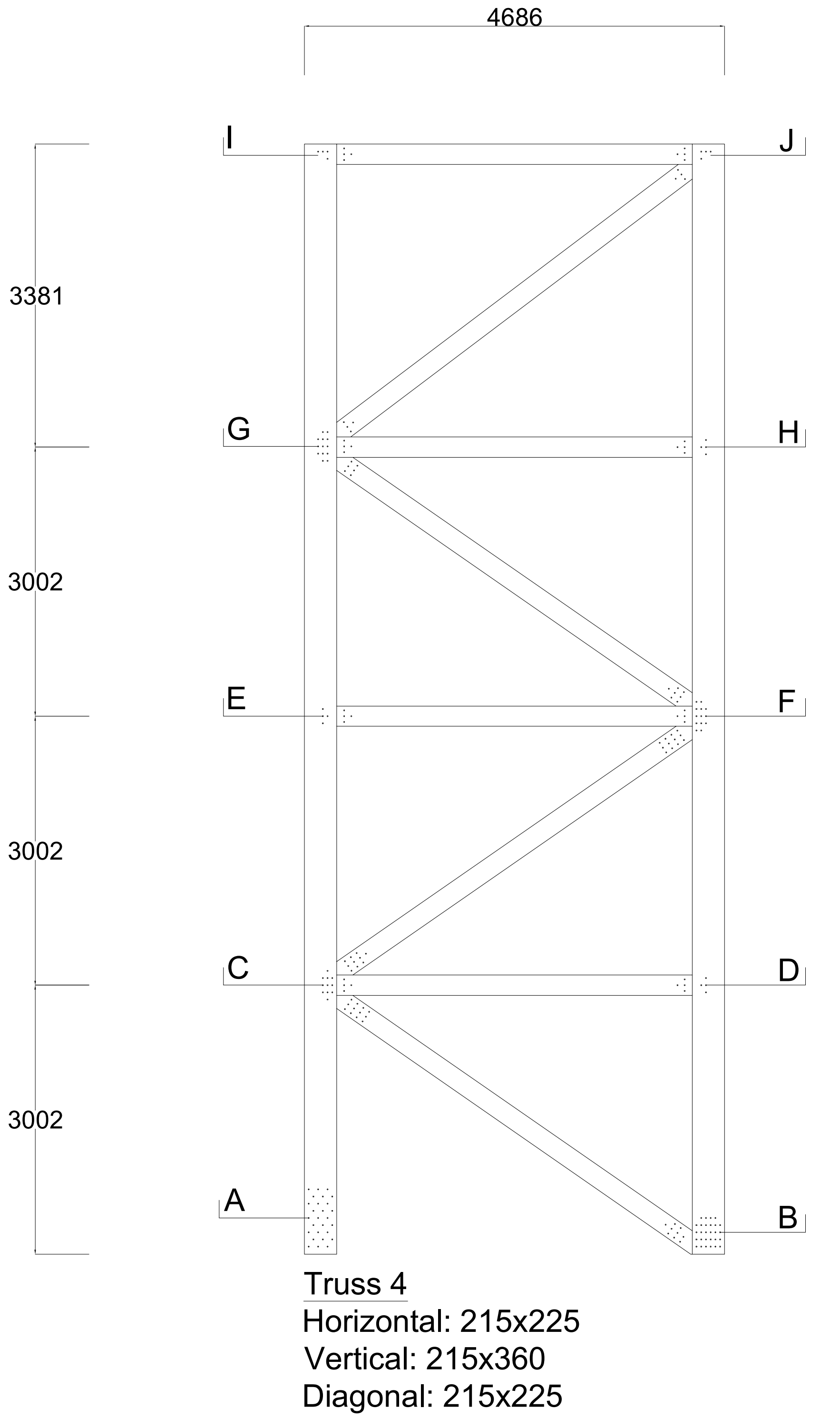
A11

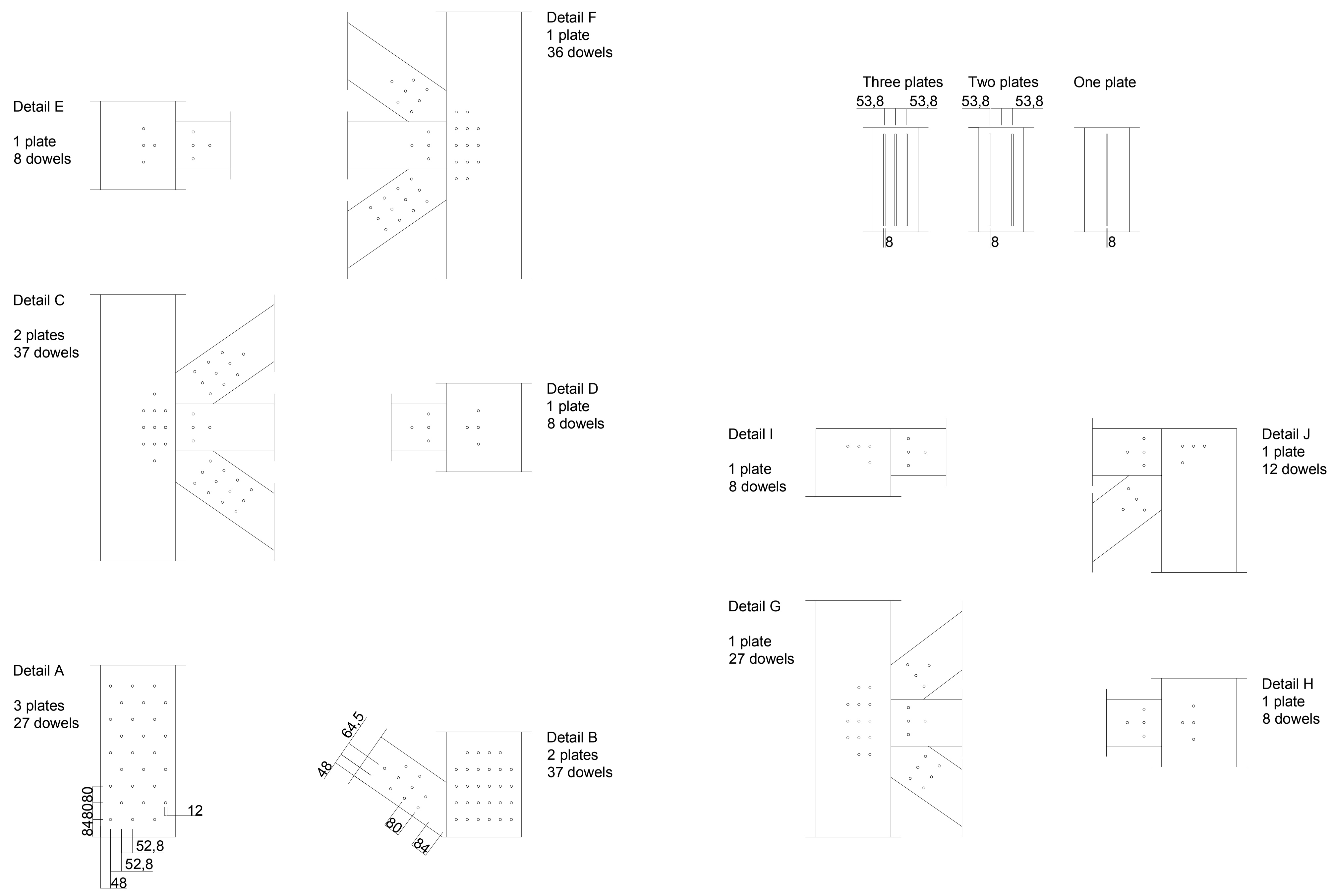


Appendix A

A.3 Trusses of four storey building

A12

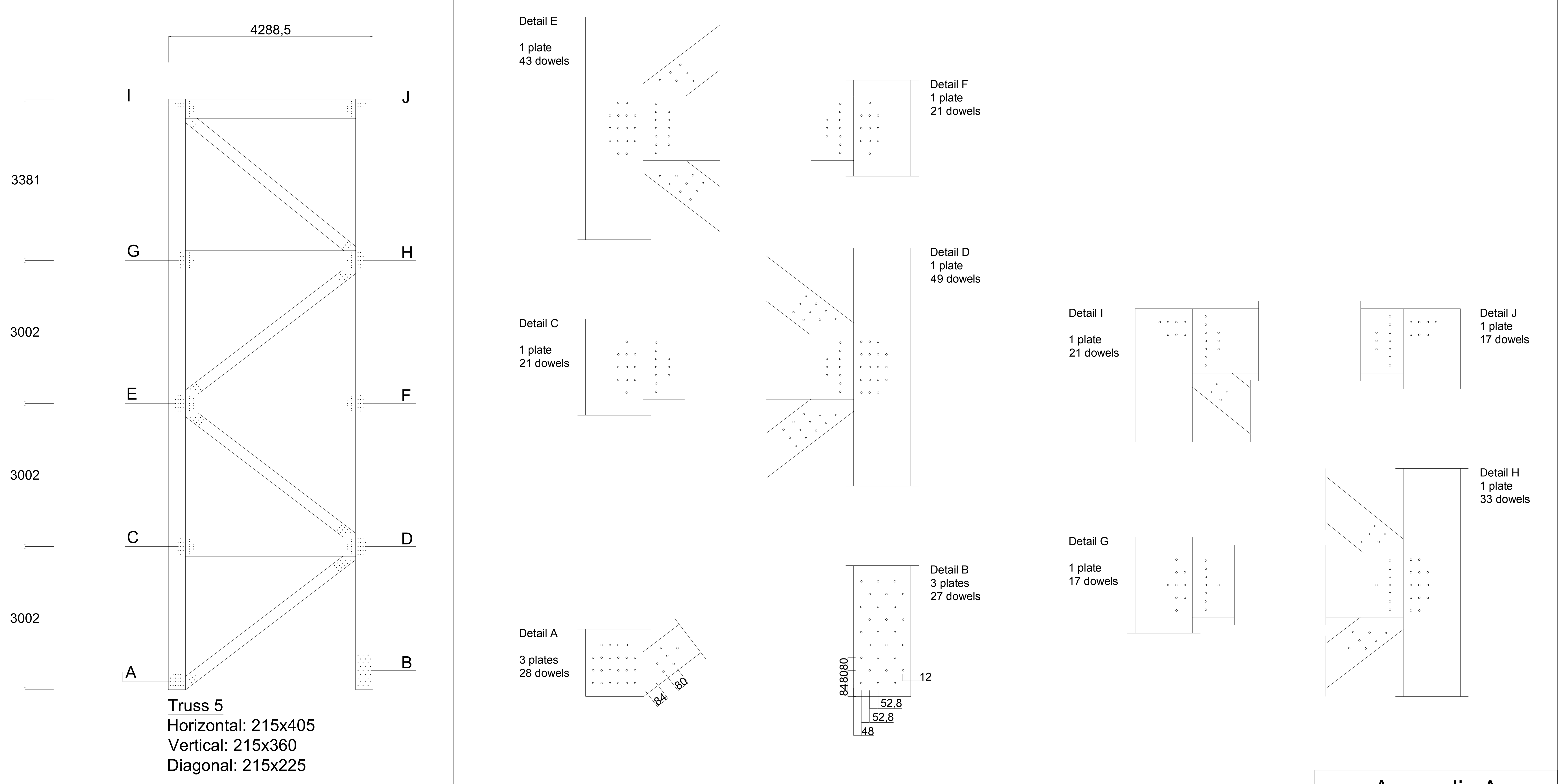




Appendix A

A.3 Trusses of four storey building

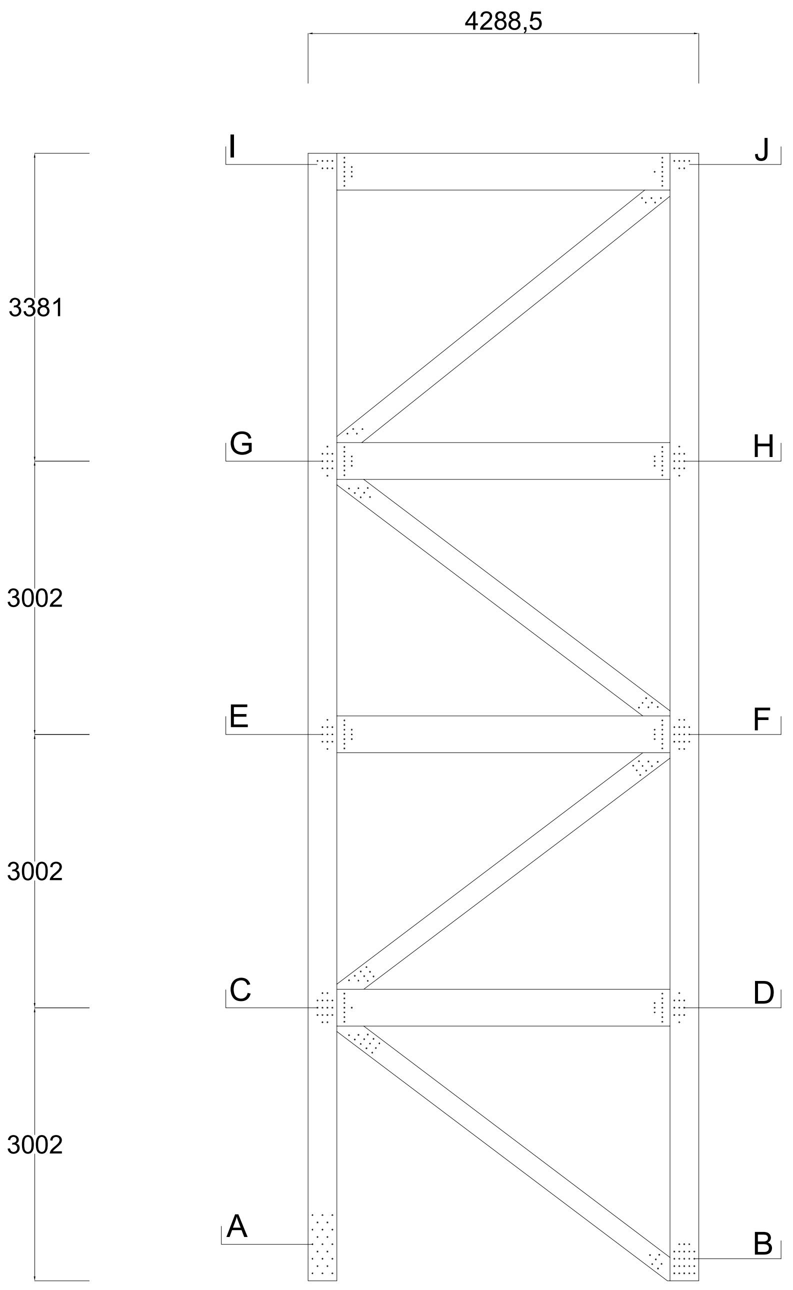
A13



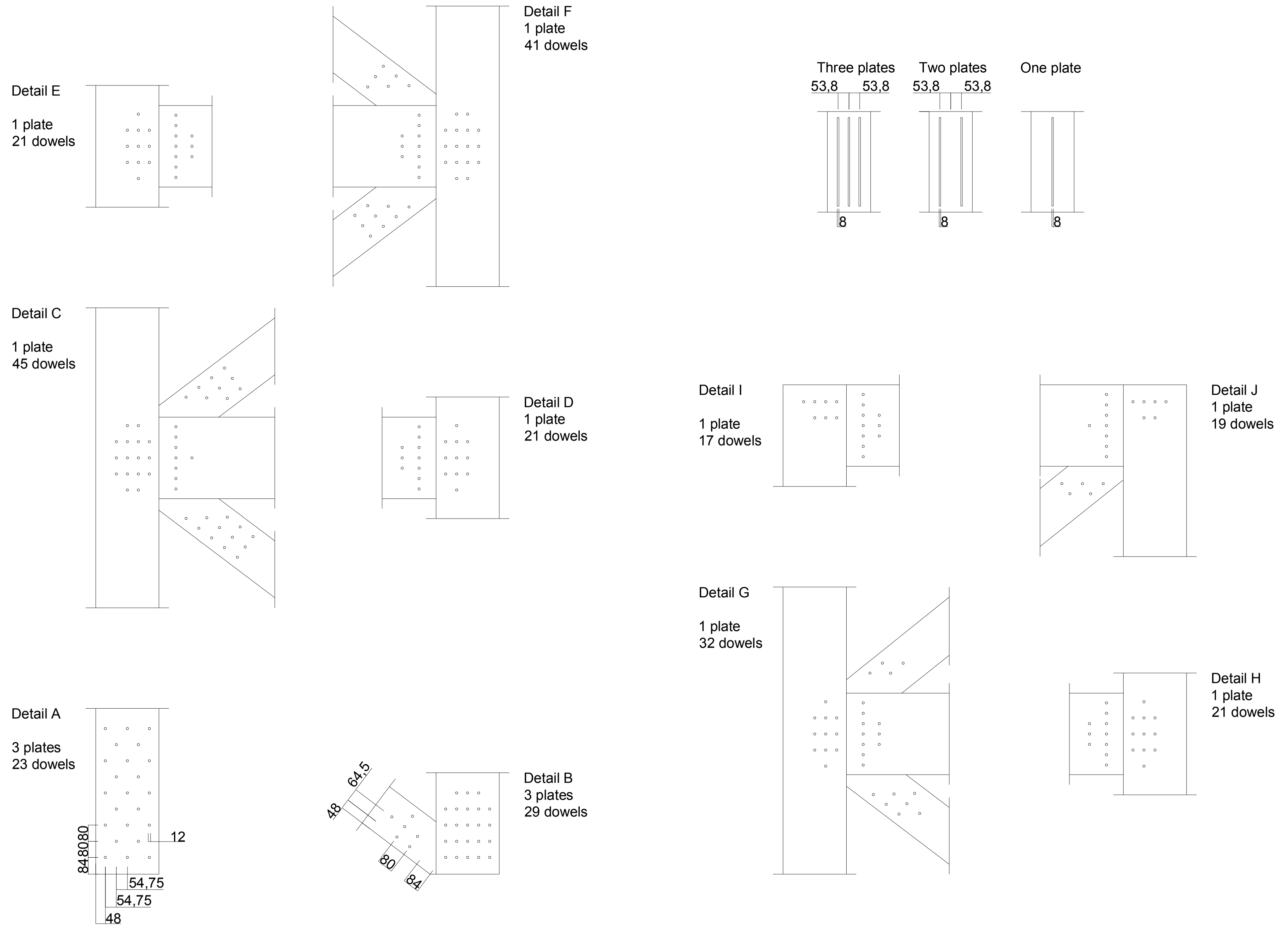
Appendix A

A.3 Trusses of four storey building

A14



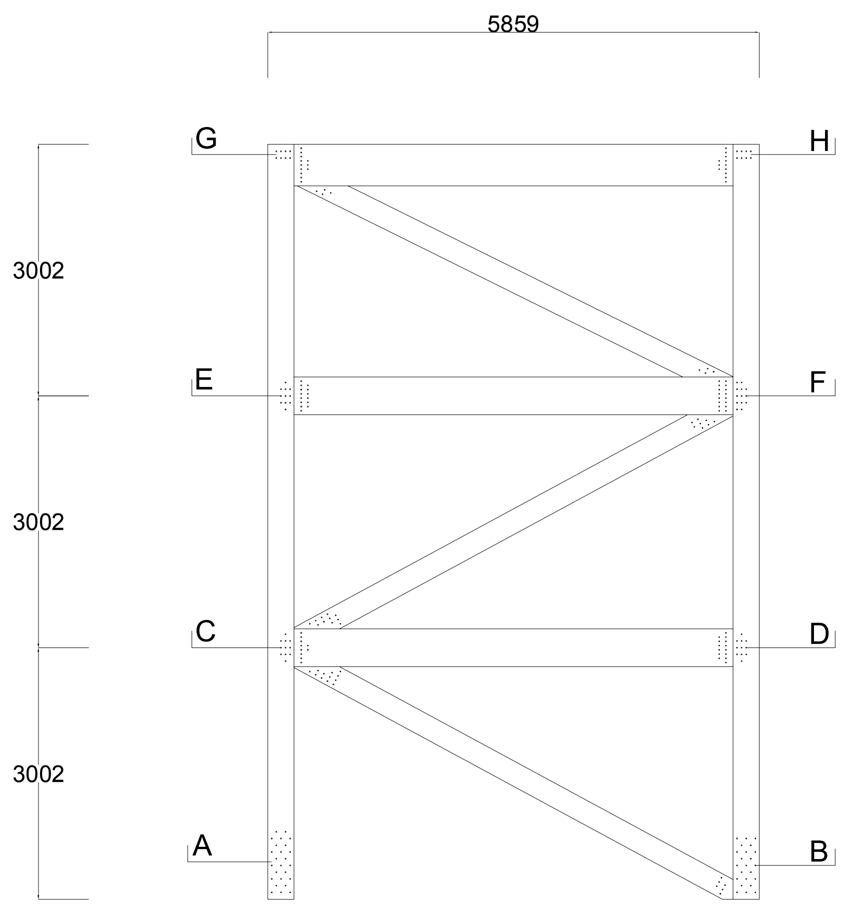
Truss 6
Horizontal: 215x405
Vertical: 215x315
Diagonal: 215x225



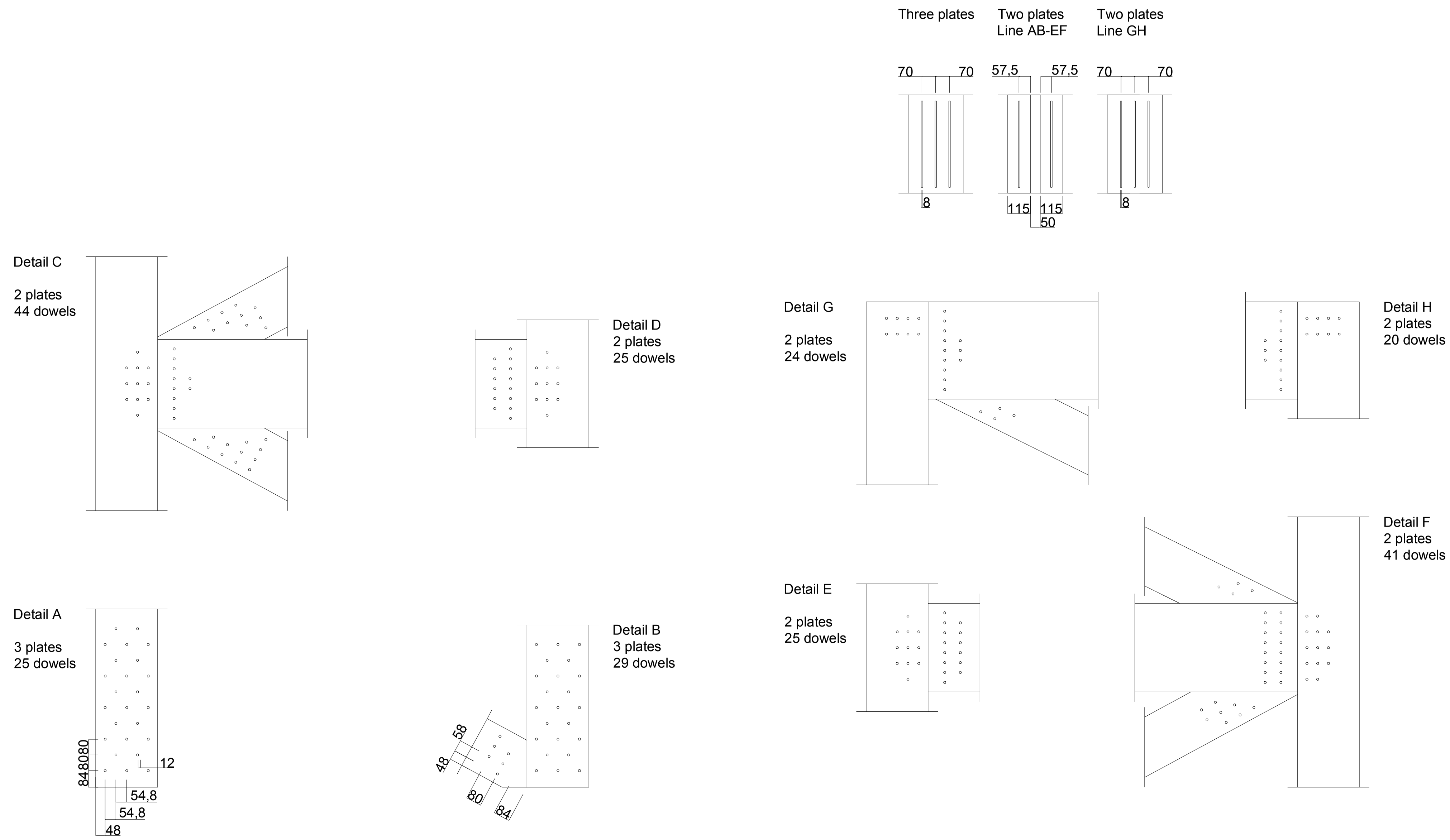
Appendix A

A.3 Trusses of four storey building

A15



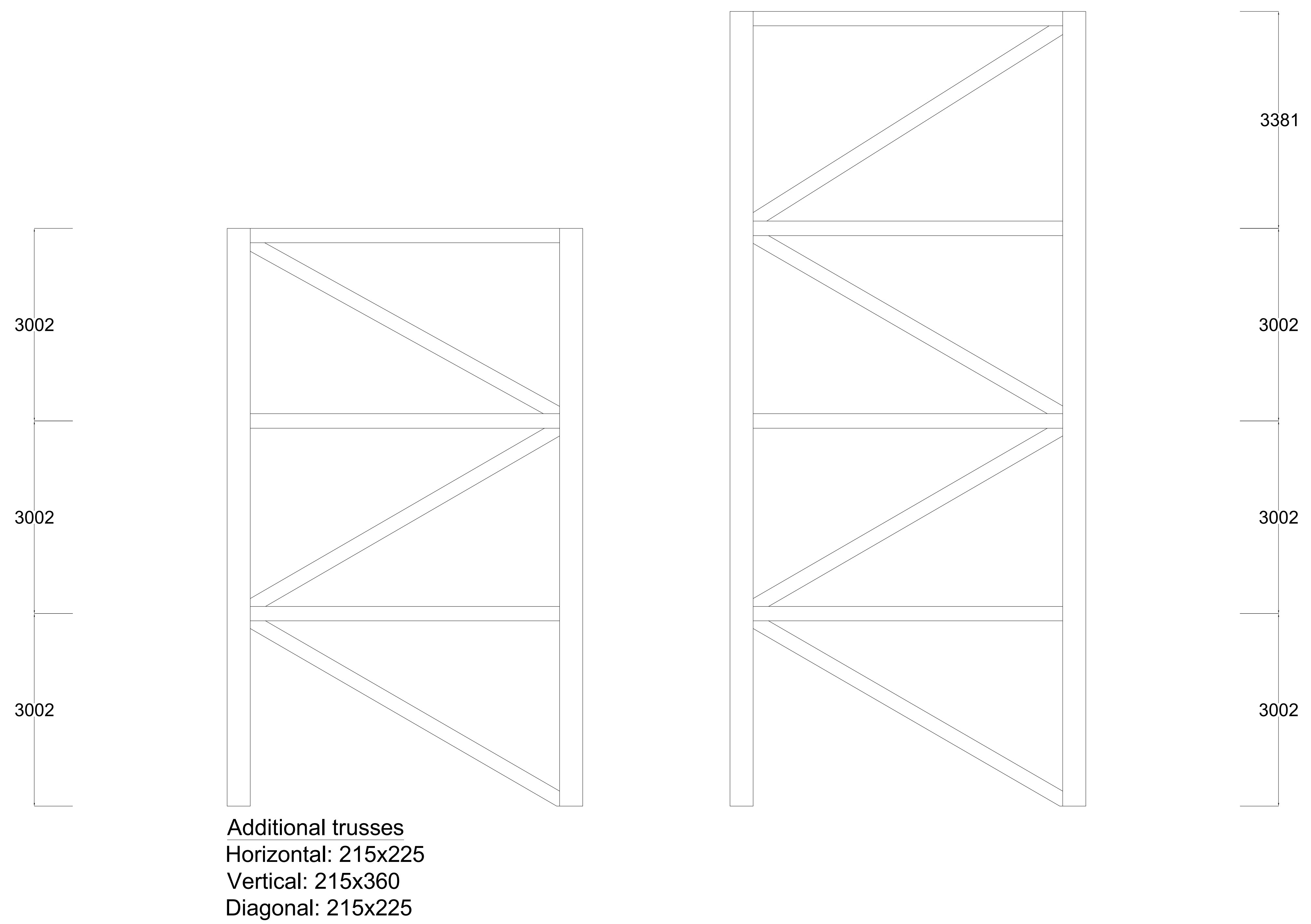
Truss 7
Horizontal: 115x450 x2
Horizontal at roof level: 280x495
Vertical: 280x315
Diagonal: 280x270



Appendix A

A.3 Trusses of four storey building

A16



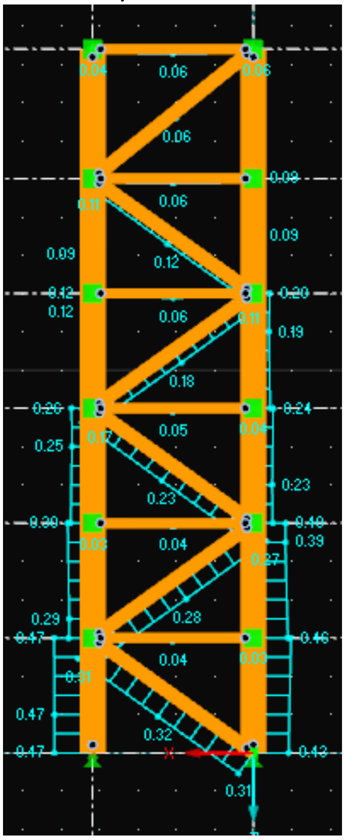
Appendix A
A.3 Trusses of four storey building
A17

Appendix B – Results from computational software

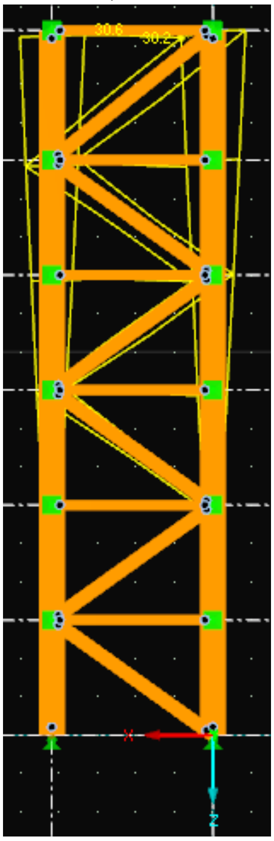
B.1 Dimensioning of trusses

Trusses of six storey building

Truss 1
Max utility 47 %

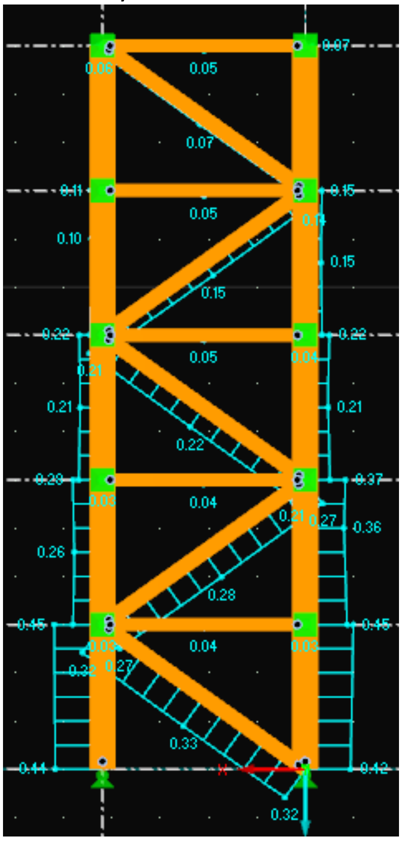


Global def, x: 30,6 mm
Local def, z: -

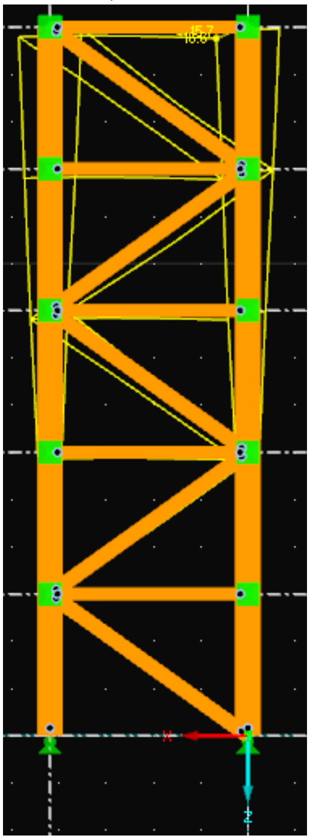


Appendix B

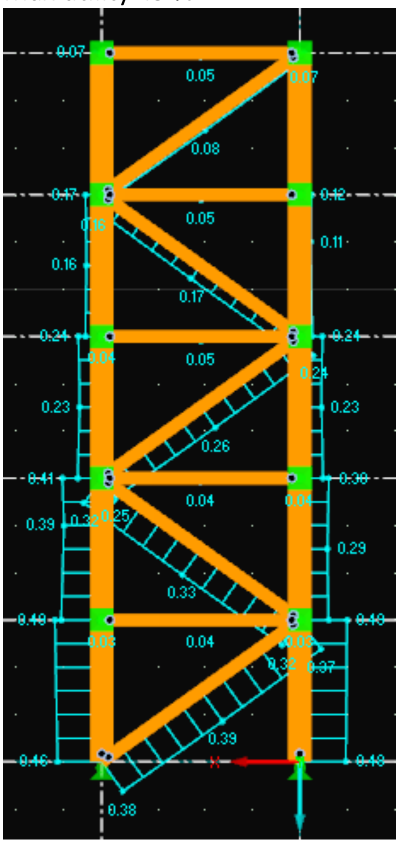
Truss 2
Max utility 45 %



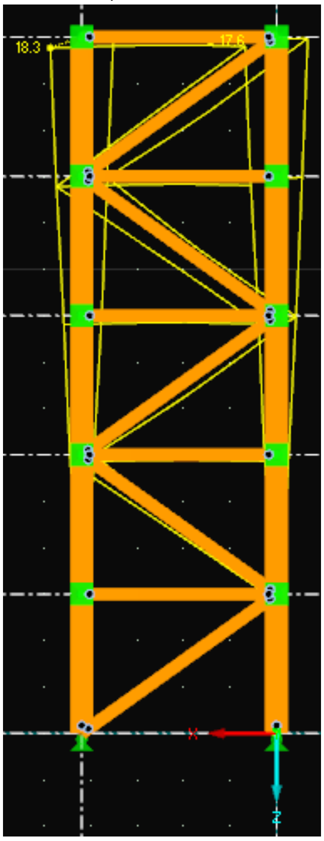
Global def, x: 15,6 mm
Local def, z: -



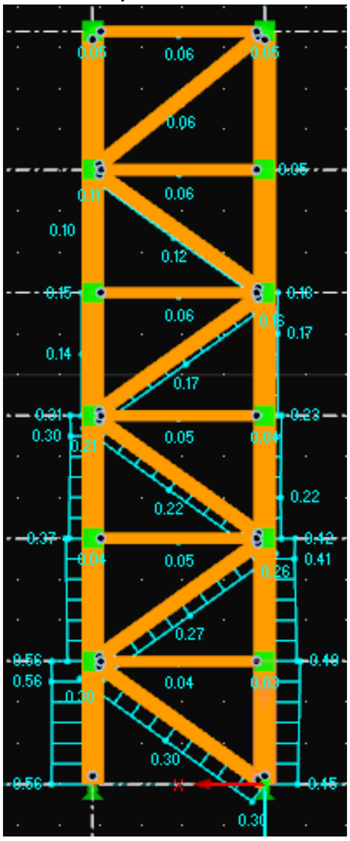
Truss 3
Max utility 49 %



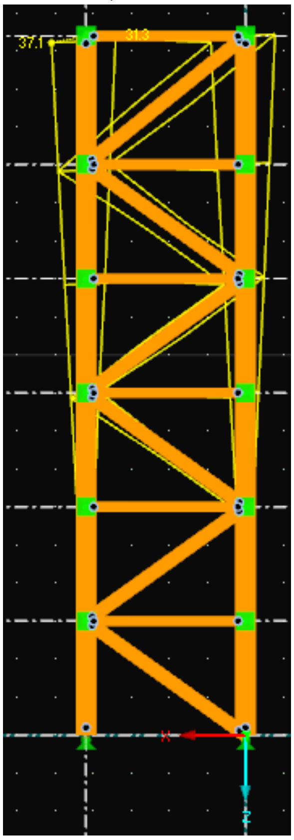
Global def, x: 17,3 mm
Local def, z: -



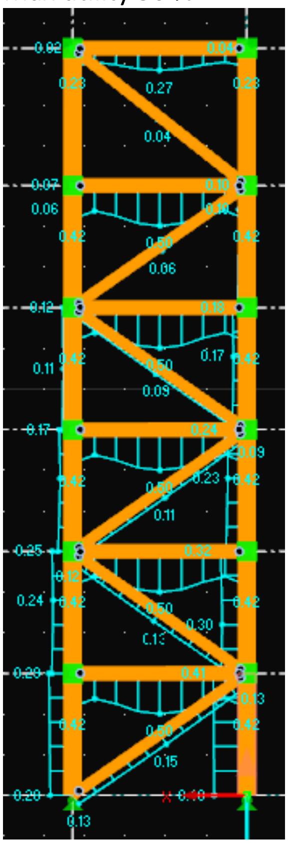
Truss 4
Max utility 56 %



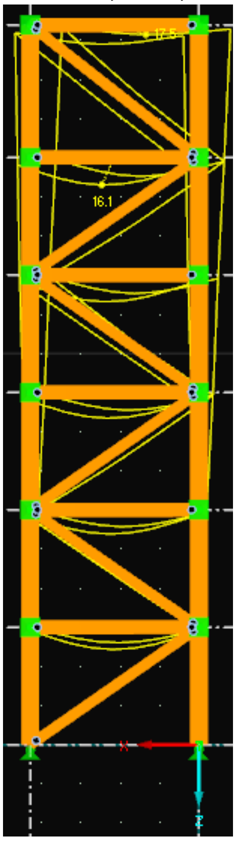
Global def, x: 36,4 mm
Local def, z: -



Truss 5
Max utility 50 %

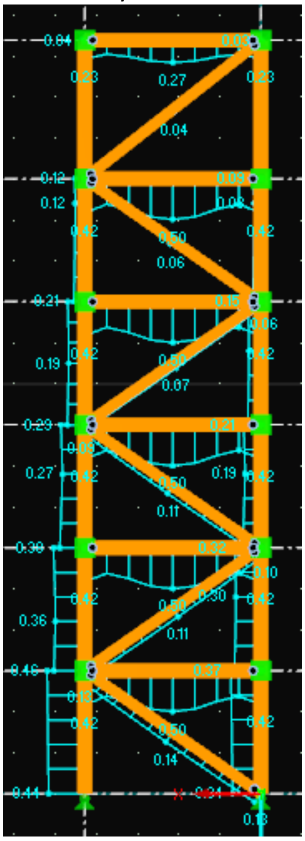


Global def, x: 16,7 mm
Local def, z: 14,7 mm

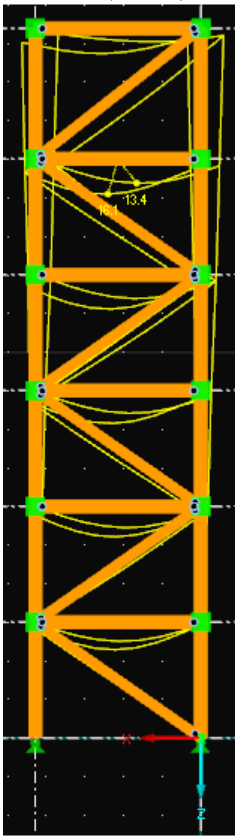


Appendix B

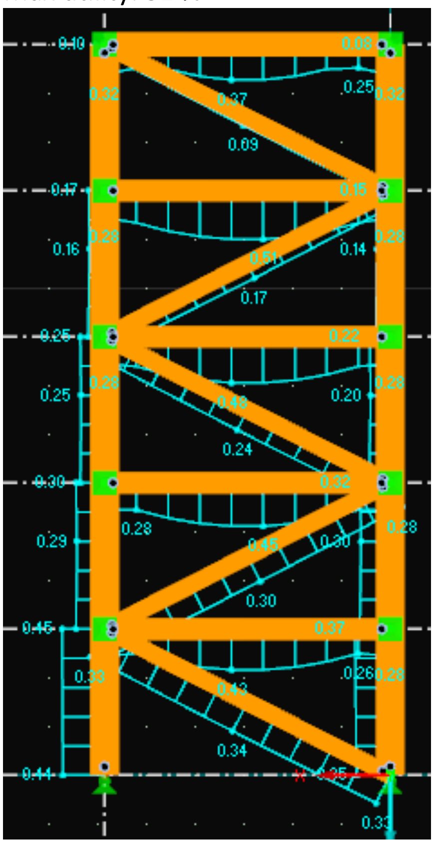
Truss 6
Max utility 50 %



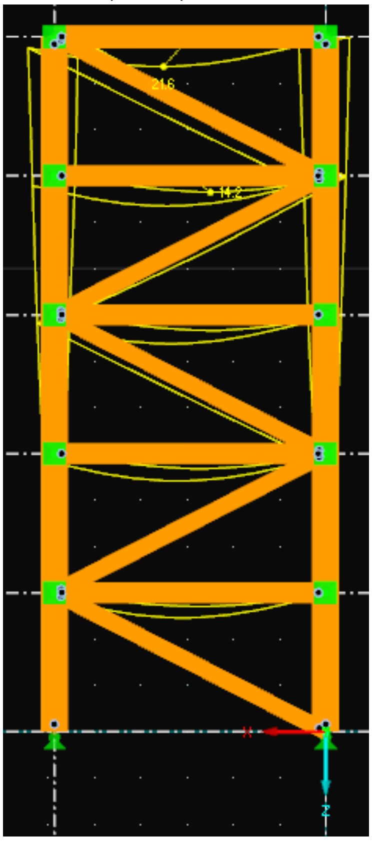
Global def, x: 10,0 mm
Local def, z: 15,5 mm



Truss 7
Max utility: 51 %



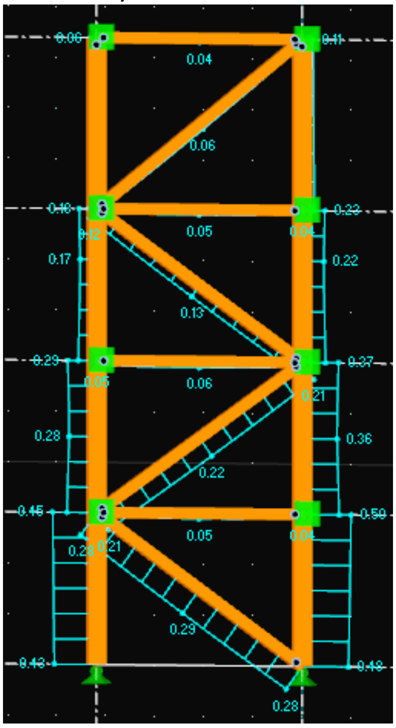
Global def, x: 14,5 mm
Local def, z: 16,4 mm



Trusses of four storey building

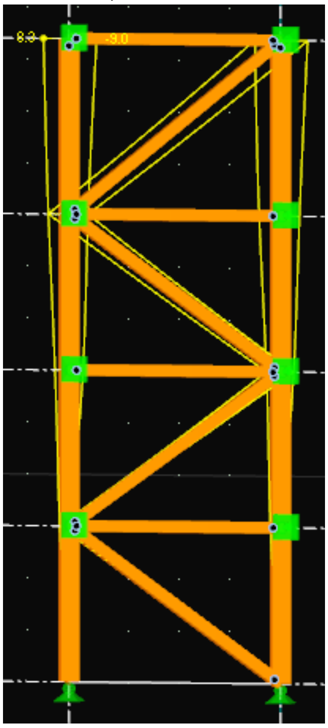
Truss 1

Max utility 50 %

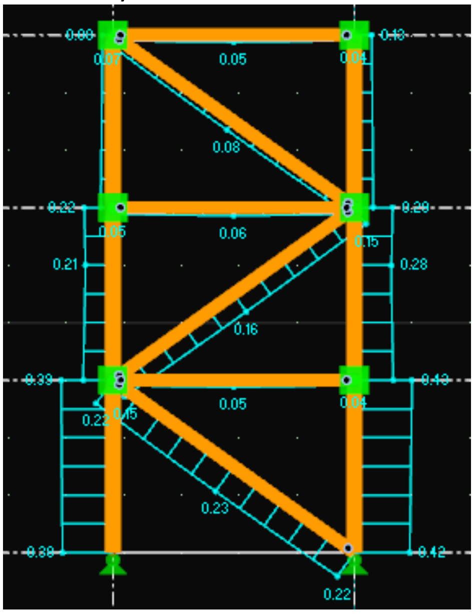


Global def, x: 9,0 mm

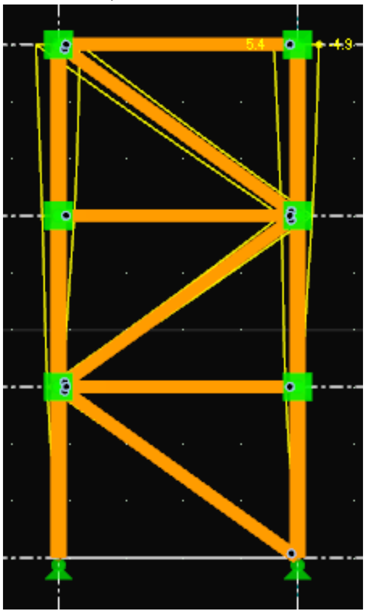
Local def, z: -



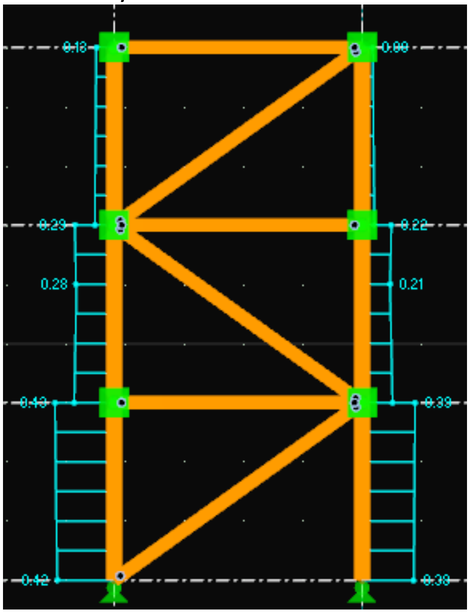
Truss 2
Max utility 43 %



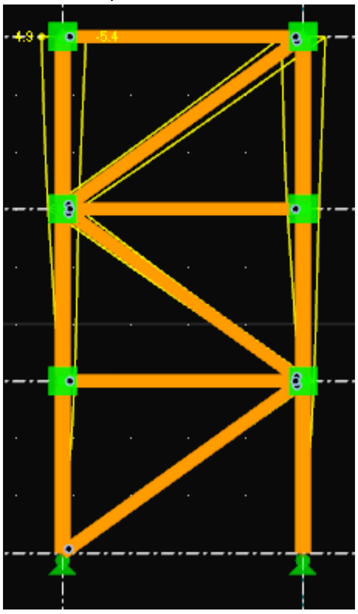
Global def, x: 5,4 mm
Local def, z: -



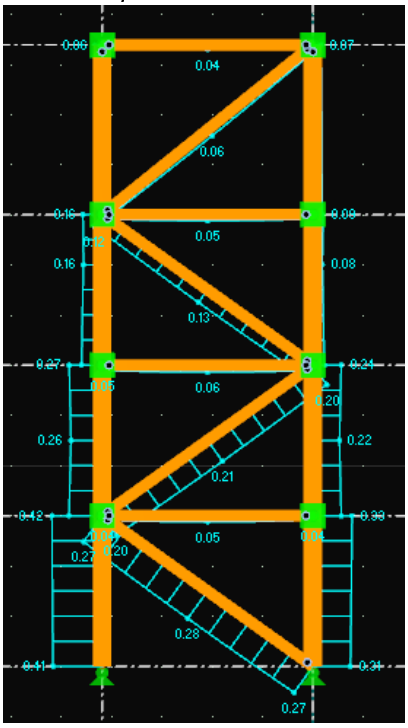
Truss 3
Max utility 43 %



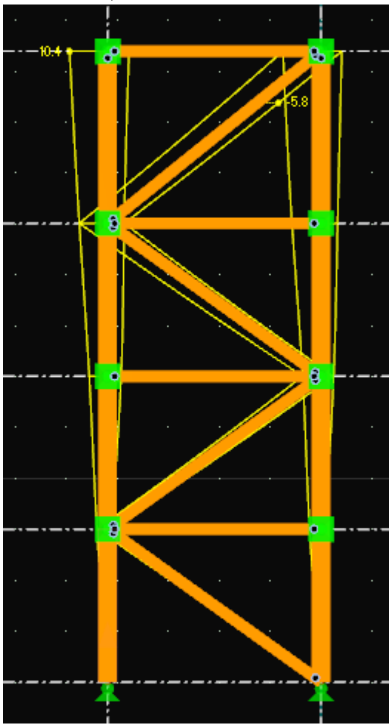
Global def, x: 5,4 mm
Local def, z: -



Truss 4
Max utility 42 %

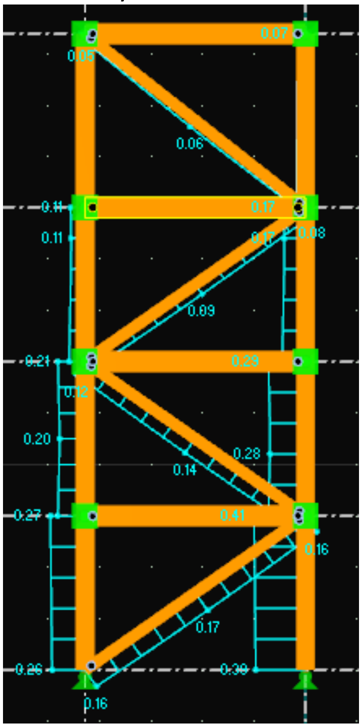


Global def, x: 10,4 mm
Local def, z: -

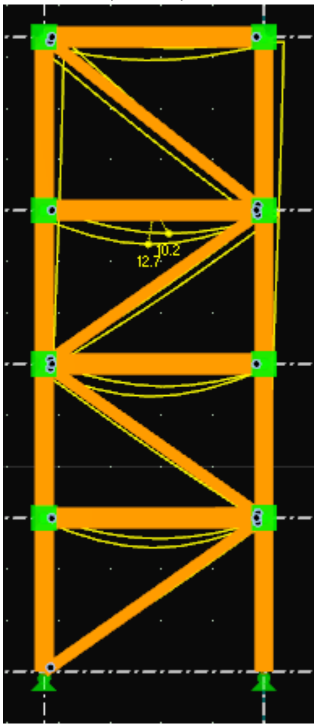


Appendix B

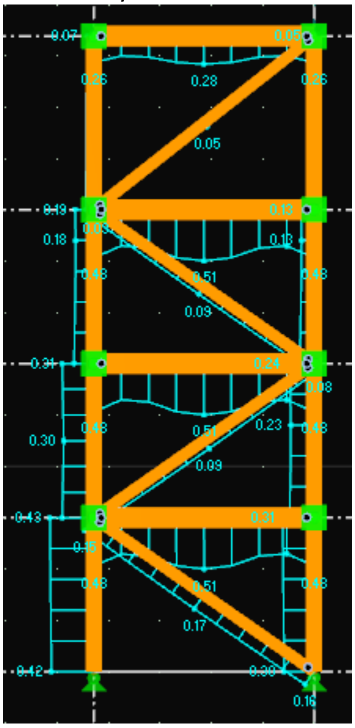
Truss 5
Max utility 41 %



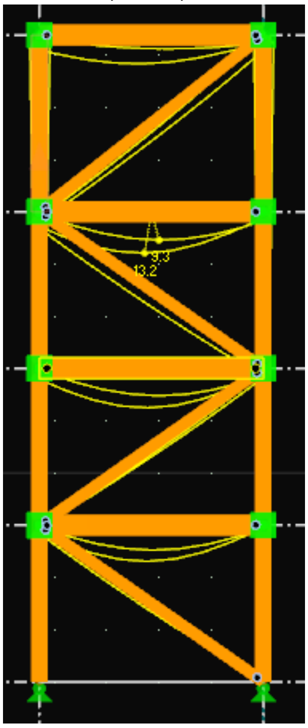
Global def, x: 7,4 mm
Local def, z: 12,5 mm



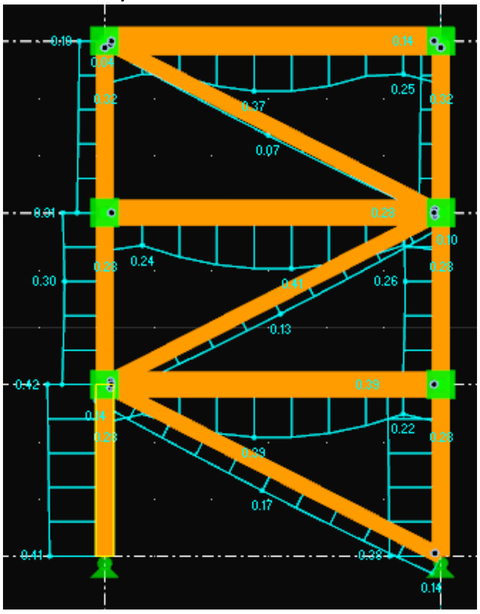
Truss 6
Max utility 51 %



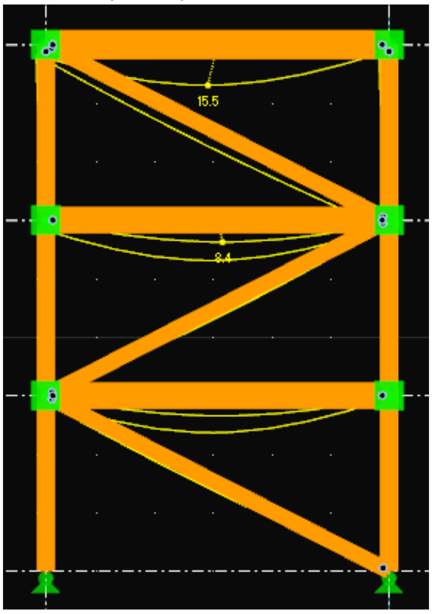
Global def, x: 3,6 mm
Local def, z: 13,0 mm



Truss 7
Max utility 42 %



Global def, x: 3,9 mm
Local def, z: 15,1 mm

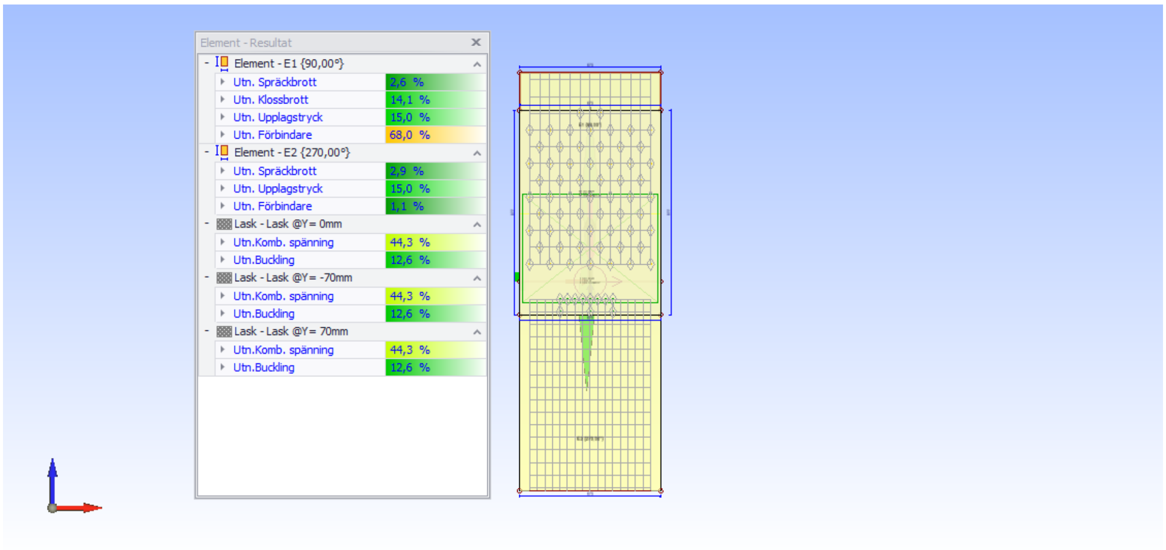


B.2 Design of connections

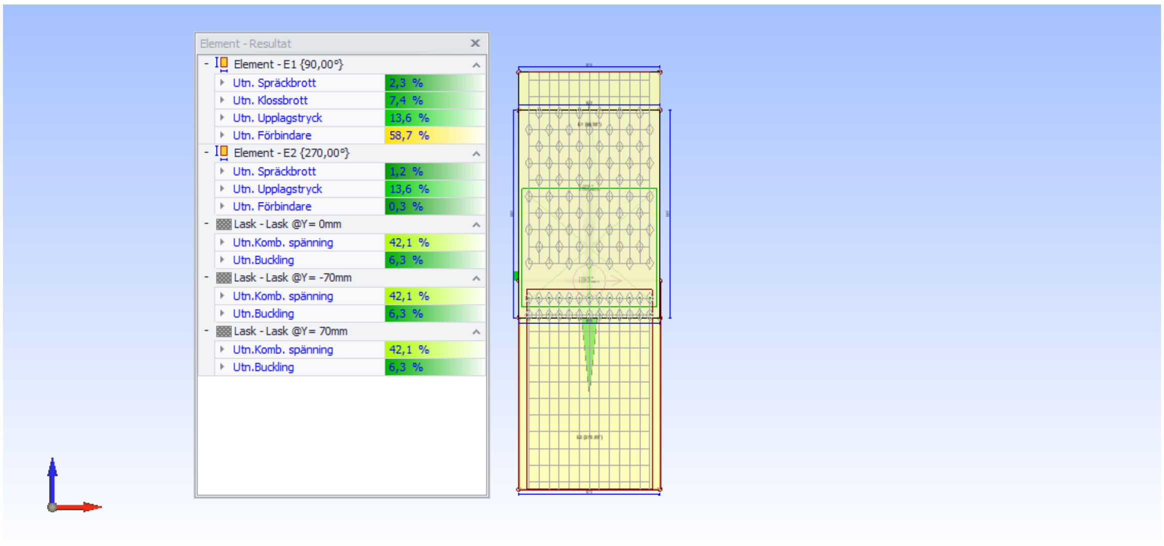
Connections of six storey building

Truss 1

Connection A

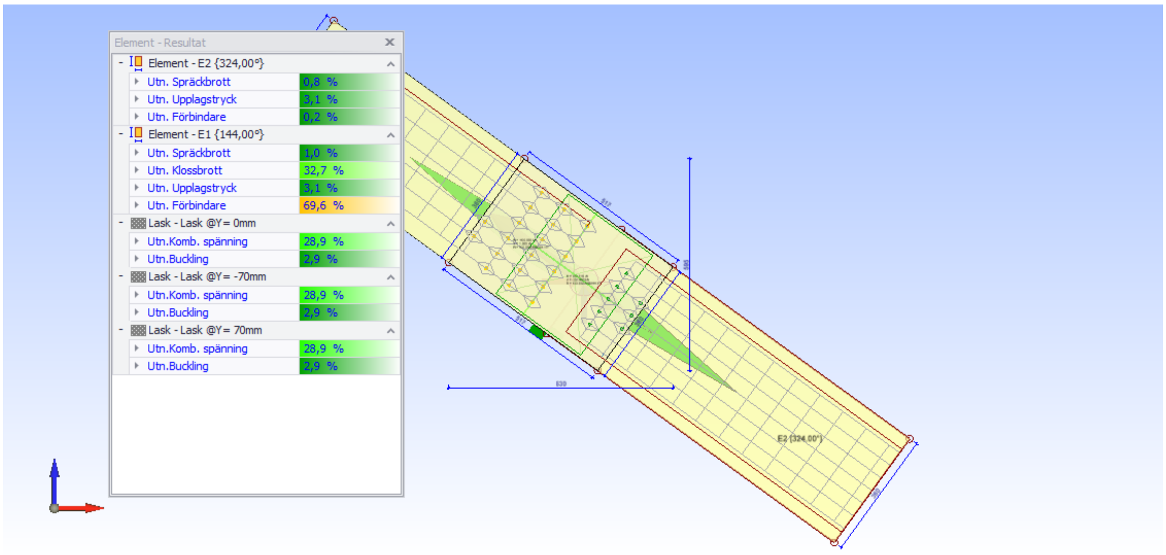


Connection B, vertical

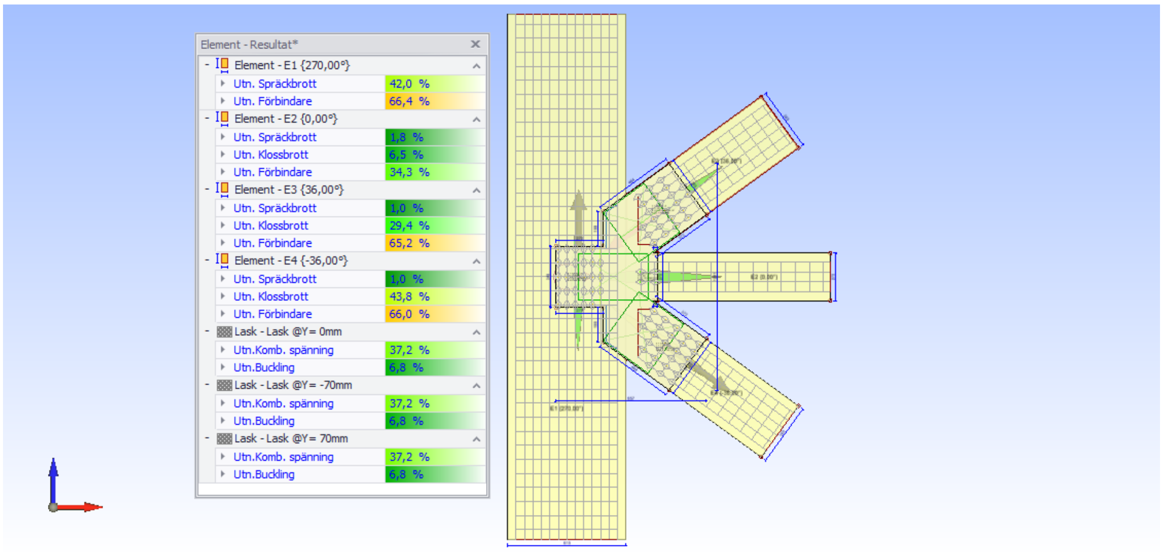


Appendix B

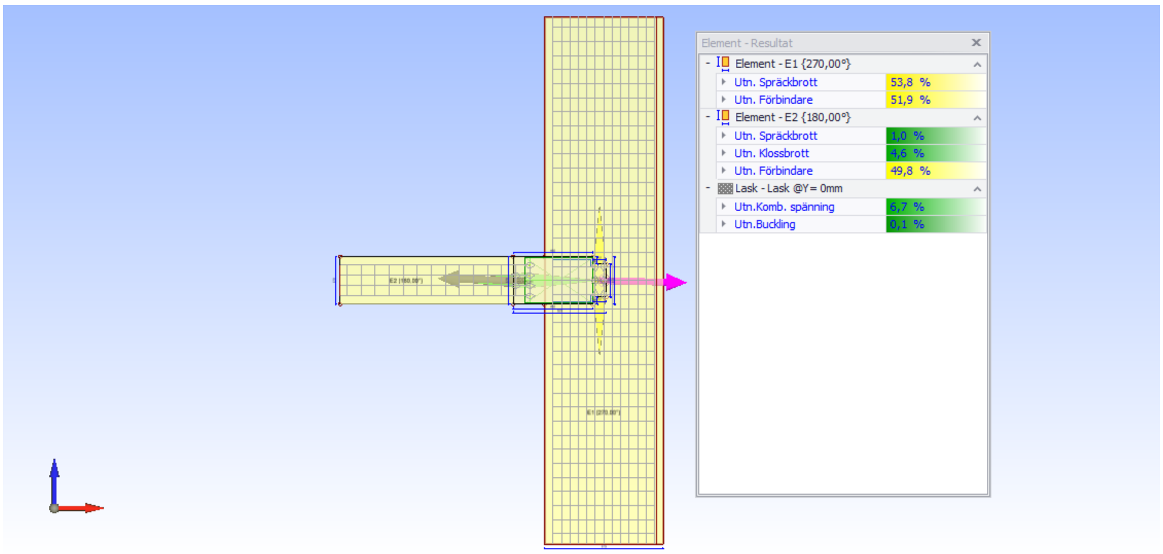
Connection B, diagonal



Connection C

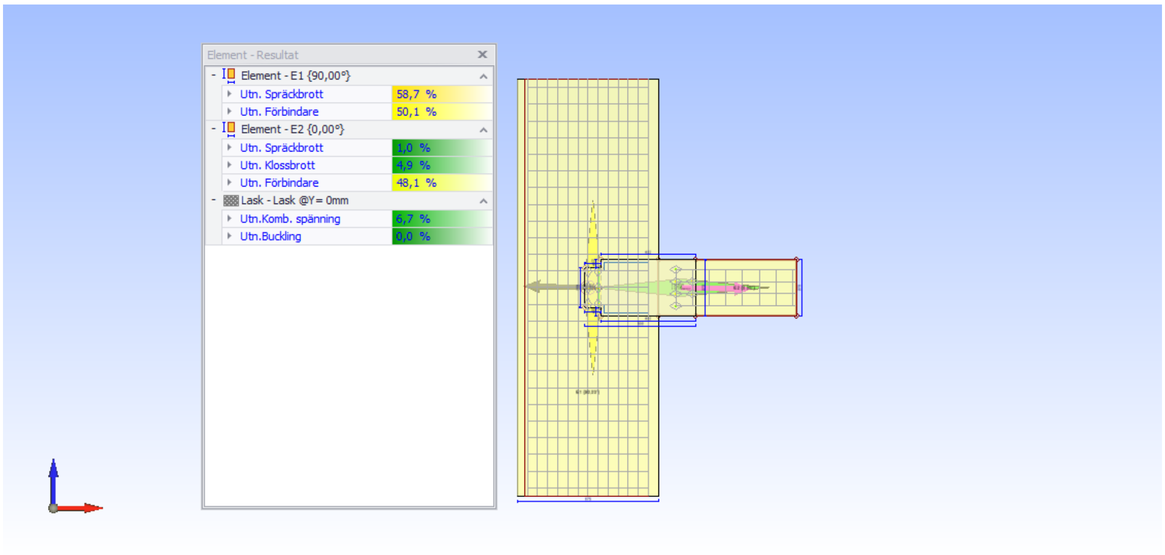


Connection D

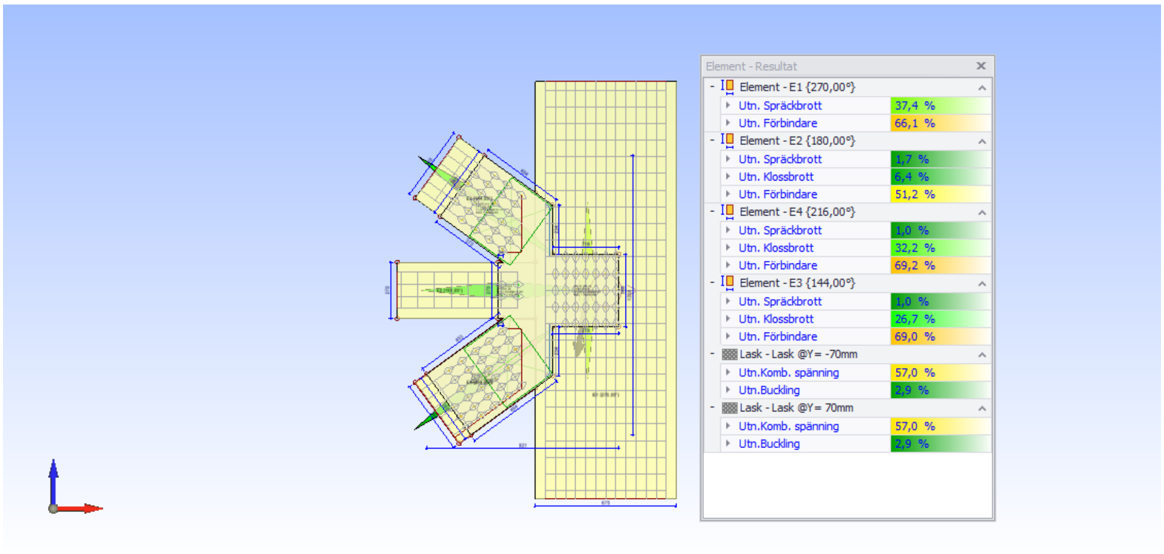


Appendix B

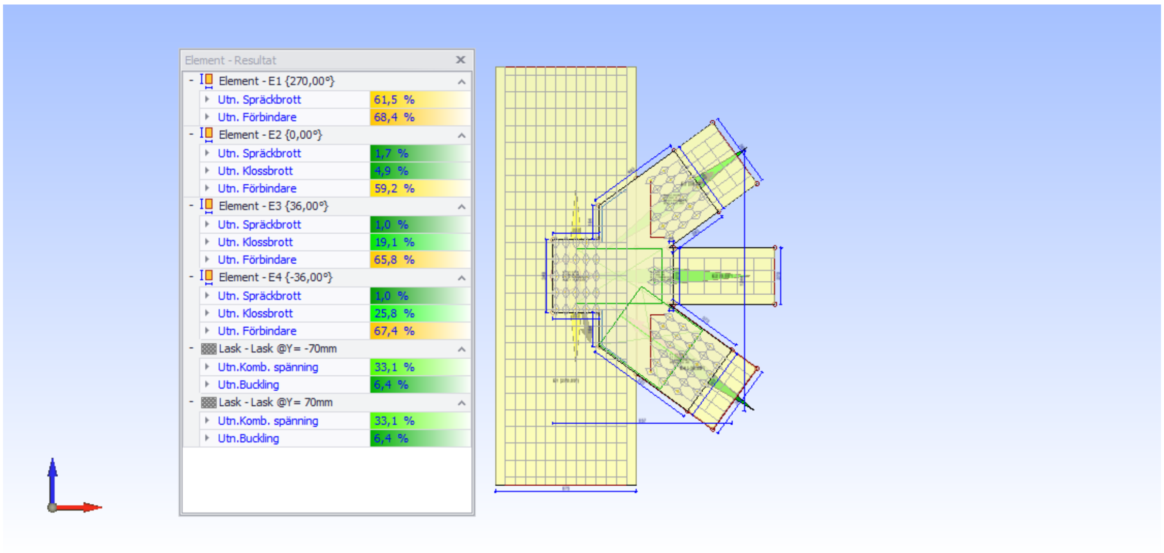
Connection E



Connection F

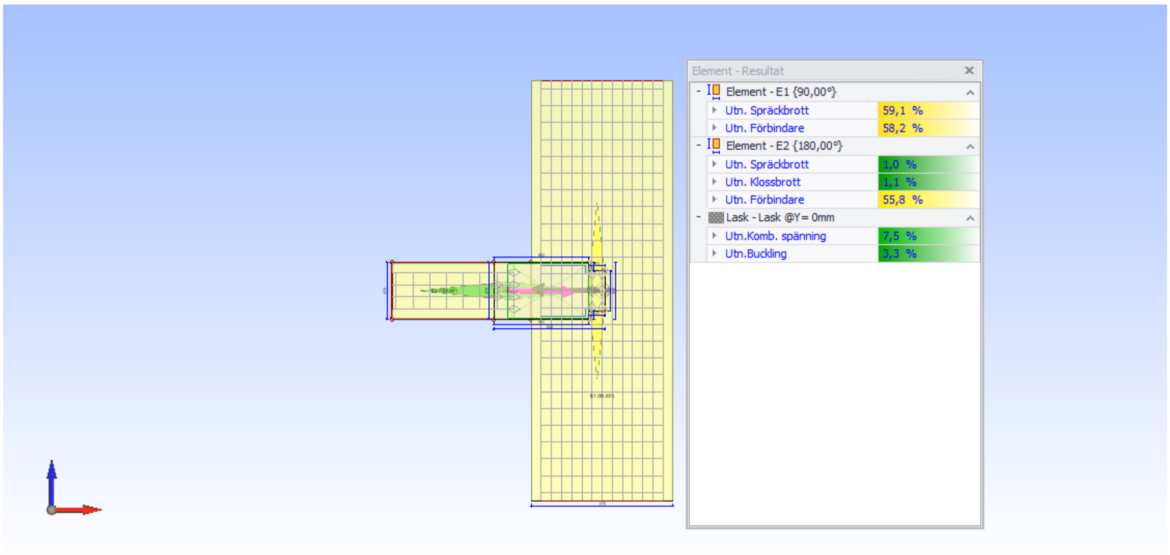


Connection G

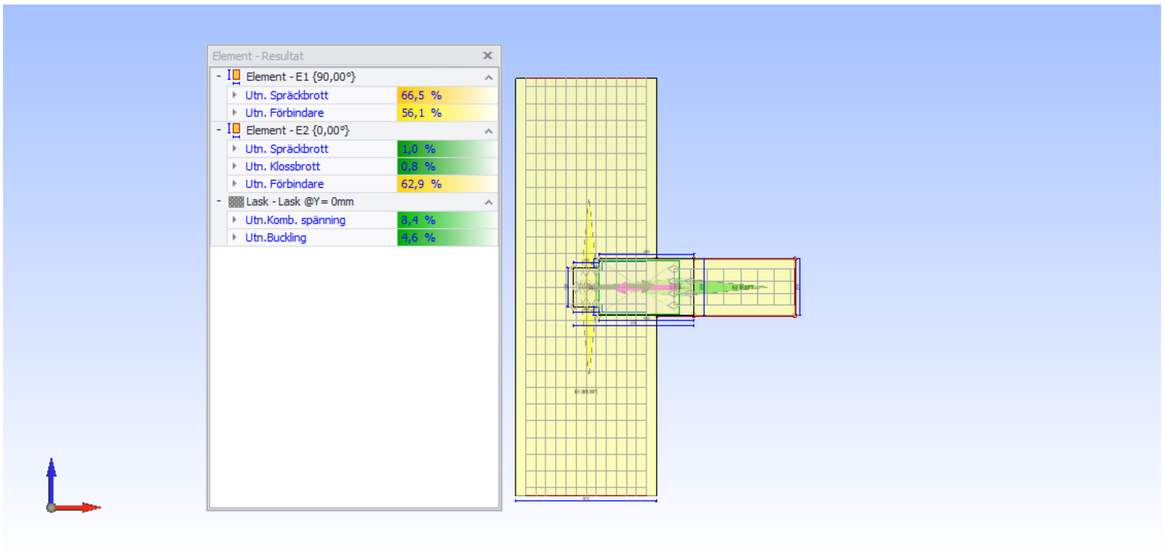


Appendix B

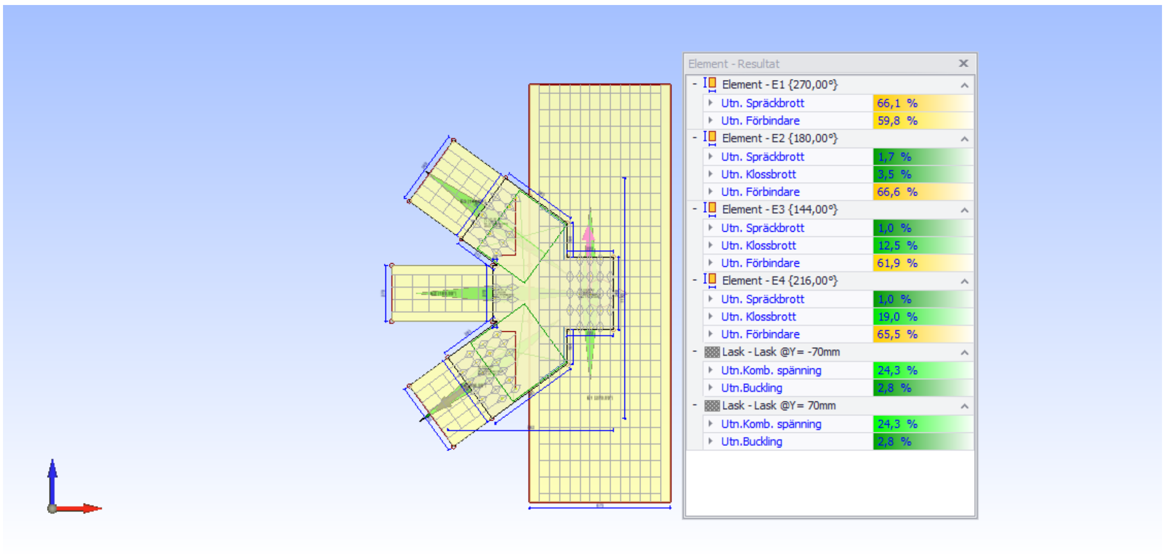
Connection H



Connection I

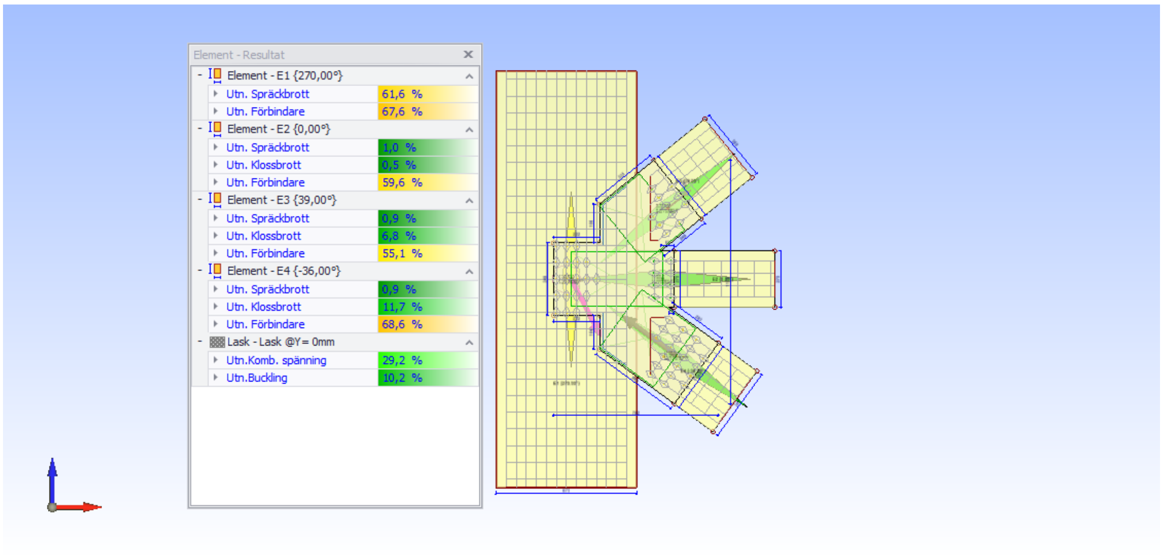


Connection J

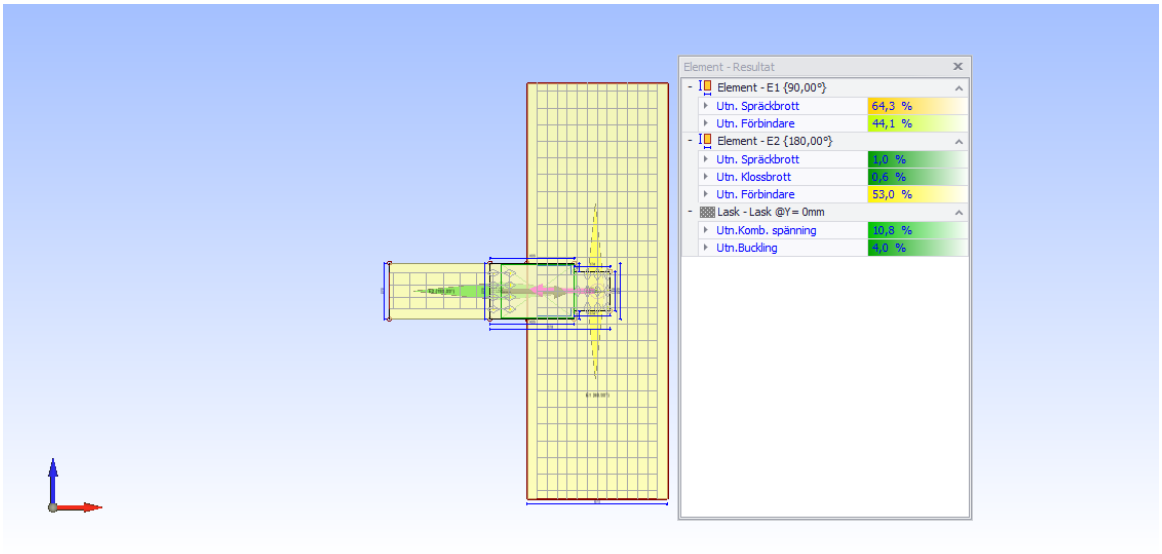


Appendix B

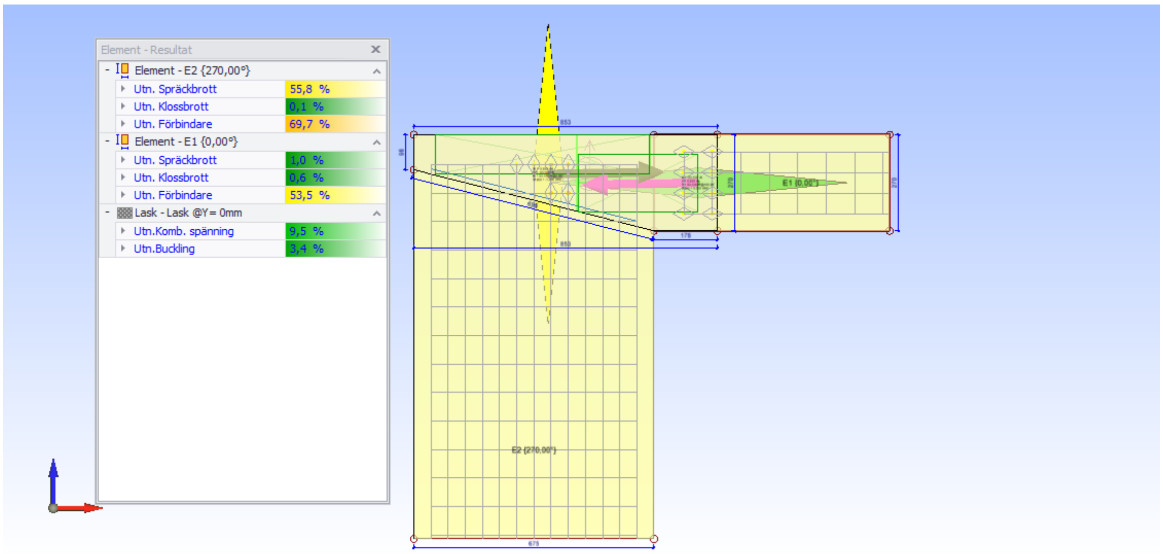
Connection K



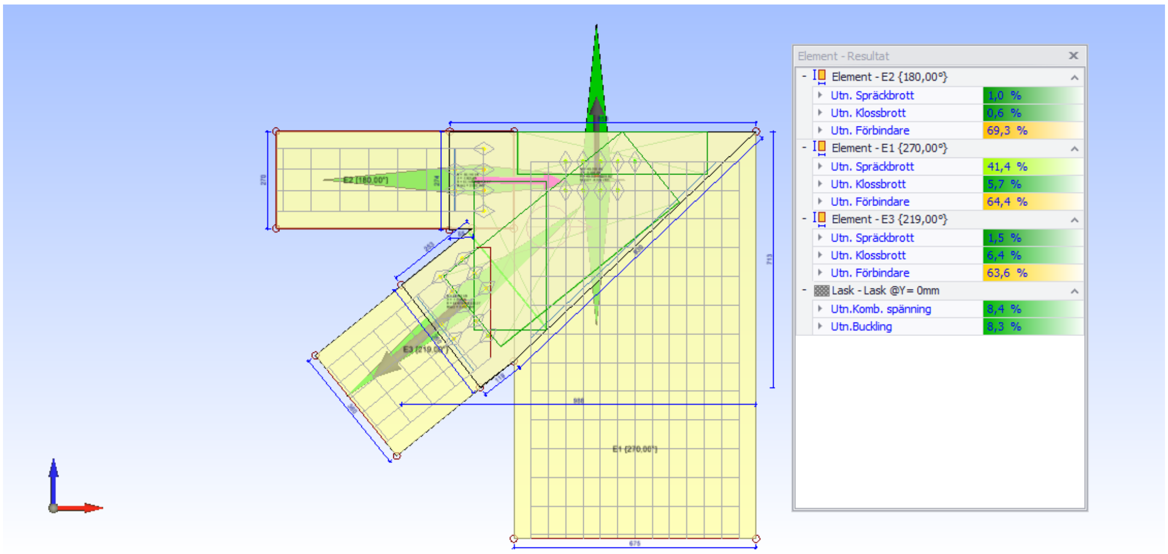
Connection L



Connection M

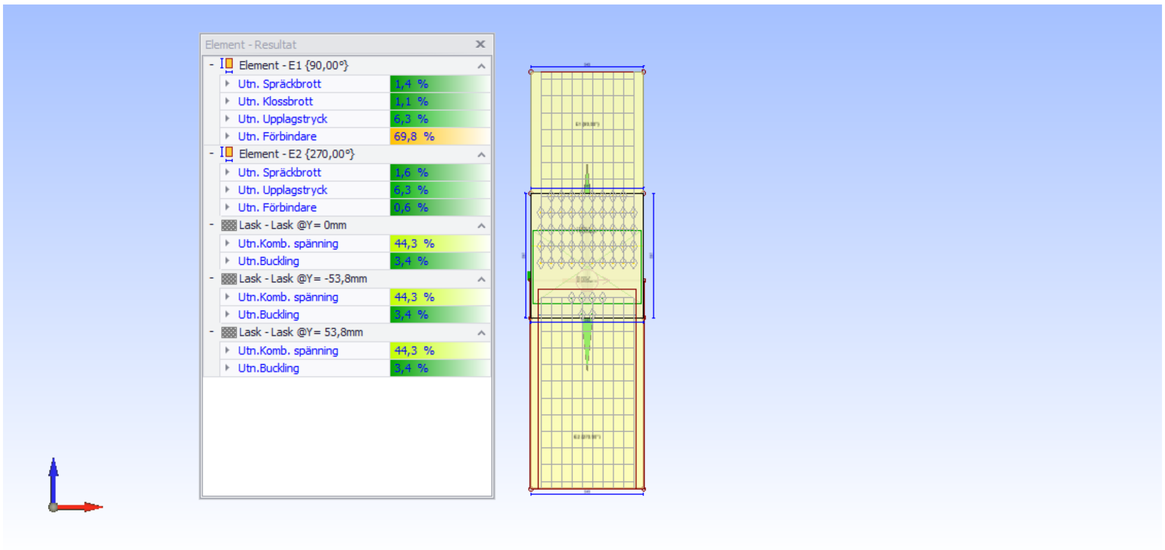


Connection N

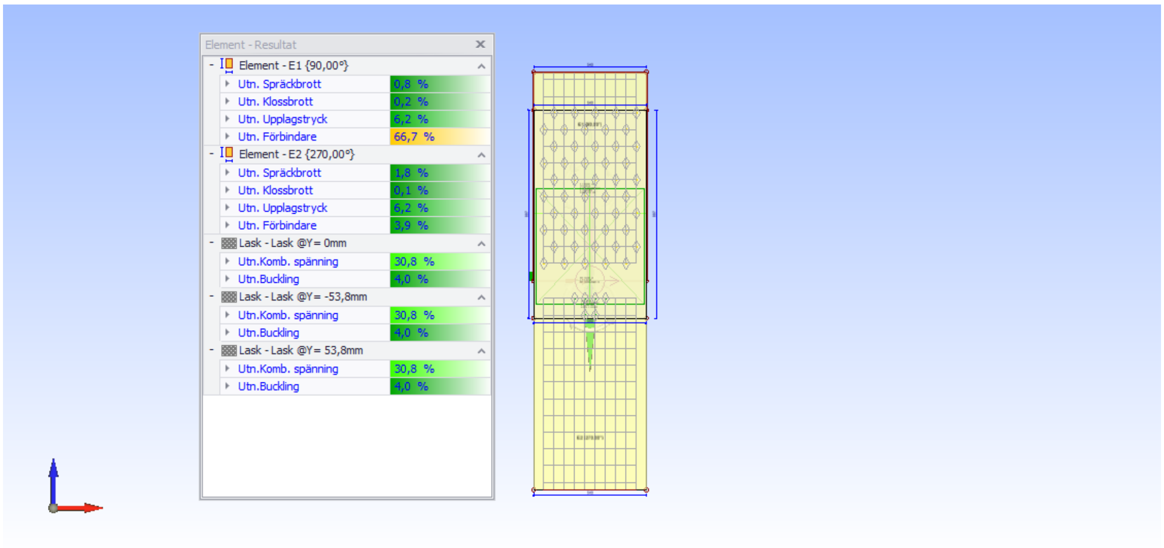


Truss 2

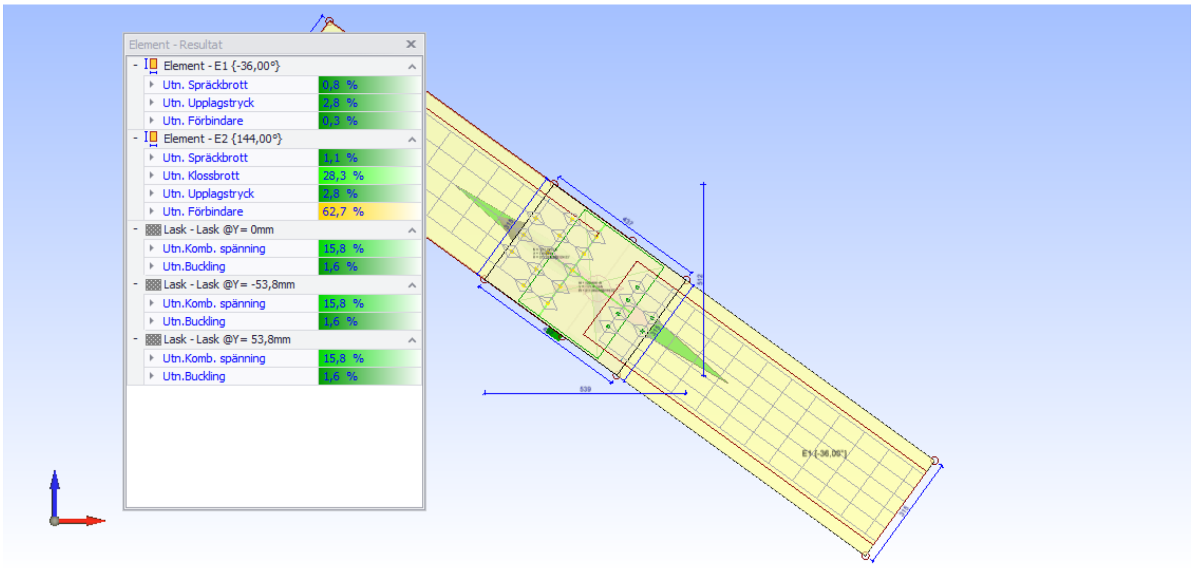
Connection A



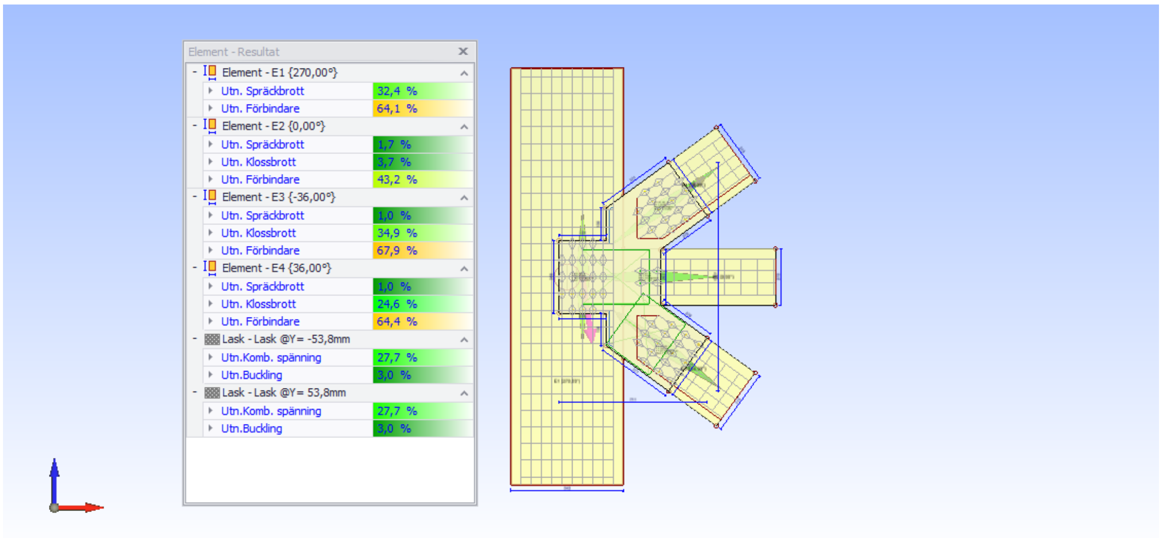
Connection B, vertical



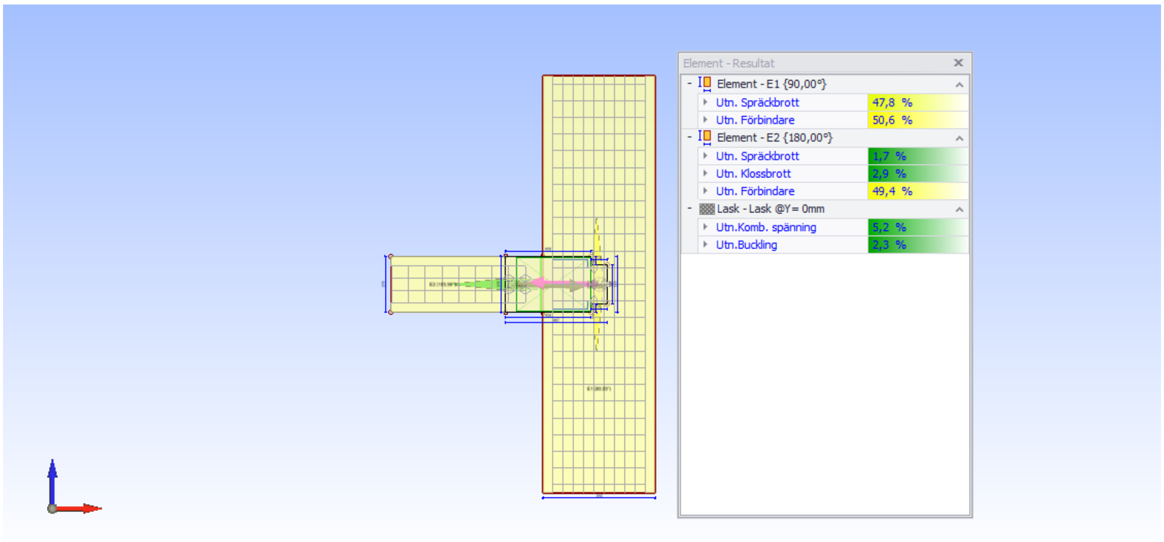
Connection B, diagonal



Connection C

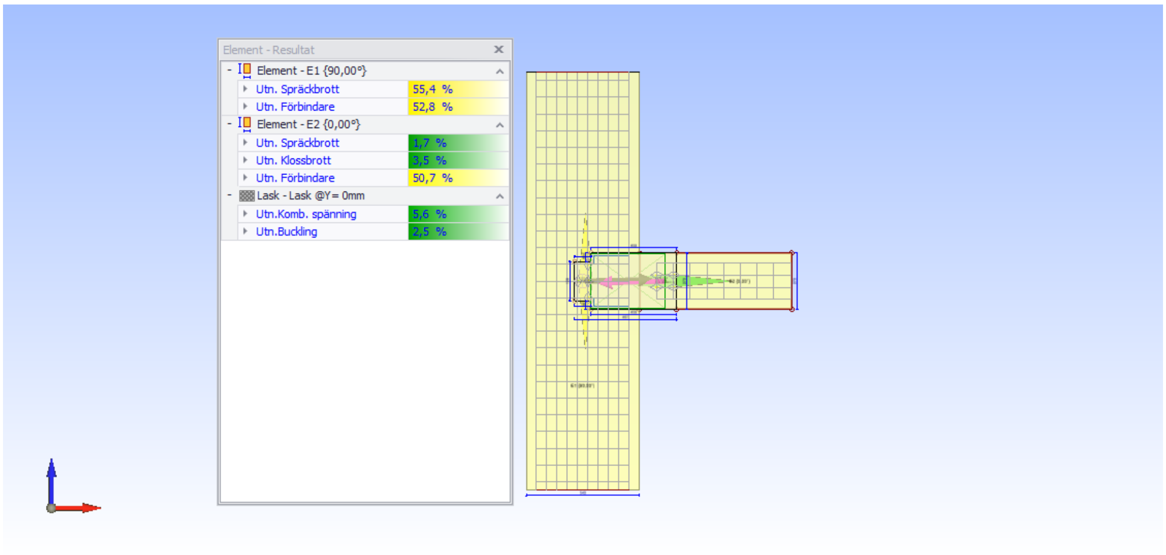


Connection D

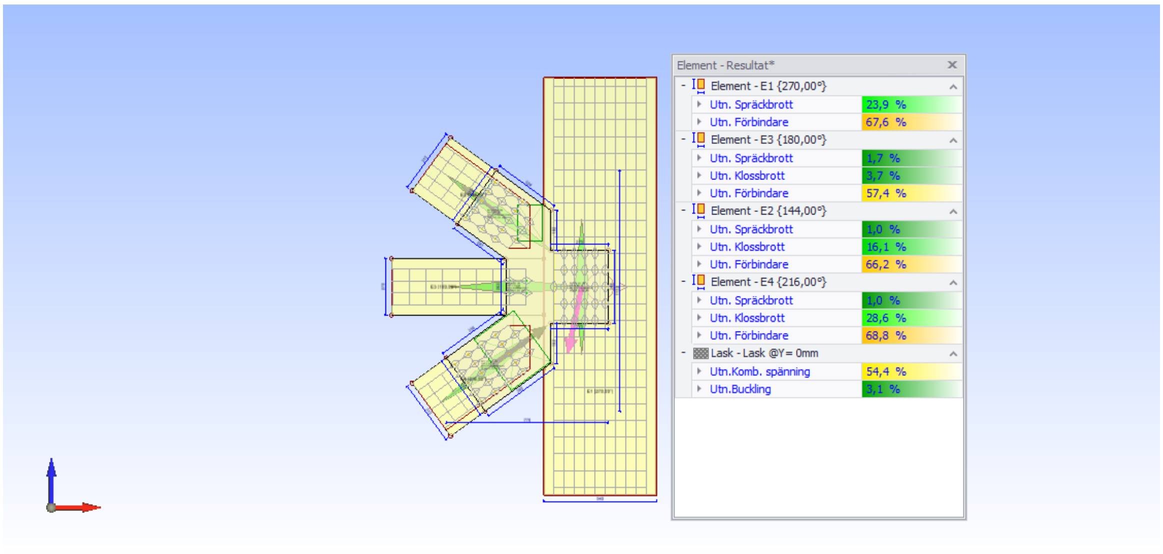


Appendix B

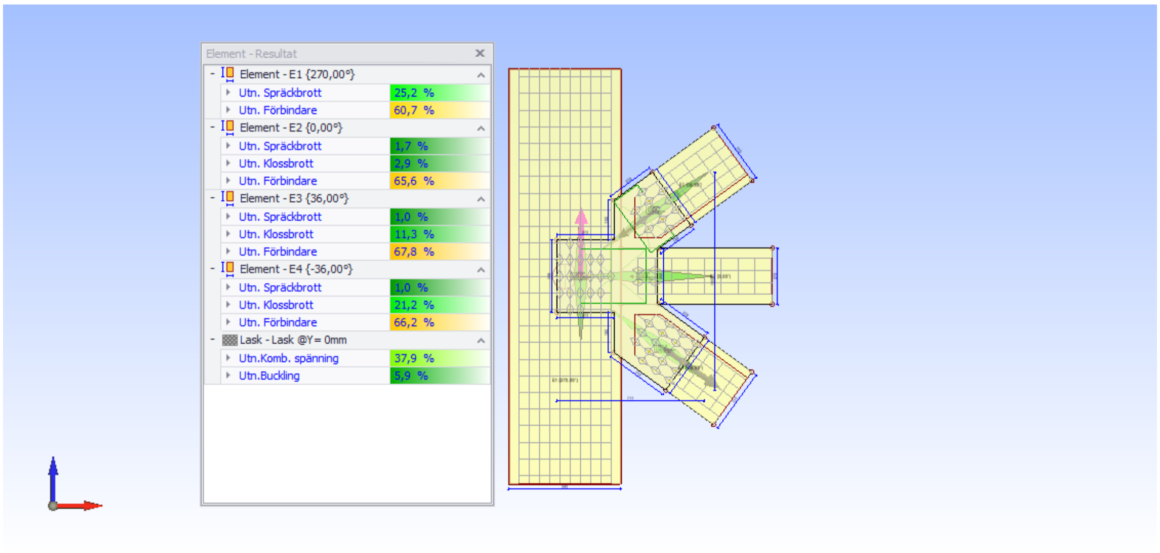
Connection E



Connection F

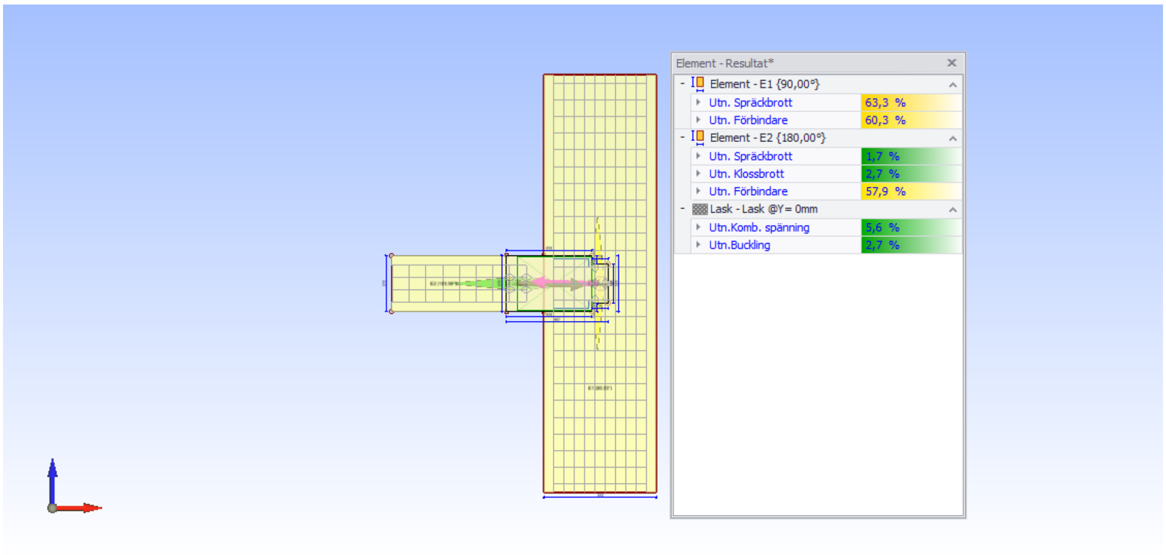


Connection G

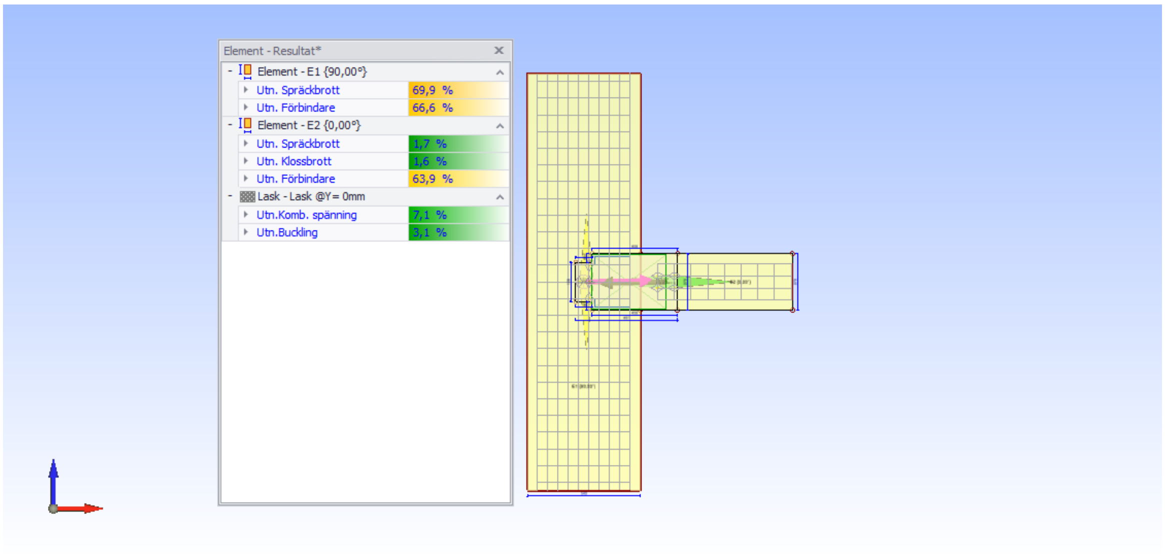


Appendix B

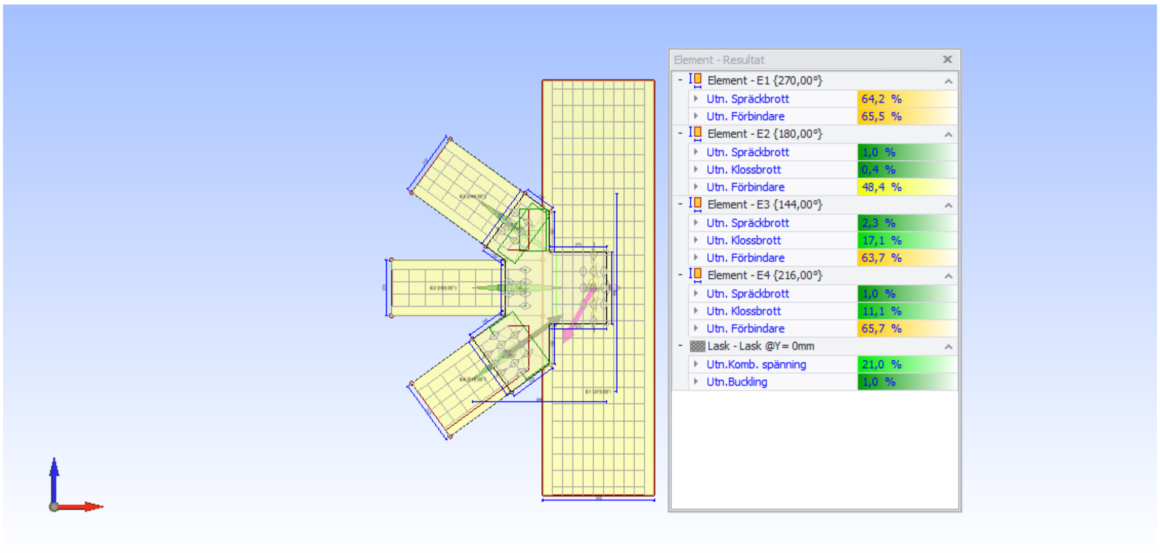
Connection H



Connection I

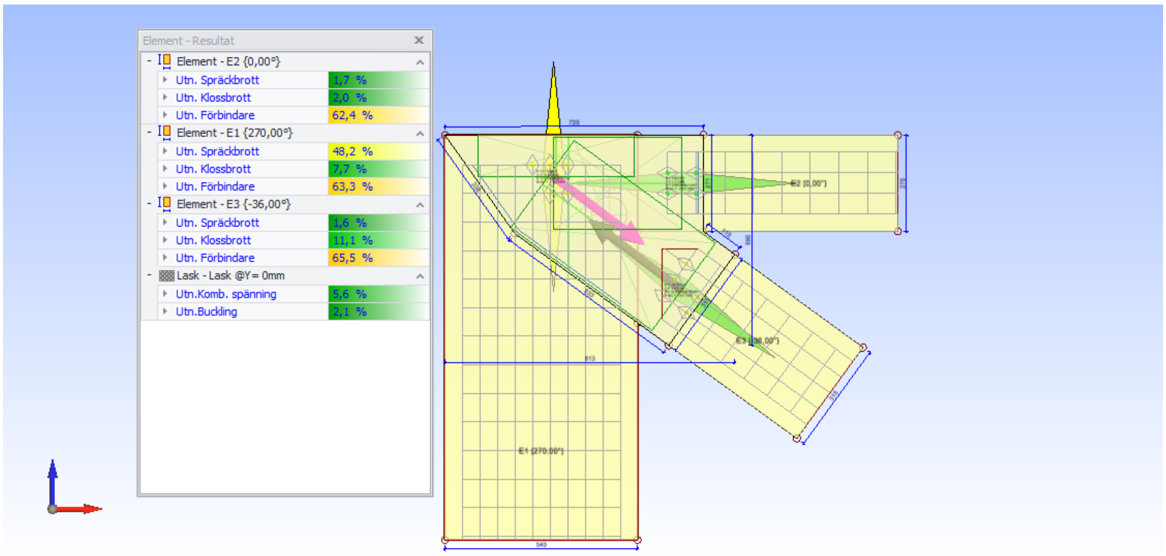


Connection J

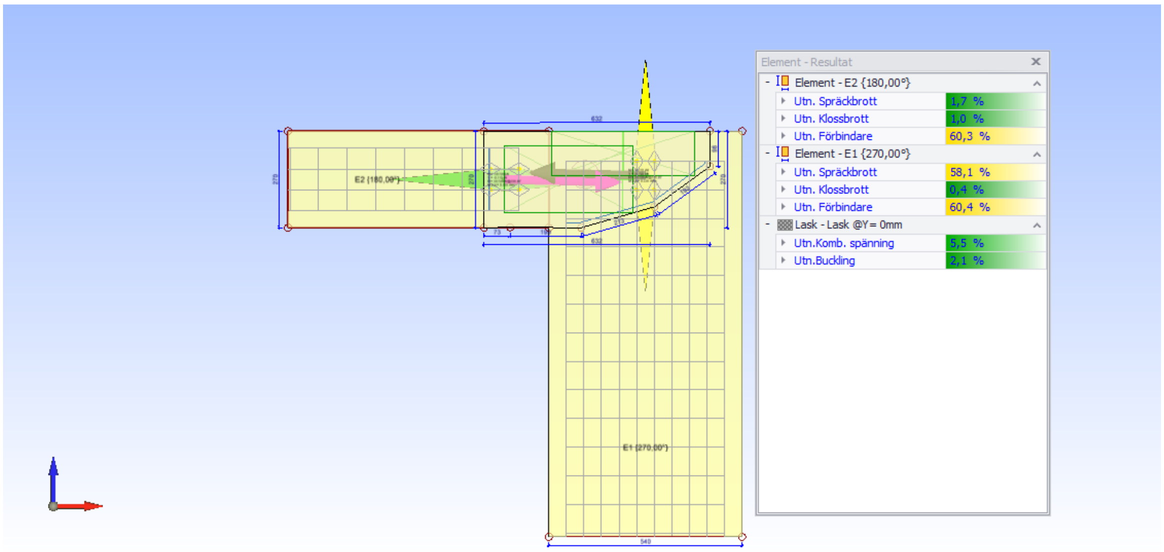


Appendix B

Connection K

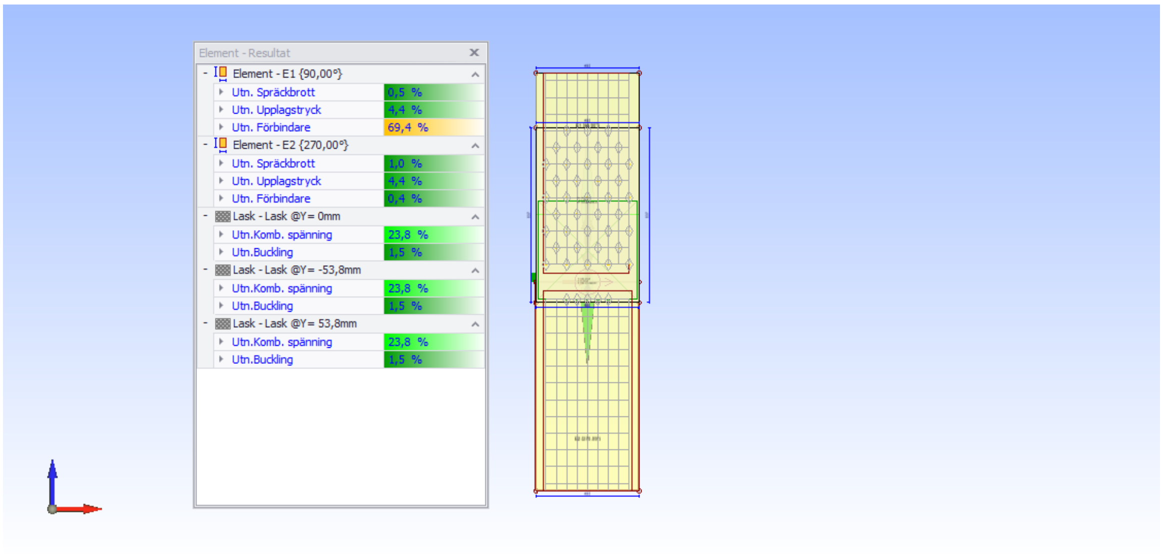


Connection L

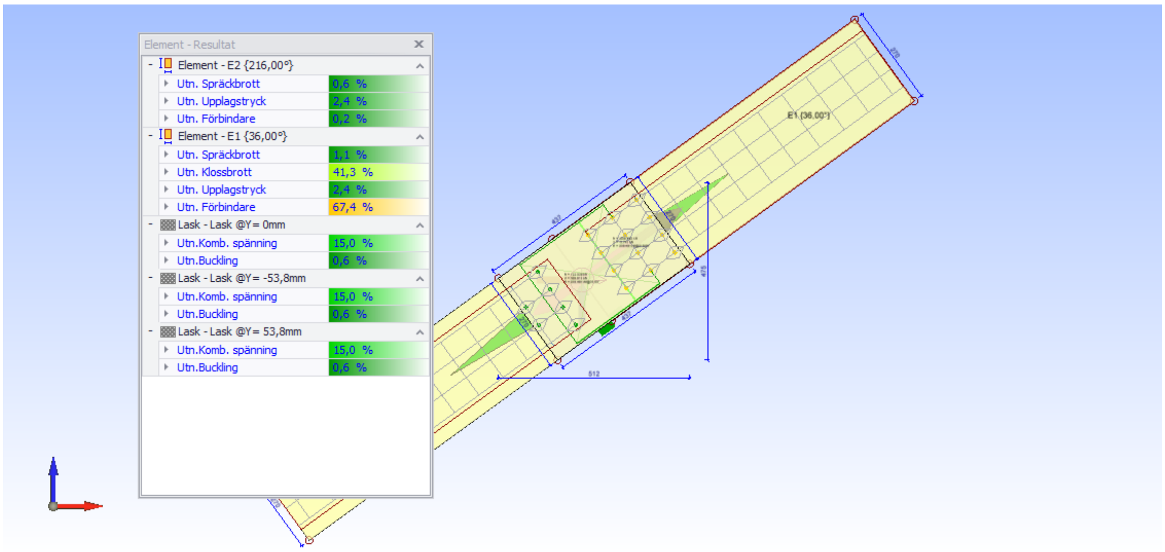


Truss 3

Connection A, vertical

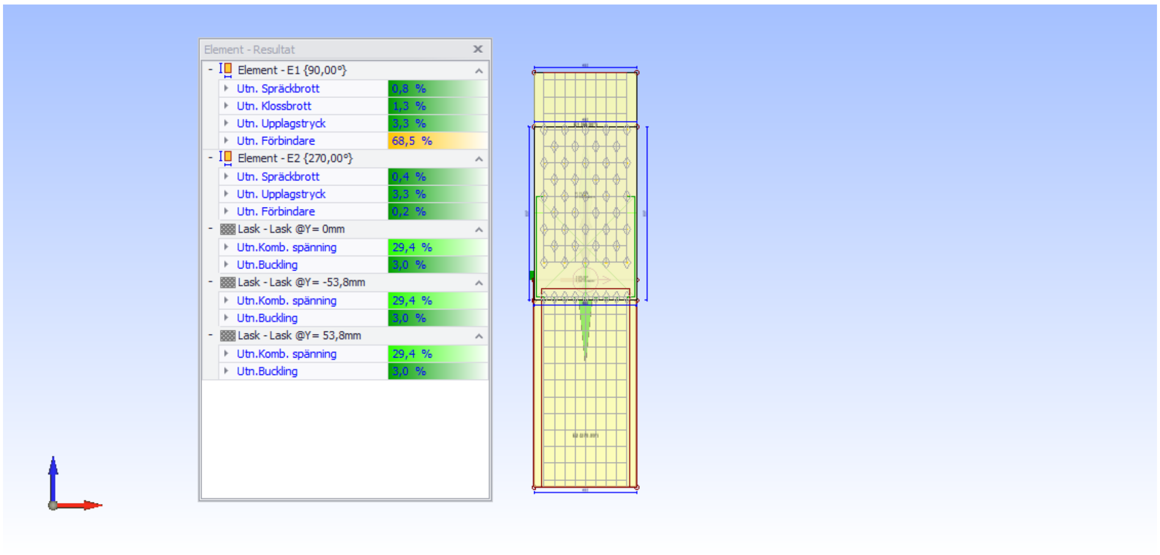


Connection A, diagonal

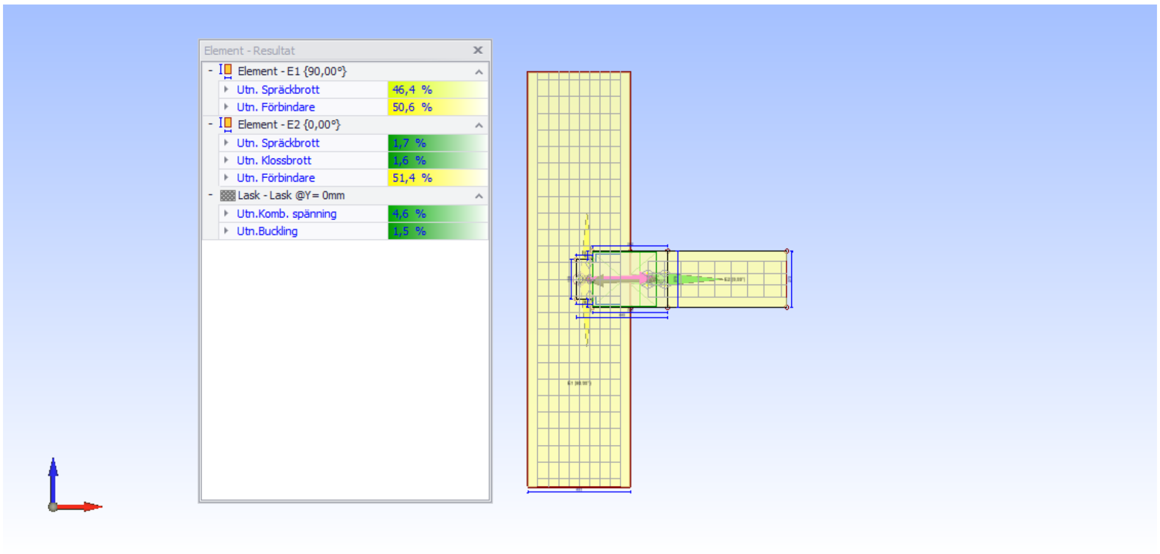


Appendix B

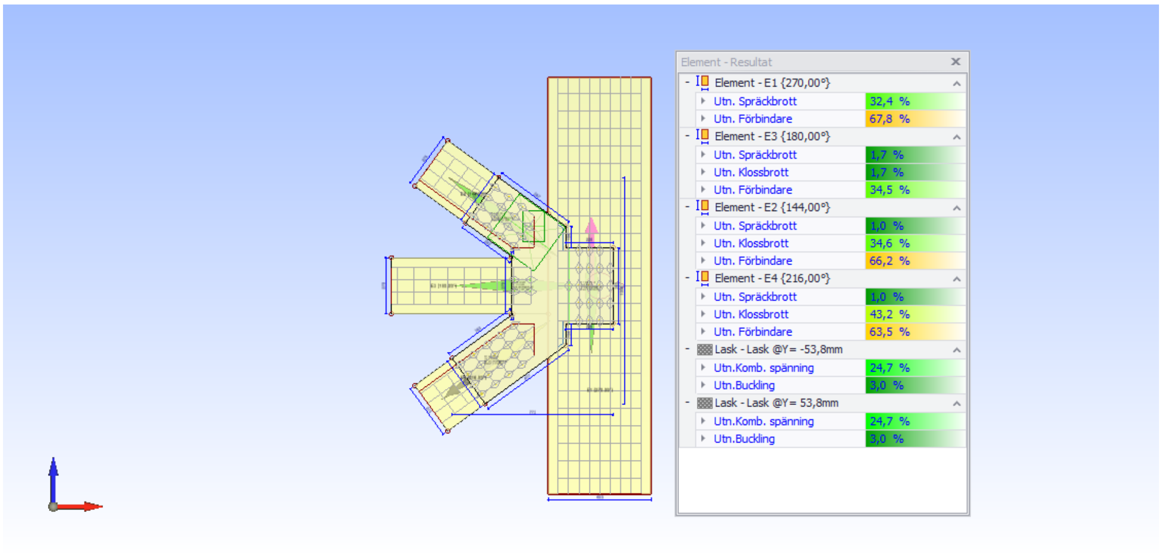
Connection B



Connection C

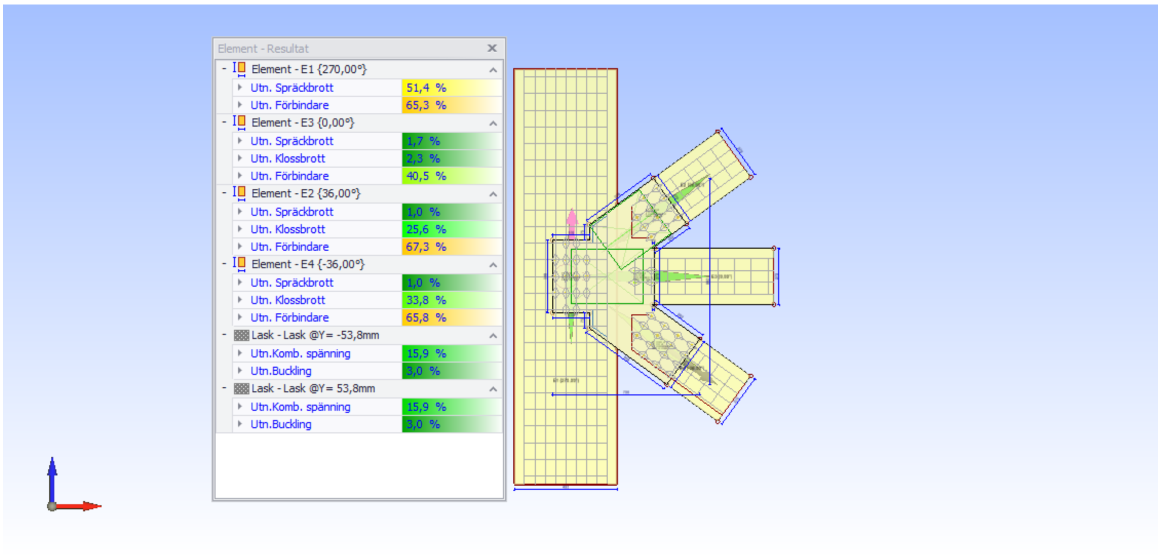


Connection D

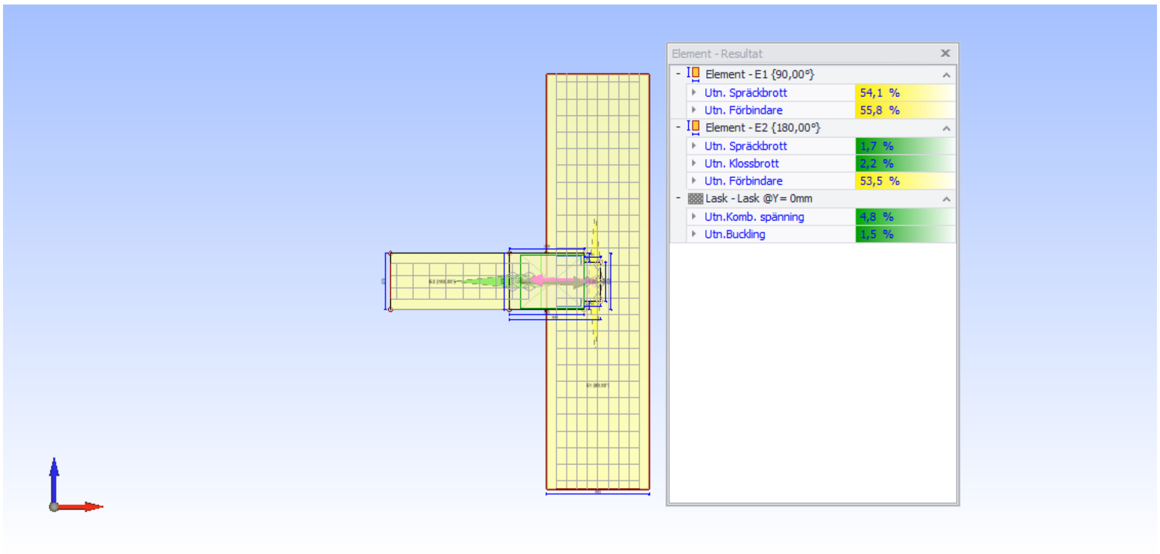


Appendix B

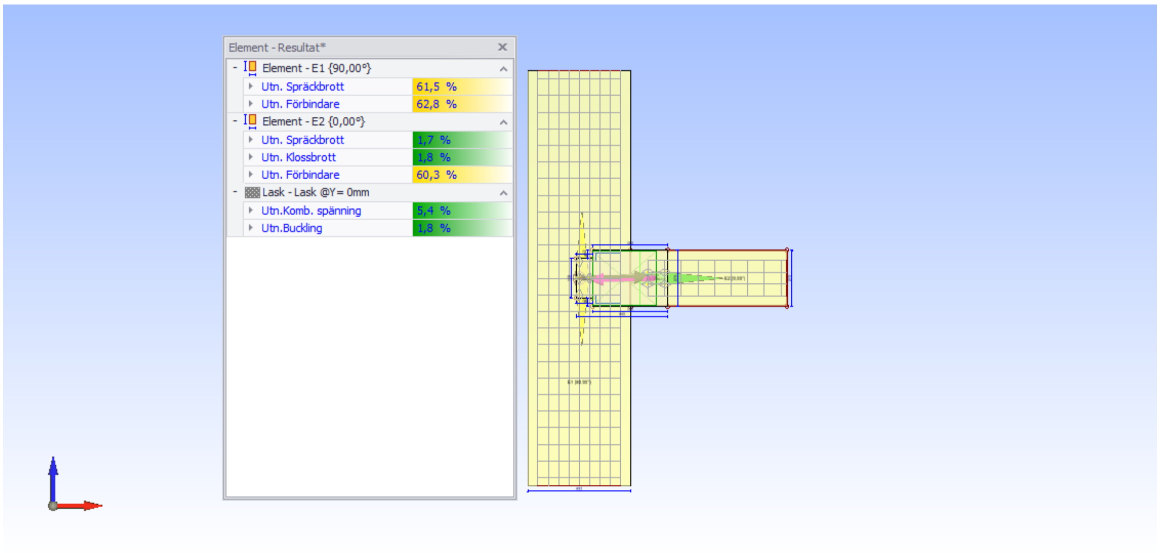
Connection E



Connection F

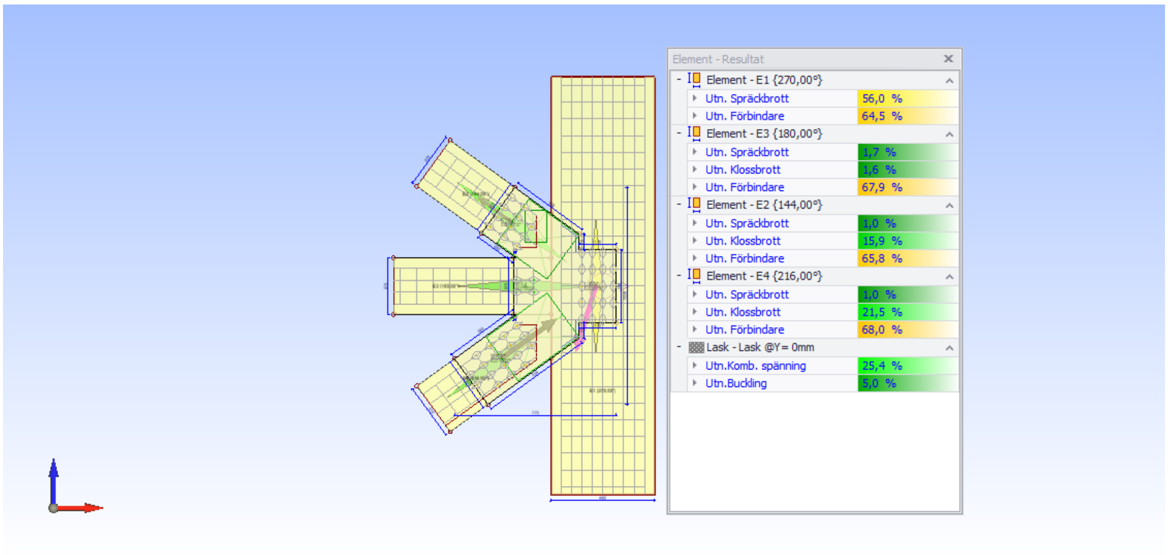


Connection G

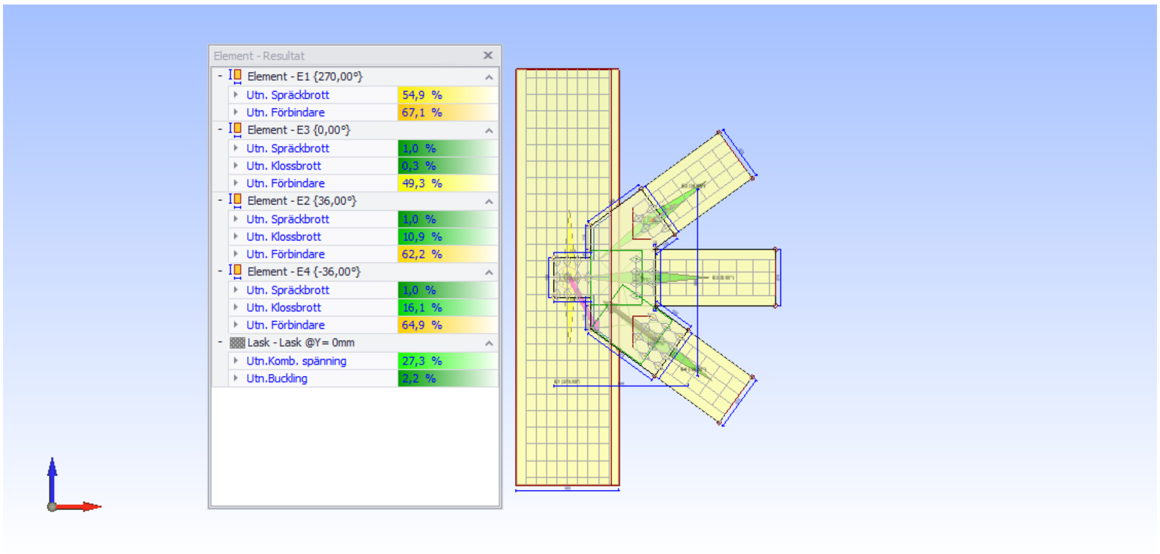


Appendix B

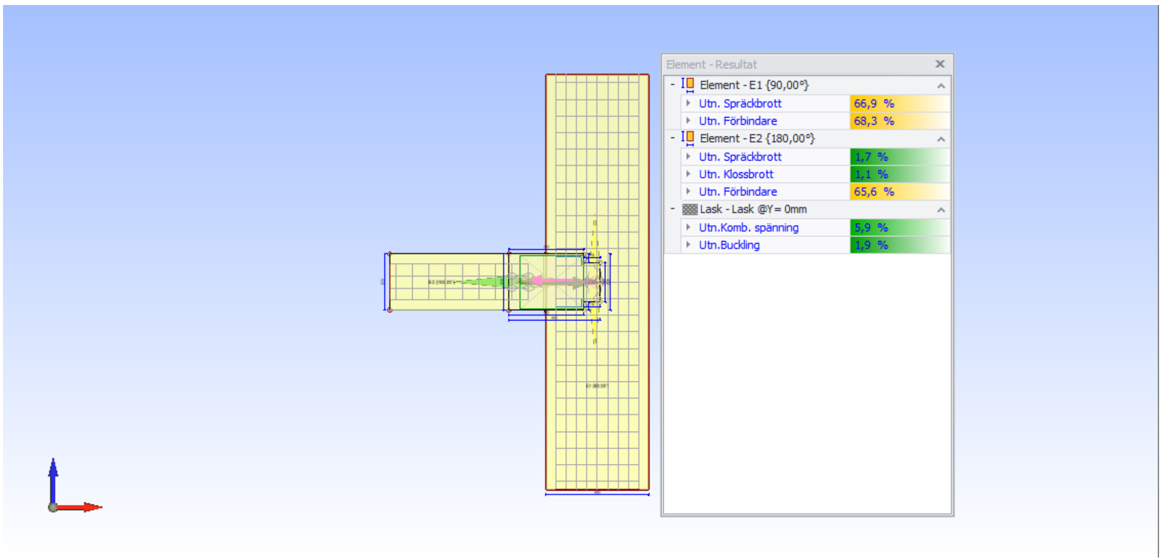
Connection H



Connection I

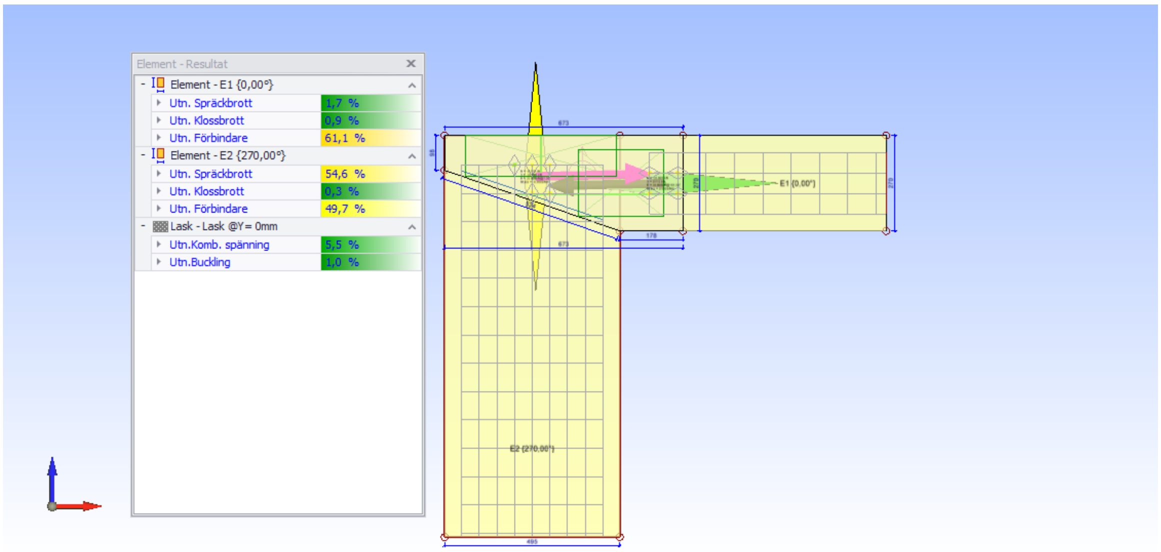


Connection J

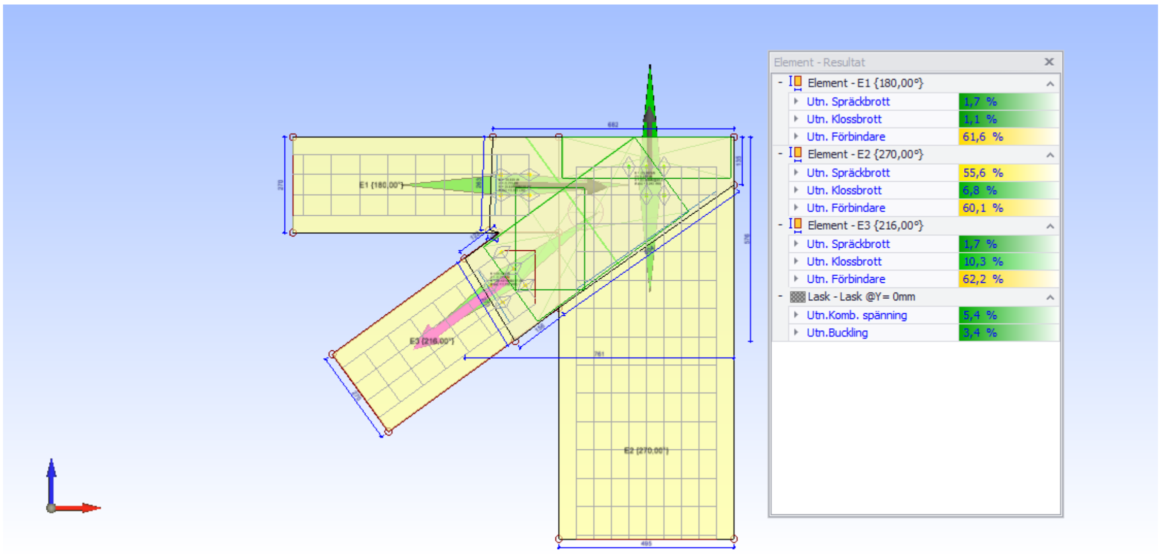


Appendix B

Connection K

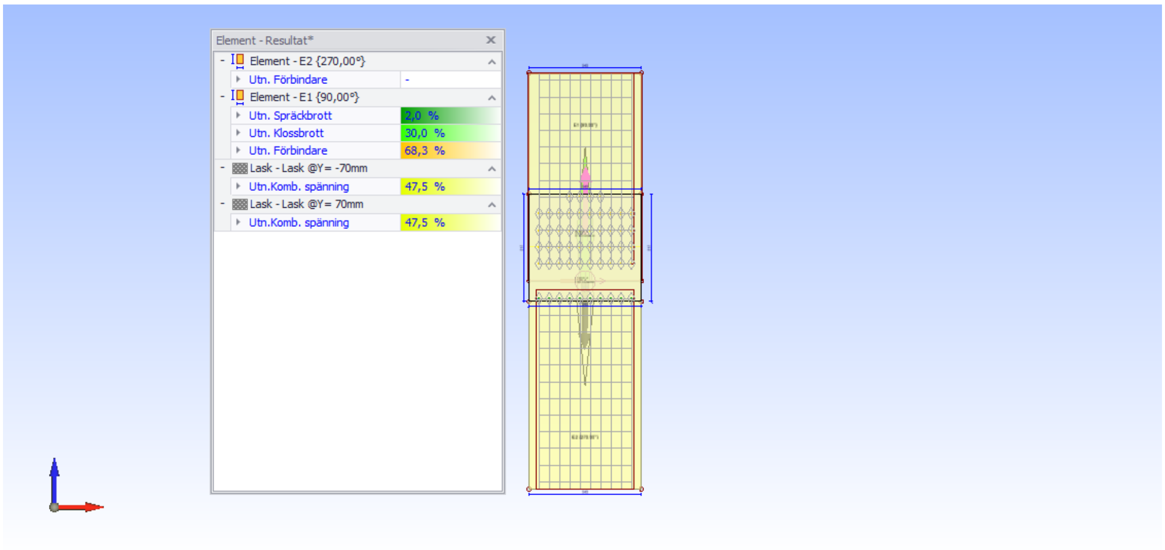


Connection L

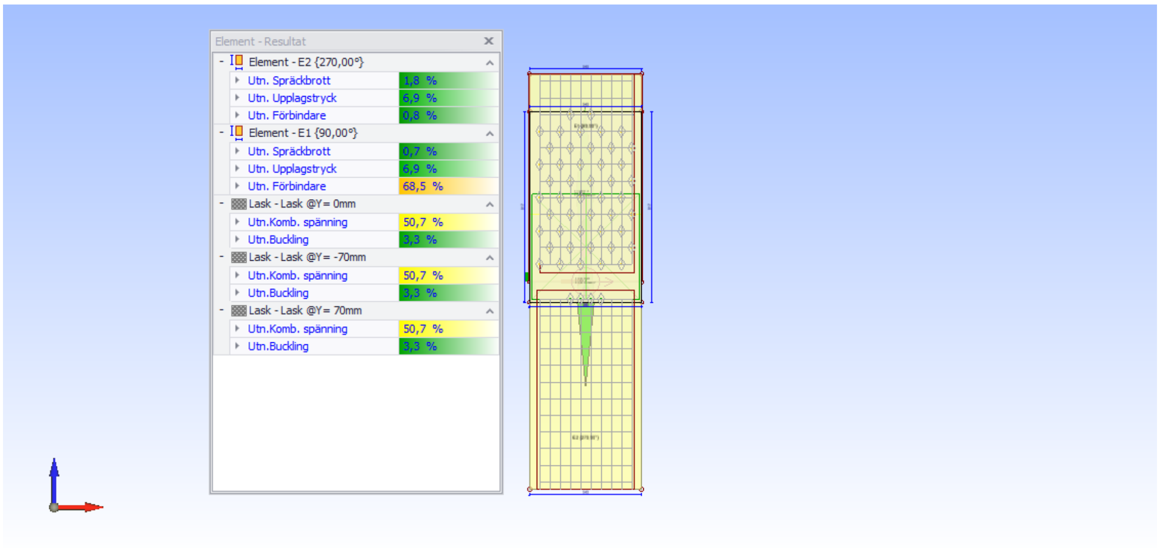


Truss 4

Connection A

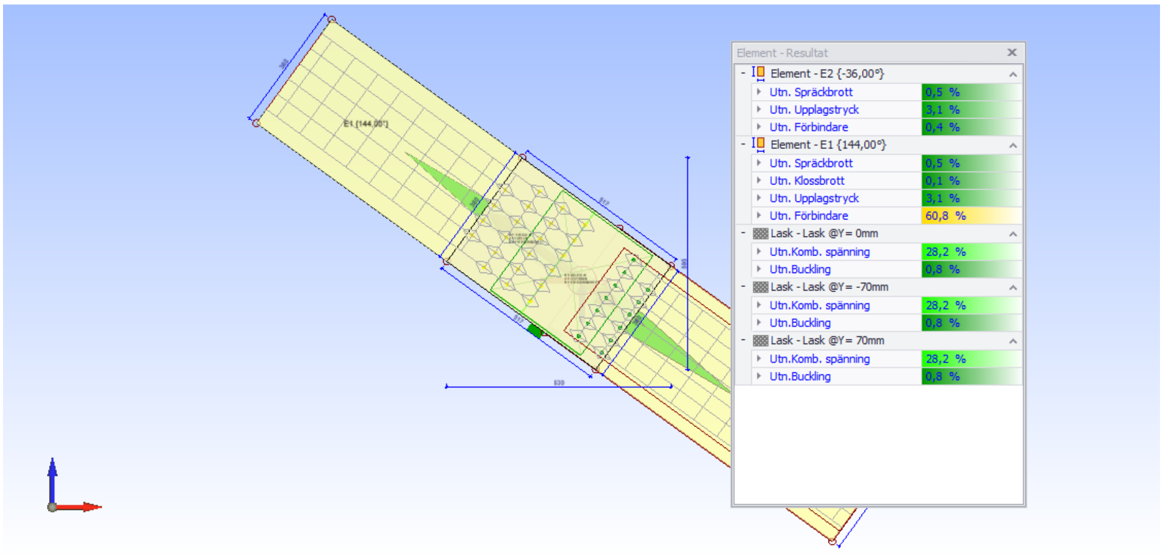


Connection B, vertical

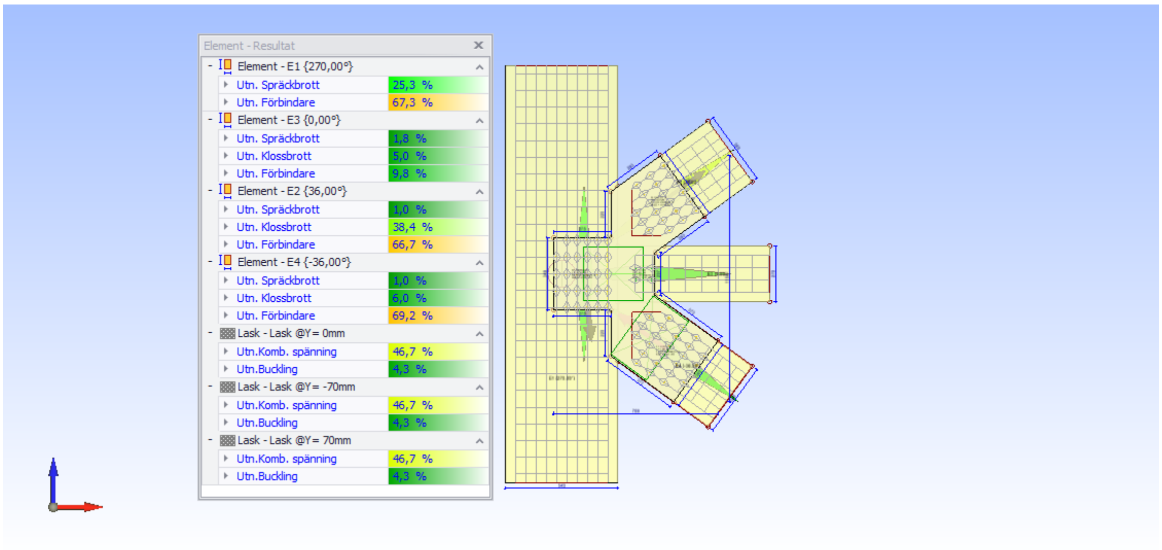


Appendix B

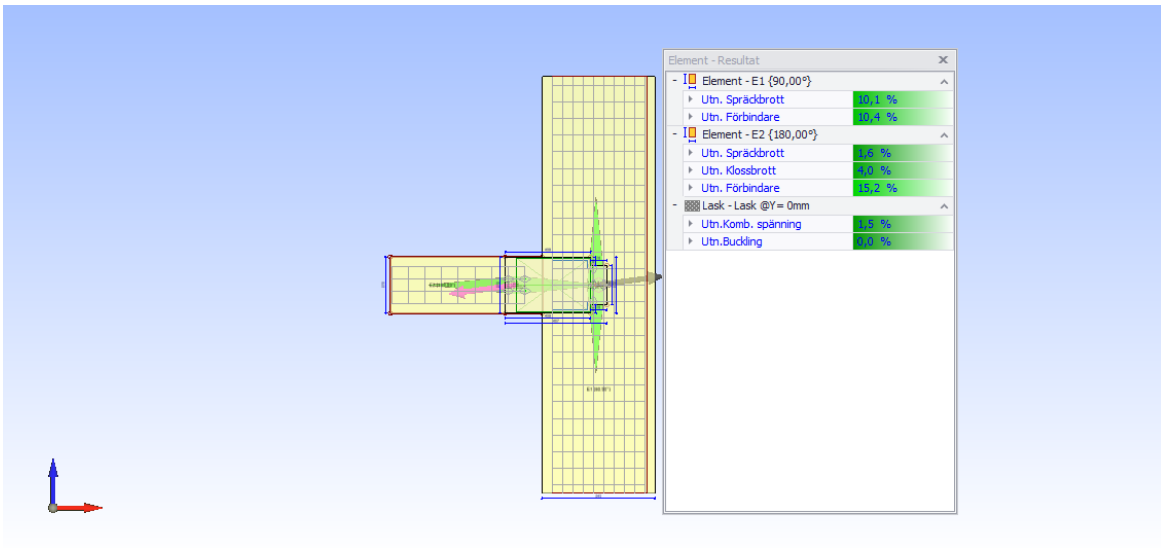
Connection B, diagonal



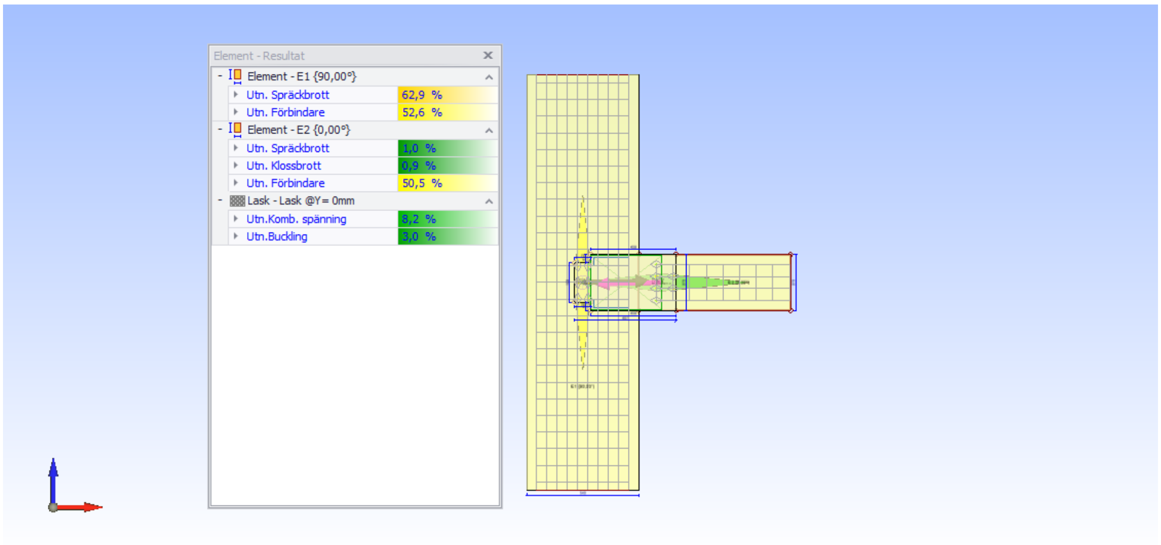
Connection C



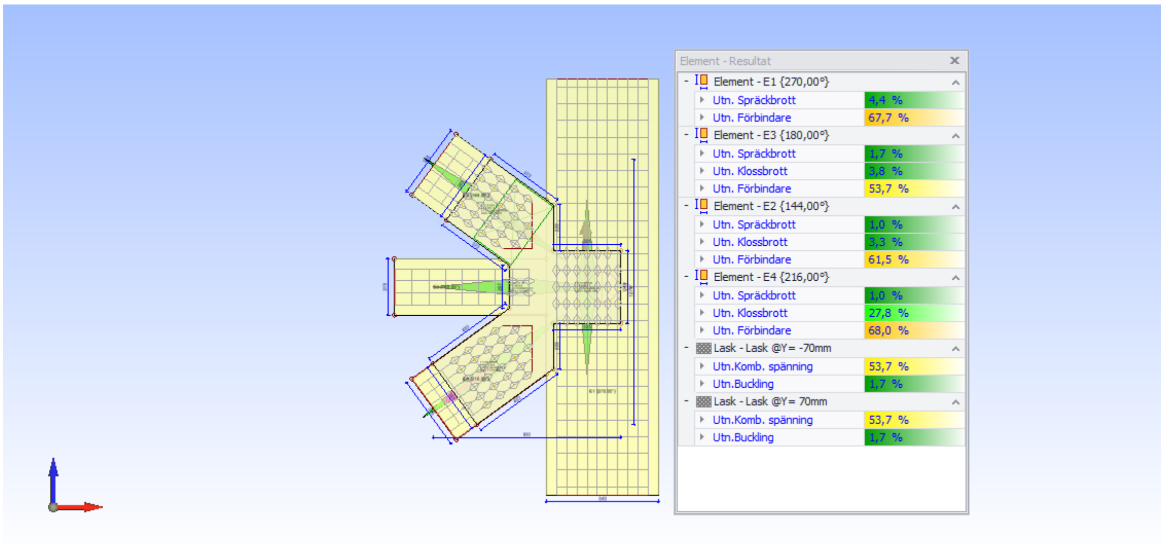
Connection D



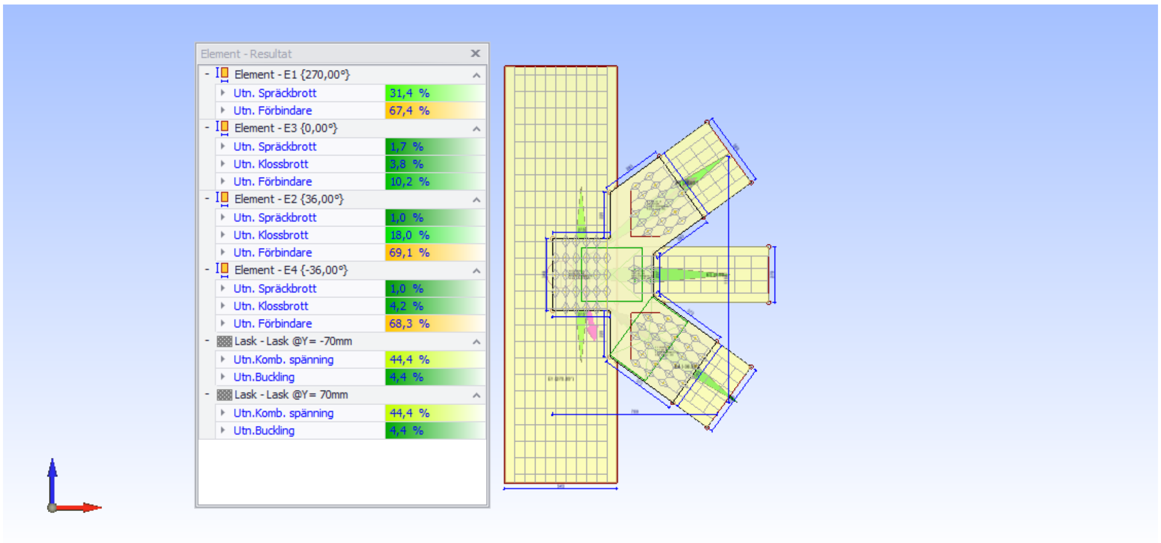
Connection E



Connection F

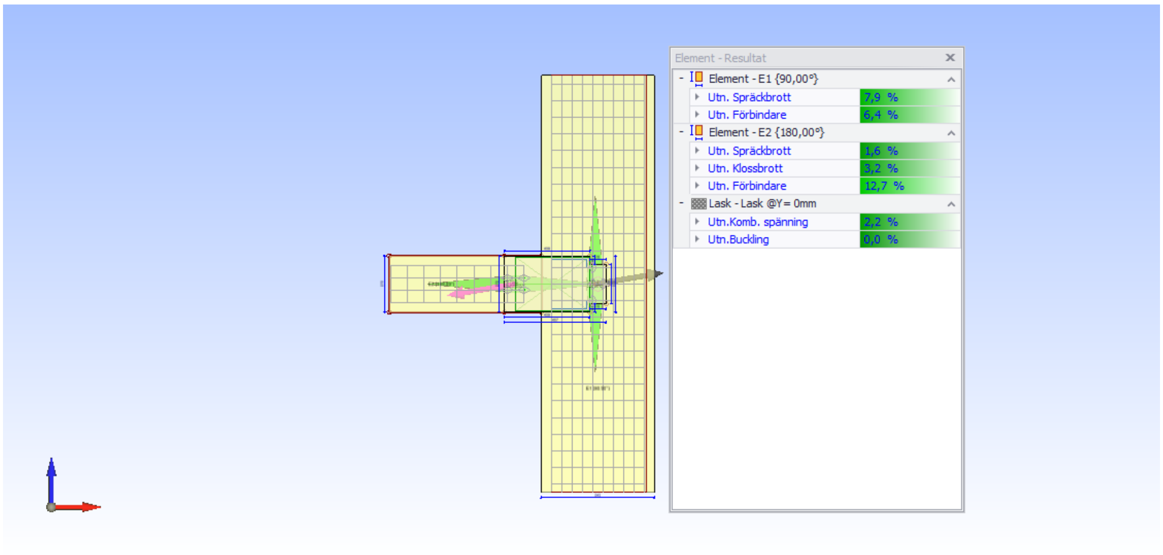


Connection G

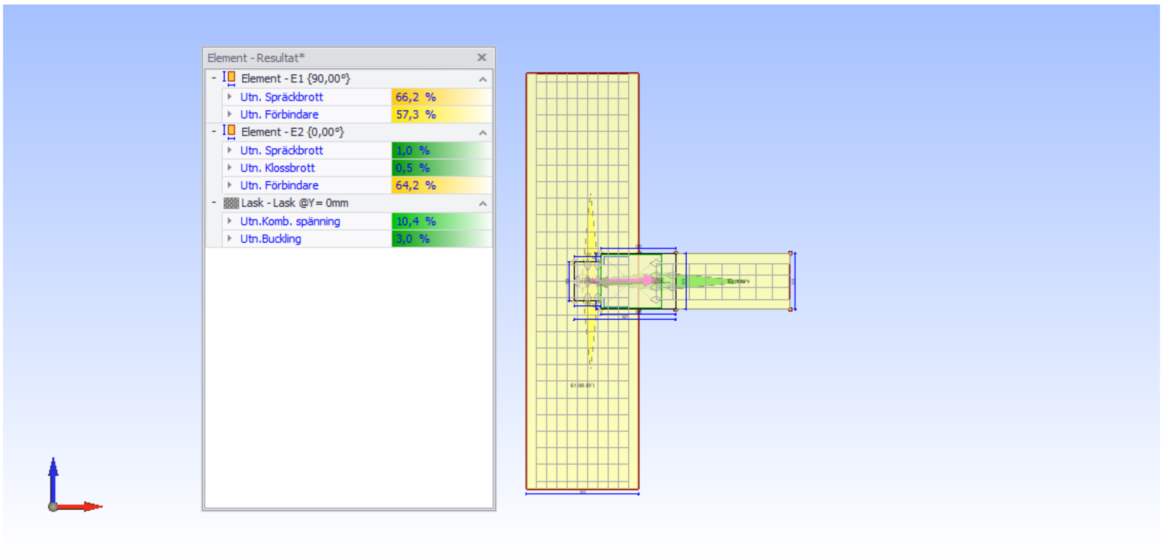


Appendix B

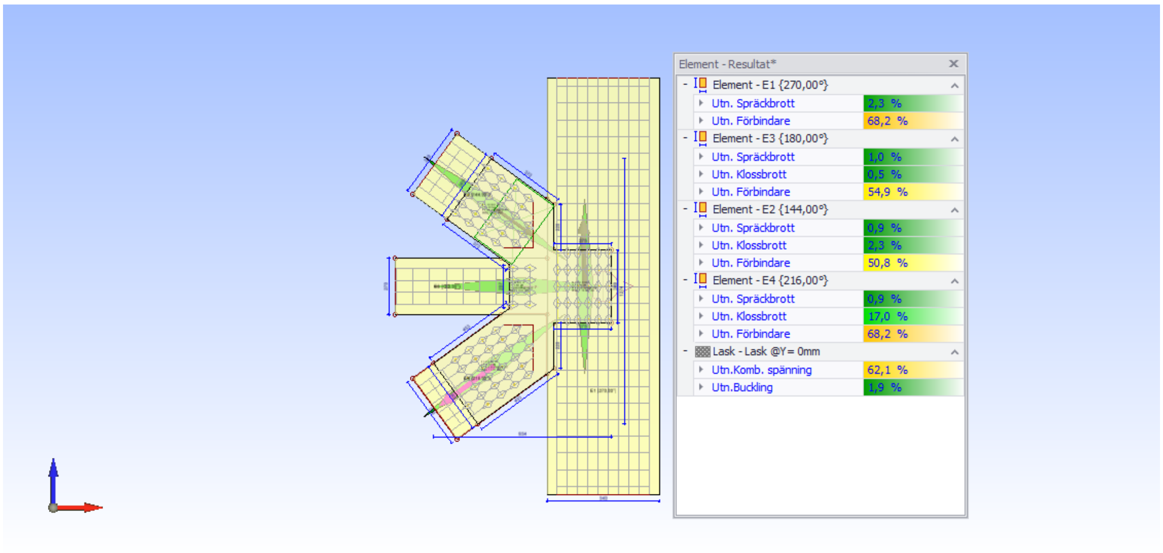
Connection H



Connection I

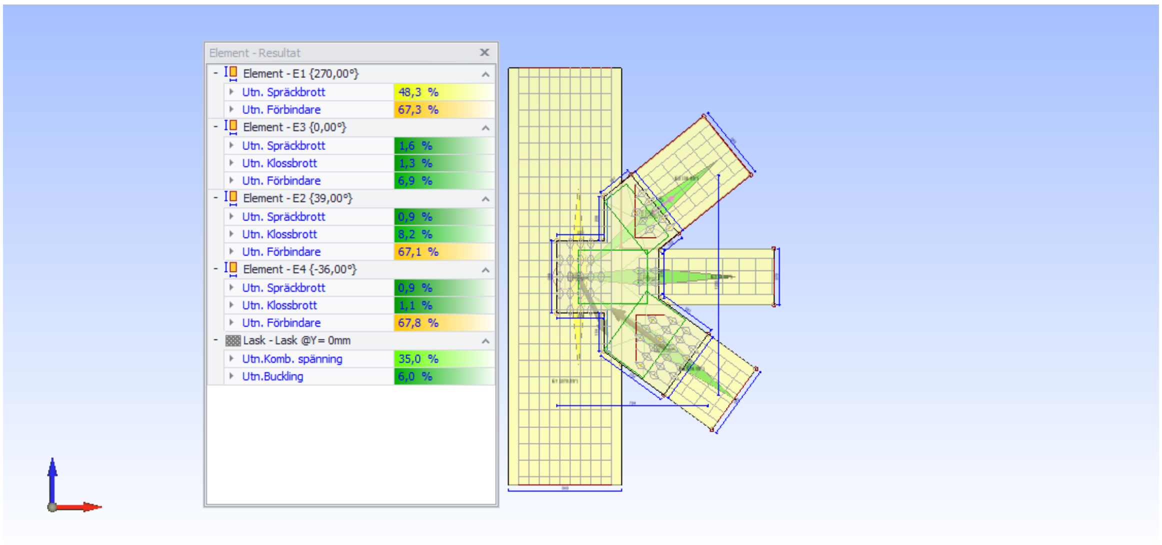


Connection J

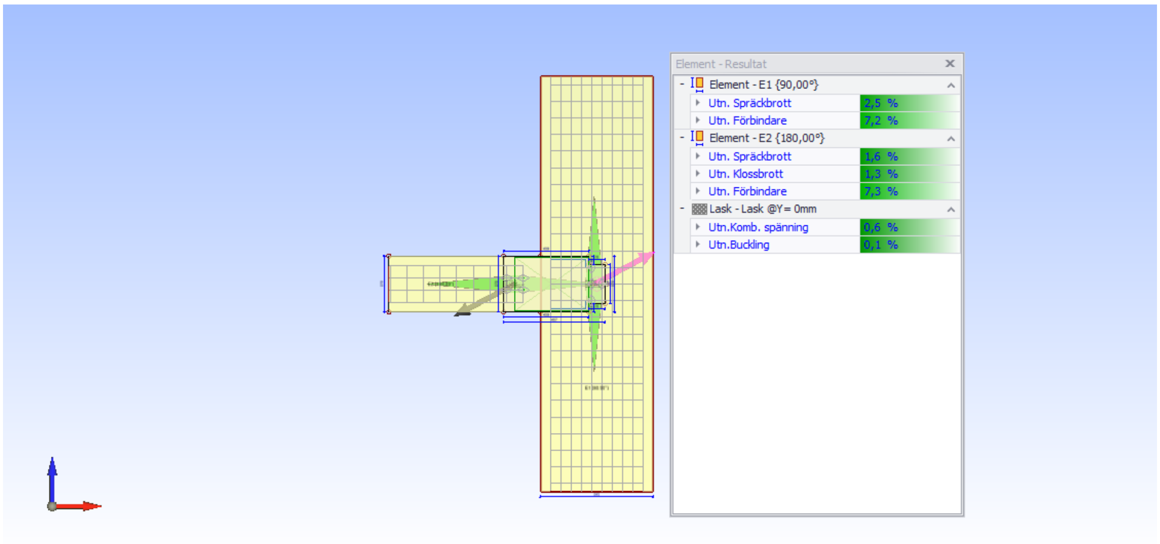


Appendix B

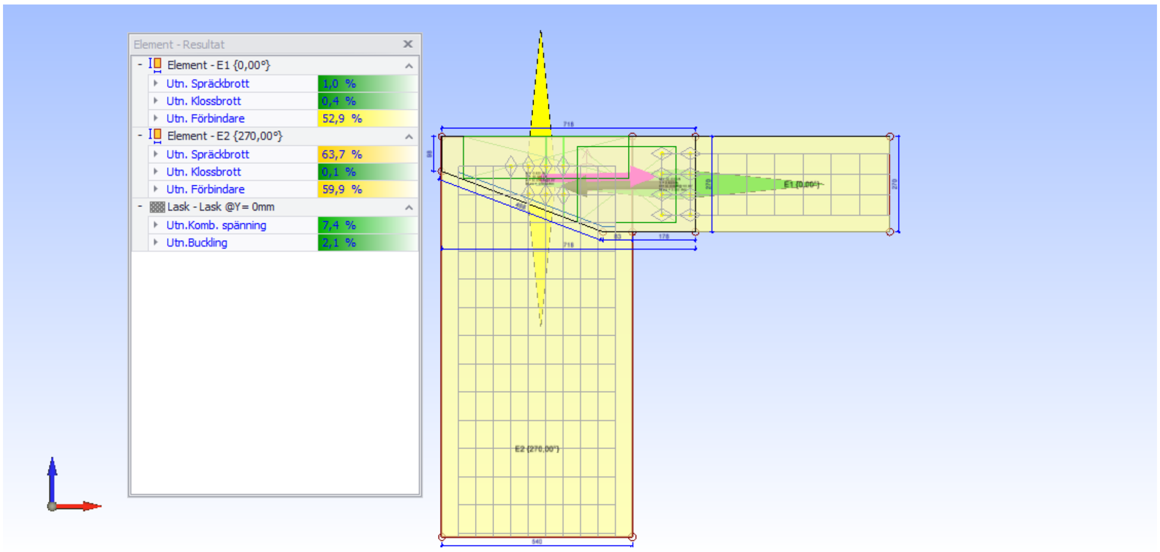
Connection K



Connection L

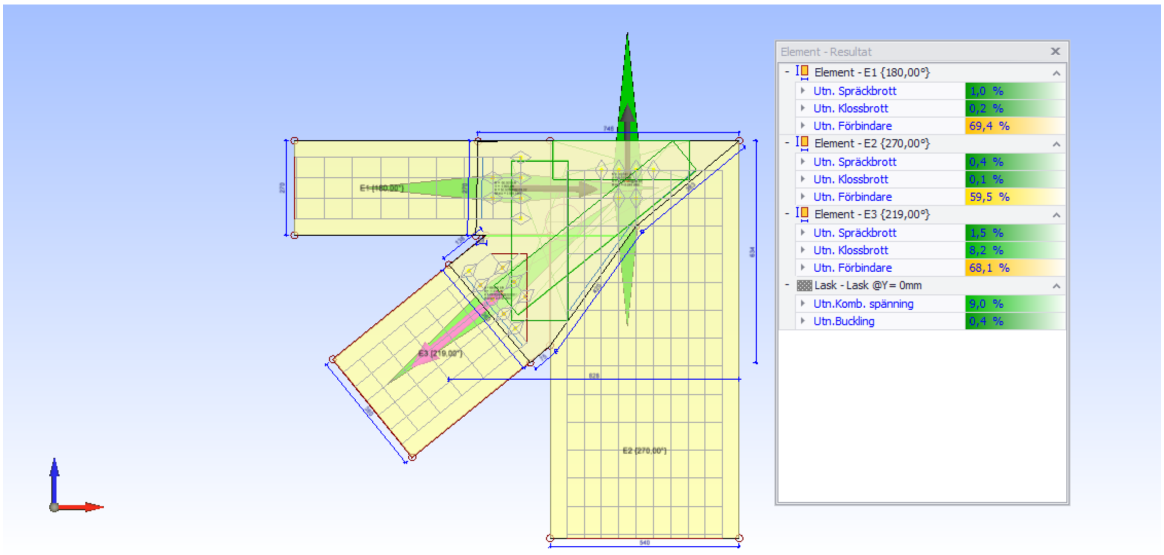


Connection M



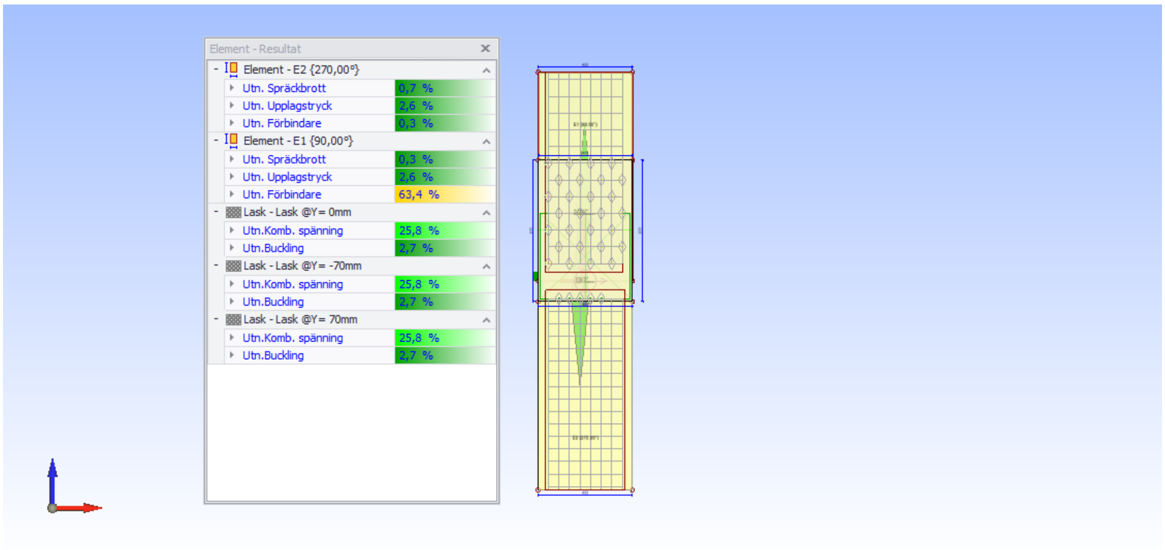
Appendix B

Connection N

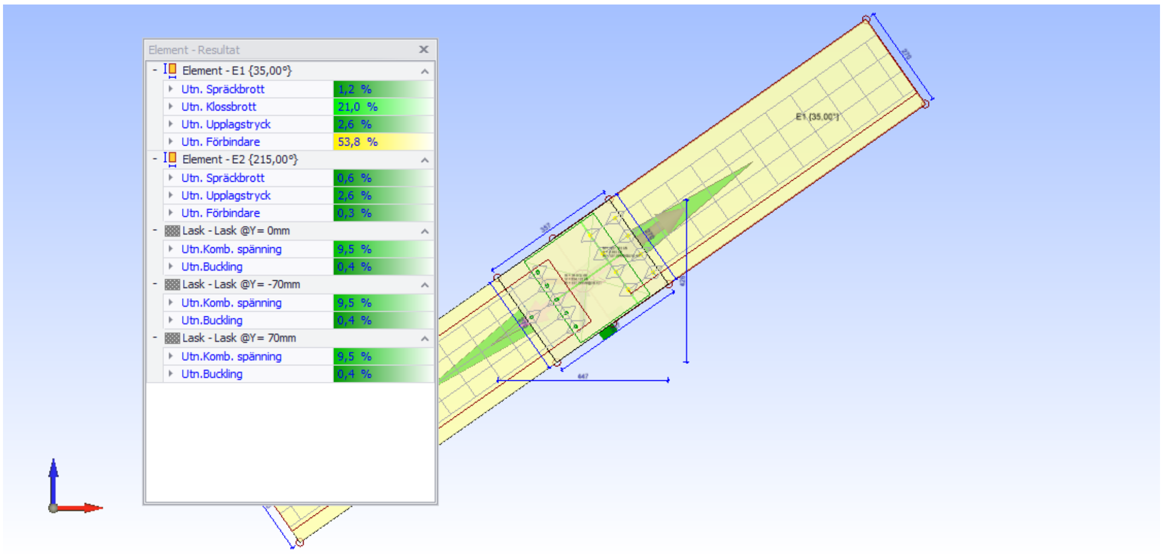


Truss 5

Connection A, vertical

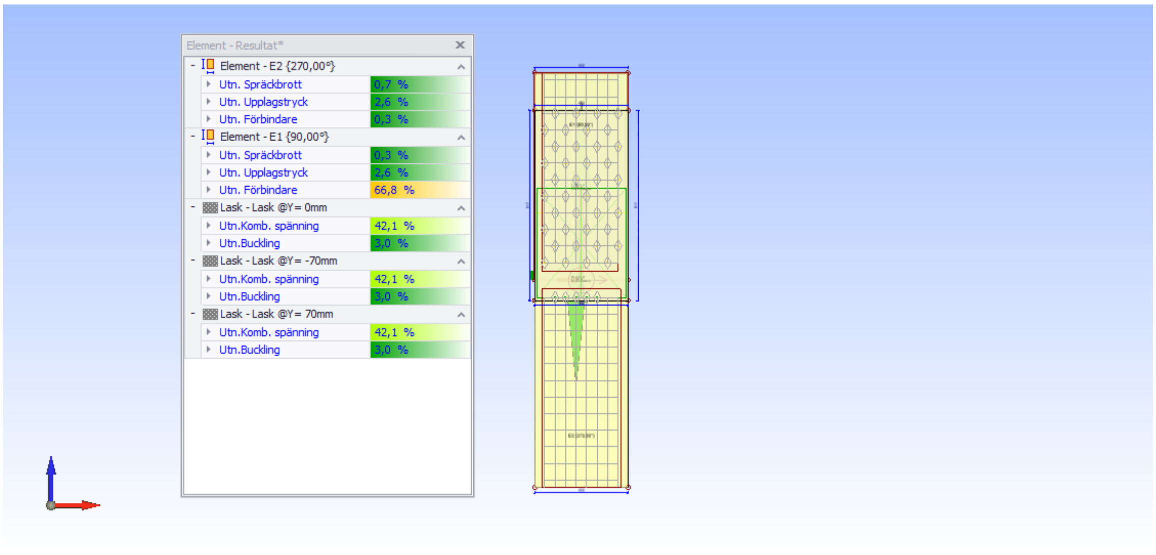


Connection A, diagonal

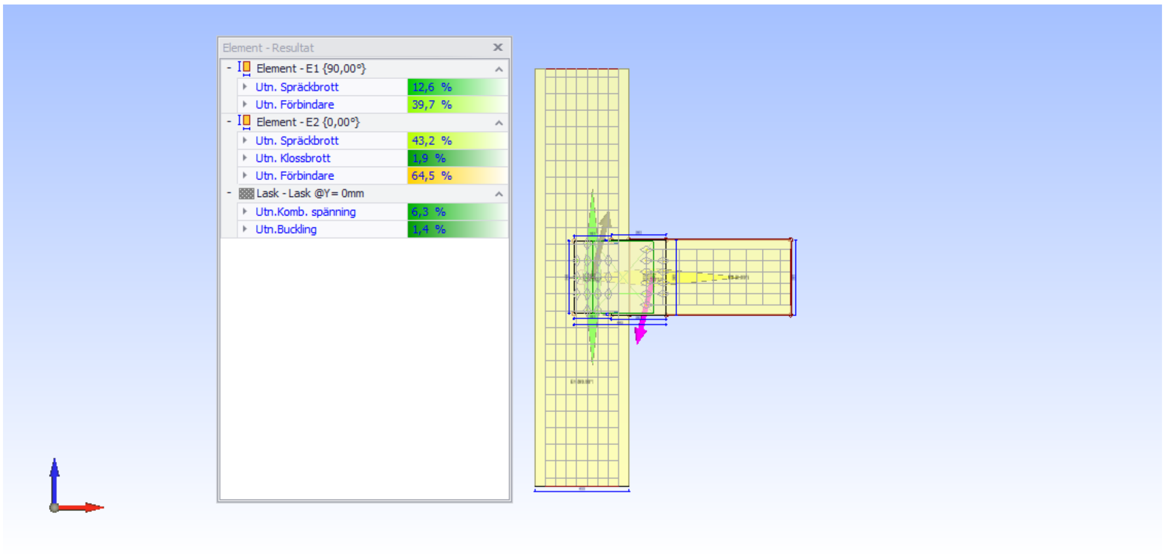


Appendix B

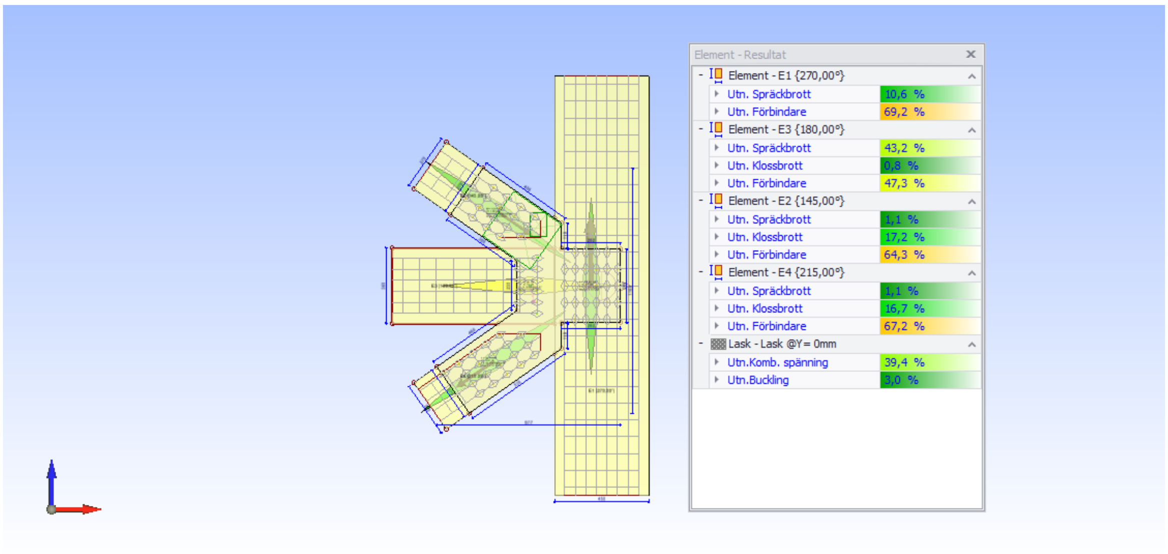
Connection B



Connection C

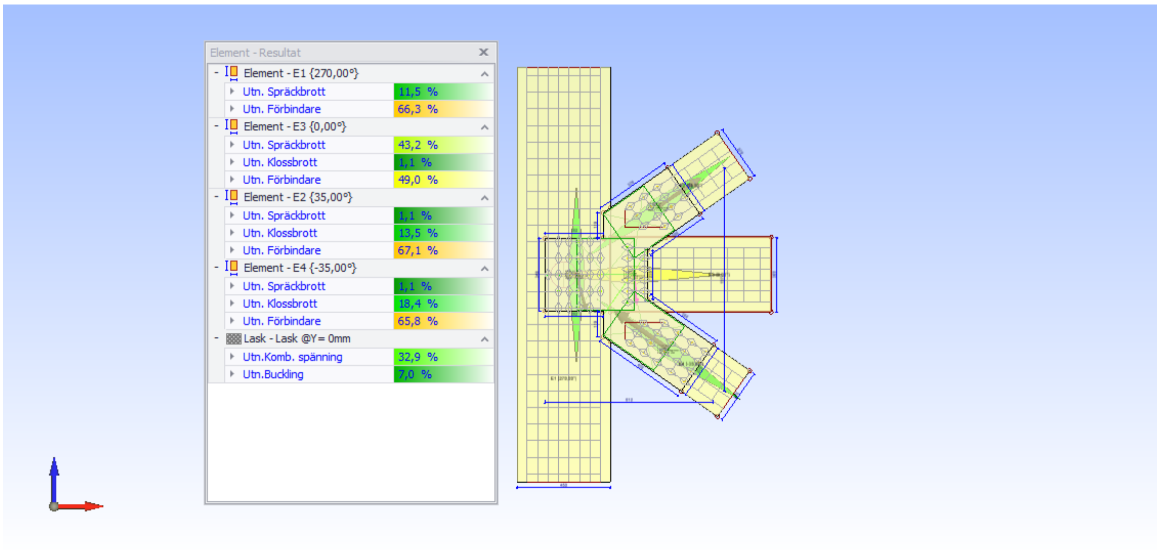


Connection D

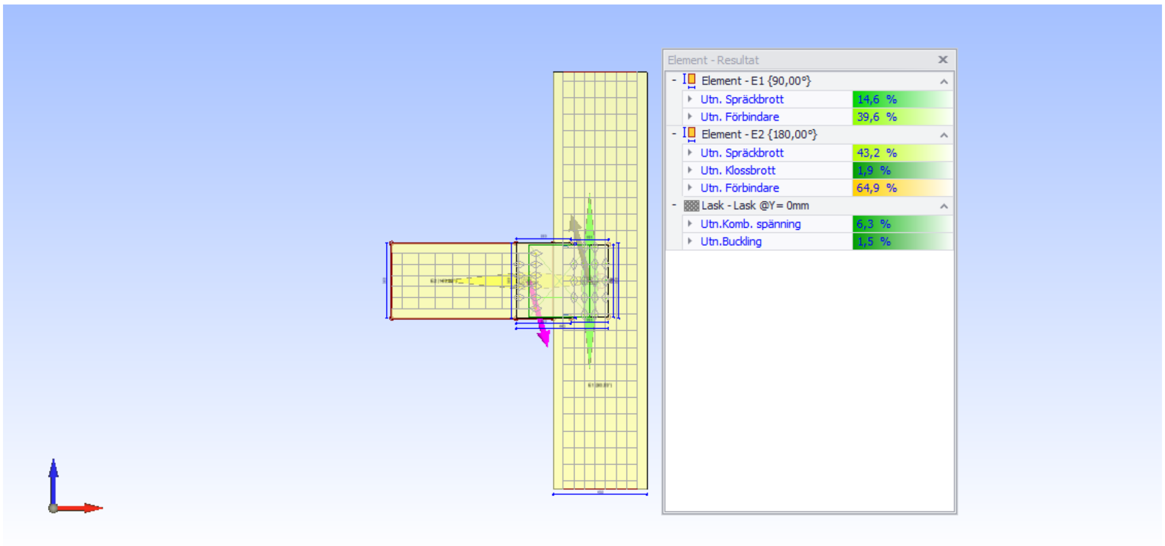


Appendix B

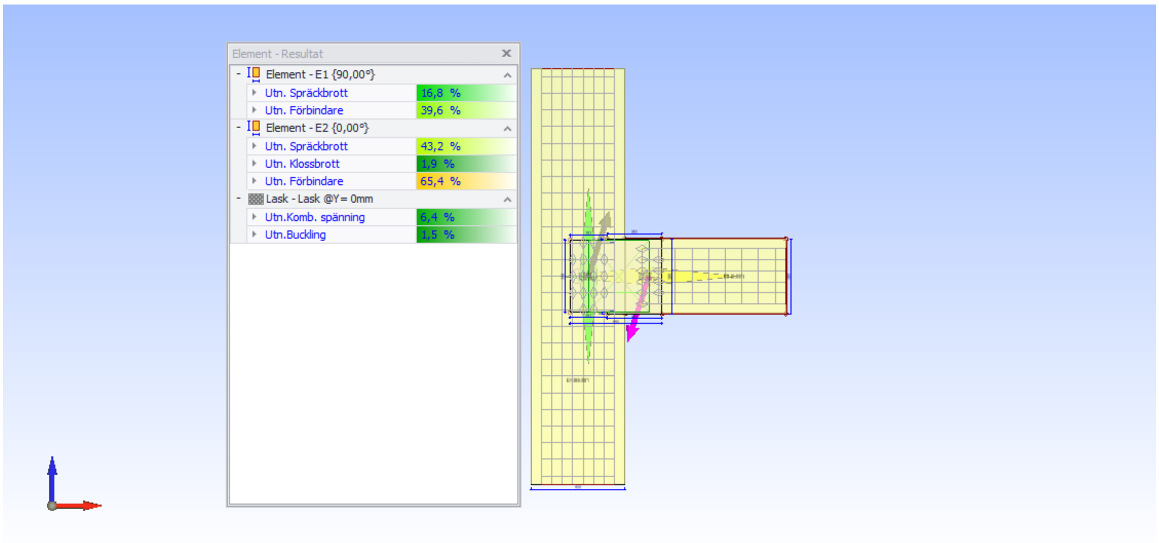
Connection E



Connection F

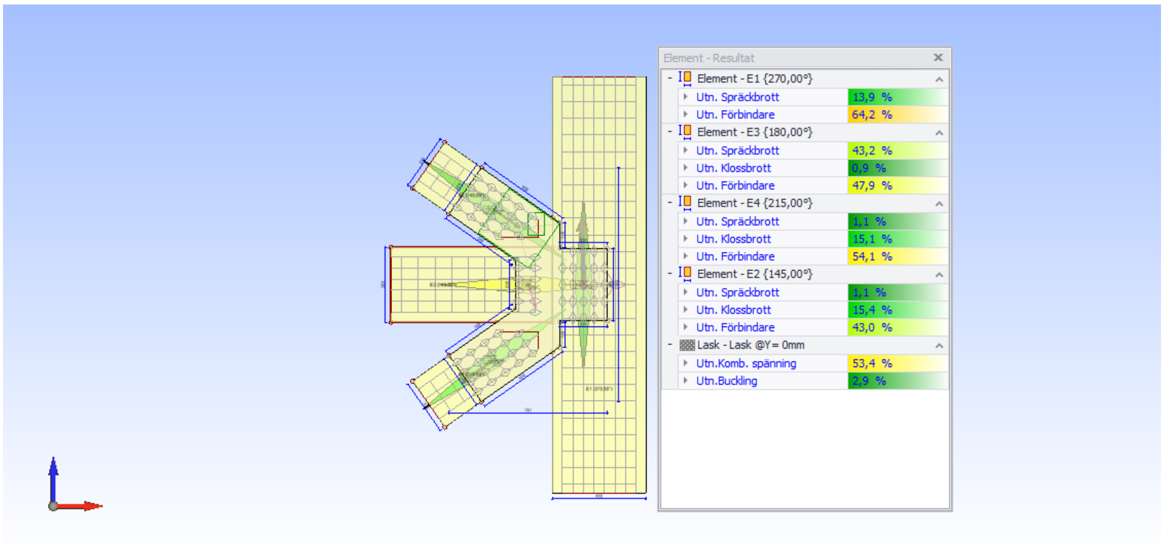


Connection G

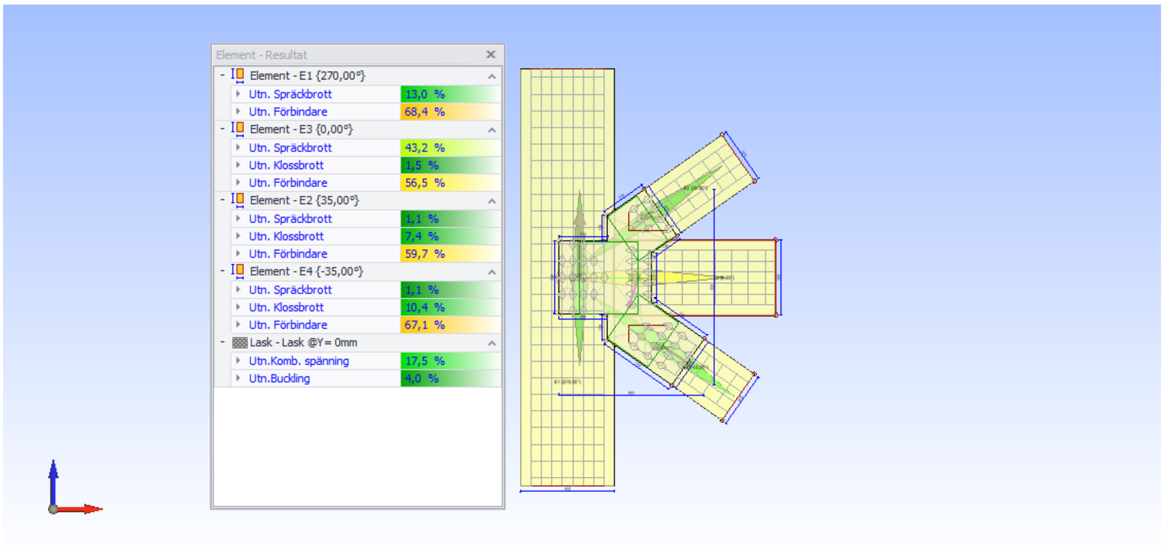


Appendix B

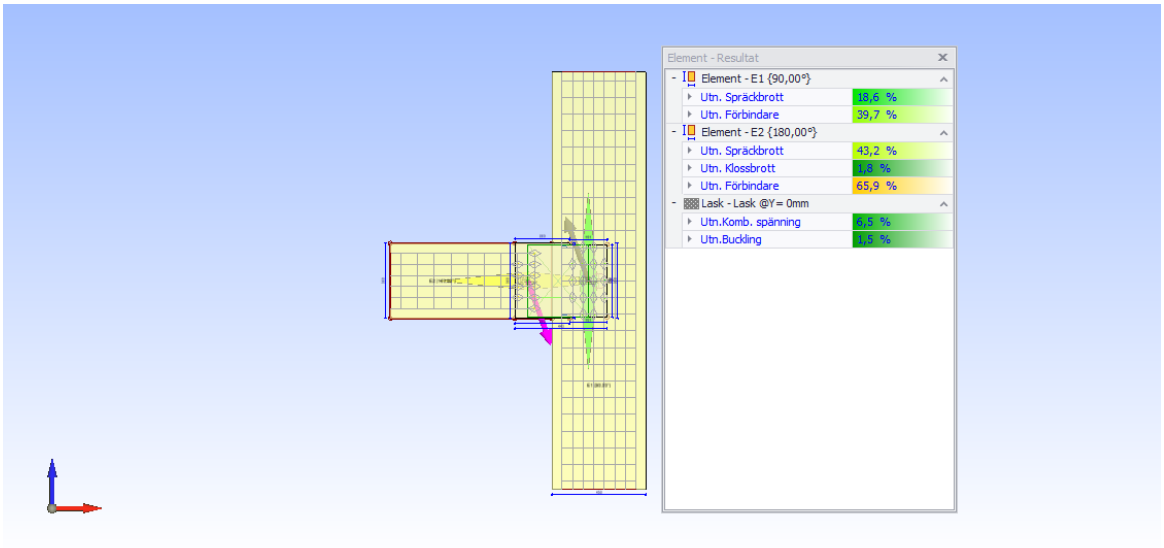
Connection H



Connection I

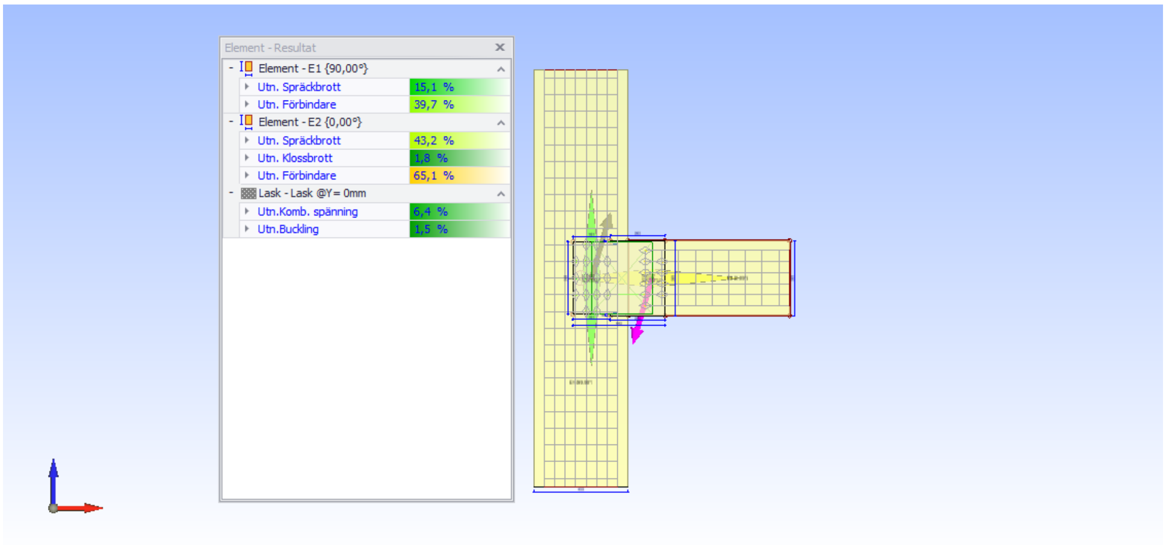


Connection J

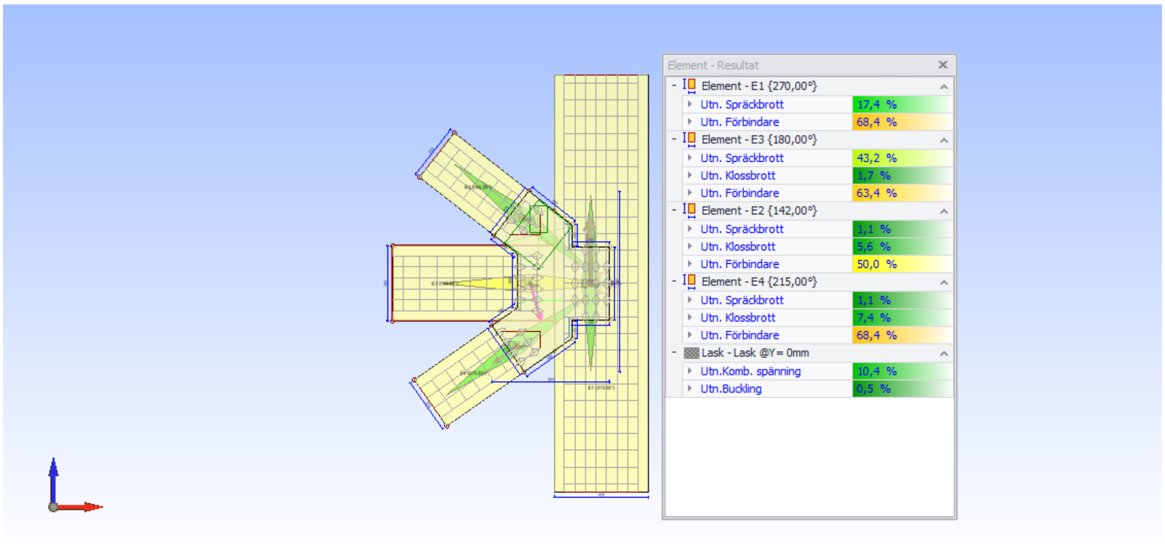


Appendix B

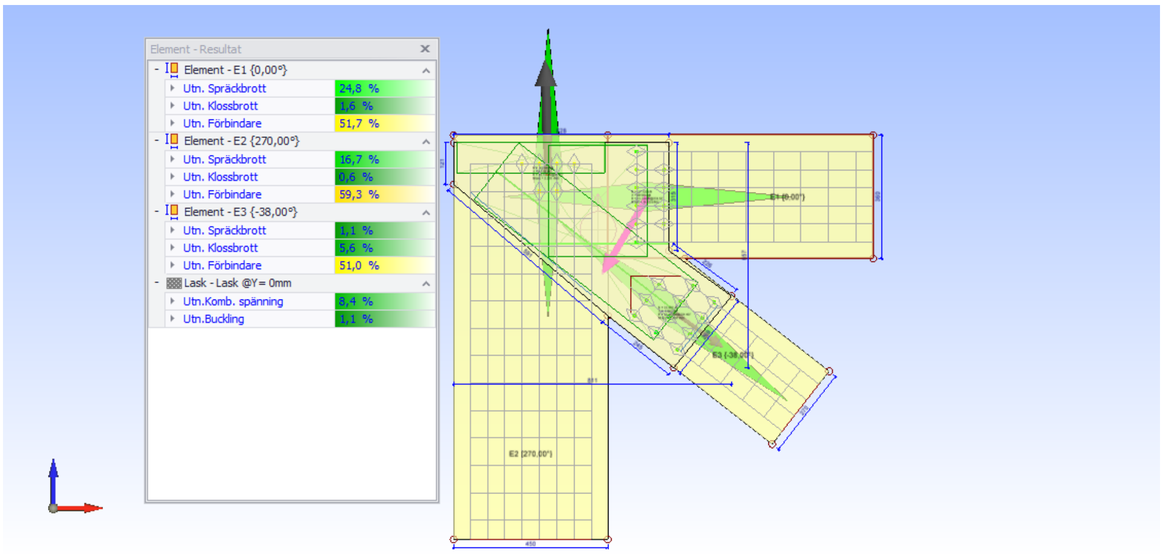
Connection K



Connection L

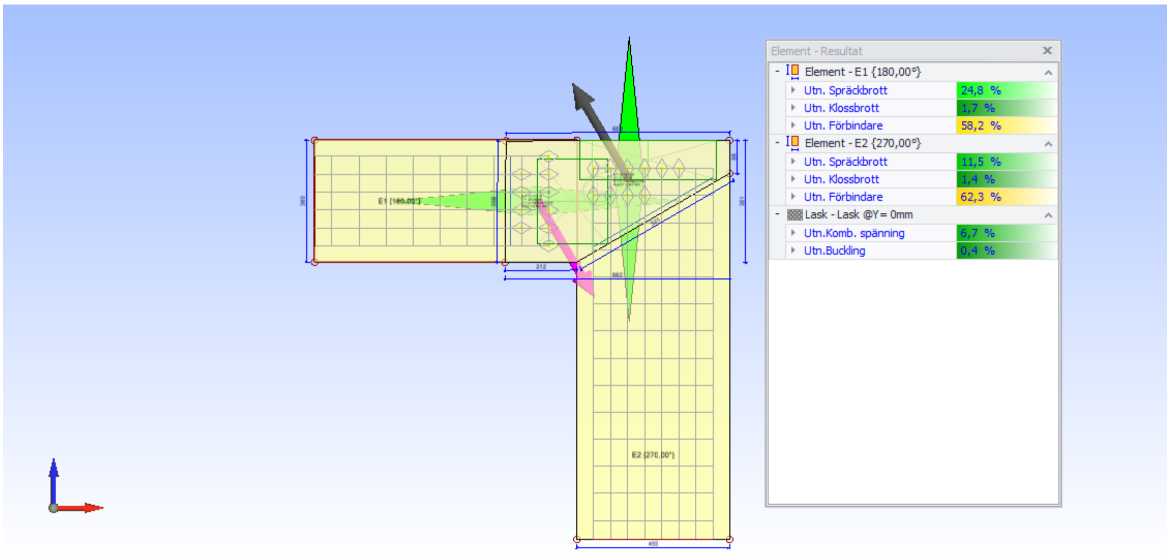


Connection M



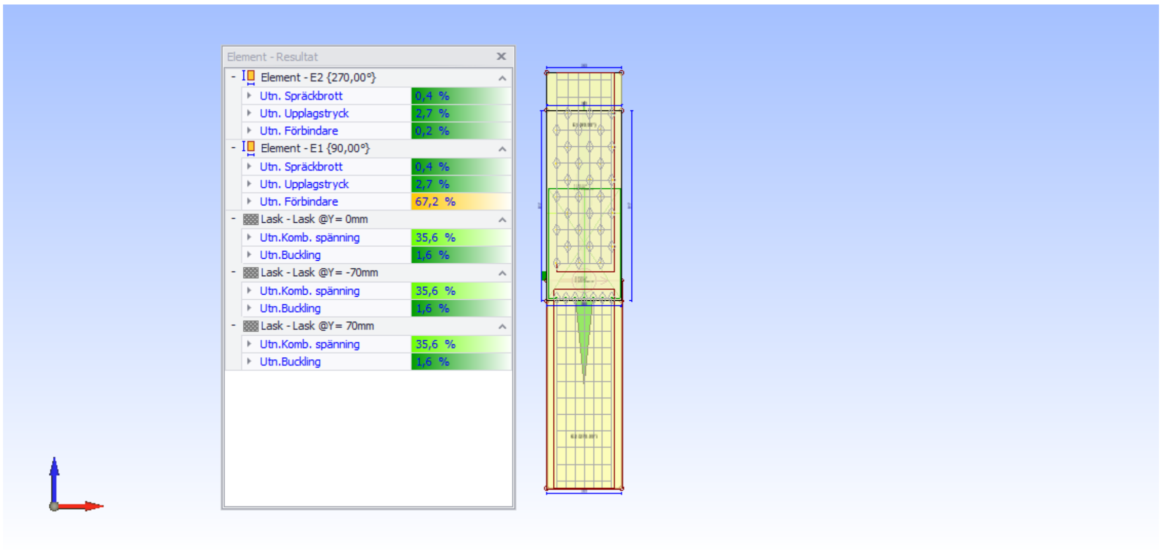
Appendix B

Connection N

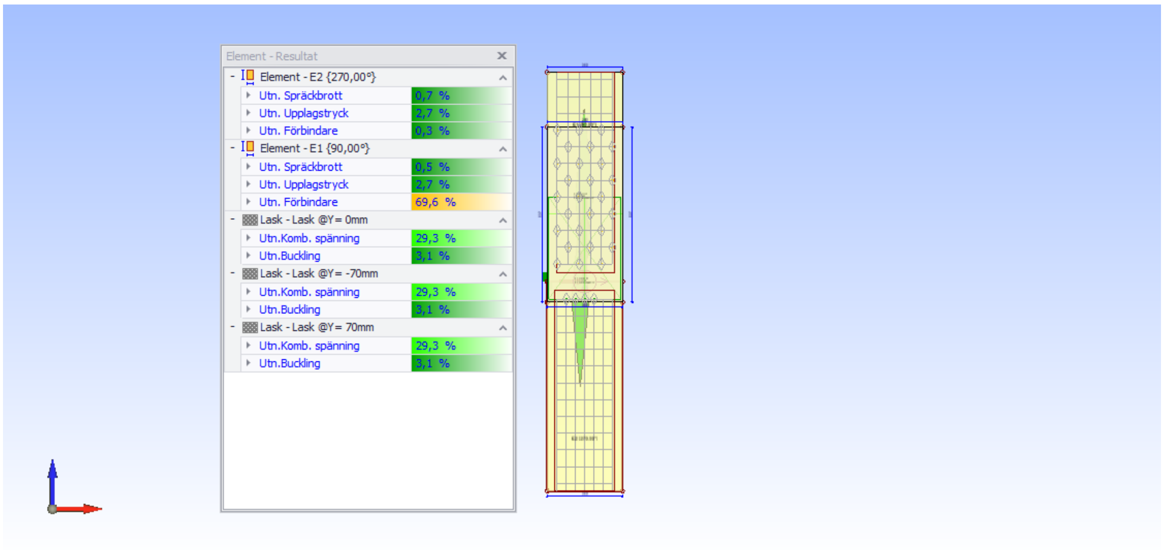


Truss 6

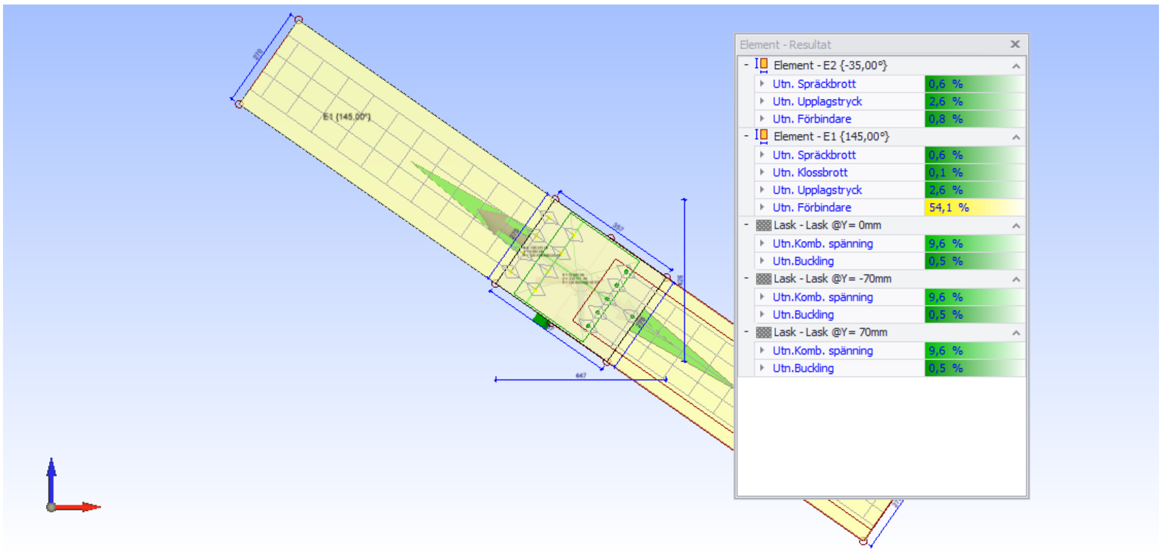
Connection A



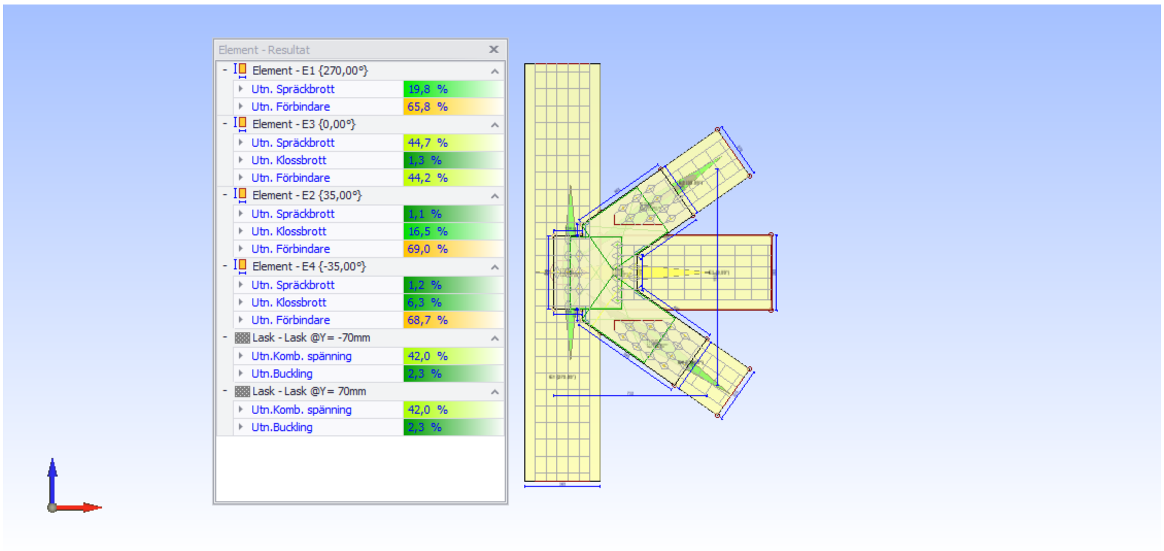
Connection B, vertical



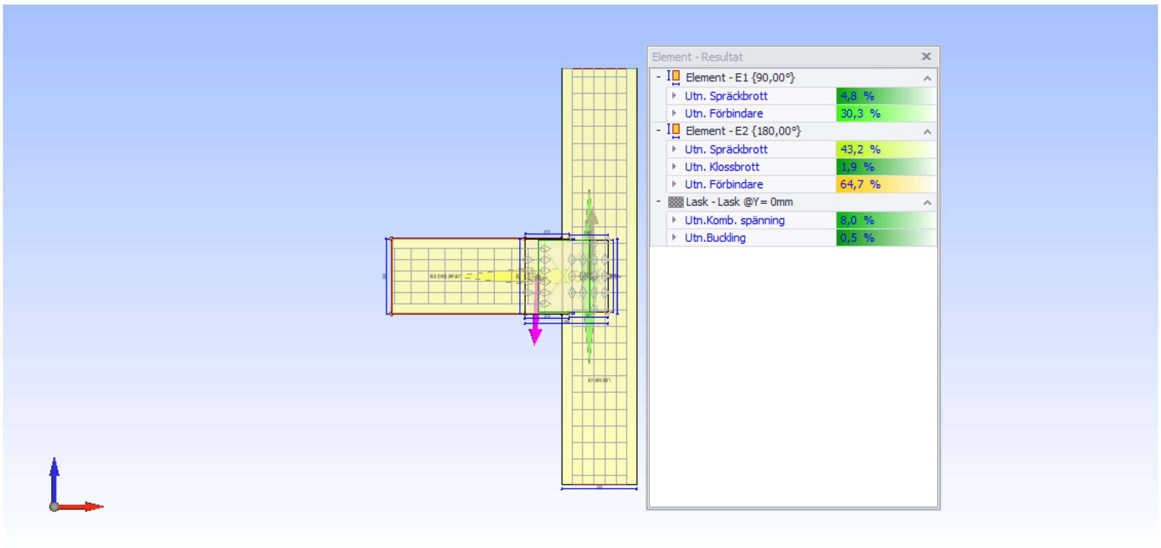
Connection B, diagonal



Connection C

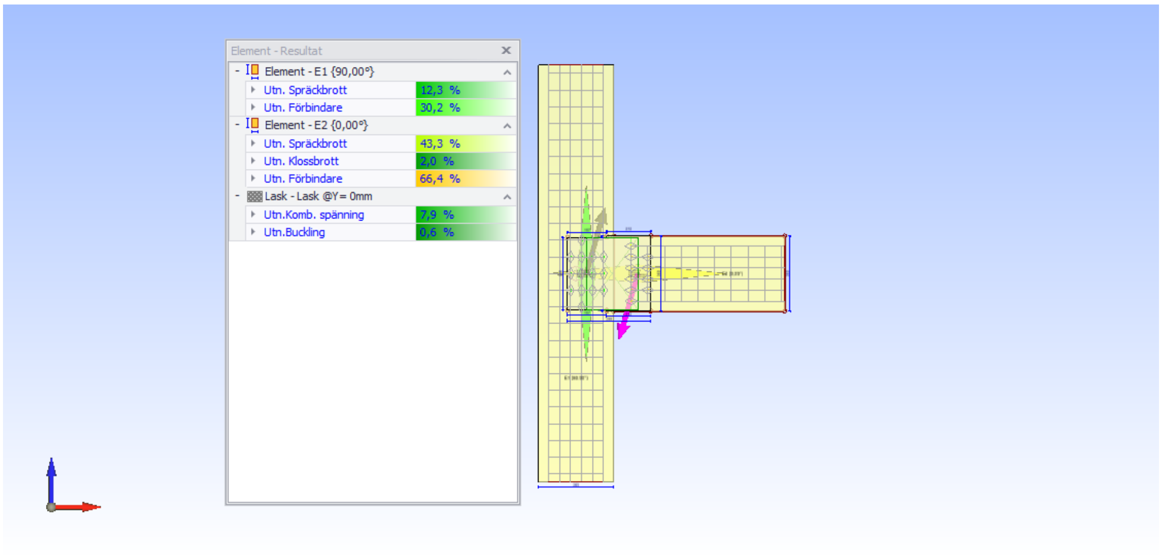


Connection D

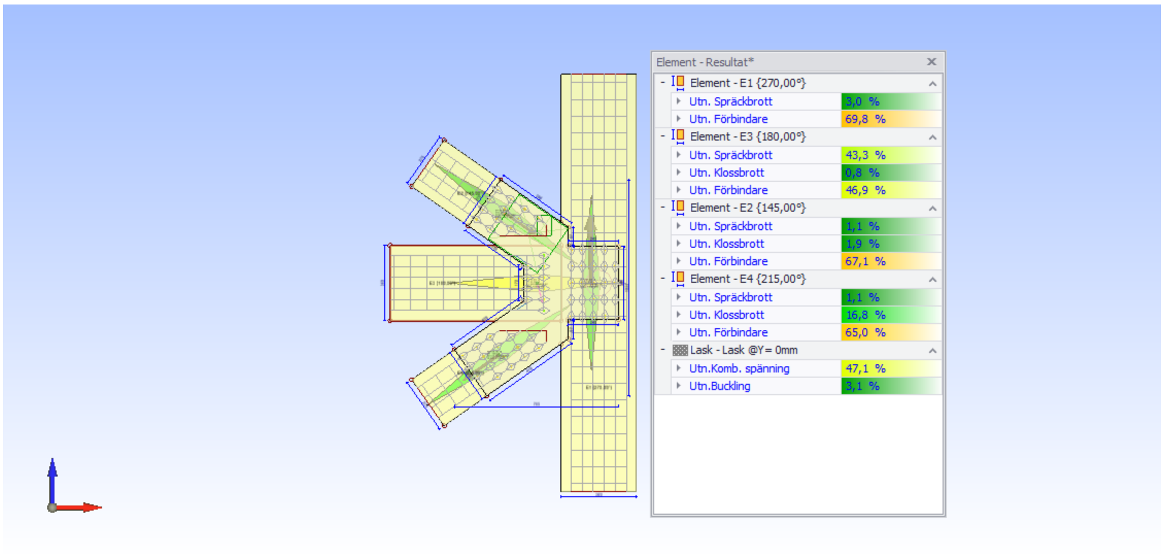


Appendix B

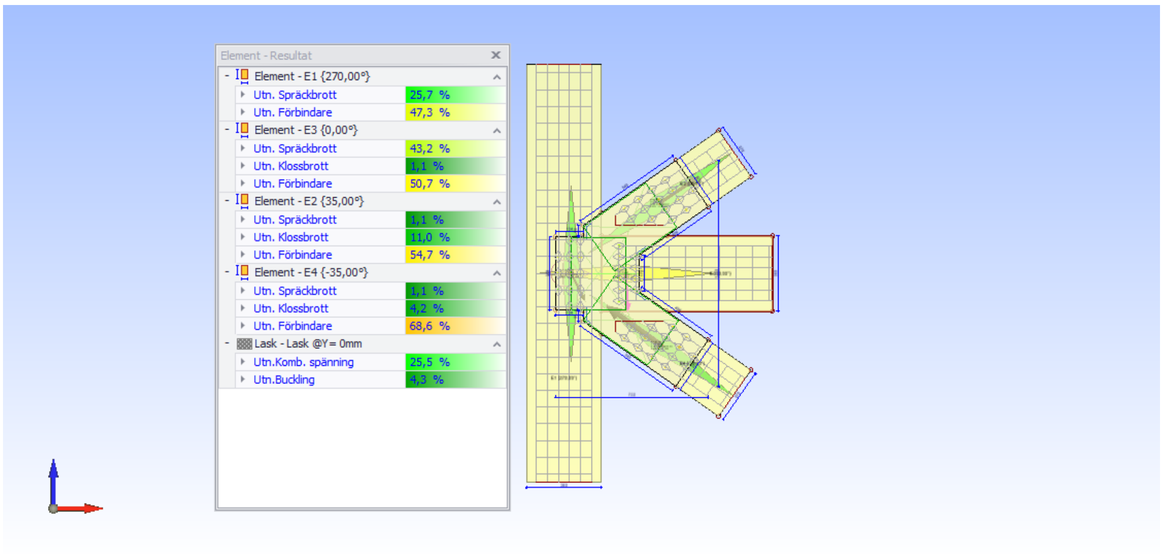
Connection E



Connection F

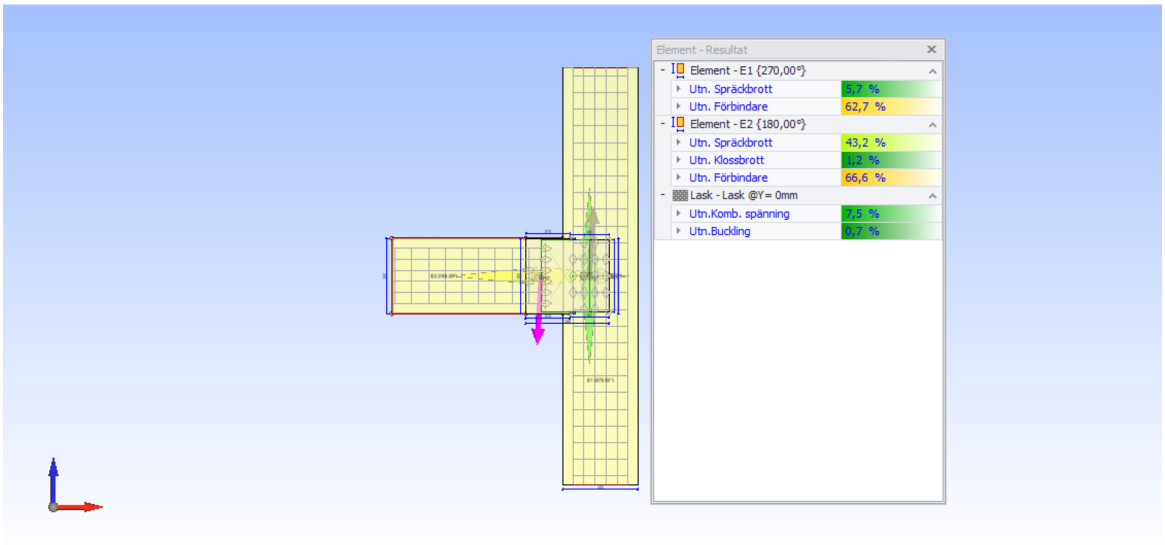


Connection G

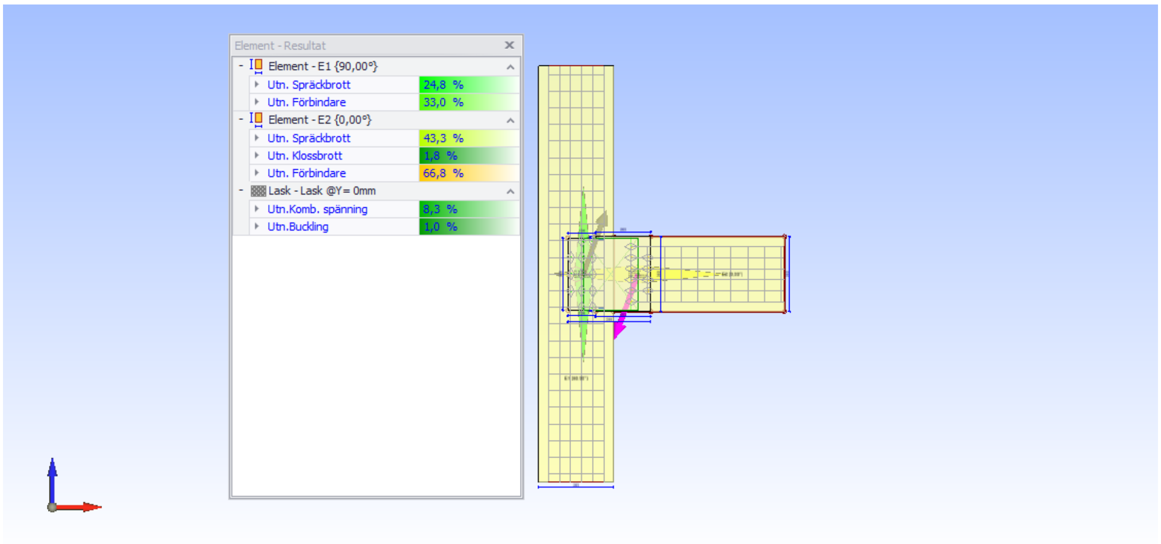


Appendix B

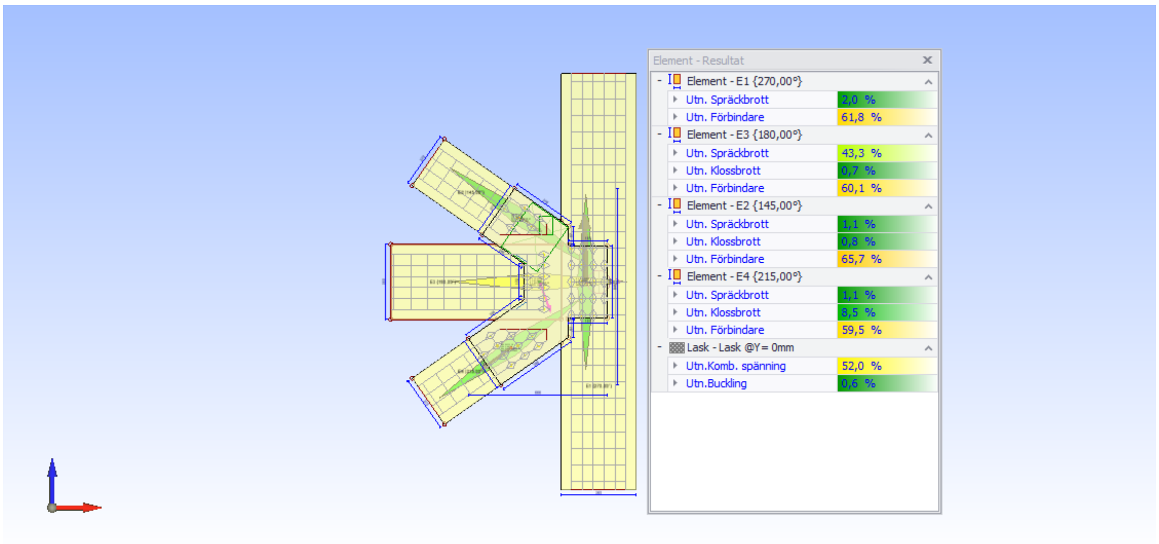
Connection H



Connection I

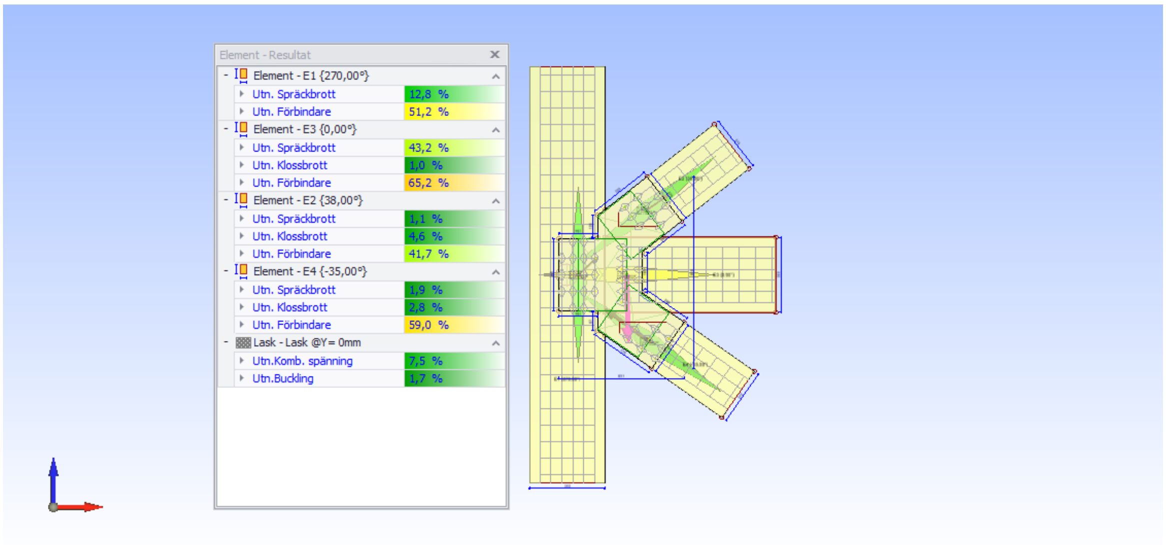


Connection J

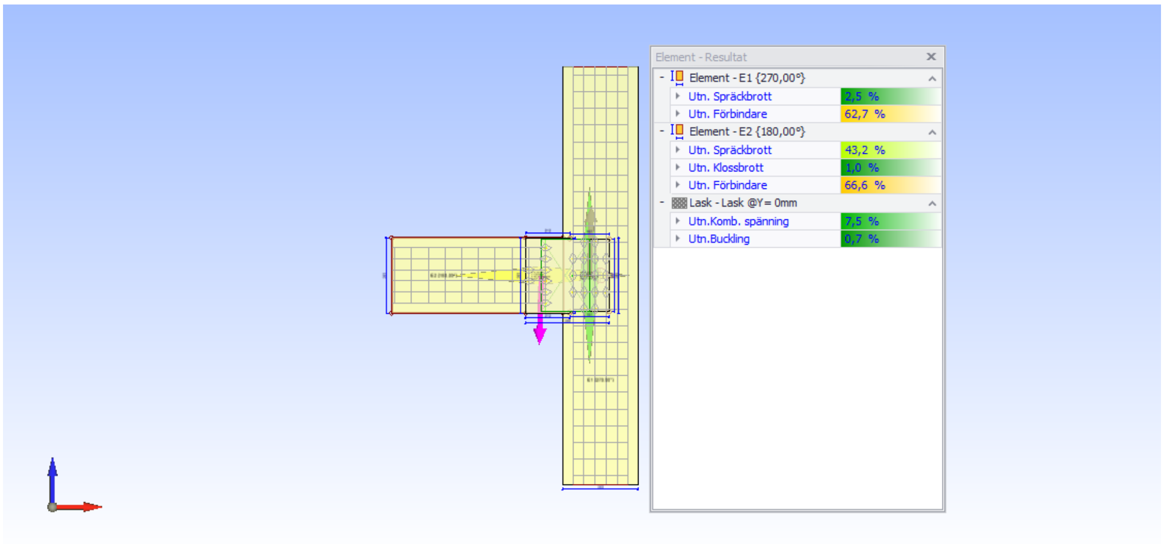


Appendix B

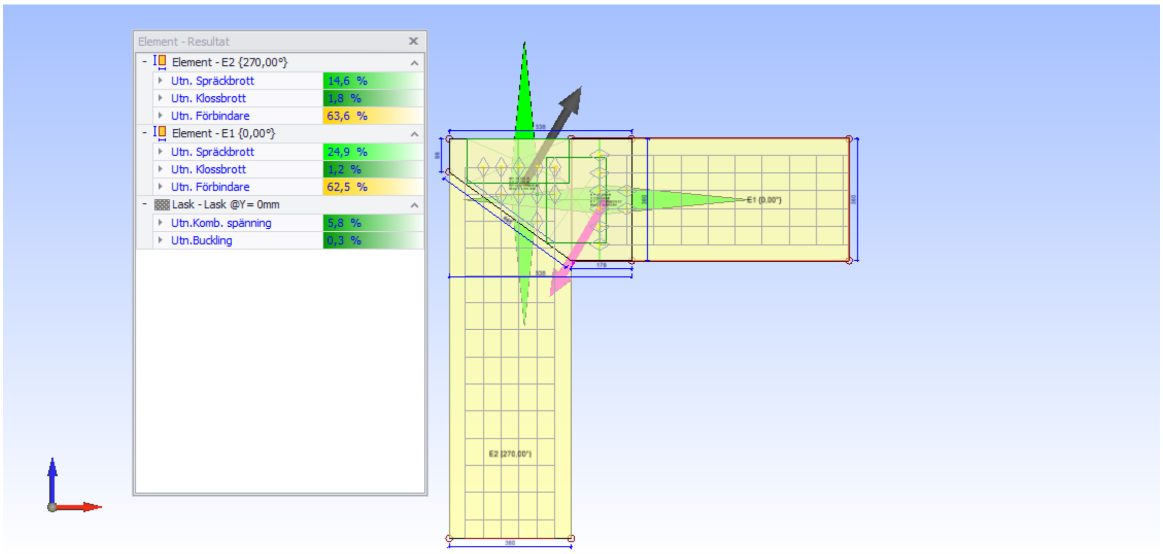
Connection K



Connection L

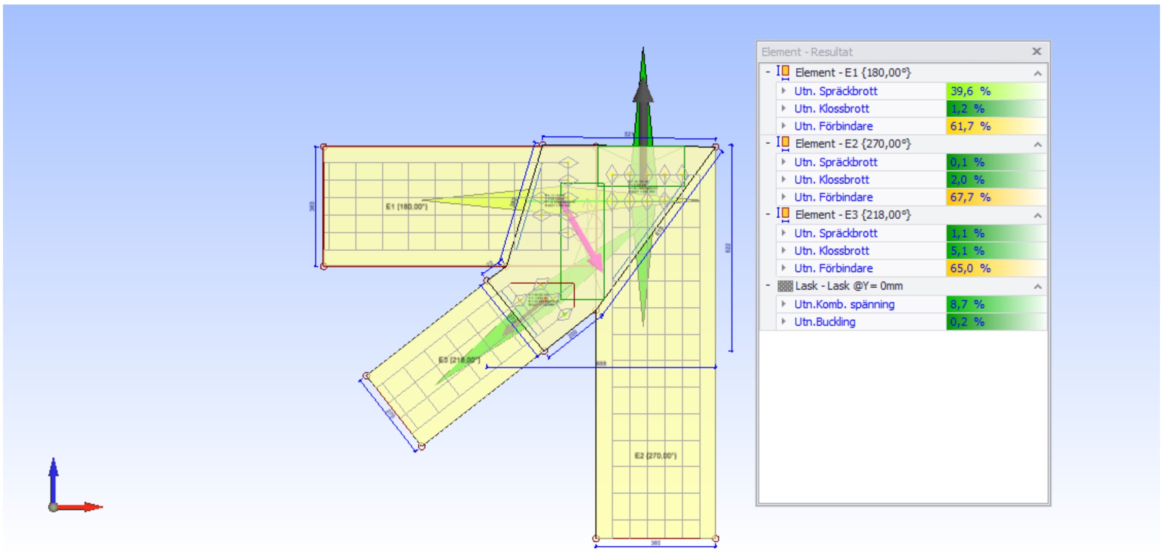


Connection M



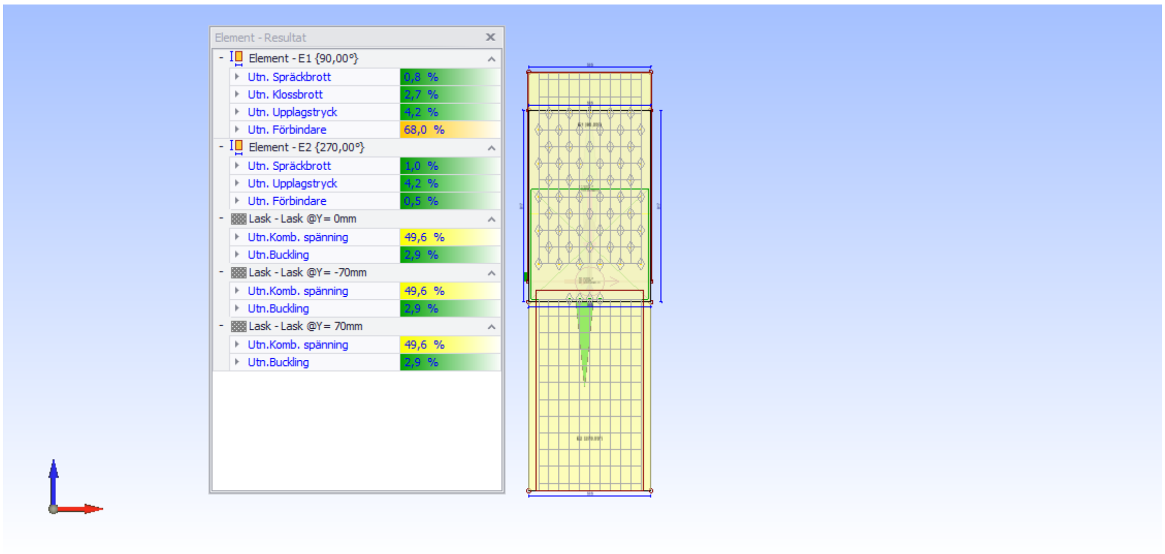
Appendix B

Connection N

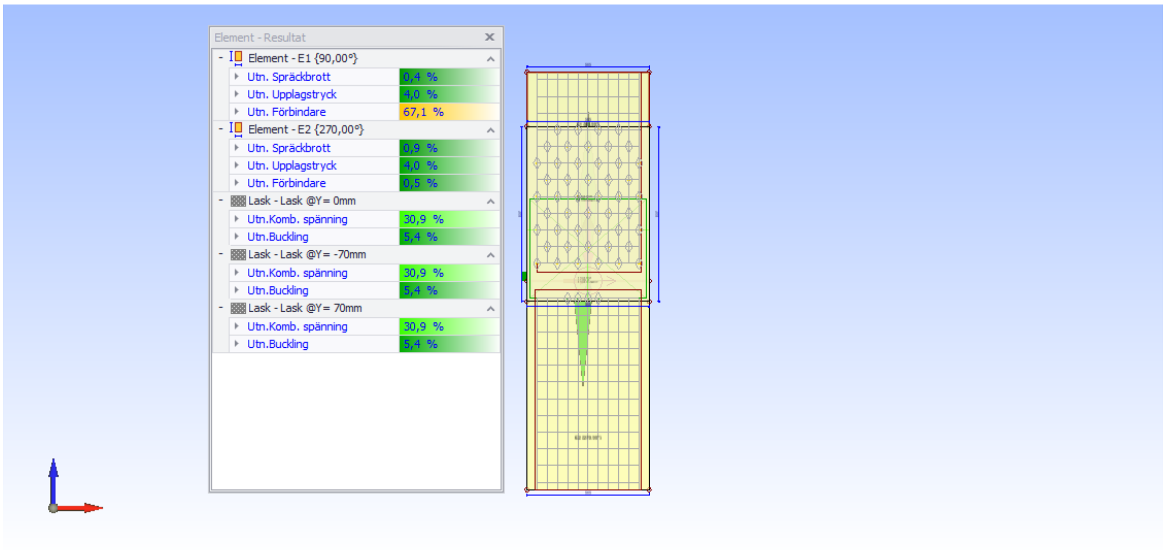


Truss 7

Connection A

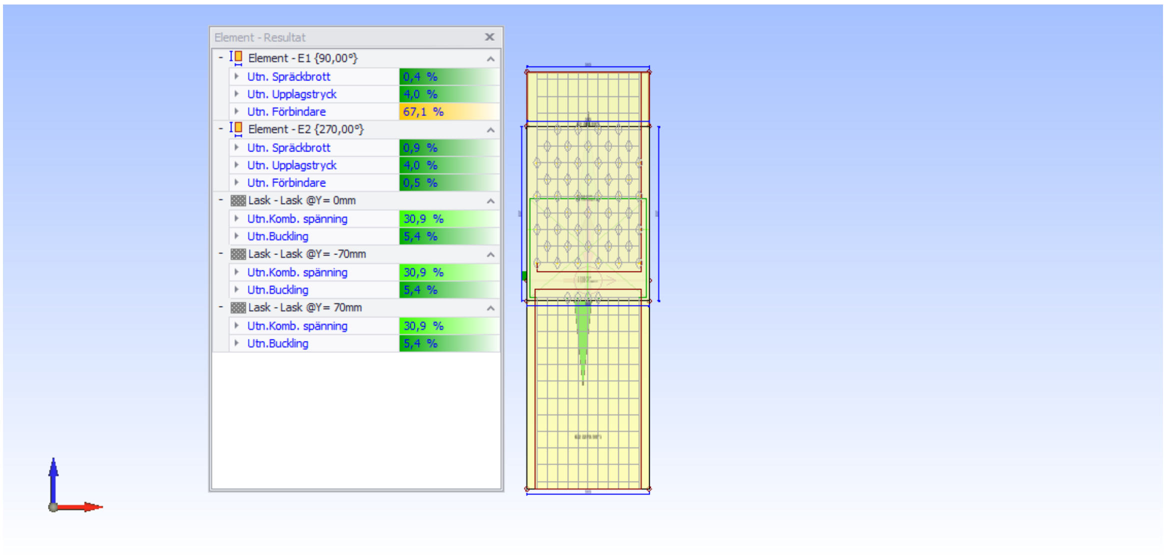


Connection B, vertical

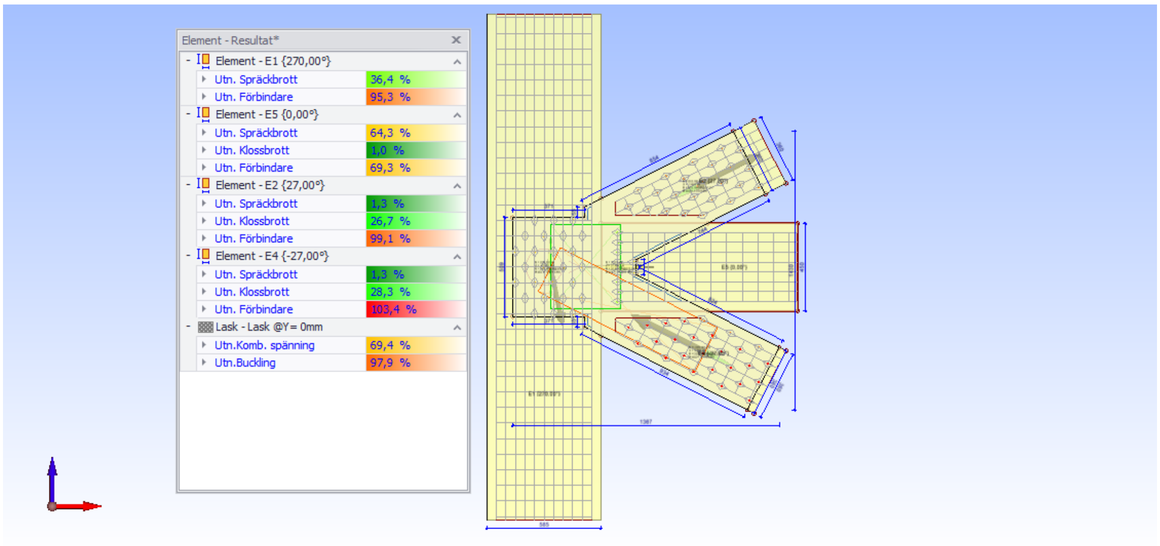


Appendix B

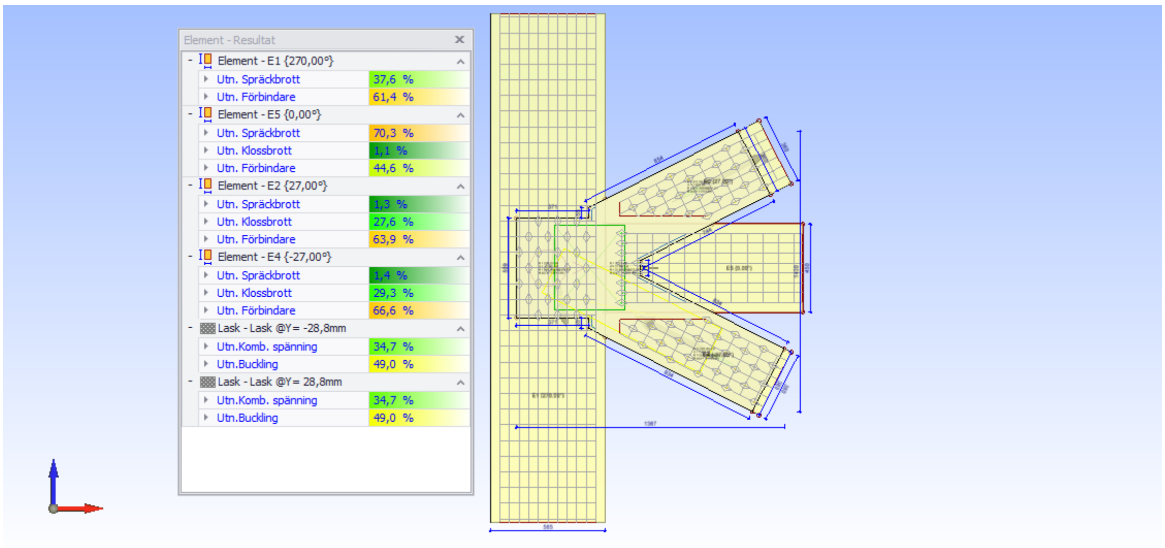
Connection B, diagonal



Connection C, horizontal member – 1 plate

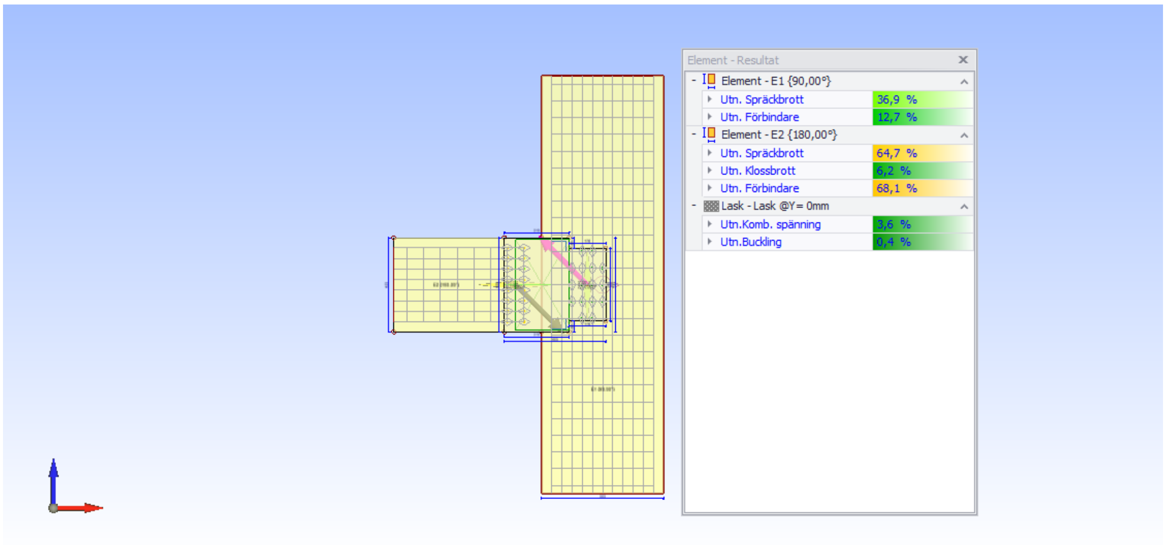


Connection C, vertical and diagonal members – 2 plates

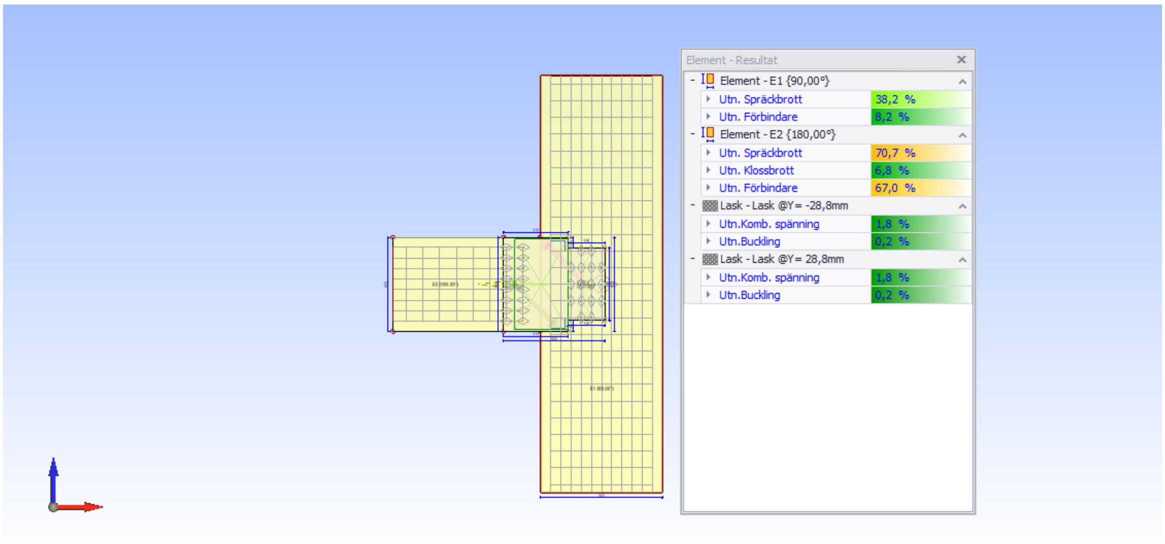


Appendix B

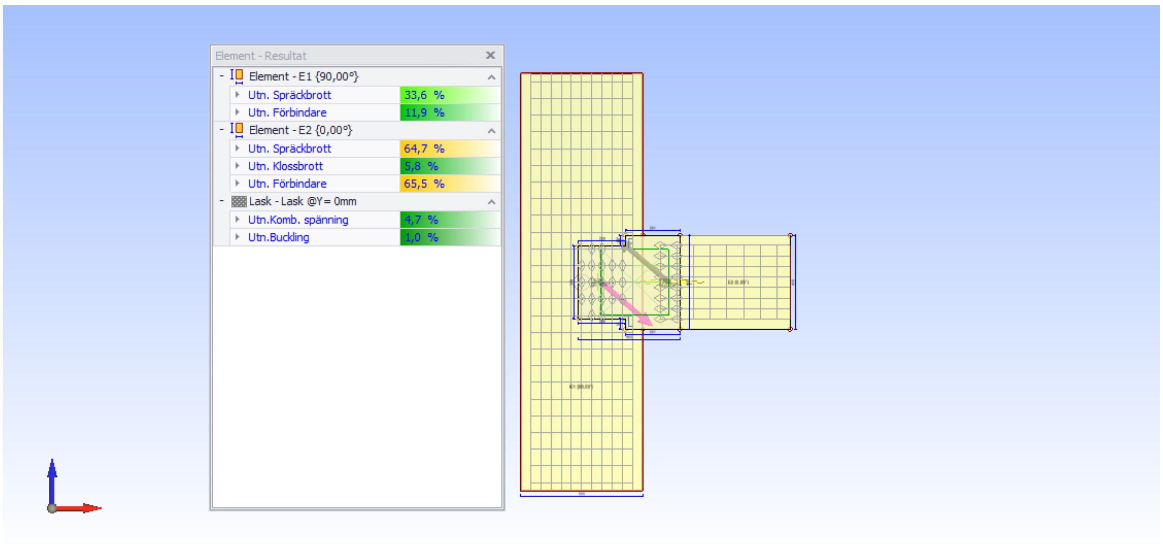
Connection D, horizontal member – 1 plate



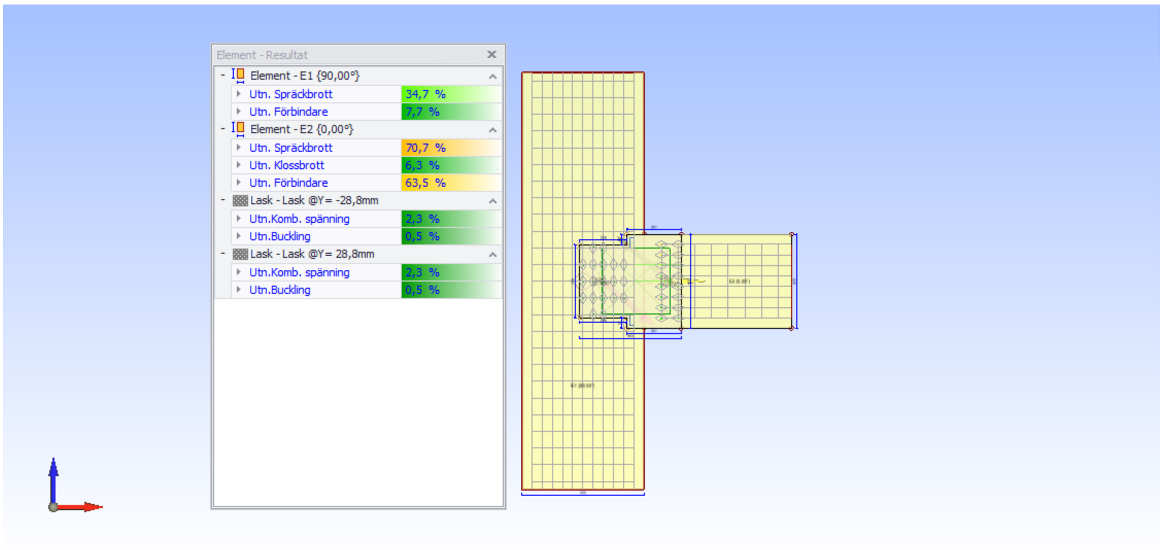
Connection D, vertical member – 2 plates



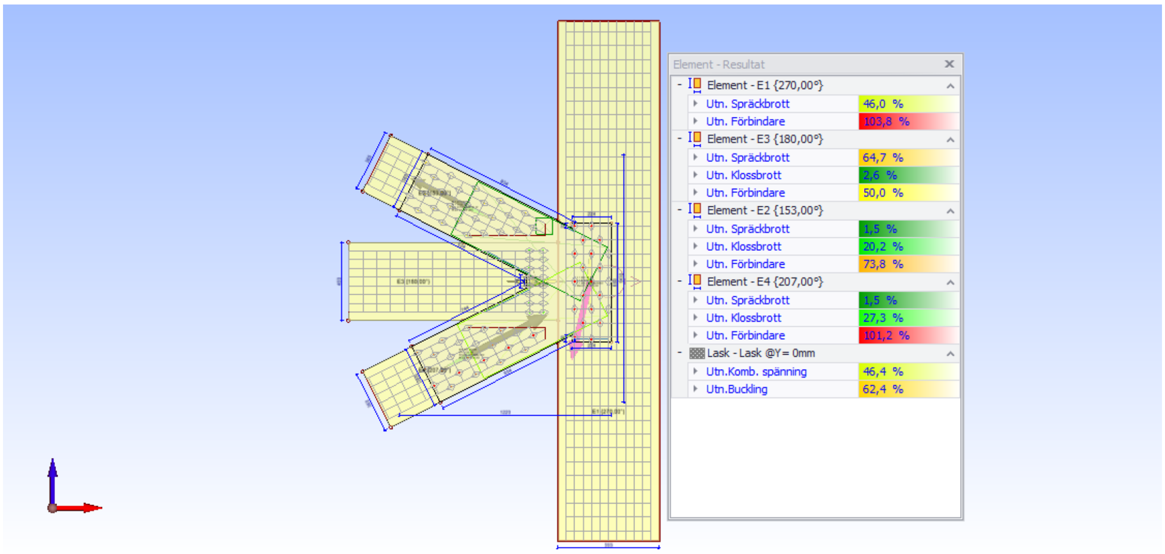
Connection E, horizontal member – 1 plate



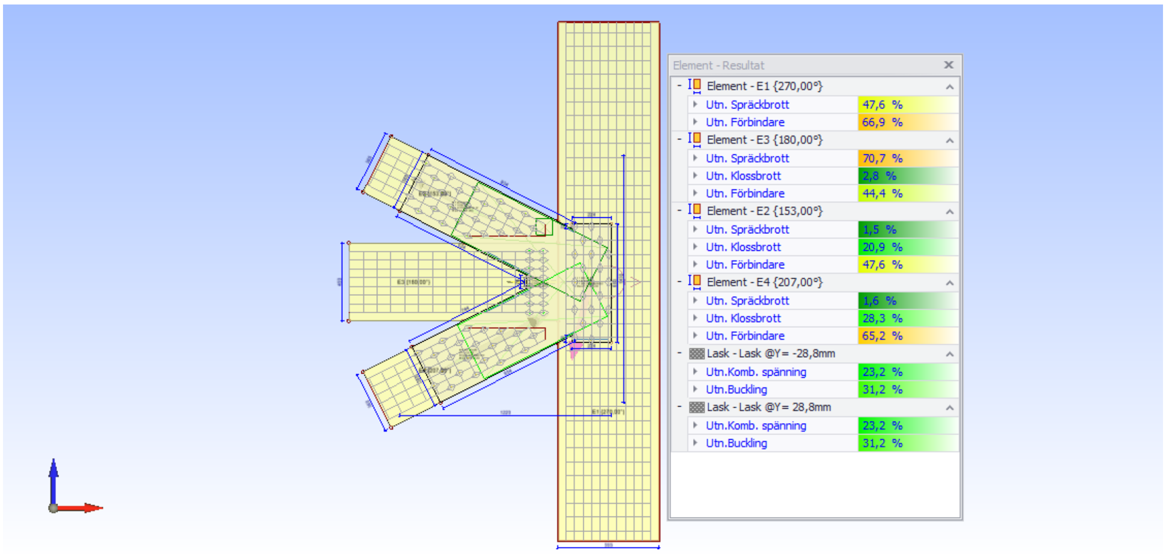
Connection E, vertical member – 2 plates



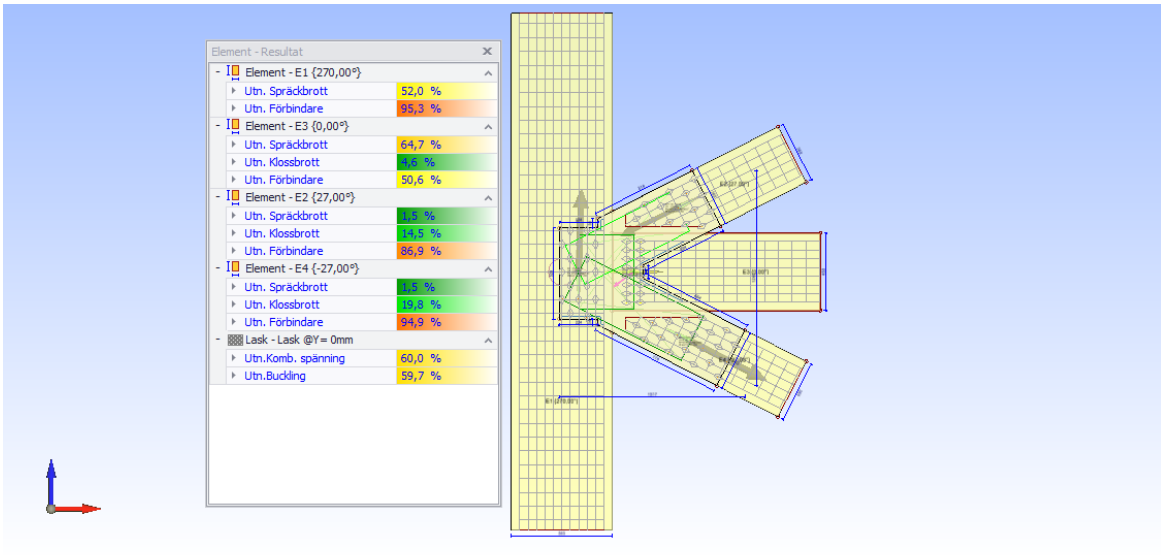
Connection F, horizontal member – 1 plate



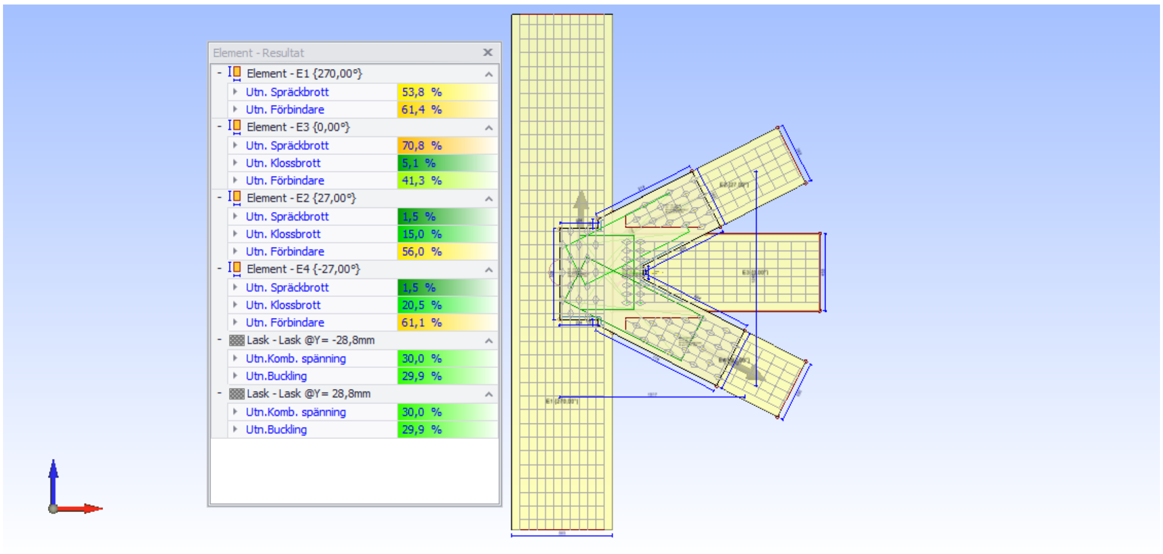
Connection F, vertical and diagonal members – 2 plates



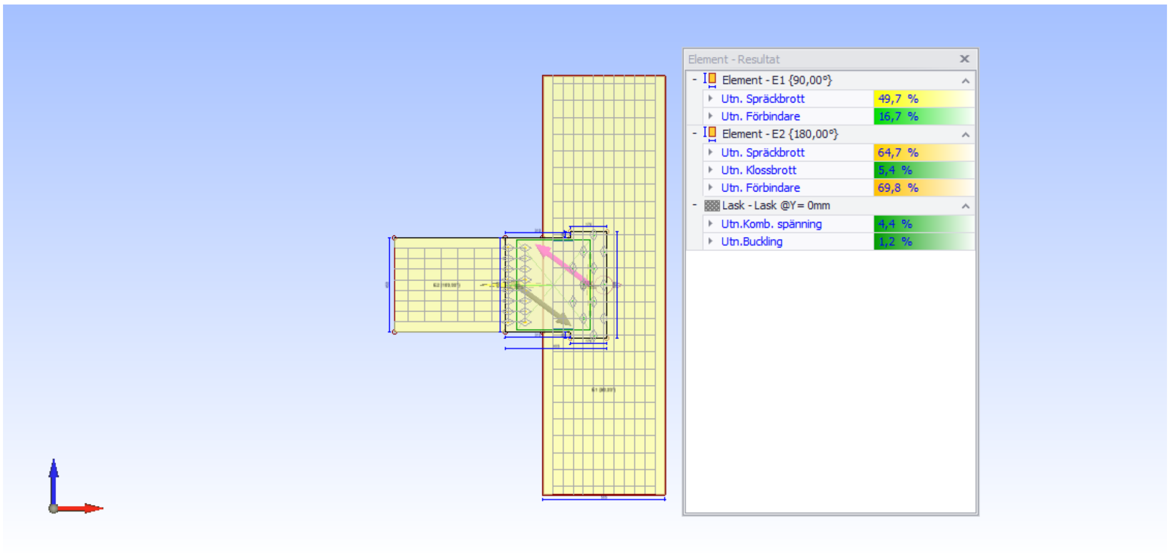
Connection G, horizontal member – 1 plate



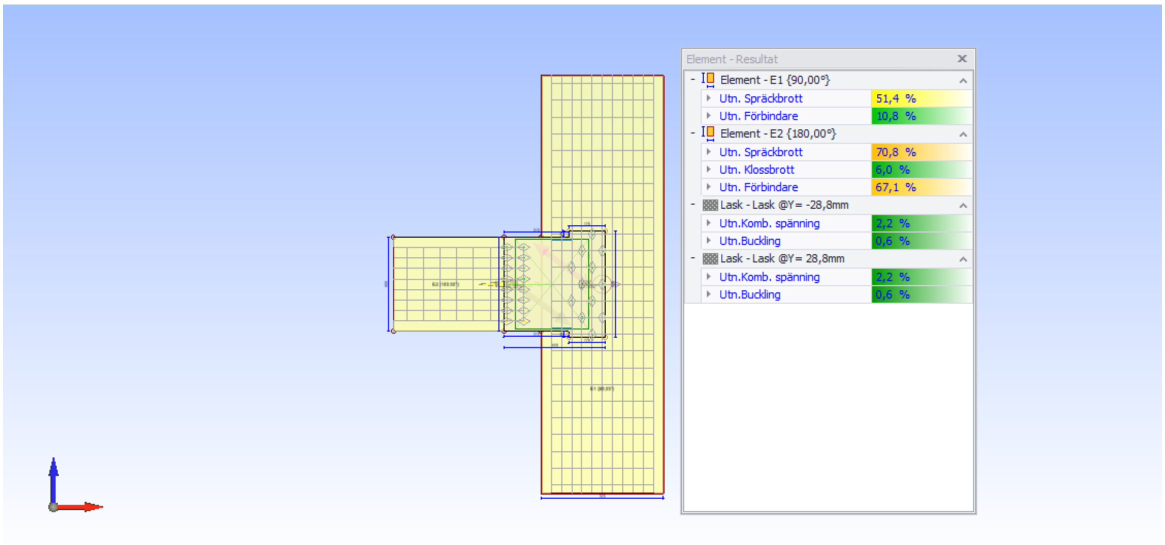
Connection G, vertical and diagonal members – 2 plates



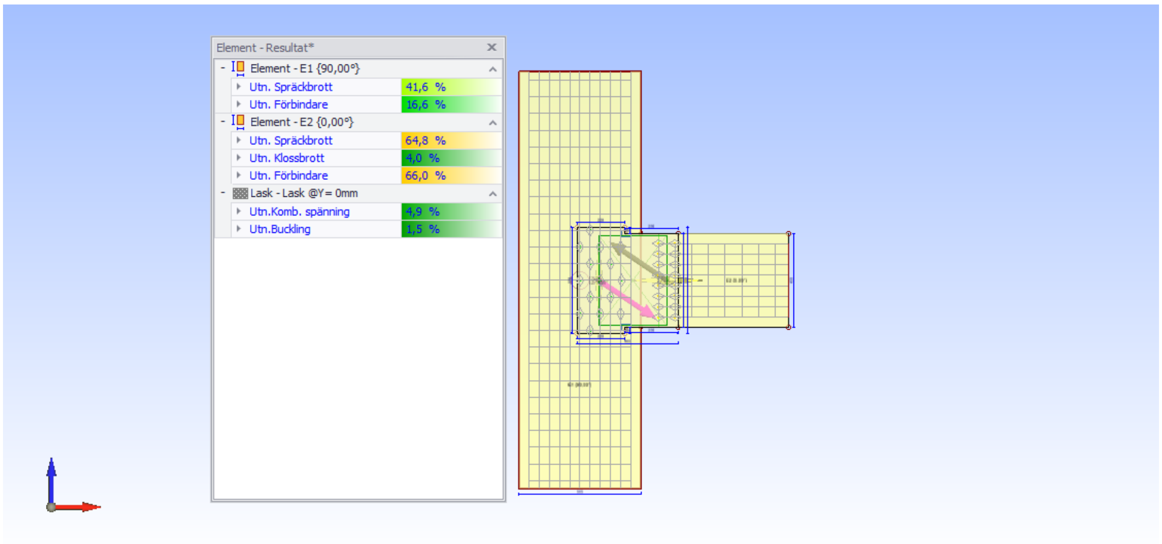
Connection H, horizontal member – 1 plate



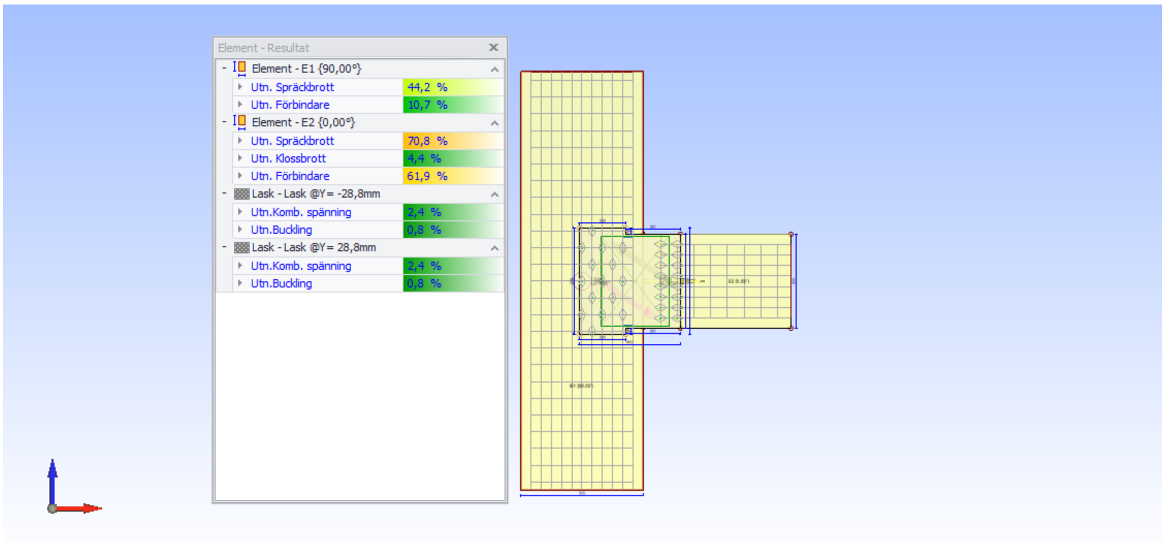
Connection H, vertical member – 2 plates



Connection I, horizontal member – 1 plate

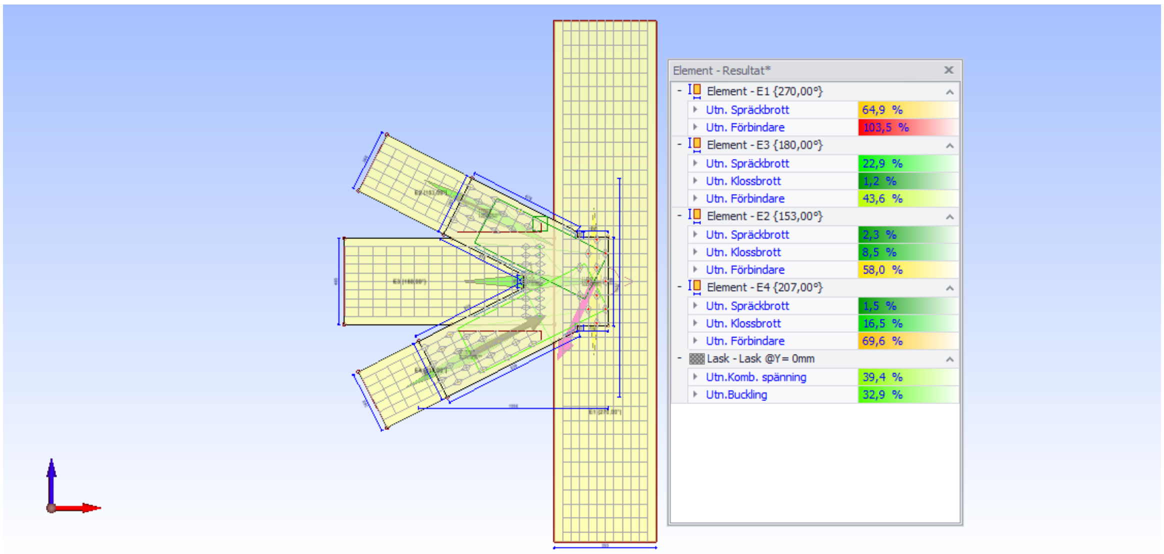


Connection I, vertical member – 2 plates

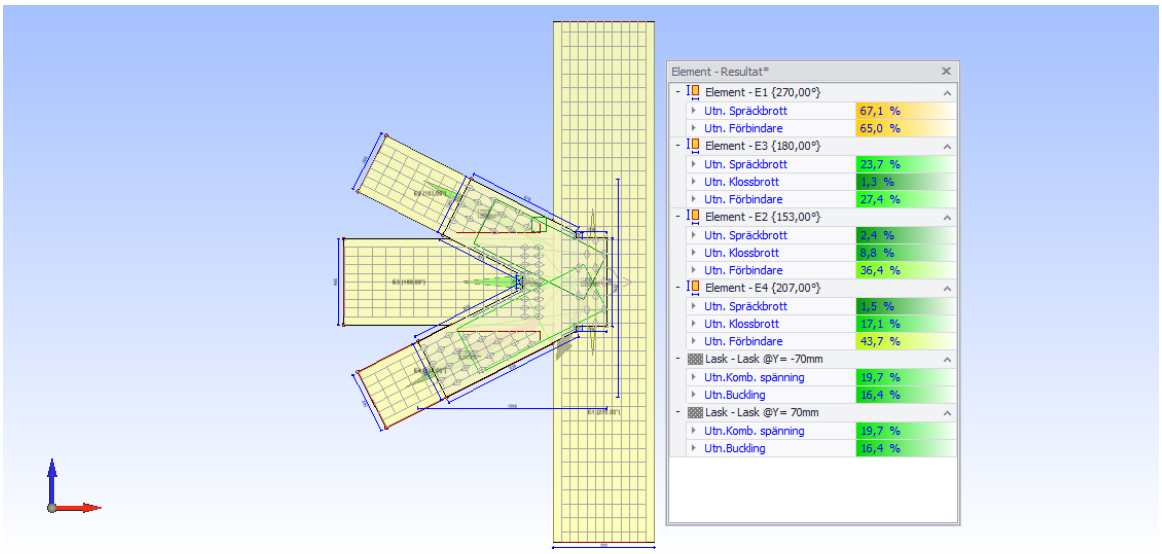


Appendix B

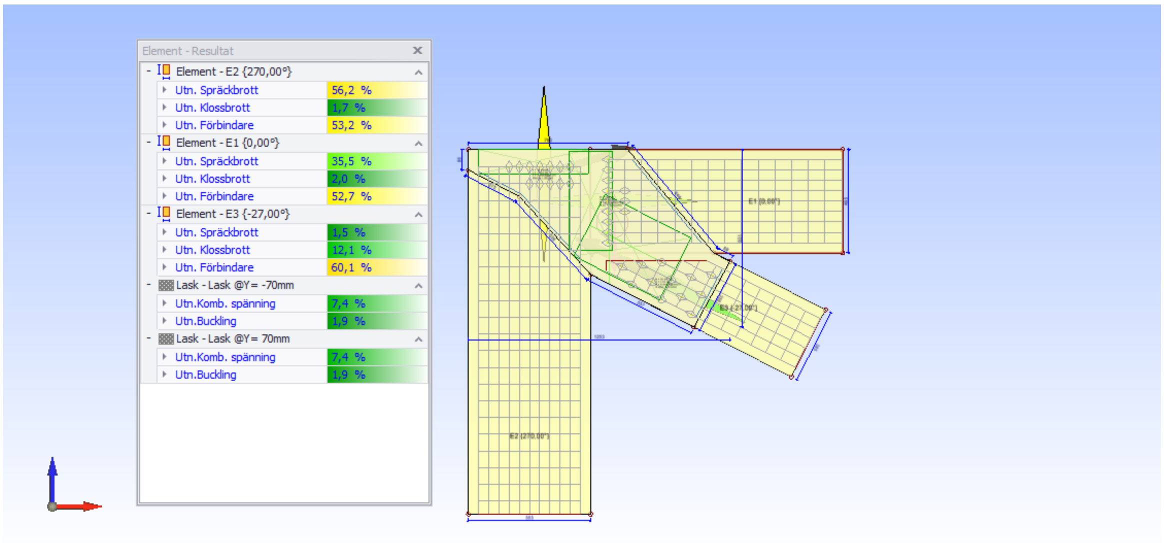
Connection J, horizontal member – 1 plate



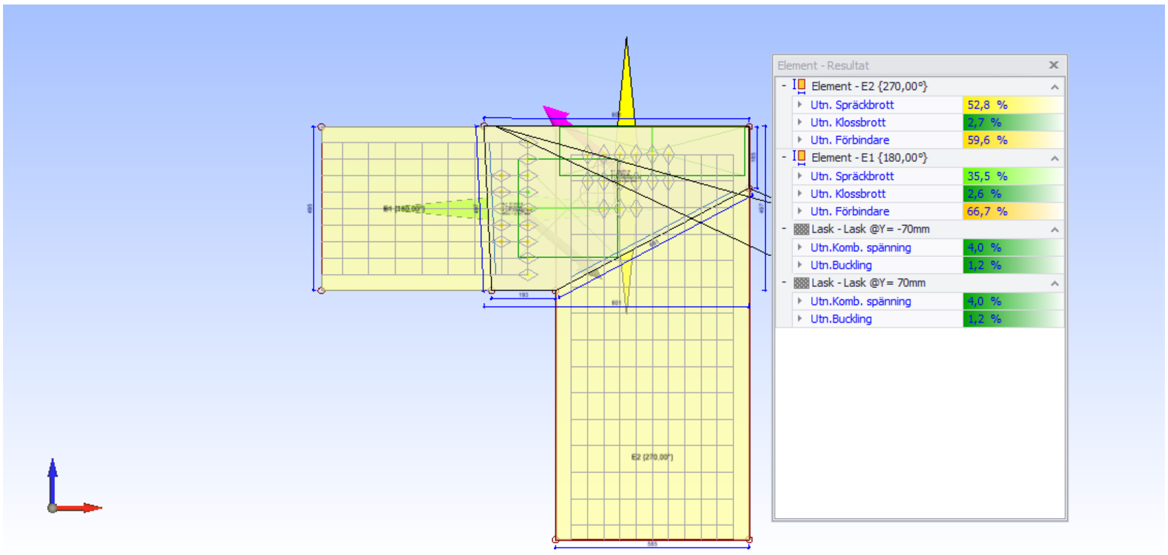
Connection J, vertical and diagonal members – 2 plates



Connection K



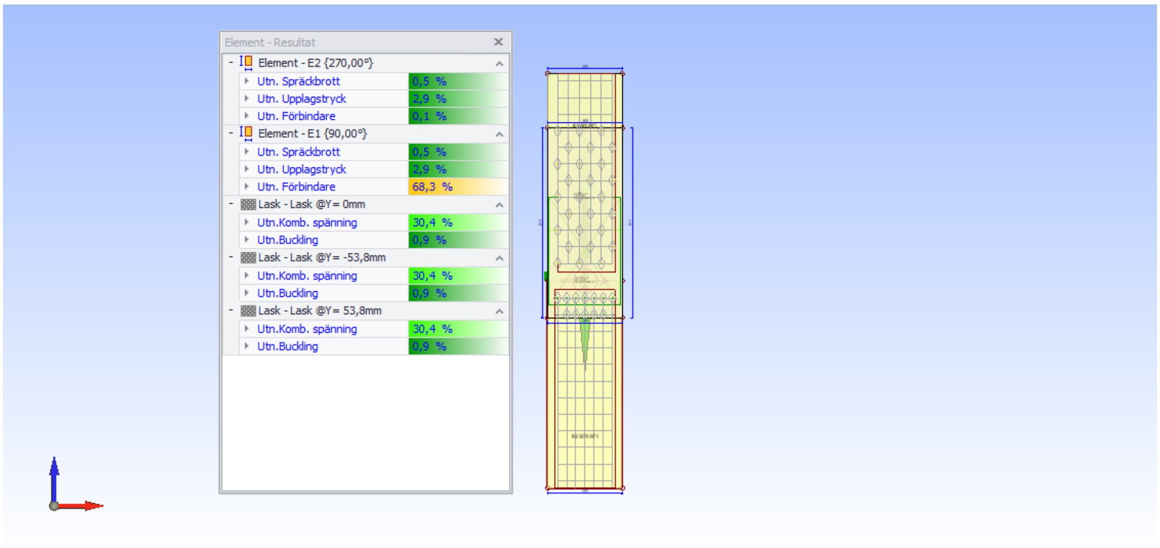
Connection L



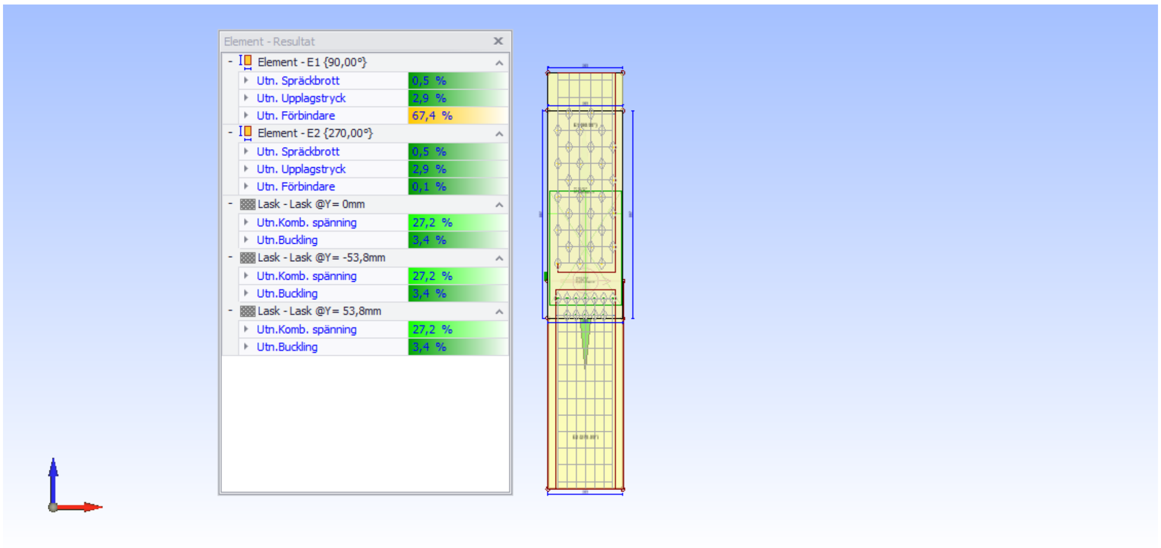
Connections of four storey building

Truss 1

Connection A

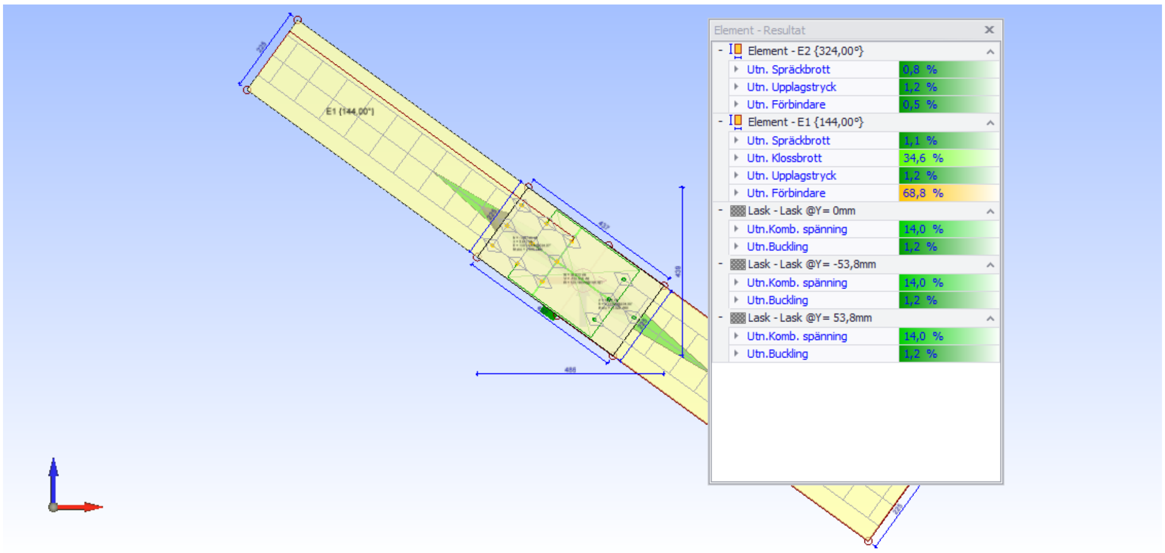


Connection B, vertical

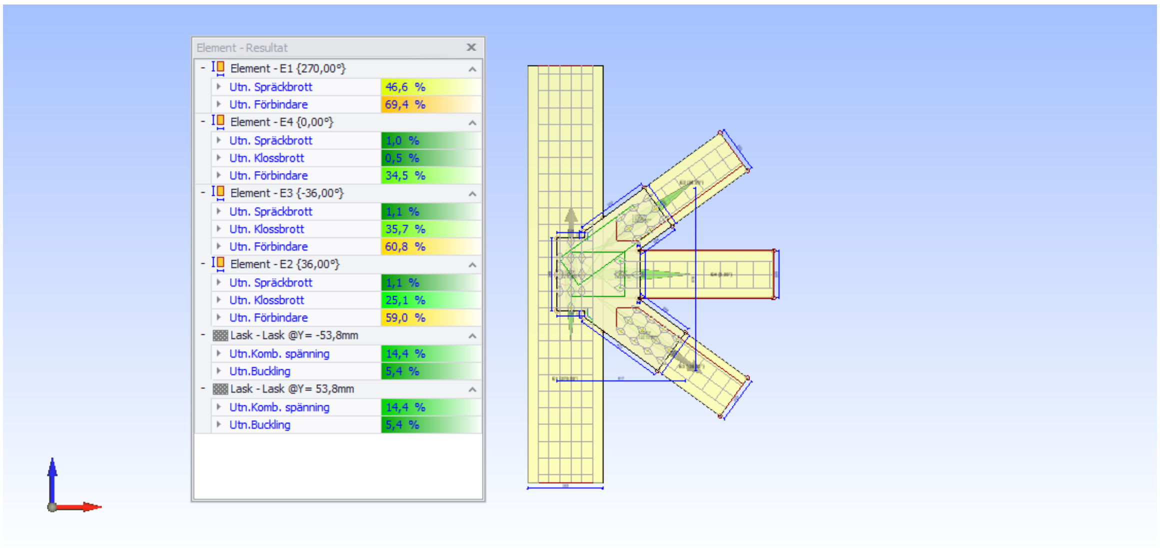


Appendix B

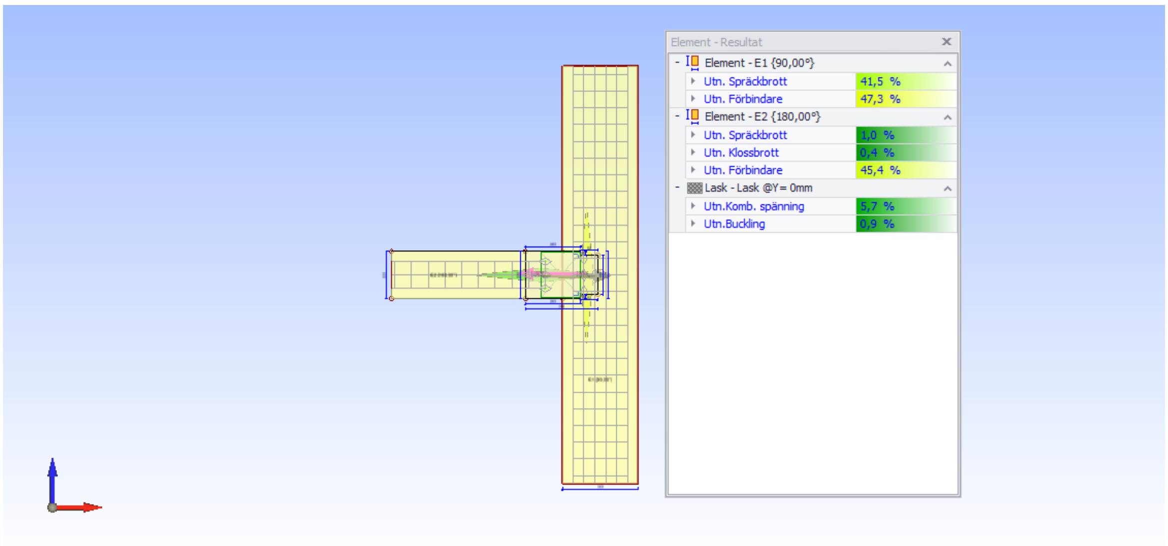
Connection B, diagonal



Connection C

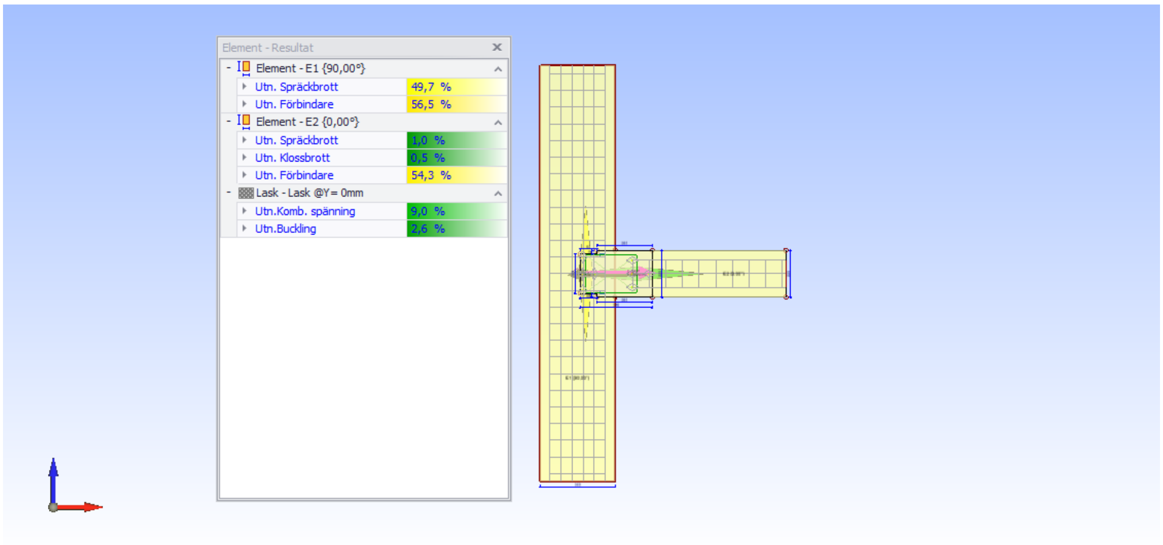


Connection D

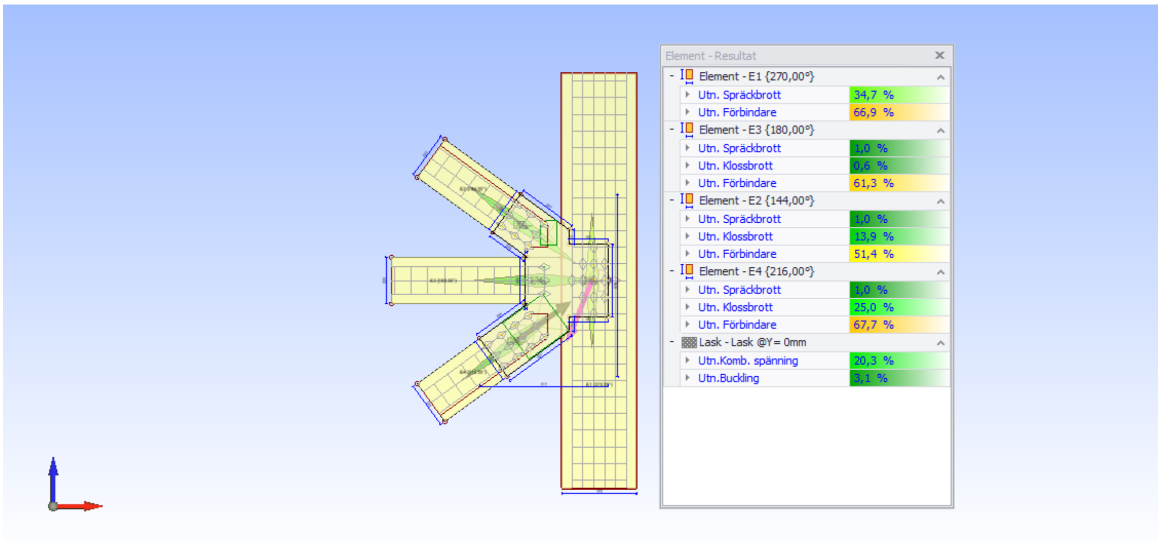


Appendix B

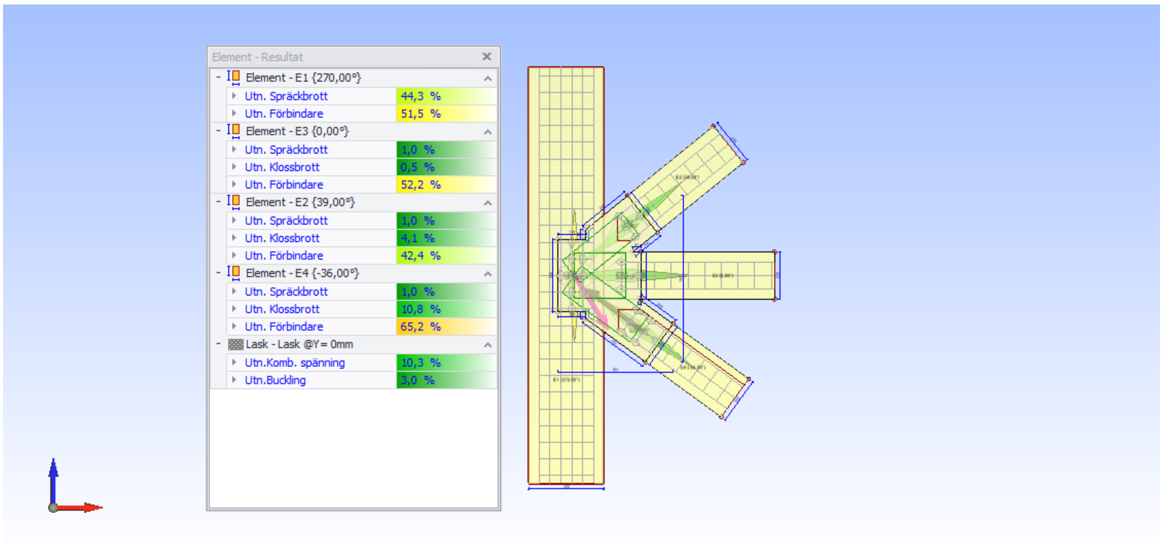
Connection E



Connection F

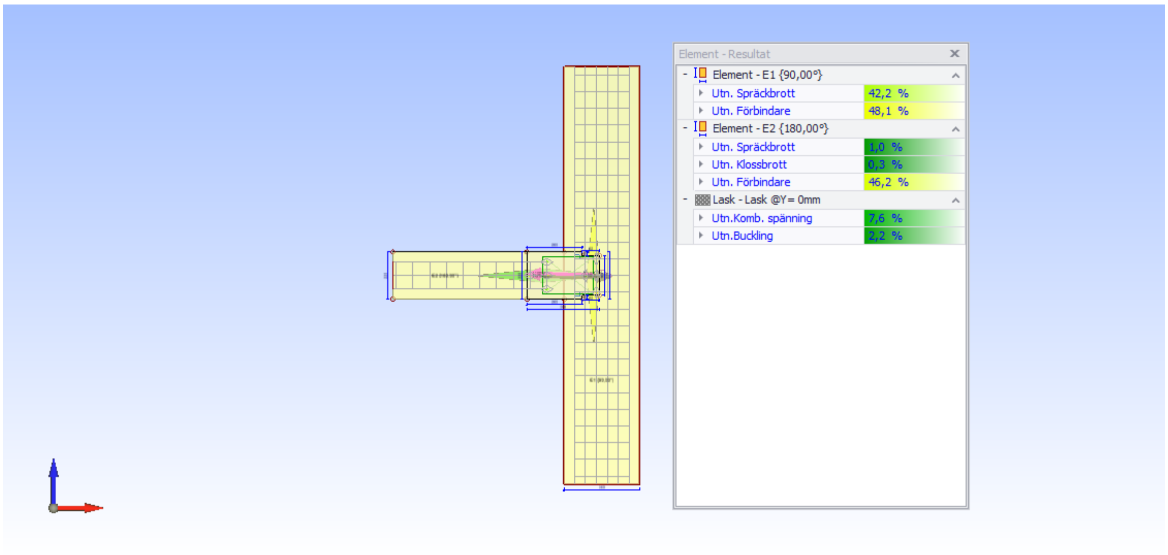


Connection G

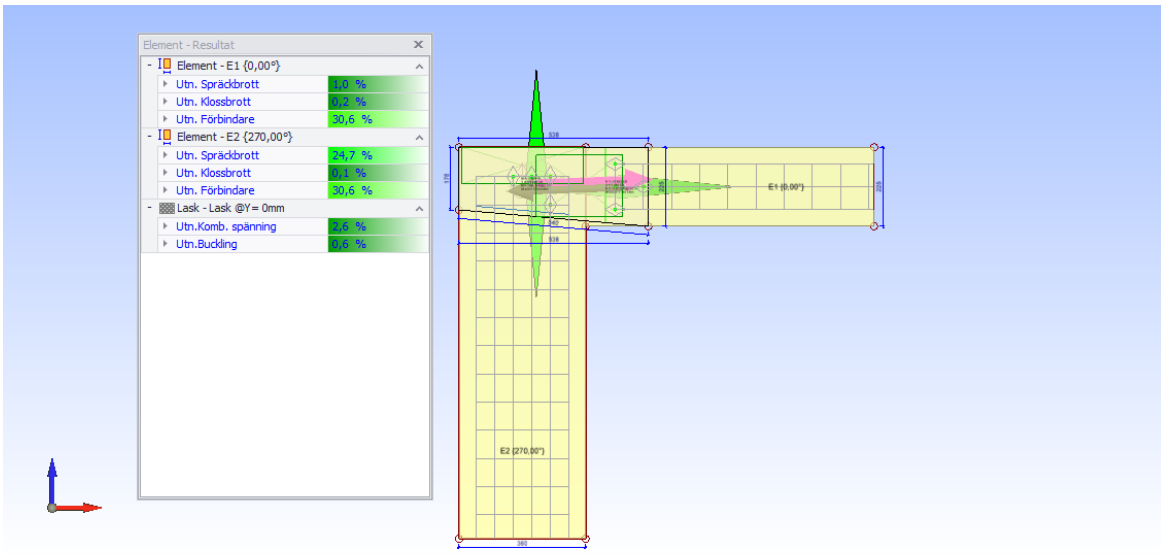


Appendix B

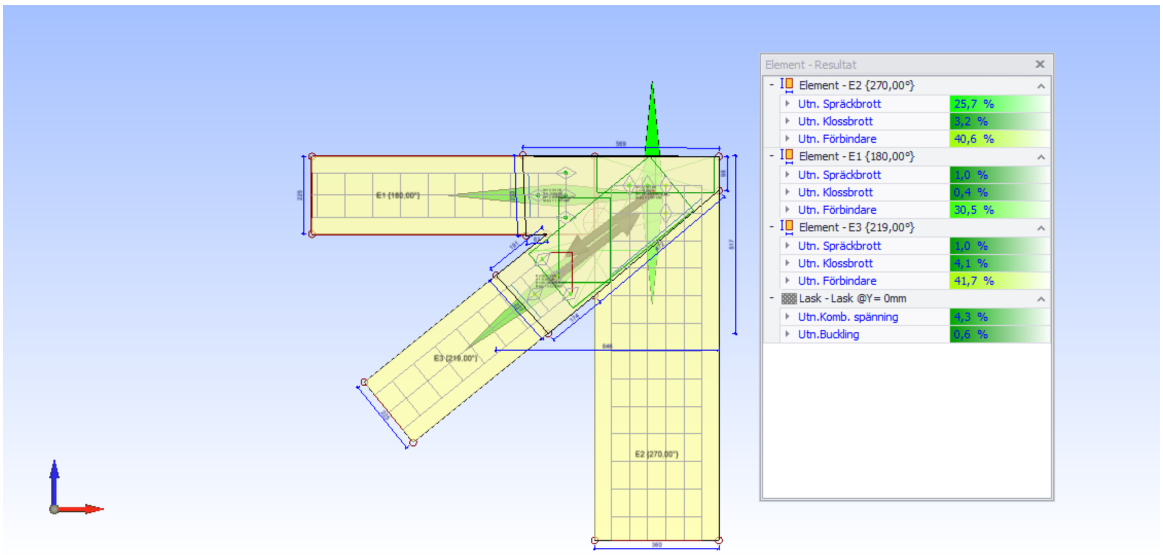
Connection H



Connection I

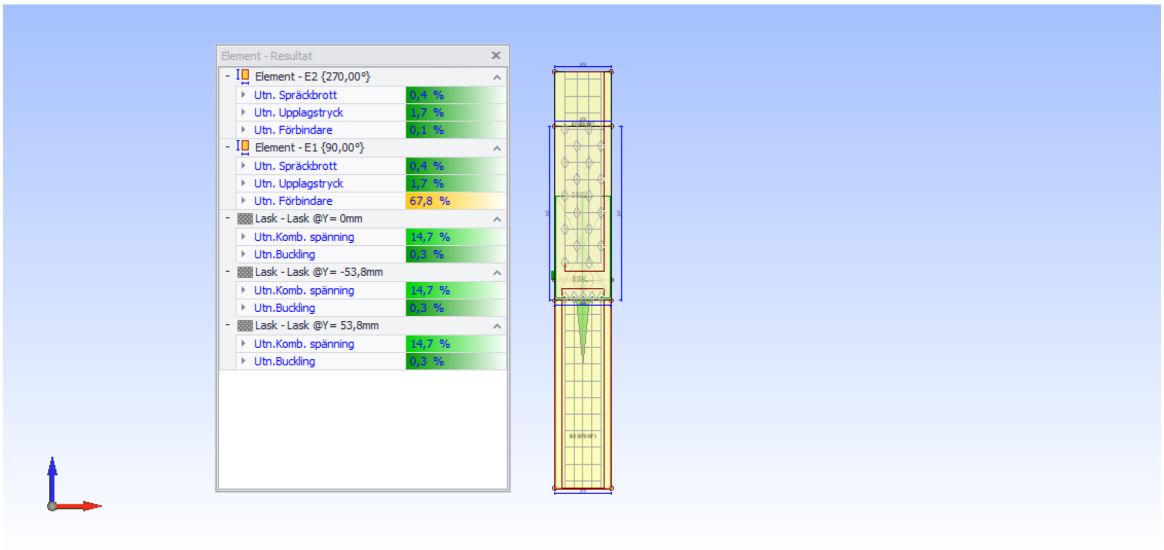


Connection J

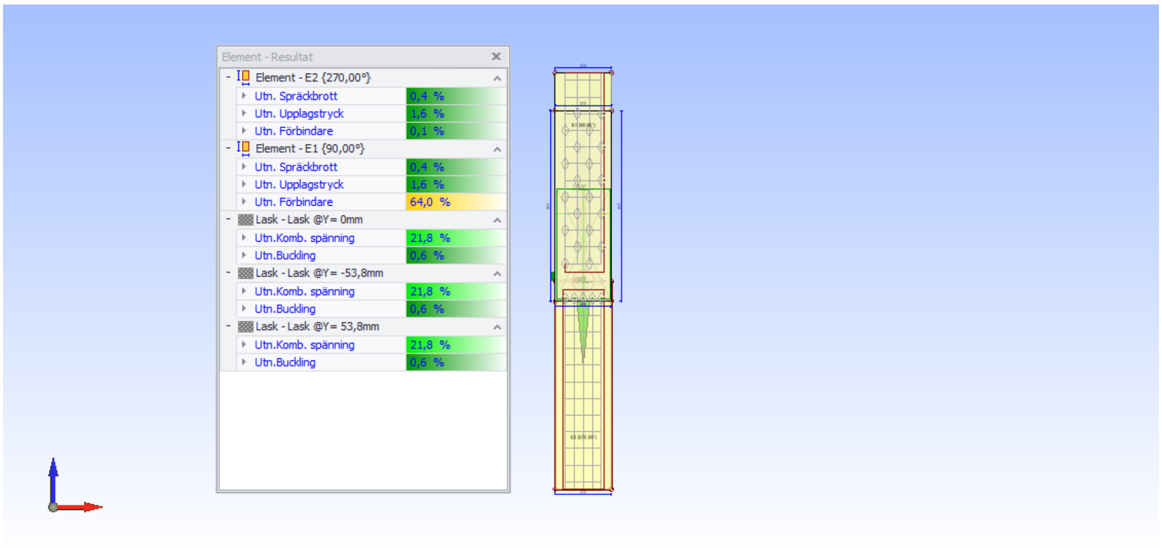


Truss 2

Connection A

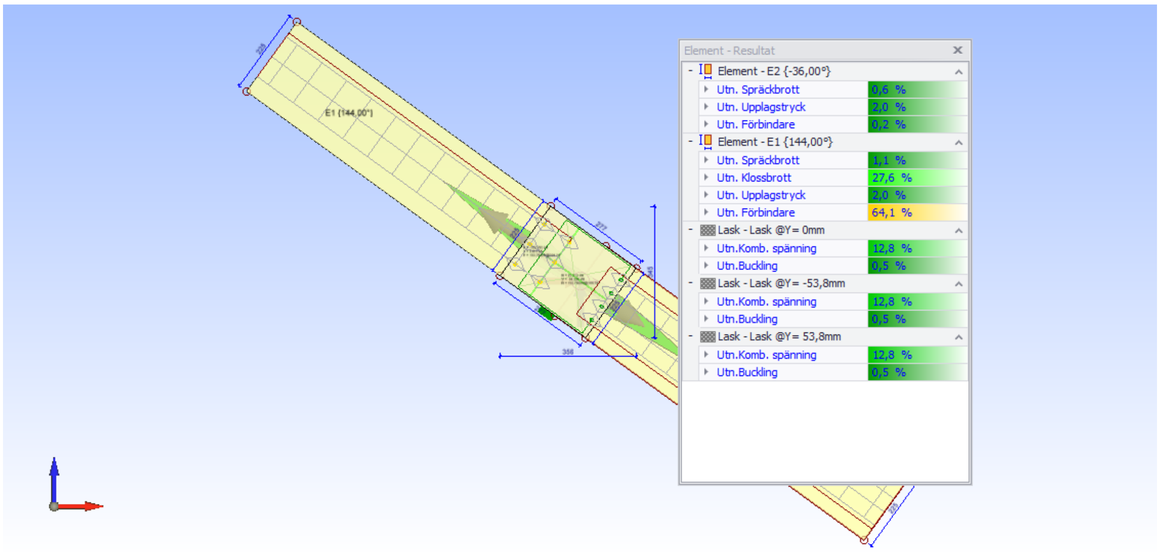


Connection B, vertical

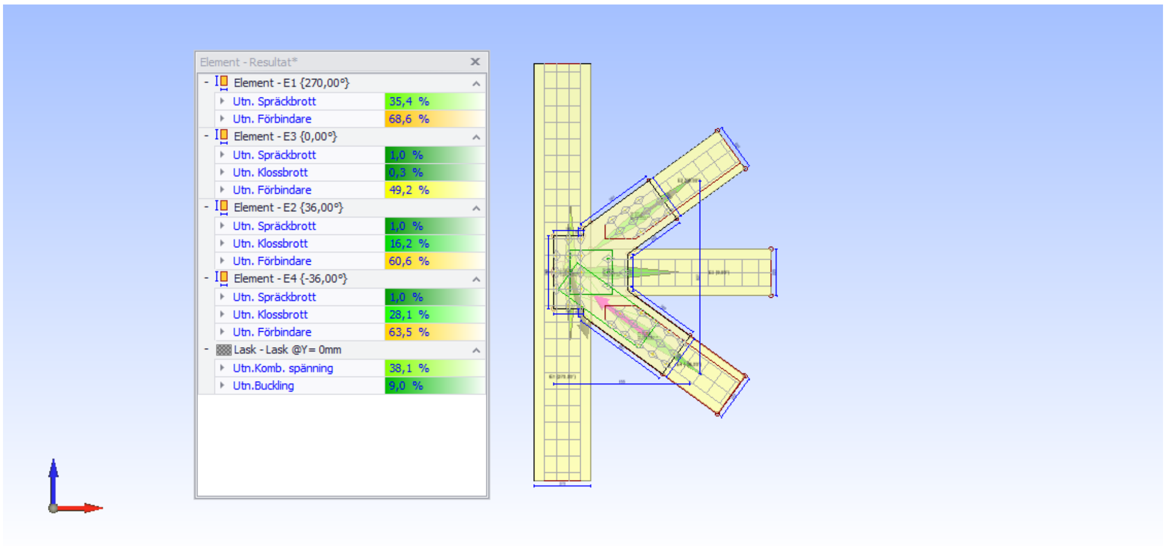


Appendix B

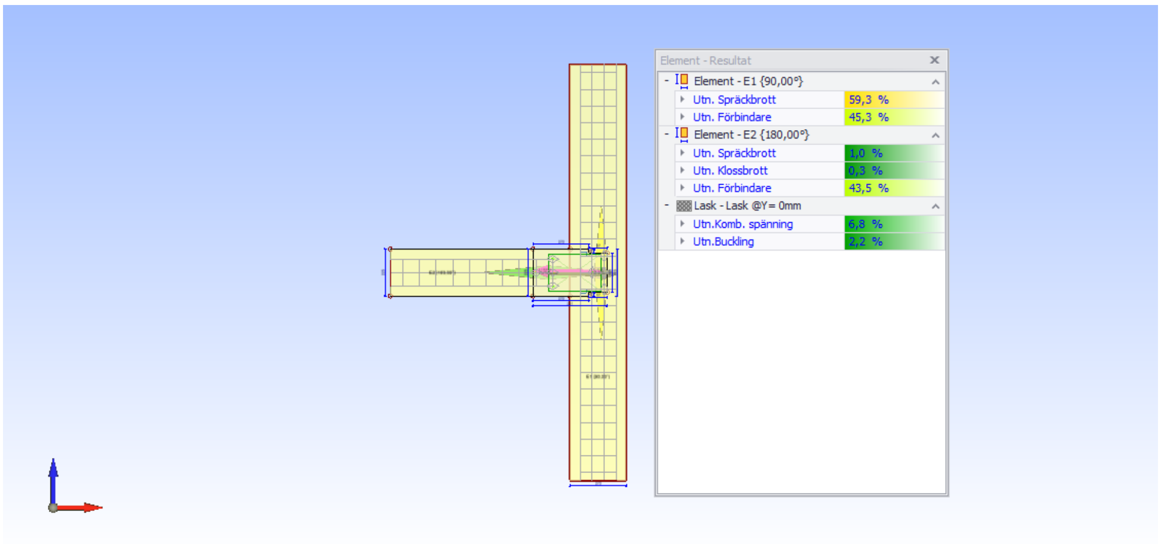
Connection B, diagonal



Connection C

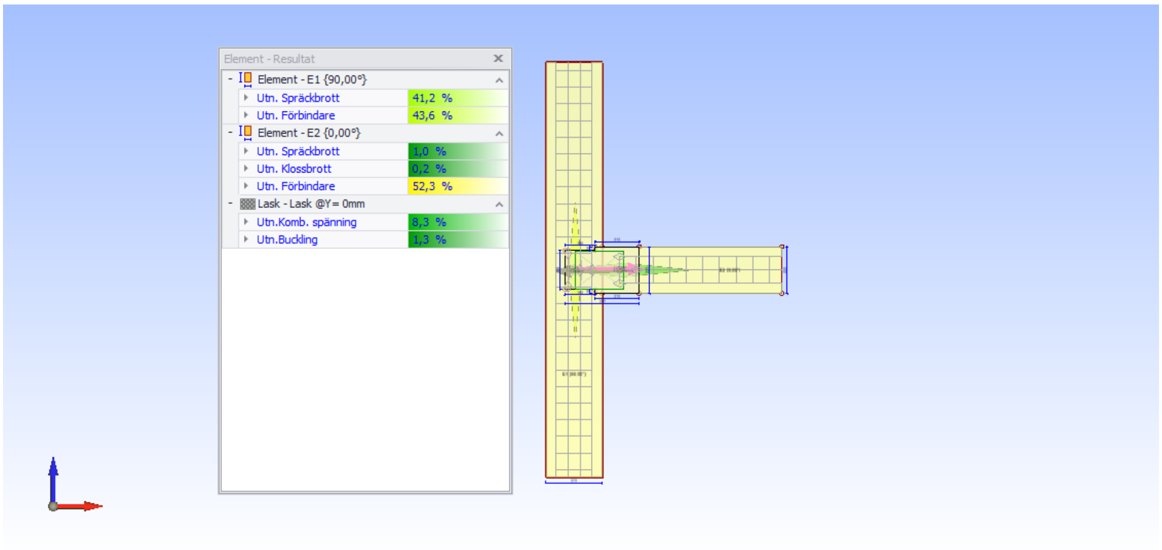


Connection D

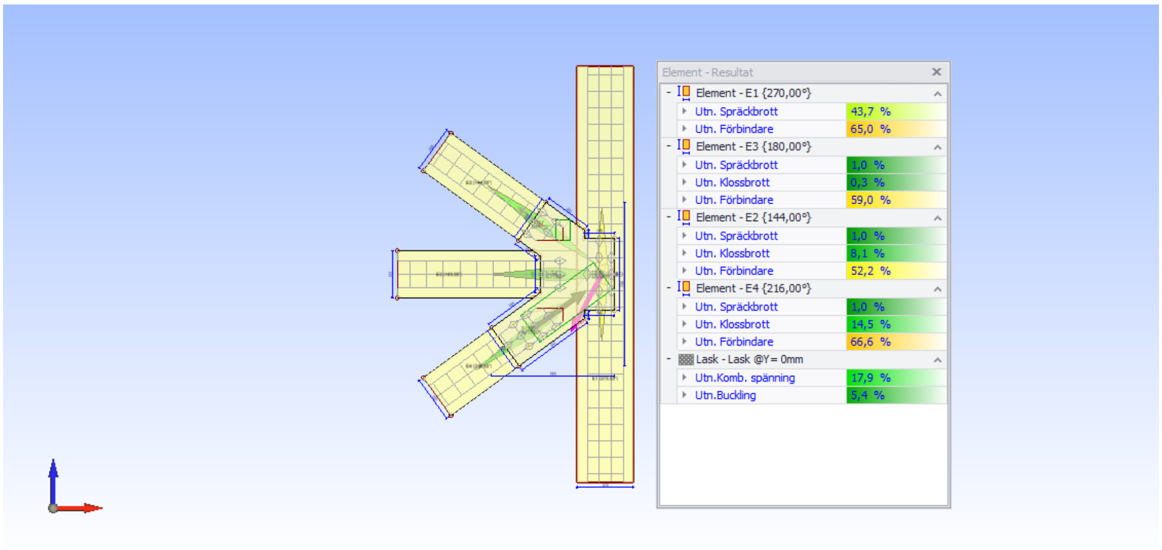


Appendix B

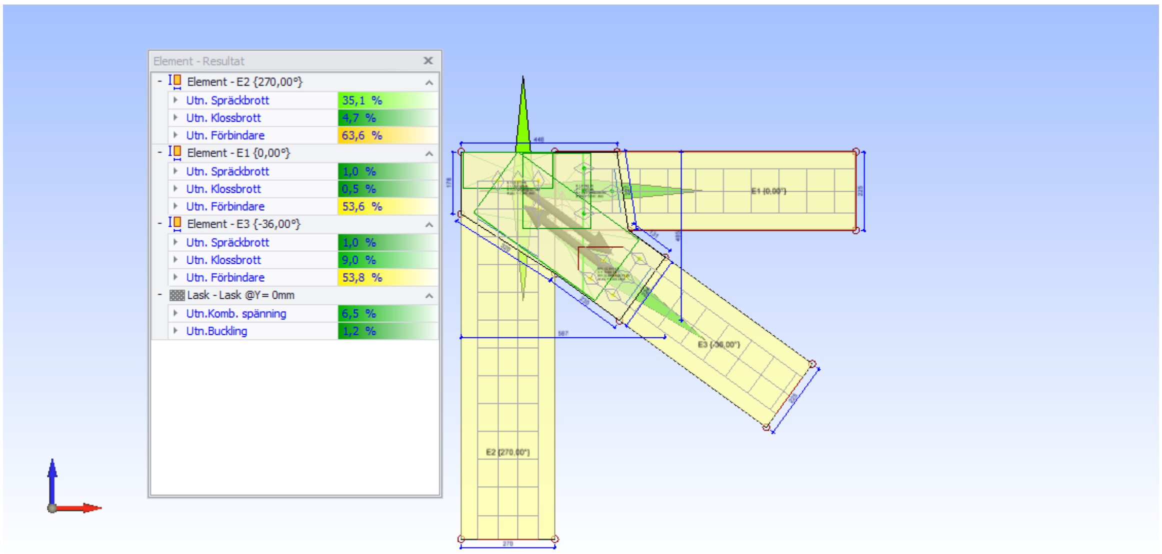
Connection E



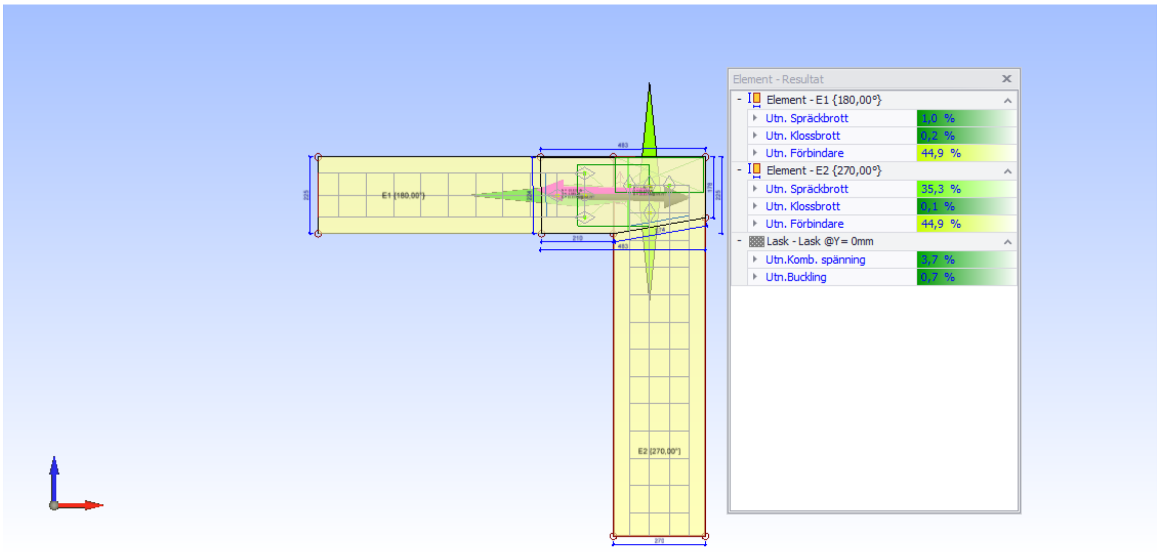
Connection F



Connection G

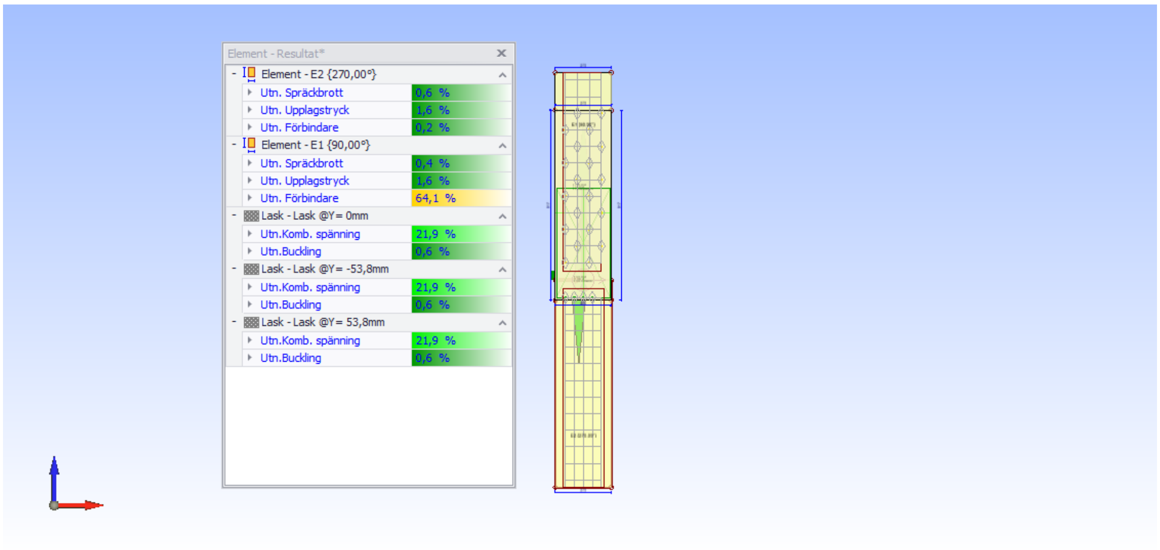


Connection H

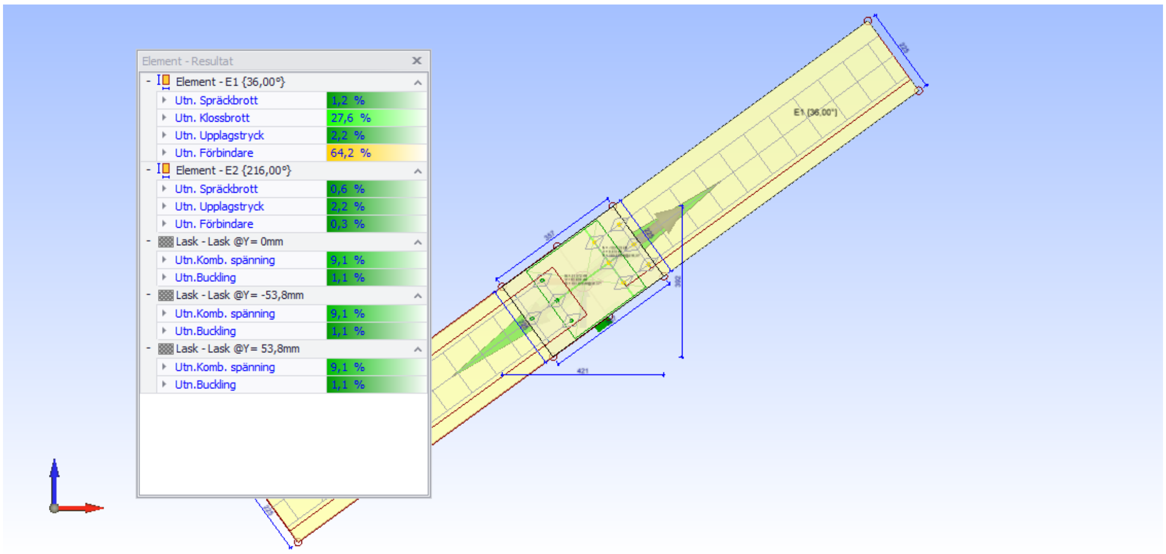


Truss 3

Connection A, vertical

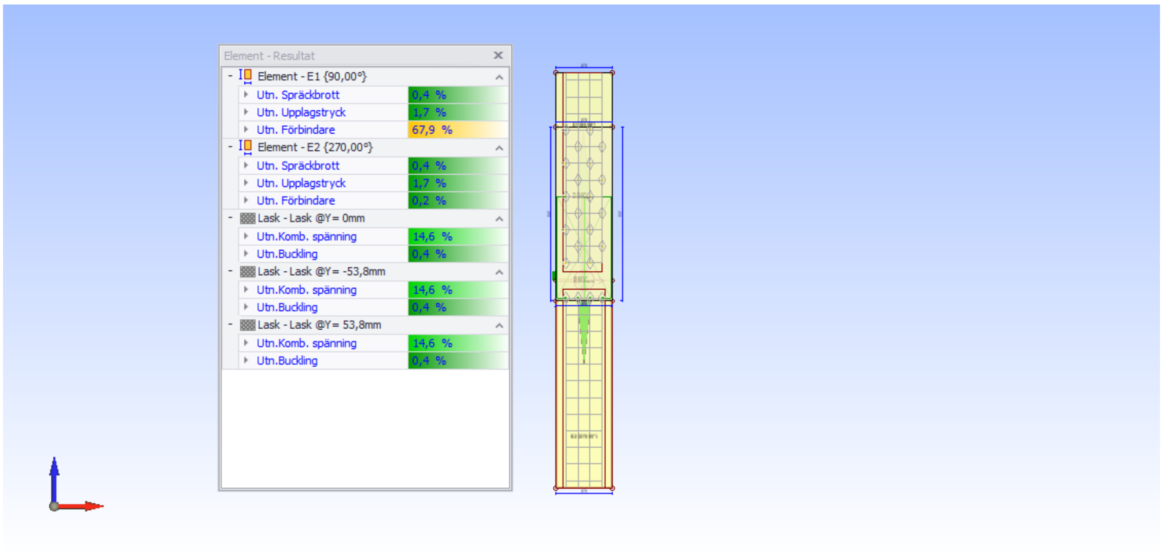


Connection A, diagonal

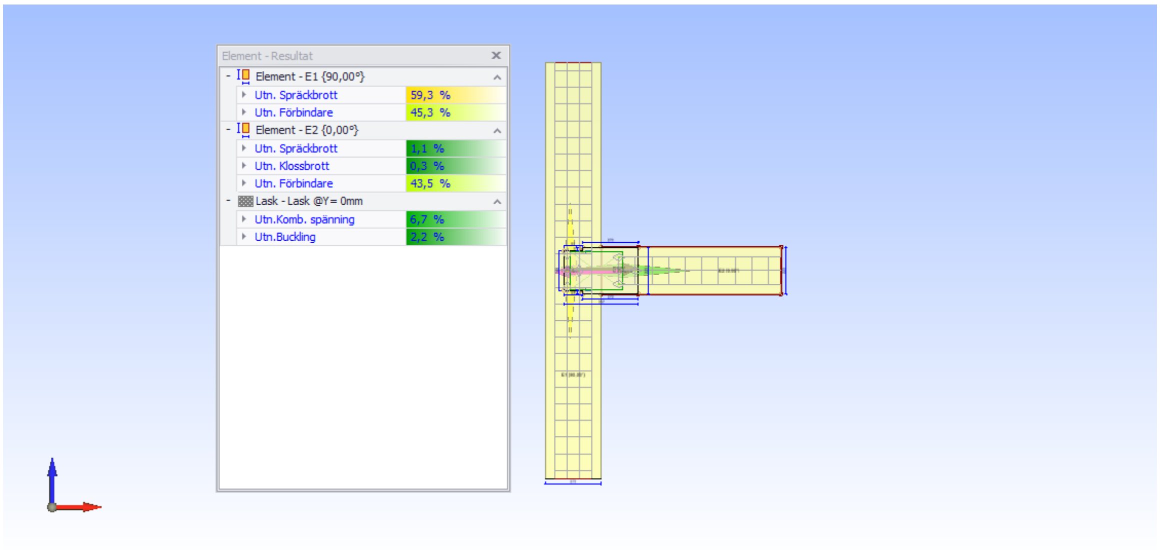


Appendix B

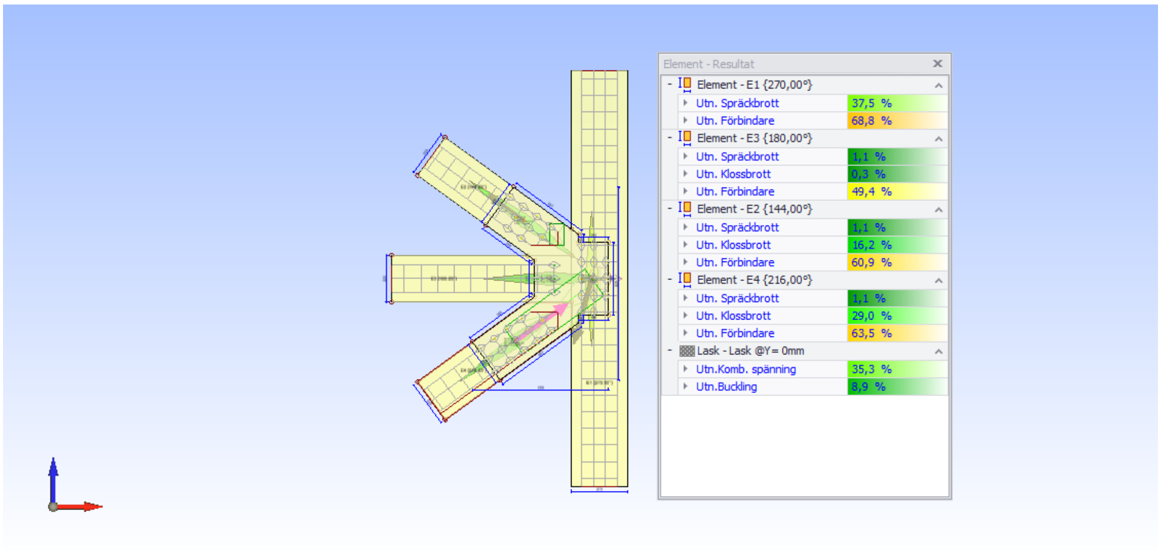
Connection B



Connection C

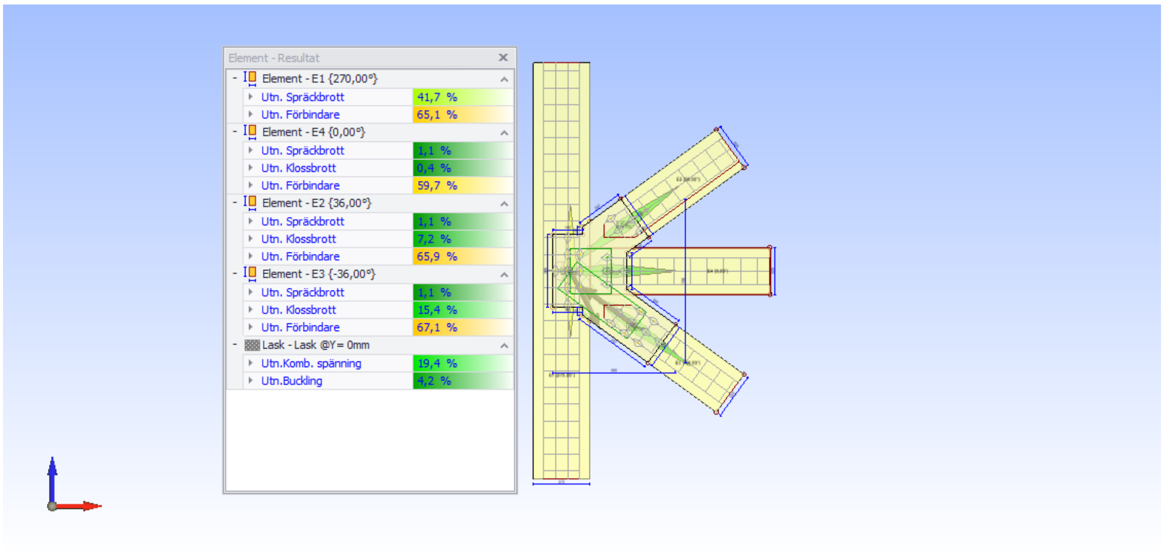


Connection D

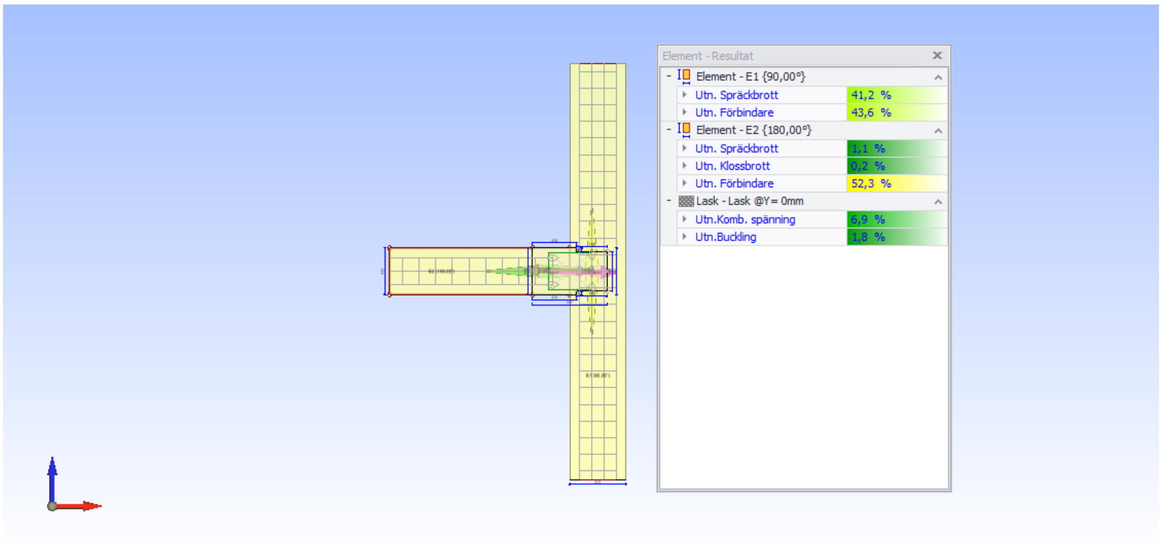


Appendix B

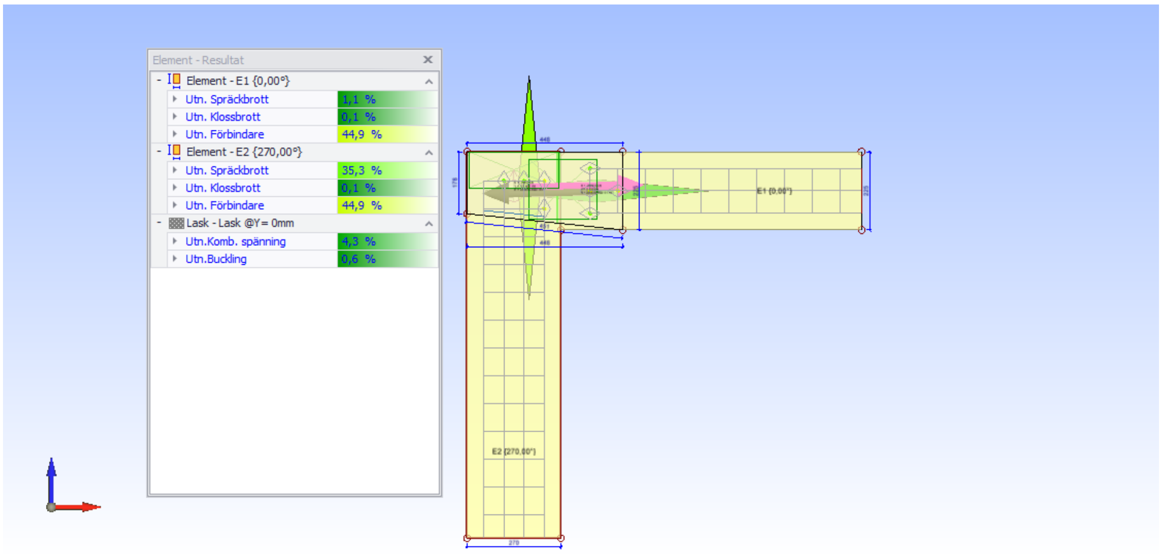
Connection E



Connection F

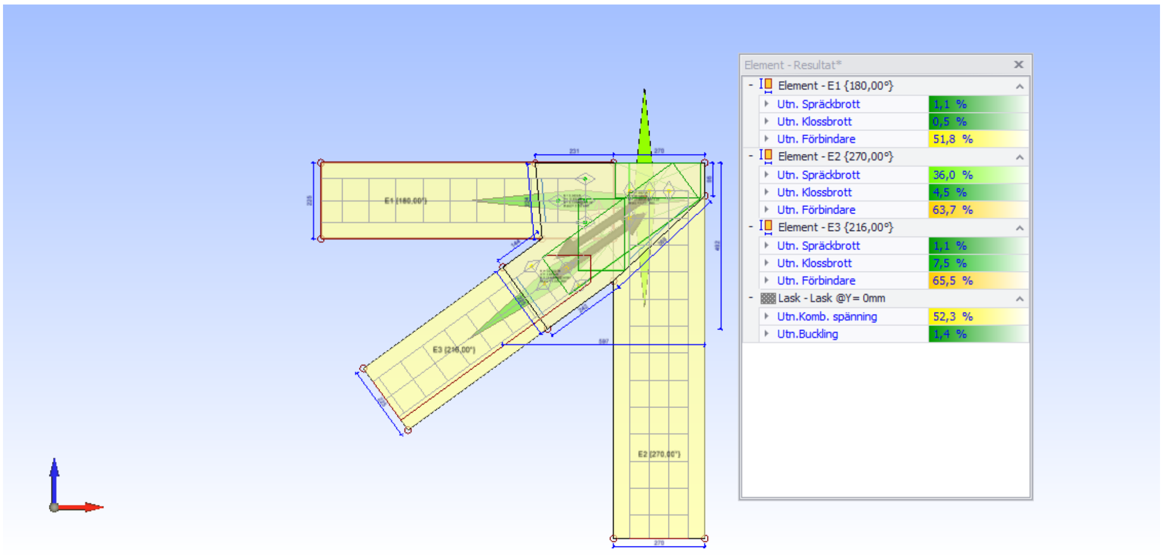


Connection G



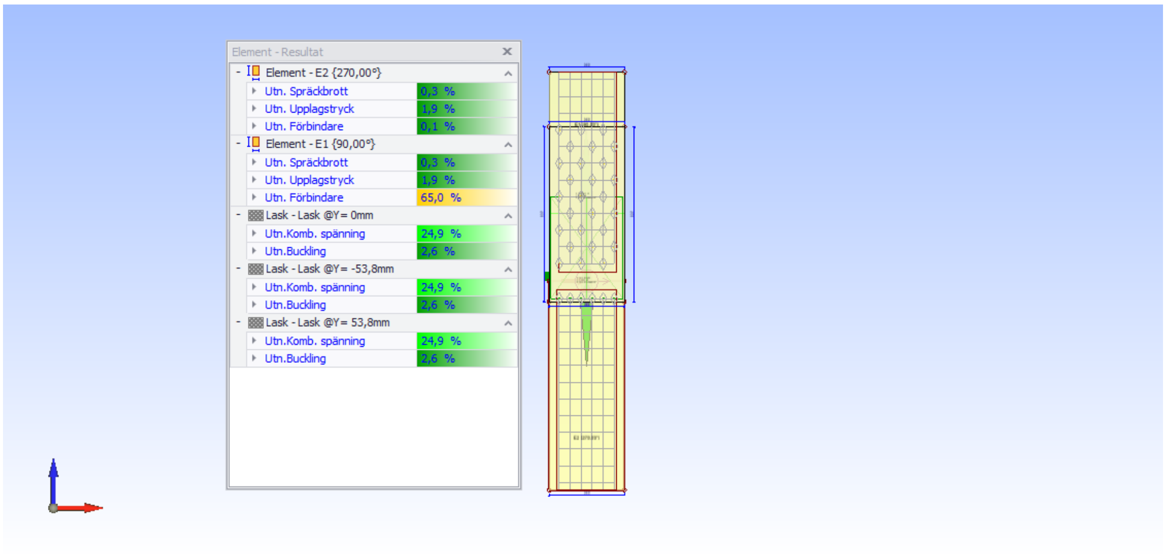
Appendix B

Connection H

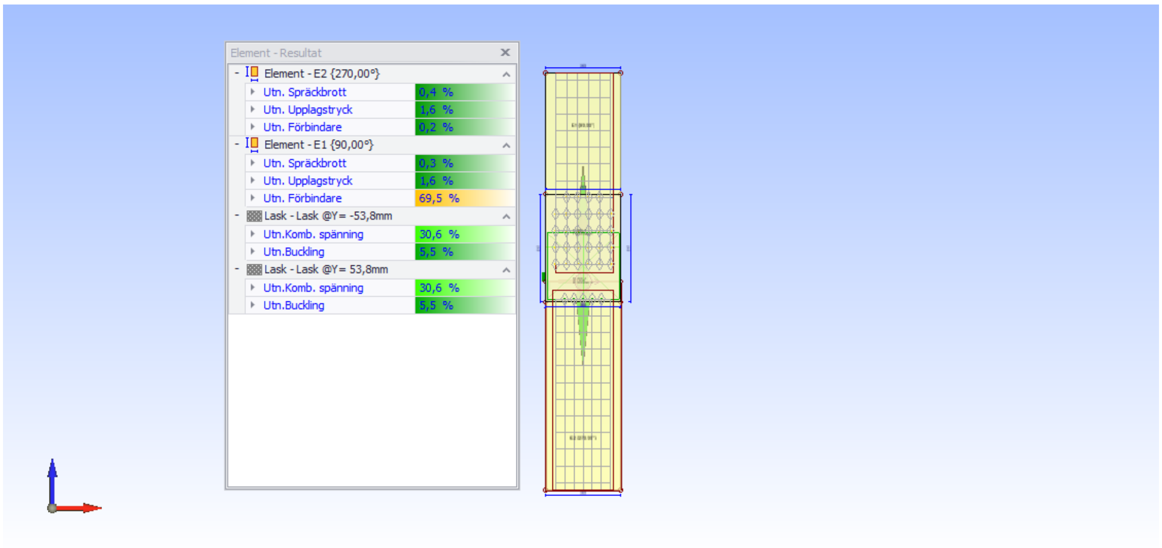


Truss 4

Connection A

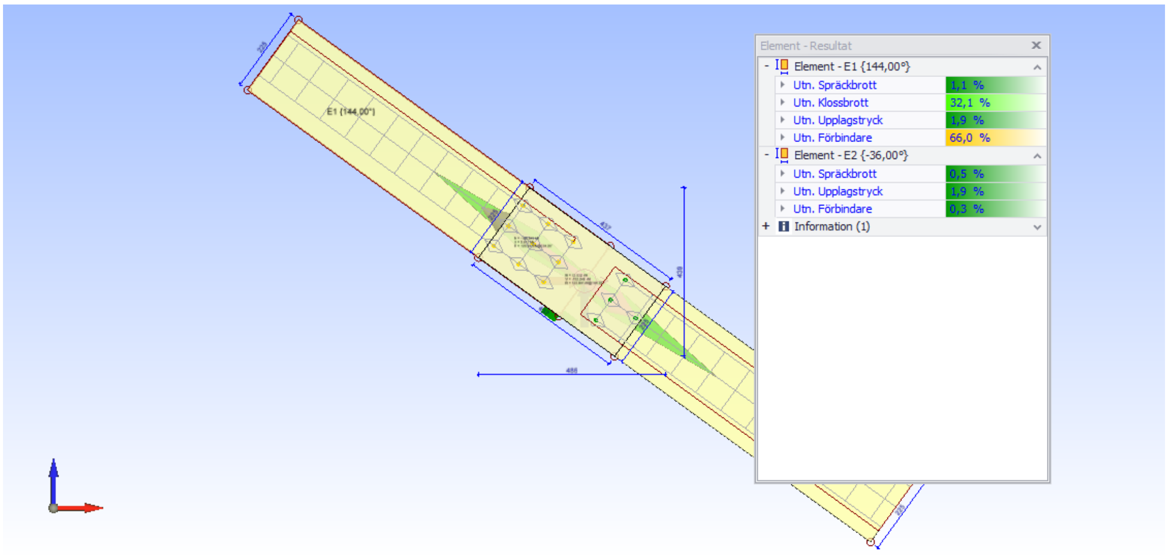


Connection B, vertical

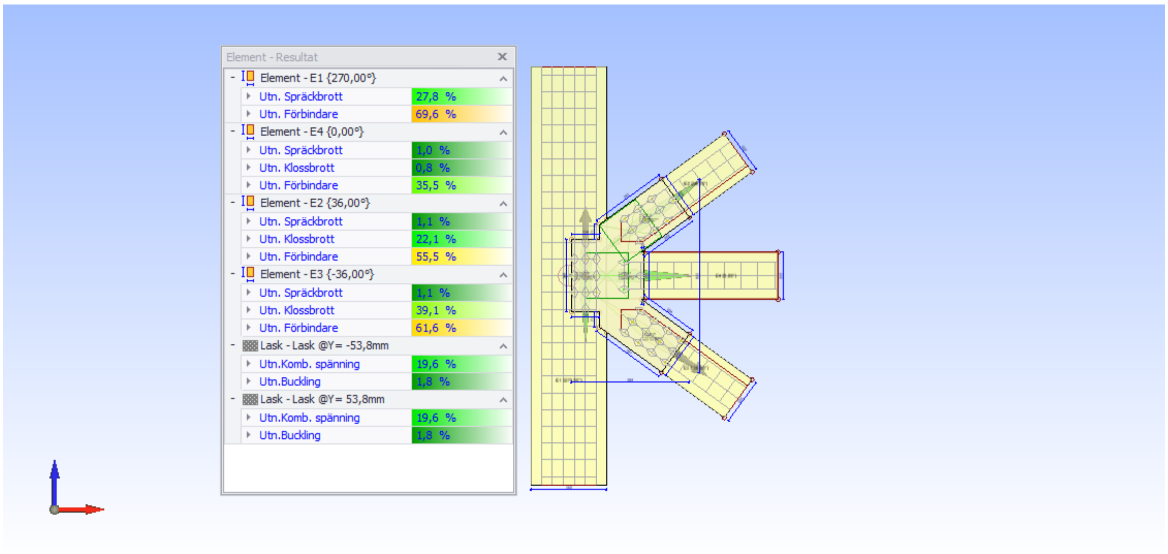


Appendix B

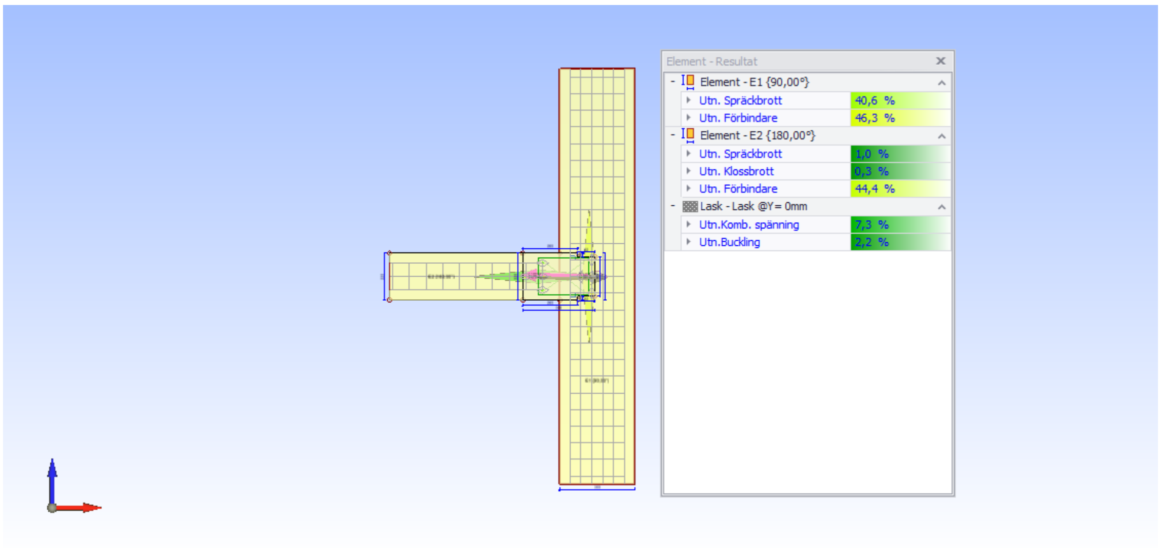
Connection B, diagonal



Connection C

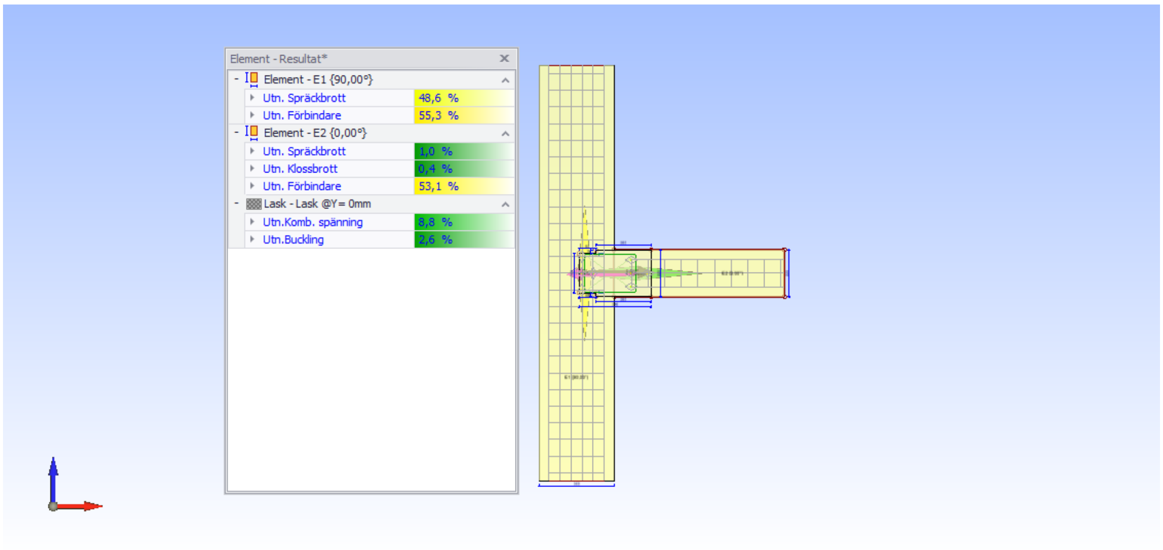


Connection D

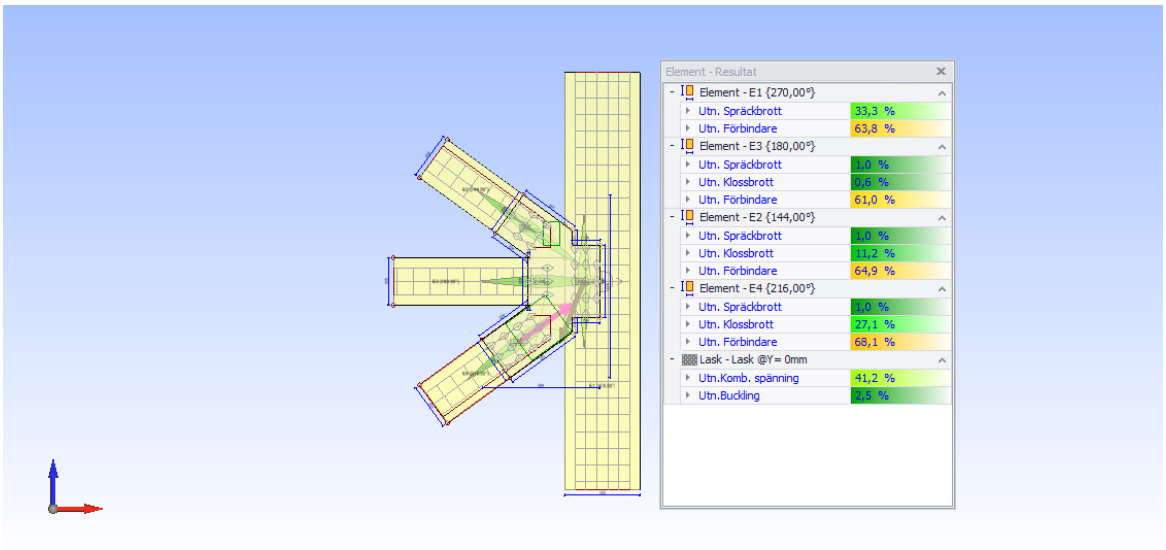


Appendix B

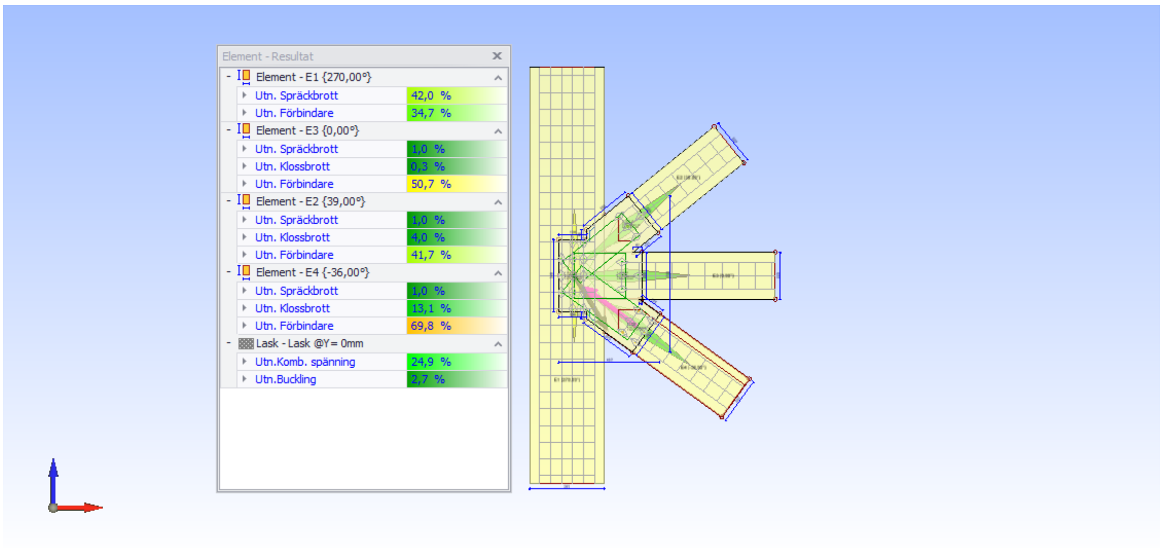
Connection E



Connection F

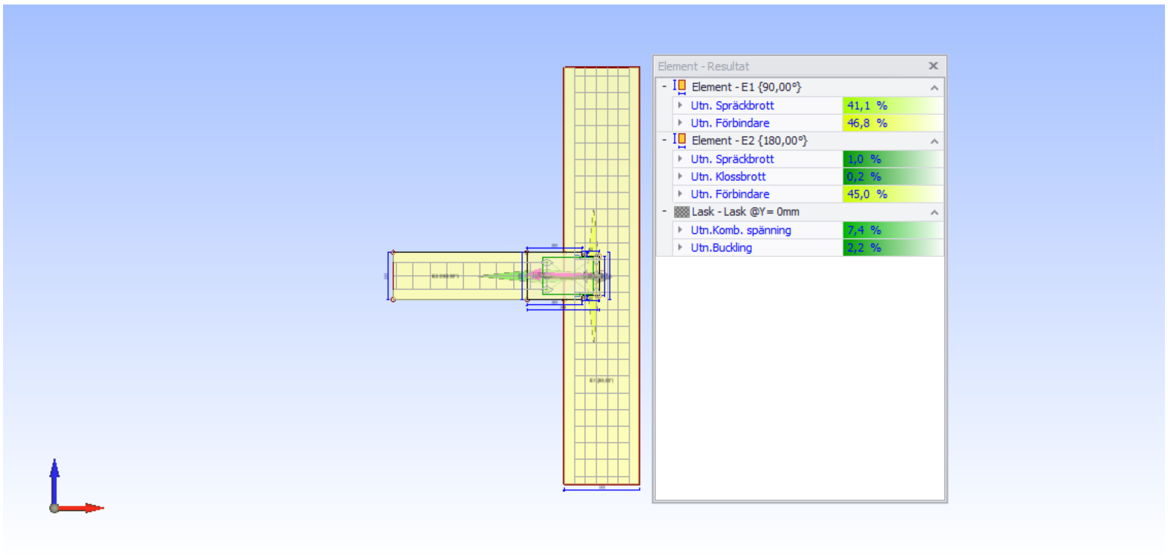


Connection G

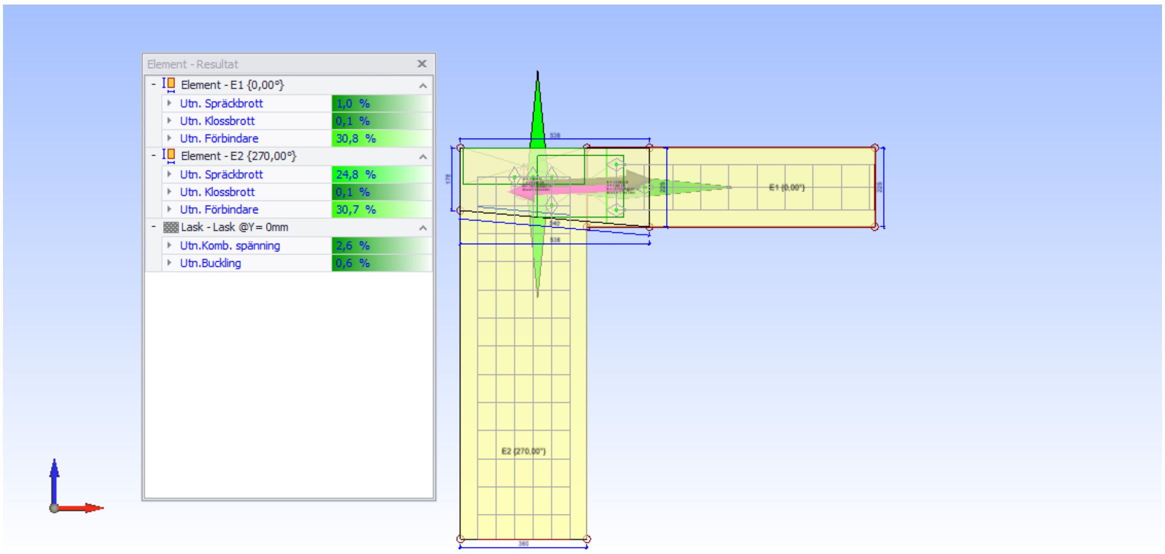


Appendix B

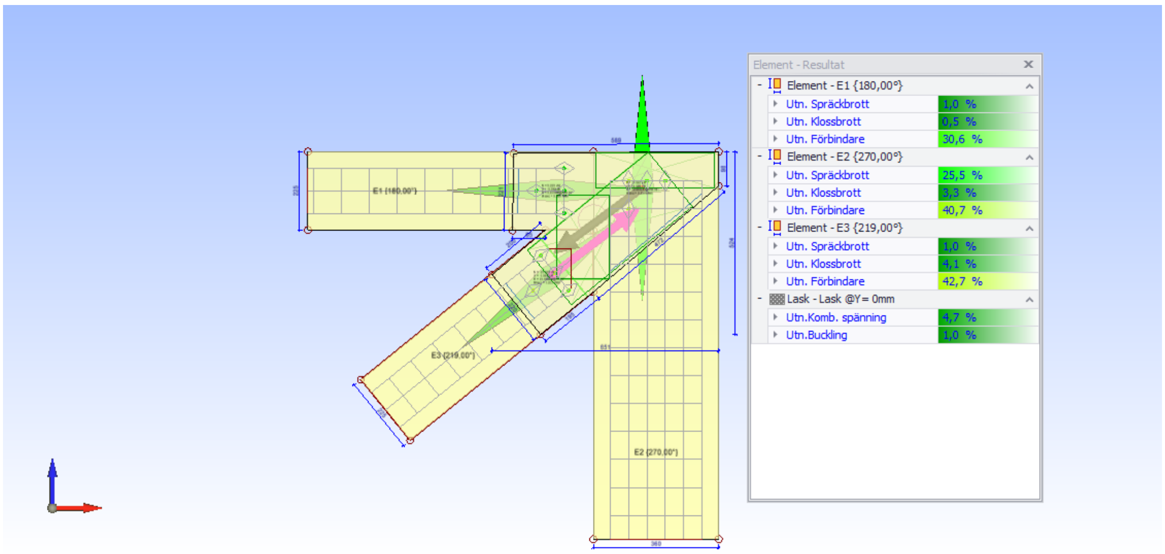
Connection H



Connection I

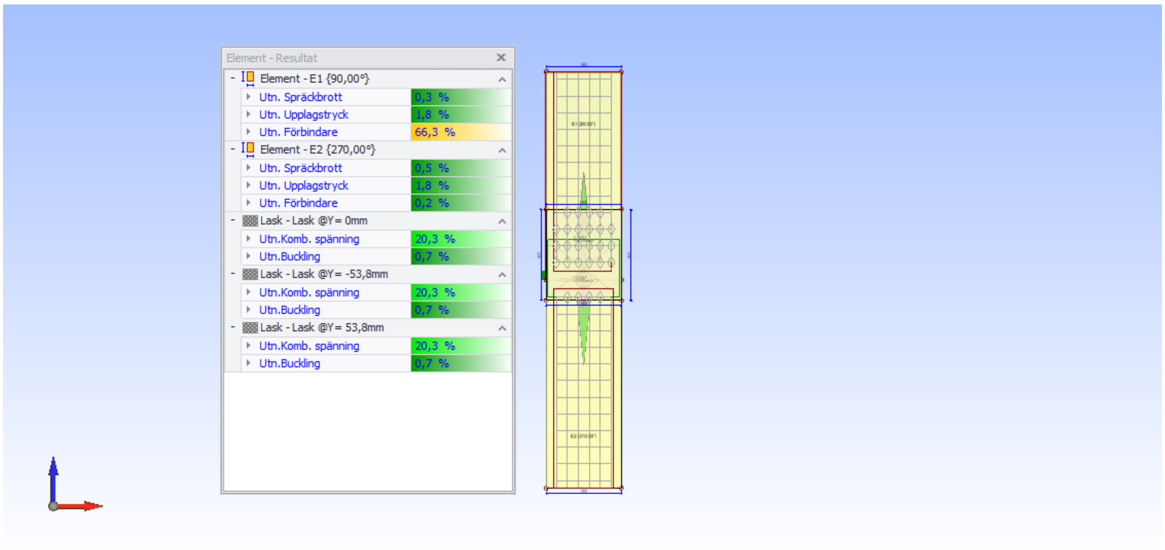


Connection J

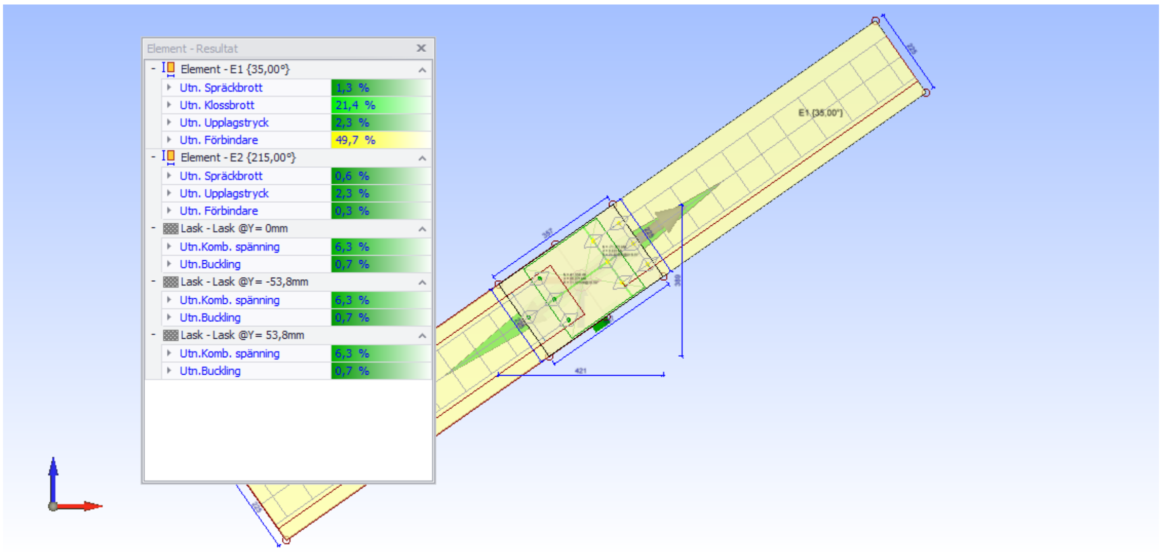


Truss 5

Connection A, vertical

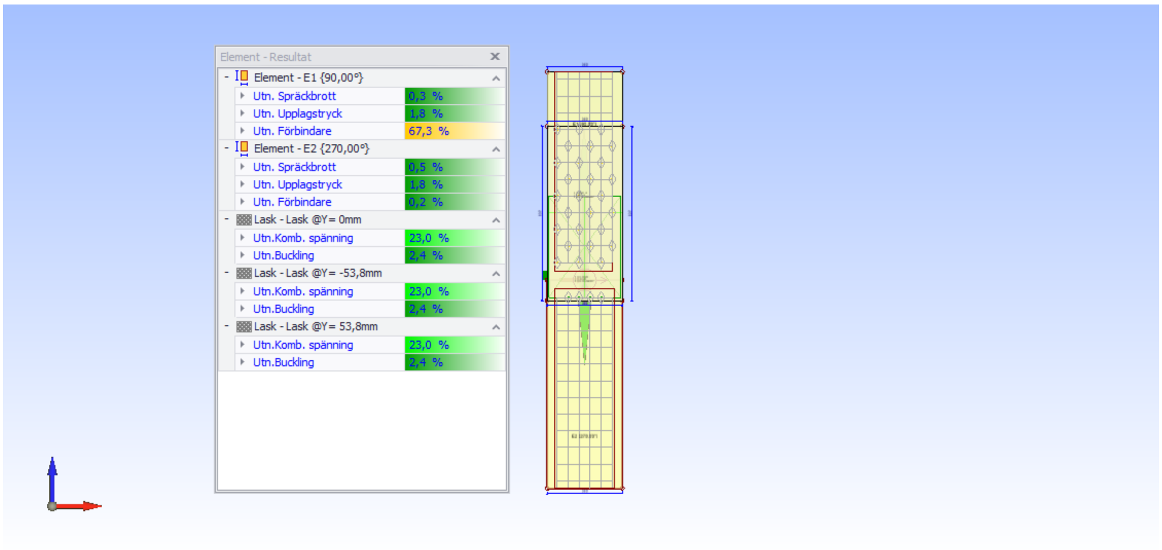


Connection A, diagonal

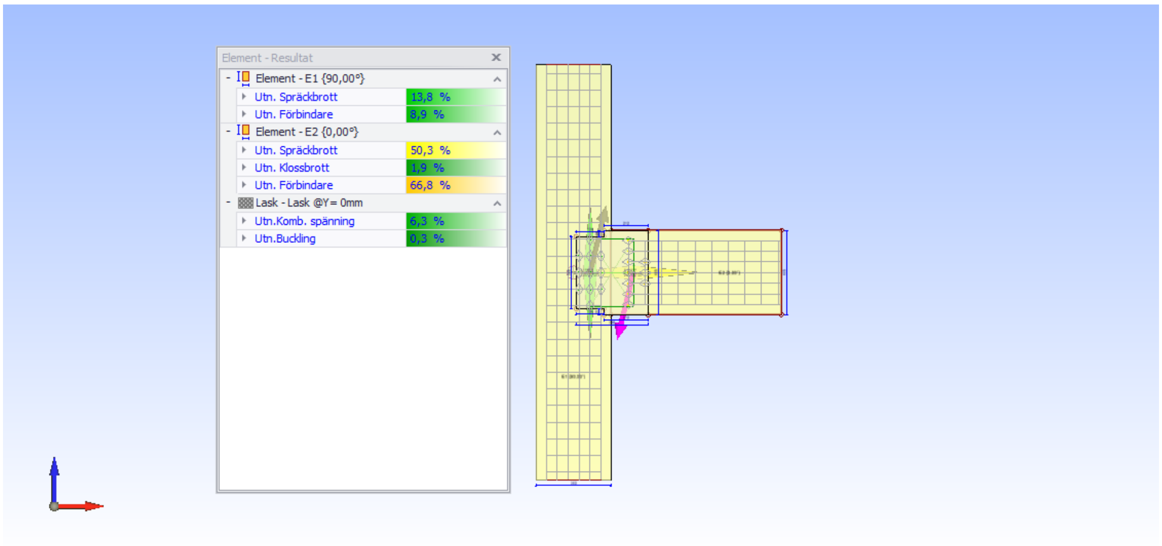


Appendix B

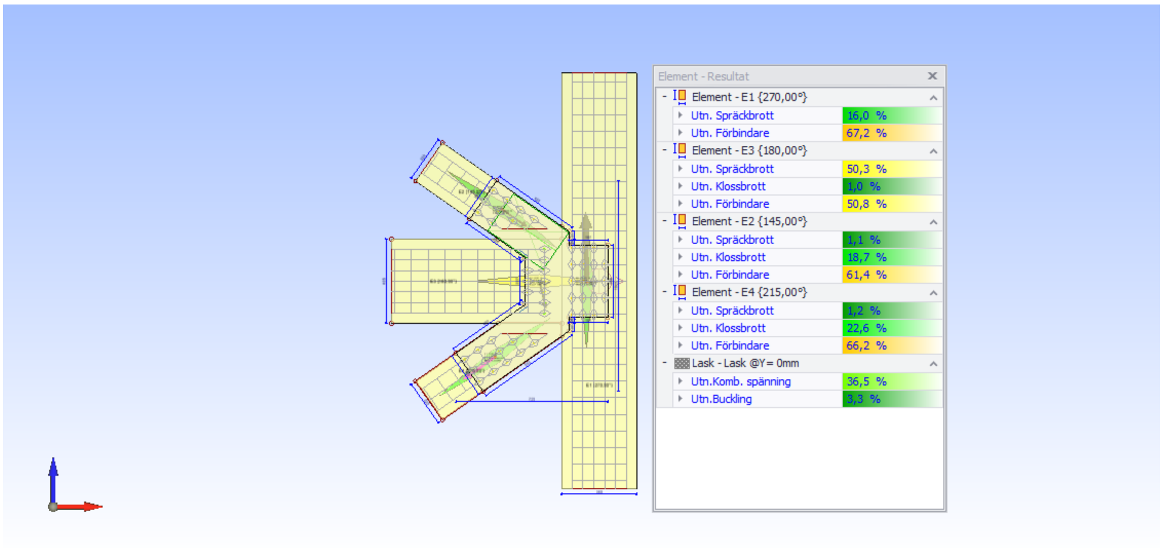
Connection B



Connection C

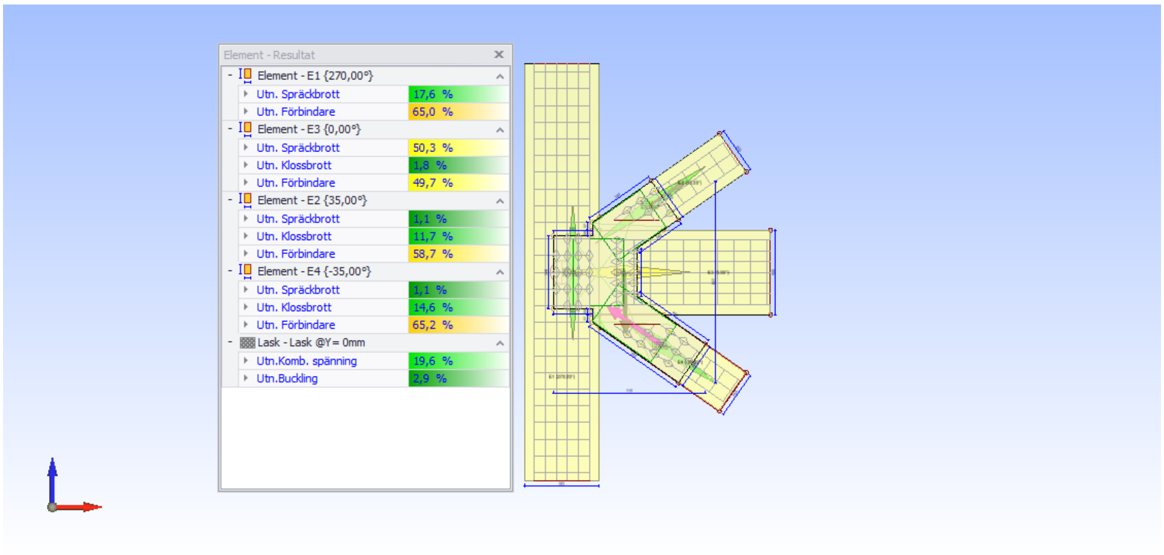


Connection D

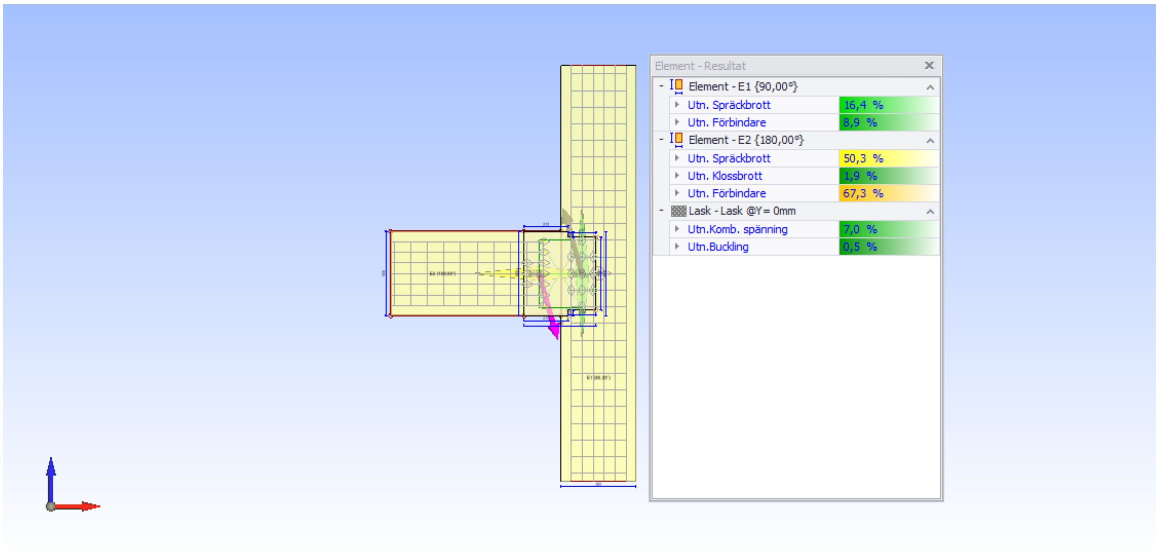


Appendix B

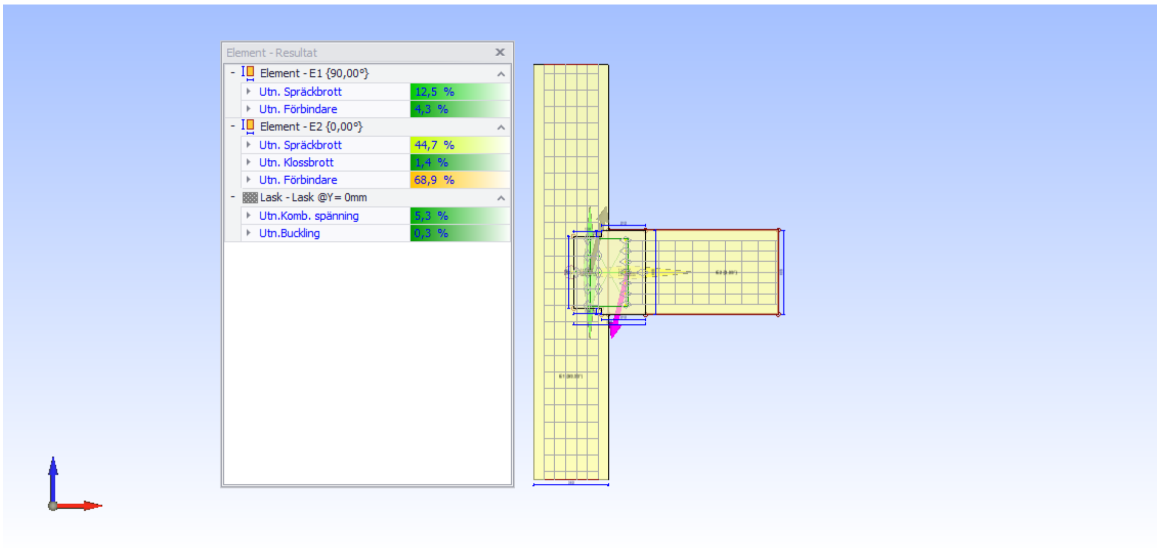
Connection E



Connection F

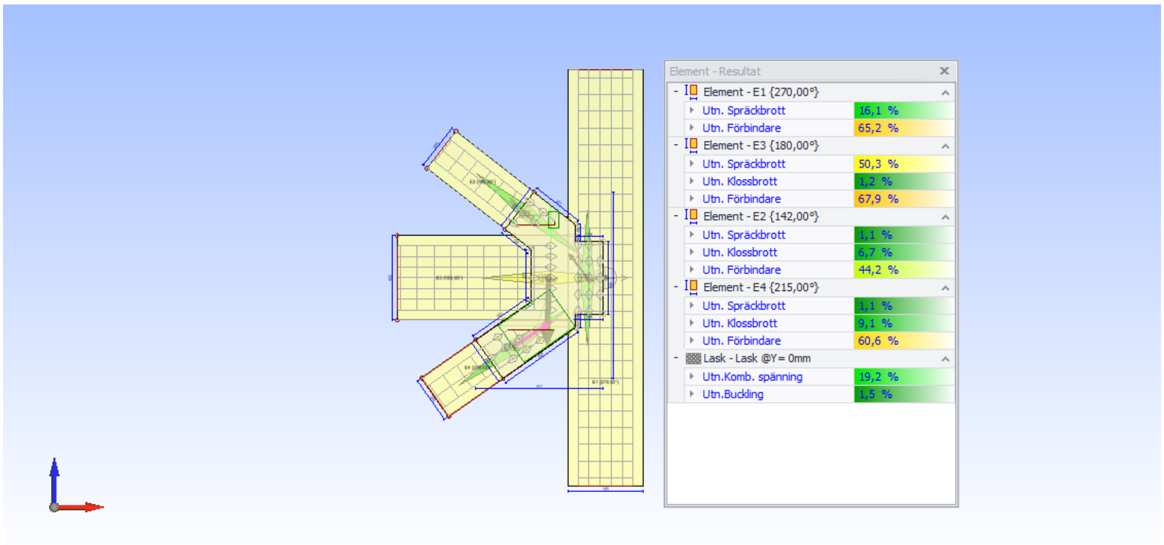


Connection G

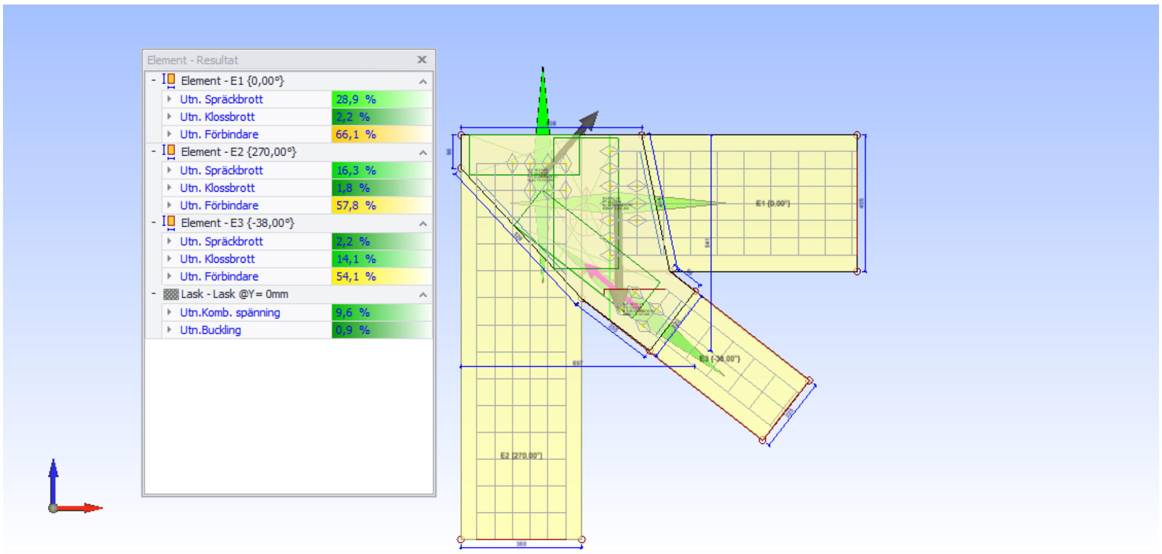


Appendix B

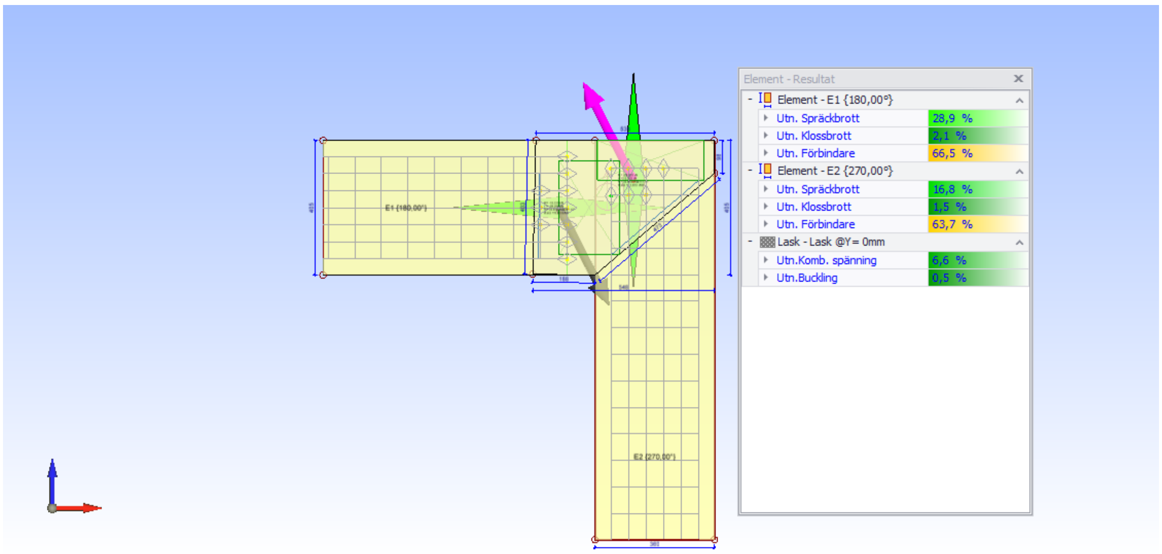
Connection H



Connection I

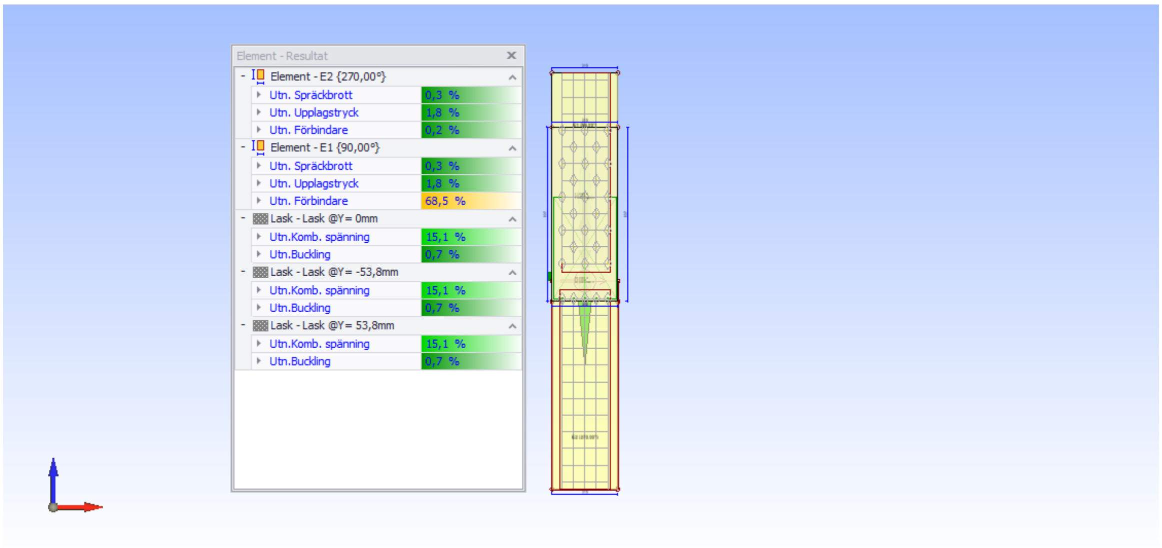


Connection J

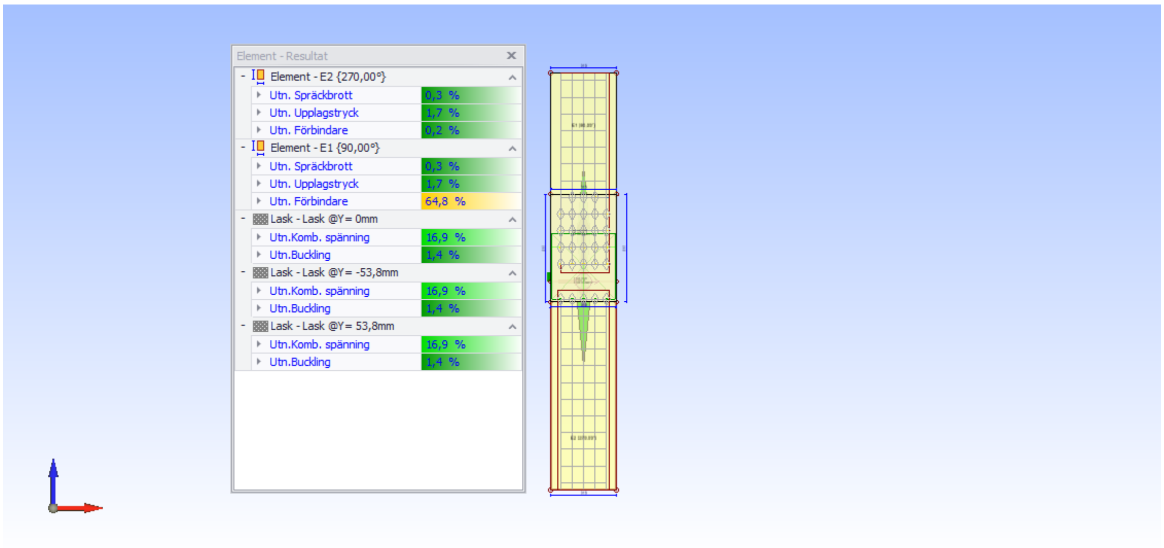


Truss 6

Connection A

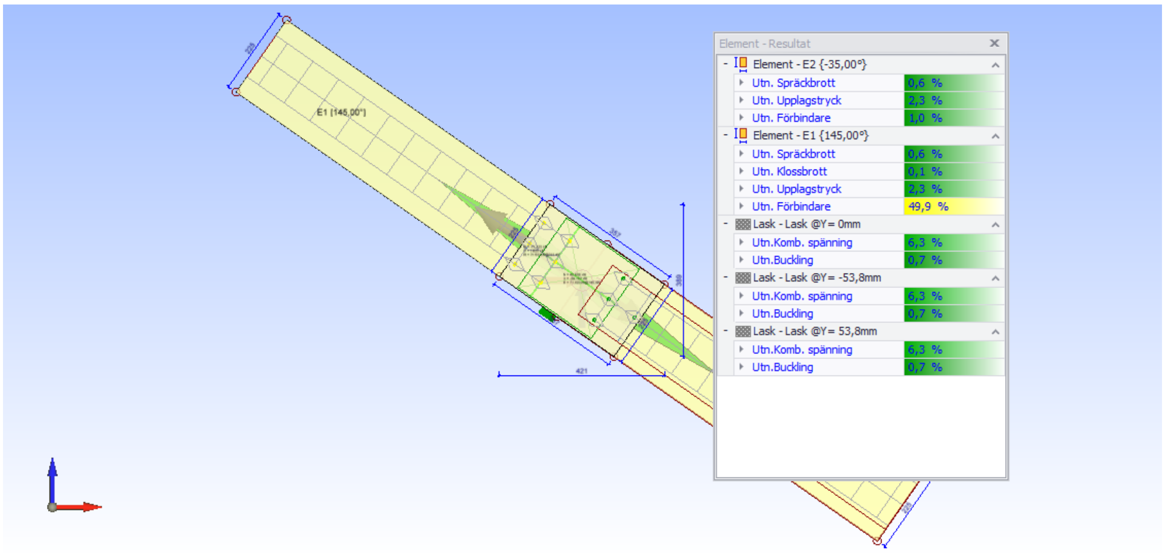


Connection B, vertical

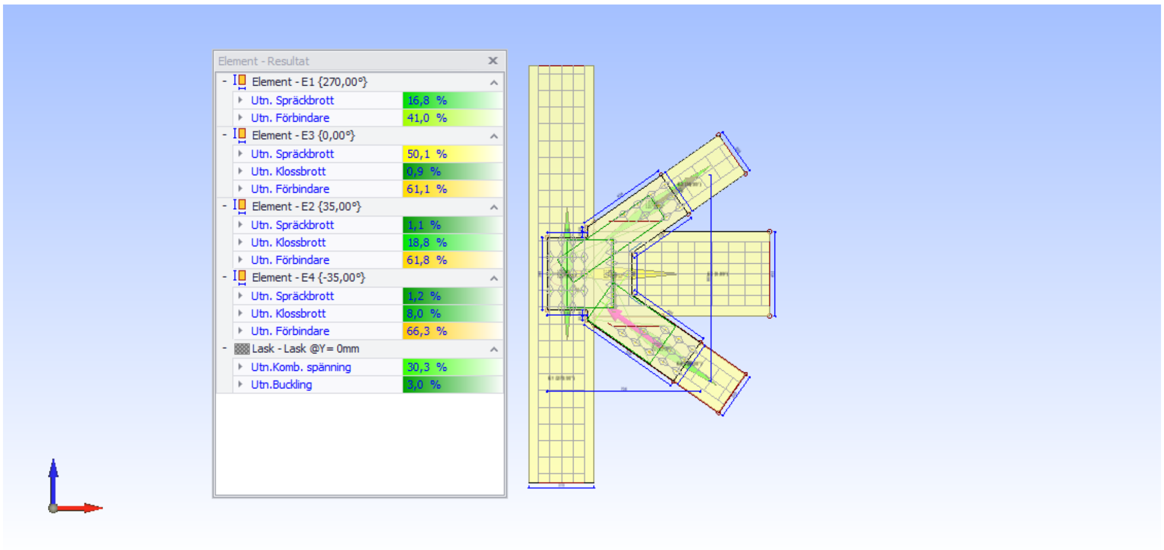


Appendix B

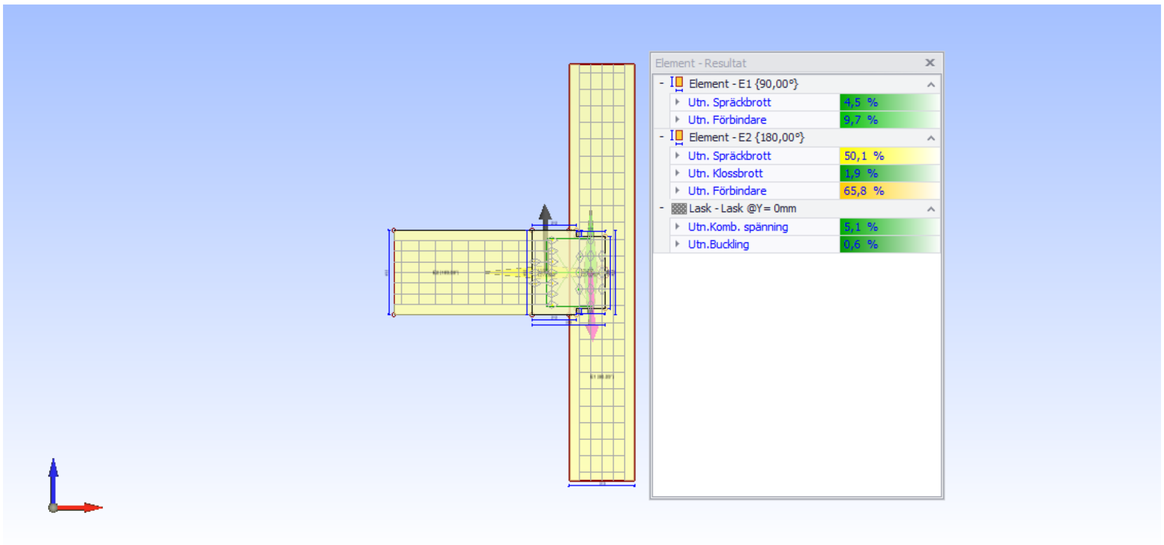
Connection B, diagonal



Connection C

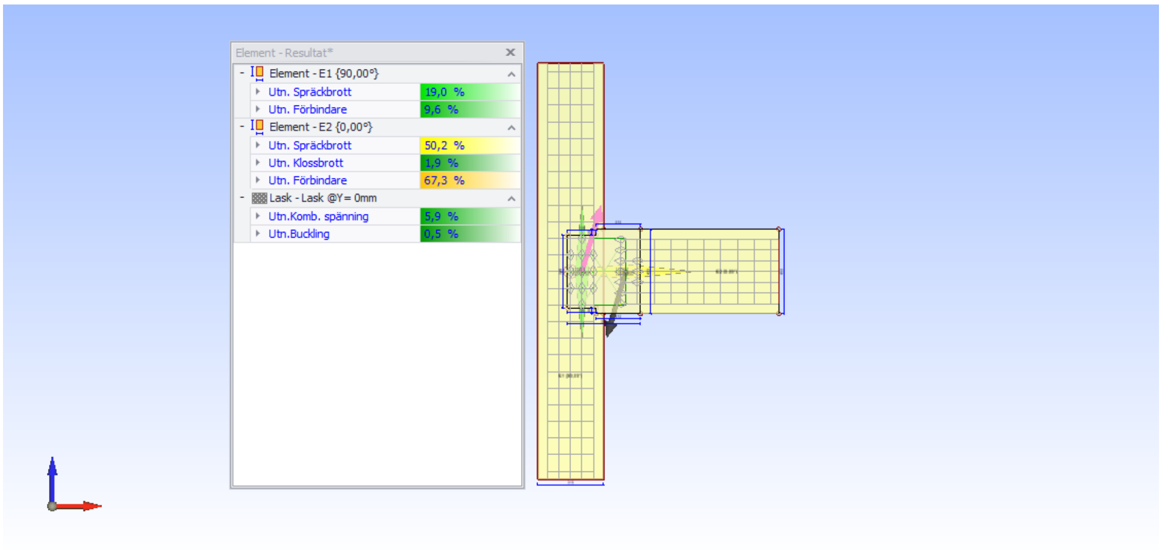


Connection D

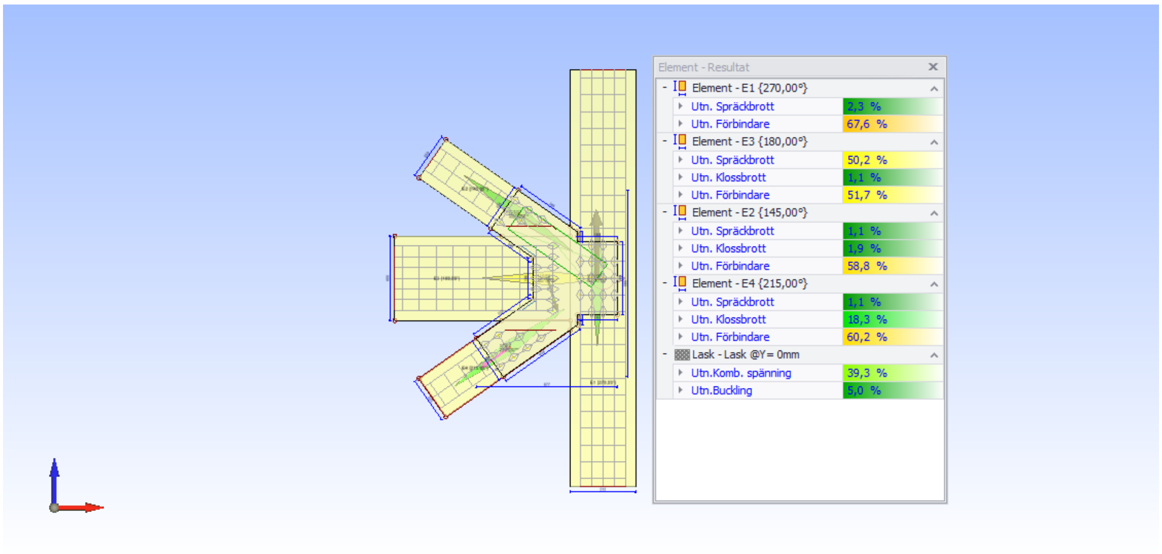


Appendix B

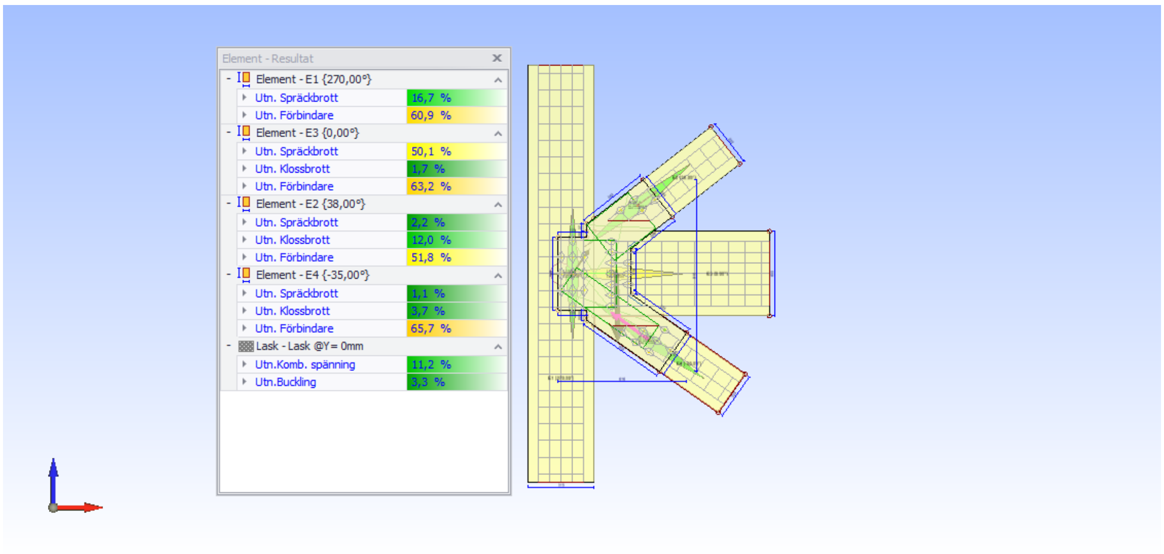
Connection E



Connection F

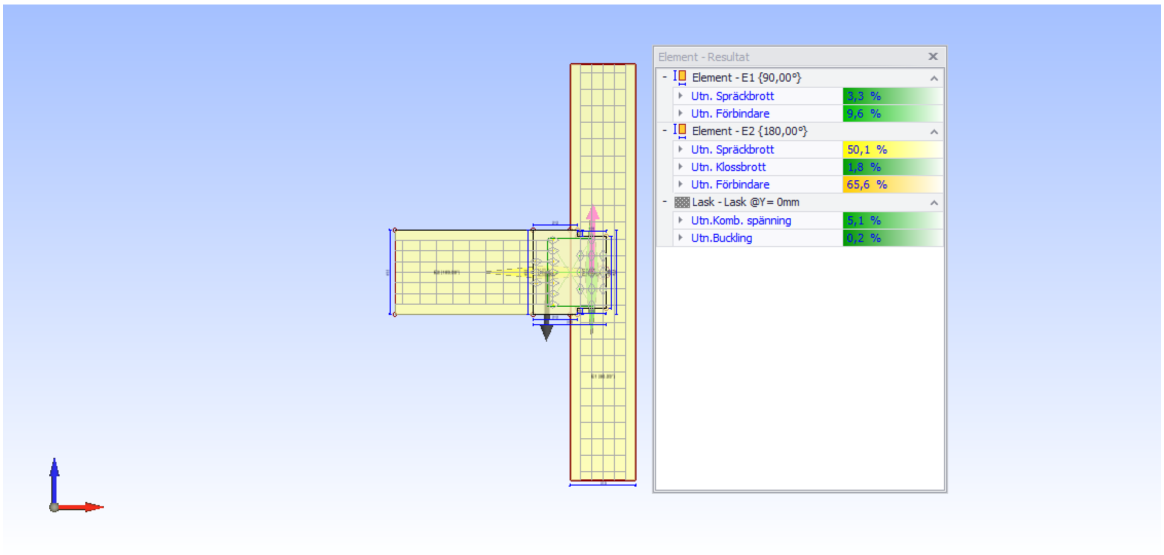


Connection G

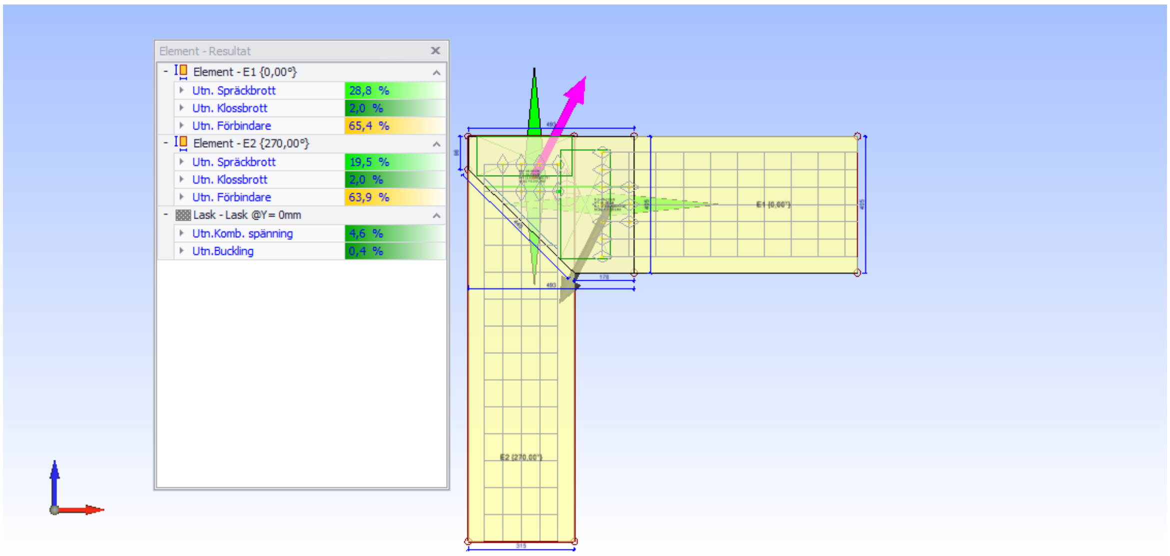


Appendix B

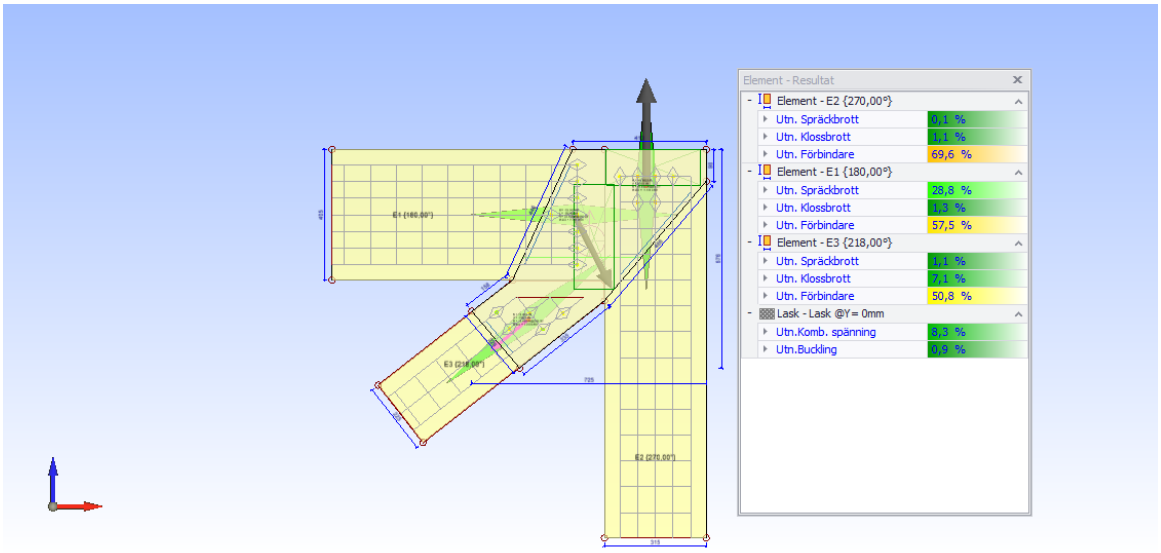
Connection H



Connection I

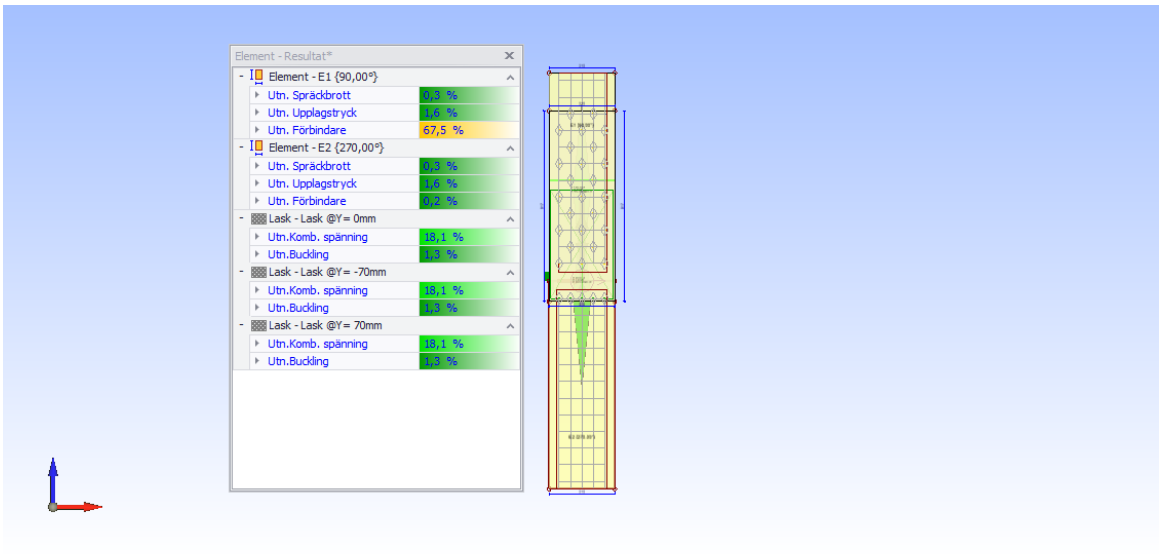


Connection J

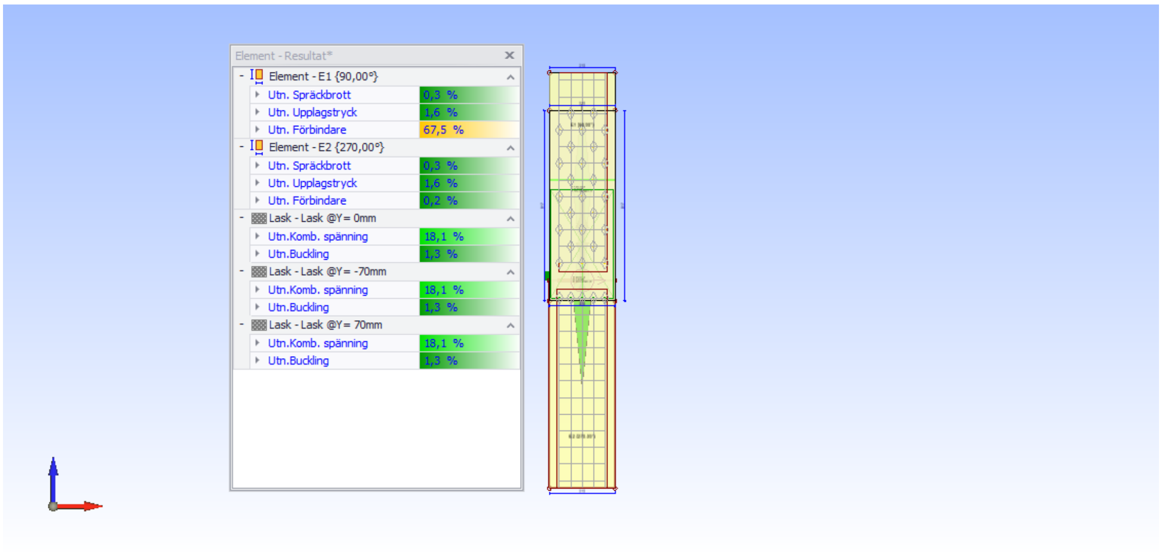


Truss 7

Connection A

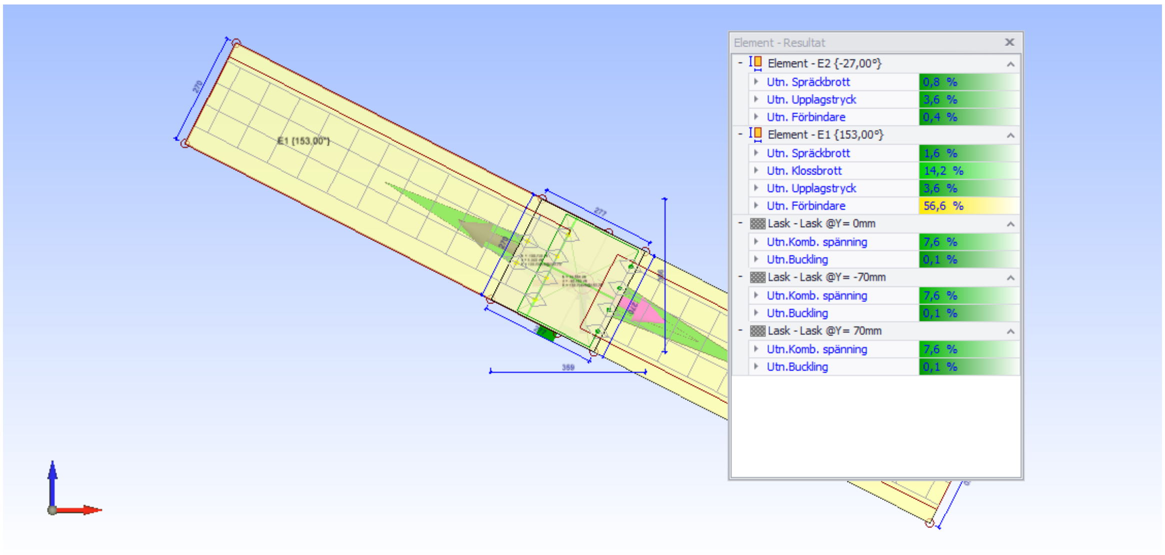


Connection B, vertical

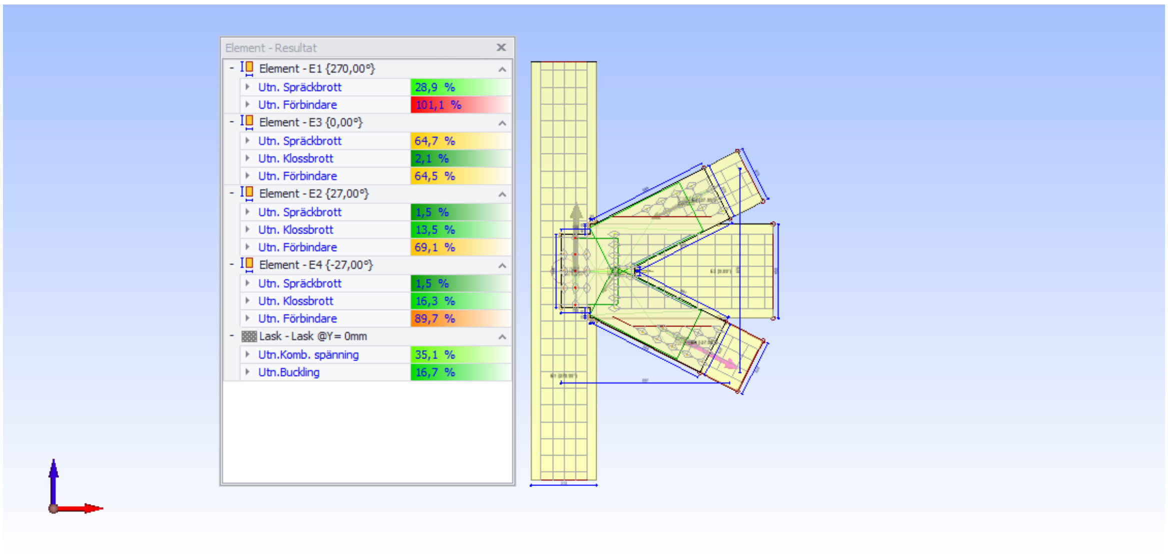


Appendix B

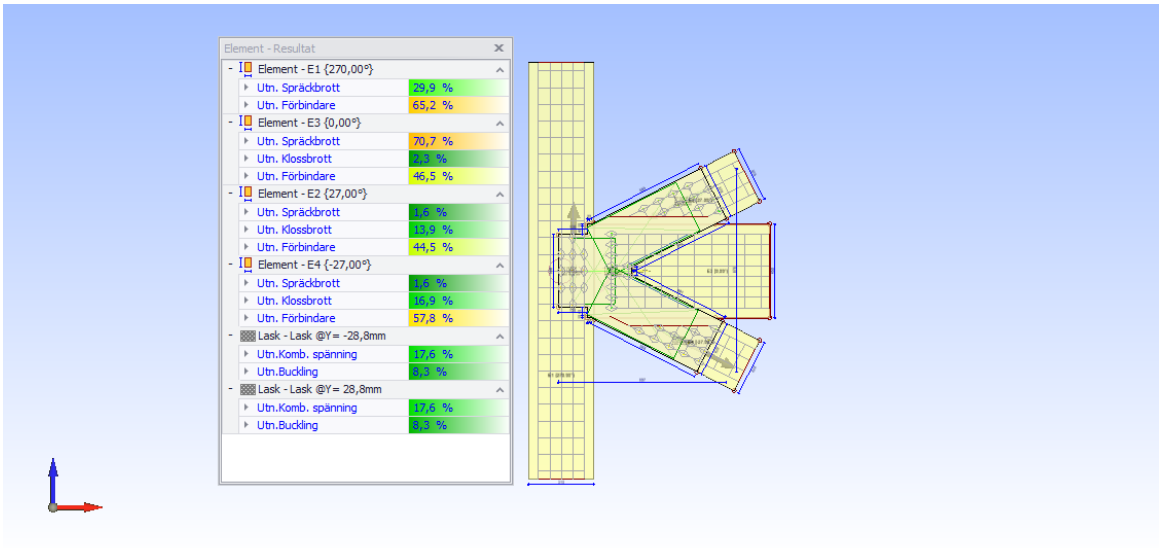
Connection B, diagonal



Connection C, horzontal member – 1 plate

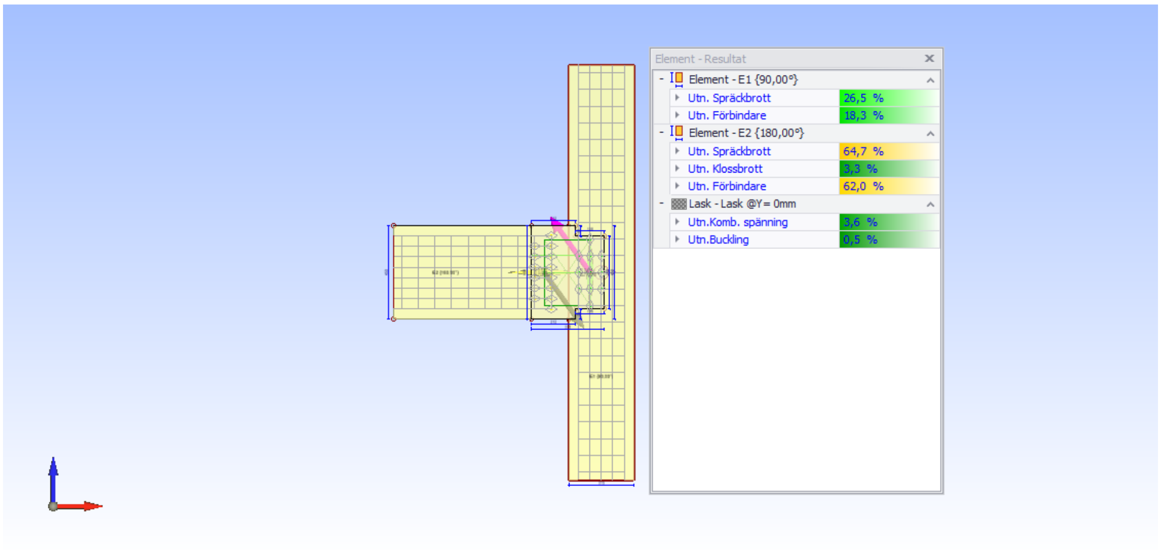


Connection C, vertical and diagonal members – 2 plates

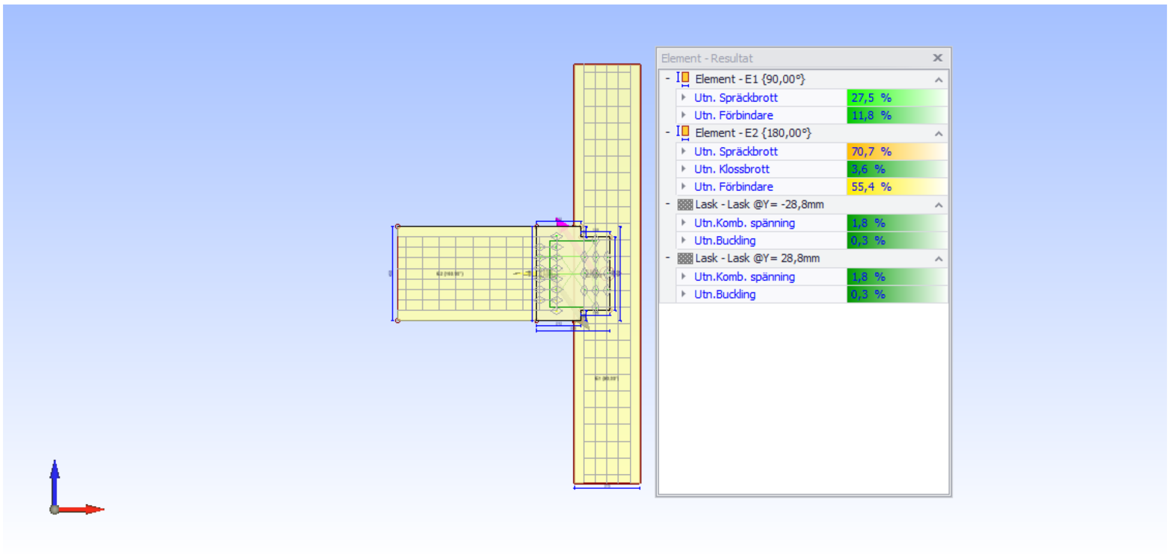


Appendix B

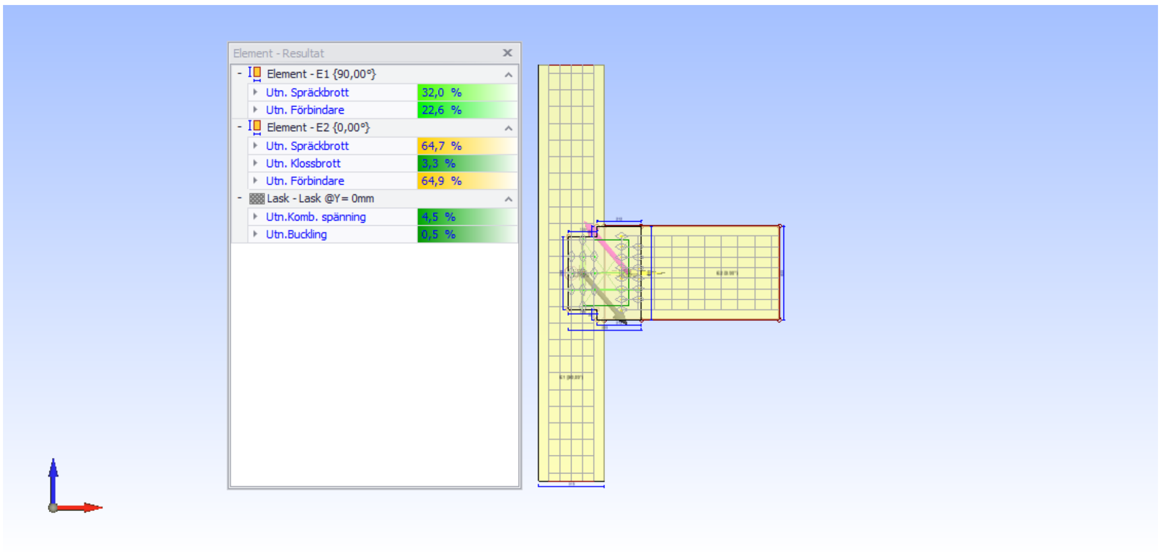
Connection D, horizontal member – 1 plate



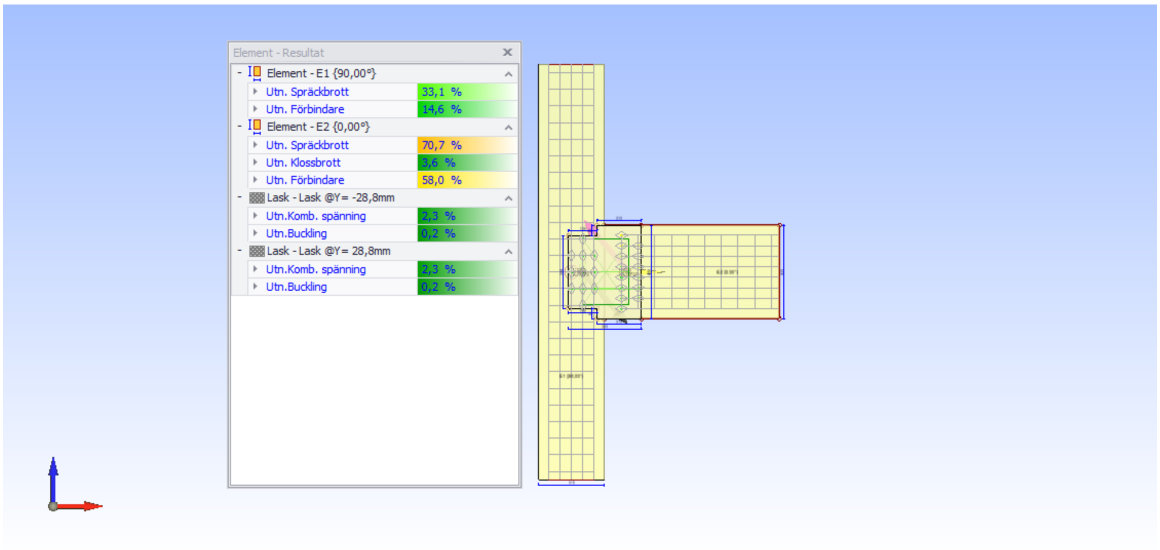
Connection D, vertical member – 2 plates



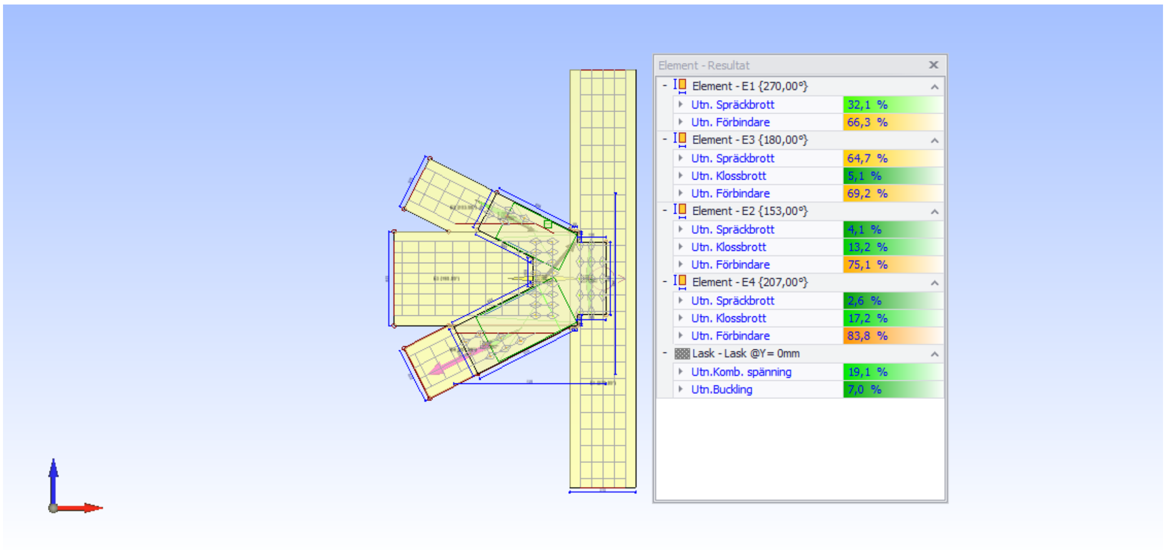
Connection E, horizontal member – 1 plate



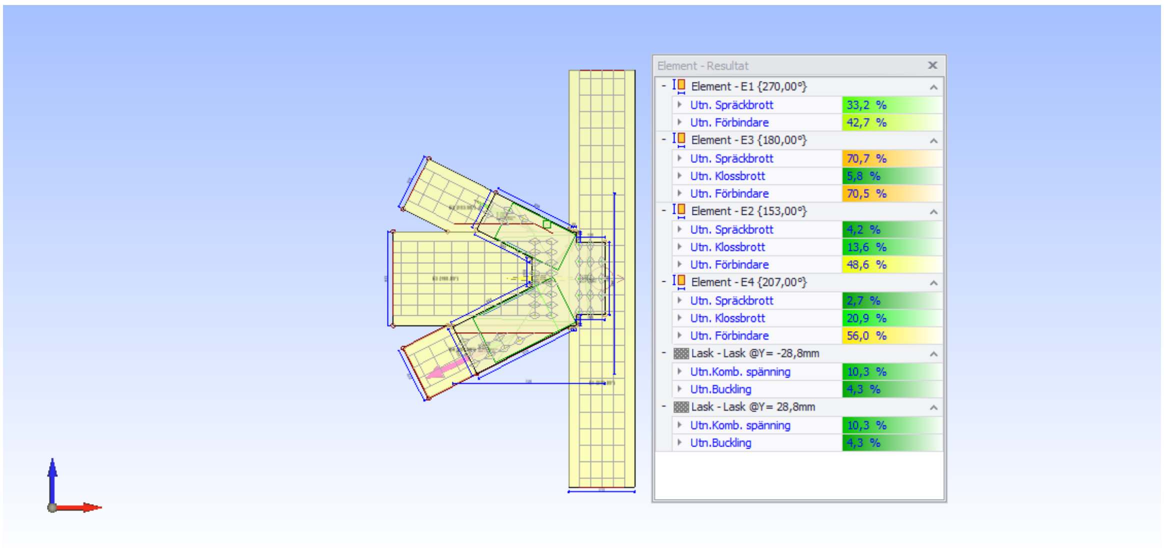
Connection E, vertical member – 2 plates



Connection F, horizontal member – 1 plate

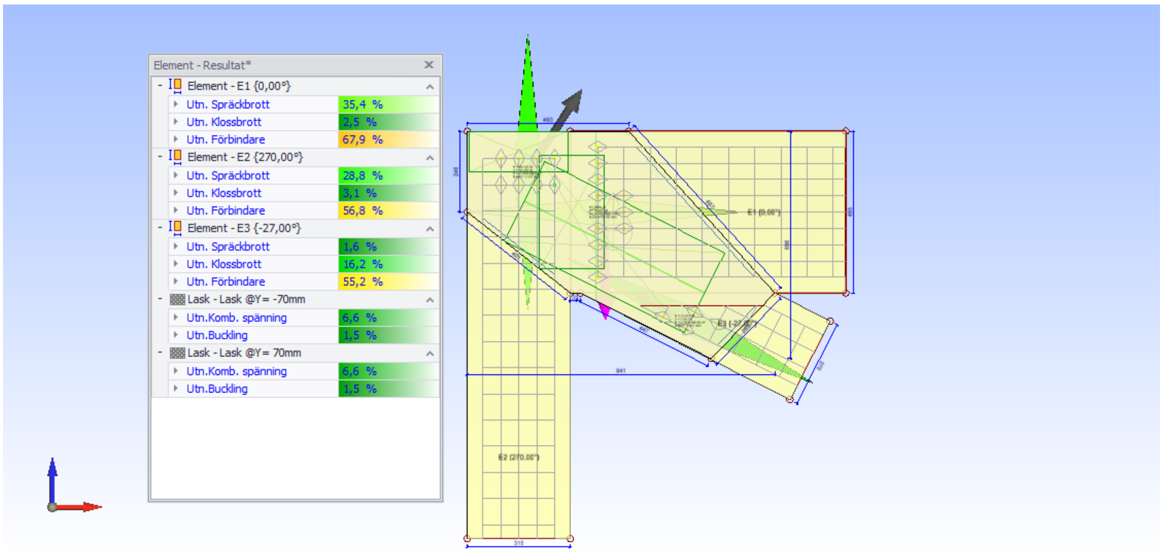


Connection F, vertical and diagonal members – 2 plates

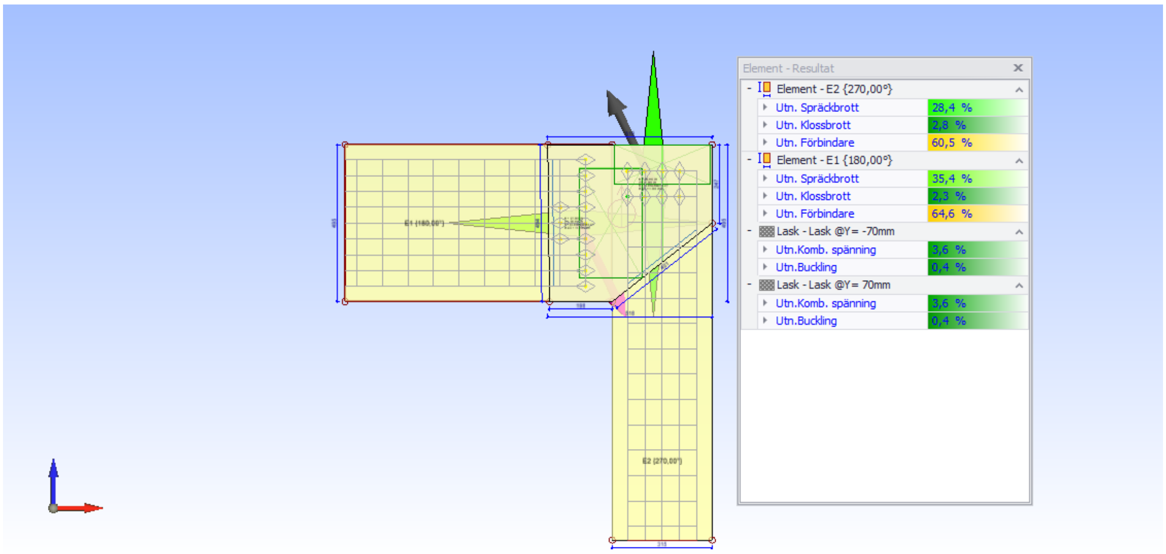


Appendix B

Connection G



Connection H



B.3 Results from static analysis

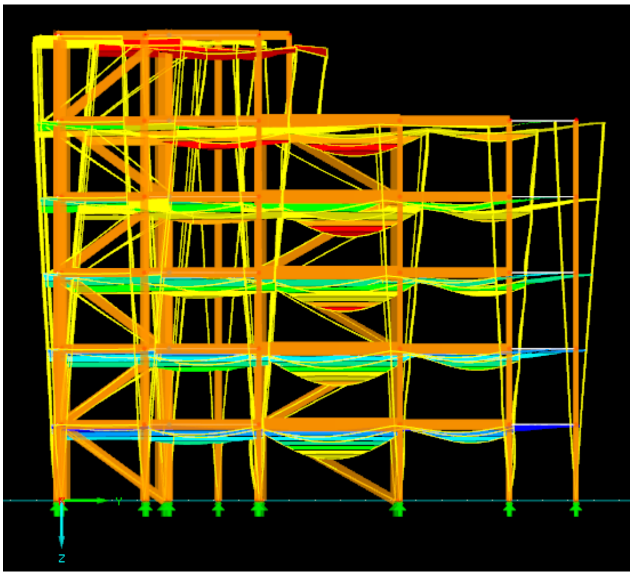
Results from six storey building

k=0,8

Displacements in y

$u_{y,max}$: 13,7 mm

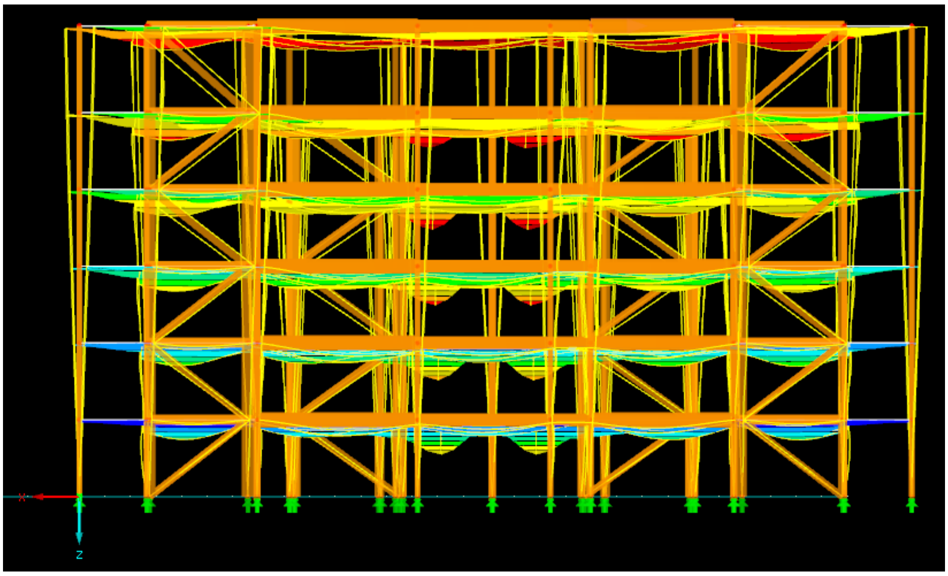
$u_{y,min}$: -8,6 mm



Displacements in x

$u_{x,max}$: 5,9 mm

$u_{x,min}$: -6,1 mm

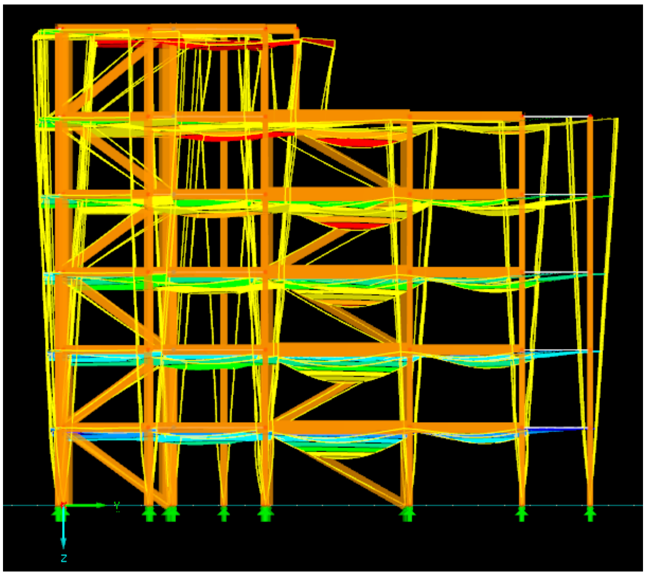


k=0,35

Displacements in y

$u_{y,max}$: 16,2 mm

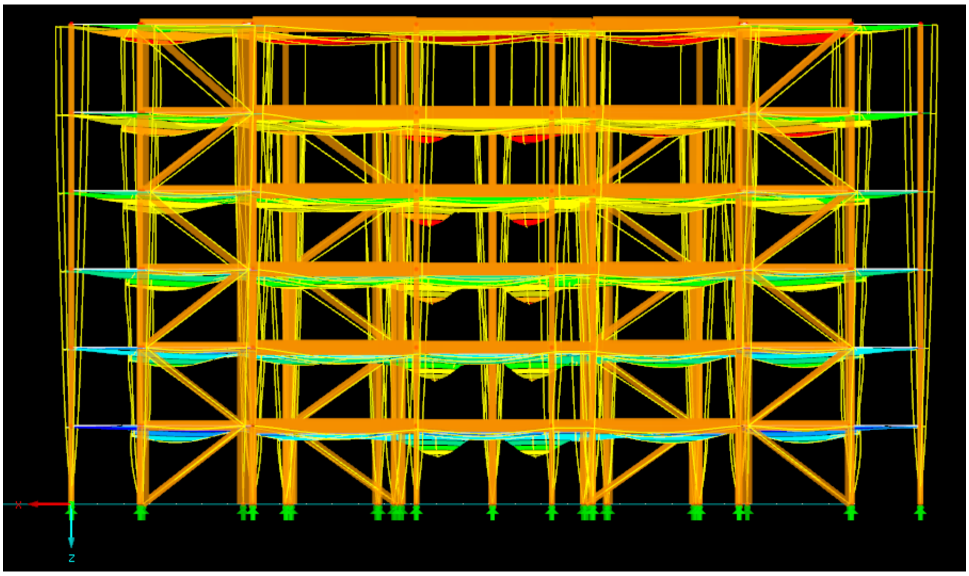
$u_{y,min}$: -10,6 mm



Displacements in x

$u_{x,max}$: 8,3 mm

$u_{x,min}$: -8,6 mm



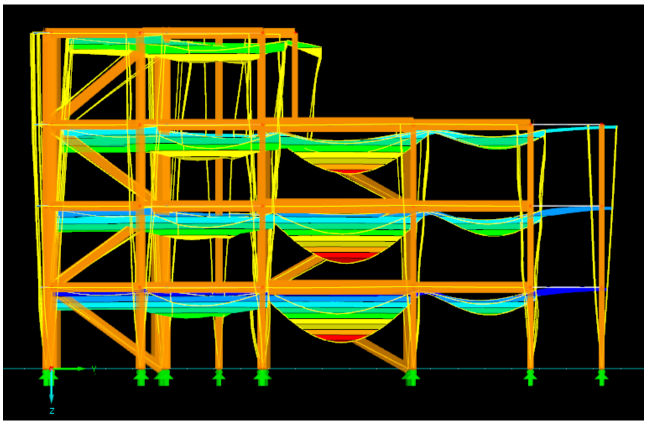
Results from four storey building

k=0,8

Displacements in y

$u_{y,max}$: 5,8 mm

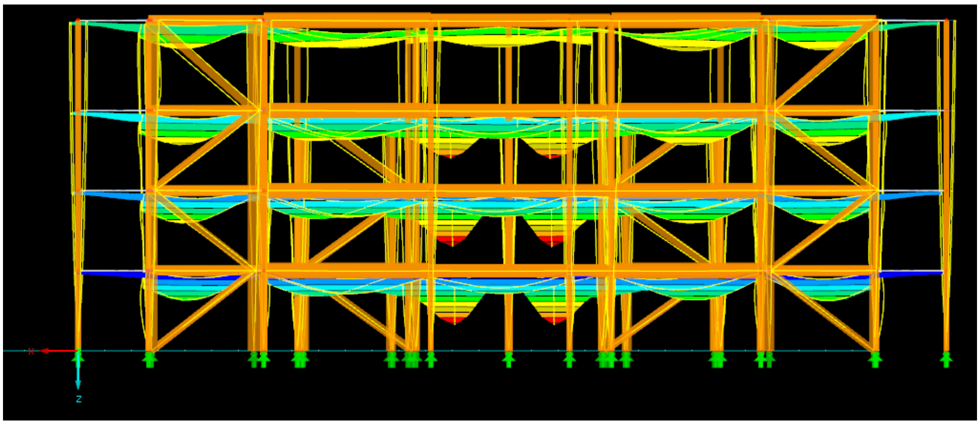
$u_{y,min}$: -3,6 mm



Displacements in x

$u_{x,max}$: 2,7 mm

$u_{x,min}$: -2,8 mm

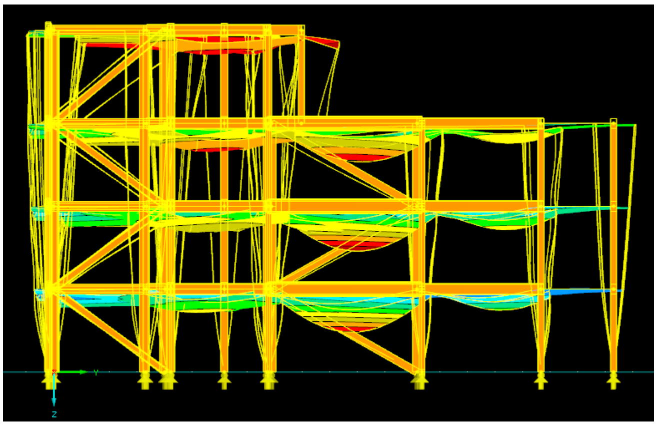


k=0,35

Displacements in y

$u_{y,max}$: 11,6 mm

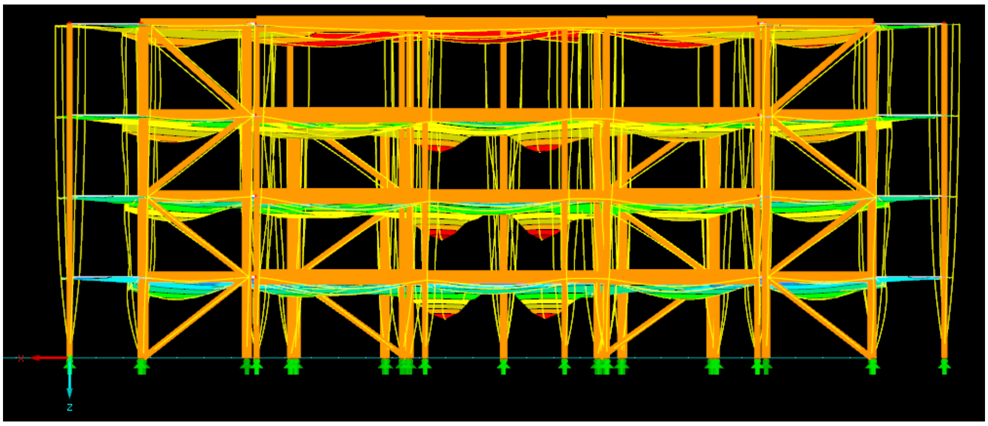
$u_{y,min}$: -6,8 mm



Displacements in x

$u_{x,max}$: 6,1 mm

$u_{x,min}$: -6,3 mm



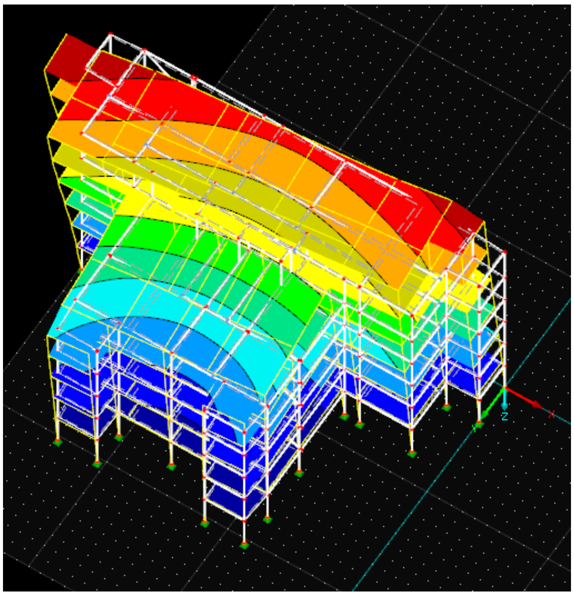
B.4 Results from dynamic analysis

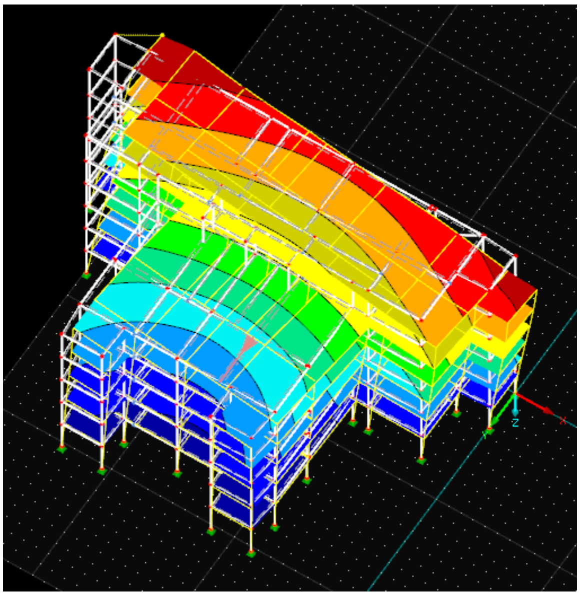
Results of six storey building

Original design

k=0,8

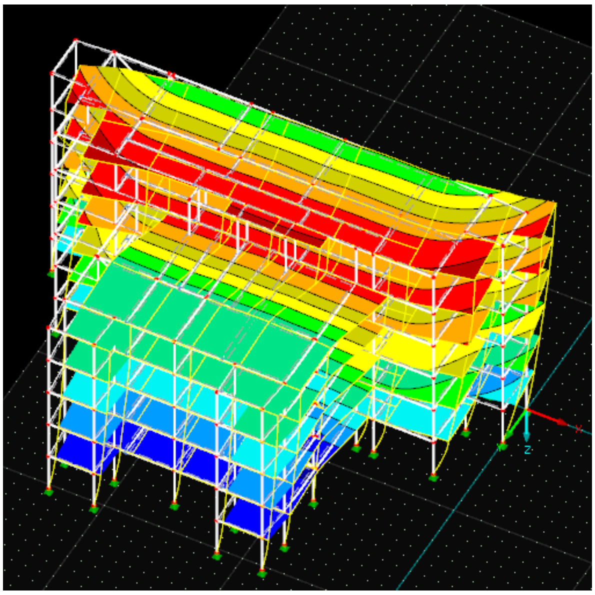
Fundamental frequency: 1,566 Hz

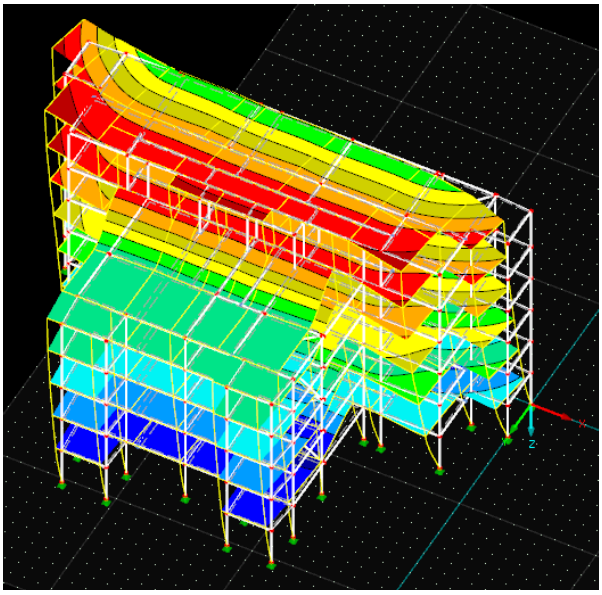




k=0,35

Fundamental frequency: 1,315 Hz

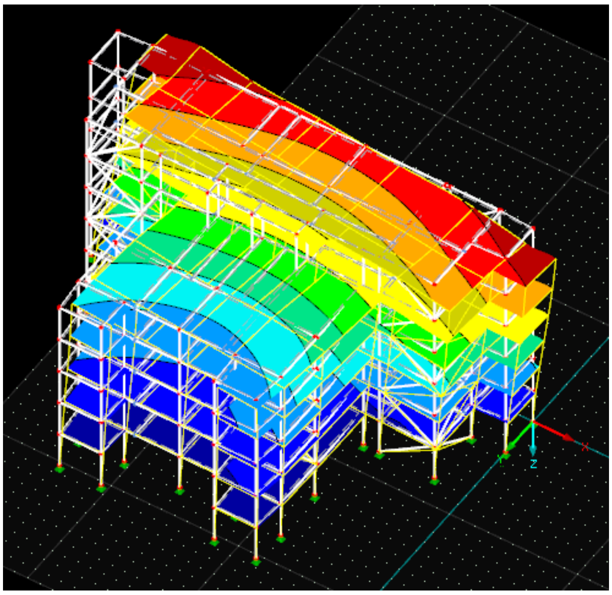


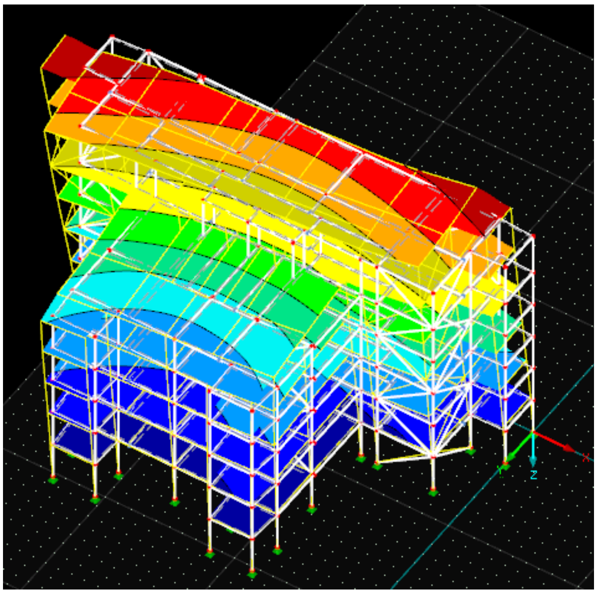


Design AB

k=0,8

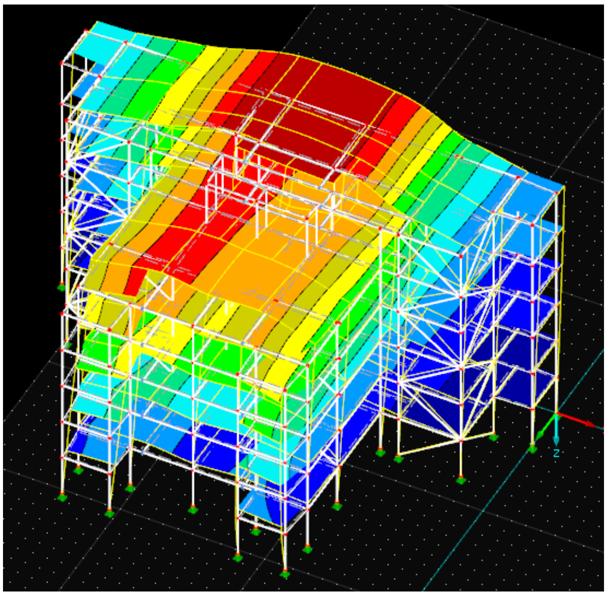
Fundamental frequency: 1,922 Hz

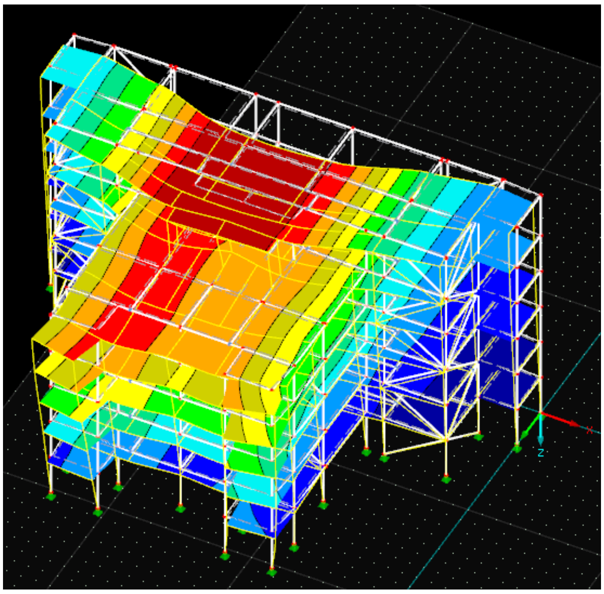




k=0,35

Fundamental frequency: 1,697 Hz

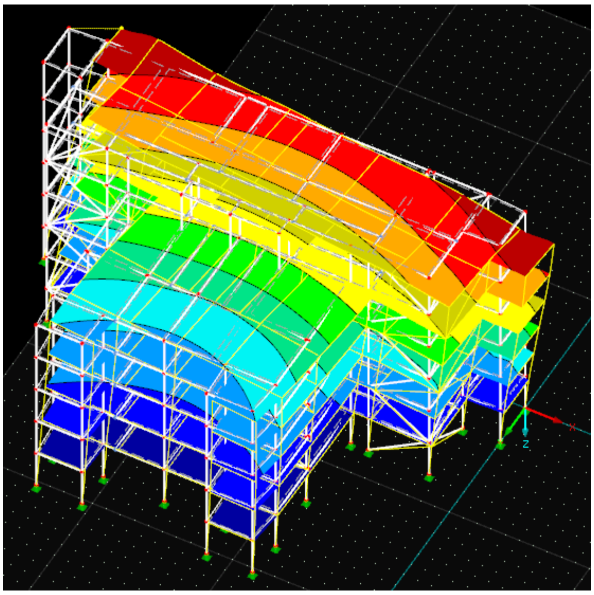
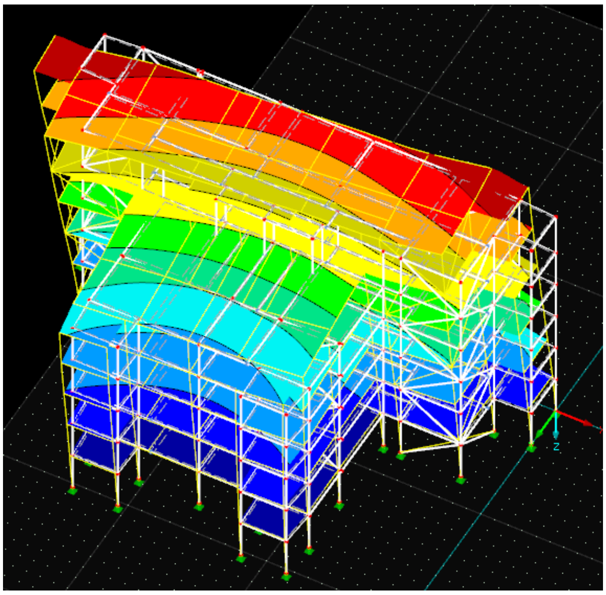




Design ABC

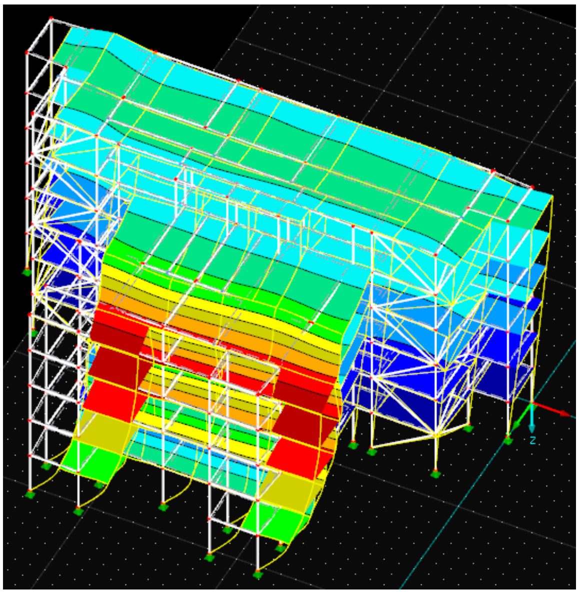
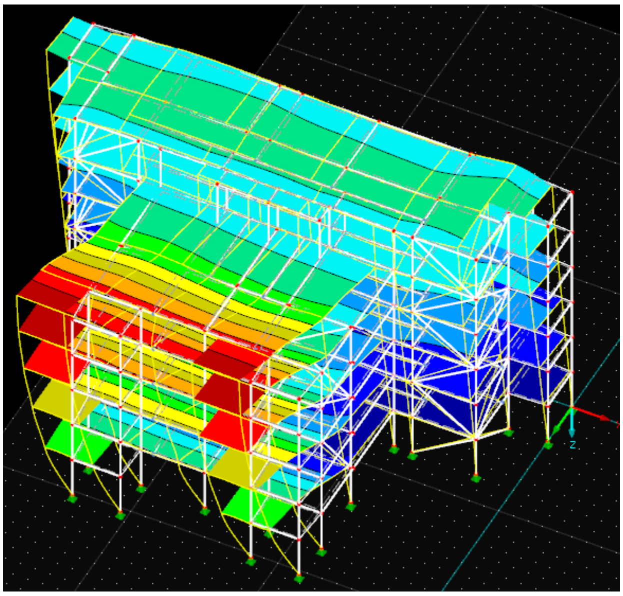
k=0,8

Fundamental frequency: 1,951 Hz

k=0,35

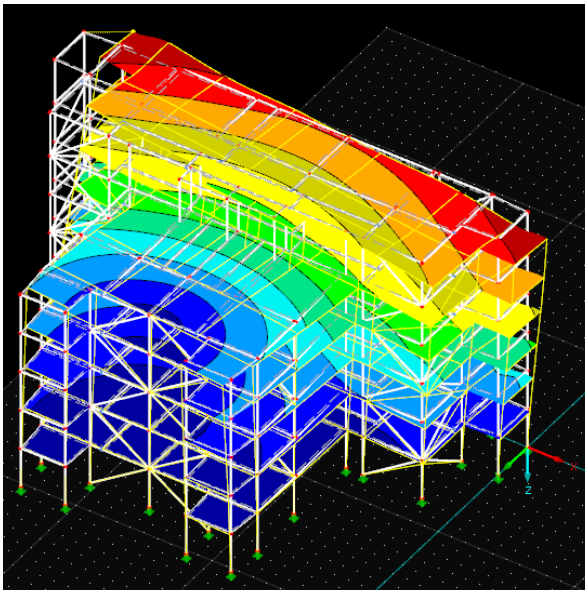
Fundamental frequency: 1,866 Hz

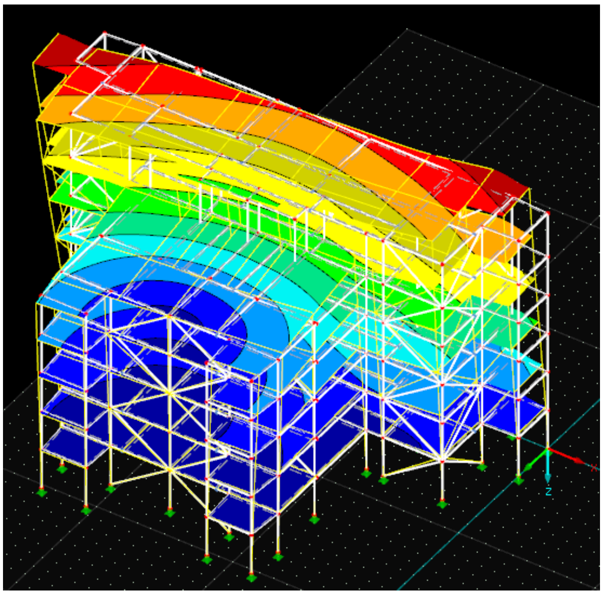



Design ABCD

k=0,8

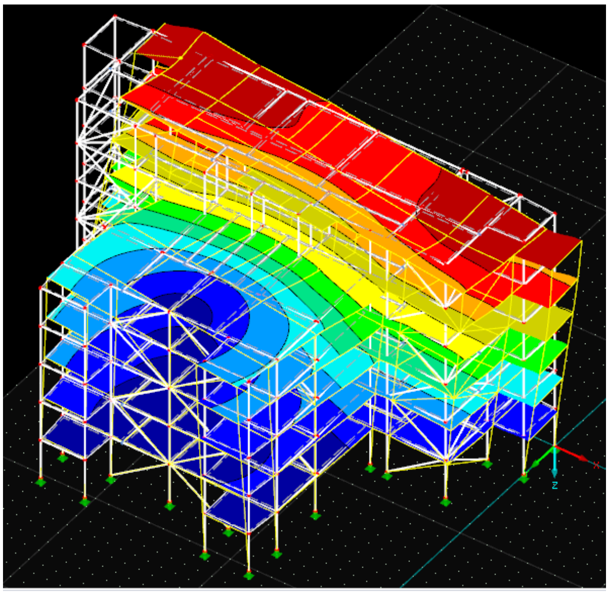
Fundamental frequency: 1,972 Hz

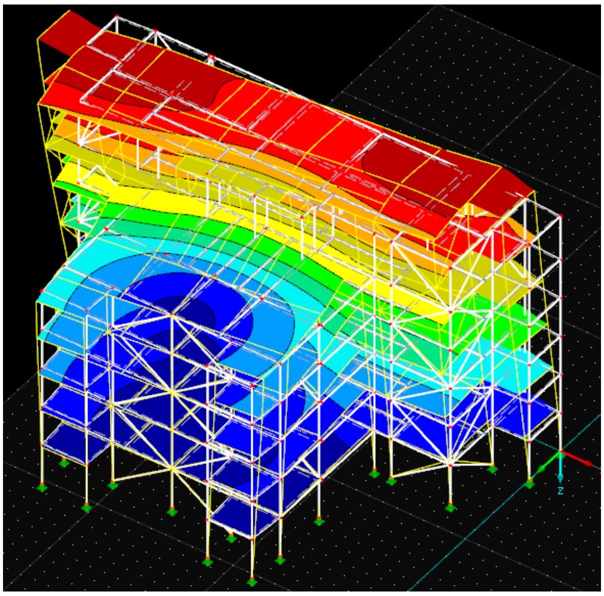




k=0,35

Fundamental frequency: 1,880 Hz



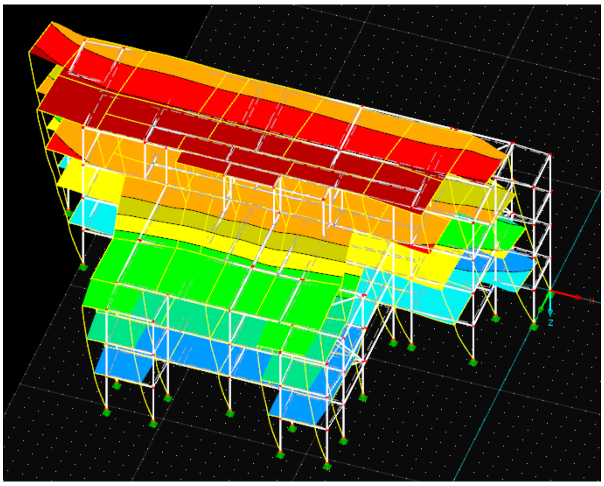
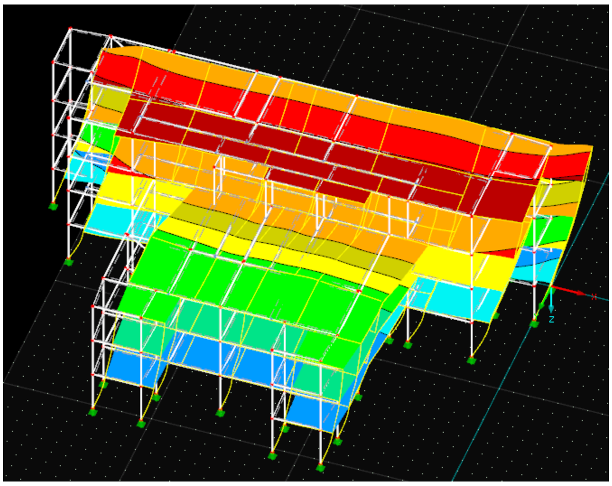


Results of four storey building

Original design

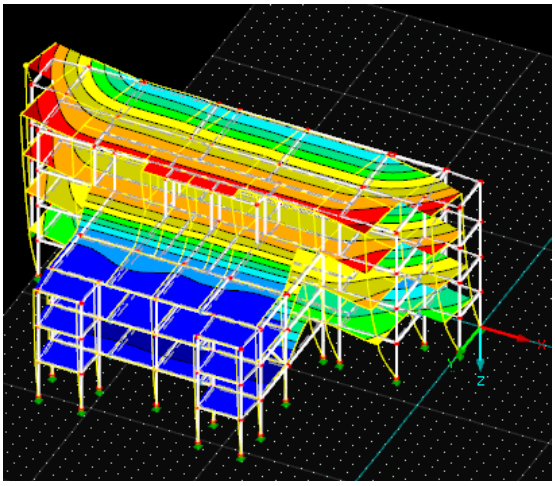
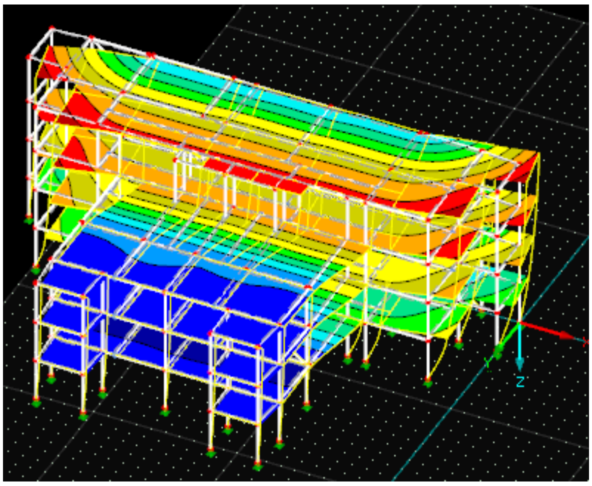
k=0,8

Fundamental frequency: 2,575 Hz

k=0,35

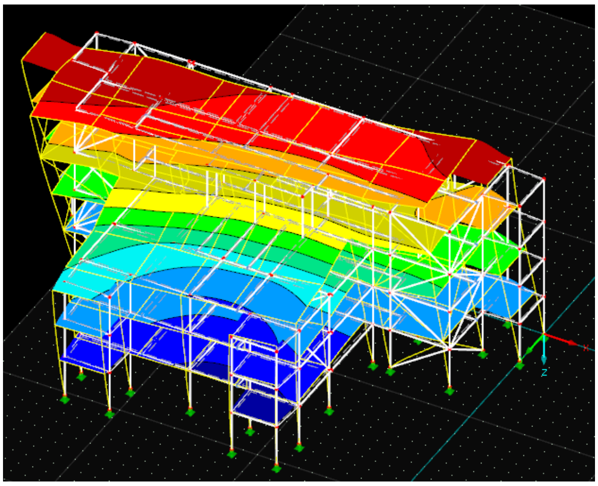
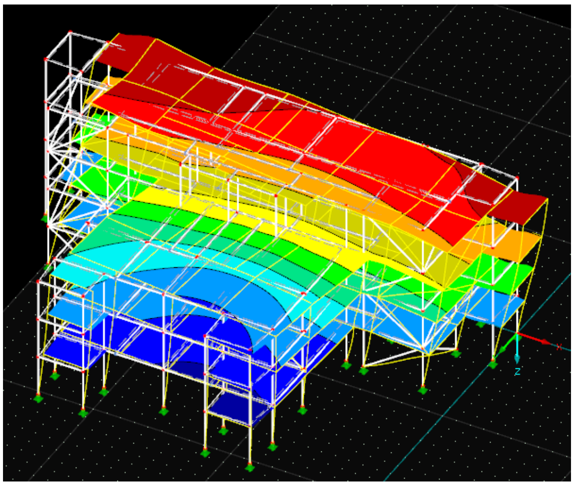
Fundamental frequency: 1,543 Hz

Design AB

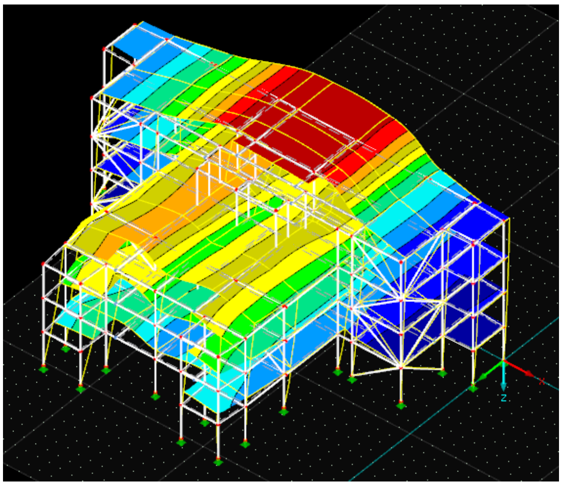
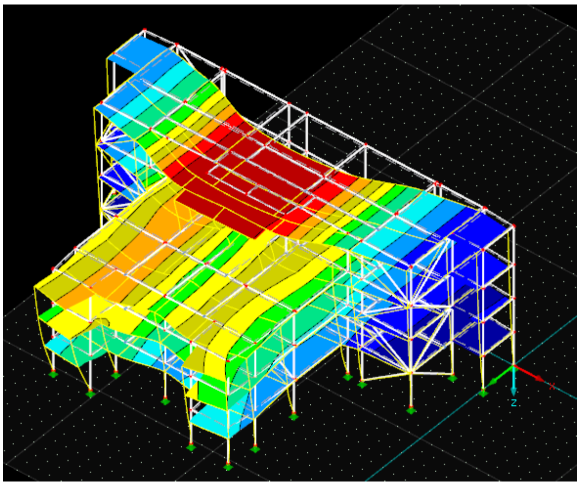
k=0,8

Fundamental frequency: 3,031 Hz

k=0,35

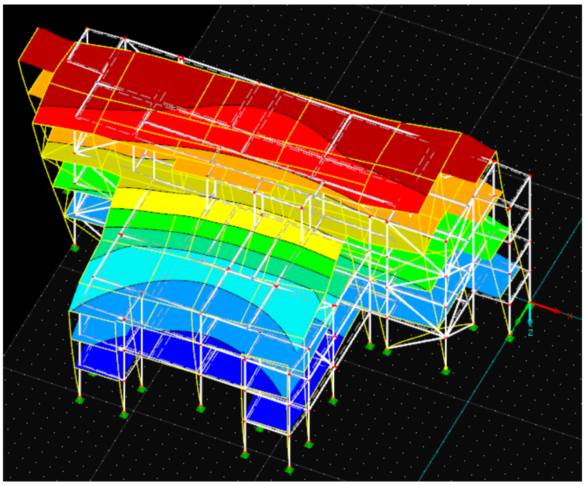
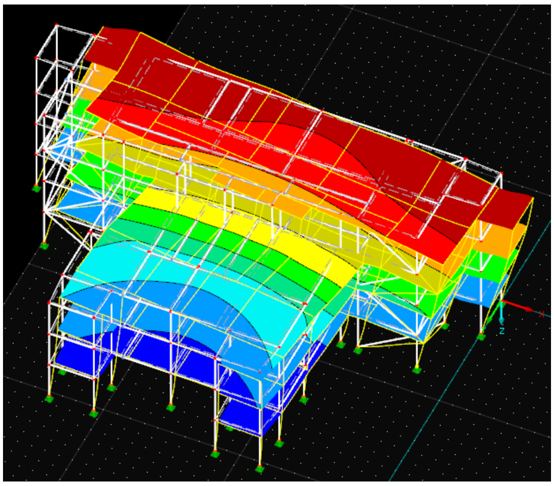
Fundamental frequency: 2,159 Hz

Design ABC

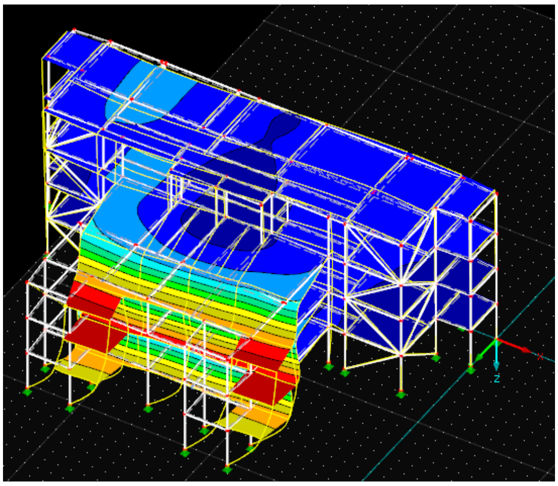
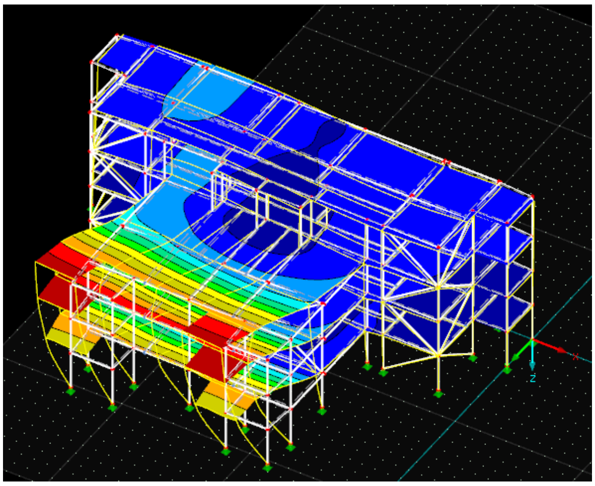
k=0,8

Fundamental frequency: 3,042 Hz

k=0,35

Fundamental frequency: 2,315 Hz

Appendix C – Hand calculations

C.1 Calculation of wind acting on the building

Six storey building

Initial imperfections

Number of columns	35
α	0,00503

Residential			Balcony			Roof		
Self weigth	2,8	kN/m^2	Self weigth	1,5	kN/m^2	Self weigth	1,4	kN/m^2
Imposed	2	kN/m^2	Imposed	3,5	kN/m^2	Snow	2	kN/m^2

	Residential		Balcony		Roof		Total		Imperfection	
Roof	0	m^2	0	m^2	286,2	m^2	973,08	kN	4,893	kN
Level 5	226,2	m^2	60	m^2	123	m^2	1803,96	kN	9,071	kN
Level 4	371,6	m^2	37,6	m^2	0	m^2	1971,68	kN	9,914	kN
Level 3	371,6	m^2	37,6	m^2	0	m^2	1971,68	kN	9,914	kN
Level 2	371,6	m^2	37,6	m^2	0	m^2	1971,68	kN	9,914	kN
Level 1	371,6	m^2	37,6	m^2	0	m^2	1971,68	kN	9,914	kN

Wind on short side

	Area		Peak velocity pressure	
Roof	12,3	m^2	0,991	kN/m^2
Level 5	38,9	m^2	0,942	kN/m^2
Level 4	53,2	m^2	0,889	kN/m^2
Level 3	53,2	m^2	0,823	kN/m^2
Level 2	53,2	m^2	0,734	kN/m^2
Level 1	52,4	m^2	0,59	kN/m^2

Shape factor

Wind facing	$c_{pe,10}$	0,758
Opposite	$c_{pe,10}$	0,416

	Wind facing		Wind opposite		Total wind	Imperfection	Total
	kN/m^2	kN	kN/m^2	kN	kN	kN	kN
Roof	0,751	9,239	0,412	5,071	14,310	4,89	19,203
Level 5	0,714	27,776	0,392	15,244	43,020	9,07	52,091
Level 4	0,674	35,849	0,370	19,675	55,524	9,91	65,438
Level 3	0,624	33,188	0,342	18,214	51,402	9,91	61,316
Level 2	0,556	29,599	0,305	16,244	45,843	9,91	55,758
Level 1	0,447	23,434	0,245	12,861	36,295	9,91	46,210

Wind on long side

	Area		Peak velocity pressure	
Roof	42,7	m^2	0,991	kN/m^2
Level 5	83,6	m^3	0,942	kN/m^3
Level 4	81,61	m^4	0,889	kN/m^4
Level 3	81,61	m^5	0,823	kN/m^5
Level 2	81,61	m^6	0,734	kN/m^6
Level 1	80,4	m^7	0,59	kN/m^7

Shape factor

Wind facing	$c_{pe,10}$	0,8
Opposite	$c_{pe,10}$	0,5

	Wind facing		Wind opposite		Total wind	Imperfection	Total
	kN/m^2	kN	kN/m^2	kN	kN	kN	kN
Roof	0,793	33,853	0,496	21,158	55,010	4,89	59,903
Level 5	0,754	63,001	0,471	39,376	102,377	9,07	111,448
Level 4	0,711	58,041	0,445	36,276	94,317	9,91	104,231
Level 3	0,658	53,732	0,412	33,583	87,315	9,91	97,229
Level 2	0,587	47,921	0,367	29,951	77,872	9,91	87,787
Level 1	0,472	37,949	0,295	23,718	61,667	9,91	71,581

Wind on roof

Shape factors

Zone F	$c_{pe,10}$	-1,8	
Zone G	$c_{pe,10}$	-1,2	
Zone H	$c_{pe,10}$	-0,7	
Zone I	$c_{pe,10}$	-0,2	0,2

Assuming $c_{pe,10}$ = -1,2 all over

Peak velocity pressure 0,991 kN/m^2

Wind -1,189 kN/m^2

Four storey building

Initial imperfections

Number of columns	35
α	0,00503

Residential			Balcony			Roof		
Self weigth	2,8	kN/m^2	Self weigth	1,5	kN/m^2	Self weigth	1,4	kN/m^2
Imposed	2	kN/m^2	Imposed	3,5	kN/m^2	Snow	2	kN/m^2

	Residential		Balcony		Roof		Total		Imperfection	
Roof	0	m^2	0	m^2	286,2	m^2	973,08	kN	4,893	kN
Level 3	226,2	m^2	60	m^2	123	m^2	1803,96	kN	9,071	kN
Level 2	371,6	m^2	37,6	m^2	0	m^2	1971,68	kN	9,914	kN
Level 1	371,6	m^2	37,6	m^2	0	m^2	1971,68	kN	9,914	kN

Wind on short side

	Area		Peak velocity pressure	
Roof	12,3	m^2	0,896	kN/m^2
Level 3	38,9	m^2	0,823	kN/m^2
Level 2	53,2	m^2	0,734	kN/m^2
Level 1	52,4	m^2	0,59	kN/m^2

Shape factor

Wind facing	$c_{pe,10}$	0,728
Opposite	$c_{pe,10}$	0,356

	Wind facing		Wind opposite		Total wind	Imperfection	Total
	kN/m^2	kN	kN/m^2	kN	kN	kN	kN
Roof	0,652	8,023	0,319	3,923	11,947	4,89	16,840
Level 3	0,599	23,307	0,293	11,397	34,704	9,07	43,775
Level 2	0,534	28,428	0,261	13,901	42,329	9,91	52,243
Level 1	0,430	22,507	0,210	11,006	33,513	9,91	43,427

Wind on long side

	Area		Peak velocity pressure	
Roof	12,3	m^2	0,896	kN/m^2
Level 3	38,9	m^2	0,823	kN/m^2
Level 2	53,2	m^2	0,734	kN/m^2
Level 1	52,4	m^2	0,59	kN/m^2

Shape factor		
Wind facing	$c_{pe,10}$	0,732
Opposite	$c_{pe,10}$	0,415

	Wind facing		Wind opposite		Total wind	Imperfection	Total
	kN/m^2	kN	kN/m^2	kN	kN	kN	kN
Roof	0,656	8,067	0,372	4,574	12,641	4,89	17,534
Level 3	0,602	23,435	0,342	13,286	36,721	9,07	45,792
Level 2	0,537	28,584	0,305	16,205	44,789	9,91	54,703
Level 1	0,432	22,631	0,245	12,830	35,461	9,91	45,375

Wind on roof

Shape factors

Zone F	$c_{pe,10}$	-1,8	
Zone G	$c_{pe,10}$	-1,2	
Zone H	$c_{pe,10}$	-0,7	
Zone I	$c_{pe,10}$	-0,2	0,2

Assuming $c_{pe,10}$ = -1,2 all over

Peak velocity pressure 0,896 kN/m^2

Wind -1,075 kN/m^2

C.2 Calculation of load dividing between trusses

Wind on short side

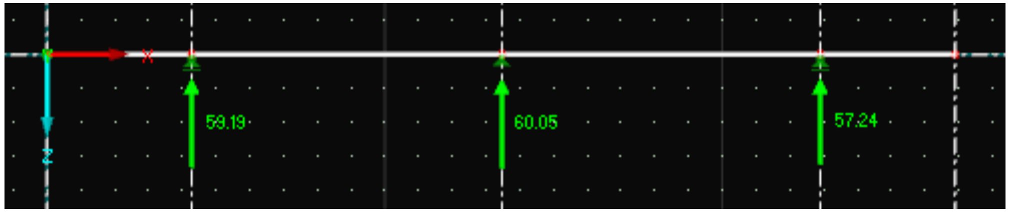
Calculation of factors:

$$36{,}87 + 72{,}16 = 109{,}03$$

First support = Truss 5 + Truss 6 $\quad \dfrac{36{,}87}{109{,}03} = 0{,}3382$

Second support = Truss 2 + Truss 3 $\quad \dfrac{72{,}16}{109{,}03} = 0{,}6618$

Wind on long side



Calculation of factors:

$$59{,}19 + 60{,}05 + 57{,}24 = 176{,}48$$

First support = Truss 1 $\frac{59{,}19}{176{,}48} = 0{,}3354$

Second support = Truss 7 $\frac{60{,}05}{176{,}48} = 0{,}3403$

Third support = Truss 4 $\frac{57{,}24}{176{,}48} = 0{,}3243$

C.3 Calculation of load acting on the trusses

Six storey building

Wind on short side

		Factors
Level 1-5	Truss 2	0,3309
	Truss 3	0,3309
	Truss 5	0,1691
	Truss 6	0,1691
Level 6	Truss 2	0
	Truss 3	0
	Truss 5	0,5000
	Truss 6	0,5000

	Level 1	**Level 2**	**Level 3**	**Level 4**	**Level 5**	**Level 6**
Total load	46,21 kN	55,76 kN	61,32 kN	65,44 kN	52,09 kN	19,2 kN
Truss 2	15,291 kN	18,451 kN	20,291 kN	21,654 kN	17,237 kN	0 kN
Truss 3	15,291 kN	18,451 kN	20,291 kN	21,654 kN	17,237 kN	0 kN
Truss 5	7,814 kN	9,429 kN	10,369 kN	11,066 kN	8,808 kN	9,600 kN
Truss 6	7,814 kN	9,429 kN	10,369 kN	11,066 kN	8,808 kN	9,600 kN

Wind on long side

		Factors
Level 1-5	Truss 1	0,3354
	Truss 7	0,3403
	Truss 4	0,3243
Level 6	Truss 1	0,5000
	Truss 7	0
	Truss 4	0,5000

	Level 1	**Level 2**	**Level 3**	**Level 4**	**Level 5**	**Level 6**
Total load	71,58 kN	87,79 kN	97,23 kN	104,23 kN	111,45 kN	59,9 kN
Truss 1	24,008 kN	29,445 kN	32,611 kN	34,959 kN	37,380 kN	29,950 kN
Truss 7	24,359 kN	29,875 kN	33,087 kN	35,469 kN	37,926 kN	0 kN
Truss 4	23,213 kN	28,470 kN	31,532 kN	33,802 kN	36,143 kN	29,950 kN

Vertical loads: snow, self-weight, imposed

Loads					
Self weigth			Imposed		
Roof	1,4	kN/m^2	Snow load	2	kN/m^2
Floor	2	kN/m^2	Residental	2	kN/m^2
Interior walls	0,5	kN/m^2	Balcony	3,5	kN/m^2
Exterior walls	1,8	kN/m^2			
Balcony	1,5	kN/m^2			

Snow load	Point load, Left			Point load, Right			Distributed		
	Area	Snow	Load	Area	Snow	Load	Width	Snow	Load
	m^2	kN/m^2	kN	m^2	kN/m^2	kN	m	kN/m^2	kN/m
Truss 1	12,3	2	24,564	21,5	2	43,100			
Truss 2	11,6	2	23,102	19,4	2	38,888			
Truss 3	19,4	2	38,888	11,6	2	23,102			
Truss 4	11,6	2	23,102	19,7	2	39,401			
Truss 5	2,4	2	4,900	7,3	2	14,592	2,3	2	4,686
Truss 6	6,5	2	13,046	2,4	2	4,900	2,3	2	4,686
Truss 7	16,0	2	31,989	17,6	2	35,236	1,868	2	3,736

Appendix C

Self-weigth Level 1-4	Point load, Left								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2,5	15,7	1,8					59,031
Truss 2	11,6	2,5	14,8	1,8					55,517
Truss 3	19,4	2,5							48,610
Truss 4	11,6	2,5	14,8	1,8					55,517
Truss 5			5,3	1,8			2,4	1,5	13,207
Truss 6	6,5	2,5	8,4	1,8					31,449
Truss 7	16,0	2,5							39,986

Self-weigth Level 1-4	Point load, Right								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2,5							53,875
Truss 2	19,4	2,5							48,610
Truss 3	11,6	2,5	14,8	1,8					55,517
Truss 4	19,7	2,5							49,251
Truss 5	7,3	2,5	9,3	1,8					35,067
Truss 6			5,3	1,8			2,4	1,5	13,207
Truss 7	17,6	2,5							44,045

Self-weigth Level 1-4	Distributed								
	Width floor	Load on floor	Height exterior wall	Load from ex. wall	Width roof	Load roof	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	m	kN/m^2	m	kN/m^2	kN/m
Truss 1									
Truss 2									
Truss 3									
Truss 4									
Truss 5	2,3	2,5	3,0	1,8					11,261
Truss 6	2,3	2,5	3,0	1,8					11,261
Truss 7	1,868	2,5							4,670

Self-weigth Level 5	Point load, Left								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2,5	15,7	1,8					59,031
Truss 2					11,6	1,4			16,171
Truss 3					19,4	1,4			27,221
Truss 4	11,6	2,5	14,8	1,8					55,517
Truss 5			5,3	1,8			2,4	1,5	13,207
Truss 6	6,5	2,5	8,4	1,8					31,449
Truss 7	6,0	2,5	10,507	1,8	6,0	1,4	10,0	1,5	57,199

Self-weigth Level 5	Point load, Right								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2,5							53,875
Truss 2					19,4	1,4			27,221
Truss 3					11,6	1,4			16,171
Truss 4	1,8	2,5							4,500
Truss 5	7,3	2,5	9,3	1,8					35,067
Truss 6			5,3	1,8			2,4	1,5	13,207
Truss 7					7,6	1,4	10,0	1,5	25,669

Self-weigth Level 5	Distributed								
	Width floor	Load on floor	Height exterior wall	Load from ex. wall	Width roof	Load roof	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	m	kN/m^2	m	kN/m^2	kN/m
Truss 1									
Truss 2									
Truss 3									
Truss 4									
Truss 5	2,3	2,5	3,0	1,8					11,261
Truss 6	2,3	2,5	3,0	1,8					11,261
Truss 7							1,868	1,5	2,802

Self-weigth Level 6	Point load, Left								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1					12,3	1,4			17,195
Truss 2									
Truss 3									
Truss 4					11,6	1,4			16,171
Truss 5					2,4	1,4			3,430
Truss 6					6,5	1,4			9,132
Truss 7									

Self-weigth Level 6	Point load, Right								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1					21,5	1,4			30,170
Truss 2									
Truss 3									
Truss 4					1,8	1,4			2,520
Truss 5					7,3	1,4			10,215
Truss 6					2,4	1,4			3,430
Truss 7									

Self-weigth Level 6	Distributed								
	Width floor	Load on floor	Height exterior wall	Load from ex. wall	Width roof	Load roof	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	m	kN/m^2	m	kN/m^2	kN/m
Truss 1									
Truss 2									
Truss 3									
Truss 4									
Truss 5					2,3	1,4			3,280
Truss 6					2,3	1,4			3,280
Truss 7									

Imposed loads Level 1-4	Point load, Left				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2			24,564
Truss 2	11,6	2			23,102
Truss 3	19,4	2			38,888
Truss 4	11,6	2			23,102
Truss 5			2,4	3,5	8,574
Truss 6	6,5	2			13,046
Truss 7	16,0	2			31,989

Imposed loads Level 1-4	Point load, Right				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2			43,100
Truss 2	19,4	2			38,888
Truss 3	11,6	2			23,102
Truss 4	19,7	2			39,401
Truss 5	7,3	2			14,592
Truss 6			2,4	3,5	8,574
Truss 7	17,6	2			35,236

Imposed loads Level 1-4	Distributed				
	Width floor	Load on floor	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	kN/m
Truss 1					
Truss 2					
Truss 3					
Truss 4					
Truss 5	2,3	2			4,686
Truss 6	2,3	2			4,686
Truss 7	1,9	2			3,736

Appendix C

Imposed loads Level 5	Point load, Left				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2			24,564
Truss 2					
Truss 3					
Truss 4	11,6	2			23,102
Truss 5			2,4	3,5	8,574
Truss 6	6,5	2			13,046
Truss 7	6,0	2	10,0	3,5	47,046

Imposed loads Level 5	Point load, Right				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2			43,100
Truss 2					
Truss 3					
Truss 4	1,8	2			3,600
Truss 5	7,3	2			14,592
Truss 6			2,4	3,5	8,574
Truss 7			10,0	3,5	35,134

Imposed loads Level 5	Distributed				
	Width floor	Load on floor	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	kN/m
Truss 1					
Truss 2					
Truss 3					
Truss 4					
Truss 5	2,3	2			4,686
Truss 6	2,3	2			4,686
Truss 7			1,9	3,5	6,538

Four storey building

Wind on short side

		Factors
Level 1-3	Truss 2	0,3309
	Truss 3	0,3309
	Truss 5	0,1691
	Truss 6	0,1691
Level 4	Truss 2	0
	Truss 3	0
	Truss 5	0,5000
	Truss 6	0,5000

	Level 1	Level 2	Level 3	Level 4
Total load	43,427 kN	52,243 kN	43,775 kN	16,84 kN
Truss 2	14,370 kN	17,287 kN	14,485 kN	0,000 kN
Truss 3	14,370 kN	17,287 kN	14,485 kN	0,000 kN
Truss 5	7,344 kN	8,834 kN	7,402 kN	8,420 kN
Truss 6	7,344 kN	8,834 kN	7,402 kN	8,420 kN

Wind on long side

		Factors
Level 1-3	Truss 1	0,3354
	Truss 7	0,3403
	Truss 4	0,3243
Level 4	Truss 1	0,5000
	Truss 7	0
	Truss 4	0,5000

	Level 1	**Level 2**	**Level 3**	**Level 4**
Total load	45,375 kN	54,703 kN	45,792 kN	17,534 kN
Truss 1	15,219 kN	18,347 kN	15,359 kN	8,767 kN
Truss 7	15,441 kN	18,615 kN	15,583 kN	0,000 kN
Truss 4	14,715 kN	17,740 kN	14,850 kN	8,767 kN

Vertical loads: snow, self-weight, imposed

Loads					
Self weigth			Imposed		
Roof	1,4	kN/m^2	Snow load	2	kN/m^2
Floor	2	kN/m^2	Residental	2	kN/m^2
Interior walls	0,5	kN/m^2	Balcony	3,5	kN/m^2
Exterior walls	1,8	kN/m^2			
Balcony	1,5	kN/m^2			

Snow load	Point load, Left			Point load, Right			Distributed		
	Area	Snow	Load	Area	Snow	Load	Width	Snow	Load
	m^2	kN/m^2	kN	m^2	kN/m^2	kN	m	kN/m^2	kN/m
Truss 1	12,3	2	24,564	21,5	2	43,100			
Truss 2	11,6	2	23,102	19,4	2	38,888			
Truss 3	19,4	2	38,888	11,6	2	23,102			
Truss 4	11,6	2	23,102	19,7	2	39,401			
Truss 5	2,4	2	4,900	7,3	2	14,592	2,3	2	4,686
Truss 6	6,5	2	13,046	2,4	2	4,900	2,3	2	4,686
Truss 7	16,0	2	31,989	17,6	2	35,236	1,868	2	3,736

Self-weigth Level 1-2	Point load, Left								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2,5	15,7	1,8					59,031
Truss 2	11,6	2,5	14,8	1,8					55,517
Truss 3	19,4	2,5							48,610
Truss 4	11,6	2,5	14,8	1,8					55,517
Truss 5			5,3	1,8			2,4	1,5	13,207
Truss 6	6,5	2,5	8,4	1,8					31,449
Truss 7	16,0	2,5							39,986

Self-weigth Level 1-2	Point load, Right								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2,5							53,875
Truss 2	19,4	2,5							48,610
Truss 3	11,6	2,5	14,8	1,8					55,517
Truss 4	19,7	2,5							49,251
Truss 5	7,3	2,5	9,3	1,8					35,067
Truss 6			5,3	1,8			2,4	1,5	13,207
Truss 7	17,6	2,5							44,045

Self-weigth Level 1-2	Distributed								
	Width floor	Load on floor	Height exterior wall	Load from ex. wall	Width roof	Load roof	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	m	kN/m^2	m	kN/m^2	kN/m
Truss 1									
Truss 2									
Truss 3									
Truss 4									
Truss 5	2,3	2,5	3,0	1,8					11,261
Truss 6	2,3	2,5	3,0	1,8					11,261
Truss 7	1,868	2,5							4,670

Self-weigth Level 3	Point load, Left								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2,5	15,7	1,8					59,031
Truss 2					11,6	1,4			16,171
Truss 3					19,4	1,4			27,221
Truss 4	11,6	2,5	14,8	1,8					55,517
Truss 5			5,3	1,8			2,4	1,5	13,207
Truss 6	6,5	2,5	8,4	1,8					31,449
Truss 7	6,0	2,5	10,507	1,8	6,0	1,4	10,0	1,5	57,199

Self-weigth Level 3	Point load, Right								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2,5							53,875
Truss 2					19,4	1,4			27,221
Truss 3					11,6	1,4			16,171
Truss 4	1,8	2,5							4,500
Truss 5	7,3	2,5	9,3	1,8					35,067
Truss 6			5,3	1,8			2,4	1,5	13,207
Truss 7					7,6	1,4	10,0	1,5	25,669

Self-weigth Level 3	Distributed								
	Width floor	Load on floor	Height exterior wall	Load from ex. wall	Width roof	Load roof	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	m	kN/m^2	m	kN/m^2	kN/m
Truss 1									
Truss 2									
Truss 3									
Truss 4									
Truss 5	2,3	2,5	3,0	1,8					11,261
Truss 6	2,3	2,5	3,0	1,8					11,261
Truss 7							1,868	1,5	2,802

Self-weigth Level 4	Point load, Left								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1					12,3	1,4			17,195
Truss 2									
Truss 3									
Truss 4					11,6	1,4			16,171
Truss 5					2,4	1,4			3,430
Truss 6					6,5	1,4			9,132
Truss 7									

Self-weigth Level 4	Point load, Right								
	Area floor	Load floor	Area ext. wall	Load ext. wall	Area roof	Load roof	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1					21,5	1,4			30,170
Truss 2									
Truss 3									
Truss 4					1,8	1,4			2,520
Truss 5					7,3	1,4			10,215
Truss 6					2,4	1,4			3,430
Truss 7									

Self-weigth Level 4	Distributed								
	Width floor	Load on floor	Height exterior wall	Load from ex. wall	Width roof	Load roof	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	m	kN/m^2	m	kN/m^2	kN/m
Truss 1									
Truss 2									
Truss 3									
Truss 4									
Truss 5					2,3	1,4			3,280
Truss 6					2,3	1,4			3,280
Truss 7									

Imposed loads Level 1-2	Point load, Left				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2			24,564
Truss 2	11,6	2			23,102
Truss 3	19,4	2			38,888
Truss 4	11,6	2			23,102
Truss 5			2,4	3,5	8,574
Truss 6	6,5	2			13,046
Truss 7	16,0	2			31,989

Appendix C

Imposed loads Level 1-2	Point load, Right				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2			43,100
Truss 2	19,4	2			38,888
Truss 3	11,6	2			23,102
Truss 4	19,7	2			39,401
Truss 5	7,3	2			14,592
Truss 6			2,4	3,5	8,574
Truss 7	17,6	2			35,236

Imposed loads Level 1-2	Distributed				
	Width floor	Load on floor	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	kN/m
Truss 1					
Truss 2					
Truss 3					
Truss 4					
Truss 5	2,3	2			4,686
Truss 6	2,3	2			4,686
Truss 7	1,9	2			3,736

Imposed loads Level 3	Point load, Left				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	12,3	2			24,564
Truss 2					
Truss 3					
Truss 4	11,6	2			23,102
Truss 5			2,4	3,5	8,574
Truss 6	6,5	2			13,046
Truss 7	6,0	2	10,0	3,5	47,046

Imposed loads Level 3	Point load, Right				
	Area floor	Load on floor	Area balcony	Load balcony	Sum
	m^2	kN/m^2	m^2	kN/m^2	kN
Truss 1	21,5	2			43,100
Truss 2					
Truss 3					
Truss 4	1,8	2			3,600
Truss 5	7,3	2			14,592
Truss 6			2,4	3,5	8,574
Truss 7			10,0	3,5	35,134

Imposed loads Level 3	Distributed				
	Width floor	Load on floor	Width balcony	Load balcony	Sum
	m	kN/m^2	m	kN/m^2	kN/m
Truss 1					
Truss 2					
Truss 3					
Truss 4					
Truss 5	2,3	2			4,686
Truss 6	2,3	2			4,686
Truss 7			1,9	3,5	6,538

C.4 Calculation of loads acting on the FE structure

Six storey building

Horisontal loads								
Wind	Short side (x), facing		Short side (x), opposite		Long side (y), facing		Long side (y), opposite	
Roof	0,751	kN/m^2	0,412	kN/m^2	0,793	kN/m^2	0,496	kN/m^2
	1,262	**kN/m**	**0,693**	**kN/m**	**1,332**	**kN/m**	**0,833**	**kN/m**
Level 5	0,714	kN/m^2	0,392	kN/m^2	0,754	kN/m^2	0,471	kN/m^2
	2,279	**kN/m**	**1,251**	**kN/m**	**2,405**	**kN/m**	**1,503**	**kN/m**
Level 4	0,674	kN/m^2	0,370	kN/m^2	0,711	kN/m^2	0,445	kN/m^2
	2,023	**kN/m**	**1,110**	**kN/m**	**2,135**	**kN/m**	**1,334**	**kN/m**
Level 3	0,624	kN/m^2	0,342	kN/m^2	0,658	kN/m^2	0,412	kN/m^2
	1,873	**kN/m**	**1,028**	**kN/m**	**1,977**	**kN/m**	**1,235**	**kN/m**
Level 2	0,556	kN/m^2	0,305	kN/m^2	0,587	kN/m^2	0,367	kN/m^2
	1,670	**kN/m**	**0,917**	**kN/m**	**1,763**	**kN/m**	**1,102**	**kN/m**
Level 1	0,447	kN/m^2	0,245	kN/m^2	0,472	kN/m^2	0,295	kN/m^2
	1,343	**kN/m**	**0,737**	**kN/m**	**1,417**	**kN/m**	**0,886**	**kN/m**

Vertical loads					
Self weigth			Imposed		
Roof	1,4	kN/m^2	Snow load	2	kN/m^2
Floor	2	kN/m^2	Residental	2	kN/m^2
Interior walls	0,5	kN/m^2	Balcony	3,5	kN/m^2
Exterior walls	1,8	kN/m^2			
Balcony	1,5	kN/m^2			

Four storey building

Horisontal loads				
Wind	Short side (x), facing	Short side (x), opposite	Long side (y), facing	Long side (y), opposite
Roof	0,652 kN/m^2 **1,096 kN/m**	0,319 kN/m^2 **0,536 kN/m**	0,656 kN/m^2 **1,102 kN/m**	0,372 kN/m^2 **0,625 kN/m**
Level 3	0,599 kN/m^2 **1,912 kN/m**	0,293 kN/m^2 **0,935 kN/m**	0,602 kN/m^2 **1,923 kN/m**	0,342 kN/m^2 **1,090 kN/m**
Level 2	0,534 kN/m^2 **1,604 kN/m**	0,261 kN/m^2 **0,784 kN/m**	0,537 kN/m^2 **1,613 kN/m**	0,305 kN/m^2 **0,914 kN/m**
Level 1	0,430 kN/m^2 **1,289 kN/m**	0,210 kN/m^2 **0,631 kN/m**	0,432 kN/m^2 **1,297 kN/m**	0,245 kN/m^2 **0,735 kN/m**

Vertical loads					
Self weigth			Imposed		
Roof	1,4	kN/m^2	Snow load	2	kN/m^2
Floor	2	kN/m^2	Residental	2	kN/m^2
Interior walls	0,5	kN/m^2	Balcony	3,5	kN/m^2
Exterior walls	1,8	kN/m^2			
Balcony	1,5	kN/m^2			

C.5 Calculation of peak wind acceleration

Six storey building

Building's properties

Terrain category	II		
Height of building	h	18,391	m
Length of building	l	27,5	m
Width of building	w	18,1	m
Height of top floor	z	15,1	m

Results from RFEM simulation		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
First natural frequency	n_1	1,566 Hz	1,315 Hz	1,566 Hz	1,315 Hz
Total self-weight	m_{total}	820857 kg	820857 kg	820857 kg	820857 kg
Approximate equivalent mass	$m_e = m_{total}/h$	44634 kg/m	44634 kg/m	44634 kg/m	44634 kg/m

Calculations

Calculation of reference wind speed, v_T, done by using following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1)

$$v_T = 0{,}75 \cdot v_{50} \sqrt{1 - 0{,}2 \cdot \ln\left(-ln\left(1 - \frac{1}{T}\right)\right)}$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Reference wind speed (50y return period)	$v_{b,50y}$	24 m/s	24 m/s	24 m/s	24 m/s
Desired return period	T	2 years	2 years	2 years	2 years
Reference wind speed, 2y return period	$v_{b,2y}$	18,648 m/s	18,648 m/s	18,648 m/s	18,648 m/s

Calculation of mean wind speed, v_m, for a 2 year return period is done by using formula (4.3) in SS-EN 1991-1-4

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b$$

Where the roughness factor, c_r, is calculated by formula (4.4) in SS-EN 1991-1-4

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

The terrain factor, k_r, is calculated by formula (4.5) in SS-EN 1991-1-4

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

The orography factor, $c_0(z)$, is set to 1,0 according to SS-EN 1991-1-4

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Roughness length	z_0	0,05 m	0,05 m	0,05 m	0,05 m
Reference roughness length	$z_{0,II}$	0,05 m	0,05 m	0,05 m	0,05 m
Terrain factor	k_r	0,19	0,19	0,19	0,19
Roughness factor	$c_r(z)$	1,085	1,085	1,085	1,085
Orography factor	$c_0(z)$	1	1	1	1
Mean wind speed (2y return period)	$v_m(z)$	20,233 m/s	20,233 m/s	20,233 m/s	20,233 m/s

The turbulence intensity, $I_v(z)$, can be determined by using formula (4.7) in SS-EN 1991-1-4 with k_l set to 1,0

$$I_v(z) = \frac{k_l}{c_0(z) \cdot ln\left(\frac{z}{z_0}\right)}$$

Turbulence intensity	$I_v(z)$	0,175	0,175	0,175	0,175

The background response, B^2, is calculated using following formula from EKS 10 Chapter 1.1.4 Section 6.3.1(1) with reference height, h_{ref}, set to 10 m according to BSV 97 Section 3.22

$$B^2 = exp\left(-0{,}05\left(\frac{h}{h_{ref}}\right) + \left(1 - \frac{b}{h}\right)\left(0{,}04 + 0{,}01\left(\frac{h}{h_{ref}}\right)\right)\right)$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Reference height	h_{ref}	10 m	10 m	10 m	10 m
Background response	B^2	0,913	0,913	0,886	0,886

The resonance response, R^2, is calculated using following formula from EKS 10 Section 6.3.1(1).

$$R^2 = \frac{2\pi \cdot F \cdot \phi_b \cdot \phi_h}{\delta_s + \delta_a}$$

The factor, F, which refers to Kármán's wind energy spectrum is calculated by following formula from EKS 10 Chapter 1.1.4 Section 6.3.1(1).

$$F = \frac{4 \cdot y_C}{(1 + 70{,}8 \cdot {y_C}^2)^{5/6}}$$

$$y_C = \frac{150 \cdot n_1}{v_m(z)}$$

The factors ϕ_b and ϕ_h depends on the size with regard to width and height respectively and are calculated by following formulas from EKS 10 Chapter 1.1.4 Section 6.3.1(1).

$$\phi_h = \frac{1}{1 + \frac{2 \cdot n_1 \cdot h}{v_m(z)}}$$

$$\phi_b = \frac{1}{1 + \frac{3{,}2 \cdot n_1 \cdot b}{v_m(z)}}$$

The δ factors refers to damping. The structural damping, δ_s, is set to 0,09 according to BSV 97 Table 3.22a. The aerodynamic damping, δ_a, is calculated using formula (F.18) SS-EN 1991-1-4.

$$\delta_a = \frac{c_f \cdot \rho_{air} \cdot b \cdot v_m(z)}{2 \cdot n_1 \cdot m_e}$$

The shape factor, c_f, is calculated using formula (7.9) in SS-EN 1991-1-4.

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda$$

The shape factor, $c_{f,0}$, is calculated using following formulas from SS-EN 1991-1-4 in Figure 7.23

$$For\ 0{,}2 < {}^{d}/_{b} < 0{,}7 \qquad c_{f,0} = 0{,}3193 \cdot \ln\left({}^{d}/_{b}\right) + 2{,}5139$$
$$For\ 0{,}7 < {}^{d}/_{b} < 5 \qquad c_{f,0} = -0{,}7121 \cdot \ln\left({}^{d}/_{b}\right) + 2{,}1460$$

The reduction factor ψ_r is set to 1,0 due to the sharp edges of the building according to SS-EN 1991-1-4 Figure 7.24. The reduction factor ψ_λ is determined from Figure 7.36 in SS-EN 1991-1-4 using the slenderness λ and the degree of fullness φ. The slenderness is calculated by the following formula.

$$\lambda = {}^{h}/_{d}$$

The degree of fullness, φ, is assumed to be 1,0.

The density of air, ρ_{air}, is set at 1,25 kg/m^3 according to SS-EN 1991-1-4 Section 4.5.

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
	y_C	11,610	9,749	11,610	9,749
Kármán's wind energy spectrum	F	0,022	0,025	0,022	0,025
Size factor regarding width	ϕ_b	0,182	0,210	0,182	0,210
Size factor regarding height	ϕ_h	0,260	0,295	0,260	0,295
Structural damping	δ_s	0,09	0,09	0,09	0,09
Size ratio	d/b	1,519	1,519	0,658	0,658
Shape factor	$c_{f,0}$	1,848	1,848	2,012	2,012
Reduction factor, edges	ψ_r	1,0	1,0	1,0	1,0
Slenderness	λ	0,669	0,669	1,016	1,016
Reduction factor, slenderness	ψ_λ	0,6	0,6	0,6	0,6
Degree of fullness	φ	1,0	1,0	1,0	1,0
Shape factor	c_f	1,109	1,109	1,207	1,207
Density of air	ρ_{air}	1,25 kg/m^3	1,25 kg/m^3	1,25 kg/m^3	1,25 kg/m^3
Aerodynamic damping	δ_a	0,0036	0,0043	0,0060	0,0072
Resonance response	R^2	0,0713	0,1038	0,0695	0,1008

The tip factor k_p can be determined using formula (3.22b) in BSV 97.

$$k_p = \sqrt{2 \cdot \ln(600 \cdot f_e)} + \frac{0{,}58}{\sqrt{2 \cdot \ln(600 \cdot f_e)}}$$

The equivalent frequency f_e is determined using following formula.

$$f_e = n_1 \sqrt{\frac{R^2}{B^2 + R^2}}$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Equivalent frequency	f_e	0,4215 Hz	0,4202 Hz	0,4224 Hz	0,4202 Hz
Tip factor	k_p	3,5009	3,5000	3,5015	3,5001

The standard deviation for the acceleration in the direction of the wind, σ, can be calculated using the following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1).

$$\sigma(z) = \frac{3 \cdot I_v(z) \cdot R \cdot q_m(z) \cdot b \cdot c_f \cdot \phi_1(z)}{m_e}$$

The velocity pressure at height z, $q_m(z)$ can be calculated by following formula.

$$q_m = \frac{1}{2} \cdot \rho_{air} \cdot {v_m}^2$$

The mode function ϕ_1 can be determined using following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1).

$$\phi_1(z) = \left(\frac{z}{h}\right)^{1,5}$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Velocity pressure	q_m	217,3	217,3	217,3	217,3
Mode function	ϕ_1	0,744	0,744	0,744	0,744
Standard deviation of the acceleration	$\sigma(z)$	0,0102 m/s^2	0,0123 m/s^2	0,0167 m/s^2	0,0201 m/s^2

The peak acceleration, a_{max}, is then determined by following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1).

$$a_{max} = \sigma(z) \cdot k_p$$

Peak acceleration	a_{max}	0,036 m/s^2	0,043 m/s^2	0,058 m/s^2	0,070 m/s^2

Four storey building

Building's properties

Terrain category	II		
Height of building	h	12,387	m
Length of building	l	27,5	m
Width of building	w	18,1	m
Height of top floor	z	9,006	m

Results from RFEM simulation

		Wind on short side				Wind on long side			
		k=0,8		k=0,35		k=0,8		k=0,35	
First natural frequency	n_1	2,575	Hz	1,543	Hz	2,575	Hz	1,543	Hz
Total self-weight	m_{total}	500651	kg	500651	kg	500651	kg	500651	kg
Approximate equivalent mass	$m_e = m_{total}/h$	40417	kg/m	40417	kg/m	40417	kg/m	40417	kg/m

Calculations

Calculation of reference wind speed, v_T, done by using following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1)

$$v_T = 0,75 \cdot v_{50} \sqrt{1 - 0,2 \cdot \ln\left(-ln\left(1 - \frac{1}{T}\right)\right)}$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Reference wind speed (50y return period)	$v_{b,50y}$	24 m/s	24 m/s	24 m/s	24 m/s
Desired return period	T	2 years	2 years	2 years	2 years
Reference wind speed, 2y return period	$v_{b,2y}$	18,648 m/s	18,648 m/s	18,648 m/s	18,648 m/s

Calculation of mean wind speed, v_m, for a 2 year return period is done by using formula (4.3) in SS-EN 1991-1-4

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b$$

Where the roughness factor, c_r, is calculated by formula (4.4) in SS-EN 1991-1-4

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

The terrain factor, k_r, is calculated by formula (4.5) in SS-EN 1991-1-4

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

The orography factor, $c_0(z)$, is set to 1,0 according to SS-EN 1991-1-4

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Roughness length	z_0	0,05 m	0,05 m	0,05 m	0,05 m
Reference roughness length	$z_{0,II}$	0,05 m	0,05 m	0,05 m	0,05 m
Terrain factor	k_r	0,19	0,19	0,19	0,19
Roughness factor	$c_r(z)$	0,987	0,987	0,987	0,987
Orography factor	$c_0(z)$	1	1	1	1
Mean wind speed (2y return period)	$v_m(z)$	18,402 m/s	18,402 m/s	18,402 m/s	18,402 m/s

The turbulence intensity, $I_v(z)$, can be determined by using formula (4.7) in SS-EN 1991-1-4 with k_l set to 1,0

$$I_v(z) = \frac{k_l}{c_0(z) \cdot ln\left(\frac{z}{z_0}\right)}$$

Turbulence intensity	$I_v(z)$	0,193	0,193	0,193	0,193

The background response, B^2, is calculated using following formula from EKS 10 Chapter 1.1.4 Section 6.3.1(1) with reference height, h_{ref}, set to 10 m according to BSV 97 Section 3.22

$$B^2 = exp\left(-0{,}05\left(\frac{h}{h_{ref}}\right) + \left(1 - \frac{b}{h}\right)\left(0{,}04 + 0{,}01\left(\frac{h}{h_{ref}}\right)\right)\right)$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Reference height	h_{ref}	10 m	10 m	10 m	10 m
Background response	B^2	0,918	0,918	0,882	0,882

The resonance response, R^2, is calculated using following formula from EKS 10 Section 6.3.1(1).

$$R^2 = \frac{2\pi \cdot F \cdot \phi_b \cdot \phi_h}{\delta_s + \delta_a}$$

The factor, F, which refers to Kármán's wind energy spectrum is calculated by following formula from EKS 10 Chapter 1.1.4 Section 6.3.1(1).

$$F = \frac{4 \cdot y_C}{(1 + 70{,}8 \cdot {y_C}^2)^{5/6}}$$

$$y_C = \frac{150 \cdot n_1}{v_m(z)}$$

The factors ϕ_b and ϕ_h depends on the size with regard to width and height respectively and are calculated by following formulas from EKS 10 Chapter 1.1.4 Section 6.3.1(1).

$$\phi_h = \frac{1}{1 + \frac{2 \cdot n_1 \cdot h}{v_m(z)}}$$

$$\phi_b = \frac{1}{1 + \frac{3{,}2 \cdot n_1 \cdot b}{v_m(z)}}$$

The δ factors refers to damping. The structural damping, δ_s, is set to 0,09 according to BSV 97 Table 3.22a. The aerodynamic damping, δ_a, is calculated using formula (F.18) SS-EN 1991-1-4.

$$\delta_a = \frac{c_f \cdot \rho_{air} \cdot b \cdot v_m(z)}{2 \cdot n_1 \cdot m_e}$$

The shape factor, c_f, is calculated using formula (7.9) in SS-EN 1991-1-4.

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda$$

The shape factor, $c_{f,0}$, is calculated using following formulas from SS-EN 1991-1-4 in Figure 7.23

$$For\ 0{,}2 < {}^{d}/_{b} < 0{,}7 \qquad c_{f,0} = 0{,}3193 \cdot \ln\left({}^{d}/_{b}\right) + 2{,}5139$$
$$For\ 0{,}7 < {}^{d}/_{b} < 5 \qquad c_{f,0} = -0{,}7121 \cdot \ln\left({}^{d}/_{b}\right) + 2{,}1460$$

The reduction factor ψ_r is set to 1,0 due to the sharp edges of the building according to SS-EN 1991-1-4 Figure 7.24. The reduction factor ψ_λ is determined from Figure 7.36 in SS-EN 1991-1-4 using the slenderness λ and the degree of fullness φ. The slenderness is calculated by the following formula.

$$\lambda = {}^{h}/_{d}$$

The degree of fullness, φ, is assumed to be 1,0.

The density of air, ρ_{air}, is set at 1,25 kg/m^3 according to SS-EN 1991-1-4 Section 4.5.

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
	y_C	20,990	12,578	20,990	12,578
Kármán's wind energy spectrum	F	0,015	0,021	0,015	0,021
Size factor regarding width	ϕ_b	0,110	0,171	0,110	0,171
Size factor regarding height	ϕ_h	0,224	0,325	0,224	0,325
Structural damping	δ_s	0,09	0,09	0,09	0,09
Size ratio	d/b	1,519	1,519	0,658	0,658
Shape factor	$c_{f,0}$	1,848	1,848	2,012	2,012
Reduction factor, edges	ψ_r	1,0	1,0	1,0	1,0
Slenderness	λ	0,450	0,450	0,684	0,684
Reduction factor, slenderness	ψ_λ	0,6	0,6	0,6	0,6
Degree of fullness	φ	1,0	1,0	1,0	1,0
Shape factor	c_f	1,109	1,109	1,207	1,207
Density of air	ρ_{air}	1,25 kg/m^3	1,25 kg/m^3	1,25 kg/m^3	1,25 kg/m^3
Aerodynamic damping	δ_a	0,0022	0,0037	0,0037	0,0061
Resonance response	R^2	0,0253	0,0790	0,0249	0,0771

The tip factor k_p can be determined using formula (3.22b) in BSV 97.

$$k_p = \sqrt{2 \cdot \ln(600 \cdot f_e)} + \frac{0{,}58}{\sqrt{2 \cdot \ln(600 \cdot f_e)}}$$

The equivalent frequency f_e is determined using following formula.

$$f_e = n_1 \sqrt{\frac{R^2}{B^2 + R^2}}$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Equivalent frequency	f_e	0,4218 Hz	0,4346 Hz	0,4268 Hz	0,4374 Hz
Tip factor	k_p	3,5011	3,5096	3,5045	3,5115

The standard deviation for the acceleration in the direction of the wind, σ, can be calculated using the following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1).

$$\sigma(z) = \frac{3 \cdot I_v(z) \cdot R \cdot q_m(z) \cdot b \cdot c_f \cdot \phi_1(z)}{m_e}$$

The velocity pressure at height z, $q_m(z)$ can be calculated by following formula.

$$q_m = \frac{1}{2} \cdot \rho_{air} \cdot {v_m}^2$$

The mode function ϕ_1 can be determined using following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1).

$$\phi_1(z) = \left(\frac{z}{h}\right)^{1,5}$$

		Wind on short side		Wind on long side	
		k=0,8	k=0,35	k=0,8	k=0,35
Velocity pressure	q_m	217,3	217,3	217,3	217,3
Mode function	ϕ_1	0,620	0,620	0,620	0,620
Standard deviation of the acceleration	σ(z)	0,0061 m/s^2	0,0109 m/s^2	0,0101 m/s^2	0,0177 m/s^2

The peak acceleration, a_{max}, is then determined by following formula from EKS 10 Chapter 1.1.4 Section 6.3.2(1).

$$a_{max} = \sigma(z) \cdot k_p$$

Peak acceleration	a_{max}	0,022 m/s^2	0,038 m/s^2	0,035 m/s^2	0,062 m/s^2