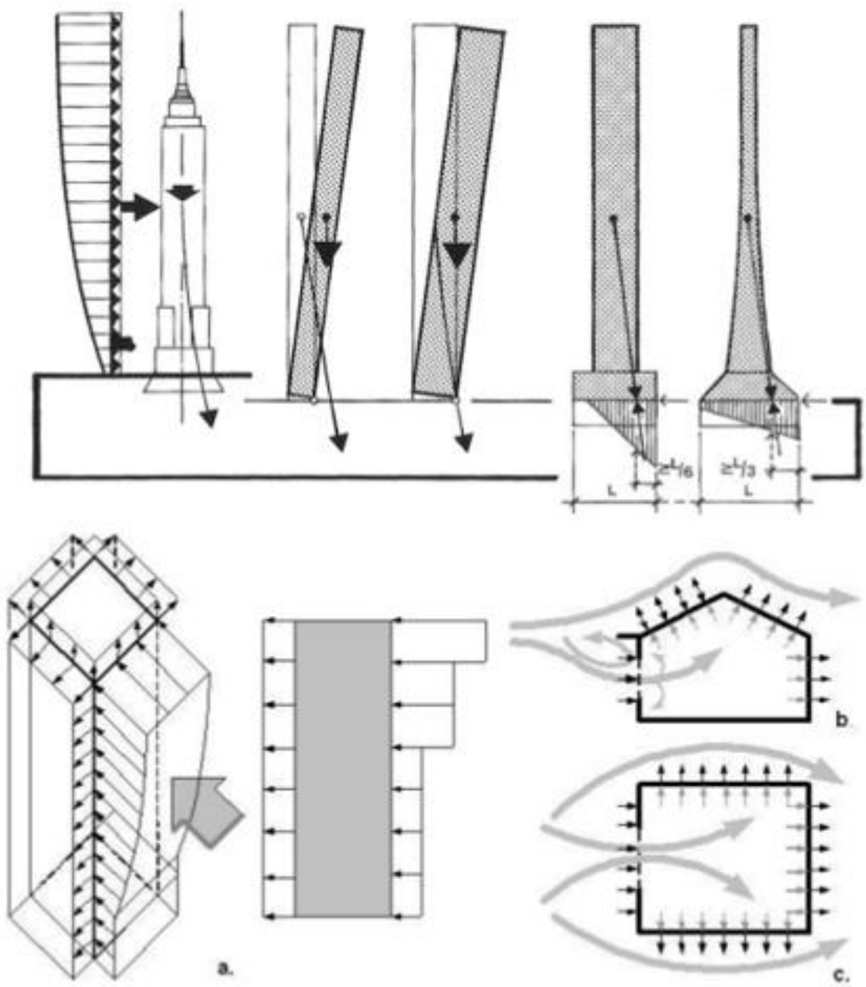
Stabilization of buildings by timber-glass facades



A.P. Azamy

MSc Civil Engineering-Structural Design

Stabilization of buildings by timber-glass façades

By

Ahmad Parwiz Azamy

in partial fulfillment of the requirements for the degree of

Master of Science

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Thesis committee TU-Delft:

Prof. dr. Jan Rots,	professor of structural mechanic
ir. Chris Noteboom	researcher & lecturer of structural glass
ir. Paul Korswagen	researcher & lecturer of applied mechanics.
Dr.ir. Roel Schippers	researcher & lecturer, track coordinator of the MSc CEG track Structural Engineering

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I. PREFACE

This research is drafted to defend my Master of Science degree in structural design at the University of civil engineering and Geoscience at the Delft University of Technology. This thesis was part of a double degree program, but due to personal circumstances, I decided at almost the end of the program to accomplish only one of my master's. The research is due to the corona pandemic executed at the Technical University of Delft under the supervision of Prof. Jan Rots, ir. Chris Noteboom, ir. Paul korswagen and dr.ir Roel Schippers (as pro-forma) without the involvement of any external firm.

I sincerely thank Chris Noteboom, my daily supervisor, for his significant input. His knowledge and passion for structural glass inspired me to choose my thesis research in this field. Besides sharing his professional knowledge, helpful comments, and feedback, he also supported me during the lockdown.

Paul korswagen assisted me with the finite modeling of various structural glass models. He taught me how to simplify a structural model to simulate finite element programs. His involvement, comments, and feedback gave me the strength to think and go beyond the boundary of this research. Therefore, I thank him for his advice and for sharing his structural skills and guidance.

Last but not least, my appraisal of Professor Jan Rots for being the chairman of my thesis committee. Despite his full agenda and supervising various researchers, he agreed to support me with my thesis. Thank you for sharing your knowledge and providing critical comments and feedback.

Furthermore, my truthful appreciation toward Dr.ir Roel Schipper for his support and understanding during my entire master's track, the arrangement and establishment of this thesis topic and the accomplishment.

I am also grateful to my wife for being patient and supporting me in accomplishing this research despite our busy lives and jobs. My sincere appraisal is also toward the entire civil engineering team at Delft University. This research results from their guidance and unconditional support, which enabled me to complete my second master's degree.

A.P. Azamy   Delft, May 2023

II. Summary

Glass is a well-known material with extensive applications in different fields. Although glass was invented about 3500-5000 BC, its application in the building industry is dated from the early 17th century. However, despite its outstanding compression strength (better than concrete), its structural application in the building industry still needs to be improved. Therefore, this research is drafted in the field of structural glass to participate in the further development of structural glass and to investigate the suitability of timber-glass façades as load bearing- and stability elements.

Currently, glass facades are applied in the buildings to bring daylight inside. These openings are assumed to weaken the structures. The structural properties and contribution of glass as a structural material could be addressed and designed to anticipate load-bearing. Three real projects are considered for this research to demonstrate the load-bearing capacity and confirm that large glass windows and facades can stabilize buildings. The similarity between these projects is the availability of large glass façades without any structural function. By studying these three projects, the appropriateness of the glass façade as a load-bearing and stability element is investigated in existing buildings as well as in new buildings.

The research starts with enhanced studying and discussing the chemical composition of the glass material, its physical strength and appropriateness for various fields, and the standard dimensions under literature study. The connection between the glass panes and the surrounding structural elements is also an important topic discussed based on available literature. A distinction is made between bolted, heat-bonded, and glued connections. The literature study is extended by examining the existing stability systems. This was necessary to understand their functionality, advantages, disadvantages, and limitations to develop a proper and effective hybrid system with a timber-glass façade.

Stabilization of the buildings in the Groningen province against seismic load investigated by Kisa (2021) and De Groot (2019) is interesting and compared with the findings of this research. In that research is laminated annealed glass used with a thickness of 20 mm (2x10 mm) and a surface of 800 mm by 1500 mm glued with Sikaflex-252 to a hardwood frame with a strength class of D60 (dimension 65 mm by 100 mm). Based on different experiments was concluded that a thinner adhesive will not improve the overall structural

behavior of the design. Thicker adhesive can increase deflection and reduce the rigidity of the structural window. Applying structural windows undergoing a shear force between 37 kN to 42 kN improved the in-plane strengthening of the houses between 137% and 300%.

The appropriateness of the timber-glass façade as load bearing and stability element of this research is investigated through three study projects. The study of these projects, development of structural models and verification through manual calculation, and finite element models can provide answers to whether the glass is suitable as a load-bearing and stability element.

Case study one is focused on the expansion of the existing buildings. Most expansion buildings have large glass façades to enable sufficient light entrance. However, these glass façades are currently not designed to participate in load bearing or stabilization of the structures. To stabilize case study one, the glass pane with a length of 4 m, height of 2.6 m, and thickness of 20 mm (2x10mm) is applied to resist the vertical and horizontal loads.

Case study two is also about an existing building with broader challenges. The expansion part is stabilized by steel portals around the glass pane, where glass has no structural function. To ensure the stability of the building, the cross-section of the steel element had to be increased by about 205%. The cross-section of the steel elements is not required to resist the combined loads but to prevent horizontal displacement. This was a suitable project to emphasize the vertical and horizontal load-bearing capacity of the timber glass façade. With 2D and 3D calculations is confirmed that the steel portal can be combined with a timber-glass façade to realize the perfect hybrid system. By activating the glass pane with a length of 4.2m, height of 2.6m, and 20 mm thickness the cross-section of the steel portal can be reduced by 105%.

Case study three is about a tall building of about 60 m. The structure is stabilized mainly by a concrete lift shaft at the ground level in the X- and Y-direction. At the same time, the longitudinal facades are entirely designed in glass (figures 43 & 44). Glass facades are excluded from the stabilization and load-bearing of the buildings, which causes tension in the foundation under a limited number of stability elements at the ground level. To emphasize the importance and contribution of the timber-glass façades, the tension in the foundation is manually calculated first. Then the building is designed with a 3D finite element program to confirm the tension in the foundation piles. Based on these calculations, the minimum number of timber-glass facades with a thickness of 60 mm (5x12 mm) are activated to stabilize the building and prevent tension in the foundation.

The first two case studies underlined also the load bearing and the possibility of stabilizing structures by timber-glass facades with different dimensions and forces. The magnitude of the wind loads in case study three was 33 times higher compared to case study two. The ratio between the required length was more than six times larger, and the required glass was almost three times thicker. Based on this can be concluded, that variation in the glass dimension can enable tailor-made design for different structures and their corresponding loads.

CONTENT

1 Introduction

This thesis studies the stabilization of buildings with the application of timber frames and glass facades. Stabilizing buildings is an essential aspect of structural engineering, which is evaluated early in the design process. This can prevent any changes in the architectural and structural design afterward. Stability elements are generally vertical load-bearing members (desired, not must) and will strengthen the building against unproportioned (large horizontal) displacement, torsional buckling, and twist due to wind compression, tension, suction, friction, and turbulation. Buildings also have to be stabilized against vibration due to the earth's quick or heavy railway traffic, which is outside the scope of this research.

Up to date, the buildings are stabilized by load-bearing elements (mainly walls) with sufficient bending stiffness. These walls are found to resist wind loads and minimize the horizontal deflection of the building. Walls with large openings for the glass windows or curtain walls are not considered to be used and are activated as a stability element. The openings are considered as a weakness, and the glass is a non-structural component. However, various research of the last decades has proved that glass consists of higher compression strength (approximately 1000 N/mm2) compared to concrete. It should also be noted that glass is very ductile and has a significantly lower bending tensile strength than its compression capacity (Kozlowski, 2019).

This research investigates the possibility of using a timber frame with a glass facade as a stability member. Execution of the practical investigation is due to the COVID-19 pandemic not possible. Nevertheless, for the numerical analysis, the outcome of the tests executed in the lab of the Delft University of Technology (hereafter referred to as TU-Delft) is applied. These experiments were executed to determine the stiffness of the framed glass panels to stabilize the buildings in the province of Groningen in the Netherlands against Earth quick.

1.1 Thesis topic

This research topic is called "stabilization of buildings by timber-glass façade." The research will be focused, i.e., on the bending stiffness and tensile strength of the glass panels enclosed by a timber frame. Due to glass members, walls with large openings are not considered as suitable stability elements. By ignoring the glass member out of consideration, the bending stiffness of the non-homogeneous wall becomes relatively small and negligible as a stability element. An existing building is provided in Figure 1, where glass facades are eliminated in the Y-direction as a suitable stability element. To stabilize the building, additional steel portals are designed around the timber-glass panes. Only the thick load-bearing concrete/brick walls are considered and activated as a stability element in X-and Y-directions.

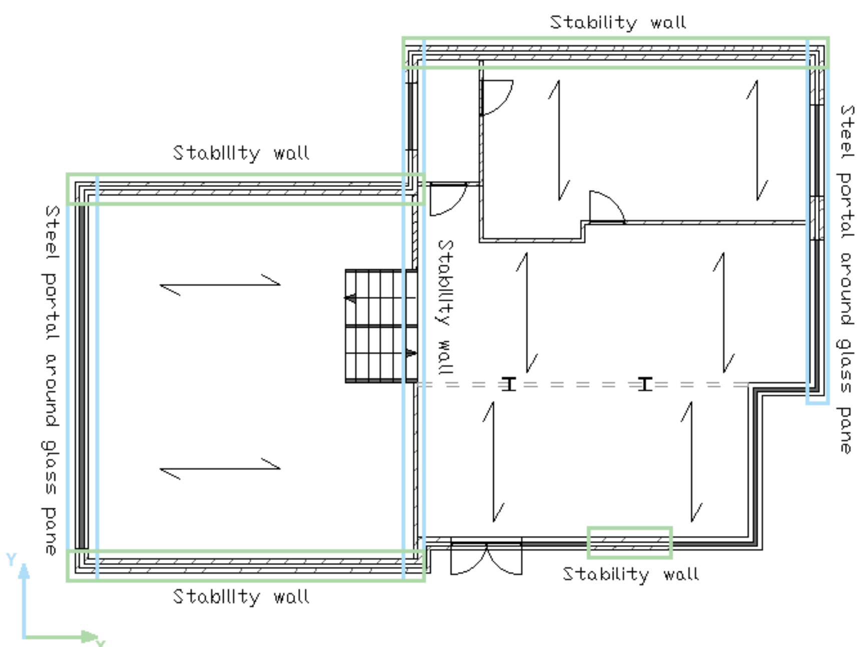


Figure 1: Floorplan with stability walls in X- and Y-direction

Openings in the façades are assumed to reduce the bending stiffness of the walls, which is related to the thickness and the length of the wall. This research aims to increase the stiffness of these nonhomogeneous walls by activating/adding the resistance capacity of the glass member. The general formula for the bending stiffness of a homogeneous structure is demonstrated below and will be elaborated later for the nonhomogeneous structures (walls including glass members).

$$EI = E\left(\frac{tl^3}{12}\right) \quad (1)$$

Equation 1: Bending stiffness of the homogeneous element

1.2 Problem statement

The stabilization of buildings is a well-known phenomenon within structural engineering. There are various techniques and possibilities to stabilize buildings against horizontal wind load. Proper stability options are available based on the applied technology and construction materials. However, the problem statement of this research is focused on the structures interrupted by large openings for the windows and curtain walls. These buildings require additional horizontal supports to resist wind loads.

Buildings with frontage glass façade are assumed to debilitate the structure and are not load bearing. The existing technology to stabilize these buildings requires complementary expenses, which makes the overall costs higher. At the same time, these adjunctive elements may also affect the architectural division and aesthetic view of buildings. By not implementing the glass façade as a stability element, the façade has to be strong to enable the bilateral load redistribution and transfer over to other stability members. Figure 2 demonstrates different buildings with curtain walls. Steel elements and cables are applied to ensure the load transfer and stabilize the structures. The structural capacity of glass members is neglected.

Figure 2: stabilizing and connection of glass façade

Stabilization of the buildings with a timber-glass façade is also not without challenges. Glass as load bearing member is a relatively new concept and is vulnerable to tension and bending tensile stresses. Stabilizing the buildings with timber-glass façade can cause tension in the glass pane and the applied connections. Due to the glass's brittle behavior and low-tension bearing capacity, the timber-glass façade could fail. A careful and new detailed design for the connection between the glass panel and timber frame is designed and discussed in chapter five.

Another critical consideration in this research is the application of the load on glass members. A clear distinction is made between line and point load, with glass thickness and panel width as constant parameters.

1.3 Research objective

The main objective of this research is to apply the theoretical knowledge of building engineering to a daily problem. Building Engineering is focused on the construction industry and providing solutions to improve the overall building performance ethically, considering the environmental aspects.

Structural glass is in development, and the scholars of the technical university of Delft are actively participating in the worldwide progress of this construction material. Different lab research and structural modules were developed by the glass & transparency research group of TU-Delft for buildings, bridges, and other glass structural applications. In partial fulfillment of the requirement for the degree of Master of Science in the field of building engineering, this thesis research is done in the field of structural glass. This thesis analyzes the structural glass's behavior under bilateral compression, bending, and tensile stresses. The focus is to evaluate, numerically and analytically, whether a timber-glass façade could stabilize buildings against horizontal wind loads. The timber-glass façade will also be loaded beside compression (vertical) by wind load s well. The wind load will cause compression, bending, tension, and friction.

An equalized balanced loading of the timber-glass façade is studied in this research. Figure 3 demonstrates various wind loads working on a random building.

Figure 3: loads on the building

1.4 Research questions

The content of this research focuses on providing answers to the below-listed research questions. The main research question is:

- **Can timber-glass facades participate in the stabilization of buildings?**

1.4.1 Sub questions

Besides the abovementioned question, the following sub-questions are answered to strengthen this research's content.

- How to design and apply the timber-glass façade as a stability element to eliminate or respond to the tension and bending stresses?

- How to design the connection between the glass façade and the prominent load-bearing members without exceeding the glass pane's compression, tension, and shear resistance capacity?

- Is the timber-glass facade exclusively suitable as the main stability element? Or is it also possible to design hybrid systems to strengthen the existing buildings for expansion/renovation by combining them with present techniques?

1.5 Scope of research

This research aims to investigate the possibility of stabilizing buildings with a timber-glass façade. This report research and discuss several aspects of structural glass and the content is subdivided into the following subparts for evaluation.

I. First, the glass material's chemical composition, mechanical behavior, and aesthetic aspects will be investigated based on existing literature.

II. To identify the magnitude of the horizontal wind loads on a random design, the wind load division associated with the wind area and structure height will is elaborated.

III. Force distribution and division per stability element are correlated to the bending stiffness of the stability element. Therefore, a particular timber-glass façade's bending stiffness must be analyzed and discussed.

IV. Performing numerical evaluation supported by finite element analysis is required to understand the stress distribution and load transfer behavior through a timber-glass façade.

This thesis's central aspect is identifying and determining the load-bearing capacity of the timber-glass façade. Therefore, various existing and new building designs with the possibility of applying a timber-glass façade as a stability element will be evaluated and discussed. The analysis will be strengthened by manual calculation and improved by finite element models. However, the existing techniques are also studied to underline the main differences with other construction materials.

The content of this thesis is drafted according to academic guidelines enforced by existing research. The outcomes will be presented in the conclusion, and the research is accomplished with sufficient findings, including recommendations and future investigations.

2 LITERATURE REVIEW

Prior to discussing the load-bearing capacity and possibility of stabilization of buildings by timber-glass façade, the existing literature in this field will be reviewed. The objective is to build further on the existing findings and not invent the wheel repeatedly.

Although glass was produced more than thousands of years ago for the first time, its main application in the building industry was, until some decades ago, limited to the accommodation of the light entrance. Therefore, the structural behavior and type of structural glass material will be discussed in this chapter to highlight the main properties from a different perspective. This will clarify some structural design aspects and the capture effect. The content of this chapter is subdivided into the following paragraphs.

- **struct. glass**
  - It's important to elaborate first on the existence and history of glass. The physical behavior and chemical composition of glass is studied in this topic.
- **connec-tion**
  - Connection of the glass pane to the timber frame and the surrounding structures is very important, which is discussed in this paragraph.
- **bolted conec.**
  - Bolt and embedment connection is quite a popular technique to create a strong and elegant connection of the glass pane with the remaining (steel) structures.
- **heat bonded**
  - Although heat bonded connection is generally used by artists to connect glass members to each other, however this technique is also studied to complete the research.
- **adhesive conec.**
  - Adhesive connection is the most appropriate connection type. This technique is discussed extensively to investigate the application criteria for this research.
- **existing models**
  - The existing research in the field of structural glass related to this topic is presented in this paragraph. The highlights are mentioned with appropriate references.

2.1 Structural glass

The glass was already in the stone age and used by humans to make weapons from black Vulcanic glass and decorative objects. In contrast, its function changed over the years after the invention of glass production technology about 3500-5000 BC. The first glass used in the building industry is dated to being applied for decorative purposes in churches (history of glass, 2021).

The main component to produce glass is sand (silicon dioxide $SiO2$), limestone (calcium carbonate $CaCO3$), and soda (sodium carbonate $Na2SO3$). Nevertheless, fused silica is an excellent glass where no other component is required, and the end product is excellent. However, the melting point where silica starts to crystalize is around 1700 C°, and to heat the furnace to this temperature, the production cost of glass will increase enormously. Therefore, adding extra supplements like soda and limestone is required to decrease the smelting point of sand up to 850 C° or lower. These supplements make glass production cost-effective and environmentally friendly (Augustyn, 2021).

The application field of glass is quite large, from the building industry to cars, airplanes, kitchen equipment, computers, sunglasses, microscopes, and many more. Therefore, every field requires a different type of glass and different components to affect the glass properties like strength, thickness, brightness, color, and more. Glass components and their (non-structural) properties are discussed in different kinds of literature. Therefore, it is redundant to mention them here again, and is therefore left out of the scope of this research.

The general perception of glass is that it is vulnerable and easy to break. Due to the application area and the slenderness of the objects, the compression strength is only sometimes expressed. However, the compression strength of the glass material is even higher than regular concrete used in the building industry, while the density is comparable to reinforced concrete. The main properties of the glass material are tested in different labs worldwide and published in various works of literature. These properties are now integrated into modern state-of-the-art programs to design and evaluate glass structures using finite element methods. Table 1 presents the essential properties of glass material mentioned in NEN-EN572-1 (NEN-EN, 2021).

Properties of glass material

Density	ρ_g	25000	N/m^3
Compression strength	$f_{g;c}$	1000	N/mm^2
Tensile strength (annealed glass)	$f_{g;k}$	± 6.0	N/mm^2
Young's modulus	E_g	70.000	N/mm^2
Poisson's ratio	v_g	0,23	

Table 1: properties of glass material

Technical development eased and increased the application field of glass materials. The modern glass material is no longer used only as a decorative element or to allow light to enter through windows and frontage but also functions as a load-bearing element. The glass & transparency research group of TU-Delft, under Professor Rob Nijsse, has designed and developed various international award-winning projects. Figure 4 presents an overview of three glass projects of TU-Delft, which are (left to right) the Glass bridge at Delft, the Glass façade in Amsterdam, and the glass swing experiment). These projects emphasize the load-bearing capacity and aesthetic value of glass materials. The typical characteristic of these projects is that glass is loaded mainly on compression stresses.

Figure 4: TU-Delft glass projects

Glass is a brittle material with very high compression strength. Using the compression strength of glass material correctly makes glass suitable as a load-bearing and stability element.

To emphasize the brittleness of the glass material, the yield point of steel with glass material is compared in Figure 5. Steel structures are going to deform irreversibly once the yield point is crossed. However, in this plastic deformation stage, the steel structure does not fail, and load redistribution will start where strain will increase until failure. This will allow us to redistribute the load and use the steel profiles.

Steel elements are generally warned of large deformation before failure. Glass acts differently and will fail once the yield point is reached without any sign or warning.

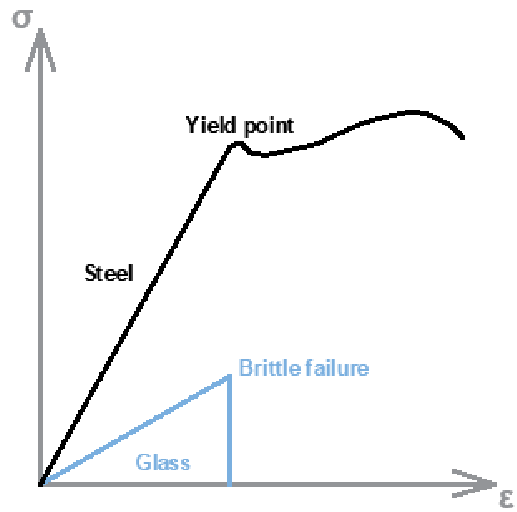


Figure 5: yield point steel vs glass

The brittleness of glass correlated with the density is studied and discussed by J. Sehgal in his book Brittleness of glass (Sehgal, 1999). In his experiments, he relates the glass to its density. The lower the density, the easier the glass will break. Besides the brittleness, the plasticity of the glass was also studied and correlated to the density. The brittleness of ordinary glass decreases with decreasing density due to plastic flow and densification ease.

2.1.1 Structural glass in construction

Only some types of glass can be applied as a structural component in building construction. The structural glass must possess sufficient strength, and the damage should be limited to failure. The behavior of structural glass is investigated and discussed in different literature. Dierks studied the possibilities of stabilizing a high-strength steel column with structural glass. He discussed his findings in a scientific paper, "A slender transparent glass supported high-strength steel column." He used heat-strengthened float glass in numerical and lab experiments (D. Dierks et al., 2014). This research verifies that heat-strengthened glass is a perfect structural material for load-bearing and supporting members.

The heat-strengthened floating glass panes support the slender high, strength steel column laterally. The glass panes are connected through the epoxy adhesive and bolt connection to the Dywidag bar. An overview of the structural drawing is presented in Figure 6.

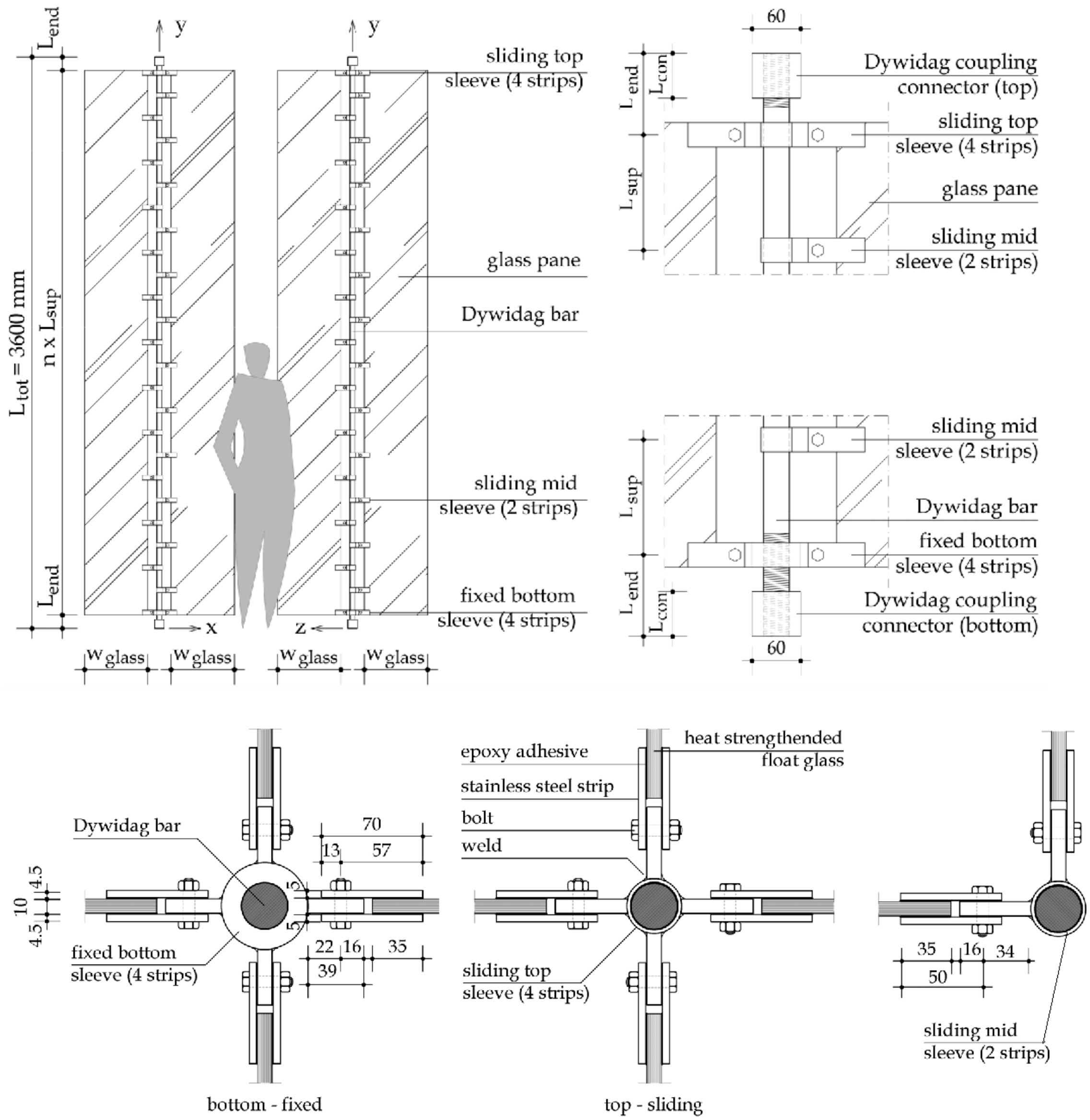
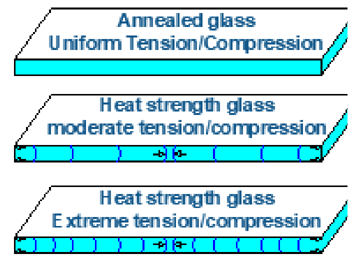


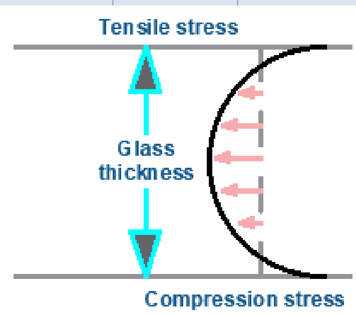
Figure 6: Dierks steel column supported by glass panes

Luible investigated in his Ph.D. research the stability of glass columns as load-bearing members. In contrast to Dierks et al., he used laminated tempered glass for his analytical and numerical models. Tempered glass is also known as thermally toughened glass (TTG), which is stronger than heat-strengthened glass but can take up relatively lower forces after failure. He also emphasized the efficiency and transparency of glass columns as future structural members (Luible, 2004).

There is also much other research discussing the various structural fields of glass and its application field. Table 2 presents the most suitable structural glass types with their structural performances investigated and applied in recent studies.

Type and main properties of structural glass			
Type of structural glass	**Bending stress f_k**	**Surface pressure**	**Standard thickness**
ANNEALED GLASS (ORDINARY FLOAT GLASS) Float glass, also known as annealed glass, is the most common type of glass with a large application field and is very suitable to be laminated and used as a structural element. The melting silicate (sand), lime, and soda in the furnace will float onto a large bed of molten tin. This mixture slowly solidifies over the molten tin as it enters the oven traveling along rollers under a controlled cooling process. From this point, the glass emerges in one continuous ribbon, then cut and further processed for customers' needs.	+/- 45	0	2-19
HEAT-STRENGTHENED GLASS (HSG) Heat-strengthening glass is about twice as strong as annealed glass. By undergoing heat treatment, the breakage potential due to wind and thermal stress will be reduced. The difference with tempered glass is in the cooling process. Heat-strengthened glass is cooled slower than tempered glass and therefore has lower compression strength, which is still higher than annealed glass. The main difference between the heat-strengthened and tempered glass is the breakage characteristic. Where tempered glass will break into small rough pieces, heat-strength glass will break into relatively large pieces.	+/- 70 N/mm^2	30-50 MPa	2-19 mm
TEMPERED GLASS OR THERMALLY TOUGHENED GLASS (TTG) Heat-strengthening glass is about twice as strong as annealed glass. By undergoing heat treatment, the breakage potential due to wind and thermal stress will be reduced. The difference with tempered glass is in the cooling process. Heat-strengthened glass is cooled slower than tempered glass and therefore has lower compression strength, which is still higher than annealed glass. The main difference between the heat-strengthened and tempered glass is the breakage characteristic. Where tempered glass will break into small rough pieces, heat-strength glass will break into relatively large pieces.	+/- 120 N/mm^2	> 90 MPa	2-19 mm





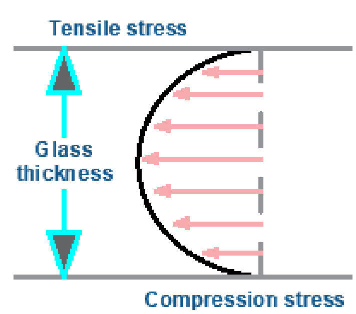


Table 2: properties of structural glass

2.1.2 Connections

To stabilize buildings with a timber-glass façade, the interconnection between the glass to the timber frame and the primary load-bearing member (concrete, brick, steel) element is essential. This connection must enable beside the vertical (live and dead) loads also the horizontal wind load and the bilateral imposed thermal deformations.

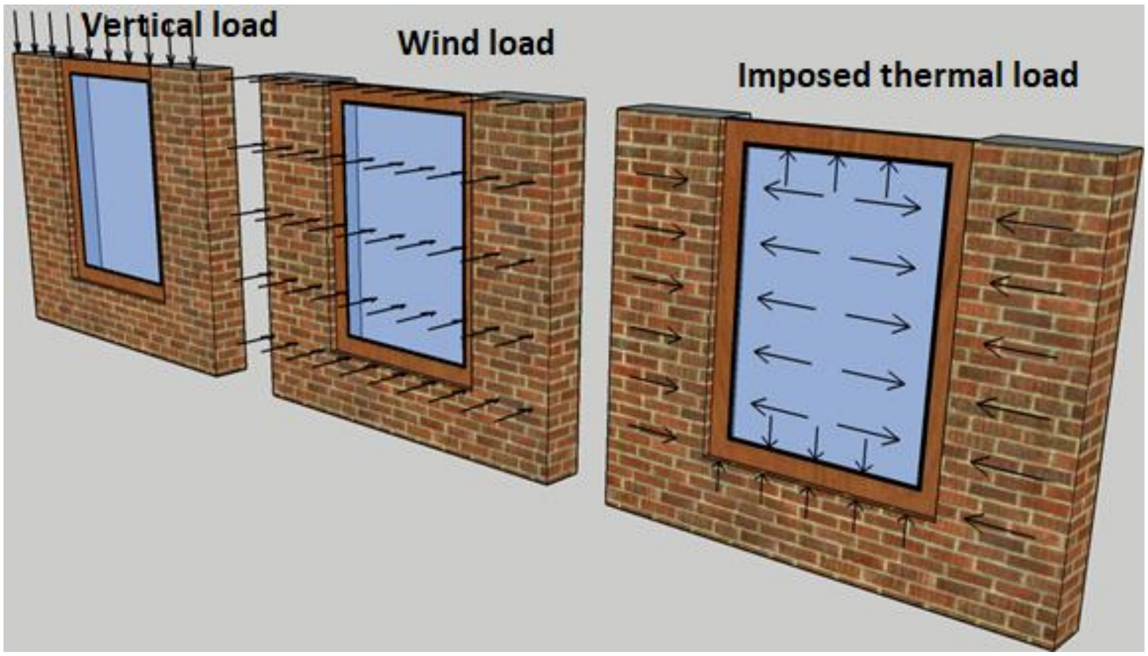


Figure 7: loads on timber-glass facade

Another critical issue to consider while designing the timber-glass façade connection is the thermal and acoustic requirements. Therefore, the connection must prevent/minimize sound and dust entry and air diffusion to reduce heating and cooling.

Figure 8: building physics requirement regarding facade

Load transfer through connection is essential while designing components as load-bearing members. Types of connections are generally interdepended to the magnitude of the applied load. There are different connection types for their application fields. The following connection types are important and discussed in this report.

- bolted and embedment connection
- Heat-bonded connection
- Adhesive connections

2.1.3 Bolt and embedment connection

Connecting through bolts is a well-known technique and applies to different materials. Bolt connections are used to connect timber structures, (prefab)concrete elements, and glass panes or to steel components, steel structures, or steel elements to concrete structures in the building industry. Bolt connections can transfer high amounts of tension and shear forces. However, they also introduce enormous, concentrated tension, compression, and shear stresses to the related material. The forces are very concentrated around the phrased hole of the bolt, especially in thin (glass/steel) plates(Figure 9).

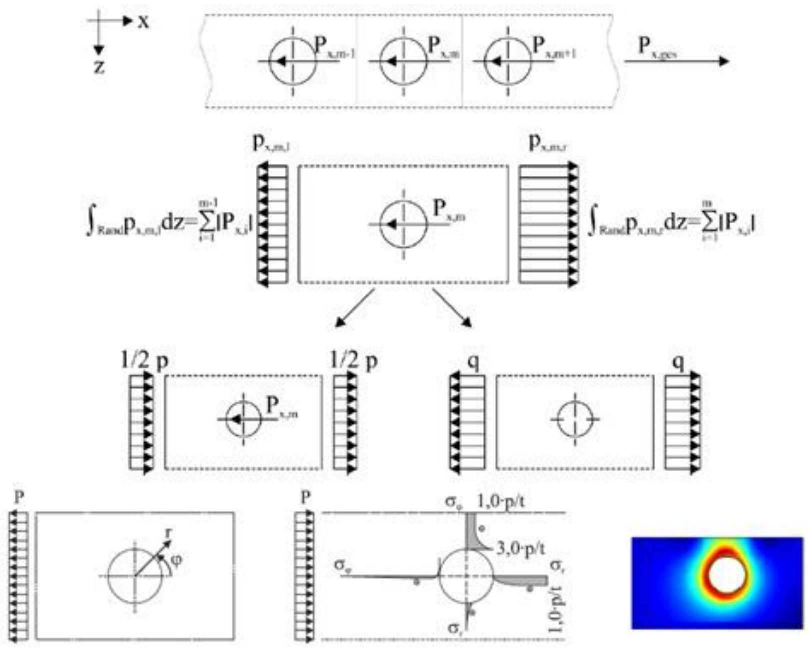


Figure 9: stress distribution in the bolted connection

Glass is a brittle material with high compression strength and minimal tension and shear stress resistance. Several disadvantages affect the bolted connection, like stress intensification at the borehole edges and reduced material resistance due to the drilling process (Feldmann et al, 2014).

Therefore, bolted connections in glass panes are suitable when the magnitude of the transfer load is limited, the distance to the edges is sufficient, and the glass plate is thick enough.

Embedded laminated glass is also a semi-bolt connection, providing an elegant solution to connect glass panels (Figure 10). Santarsiero et al. discussed in their experiment and numerical analysis of thick embedded laminated glass connections and concluded that this type of connection is susceptible to temperature. The embedded connection is also exquisite and suitable for various purposes. The embedded connection is realized by a metal plate encapsulated and glued in multi-ply laminated glass components (Santarsiero, Bedon, & Louter, 2018).

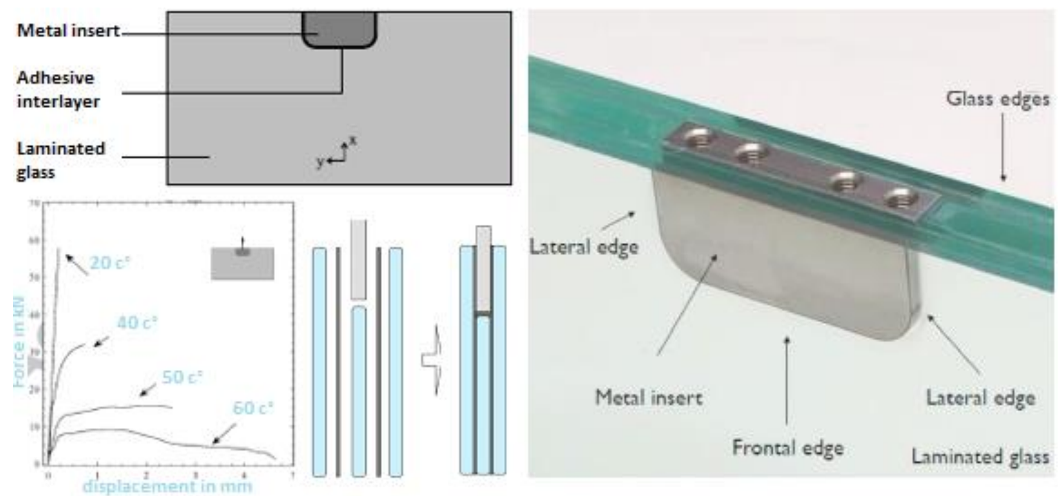


Figure 10: embedded laminated glass connection

The load distribution in an embedded connection is non-linear and transferred through the metal plate to the parallel thin glass and the glued connections of the lateral and frontal edges (Figure 11). Therefore, the failure will occur first in the glass layers around the metal plate where the concentrated forces are the highest under normal circumstances (room temperature).

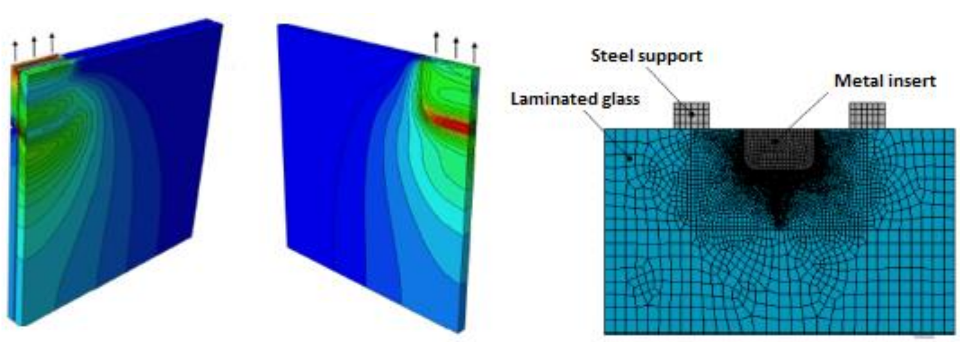


Figure 11: load transfer through embedded connection

2.1.4 Heat-bonded connection

Connecting glass members by bonding could be achieved through local heating of the connection area or glass fusion through global heating. Bonding through local heating is comparable with steel welding. However, the connected glass members will be preheated to prevent substantial local tension stresses, deformation, and thermal shocks. Eskes et al. 2020 studied these two technologies and elaborated on the research related to heat-bonded glass executed by Bos et al. 2007/2008. These investigations proved that weld bonding is more suitable for borosilicate glass than annealed glass due to its lower thermal stress, which prevents thermal shock and failure (Eskes, et al., 2020). The thermal shock of soda lime glass could be reduced by preheating and heating the glass from both sides with a double torch (Jan Belis, 2006). The preheating and welding of glass will occur in the oven, and the size of the connected glass members is related to the oven size.

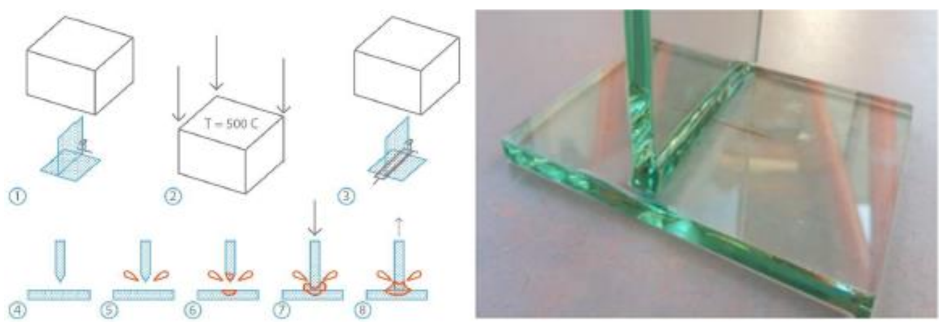
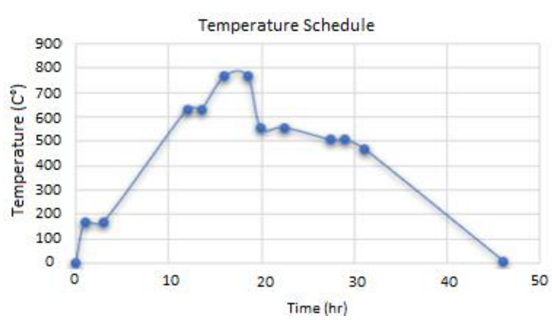


Figure 12: heat bonding of glass by Eske

The connection of glass through fusion is experimented with and studied at TU-Delft. For this experiment, two gypsum molds were designed to place each part of the glass member separately. The molds were placed perpendicularly to each other in the oven to create a T-shaped connection. The maximum temperature in the oven was 765 °C with a cycle period of 45 hours. Dwells, periods of constant temperature, were included at 630 °C to prevent air inclusions (Mitchell, 2015), at the maximum temperature to ensure fusion, and at 555 °C and 505 °C to anneal the glass. The vertical displacement of the specimen due to gravity was 25 mm without any visible crystallization. The main goal was achieved by proving that glass is connectable by fusion. Further influence of this process on glass is left out of this research.



Graph 1: Oven temperature for glass fusion

2.1.5 Adhesive connections

The type of connection is not only related to the aesthetic desire of the designer/owner but also strongly interconnected to the load to transfer, the airtightness and moisture-repellent, the construction environment, and more. Glued connection is the most suitable technique to apply to curtain walls. In contrast to the bolted connection (point contact), where the stresses are concentrated around the drilled opening of the bolts, glued connections enable uniform load transfer. The cross-section of the connected glass elements will be used, reducing and distributing the stresses over a larger area (Figure 13).

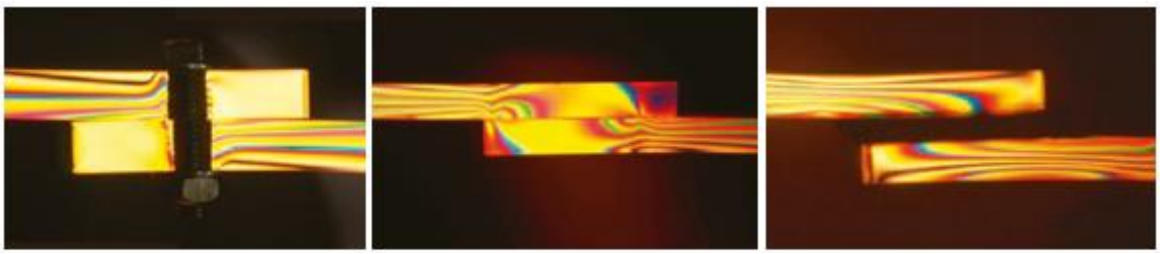


Figure 13: stress distribution in drilled bolted connection, lapped connection with adhesive, and connection with thick elastic adhesive Jan Wurm

The service life of the adhesive is related to the type and duration of the load, the quality of its installation and surfaces, and environmental influences such as UV (Wurm, 2007).

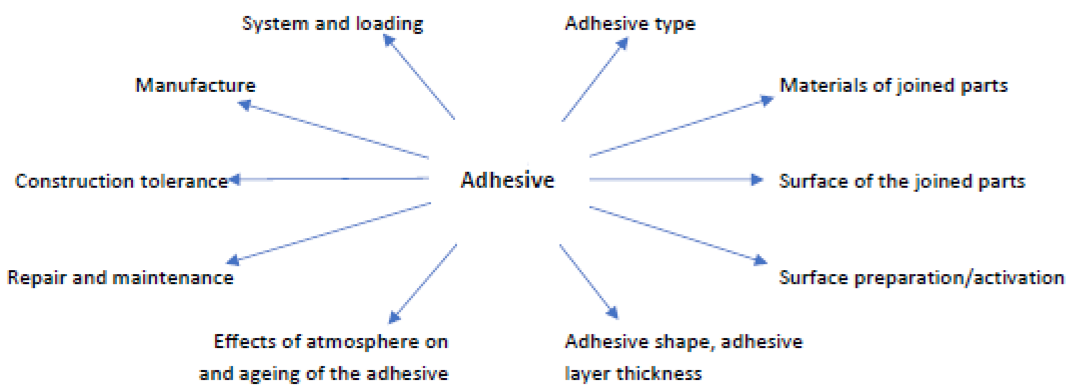


Figure 14: factor influencing the strength of an adhesive

Various adhesives are available for different purposes, tested, and applied in the automobile, aircraft, and industry. Considering the factors mentioned above (Figure 14) will guide the designer to an appropriate adhesive type to ensure sufficient service lifetime of the connection for the field of structural glass. The adhesive types are generally subdivided into categories based on their moduli of elasticity, shear capacity, though elastic, and brittleness. Most types of adhesives are suitable for this.

Structural glass is silicones (modified silicones), polyurethanes, epoxy resins, and acrylates. Silicon and polyurethanes are suitable for flexible systems with up to 150 % elongation, and epoxy resins and acrylates as rigid adhesives.

(Huveners, 2009) investigated in his research the behavior of adhesively bonded glass panes for bracing steel frames in facades. The steel frame functioned as a supporting element to transfer the glass panels' in-plane vertical and horizontal loading to the prominent load-bearing members. This research uses three standard analysis types: theoretical, numerical, and practical.

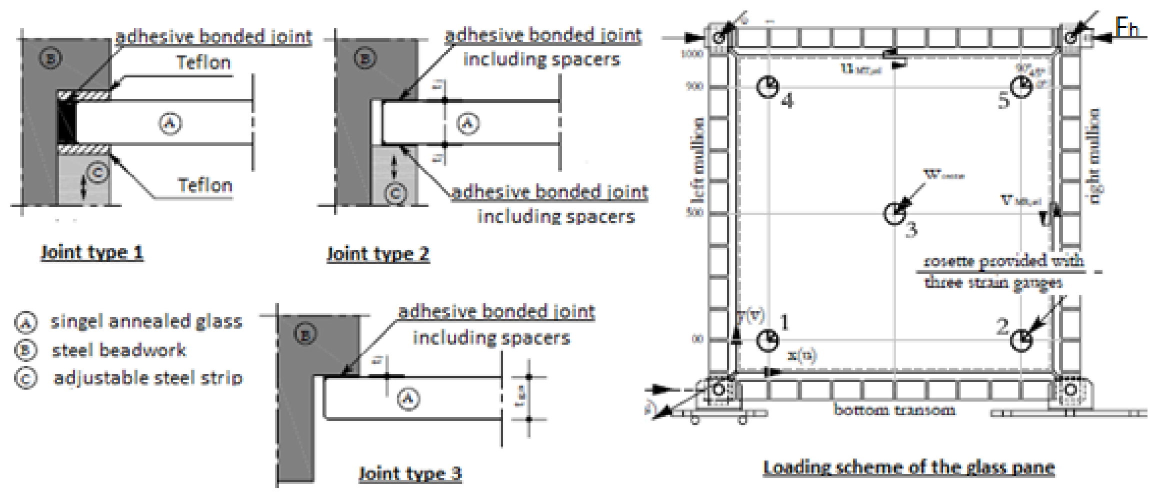


Figure 15:adhesive joint and loading scheme of Huveners

Huveners concluded that the steel frame of joint 1 had a relatively more significant displacement due to the horizontal load, which led to direct contact between the glass pane and the steel structure. The local stresses in the corners were increased beyond the stress resistance of the glass pane, and the first cracks occurred.

The glass pane of joint two was glued two-sided to the steel frame to transfer the load. This adhesive-bonded joint prevented large displacement, and the load distribution was better regulated. Therefore, the failure load was significantly increased in this rigid connection. The connection of the glass pane with the steel frame of joint 3 was idem glued and had similarities with joint 2. However, due to one side adhesive-bonded joint, the displacement was relatively higher, and the local stresses at the corners caused the failure of the glass pane. Figure 16 shows the three joints' maximum stress and horizontal failure load.

Mehmet Kisa has investigated in his thesis research the validation of structural glass window design for in-plane seismic to strengthen the houses in the province of Groningen against earthquakes. He also discussed the suitability of the glued and mechanical fasteners (Kisa, 2021). Composite (timber-glass) structures are mainly equipped with a glued connection between the timber-glass members and (screw/bolt) fasteners to connect the timber with the surrounding structures.

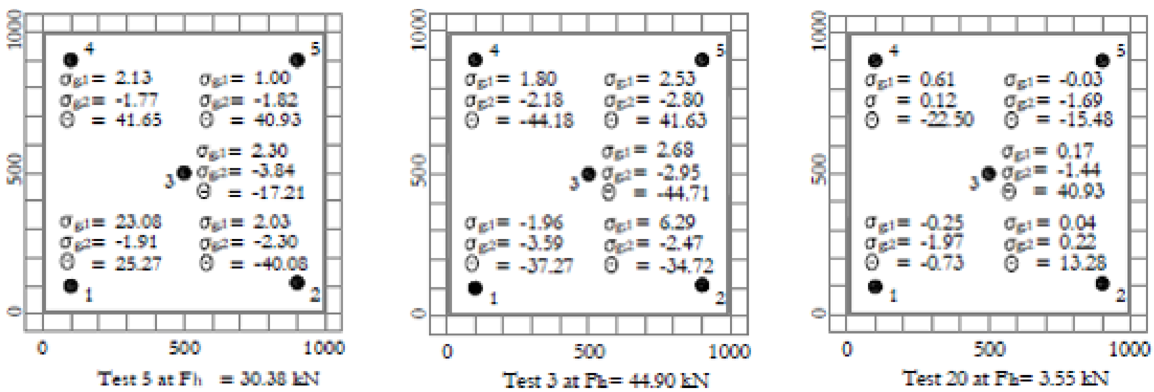


Figure 16: stresses and failure load in joints 1to 3 Huveners

2.1.6 Existing research and models

Glass, as a structural element, is a relatively new material. Due to its elegant appeal, transparency, and high compression strength, the demand for glass designs is increasing. However, the connection and brittleness of the glass require some attention to make glass suitable for various application purposes. Investigation in the field of glass structure is increased in the past two decades to answer the questions and fulfill the demands.

The application of glass material as a structural element is also quite popular and investigated by various scholars and institutes. Transparent glass facades, columns, bridges made of glass, and steel structures supported by glass are studied. The stabilization of houses against seismic load in the province of Groningen is also studied and discussed in several theses and articles. Some brilliant and practical houses are built with more focus on structural glass. One of these projects is the glass penthouse designed by Powerhouse Company architecture. The structural engineers of ABT Engineering, BREED ID, and Glasimpex were responsible for the structural design of the glass dimensions, the connections, and the realization of the project. The project's highlight is summarized by Josine Crone in a paper with sufficient images to illustrate the penthouse on the roof of an existing building in Rotterdam, the Netherlands.

Josine Crone discus in her article (XL-glas draagt penthousedak) the design and technical specification of the 18 m long structural glass façade of a penthouse supporting the roof in Rotterdam (crone, 2020). The article focuses on the design and execution of a sizeable slender glass structure and its connection with the surrounding structural members to transfer the load. The article presents the experience of the involved architect, structural engineer, glass engineer, and contractor. They are also asked to motivate their decision for their applied technical solutions. The essential message of the penthouse project for this research is the application of the load on the glass façade. The structural and glass engineer decided to apply and calculate the load as a point load and not to account for the entire glass length to resist the load. The roof is supported by a steel beam, which is tangible to deflection and will not distribute the load over the entire glass pane equally.

Figure 17 presents an overview of the connection between the steel beam supporting the roof and transferring the load to the glass pane. For the structural calculation, only the interlayer glass panes will be accountable. The load on the glass panes is applied as a concentrated point load through specially designed synthetic HDPE blocks. Additional aluminum load distributors are applied to ensure the load transfer only through the HDPE blocks. This technique is generally applied in the bridging industry.

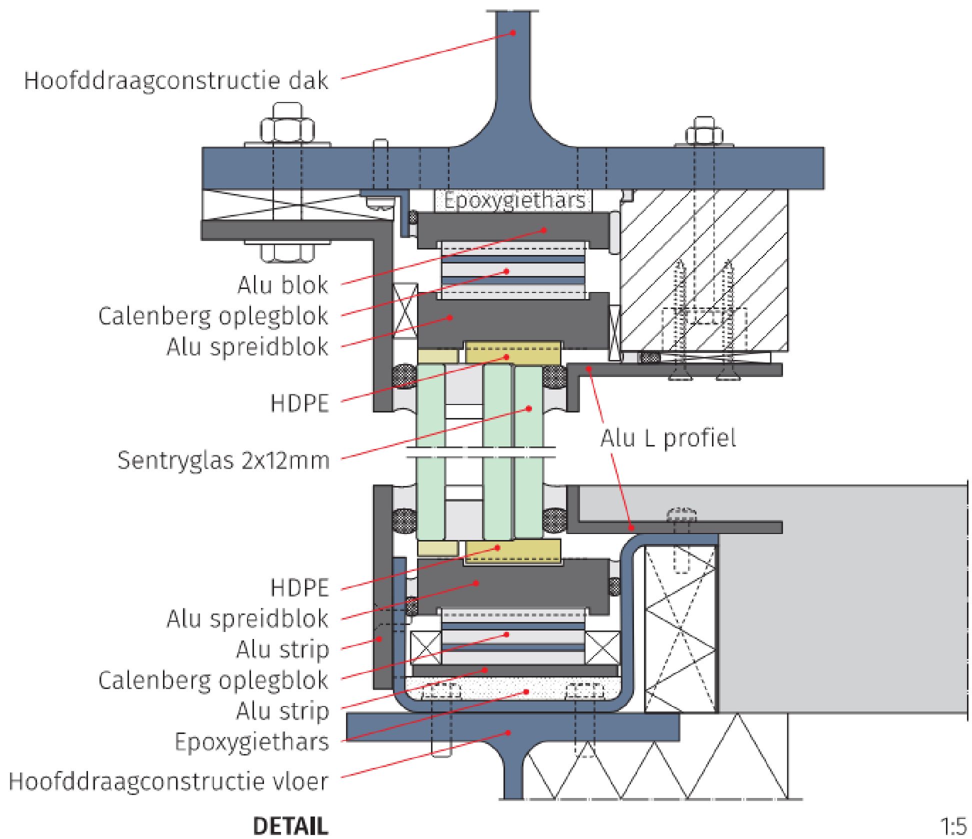


Figure 17: detail design of load transfer to the glass pane (source: XL-glas draagt penthouse dak)

2.1.7 Conclusion

In this chapter, the existing literature and their findings have been mentioned and related to the content of this report. This review aimed to enlarge the theoretical background of structural glass and not waste too much time on existing knowledge by inventing it in this research again.

The invention of glass goes back up to 3500-5000 BC, and the structural application is more from the last decades. Although structural glass is developing and extending its application field, its essential components (silicon, calcium, and sodium) have remained the same. The variety and application field of glass material is extensive and discussed in this report. The appropriate glass types for the construction industry are annealed, heat-strengthened, and tempered glass. The resistance capacity of annealed glass can improve by undergoing a heat treatment, which creates more compact glass and prevent breakage due to impurities. Glass has a relatively large compression strength (1000 N/mm2) compared to its characteristic bending tensile strength (45 N/mm2). Its density is comparable with the reinforcement concrete (25 kN/m3), making it a very suitable material for compression loading. Based on the properties discussed in this report, heat-strengthened glass is the most appropriate type to evaluate for this research. Heat-strengthened glass possesses higher compression strength than regular annealed glass, while the bending tensile strength and crack sensitivity are better than thermally toughened glass. However, lamination and composition of different glass types as one element provide the best result for various applications.

The existing connection technics (bolted, heat bonded, and adhesive) are also studied. Based on this analysis and discussed literature by various scholars, adhesive connections are suitable for realizing the connection between the glass pane and the timber frame. This will accommodate the transfer of the shear stress from the timber frame to the glass pane and vice versa. Transfer of large magnitude loads through-bolted connection also introduces high concentrated stresses around the connection. The phrasing of the hole for the bolts at the edge of the glass pane requires extra precaution. Glass is more vulnerable at the edges than the rest of the surface. On the other hand, screw and bolt connections are suitable for transferring the combined loads from the timber frame to the surrounding structures. Detailing and load transfer at the connection are also discussed under case study 3 in chapter 5.

2.2 Existing stability systems

The main objective of this thesis is to investigate the stabilization of the buildings by a timber-glass façade. Application of the timber glass façade will not be limited only to new buildings but will also provide solutions to the expansion of the existing buildings. Before starting modeling and discussing the possibilities of stabilizing the buildings with a timber-glass façade, the existing stability techniques must be underlined. Therefore, a brief explanation of the existing stability systems and their main application area is discussed in this chapter. At the same time, a first attempt is made to strengthen the existing techniques with a timber-glass façade.

The theme of this chapter is presented in the chart below. This chart emphasizes the content of the subparagraphs.

stability	• Stability of structures is an important aspect which has to be identified during the design stage. There are various techniques to achieve this.
wind load	• Vertical structures must be able to resist besides vertical loads, the horizontal wind load. The magnitude of the wind load is related a.o. to the dimension of the structure discussed in this report.
wind area	• Wind area is another important aspect in determining the magnitude of the wind load on the structure. The surface of the Netherlands is subdivided into 3 wind areas presented in this paragraph.
bending	• To resist the wind load and prevent torsion, several stability elements are required in the X- and Y-directions. The wind load division per stability element is related to its bending stiffness, which is derived from its dimension and material properties.
concrete	• Concrete elements are suitable to ensure the stability of utility buildings. Reinforced concrete is an outstanding composite material, which can resist compression as well as tension stresses.
masonry	• Masonry walls are also very suitable stability elements for small to midsize buildings. Stabilization with masonry walls is financially interesting, but the number of floor levels should not exceed (currently) 11 to 13.
steel	• Steel diagonal braces are very suitable techniques for industrial buildings. These diagonals are mainly applied behind closed (curtain) walls. Steel diagonals enforce cooperation between steel columns and beams.

2.2.1 Wind load

Before diving into the calculation of the stability of the buildings by timber-glass members, it is crucial to underline first the variety of the loads and their mutual relations working on the buildings.

Vertical structures are loaded besides upstanding forces also by horizontal wind loads. The magnitude of the horizontal loads is related to the location of the building and the height of the structure. The magnitude of the wind load differs per country and region. This research focuses on buildings in the Netherlands. However, the general method is valid for other areas with different wind load quantities.

2.2.2 Wind load in the Netherlands

The surface area of the Netherlands related to the wind load is subdivided into three main parts, shown in Figure 18. The wind pressure is the highest in region I, followed by regions II and III.

Figure 18: wind load area

Besides the wind region category, the built-up area is also an important issue to consider. The wind load in the coastal areas is relatively higher than inland. One also has to distinguish between the wind load of a built-up area and a not-built-up area. The wind load pressure in non-built-up areas is higher on buildings compared to built-up areas. The magnitude of the wind load per area related to the structure's height is discussed in Eurocode (Eurocode-1, 2021) and mapped as an appendix of this report.

The general formula and calculation of wind compression, suction, and friction are also discussed and presented in Eurocode-1 (NEN-EN 1991-1-4 up to 7). The magnitude of the wind friction is relatively small compared with wind compression and suction. This is especially the case with smooth facades (like curtain walls). Therefore, wind friction will be neglected. The general formula to determine the wind load per square meter is as follows;

$$q_w = C_s C_d \left(C_{pe} + C_{p\,i}\right) q_p(z) \tag{2}$$

$$F_w = C_s C_d \, C_f \, q_p(z) \, A_{ref} \tag{3}$$

$$F_w = C_s C_d \cdot \sum_{elements} \left(C_f\right) \cdot q_p(z) \cdot A_{ref}$$

Where:

q_w	total wind load on the structure per m2
$C_s C_d$	factor to account the wind turbulation
$C_{pe} + C_{pi}$	factor to account wind compression and suction
$q_p(z)$	wind load related per area and height of the structure
F_w	wind as point load on the structure
C_f	factor to account wind friction

The importance and magnitude of the wind load on the structure have been mainly highlighted so far. Before investigating the possibilities of stabilizing buildings with a timber-glass façade, it is also essential to underline how the existing buildings are stabilized against horizontal wind loads and the possibilities to strengthen these techniques with timber-glass facades. Nevertheless, first, the theory of bending stiffness will be highlighted.

2.2.3 Bending stiffness of stability elements

The bending stiffness of the stability elements is an important issue. The bending stiffness of the stability elements is related to their young's modulus and the moment of inertia. Where young's modulus is related to the material property, it is the moment of inertia related to the dimension of the stability element. The timber-glass façade is a hybrid system with two very different materials. Given that the young's modulus of the glass material is higher than any wood species and the glass member's surface is larger than the timber frame, the glass component becomes the decisive part of the stability element. An overview of a random building is presented below to underline the stability element's displacement and bending stiffness.

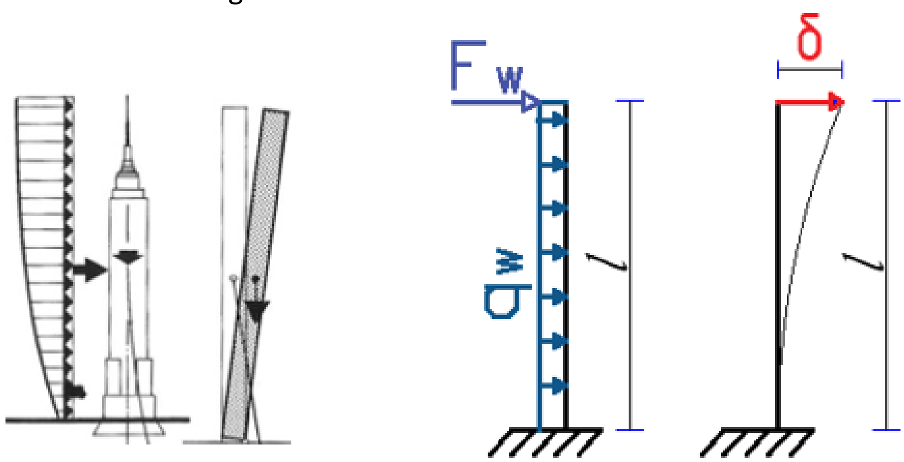


Figure 19: displacement of a random building due to wind load

$$\delta_{tot} = \frac{q_w\, l^4}{8\, EI_{eff}} + \frac{F_w\, l^2}{2\, EI_{eff}} \tag{4}$$

Where :

δ_{tot} = total displacement due to wind load

F_w = friction load due to wind at the roof surface

q_w = wind load enforcing compression, suction and friction on facade

$$EI_{eff} = E_{timber}\left(\frac{1}{12}b_t h_t^3 + b_t * h_t * a_t^2\right) + E_{glass}\left(\frac{1}{12}b_g h_g^3 + b_g * h_g * a_g^2\right)$$

The above-provided equation enhances the importance of the dimensions of the stability elements, which emphasizes that single columns are not very suitable as a stability element.

2.2.4 Timber-glass façade and existing stability systems

The criteria for enforcing a vertical structure against horizontal wind load depends on its construction material. The first attempt of the structural engineer is to evaluate whether the vertical load-bearing members fulfill the strength and displacement requirements to function as stability elements without disproportionally enlarging the cross-section of the element. Using existing load-bearing elements (walls, portals, arches, etc.) will save additional costs and ease the building division, preventing interrupting the architectural view (think of massive concrete walls or steel bracing in curtain walls). However, there are also some requirements (like bending stiffness Etc) that stability elements must fulfill, which are discussed in the following paragraphs, but first, a list of the most commonly applied stability systems to combine with timber-glass is presented here below; Strengthening of the concrete structures with timber-glass façade

- Strengthening of the masonry work with timber-glass façade
- Strengthening of the steel structures with timber-glass façade
- Strengthening of the wooden structures with timber-glass façade

2.2.5 Stability of concrete structures

Concrete elements like columns, beams, slabs, and walls loaded on compression and bending are equipped with steel reinforcement. Concrete structures can resist compression stresses but fail under relatively low-tension stresses. To compensate for this, steel reinforcement is designed to prevent large cracks in areas with higher tension than concrete can resist (Munkelt, 2018). This hybrid material is very suitable to be used as a stability element. Generally, these load-bearing elements stabilize buildings constructed with concrete walls. Concrete columns are less suitable for stability due to their low bending stiffness and dimensions. Therefore, curtainwalls supported by concrete columns and facades with enormous openings for glass windows should be addressed as a stability element. The contribution of the glass members is not counted in these structures. The following concrete members are generally considered stability elements;

- Elevator shaft
- Staircase
- Separation walls

Traditional elevator shafts consist of three closed walls transferring the load from the top to the foundation are favorable stability elements. These massive structures and the dimensions available on each floor make them suitable for resisting vertical loads. Also, the horizontal loads. The elevator shaft possesses sufficient bending stiffness without interruption of openings. The staircase is quite like the elevator shaft and suitable for stability.

Separation walls are suitable stability elements. However, the challenge starts when the developer desires to construct a multifunctional and flexible office building that could be adjusted, and the walls could be shifted to the functional requirements of the future user. This is only possible with separation walls and not load-bearing walls. These buildings could be realized with concrete columns and beams. Different measurements have to be taken into account for the stability of these buildings, and the timber-glass façade is a very suitable one compared to most other (not environmentally friendly and expansive) options.

It is impossible to emphasize the magnificence and contribution of the glass panel for each situation separately, but a general overview of the situation where timber-glass windows could have an essential contribution as a stability member for concrete structures is listed here;

- Large buildings stabilized by a single lift shaft or limited concentrated stability elements could undergo torsional stresses. A Timber-glass façade could solve this problem as a supporting stability element located at the edge of the floor.

- Stabilizing high-rise buildings with limited stability elements will cause massive tension in the foundation under these stability elements. Activation of the curtain walls or timber-glass façade will distribute the tension over the foundations and prevent failure or additional expansive geotechnical measurements.

- Combining the concrete façade column-beam with a timber-glass facade will increase the bending stiffness and reduce horizontal displacement.

- It is also quite challenging to deal with tall buildings (above five levels), with different layouts of the floors where the walls are not on top of each other. In these situations, activating the outer walls as a stability element with large openings for the glass windows is imperative.

2.2.6 Stabilization with masonry brick walls

Construction of (reinforced) masonry is gaining popularity and is relatively cost-effective and sustainable compared to most other construction materials. However, this technique also has its limitations. Failure due to a large horizontal load is one of them. The height of the masonry outer walls (non-load bearing) is limited to 3 to 4 levels, equivalent to 10-15 meters. Above these heights, the façade has to be supported by a façade carrier (KNB, 2020). The load-bearing walls, which are also responsible for the stability of the building, are generally thicker (minimum 120 mm). These walls are supposed to be on top of each other, and the tension must be limited. The maximum height of buildings with masonry brick walls is constantly improving, and currently, it is possible to construct buildings with masonry brick walls up to 13-17 floors. However, many walls are required to sustain tall buildings' stability. These walls resist the horizontal loads to minimize the tension stresses. Figure 20 demonstrates an overview of a random building stabilized by masonry brick walls.

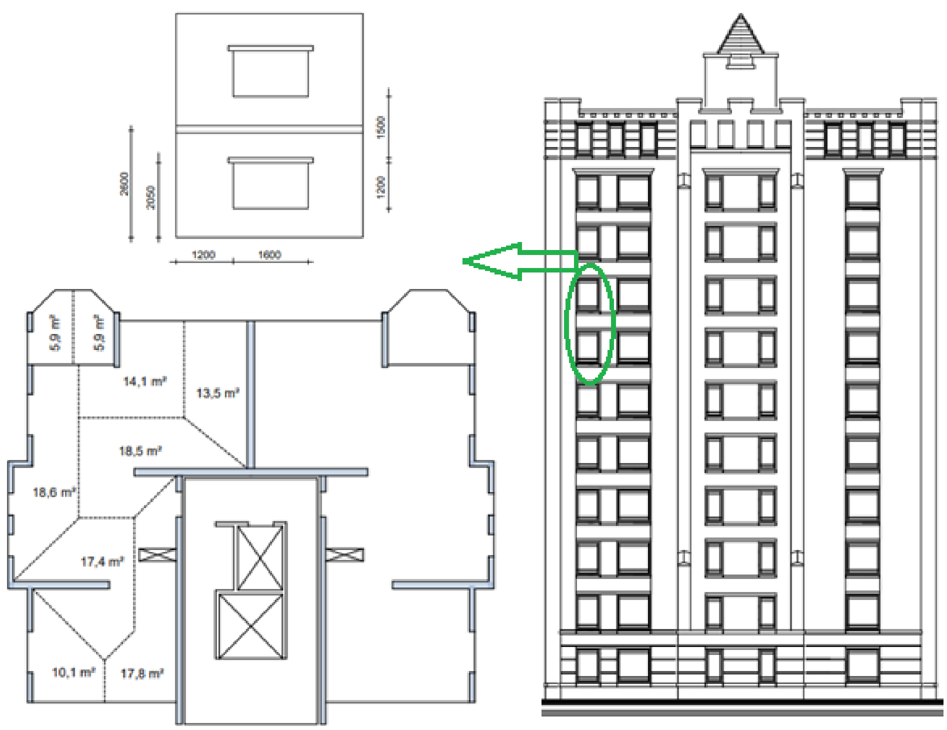


Figure 20: masonry stability brick walls of a building

The façade walls are also needed to stabilize the building; therefore, the window openings must be limited. By applying timber-window as a stability element, the architect will be free to enlarge the openings and play with the windows' location for a dynamic look of the façade. The timber-glass window and the brick walls will create a hybrid stability element with much higher bending stiffness.

2.2.7 Stabilization with steel diagonal bracing

Buildings constructed with steel elements are designed with steel columns and beams. These steel elements are slender with low bending stiffness. By rigidly connecting the beams with the columns, the structure (portals and arches) could resist a certain amount of wind load parallel to the steel structure (John Wiley, 2008). Steel diagonals could stabilize the counter direction, which is not always desired from an architectural and aesthetic perspective to apply steel diagonals in the façade.

These diagonals could block the view and affect the design. However, there are also steel/concrete diagonals, stabilizing and making the building remarkable. It is also possible to use the concrete core and walls as stability elements if available. Steel diagonals are very popular and primarily applied in silos and warehouse buildings.

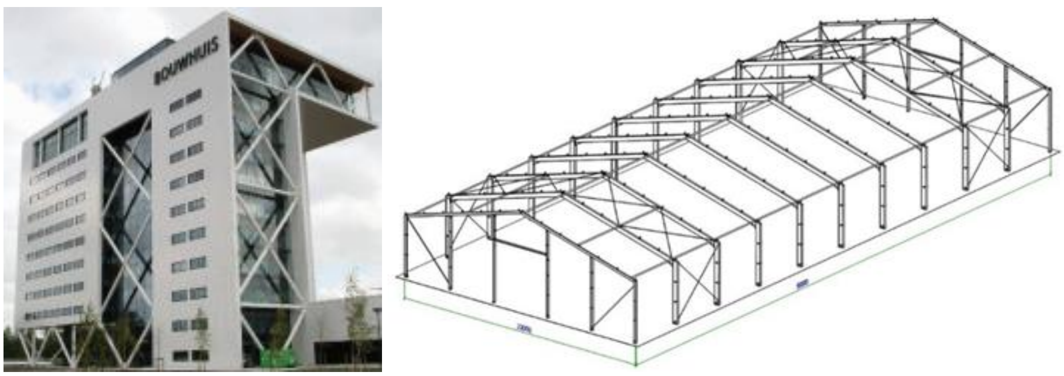


Figure 21: high-rise building and silo stabilized by steel diagonals

From a load-bearing and division perspective, steel diagonals require quite expansive connections. The cost and attention required for the joints are linearly related to the height of the buildings. The higher the building, the larger the concentrated stresses on the connections of the diagonals. High-rise buildings stabilized by single steel diagonals are also tender for torsion. Therefore, accurate design and additional measurements are required to prevent damage due to horizontal loads.

Timber-glass façade is an appropriate solution to stabilize buildings designed with steel columns and beams. The timber-glass façade will stabilize the building and prevent any disadvantages affecting the architectural and aesthetical issues. The timber-glass façade could be loaded out of the plane and function as a shear wall. It is possible to apply the timber-glass façade in the counter direction of the steel portals and arches and in the same direction to strengthen the steel members. This will increase the total bending stiffness of the applied stability members.

2.2.8 Conclusion

The timber-glass façade is introduced in this chapter, and the importance of the bending stiffness to resist horizontal forces is underlined. Although glass material was discovered thousands of years BC, its application in the building industry started relatively late (R. Nijsse, 2019). Glass is primarily used in the building industry to allow daylight entrance and later as moving parts (windows) to enable air ventilation. Glass as construction material and load bearing member has improved in the last decades. Its application field is increasing. Glass is a very ductile material with quite a significant compression strength (even higher than concrete). Using the compression strength properly, Glass could be usefully applied as a load-bearing element (Kozlowski, 2019).

The primary existing construction materials and how to stabilize buildings made of these materials (concrete, steel, masonry, and brick walls) are highlighted. The attempt was to investigate the possibilities of strengthening this existing construction methodology with a timber-glass façade. The timber-glass façade can function as a shear wall loaded out of the plane. These façade elements are suitable to allow more light entrance, to improve the architectural design, but could also function as stability members. Openings for glass facades are assumed to be weaknesses of the bearing structure. This research investigates whether it is possible to apply timber glass to strengthen the structures.

2.3 Application field of timber-glass façade

Glass is an important construction material and is available in almost every building. However, glass has so far not or very limited load-bearing function. To improve this and to use the available glass in new designs, its suitability and benefits are studied in this chapter. The goal is not to use glass facades only as load bearing but also to stabilize the buildings by timber-glass façade.

This chapter also studies the activation and application of the timber-glass façade in a hybrid system. The objective is to investigate whether glass can participate as a load-bearing and stability element with existing techniques. The creation of a hybrid system will be investigated in broad content and must be applied to existing and new buildings.

Last but not least important is applying the timber-glass façade in expansion buildings. The number of expansions in the existing buildings in the Netherlands is increasing, with large openings for the glass facades to accommodate light entrances, create transparency and modern architecture, reduce dead weight on the foundations, and create a better view. However, these glass panes are not meant yet to stabilize the buildings, which gives sufficient reason to be investigated here. The content of this chapter is pointwise presented in the chart below.

new building
- Suitability and importance of the timber-glass façade in new buildings is discussed in this paragraph. Glass facade in new design could function as stability element.

expansion building
- Shortage of housing in the NL increased the number of expansions. Timber-glass facades are suitable stability element to stabilize the existing and expansion part.

hybrid system
- Timber-glass façade is also very suitable to be applied as stability element in hybrid system. The importance, financial benefits and functionality is discussed in this paragraph.

conclusion
- final discussion and assesment to determine the boundary conditions of timber-glass facade as stability element in different type of building is presented under conclusion.

2.3.1 Stability of new buildings

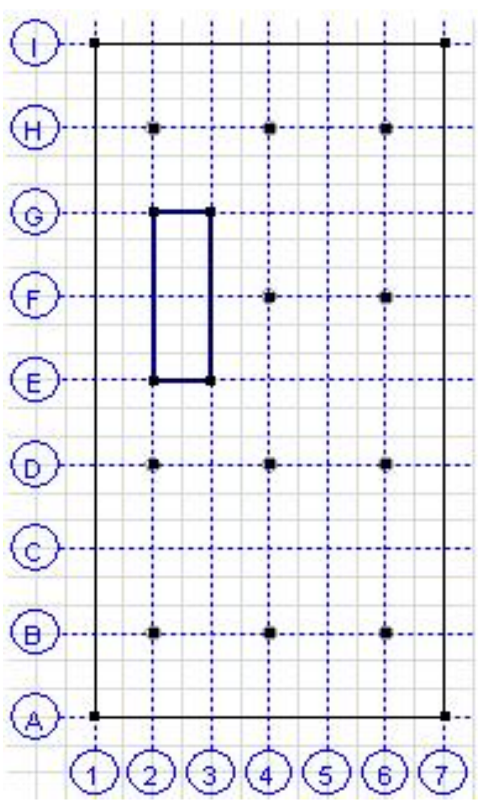
The concept of stability is a well-known phenomenon within building engineering, which is already discussed in this report (chapter 2.2). To prevent undesired cracks and askew of buildings, proper measurements are required. Based on the function of the building and the applied construction material, the Eurocode prescribes the maximum allowed deflection due to horizontal wind loads (CUR-Aanbeveling, 2021). Total deflection determines the number and dimension of the stability elements to secure the requirement.

Stability elements are crucial safety measurements and are included in a very early design stage. The structural engineer enforces the building with sufficient stability elements to stabilize the vertical structure. Although every building has its authentic character, function, and dimension, the applied stability technique and material are generally identical. Most skyscrapers are stabilized by concrete, steel, or a combination. These materials have been applied over the last centuries and proved themselves. Therefore, it is also convenient for structural engineers to stabilize new buildings with the same technique and material.

Figure 22: stability of high-rise buildings in concrete, steel, and composite (concrete and steel)

Figure 22 presents three high-rise buildings realized with different stability materials and techniques. The Trump International Hotel and Tower in Chicago (1e left) are stabilized with a concrete core. The middle building is the Six Green Start 8 (Sydney, Australia), stabilized by steel diagonals. The right one is the tallest building of this moment (Burj Khalifa), which is stabilized by a combination of concrete and steel elements. The common thing about these high-rise buildings is the availability of sufficient glass panes. These glass facades are currently not considered stability elements. By stabilizing such a structure only by lift shaft, staircase, or a limited number of concrete walls, the entire horizontal wind load will be transferred through these stability elements to the foundation piles. This will lead to highly concentrated vertical load and possible tension in these foundation piles, which has to be compensated with additional measurements.

Figure 23 presents the location of a random high-rise building stabilized only by a lift shaft located eccentrically at the center of gravity of the building. This is generally the case when facades are designed with transparent glass panes, and walls are not desired at the outline of the building. In these specific cases, torsional stresses can occur due to horizontal wind loads.



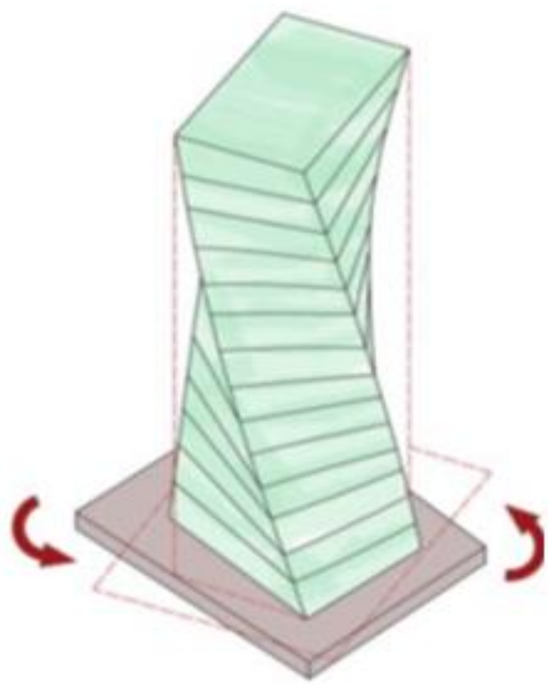


Figure 23:possible torsion due to wind load of eccentrically located stability walls

The main message of this paragraph is clear now. Substantial glass panes are available in the design of almost all high-rise buildings, but they are not used as load-bearing and stability elements. Instead, several expansive measures are taken to strengthen the building against torsional stresses, the tension in the foundation piles, and other combined loading to stabilize the structures. Therefore, it is crucial to investigate the suitability of timber-glass facades and their contribution to the stabilization of the building to prevent torsion and better distribution of the combined load over the foundations.

The timber-glass façade is very suitable as the main stability element in buildings with curtain walls. Curtain walls are generally self-supporting systems that use additional structural supporting elements and needles. Timber glass façade can resist vertical and horizontal wind loads (Figure 24).

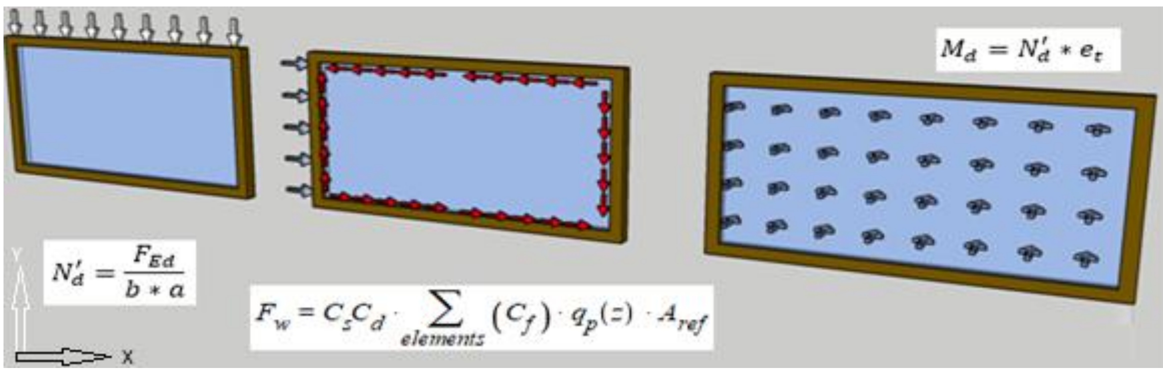


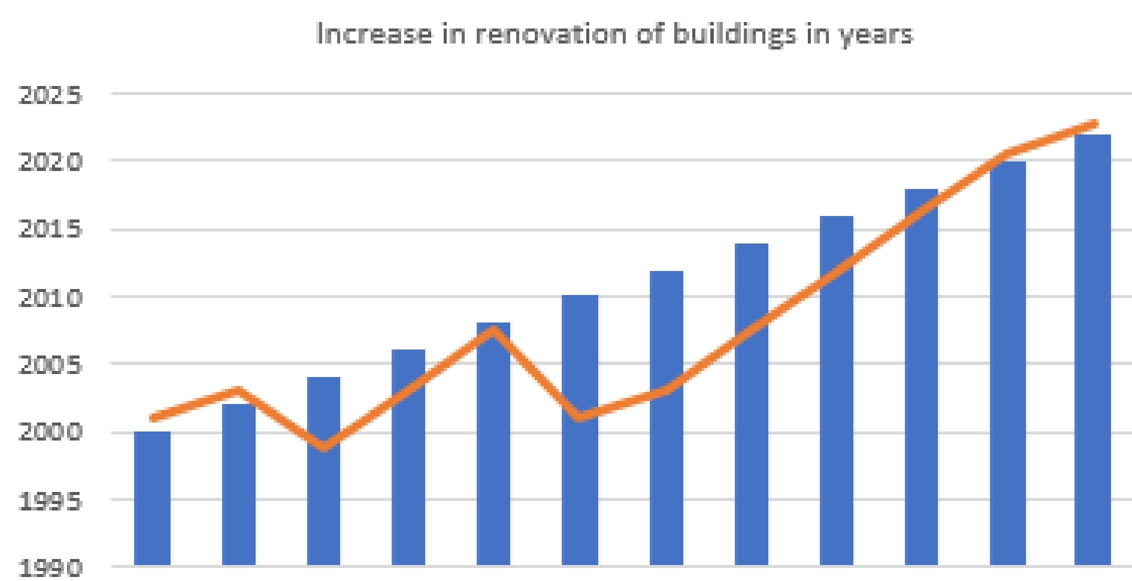
Figure 24: normal forces in combination with wind load in X- and Y-direction

Besides building stabilization, the timber-glass façade can distribute the vertical load over the foundation piles, provide resistance against torsional bending, and make additional measurements redundant to decrease costs.

Chapter 5, under case study 3, is an actual project presented and investigated to study the appropriateness of the timber-glass façade in new projects. Manual calculation and finite element models are applied to understand and demonstrate the load distribution over the glass façade and load transferring to the foundation piles.

2.3.2 Stability of buildings at expansion

Renovation, transformation, and expansion of existing buildings are becoming more crucial with the growing demand for leaving space. Renovation of existing buildings helps reduce the energy consumption for heating and cooling (ancient) buildings, which need to be adequately insulated. By renovating them, insulation will reduce energy consumption, and the extension of their life cycle will also save new construction materials and energy. An essential issue for building engineers is also to reduce CO2 emissions and pollution in the construction industry. The renovation will not only improve the architectural look of the facades but will also increase the comfort of the inner quality of the buildings. Consider for example light entrance, additional square meters, a better view of the garden or street, reduced consumption, and so on.



Graph 2: increase in renovation and isolation of buildings in NL

The transformation of empty offices and other buildings is also becoming popular to accommodate the increasing demand for extra living space in the Netherlands. Expansion is generally accompanied by additional openings for more daylight entrance to increase comfort and quality. The Dutch building regulation also requires a minimum daylight entrance for new buildings and buildings at expansion (Bouwbesluit, 2012).

The application and importance of the timber glass façade for new buildings are discussed in the previous paragraph. This chapter aims to evaluate whether timber-glass façade is suitable to be applied as an independent stability element to stabilize buildings at expansion due to changes in their existing stability system.

Structural glass could be applied and combined very well in the expansion and renovation of the buildings. The building industry in the Netherlands has been lacking in the past decades, especially after the financial crisis of 2008-2009, also known as "the housing market bubble" or The Great Rescission. Construction and development of new houses were minimal for about five years, and after 2014 the construction industry recovered very slowly and cautiously. Therefore, a considerable housing shortage is created in the Netherlands, several EU countries, and beyond.

Expansion of the existing buildings is currently considered one of the main objectives to create more living space. However, to expand a building, various structural measures are required to resist the combined forces and to guarantee the stability of the building. Due to technical development and better performance of isolated double and triple glass windows and frontage, the application of a broad glass façade is becoming increasingly popular. However, in most of these cases, the function of the glass façade is limited up to separation and is not used as a structural member and particularly not as a stability element. In a record number of expansion projects, steel portals or arches are applied to stabilize the building.

Figure 25 presents an overview of a building at expansion. The expansion adds an adequate amount of surface to the existing buildings, but it is also essential to accommodate sufficient light entrance in the existing part and the expanded area.

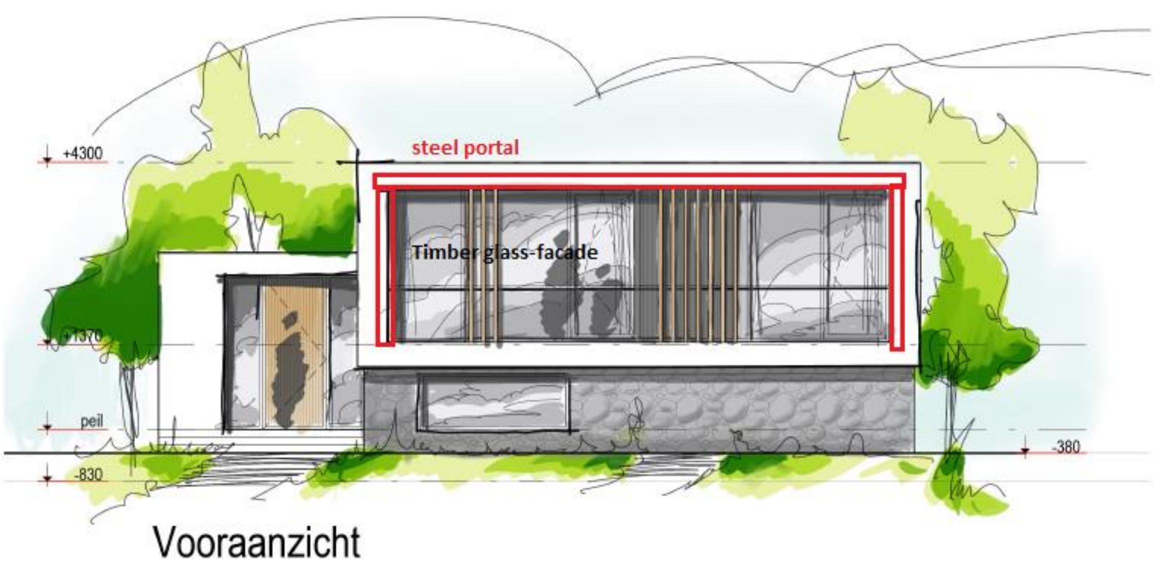


Figure 25: expansion building with steel portal and glass façade

Therefore, the façade of the expansion part is designed with many glass panes. However, as mentioned before, opening for glass facades and curtain walls is currently assumed to be the debilitation of the structure, and glass members are not participating as load-bearing members. Therefore, additional steel portals are thought to be applied to enforce the building against horizontal wind load.

Figure 26 demonstrates three different types of steel portals. The most left one is meant to transfer the vertical load around the opening realized for the glass window/façade to the foundation and is not participating in stabilizing the building. The middle one is loaded by horizontal wind load and combined vertical dead and live load. This portal has, therefore, a double function and will transfer the load through the steel columns very locally to the foundation (piles). The most right one, also known as the ring portal, is suitable to accommodate an opening in the face and distribute the combined loads over the foundation piles located under the portal. However, due to the steel elements' slenderness and relatively low bending stiffness, huge cross-sections are required to fulfill the admissible horizontal displacements.

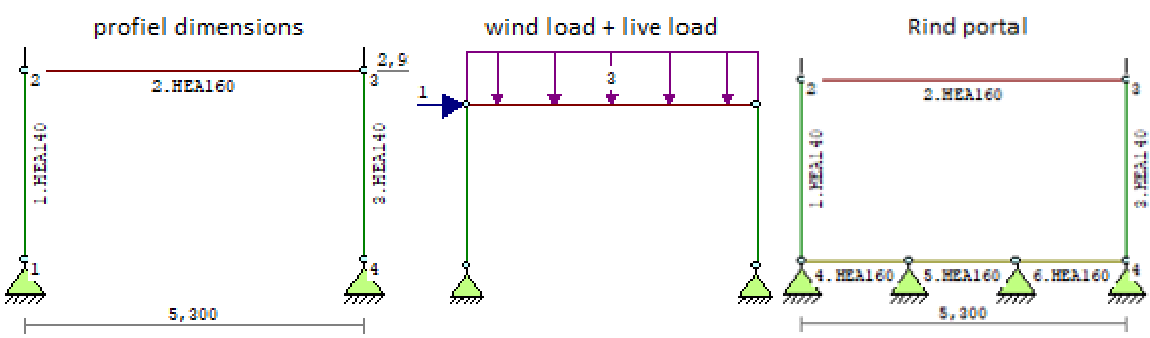


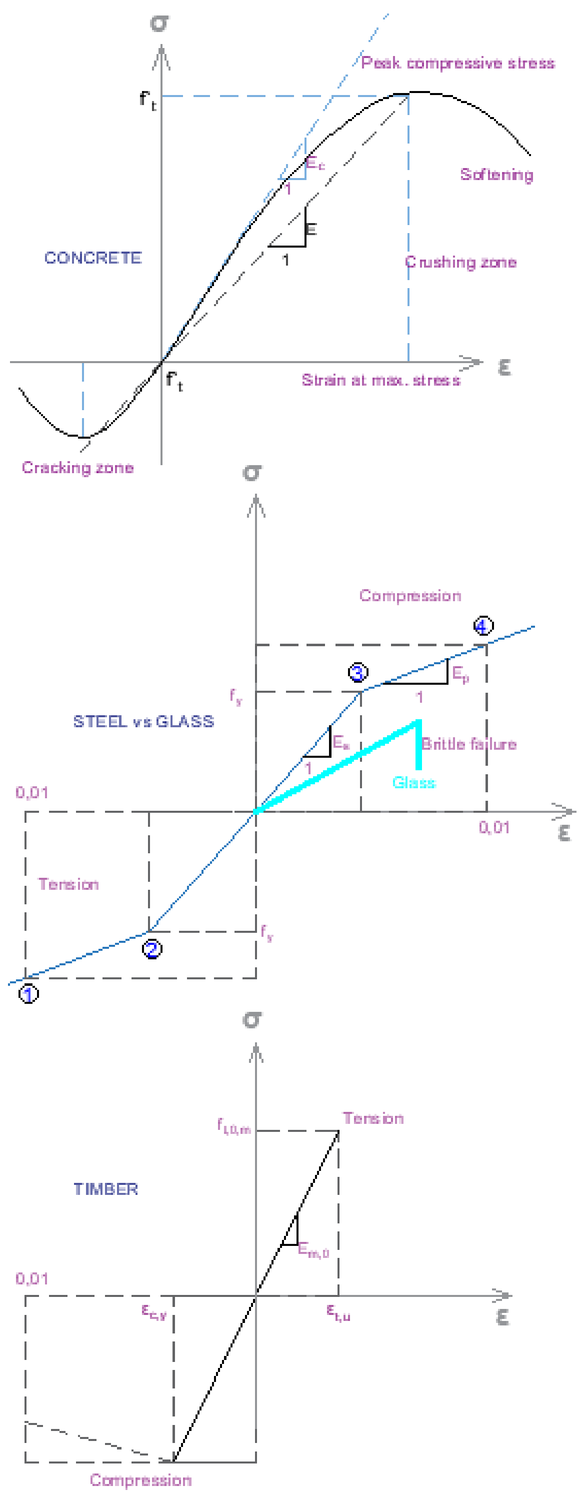
Figure 26: steel portals to stabilize at expansion

By designing a timber-glass façade as a load-bearing and stability element with sufficient glass thickness, an additional steel portal around the glass façade will be redundant. The timber-glass façade will stabilize the building and distribute the combined loads proportionally over the foundation piles under the glass façade. For further evaluation of the timber-glass façade as a stability element, an ordinary project is investigated in chapter 3 under case study 3. The aim is also to identify the magnitude of the glass panes' horizontal load and load resistance capacity.

2.3.3 Hybrid systems

The correlation between the stress resistance and the dimensions of contrary construction materials is very different. For example, concrete is well-known for its compression resistance behavior and relatively low tensile strength, while steel can resist compression and tension stresses. Timber structures are environ-mental friendly organic materials, which are tangible to deflection while loaded on bending. The list of construction materials with their physical performance is extensive, but the point is clarified now. Using different construction materials to resist the combined loads with different properties is not an invention. Quite the opposite, steel rebars are applied to reinforce the concrete and resist bending with tension.

Combining various construction materials as a hybrid system could improve the material's performance. The physical properties of glass are exuberantly discussed in chapter 2.1 of the literature studies. Glass material is comparable to concrete, having high compression strength and relatively low tensile resistance. Applying load on a glass structure also requires attention not to exceed the local compression and to bend tensile stress resistance. Therefore, a timber frame is required to distribute the load properly on a glass pane and minimize local peek stress.



Timber glass-façade could be applied as a hybrid system with existing construction materials and technics.

Timber glass-façade could be applied as a hybrid system with existing construction materials and technics. Due to the fragility and vulnerability of glass material, it is crucial to carefully investigate the suitability of timber-glass façade in hybrid systems. Glass is not as elastic as steel or timber structures. By applying a glass façade in a hybrid system, one should realize that Glass can break suddenly once the elastic capacity is exceeded, which is against the perception of what is known and expected from steel, concrete, timber structure, synthetic material, and others.

Figure 27 presents three situations where a glass pane is available in the structure. However, a timber-glass façade cannot be applied in all these situations as a hybrid system. The first situation presents a timber-glass façade on top of a steel portal. These two stability elements can operate independently and are at different levels. The load transfer is mainly accommodated through the concrete slab. In the second situation, the timber-glass façade is integrated with the steel portal, creating an excellent hybrid system. By integrating a timber-glass façade, the cross-section of the steel elements could be significantly reduced. The third position is risky and can cause sudden failure of the glass pane. The timber-glass façade is applied in a serial system with the steel portal in this scenario. This means that the timber-glass façade should undergo a sufficient magnitude of horizontal displacement to activate the steel portal. As discussed previously, steel is relatively elastic and will deform plastically before failure. Therefore, the serial application of timber-glass façade to create a hybrid system is risky.

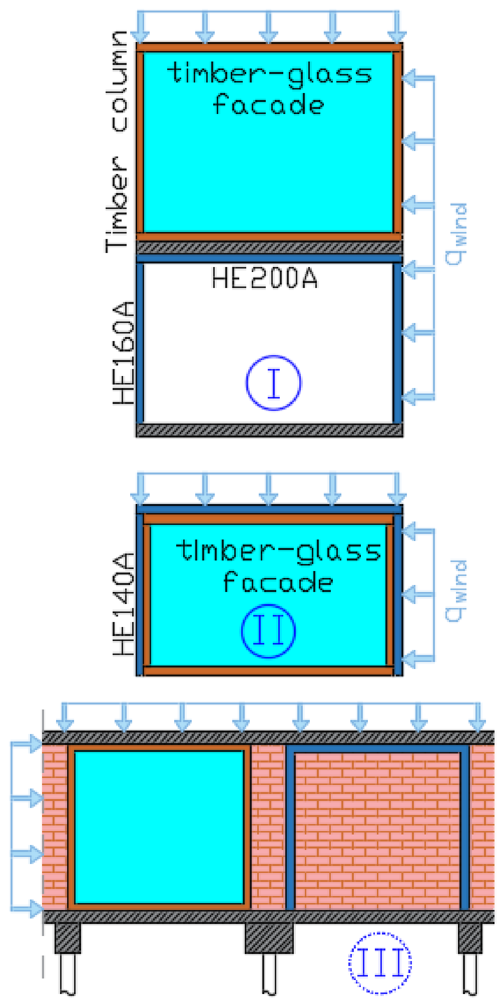


Figure 27: application of glass in a hybrid system

Based on the above discussion, timber-glass facades are suitable to be applied in new designs and existing buildings as long as the mentioned characteristic of Glass is considered. Further investigation concerning timber-glass façade in the hybrid system is executed in chapter 4 under case study 2.

2.3.4 Selection of case studies

The relevant literature study for this research has come to an end. The next step is to select three appropriate real projects to apply the discussed theory and investigate the possibility of stabilization of the buildings by timber-glass façade. However, the selected projects must fulfill some essential requirements. These requirements differ per case study and the target to be achieved. The essential characteristic that the case studies must fulfill is presented here.

Case study 1

The selected project for case study one must enable activation of the timber-glass façade as a stability element. This means there should be sufficient glass façade available in the existing design or at least the possibility and willingness to integrate timber-glass façade in the existing building. The glass must also be applied over the entire height of the level and is connected to the floors. Herewith, the timber-glass façade can strengthen the building against horizontal forces. Activated timber-glass façade cannot be used as an open element (e.a. window, doors, sliding panel, Etc.) which must be encountered while designing.

Case study 2

To investigate the suitability of the timber-glass façade as a hybrid system, a project is required where an existing stability system is also available. However, this is not an issue in existing buildings stabilized by steel, concrete, timber, or masonry brick walls. To increase the magnitude of the forces, a larger building is preferred to study the behavior of the timber-glass façade under combined load. Case study 2 distinguishes itself from the previous case study by investigating the load distribution based on the bending stiffness of the timber-glass element. Besides, serial and parallel system applications and the function of the glass as an independent stability system will be studied.

Case study 3

Glass façades are applied multiply in the design of new high-rise buildings without any structural contributions. In some designs, it is even a challenge to integrate the amount of glass and assure the stability of buildings traditionally and cost-effectively. Therefore, selecting a high-rise building where glass is available is crucial to study the contribution of the timber-glass façade as structural/stability element. The size of the project, the magnitude of the forces, the required type of connection, and the application of engineering tools (FEM) to investigate the appropriateness of the timber-glass façade as a stability element differentiates this project from the previous case studies.

2.3.5 Conclusion

Structural glass is used in various projects as load-bearing members. However, its application field is lacking compared to other construction materials. Glass is assumed vulnerable to cracks and has a more aesthetical and light entrance purpose than load bearing. This chapter aimed to highlight first that glass is available in every building, but its load-bearing function is almost neglectable. Therefore, it is crucial to start using glass panes as load-bearing members, mainly when glass is applied in those specific locations where additional load-bearing members (of concrete or steel) are applied to resist the load. Double load-bearing member increase only the construction cost and emissions, which can be reduced.

This paragraph also discusses the three different situations where timber-glass façade is applicable. Considering Glass's limitation and vulnerability, it is very well possible to stabilize buildings with timber-glass façade. Timber-glass façade could be used to stabilize new buildings and existing ones. Based on this short investigation, where timber-glass façade is evaluated as a structural element, one can conclude that there is sufficient potential to investigate this further with some real projects to evaluate the feasibility of timber-glass façade as stability and load-bearing elements.

Another essential aspect of adopting a timber-glass façade as a load-bearing and stability element is creating better load distribution over the foundation piles. A perfect hybrid system could be realized by applying the timber-glass façade parallel to the lift shaft, staircase, stability wall, and steel portal. This will provide a better structural design.

The literature study is accomplished by concluding that, the load-bearing capacity of glass structure is heavily underestimated, which increases the overall cost of construction, the construction emissions, and decrease efficiency. These aspects are discussed in the following chapters based on some real projects enhanced by manual calculation and finite element models and analysis to stimulate the application of glass as a structural material.

3 CASE STUDY 1: EXPANSION BUILDING

So far, the theoretical background and structural properties of glass materials have been discussed. Existing techniques to stabilize buildings against wind load are also explained. To extend the content of this research, it is crucial to study the application field and suitability of timber-glass façade in real projects. Case study 1 is carefully selected based on the building's availability of large glass panes. Steel portals around the glass façade stabilize the existing structure. This chapter aims to activate the available glass façade as a stability element to make the application of an additional stability element redundant.

Chapter 2.3 presents different scenarios where glass façades are applied without any structural function. In contrast, a timber-glass façade can provide a sufficient contribution to stabilizing the building due to changes in the existing stability system. In this case study, the glass façade will be highlighted from a structural point of view. It will be demonstrated with manual calculation whether a timber-glass façade could be applied as a load-bearing stability element for expansion buildings.

load application	• Due to brittleness of the glass, it's very important to consider a proper way to apply the load on glass. Application of line load and point load and their consequences on glass is studied here.
manual calculation	• The in plane loaded tensile bending capacity of the applied glass façade is manually determined in this paragraph. With this information in hand, the required thickness of the glass façade could be determined.
timber frame	• The importance of the timber frame is briefly discussed in this paragraph. Stress distribution and examination of the timber cross-section is examined manually
connection	• Connections are discussed under literature study. The appropriate connection technique for expansion buildings is briefly discussed here.

3.1 INTRODUCTION OF CASE STUDY 1

This case study is about an existing building with a large glass façade applied in the expanded part (Figure 28). Load-bearing masonry sand-lime bricks stabilize the current structure, which is standard in the Netherlands for residential houses. However, the expansion part is designed with large glass facades without brick walls.

To strengthen the entire building against a horizontal wind load, a steel portal around the glass façade is applied. The glass façade is excluded from participating in stabilizing the structure and the expanded part.

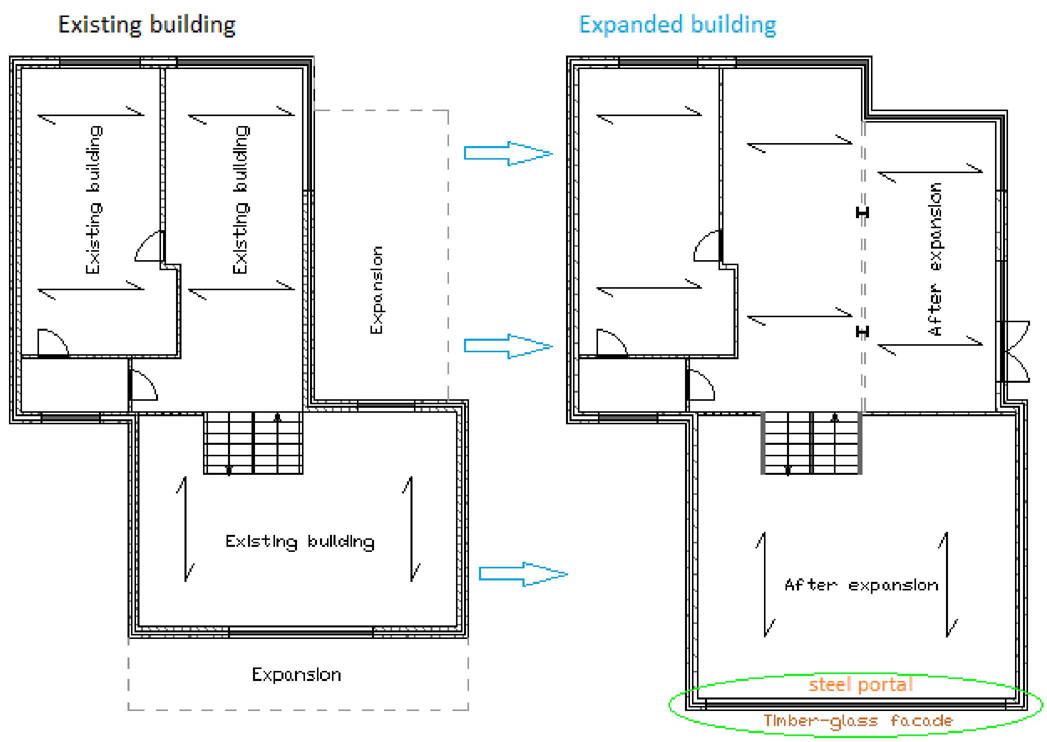


Figure 28:Layout of the expanded building

Glass panes are available in almost every building to accommodate light entrance and ventilation. The size and thickness vary, but they are generally not load-bearing. Therefore, additional research and care are required for different situations to apply the load very carefully on the glass façade. Given that the glass façade is very sensitive to tension with low bending stiffness, local stress under the loads must be examined while designing. This could be achieved by manual calculation or by applying for finite element programs. The manual calculation will be applied for this case study to underline the glass properties. FE analysis will be used for the following case studies to replenish and accomplish this research.

3.2 Application of load

In this paragraph, the load division on the glass façade for the residential building at Platteweg in Reeuwijk will be discussed and followed by manual calculation. The goal is not only to determine the thickness of the glass pane but also to discuss and investigate the load application possibilities on the glass elements. Glass is a vulnerable material and can break when the magnitude of the concentrated load increases unproportionally to the glass component. Therefore, it's crucial to understand whether the applied load on the glass element will work as a line- or concentrated-point load.

Figure 29 demonstrates the size and location of the glass façade. This façade has recently been applied to enlarge the building and to allow more light entrance. However, the used glass has no structural function in the present design. Therefore, the glass façade does not span from the roof to the slab; instead, it's separated by a thin steel plinth.

Figure 29: potential glass facade to be used as the stability element

This chapter aims to investigate, through structural calculation, whether this glass façade could participate in the stabilization of the building and expansion. At the same time, the timber-glass façade will be loaded with the vertical dead and live load of the roof. The following specification according to NEN-EN 1990 NB will be considered while calculating.

Building classification according to Eurocode

Reliability class	RC1
Consequence class	CC1
Design life	50 year
Project location	Platteweg 21, Reeuwijk
Wind area	II, undeveloped
Height of building	4.3 m
The magnitude of the wind load	0.73 kN/m^2

Table 3: building specification for determination of safety factor and wind load

Before determination of the combined loads (dead-live- and wind load) on the timber-glass facade, it's essential to analyze first how the loads will be applied on the timber-glass pane. Figure 30 presents a cross-section of the timber-glass façade with the roof, where both structural elements are visible. The figure demonstrates load transfer from the top through the truss on the timber-glass façade, which is very concentrated.

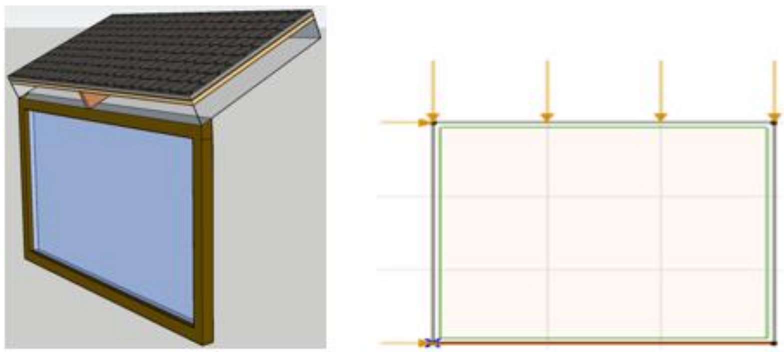


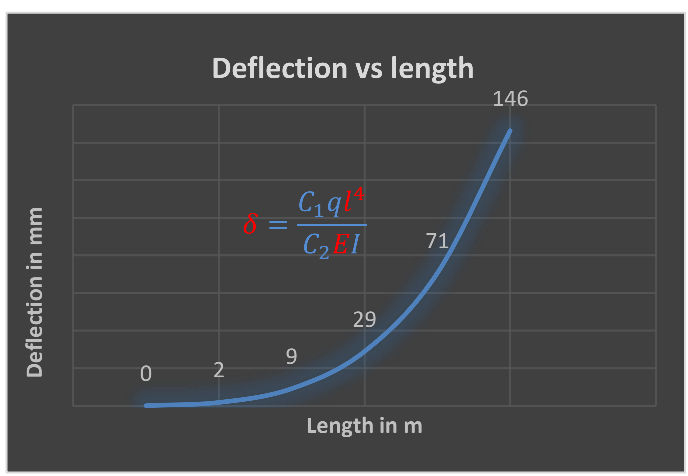
Figure 30: cross-section of the roof with timber-glass façade

The connection of the timber frame with the glass pane is significant to be considered while designing. Especially when timber is not in full contact with the glass pane, concentrated load transfer can be achieved by phrasing compartments inside the timber frame between the contact points of timber and glass pane (demonstrated in chapter 5). Timber structures are very tender to deflection. From structural mechanics, it's known that there is an exponential relationship between the length and deflection of the beams. Timber is even more tangible to deflection due to its lower modulus of elasticity than concrete and steel beams. Therefore, it is essential to design sufficient thickness for the timber frame to prevent undesired contact with the glass pane due to deflection.

$$\delta = \frac{C_1 q l^4}{C_2 EI} \quad of \quad \delta = \frac{C_1 F l^3}{C_2 EI}$$

Where:

δ	deformation (mm)
l	length (m)
$\frac{q}{F}$	loads
C	constant
E	modulus of elasticity
I	moment of inertia



Graph 3: deflection versus length of the beams

3.3 MANUAL CALCULATION

Load application was discussed in chapter 3.2. Although concentrated load on glass panes is generally assumed to be decisive, both options (point load and line load) will be considered while determining the thickness of the required glass pane in this report. The concentrated load will be applied with the same technique and specified detail discussed in the previous chapter.

Based on Table 3, the magnitude of the combined load on glass panes can be determined by calculating the required thickness. Table 4 presents an overview of the applied materials, their dimensions, and the vertical and horizontal loads on the specific timber-glass façade of the expansion building.

Dead & live loads of expansion building (Platteweg 21)						
Roof	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
roofing, underlayment purlins		0,40	-			
pipes, wiring, and ceiling		0,35	-			
use category H		-	1,00	0	0	0
		0,75	1,00			
Walls/facade	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
brickwork	-	2,00	-			
Timber-glass facade	-	0,60	-			

Table 4: the dead and live load of the expansion building

The wind load will be applied as a distributed horizontal line load or concentrated point load at the top corner of the timber-glass façade. The magnitude of the wind load, which is related to the height and location of the structure presented in Table 3, can be determined. The calculation can be executed according to the following formula originating from Eurocode.

$$q_{wind} = C_S C_d * C_{pe} * q_p(z) \rightarrow q_{wind} = 0.85 * (0.8 + 0.5) * 0.73 = 0.81 \frac{kN}{m^2}$$

$C_S C_d$ wind coefficient, related to the height and slenderness ratio of the structure.

C_{pe} wind compression/suction related to surrounding structures and turbulation.

$q_p(z)$ wind load related to the height and location (province/country) of the structure.

3.3.1 Loads

Various aspects affecting loading the timber-glass façade have so far been discussed. By clustering all this information, the manual examination can start. The main goal of this calculation is to answer whether the glass façade could stabilize the building. At the same time, the magnitude of the required glass thickness can be determined.

Figure 31 presents the layout of the case study-1, including wind load in the X-direction and corresponding stability elements (canvas A, B, C). The intended stability element stabilizes the building with a timber-glass façade on the C-axis.

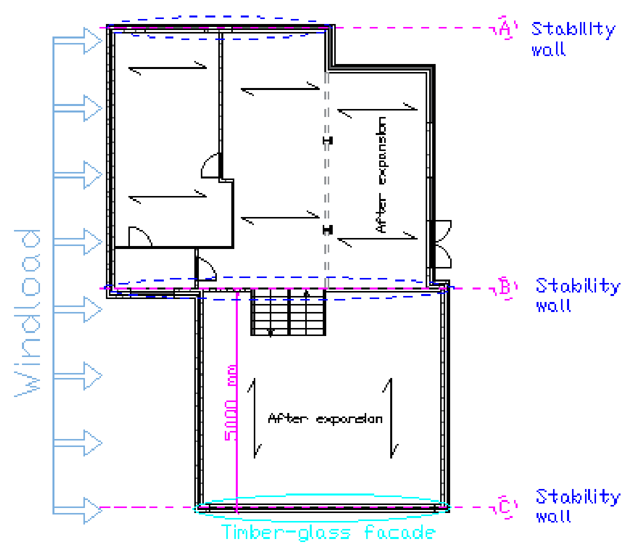


Figure 31: wind load & stability elements

The heart-to-heart distance of the stability walls is known now. The weight calculation is already determined in Table 4, and based on this information, the dead and live load on the timber-glass façade (C-axis) could be determined at kN/m.

On timber glass facade

	q_G	q_Q	w	l	h	ψ_0	G_k	Q_k	
Roof	0,75	-	2,5	-	-	-	1,90	-	kN/m
	-	1,0	2,5	-	-	-	-	2,5	kN/m
Timber-glass façade	0.60	-	-	-	3,0	-	1,80	-	kN/m
							3,7	3,0	kN/m
Wind load	-	0,81	2,5	-	-	-		2,0	kN/m
								2,0	kN/m

Table 5: load division on the timber-glass facade

Based on the provided dimensions and structural drawing depicted in Figure 31, the weight calculation is executed and presented in Table 5. The roof structure is assumed to be statically performed, and therefore half of the roof beam (2.5m) will be resisted by the timber-glass façade. The height of the glass façade is 3m.

The following equation must satisfy the requirement to ensure that there is no tension due to horizontal wind load at the connection between the glass facade, timber frame, and foundation beam.

$$\gamma_Q * \sum F_{H,wind} \leq \gamma_G * \sum F_{V,G}$$

$The\ following\ values\ are\ valid\ for\ CC1$

$\gamma_Q = 1.35$ safety factor for life load

$F_H = 2.0 \frac{kN}{m}$ total magnitude of horizontal wind load

$\gamma_G = 0.9$ safety factor for dead load

$F_{V,G} = 3.7 \frac{kN}{m}$ total magnitude of the dead load

$$1.35 * 2.0 \leq 0.9 * 3.7 \rightarrow 2.7 \frac{kN}{m} < 3.3 \frac{kN}{m}$$

The tension in the foundation is with the calculation hereabove eliminated, which simplifies the connection between the timber-glass facade and the foundation slab/beam. However, the timber-glass façade must be designed and structurally calculated on compression and bending tensile stresses. To do so, first, the maximum bending tensile strength resistance capacity of the prestress glass has to be calculated. Followed by the magnitude of the bending tensile stress due to wind load. The glass façade should be designed with sufficient thickness to resist the occurring bending tensile stresses. The following formula will determine the bending tensile stress resistance of the glass façade according to NEN-EN 2608;

$$f_{mt;u;d} = \frac{k_a * k_e * k_{mod} * k_{sp} * f_{g;k}}{\gamma_{m;A}} + \frac{k_e * k_z * (f_{b;k} - k_{sp} * f_{g;k})}{\gamma_{m;V}}$$

Where

- $f_{mt;u;d}$ bending tensile strength of prestresses glass
- k_a factor for suface effect (For point load) = $1 * A^{-\left(\frac{1}{25}\right)} \rightarrow 1 * (2 * 2.6)^{-\left(\frac{1}{25}\right)} \rightarrow 0.94$

- k_e factor for edge qulity (in plane loading) = 1.0
- k_{mod} load duration factor = $\left(\frac{5}{t}\right)^{\frac{1}{c}} \rightarrow \left(\frac{5}{50*365*24*3600}\right)^{\frac{1}{16}} = 0.29$
- k_{sp} factor for surface structure = 1.0
- $f_{g;k}$ characteristic bending tensile strength $float = 45 \frac{N}{mm^2}$
- $\gamma_{m;A}$ material factor of glass = 1.8
- k_z factor zone of the pane = 1.0
- $f_{b;k}$ characteristic value of prestress heat strenghtened glass =70 $\frac{\mathrm{N}}{\mathrm{mm}^2}$
- $\gamma_{m:V}$ material factor for prestressing 1.2

$$f_{mt;u;d} = \frac{0.94 * 1.0 * 0.29 * 1.0 * 45}{1.8} + \frac{1.0 * 1.0 * (70 - 1 * 45)}{1.2} = 27.7 \frac{N}{mm^2}$$

The bending tensile strength of the prestressed heat-strengthened glass is just calculated hereabove. The next step is to determine the thickness of the glass façade to resist the combined loads. Examination of the glass thickness on bending is not normative. However, this is executed to demonstrate that the applied glass thickness also fulfills this requirement.

$$\sigma_{Ed} = \frac{M_{wind} * z}{I_y}$$

- σ_{Ed} stress due to wind load on glass pane
- M_{wind} moment on glass pane due to wind load
- I_y moment of inertia

$$I_y = \frac{1}{12} h t^3 \rightarrow \frac{1}{12} * 3500 * (12 * 2)^3 = 4.0 * 10^6\ mm^4$$

$$M\_wind = \gamma_Q * (0.5 * q_{wind} * h^2) \rightarrow 1.35 * 0.5 * 2.0 * 2.6^2\ ) = 9.2\ kNm$$

$$z = 0.5 * t_{glass\ thickness} \rightarrow\ 0.5 * 20mm = 0.01\ m$$

$$\sigma_{Ed} = \frac{9.2\ 10^6\ Nm * 10\ mm}{4.0 * 10^6\ mm^4} = 23.0 \frac{N}{mm^2}$$

$$U.C = \frac{\sigma_{Ed}}{f_{mt;u;d}}\ \leq 1.0 \rightarrow \frac{23.0}{27.7} = 0.83$$

In the calculation is, the glass tolerances left out of consideration. To include sudden breakage, accidental load, glass drilling, and other influences disadvantaging the glass performance, thicker glass plates are required. This is contrary to steel, concrete, timber, and other construction material, where only material factors are satisfied to achieve the material's final resistance capacity.

3.3.2 Stress at the connection

The application of load and connect ability of timber-glass façade to the surrounding structural members is discussed already in chapter 3.2. The magnitude of vertical and horizontal forces on a timber-glass façade is determined in chapter 3.3.1. In the same chapter, it is proved by calculation that there is no tension in the connections. However, the timber frame around the glass façade must be examined on combined stresses to accomplish the manual calculation.

As mentioned before, loads could better be applied as a point load rather than a line load. This is to prevent unexpected, concentrated loads on the glass façade. Therefore, the maximum compression stress will also occur under the point load. The stress resistance of the connected materials and dimensions is required to examine whether a timber-glass façade can resist the stress at the connection. Figure 17 presents a very suitable connection, which is also applied in the penthouse project in Rotterdam. However, detailed information, material properties, and the exact dimensions of the involved member parts of the suppliers are missing. Therefore, the detailed drawing presented in Figure 32 will be examined on combined stresses and elaborated in chapter 5. The following properties could be used for the verification.

Timber strength class D50

Bending strength $f_{m,k} = 50 \frac{N}{mm^2}$

Compression strength $f_{c,0,k} = 29 \frac{N}{mm^2}$

Shear strength $f_{v,k} = 4.0 \frac{N}{mm^2}$

Modulus of elasticity $E_0 = 14000 \frac{N}{mm^2}$

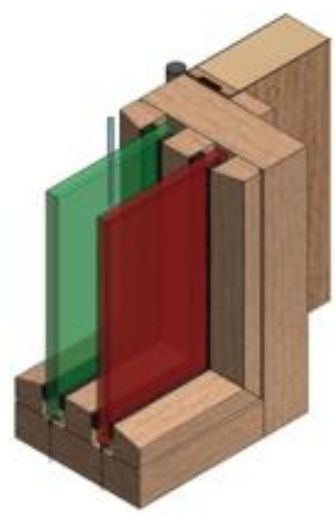


Figure 32: connection of timber-glass facade

Heat-strengthened glass

Bending tensile strength $f_{mt;u;d} = 27.7 \frac{N}{mm^2}$

Compression strength $f_{g,c} = 1000 \frac{N}{mm^2}$

Modulus of elasticity $E_{0,mean} = 70.000 \frac{N}{mm^2}$

Poisson ratio $v_g = 0.23$

By considering the connection of the timber-glass façade to the surrounding structural elements (concrete, timber, steel, glass, and masonry walls), the timber material with the lowest strength properties will remain normative to be examined on compression stresses. The Eurocode provides the following formula to examine the compression stress on timer structures.

$$\sigma_{C,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,90} * f_{c,90,d}} * \sin^2 \alpha + \cos^2 \alpha}$$

Before starting with the compression strength calculation, the timber structure's environment must be determined. The critical parameter affecting the strength of the timber structure is the modification factor k_{mod}, related to loading duration and the service class. With permanent load duration and timber surface exposed to moisture and rain, the modification factor becomes k_{mod} =0.5. Based on this information, the calculation can proceed:

$$k_{mod} = 0,5$$

$$f_{c,0,d} = \frac{f_{c,o,k} * k_{mod}}{\gamma_m} \rightarrow \frac{30 * 0.5}{1.3} = 11.5 \frac{N}{mm^2}$$

$k_{c,90} = 1.0$ (for rectangular shapes)

$$f_{c,90,d} = \frac{f_{c,90,k} * k_{mod}}{\gamma_m} \rightarrow \frac{8.1 * 0.5}{1.3} = 3.1 \frac{N}{mm^2}$$

$\alpha = 90°$ (load is vertically applied)

$$\sigma_{C,\alpha,d} = \frac{F_{Ed}}{b * h}$$

(F_{Ed} calculation based on Table 4 and b*h related to Figure 32)

$$\sum F_v = (\gamma_G * q_G + \gamma_Q * q_Q) * l * b \rightarrow (1.08 * 0.75 + 1.35 * 1.0) * 3.5 * 2.5 = 18.9\ kN$$

$$\sigma_{C,\alpha,d} = \frac{18{,}9 * 10^3}{71 * 120} = 2.25 \frac{N}{mm^2}$$

$$2.25 \leq \frac{11.5}{\frac{11.5}{1.0 * 3.1} * \sin^2 90 + \cos^2 90} \rightarrow 2.25 \frac{N}{mm^2} < 3.1 \frac{N}{mm^2}$$

Shear stress

Horizontal wind surface loads are transferred through the slabs to the stability elements. Slabs are much stiffer in the x-direction, which shares the load with the corresponding stability elements based on their bending stiffness. This load is mainly schematized as a point load at the top corner of the stability element schematized in Figure 33.

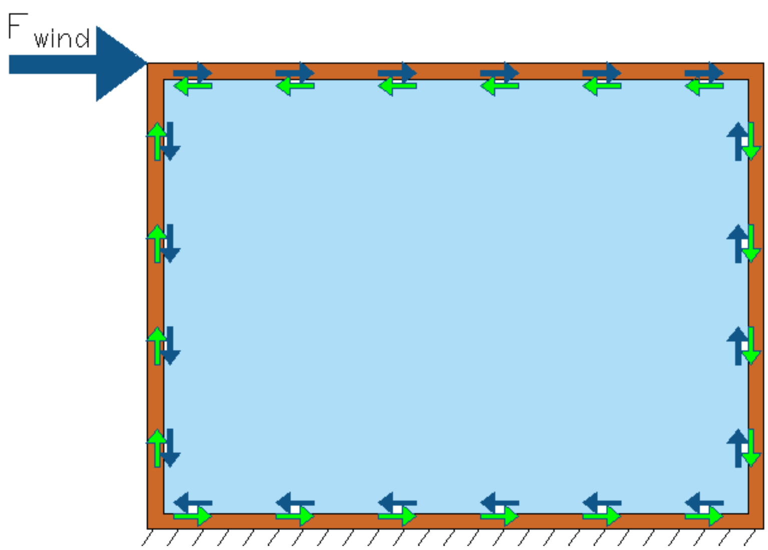


Figure 33: shear stress at connections

The inhomogeneous timber-glass façade might undergo shear stress at the connection due to the wind load. The relationship between the glass pane and the timber frame is glued and quite strong. However, the decisive member must be examined on shear stresses, which is the timber frame. The following equilibrium must satisfy the requirement.

$$\sigma_v \le f_{v,d}$$

$$\sigma_v = \gamma_Q * \left(\frac{F_{wind}}{t_{glass} * l_{connection}}\right) = 1.35\left(\frac{2.0 * 2.6 * 10^3}{30 * \left(2 * (3500 + 2600)\right)}\right) = 0.02\frac{N}{mm^2}$$

$$f_{v,d} = \frac{f_{v,k}}{\gamma_m} \to \frac{4.0}{1.3} = 3.1\frac{N}{mm^2}$$

$$0.02\frac{N}{mm^2} < \frac{3.1N}{mm^2}$$

The shear stress will be distributed over the entire edge of the rectangular-shaped timber-glass façade. With the above calculation, it is demonstrated that the equilibrium is fulfilled. Further investigation and deep analysis will be done with the FEM program.

3.4 CONCLUSION

The main goal of this chapter was to determine the bending tensile resistance of the glass through manual calculation. However, before starting with the glass structure calculation, it was necessary to identify first the variety of forces working on the timber-glass façade. Where the magnitude of the dead load is related to the thickness of the applied material, the live load is associated with the function of the building. Given that the timber-glass façade will also be accountable for stabilizing the structures, the magnitude of the wind load had to be identified as well. Wind load is not only related to the height of the building, but the building environment is also crucial. The surface of the Netherlands is subdivided into three wind areas, and the Eurocode prescribes different wind loads per area.

One of the essential tasks of manual calculation was identifying the glass pane's load transfer capacity. The compression strength of the glass is already discussed, which is even higher than concrete. However, glass is vulnerable to tension stresses, significant bending, and concentrated loads, especially when glass pane edges have some imparity. Based on discussed glass projects and analysis of the load transfer according to a structural mechanic, it’s concluded to apply the load as a point load rather than a distributed line load. This will prevent local glass damage and unexpected failure.

The load-bearing capacity of the timber-glass façade is, according to NEN 2806, manually examined. Based on the bending tensile resistance capacity of the heat-strengthened glass determined by manual calculation, further investigation of timber-glass façade as load bearing and stability element can proceed. The following chapters will investigate the suitability of a timber-glass façade as a hybrid system. To demonstrate the load distribution and resistance capacity of the timber-glass façade. A finite element program is applied to achieve this and support the manual calculation.

4 CASE STUDY 2: HYBRID SYSTEMS

The definition of hybrid systems in the construction industry is broad. Generally, the term hybrid is used for precast concrete in combination with cast in situ. However, the primary purpose of the hybrid system is to get the best of the combined material/technique with lower costs in a shorter time and better performance or at least one of these.

The term hydride in this research combines a timber-glass façade with other construction materials to stabilize the structures against horizontal (wind) loads. An actual project is investigated in this chapter to examine the appropriateness of a timber-glass façade as a hybrid material with other stability elements. The main goal is to demonstrate the load division between various structural elements based on their bending stiffness and dimensions.

FE-program
- Application of modern state of the art programs cannot be excluded anymore for complex structural analysis. AxisVM program is applied in this research to study the designs.

FE-modeling
- The existing steel structure of this case study was first modeled with 2D-software followed by the FE-program. The aim is to study the behavior of the hybrid system and compare it with the existing one.

hybrid system
- The climax of this case study is discussed in this paragraph. The ability to combine an existing system with a timber-glass façade is elaborated.

optimize
- The intersection of the existing steel elements is optimized after application of the timber-glass facade in the hybrid system. The outcomes are expressed with unity checks.

FE-analysi
- FE-designs reduce the calculation time, the results will be accurate, and the analysis will be of more value. Nevertheless, understanding of structural knowledge is a must to prevent misinterpretation of analysis.

4.1 Introduction of case study-2

This case study is very carefully selected to demonstrate the load-bearing capacity of the timber-glass façade as load bearing member and, at the same time, stabilize the building together with another structural component against horizontal wind load. The important aspect which makes this project appropriate for this case study is the availability of the glass façade and steel portals. However, it should be noted that the glass façade in the original design is not load-bearing or makes part of the stability elements. This building was renovated in 2021, and additional steel portals were applied to stabilize the new expansion part. An overview of the original structure with the expansion part is presented in Figure 34.

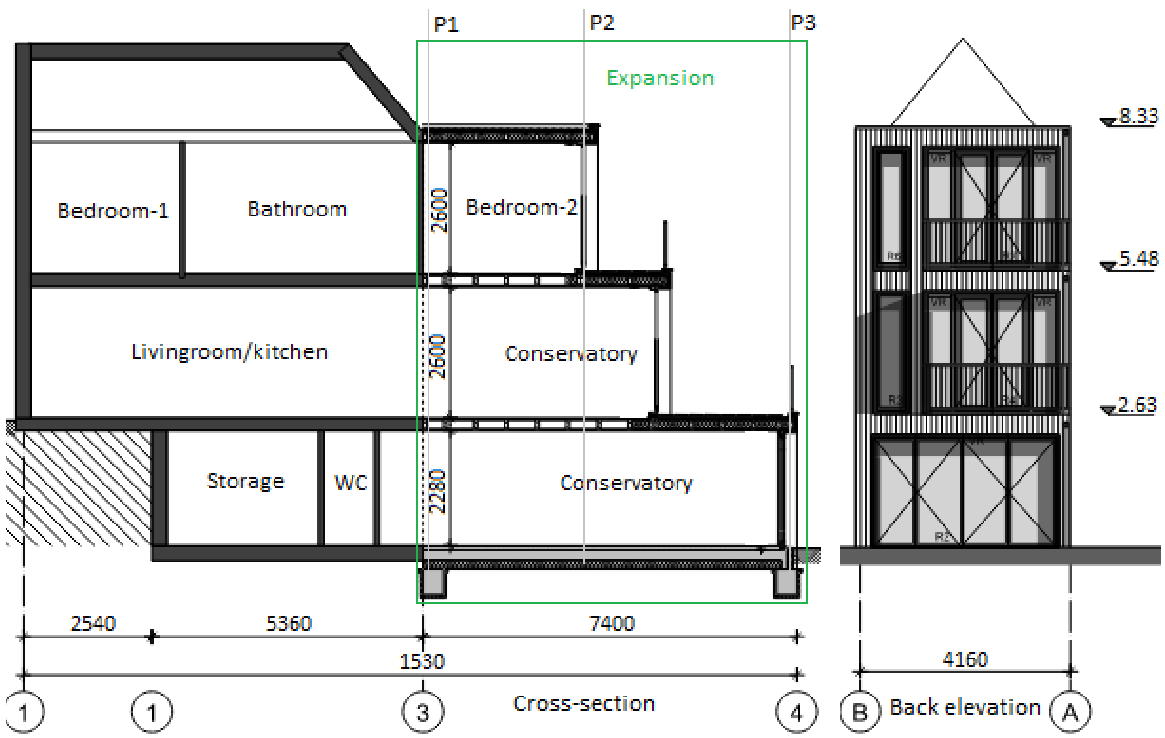


Figure 34:overview of the building at Maassluis

The newly erected part of the building is mainly designed with glass panes at the back elevation. Steel portals are applied to stabilize the structure, which is quite common in the Netherlands for this type of expansion. The glass façade is like the previous project (case study 1), left out of consideration and did not participate in stabilizing the building. P1 highlights the location of the steel portals to P3 in Figure 34—every floor end with a wide glass façade at the back elevation surrounded by a portal. This research investigates where it is possible to make the glass façade load bearing and allow them to stabilize the expansion building. This means that the steel portal will be partly substituted by a timber glass façade, which makes this project a suitable hybrid system to investigate.

There are three steel portals applied in Figure 34. Portal-1 connects the existing building with the expansion part without the involvement of any glass panes. Portal-2 and portal-3 are used around the glass façade to transfer the vertical load to the foundation and ensure the stability of the expansion building in a transversal direction. Therefore, the best position to create a hybrid system of steel portals with a timber-glass façade is portal-2. Substitution of portal-3 will not provide any hybrid system. Although theoretically, it is possible to apply a steel portal in combination with a timber-glass façade in a horizontal parallel system (Figure 35, III) as load bearing and to create a hybrid system, due to their significant differences in bending stiffness and modulus of elasticity, it will not work. Steel portals possess sufficient elastic and strain deformation, while glass is a very stiff material. Therefore, applying two load-bearing materials, which cannot cooperate, wastes material, and the decisive one will resist the entire load until failure.

In Figure 35, there are three structures displayed. Structure I represent the cross-section of portal-2 discussed in Figure 34. Structure II is the transformation of structure I, where a timber-glass façade substitutes a steel portal at the top level. Structure III is a random location where a timber-glass façade is applied while a steel portal stabilizes the structure. As mentioned before, structure III is less suitable to analyze as a hybrid system due to significant differences in material properties of the structure material.

A finite element analysis program will be applied to understand the hybrid system's behavior and load transfer. Based on this analysis, the suitability of the timber-glass façade will be discussed.

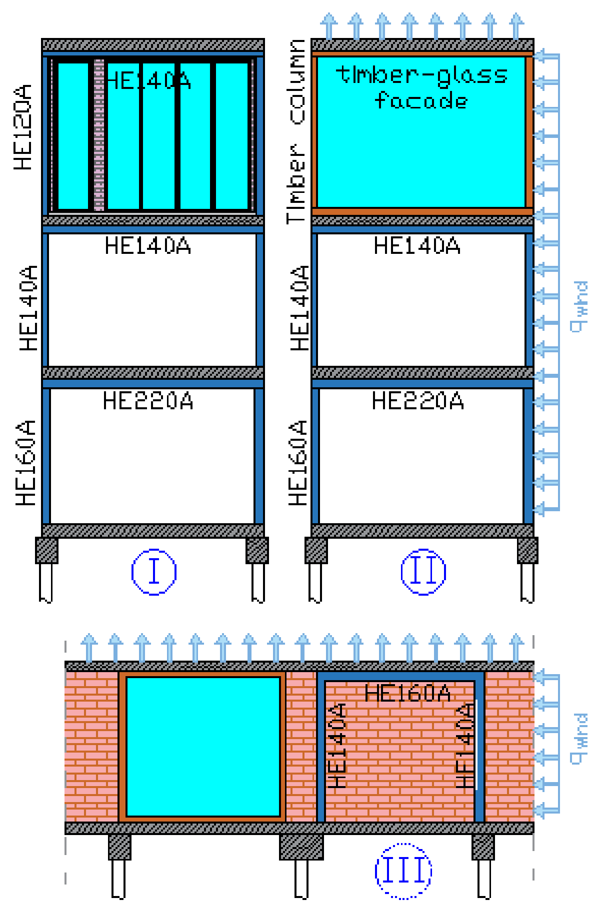


Figure 35: vertical & horizontal hybrid systems

4.2 FINITE ELEMENT PROGRAM

To understand the behavior of the glass structure under combined load and to determine the structural possibilities of the timber-glass façade, a finite element analysis is necessary. This will determine the load-bearing capacity of the timber-glass façade and the load division over the glass pane. Invention, extension, and accessibility of the modern state of the art enabled structural engineers to design structures beyond the traditional ones. The Eurocode requires more accuracy and examination of various combined loads on structural elements, like flexural buckling and lateral torsional buckling on beams. Examining the torsional buckling of a column loaded by normal force and bending in X- and Y-direction to design the cross section is another example of combined loading, which is very time-consuming to execute manually.

The invention and exploration of the latest design techniques, like parametric design combined with finite element analysis, enabled structural designers to go beyond standard calculations and shapes. With FE-modeling, the engineers could follow and present the load transfer and identify the local stress peaks. With this information in mind, the discussed project will be modeled with a finite element program to determine the load transfer from the roof through a timber frame to the glass pane.

4.3 FINITE ELEMENT MODELING

To understand the behavior of the glass structure under combined load and to determine the structural possibilities of the timber-glass façade, a finite element analysis is necessary. This will determine the load-bearing capacity of the timber-glass façade and the load division over the glass pane. Invention, extension, and accessibility of the modern state of the art enabled structural engineers to design structures beyond the traditional ones. The Eurocode requires more accuracy and examination of various combined loads on structural elements, like flexural buckling and lateral torsional buckling on beams. Examining the torsional buckling of a column loaded by normal force and bending in X- and Y-direction to design the cross section is another example of combined loading, which is very time-consuming to execute manually.

The invention and exploration of the latest design techniques, like parametric design combined with finite element analysis enabled structural designers to go beyond standard calculations and shapes. With FE-modeling, the engineers could follow and present the load transfer and identify the local stress peaks. With this information in mind, the discussed project will be modeled with a finite element program to identify the load transfer from the roof through a timber frame to the glass pane.

4.3.1 Layout

The layout and dimension of the building discussed in chapter 4.1 is required to model the structure with the finite element program. Figure 36 presents the design of the three-story building at Maassluis. The existing part is between canvases 1 to 3, and the expansion part is situated between canvases 3 to 4. As mentioned in chapter 4.1 and presented in Figure 34, the best location to substitute the steel portal by load bearing and stabilizing with a timber-glass façade is the location of portal-2. This location is also highlighted in the figure below. The steel portal of the first floor, with a width of 3800 mm, will be replaced by a timber-glass façade. The remaining steel portals at the ground floor and basement will remain. These two structural elements are integrated between the masonry brick walls.

Figure 36: layout of the building at Maassluis

4.3.2 Load

Based on the center-to-center distance between the load-bearing members, the load on the steel portal and timber-glass façade could be calculated to enable the structural analysis. An overview of the dead and live load, including horizontal wind load with other building specifications to classify the buildings according to the Eurocode for further calculation, is presented below. Load application on a timber-glass façade is already discussed in chapter 3.2. The important message was to apply the load as a point load to prevent unexpected, concentrated peak stresses on the glass façade due to deformation of the structural beam (steel, timber) under the line load.

The boundary condition for this load calculation is that (roof) load will be transferred through purlins (Dist. 1,2m) as a point load to the timber-glass façade. The information below is necessary to examine the cross-section of the steel portal and to design the thickness of the glass façade.

Construction class	CC1
Reliability class	RC1
Center to center distance	3.7 m
Building height	8.3 m
Floor height	3.0 m
Wide	4.2 m
Wind area	II, built
C_{pe}	0.8+0.5=1.3
$C_s C_d$	0.85
$q_{p(z)}$	0.63 kN/m^2
$q_{G,roof}$	0,7 kN/m^2
$q_{Q,roof}$	1,0 kN/m^2
$q_{G,floor}$	5.0 + 1.6 = 6.6 kN/m^2
$q_{Q,floor}$	1.75 + 0.8 = 2.55 kN/m^2

4.3.3 2D-ELEMENT

The initial steel portal to stabilize the expansion building is designed with a 2D-element program (Technosoft) presented in Figure 37. The dimensions of the steel columns and beams are generally determined based on applied loads and generated shear forces, bending moments, buckling and torsional buckling, or a combination of these. However, the dimension of the steel portals, which is accountable for the stability of the expansion part, is related to the maximum permissible displacement. To prevent damage to the concrete slab or masonry brick wall, the steel portal should not exceed the maximum permitted deformation/ displacement prescribed in the Eurocode (CUR-Aanbeveling, 2021).

To compare the rigidity of the steel portal with the timber-glass façade, the applied steel portal of the original design is inserted was a 2D element in Technosoft and recalculated. The following load cases are applied on the steel portal for calculation.

$$F_{wind,\ roof} = C_s C_d * C_{pe} * q_{p(z)} * b * h_3 \rightarrow 0{,}85 * 1{,}3 * 0{,}63 * 3{,}7 * 1{,}5 = 3.9\ kN$$

$$F_{wind,\ 1e\,\&\,2e\,fl} = C_s C_d * C_{pe} * q_{p(z)} * b * h_{2,1} \rightarrow 0{,}85 * 1{,}3 * 0{,}63 * 3{,}7 * 3.0 = 7.7\ kN$$

$$F_{G,roof} = 0.7 * 3.7 * 1.2 = 3.1\ kN \quad and \quad F_{G,roof} = 1.0 * 3.7 * 1.2 = 4.4\ kN$$

$$q_{G,1e\,\&\,2e\,floor} = 6{,}6 * 3.7 = 24.4\frac{kN}{m} \quad and\ q_{Q,1e\,\&\,2e\,floor} = 2.55 * 3.7 = 9.40\frac{kN}{m}$$

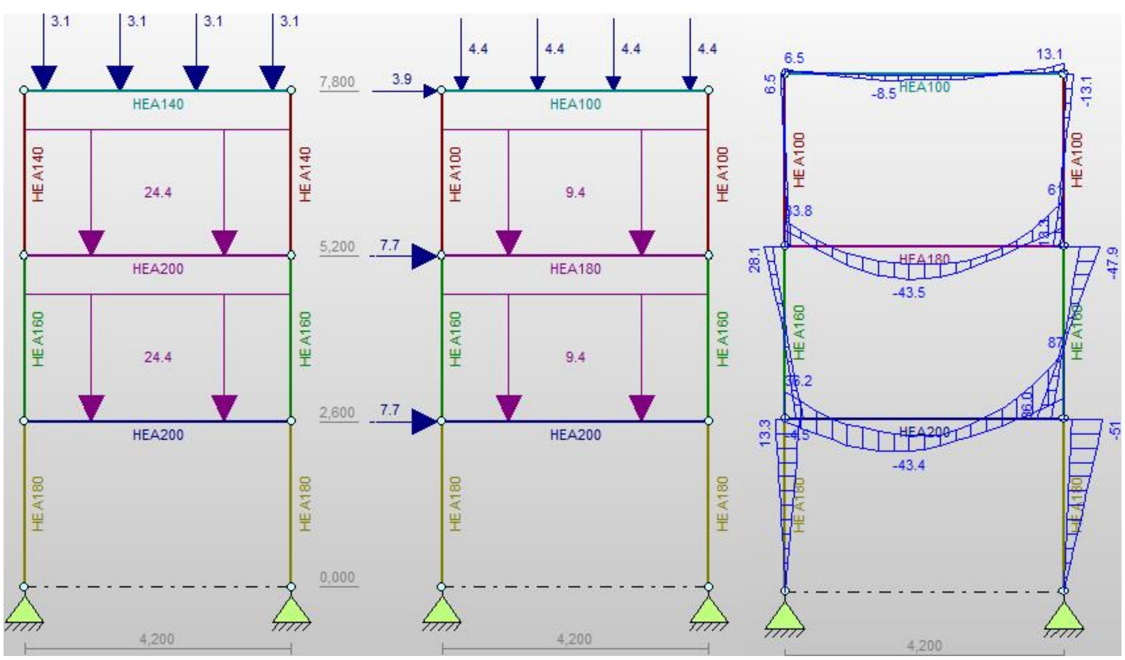


Figure 37: steel portal with corresponding loads

4.3.4 Cross-sectional examination

The portal presented in Figure 37 with the applied cross-section of the columns and beams fulfills the requirement to satisfy the bending moment, shear forces, regular forces, flexural buckling, lateral-torsional buckling, and the combination of flexural buckling with lateral torsional buckling according to Eurocode-3. An overview of the unity checks of this examination is presented in Figure 38. Each diagram with a unique color represents a different structural examination of the steel element.

The main goal of this examination is to emphasize the rigidity of the steel portal. The steel elements' cross-section is determined based on the applied vertical loads in this chapter to focus on that specific point. Steel elements can resist compression and tension stresses very well. However, due to their thin cross-section, they can undergo a large deformation.

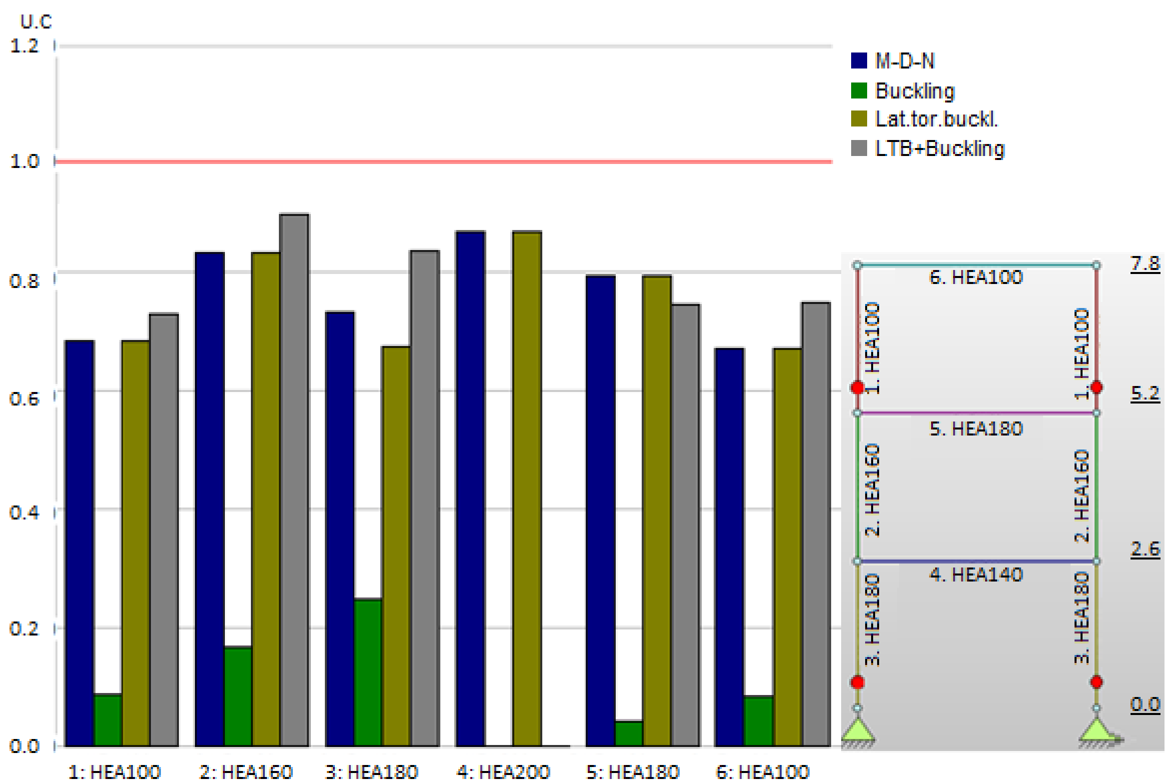
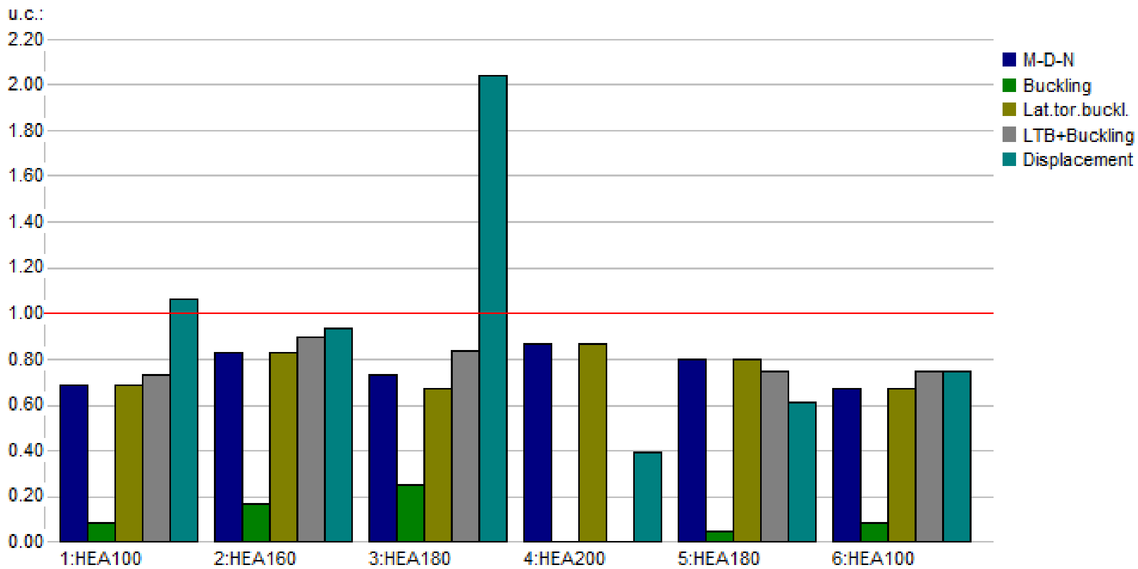


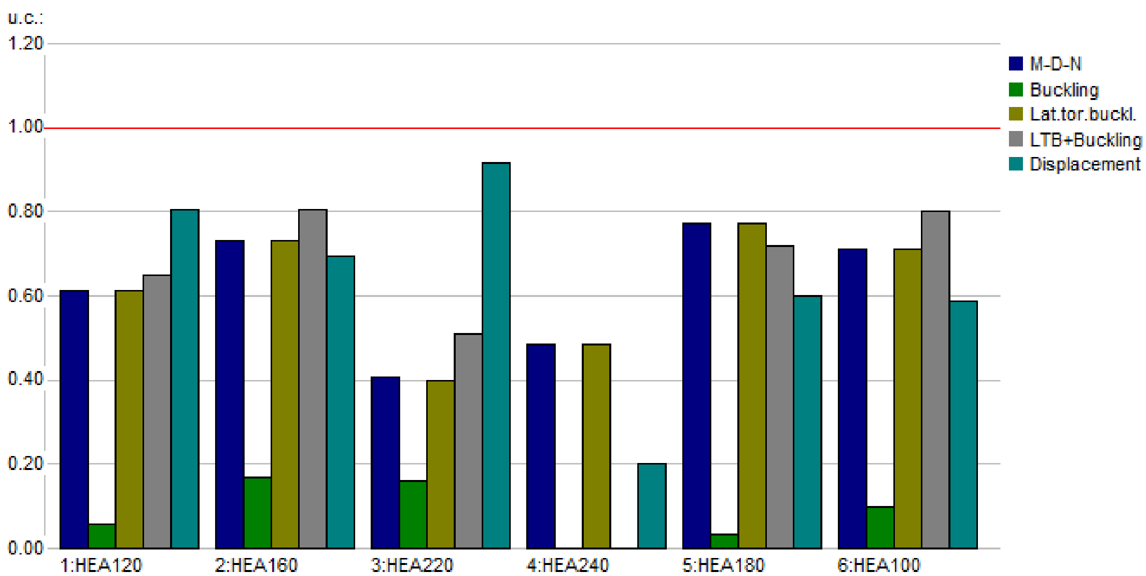
Figure 38: calculation of steel portal with a corresponding unity check

4.3.5 Cross-sectional examination inc. displacement

In this paragraph, the cross-section of the steel elements for the portal is reexamined. The difference from the previous examination is the application of horizontal displacement.



To fulfill the displacement requirement, the cross-section of the steel elements must be enlarged. The connection between the column and beam is significant in response to the horizontal load and displacement. It should also be considered not to enlarge only the cross-section of the column until it fulfills the requirement but also to think of the manufacturability and ratio of the steel column and beam. Figure 56 overviews the steel elements which fulfill the strength and displacement requirements.



4.4 FE-DESIGN OF THE HYBRID SYSTEM

The steel portal has been discussed, and the cross-section is optimized to resist the combined vertical and horizontal loads. These steps were required and essential to compare and emphasize the difference between the steel portal and the hybrid structure of this paragraph. In Figure 39, two structures are designed with the FE program for structural analysis. The cross-section of the steel elements is the same as demonstrated and calculated with the 2D program in paragraph 4.3.5 of this report. The constraints are rigid and fixed in X-, Y- and Z-directions to provide sufficient resistance against vertical and horizontal stresses and prevent the structure's instability. To understand the differences in stress distribution, displacement, and load transfer, the magnitude of the loads presented in Figure 37 is taken over to design the presented structure of Figure 39.

The hybrid system is established by substituting the steel portal at the top part with a timber-glass façade. The timber frame around the glass façade is designed with hard timber (D70), and, given that the structure is accessible for rainwater, climate class 3 is selected for structural analysis. The glass façade is 24 mm, and the final thickness will be determined after the execution of the structural analysis.

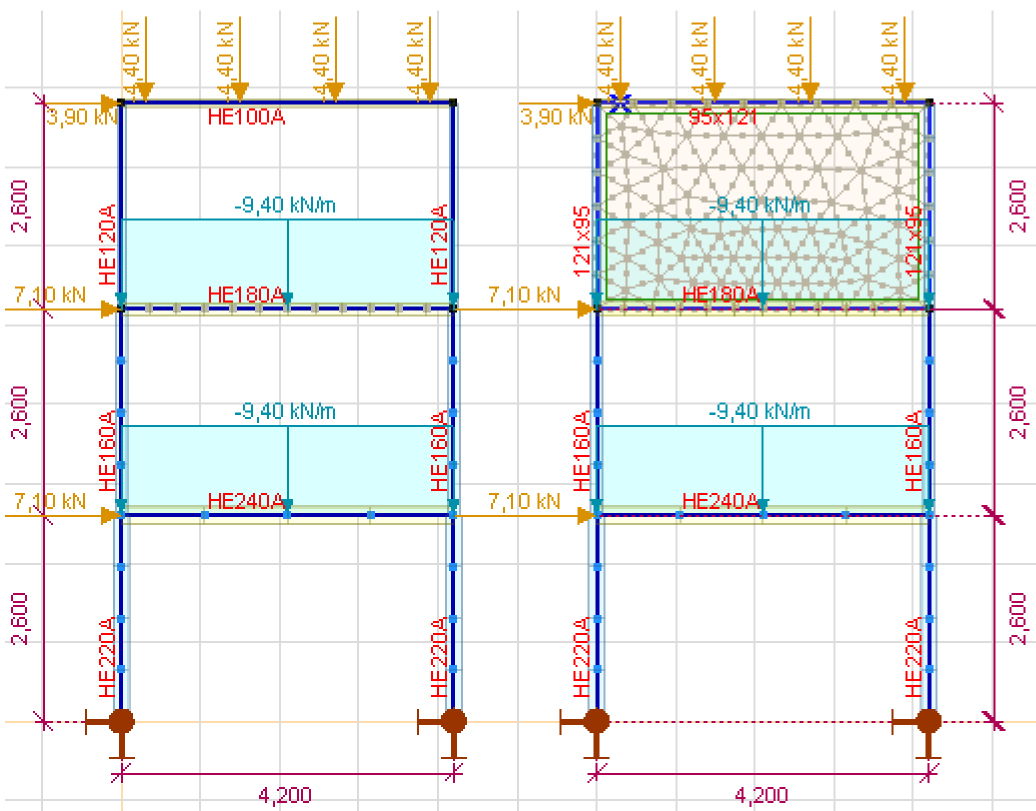


Figure 39: steel portal (left) and hybrid system (right)

4.4.1 FE-ANALYSIS

Before discussing the result of the finite element models, a structural engineer needs to have clear expectations and approximate values in mind. The output should be in line with the input to trust the model. The timber-glass façade is rectangular and loaded in the Z-direction by the dead and live load. Wind load works in the X-direction and will create compression directly under the applied load and at the supports. From the theory of the FE method, loads can cause peak stresses under concentrated point loads, at point supports, at the edges, when cross-section changes in dimensions, etc.

To understand the behavior of inhomogeneous materials under various loads, the structure presented in Figure 39 (right part) is subdivided into a timber-glass facade and a steel portal. First, the stresses in the glass pane will be analyzed to check whether the thickness satisfies the strength requirement. The next step of the FE analysis is to examine the cross-section of the steel elements. Once both structures fulfill the requirement, the connection and total displacement of the two structures will be investigated to realize the hybrid system.

Figure 40 demonstrates the output of the timber-glass façade. Not only are the loads applied as concentrated point loads, but the constraints are also designed as point support to analyze the stress distribution over the glass pane. Comparing the point load through the timber frame and point support at the bottom, one can conclude that the timber frame distributes the load over a certain distance and reduces the peak. Therefore, the maximum stresses also occurred around the constraints. To prevent the glass pane's sudden failure and fulfill the structural requirement, the glass pane must have sufficient thickness to resist stresses at every point. An overview of the maximum stresses is presented in the figure below;

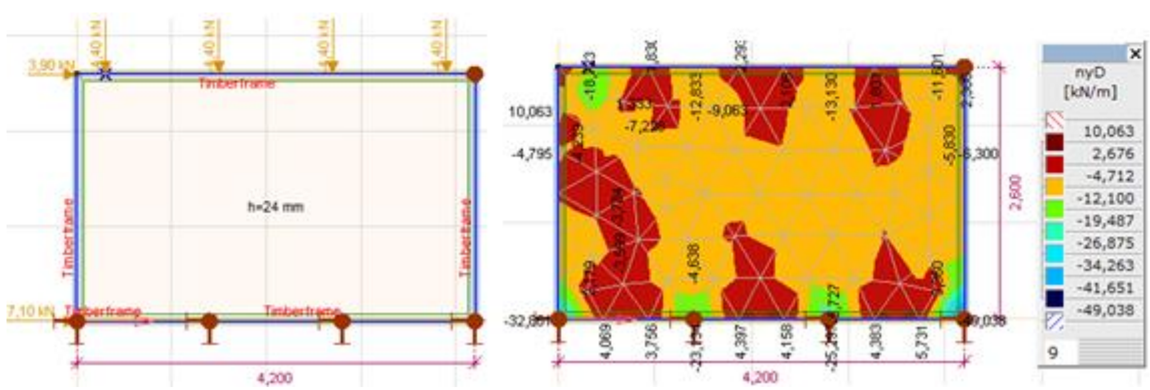


Figure 40: stress due to concentrated point load

Given glass material is not integrated in every finite element program, additional improvision is required to calculate the final stress in the glass pane. Therefore, the following equation must satisfy;

$$f_{m;u;t;d;} \geq S_{xx} \;\; \& \; f_{m;u;t;d;} \; \geq S_{yy} \; with \; f_{m;u;t;d;} = 27{,}7 \frac{N}{mm^2}$$

Steel portal

The maximum stresses in the cross-section of the steel elements are also presented as a unity check in Figure 41. The applied cross-section determined with the 2D program fulfills the requirements. By comparing the steel portal (left) with the hybrid model (right), one can conclude that the unity checks for the steel portal are higher compared to the hybrid system (columns 0.554 vs. 0.769, beam 0.543 vs. 0.674). The reason is predictable and could be derived from the displacement, already discussed in paragraphs 4.3.5 and ff.

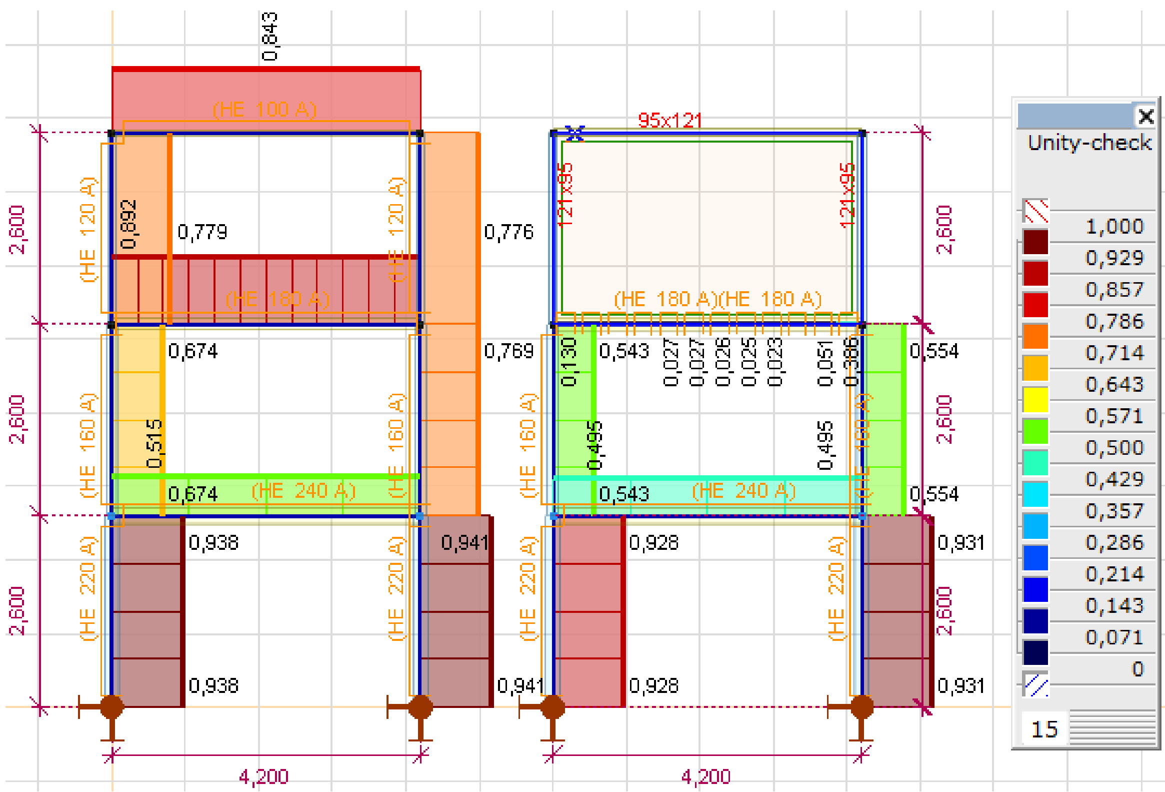
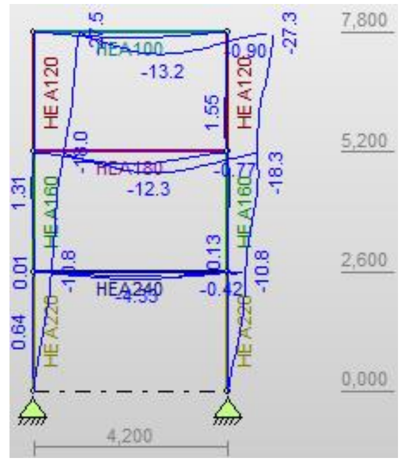


Figure 41: unity checks of the applied cross-section of the steel portal

The displacement of the steel portal is once again presented here. Numerical information about how the unity checks are established is presented in Figure 42. The presented result is of the basement column, calculated on combined loads. The maximum U.C. occurs at the SLS check, where displacement is decisive. The U.C. for the remaining structural strength examination (ULS) remains below 0.6, clarified through various diagrams in chapter 4.3.5. The same is valid for the steel portal's remaining column and beam structures.



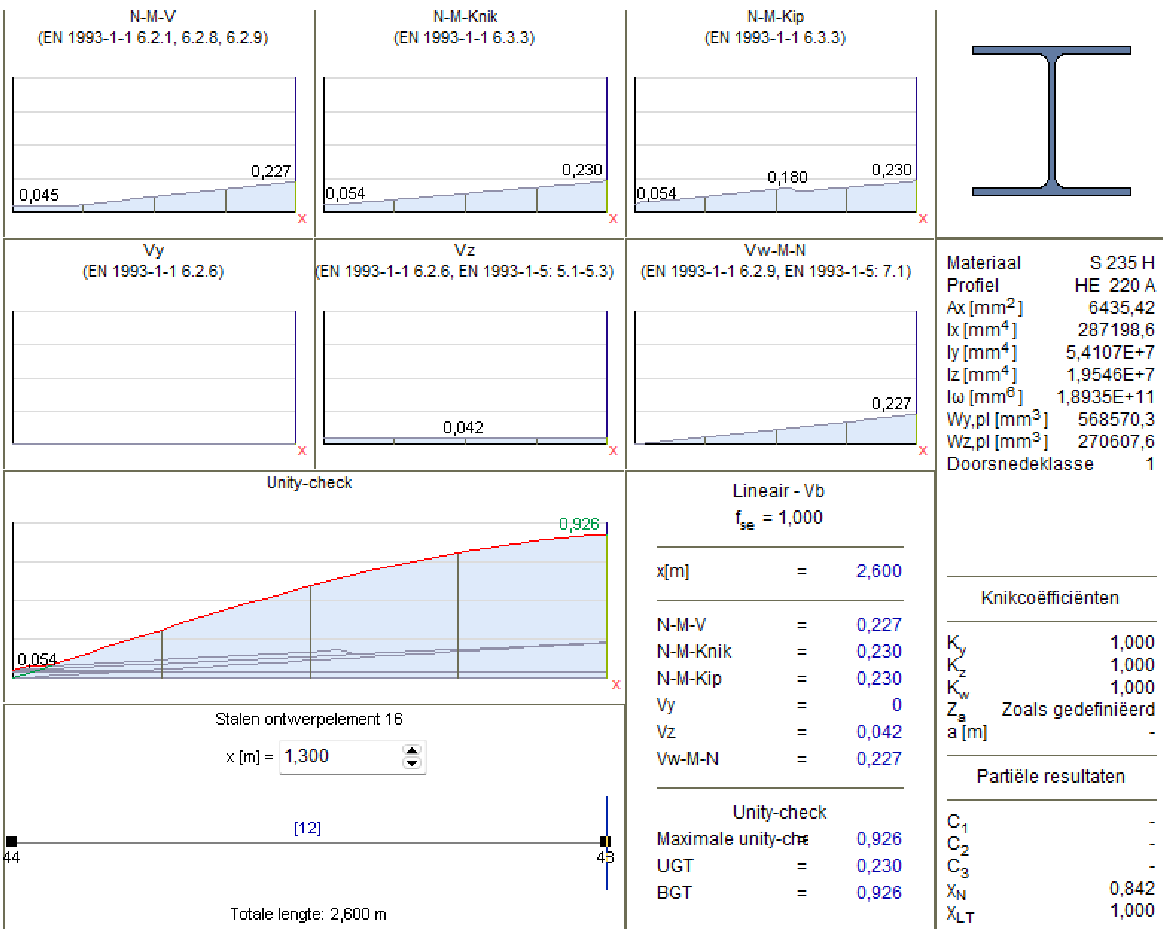


Figure 42: detailed information on the unity checks

Although the structural properties and rigidity of the glass pane were sufficient to conclude that displacement of the structure will be reduced by combining the steel portal with a timber-glass façade, the structural calculation and FE analysis confirmed and strengthened the theory. The glass pane is rigid, and the displacement will be compared to the steel portal almost negligible. Therefore, combining the timber-glass façade along with the concrete slab with the steel portal will reduce the total deflection compared to the steel portal. Load transfer from the timber-glass façade to the steel portal could be enabled through a concrete slab or directly connected to the steel beam of the portal. Connecting the ability of a timber-glass façade to another structural medium is discussed in the next paragraph.

4.4.2 Connection of hybrid systems

The main goal of this research is to identify the load-bearing capacity of the glass façade as a load-bearing member and stabilization of the buildings. The possibility of connecting the glass façade to the surrounding structures is generally discussed in literature studies and is not the main issue of this research. However, the load must be transferred from the timber-glass façade to the surrounding structures to realize a hybrid system. Therefore, a general design is presented in figure xx to demonstrate how to connect the timber frame to the steel portal.

The glass façade to the timber frame is based on glued connection. The theoretical background of the glued connection is extensively investigated by Huveners (Huveners, 2009). The timber frame could be easily connected with bolts and steel plates to the concrete slab like a load-bearing timber wall to transfer the vertical and shear forces.

4.5 CONCLUSION

This chapter aimed to determine, by 2D element calculation and finite element analysis, whether a timber-glass façade is suitable to be applied as a hybrid system. The load-bearing capacity of the timber-glass façade is already investigated and elaborated on in chapter 3. In an extension of that, a combination of the timber-glass façade is being investigated with other load-bearing materials to examine the possibility of creating a hybrid system to reduce the overall costs, prevent dimension of the structures and use the applied material (timber-glass façade) as a structural member for better performance and reduction of emission.

To investigate the possibility of a hybrid system made of timber-glass façade with other structural materials is, a real case study applied. The building discussed has recently been realized, but without any structural function for the timber-glass façade. The application of a 2D program in combination with a finite element program is the suitability of the timber-glass façade to create a hybrid system investigated. The hybrid system is achieved by substituting only the steel portal on the top floor with a timber-glass façade and connecting through concrete to the steel portal for load transfer to the foundation.

The main goal of this chapter is achieved by demonstrating that a timber-glass façade can be applied as a hybrid system and reduces the deformation of the expansion building. This is emphasized by applying the finite element program and comparing the displacement differences and the magnitude of the unity checks between the regular steel portal and the hybrid system composed of steel elements and a timber-glass façade.

Based on the content of this chapter, one can conclude that a timber-glass façade is a very suitable structural element to be combined with steel or concrete elements to create a hybrid system. The thickness of the glass member is also examined on combined loads and fulfills the bending tensile stresses. Connection of the timber-glass façade is also possible and comparable with structural timber wall to the steel element or concrete slab.

5 CASE STUDY 3: HIGH RISE BUILDINGS

The application of glass in the building industry has changed from a basic to a modern architectural design of the last century. The size of the glass facades has been increased in parallel with technological possibilities and better physical and thermal performance. Curtain walls represent the modern application of the glass façade in buildings without participating as a load-bearing, the glass façade must pose a particular strength to transfer the wind load to the surrounding structures.

Opening glass windows/ facades has become a primary part of building design in the past century to allow light entrance into the building and to increase the overall performance of the building. The size and location of the opening for the light entrance depend on the functionality of the space and could be determined according to NEN-2057 and the latest version NEN-EN 17037 (daglicht, 2021). The application of the glass façade as a load-bearing element in the new design and the possibility to stabilize new buildings with glass façade has lacked behind.

The goal of this chapter is to investigate the application of the glass façade as a structural member in new buildings. This will be demonstrated through a case study. The selected case study is about a large building stabilized only by a concrete lift shaft, while many glass facades are available. The glass façades on the ground floor are not used as structural members, and the remaining concrete walls are limited to create an open space. Therefore, tension occurred in the foundation piles under the concrete walls. To compensate for that, additional concrete blocks are poured in the current location to increase the dead load of the building. Additional pile reinforcement and geotechnical measurements are taken for the remaining tension to prevent failure.

Activating the glass façade as a load-bearing could make all these expensive measurements redundant and provide better load distribution over the foundation piles. To demonstrate that, different aspects must be highlighted and investigated first.

A summary of the important topics investigated in this chapter is presented in the following chart.

Introduction & Load scheme

- after introduction of the case study, an overview of the stability- and load bearing elements of the existing building is provided. The aim is to compare the load transfer from the top to the foundation piles in the existing design and after activation of the glass facade as stability element.

Weight calculation

- there are different types of loads and load combinations to be considered while designing. An overview of the suitable loads for this building according to NEN-EN is discussed under this topic. Proper weight calculation is important to be considered for structural calculation.

load distribution

- load distribution among load bearing elements is related to their bending stiffness. To determine the required thickness of the glass facade and the distribution of the load among concrete walls and glass facade, their bending stiffness is required, which is discussed under this paragraph.

wind load on glass facade

- based on determined bending stiffness, the magnitude of the load on glass facade is calculated. With this information in hand, the magnitude of the stress on the glass facade and the thickness of the glass is examined.

Tension in foundation

- after determination of the location and dimension of the glass facade as load bearing member, the tension in the foundation is re-examined. The goal is to evaluate, whether there are still tension in the foundations with increased number of stability elements.

FEM-analysis

- FE-analysis is applied to strength the accuracy and reliability of this research. FE-analysis does not identifies only the magnitude of the stress distribution in glass, but also determines tension in the foundation piles and siplacement of the building before and after activation of the glass facades.

Conclusion

- the overal benifit and pitfall of adding timber-glass facade as stability element for new building is dicussed in this paragraph. This conclusion provides suffiicient answers whether timber-glass facade is suitable to be applied as stability element in new buildings.

5.1 Introduction of case study 3

This case study is selected to investigate the appropriateness of the timber-glass façade as a load-bearing and stability element in new buildings. Case study 3 is about van der Valk hotel in Schiedam, the Netherlands. This project is selected based on the availability of many glass façades in the longitudinal direction without any structural function. To transfer the dead and live load of the roof and slabs, including separation walls, additional concrete columns, and beams are designed directly behind the glass façade. An overview of the front- and side view of the building is presented below.

43: layout of van der Valk hotel

The thermal performance of the glass facades is enormously improved, which leads to more application of the glass façade in the new buildings. Although glass has sufficient structural capacity, its structural application in the building industry is left far behind.

Longitudinal glass façade created a transparent and open design in the van der Valk hotel but also caused some challenges concerning stability. The number of stability elements at the ground level for the longitudinal direction is limited to the concrete lift shaft (Figure 44). Concentrated wind load resisted by the lift shaft, causing tension on the foundation piles under the lift shaft. To compensate for this tension, additional huge concrete foundation blocks were poured to increasing the dead load of the foundation pad. The remaining tension is allocated to the concrete piles.

This chapter aims to investigate whether tension could be eliminated/reduced at the foundations by activating the glass facades as a stability element in the longitudinal direction. This will not disturb the architectural layout of the design but will affect the building's structural behavior and load distribution.

5.2 Stability elements & load scheme

Before activating the glass façade as a stability element, it is essential first to identify and study the current location and number of the existing load-bearing and stability elements. Activating the glass façade as a stability element will be the next step to strengthening the building against horizontal wind loads.

The load transfer of van der Valk hotel is mainly regulated through the concrete from the top to downward. However, the layout of the load-bearing, and stability elements differs at the lower levels, which is presented in Figure 44. The building is about 60 m high, and the slabs and walls are designed in reinforcement concrete to resist the combined loads. To ensure the division of the ground floor and the freedom of movement for guests while checking in/out and suppliers, the concrete walls are mainly replaced by concrete columns and beams.

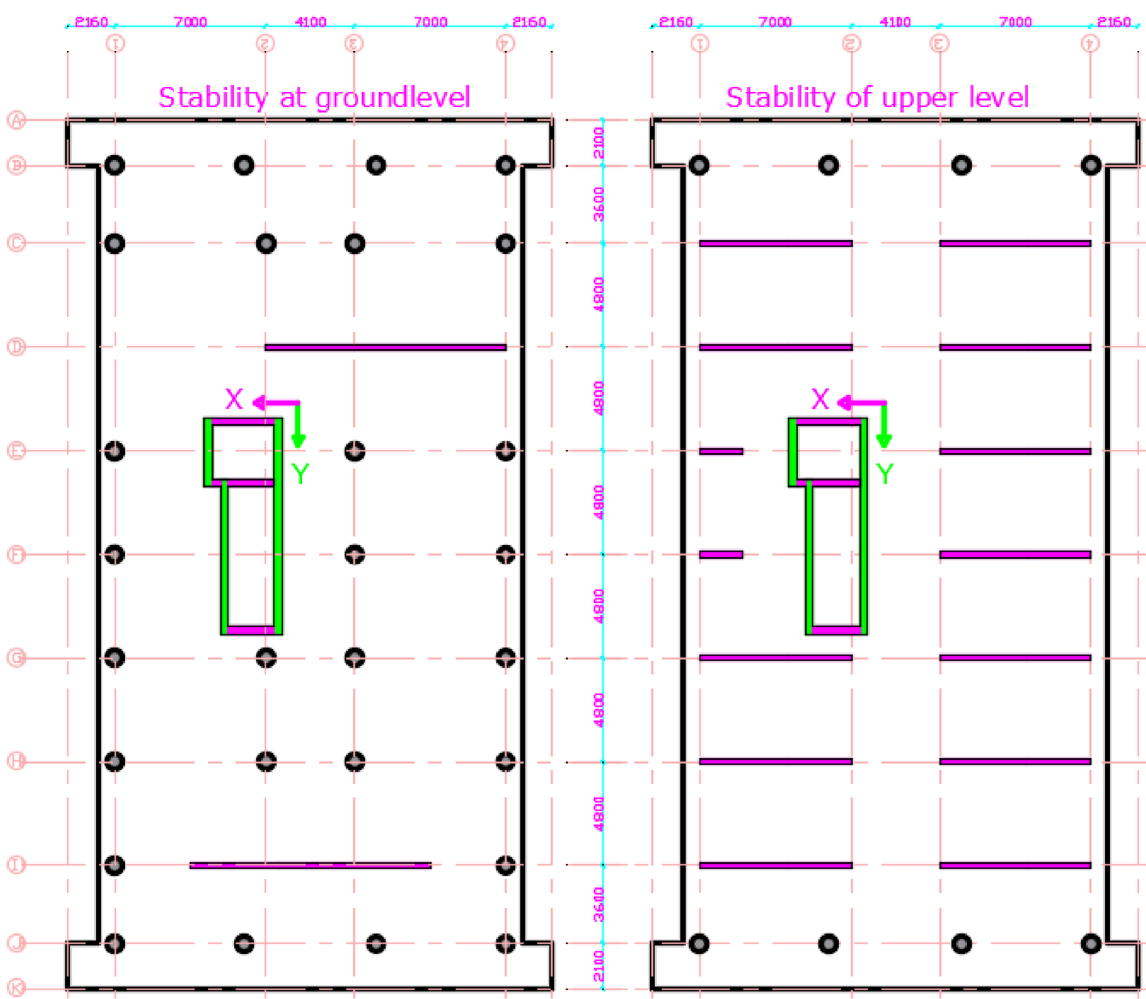


Figure 44: stability elements at ground level (left) versus stability of intermediate floor (right)

The span direction of the concrete slabs is presented in Figure 45 (left) together with the load-bearing elements. The red arrow represents the slab load direction toward the concrete walls and the yellow one represents the slab load on the lift shaft. The blue lines visualize the magnitude of the combined dead and live load from the slab vertically on the concrete walls. Given the number of concrete walls is limited on the ground floor, tension occurs under the stability walls, which will be discussed through calculation in the next paragraphs. By integrating the timber glass façade as a load-bearing element, load distribution on stability elements and the foundation piles will be more efficient. The application of a finite element program can confirm this theory and determine the required thickness of the glass façade a load-bearing member. Figure 45 (right) presents the load division once the glass façade is activated as a stability and load-bearing member. A portion of the vertical slab load will be transferred to the glass façade.

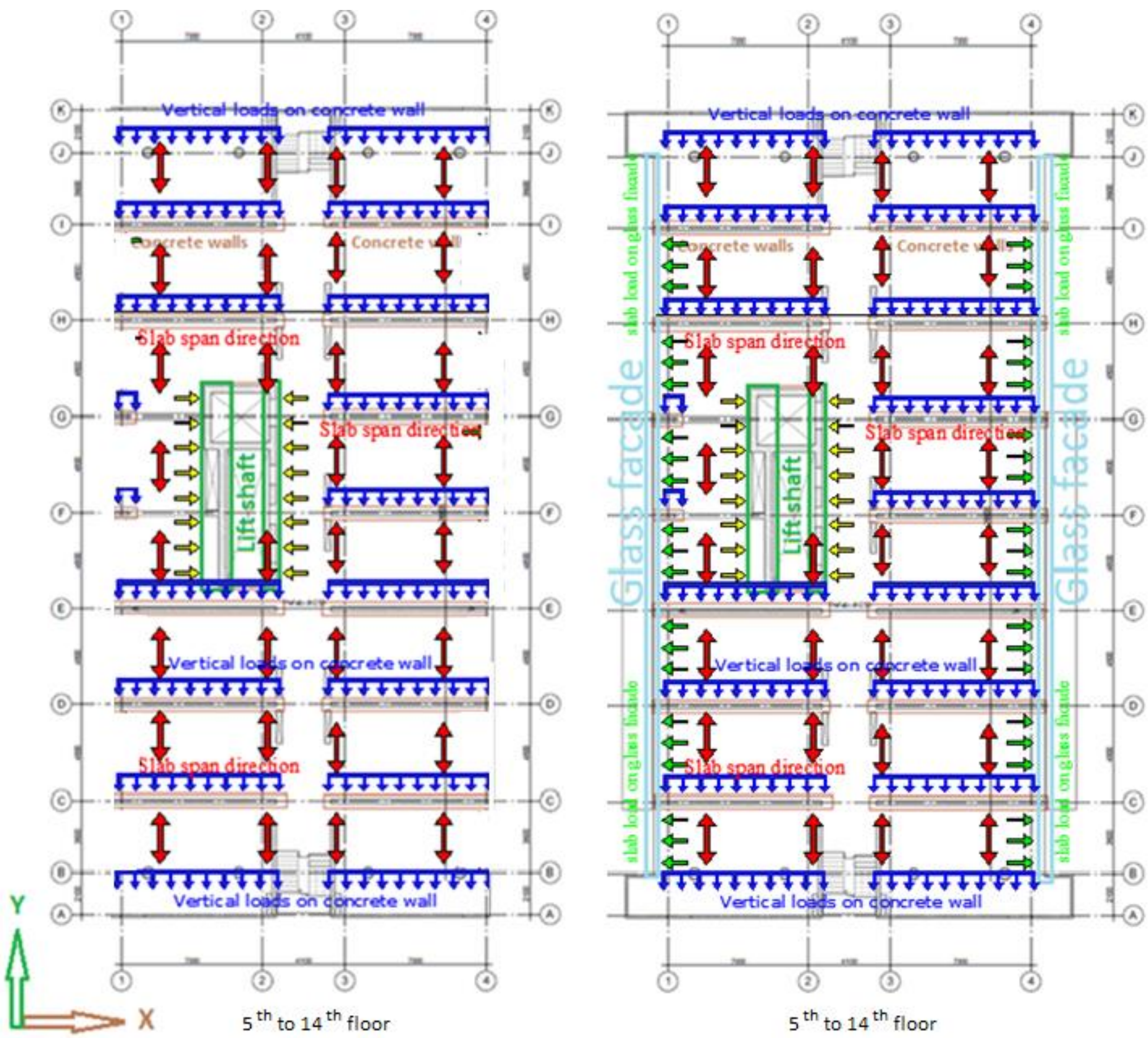


Figure 45: slab span direction on walls (left) slab direction on walls and glass façade (right)

Load-bearing capacity and the magnitude of the stress in the glass façade will be examined based on this load. The slab load on the glass façade is limited given the façade is parallel to the span direction of the slab. The slab load on the glass façade is due to the deflection of the concrete slab. The mutual vertical (live & dead) loads, and the horizontal wind load make the powerful load combination to examine the maximum stress on the glass façade. The load demonstrated in Figure 45 is explained in Table 6.

An explanation of loads applied on van der Valk hotel is demonstrated in Figure 45 and Figure 46

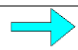
	Horizontal wind load (compression, friction, and suction) on the building structure
	Vertical dead and live loads of the slabs are transferred through concrete walls/columns to the foundation piles.
	A portion of the slab (dead + live) load transferred through the glass façade to the foundation piles.
	The reaction force of the foundation piles to the applied vertical and horizontal forces.

Table 6: definition of combined loads

5.3 Structural schemes

To enable the structural calculation and application of the structural technics, load-bearing members, and the applied load must be identified. The magnitude of the vertical load is related to the dead and live load of the structure, and the horizontal wind load is related to the location and height of the building discussed in chapter 2.2.1 and further. Figure 46 presents the combined loads working on the glass facades. The magnitude of the loads will be identified and discussed in the next paragraphs. The definition and combination of the applied loads on the façade are clarified here below.

q_{Ed} is the magnitude of the dead- and live load consisting of the weight of the concrete slabs/walls/columns and applied live load according to Eurocode-0 related to the use class of the building.

F_W is wind friction on the roof and facades, according to Eurocode-1, applied as a point load.

q_W is the horizontal wind load consisting of wind compression and suction on the building according to Eurocode 1991-1-4.

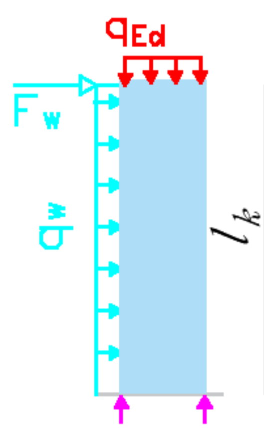


Figure 46:structural scheme of glass façade

5.4 Weight calculation

Weight calculation and load allocation to the responsible structural element is one of the fundamentals of structural assessment and designing. Once the applied load is identified, the number of load-bearing elements, location, and cross-section can be determined. Although modern state-of-the-art art programs are sufficiently equipped to determine the load division per element based on their thickness (3D designs), most experienced engineers are accustomed to executing the weight calculation manually. This provides more reliability and understanding of that specific structural element. It is also relevant to mention that the performance of weight calculation for projects such as the van der Valk hotel from the roof to the foundation piles, is quite time-consuming. Therefore, only the load on an important timber-glass façade element will be determined for further design purposes.

There are many types of loads working on the structures related to their purpose. The magnitude and type of the load are also mentioned in the Eurocode. Figure 47 presents an overview of the most common loads.

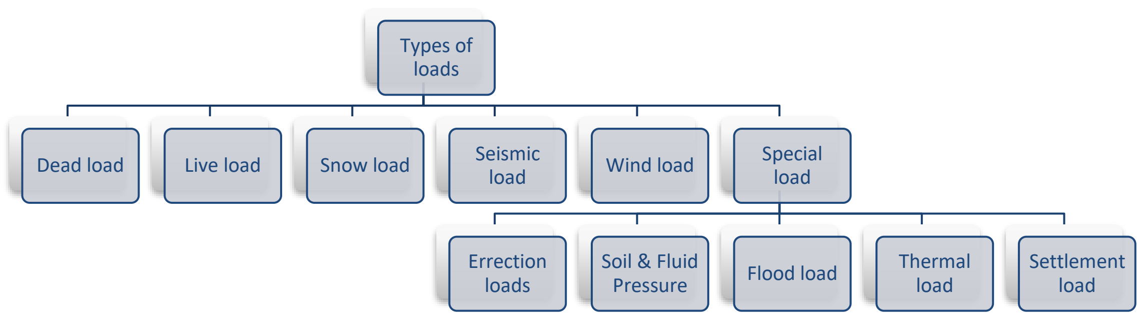


Figure 47: types of loads

However, not all the above-mentioned loads apply to every building. The types of loads are generally related to the function of the building, the climate, and the environmental impact of the situated building. The most appropriate types of loads for structural designing of the buildings in the Netherlands without considering the province of Groningen (referred as to earthquake) are presented in Figure 48.

Loads acting on structure		
	Vertical loads	- Dead load - Imposed load - Impact load
	Horizontal load	- Wind load
	Logitudinal load	- Moving load

Figure 48: a common type of load for buildings in the Netherlands

5.4.1 Vertical loads

To examine the load-bearing capacity of the timber-glass facade, the magnitude of the normal stresses is very important. Normal stress is linearly related to the immensity of the vertical dead and live load, where the dead load is related to the cross-section and mass density of the element, and the live load is related to the consequence class and function class of the structure. There are three main consequence classes underlined in the Eurocode, presented in Table 7.

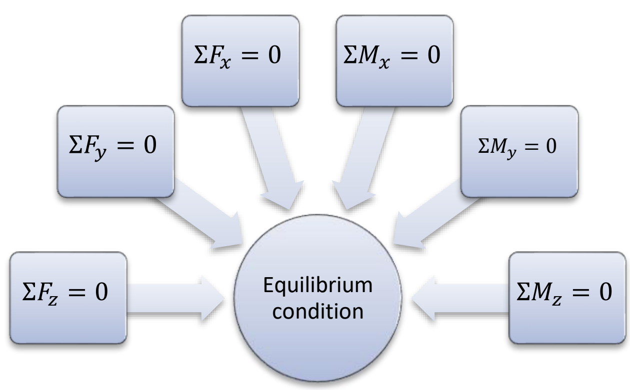
Class	Description	Examples
CC1	Low consequences for loss of human life, and economic, social, or environmental consequences are small or negligible.	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses
CC2	The consequences for loss of human life, and economic, social, or environmental consequences are considerable.	Residential and office buildings, and public buildings where failure consequences are medium (e.g. office building).
CC3	The high consequence of loss of human life, or economic, social, or environmental consequences is very great.	Grandstands are public buildings where the consequences of failure are high (e.g. concert halls.

Table 7: consequence class according to Eurocode

The reliability classes are also linked to the consequent classes and together they determine the safety factors to execute the structural design. Considering Table 7 as the point of departure, the residential buildings mentioned in chapters 3 and 4 could be subdivided into consequence class 1 and the hotel, in consequence, class 2 or 3. Given this project is already designed with consequent class 3, the research will also be executed with the same consequent class. The magnitude of the live and dead load for the van der Valk hotel is presented in Table 8.

5.4.2 Horizontal load

Wind load is generally identified as horizontal force and exercises compression, suction, and friction on buildings. Wind loads on buildings are extensively discussed in paragraphs 2.2.1 and further. The magnitude of the wind load on the timber-glass façade will be determined per façade element further in this report. For static calculation, the sum of forces must be in equilibrium.



Dead & live loads v/d Valk						
Roof	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
gravel, bituminous, isolation		0,80	-			
concrete roof	250	6,25	-			
pipes, wiring, and ceiling		0,25	-			
use category H		-	1,00	0	0	0
		7,30	1,00			
Intermediate floors	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
topping	50	1,00	-			
concrete slab	200	5,00	-			
pipes, wiring, and ceiling		0,20	-			
separation walls		-	0.80			
use category C		-	2,50	0,4	0,7	0,6
		6,20	3,30			
Ground floor	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
topping	50	1,00	-			
concrete slab	280	5,00	-			
pipes, wiring, and ceiling		0,20	-			
separation walls		-	0.80			
use category C		-	5,00	0,4	0,7	0,6
		8,20	5,80			
Basement floor	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
concrete slab	300	7,50	-			
separation walls		-	0.80			
use category F		-	2,00	0,7	0,7	0,6
		7,50	2,80			
Walls/facade	thickness	dead load	live load	instantaneous factor		
	(mm)	q_p (kN/m^2)	q_k (kN/m^2	ψ0	ψ1	ψ2
concrete walls	150-250	3,75-6,25	-			
Timber-glass facade		1,00	-			

Table 8: vertical loads on v/d Valk hotel

5.5 Load distribution

The vertical load division of the concrete slab on the lift shaft, concrete columns/walls and glass facade depend on how the concrete slab is designed. Slabs could be reinforced in one or two directions. In the original design of the van der Valk hotel, the slab was reinforced in one direction and would transfer the vertical loads mainly onto the concrete walls. The glass façade is positioned in the transversal direction and can only be loaded when there is sufficient contact between the glass façade and the concrete slab. The magnitude of the load on the glass façade will also be limited up to the immensity of the deflection of the concrete slab.

Nevertheless, the focus of this research is on the stabilization of the building by a timber-glass façade and how the horizontal wind load division is correlated to the bending stiffness of the stability elements. To elaborate on this further, a quick look at Figure 44 and Figure 45 is necessary. In the mentioned figures, the span direction of the concrete slab and the location of the load-bearing members are demonstrated. By adding the timber-glass façade as a stability element, Figure 49 could be redesigned with additional constraints representing the timber-glass façade. An overview of the adapted design including the timber glass façades is provided in Figure 49.

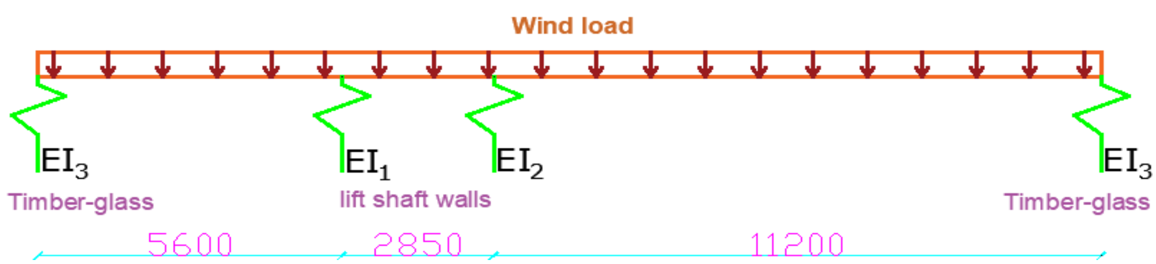


Figure 49: horizontal wind load on stability elements of vd Valk

5.5.1 Center of gravity & moment of inertia

Division of the wind load on the stability elements is related to their bending stiffness. The concrete slab is loaded in its cross-sectional direction and therefore could be assumed to be infinitely stiff. Therefore, the wind load over the height of the buildings is mainly schematized as a point/line load against the slab. The slab subsequently distributes the load over the number of stability elements based on their stiffness.

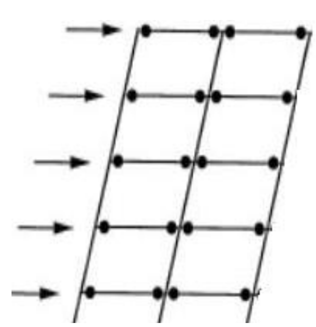


Figure 50: wind load division through slab

The building of the van der Valk hotel is mainly stabilized by a lift shaft. To determine the load division between the lift shaft and the timber-glass façade, their bending stiffness is required. The bending stiffness of a concrete lift shaft could be better determined by a rigid body. This will provide a higher bending stiffness compared to a lift shaft consisting of separate walls. The monolith concrete lift shaft elements foresee rigid bending stiffness connections will also resist the stress as a rigid body. The theory of bending stiffness and the calculation of it for various shapes are discussed in many literatures and is a foundation of civil engineering knowledge (Bai, 2014). Therefore, additional explanation on this topic will be redundant and the focus will be on the application of the existing theory in this paragraph.

Figure 51 demonstrates an overview of the lift shaft with the corresponding dimensions of the walls. Prior to the determination of the bending stiffness of the lift shaft, it's necessary to identify first the location of the central gravity. For simplicity reasons and to fasten the procedure, the reinforced concrete walls are assumed as a homogeneous material and the central gravity is determined by the following formula.

$$Z_x = \sum \left\{ \frac{a_i * b_i * z_{x,i}}{a_i * b_i} \right\} \quad ; \quad Z_y = \sum \left\{ \frac{a_i * b_i * z_{y,i}}{a_i * b_i} \right\}$$

Based on the presented overview of Figure 51, one can conclude prior to any calculation, that the central gravity is not located at the center of the lift shaft. The openings in the walls for the doors and the unsymmetrical position of the separation wall inside the lift shaft make the structure unsymmetrical. The center of gravity, the moment of inertia, and the moment of resistance will be calculated by the rule of Steiner. The concrete structure is mainly loaded by compression and therefore, the magnitude of the reinforcement is limited up to the practical amount of rebar. To speed up the calculation, the reinforcement bars are left out of the calculation for the determination of the bending stiffness. The concrete elements are assumed to be homogeneous materials. Including the reinforcement, rebars will only increase the bending stiffness of the elevator shaft, which is a safe assumption and will be determined under FE analysis

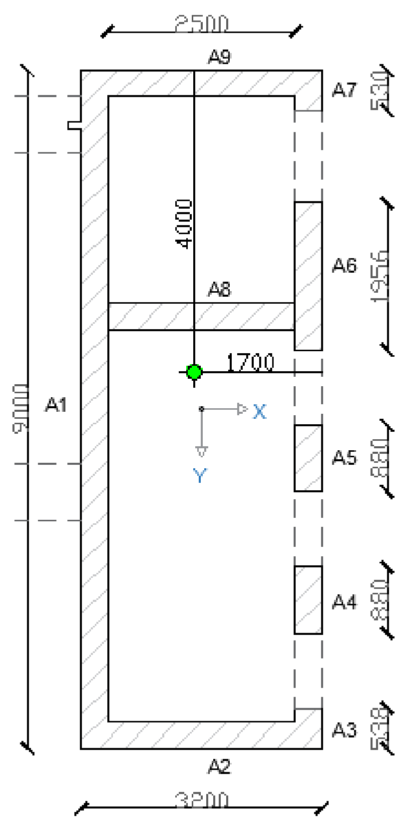


Figure 51: central gravity of lift shaft

The general formula to determine the bending stiffness of a composed structure is presented below. Based on the calculated bending stiffness of the structure and the corresponding center of gravity, the moment of resistance can be calculated.

$$I_y = \sum \left\{ I_{y,i} + a_i * b_i \left(\left| z_y - z_{y,i} \right|^2 \right) \right\} \qquad \& \qquad W_y = \left(\frac{I_y}{Z_{y,max}} \right)$$

To prevent a mass of paperwork, the general formulas have been integrated into the excel sheet and the required outcomes are demonstrated in Table 9. The center of gravity is shown with a green dot in Figure 51. The coordinate in X-direction is about 1700 mm and in the Y-direction 4000 mm concerning the top left corner.

$$center\ of\ gravity\ in\ X-direction \quad Z_x = \sum \left\{ \frac{a_i * b_i * z_{x,i}}{a_i * b_i} \right\}$$

	A1			A2			A3			A4			A5			A6			A7			A8			A9			Σ
	a_1	b_1	z_{x1}	a_2	b_2	z_{x2}	a_3	b_3	z_{x3}	a_4	b_4	z_{x4}	a_5	b_5	z_{x5}	a_6	b_6	z_{x6}	a_7	b_7	z_{x7}	a_8	b_8	z_{x8}	a_9	b_{10}	z_{x9}	
	9	0,35	3	2,5	0,35	1,4	0,5	0,35	0,2	0,9	0,35	0,2	0,9	0,35	0,2	2	0,35	0,2	2,5	0,35	1,4	2,5	0,35	0,2	2,5	0,35	1,4	
$a_i*b_i*z_{x,i}$ =	9,5			1,2			0			0,1			0,1			0,1			1,2			0,2			1,2			12,4
$A = a_i*b_i$	3,2			0,9			0,2			0,3			0,3			0,7			0,9			0,9			0,9			7,26
Z_{x1} =	1,7 m			1,7			1,7			1,7			1,7			1,7			1,7			1,7			1,7			
Z_{x2} =	0,8 m																											

$$I_x = \sum \left\{ I_{x,i} + a_i + b_i \left(\left| z_x - z_{x,i} \right|^2 \right) \right\} \ \& \ W_x = \left(\frac{I_x}{z_{x,max}} \right)$$

	A1			A2			A3			A4			A5			A6			A7			A8			A9			Σ
	$I_{x,1}$	A_1	$\|z_1\|^2$	$Ix_{,2}$	A_2	$\|z_2\|^2$	$I_{x,3}$	A_3	$\|z_3\|^2$	$I_{x,4}$	A_4	$\|z_4\|^2$	$I_{x,5}$	A_5	$\|z_5\|^2$	$I_{x,6}$	A_6	$\|z_6\|^2$	$I_{x,7}$	A_7	$\|z_7\|^2$	$I_{x,8}$	A_8	$\|z_8\|^2$	$I_{x,9}$	A_9	$\|z_9\|^2$	
	0	3,15	9,9	0,5	0,88	0,3	0	0,19	0	0	0,88	0	0	0,88	0	0	1,96	0,3	0,5	2,5	0,3	0	2,5	0,5	0,5	0,88	0,3	
$I_{x,i}$	31			0,7			0			0,3			0,3			0,7			0,9			0,9			0,7			35,8
I_x =	36 m^4																											
W_x =	21 m^3																										$Z_{x,max}$	1,7

$$center\ of\ gravity\ in\ Y-direction{:} \quad Z_y = \sum \left\{ \frac{a_i * b_i * z_{y,i}}{a_i * b_i} \right\}$$

	A1			A2			A3			A4			A5			A6			A7			A8			A9			Σ
	a_1	b_1	z_{y1}	a_2	b_2	z_{y2}	a_3	b_3	z_{y3}	a_4	b_4	z_{y4}	a_5	b_5	z_{y5}	a_6	b_6	z_{y6}	a_7	b_7	z_{y7}	a_8	b_8	z_{y8}	a_9	b_{10}	z_{y9}	
	9	0,35	4,5	2,5	0,35	8,8	0,5	0,35	8,7	0,9	0,35	7	0,9	0,35	5,2	2	0,35	2,8	2,5	0,35	3	2,5	0,35	0,3	2,5	0,35	0,2	
$a_i*b_i*z_{y,i}$ =	14			7,7			1,6			2,2			1,6			1,9			2,6			0,2			0,2			32,2
$A = a_i*b_i$	3,2			0,9			0,2			0,3			0,3			0,7			0,9			0,9			0,9			8,14
Z_{y1} =	4,0 m			4			4			4			4			4			4			4			4			
Z_{y2} =	5,0 m																											

$$I_y = \sum \left\{ I_{y,i} + a_i + b_i \left(\left| z_y - z_{y,i} \right|^2 \right) \right\} \ \& \ W_y = \left(\frac{I_y}{z_{y,max}} \right)$$

	A1			A2			A3			A4			A5			A6			A7			A8			A9			Σ
	$I_{y,1}$	A_1	$\|z_1\|^2$	$I_{y,2}$	A_2	$\|z_2\|^2$	$I_{y,3}$	A_3	$\|z_3\|^2$	$I_{y,4}$	A_4	$\|z_4\|^2$	$I_{y,5}$	A_5	$\|z_5\|^2$	$I_{y,6}$	A_6	$\|z_6\|^2$	$I_{y,7}$	A_7	$\|z_7\|^2$	$I_{y,8}$	A_8	$\|z_8\|^2$	$I_{y,9}$	A_9	$\|z_9\|^2$	
	21	3,2	0,3	0,5	0,9	24	0	0,2	23	0	0,3	9,5	0	0,3	1,5	0,2	0,7	1,4	0,5	0,9	0,9	0,5	0,9	14	0,5	0,9	14	
$I_{y,i}$	22			21			4,2			2,9			0,5			1,2			1,2			12			13			78,8
I_y =	79 m^4																											
W_y =	16 m^3																										$Z_{y,max}$	5,0

Table 9: center of gravity, the moment of inertia, and the moment of resistance

5.5.2 Bending stiffness of the elevator shaft

The bending stiffness of the stability elements is a very crucial requirement to determine wind load division per stability element. The stiffer the element, the higher is the portion of the wind load allocated. The bending stiffness (EI) depends on the elasticity modulus of the material and the moment of inertia of the stability elements. The determination of an accurate young's modulus of a cracked (nonlinear) reinforced concrete structure loaded on bending is quite complex. However, various literatures are recommending the application of table 15 of NEN 6720 (Braam, 2012) or the fictive modulus of elasticity (E_f) value of table 3.1 mentioned in VBC (Dongen, 2003). For efficiency reasons, many engineers in practice are also applying the practical estimation value of $E_f = \frac{1}{3}E_c$ (R.Sagel, 2004), which is a good approximation. There is also a lot of structural software equipped to calculate the bending stiffness through $MN\kappa$ diagram, which will provide similar results.

To demonstrate the calculation of the elastic modulus of the cracked concrete, the theoretical approach according to the Dutch book is applied (GTB, 2010-9.1). This approach is based on a combination of various concrete lab tests and theoretical knowledge of the concrete material (Betonvereniging, 2013).

The general formula is;

$$E_f = 1960 + 432000\rho + (20000 - 196000\rho)\alpha_n \geq 4450$$

$$\rho = \frac{A_{s,x} + A_{s,y}}{A_c} \quad \text{and} \quad \alpha_n = \frac{N_{Ed}}{\left[f_{cd}A_C + \left(A_{s,x} + A_{s,y}\right)f_{yd}\right]}$$

<u>Where:</u>

- A_s $\quad$ $steel\ reinforcement\ surface$
- A_c $\quad$ $concrete\ surface$
- f_{cd} $\quad$ $concrete\ compression\ strength$
- f_{yd} $\quad$ $yield\ strength\ of\ steel\ reinforcement$
- N_{Ed} $\quad$ $normal\ force$
- E_c $\quad$ $modulus\ of\ elasticity\ of\ uncracked\ concrete$
- E_f $\quad$ $modulus\ of\ elasticity\ of\ cracked\ concrete$
- EI $\quad$ $bending\ stiffness$
- ρ $\quad$ $ratio\ between\ reinforcement\ steel\ and\ concrete\ surface$

The mentioned parameters are required to accomplish the calculation. To not wander away from the main topic, which is focused on glass structure and not on a concrete structure, the values are determined roughly. However, the values are accurate enough to provide reliable results for the conclusion. The output will also be compared with the finite element calculation, which is more precise.

- $N_{Ed} = 1.88 * 10^7\ kN$
- $A_{s,x} = A_{s,y} = 8635\ mm^2\ which\ is\ \emptyset_{10} - 150\ mm\ in\ both\ direction$
- $A_c \quad = 8.93E06\ mm^2$
- $f_{cd} \quad = 20 \frac{N}{mm^2}\ for\ C30/37$
- $f_{yd} \quad = 435 \frac{N}{mm^2}$

Based on the discussed formula mentioned in the GTB and the identified values for various parameters, the young's modulus for the cracked concrete can be calculated here;

$$\rho_{elevator} = \frac{A_{s,x} + A_{s,y}}{A_c} = \frac{8635 + 8635}{8.93 * 10^6} = 0.0019$$

$$\alpha_{n-elevator} = \frac{N_{Ed}}{\left[f_{cd}A_C + \left(A_{s,x} + A_{s,y}\right)f_{yd}\right]} = \frac{1.88 * 10^7}{[20 * 8.93E06 + (8635 + 8635)435]} = 0.101$$

$$E_{f.shaft} = 1960 + 432000 * 0.0019 + (20000 - 196000 * 0.0019)0.101 = 4781 \geq 4450$$

With this young's modulus value of the cracked concrete in hand, the bending stiffness of the elevator shaft could be calculated to proceed further with wind load division per stability element. The moment of inertia for the elevator shaft in Y-direction is already determined in Table 9. Multiplication of young's modulus and the moment of inertia provides the final value of the cracked bending stiffness of the elevator shaft.

$$EI_{f.shaft} = 4781 * 7{,}9 * 10^{13} = 3{,}78 * 10^{17} Nmm^2$$

To determine the load division between the glass façade and the concrete elevator shaft, the bending stiffness of the glass façade is required as well. Given the elements are positioned parallel to each other in the Y-direction (Figure 45), the bending stiffness of the glass façade will also be determined only in the Y-direction.

5.5.3 Tension in the foundation

To understand the importance and contribution of the timber-glass façade in this design, the first examination will be about tension in the foundation piles without a timber-glass façade. This will be a rough calculation given that the load division of the slabs on the lift shaft will be determined based on reasonable assumptions, which are conservative and safe. The slabs are spanned parallel to the lift shaft, and the structure is statically undetermined. Therefore, the result of the manual calculation will be different compared to the FE program. Figure 52 demonstrates the load division per unit load on lift-shaft walls.

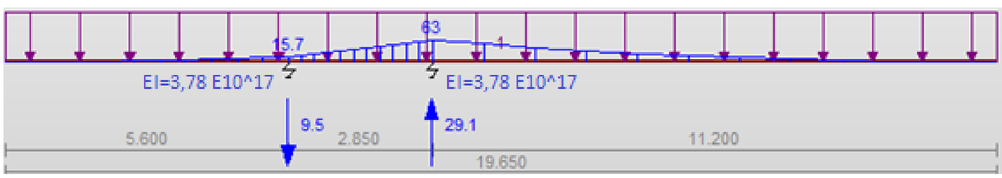


Figure 52: load division on lift-shaft based on 1-unit load

To examine whether tension occurs in the foundation piles, the magnitude of the dead load and wind load is calculated in Table 10. The sum of the loads must be more significant everywhere than zero to prevent tension in the foundation. If this is not the case, then tension will occur in the foundation. Figure 53 confirms tension in the foundation.

On lift shaft									
	q_G	q_Q	w	l	h	n	G_k	Q_k	
roof	7,3	-	0,5	-	-	1,0	4,0	-	kN/m
lift-shaft wall	3,8	-	-	-	3,0	19,0	214	-	kN/m
floors	6,2	-	0,5	-	-	19,0	59	-	kN/m
							276	0,0	kN/m
$F_{wind,\ compression\ +suction}$	-	1,8	19.7	-	60,0	-	-	2164	kN
$F_{wind,\ friction}$	-	0,1	19,7	-	60,0	-	-	65	kN
							0	2229	kN

Table 10: loads on the lift-shaft

$$q_{Ed} = \gamma * q_G + \gamma * q_{wind} \rightarrow 0.9 * 276 - 1.65 * \frac{2229}{9.0} * \frac{29.10}{19.65} = -605 \frac{kN}{m}$$

Figure 53: tension in the foundation due to wind

5.6 BENDING STIFFNESS OF GLASS FAÇADE

To compensate for tension in the foundations, the available glass façade will be activated in this paragraph. Figure 44 and Figure 45 demonstrate the location and length of the applied glass façade. Given that the load will be applied as a concentrated point load (see also paragraph 3.2), a selected length of the glass façade will be encountered as a stability element. The timber frame around the glass façade is, compared to the surface of the glass façade, quite small. For this reason, the timber frame's participation in stabilizing the building is negligible. However, the contribution of the timber frame and the stress climax under the point load will be studied under finite element analysis.

The bending stiffness of the concrete lift shaft is already calculated, and to determine the load division between the glass façade and the concrete lift shaft, the bending stiffness of the glass façade is required. With this in mind, the moment of inertia and the bending stiffness of the glass façade can be determined. The minimum length of the applied glass façade has the following dimensions/properties;

- *T = 5*12=60 mm thickness of the glass façade*
- *E = 70.000 N/mm² young's modulus*
- *L = L_t=3.6m length is applied*

The bending stiffness of a single glass pane with a length of 3.6 m is calculated here.

$$I_y = \frac{1}{12} * tl^3 \rightarrow \frac{1}{12} * 60 * 3600^3 = 2.33 * 10^{11} mm^4$$

$$EI_{glass} = 70.000 * 2.33 * 10^{11} = 16.3 * 10^{15} Nmm^2$$

5.6.1 WIND LOAD ON GLASS FAÇADE

The wind load division can be determined based on the bending stiffness of the elevator shaft and glass façade. Computer software is applied to demonstrate the correlation between the bending stiffness and load division on the stability elements. Figure 54 presents the reaction forces per stability element. The figure presents three scenarios for different bending stiffnesses. The bending stiffness of the elevator shaft in the first and second scenarios is constant. The bending stiffness of the glass façade (the constraints at the corners) is changed to express the correlation between different bending stiffnesses. By comparing the outcomes, one can conclude that, beyond a certain level, the bending stiffness differences do not cause any deviation in the distribution of the loads anymore. The final load division with solid and rigid constraints per stability elements is presented in Figure 54.

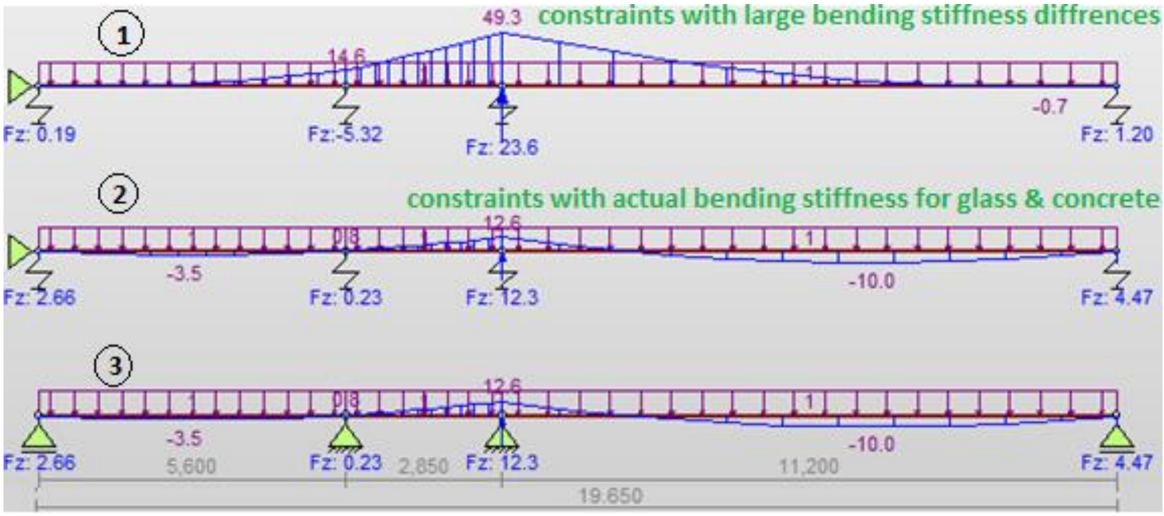


Figure 54: load division and bending stiffness of constraints

The magnitude of the vertical load on the lift-shaft is already calculated in paragraph 5.5.3. After activation of the glass façade as stability element, the wind load division per unit load is determined and presented in Figure 54. The tension on the foundation piles could be reexamined with redistributed wind load.

$$q_{Ed} = \gamma * q_G + \gamma * q_{wind} \rightarrow 0.9 * 276 - 1.65 * \frac{2229kN}{9.0m} * \frac{12.3}{19.65} = -7.0\ \frac{kN}{m}$$

By comparing the result of Figure 53 with Figure 55, one can conclude that the magnitude of tension on the foundation piles is significantly reduced. The reduction of tension on piles has confirmed the assumption due to the activation of a single glass pane at both sides of the building parallel to the lift shaft in the Y-direction. However, there is still tension in piles. To eliminate and prevent tension on piles, several glass façades must be activated. This will be examined with the application of the FE program. FE-analysis will also provide detailed information about stress distribution on the glass façade and the required thickness.

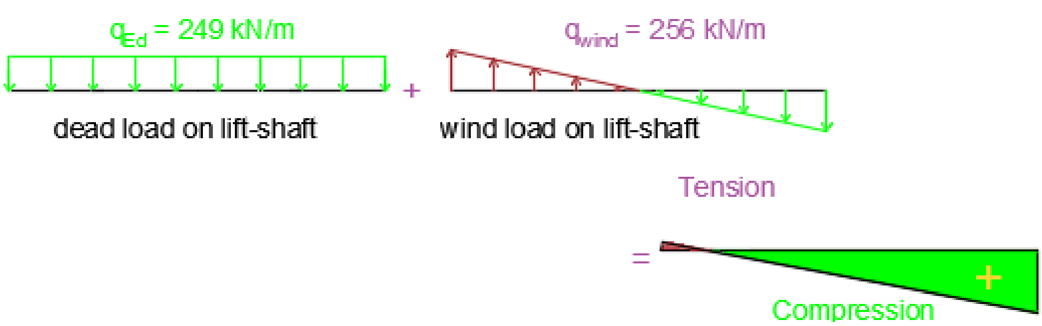


Figure 55: reduction of tension in the foundation after application of glass facade

5.7 FINITE ELEMENT ANALYSIS OF V/D VALK

A finite element program is applied to enlarge the reliability and accuracy of this research. By designing the structure of the van der Valk hotel as a 3D model, the load distribution over the stability elements will be precisely calculated. The location of the stability elements and the magnitude of the dead load for the van der Valk building has already been discussed and determined under manual calculation. A finite element analysis is necessary to understand the behavior of the glass structure under combined load and the exact division of the load in a 3D model based on the bending stiffness of the stability elements. Finite element analysis will demonstrate, besides load distribution, the exact location and magnitude of the tension on piles to take countermeasures. Another advantage of the finite element design is to study the behavior and deformation of the entire building, which is significant for the glass structure. The Eurocode requires more accuracy and examination of various combined loads on structural elements, like flexural buckling and lateral torsional buckling on beams (Eurocode, 2022). Examining the torsional buckling of a column loaded by normal force and bending in X- and Y-direction to design the cross section is another example of combined loading, which is very time-consuming to execute manually but straightforward and possible with a finite element program.

The invention and exploration of the latest design techniques, like parametric design combined with finite element analysis, enabled structural designers to go beyond standard calculations and shapes. With FE-modeling, the engineers could follow and present the load transfer and identify the local stress peaks. With this information in mind, the structure of case study-3 will be designed with a finite element program (AxisVM) to identify the load transfer through a timber-glass façade. The finite element analysis result will be compared with manual calculation and equipped with necessary findings and discussion.

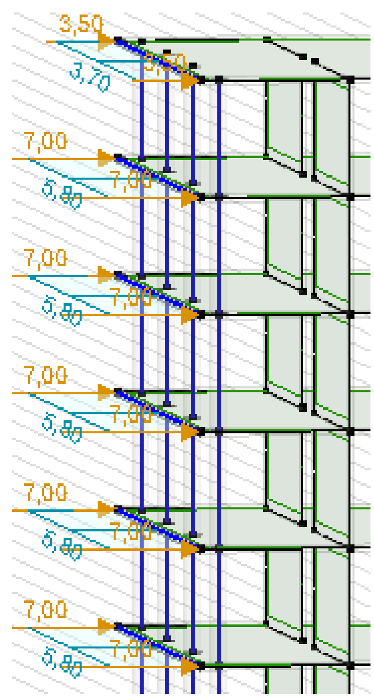
5.7.1 Finite element design

The discussed boundary conditions and determined values of the previous paragraphs are used as the point of departure to design the hybrid structure of the van der Valk hotel. To determine the required thickness of the glass façade, the timber-glass façade is simplified. In the preliminary design stage, only a glass façade is applied in the FE program. Once the exact location and required number of glass façades are identified to reduce/eliminate tension in the foundation, the timber-glass façade will be designed with a single model. This model will only be focused on stress distribution in the timber frame and glass pane.

Glass structures can resist compression stresses very well. In an ideal structural design, tension stresses should be avoided, which is not always possible. The connection of the glass pane with the surrounding elements is quite a complex problem when the connection is loaded by shear and tension stresses. Therefore, the aim is to avoid tension in the foundations under the glass façade. To examine this, the FEM program will determine the portion of the exact wind load on the glass façade. The following inputs are essential to model the building with the FE program, which has been discussed already.

- Height of the building — 60 m
- Length — 40,2 m
- Width — 22.42 m
- Wind area — II, unbuilt
- $q_P(z)$ — 1.45 kN/m^2
- C_sC_d — 0.95 (based on NEN-EN 1991-1-4)
- C_{pe} — (0.8+0.6) 1.4

Wind compression and suction will be applied as a line load against the concrete slab. The magnitude of the wind compression and suction per floor level of 3m height is calculated here;



$$q_{w,comp+suc} = C_S C_d * C_{pe} * q_P(z) * h$$
$$= 0.95 * 1.4 * 1.45 * 3.0 = 5.8 \frac{kN}{m}$$

There is also wind friction parallel to the building alongside the façade. NEN-EN specifies the friction coefficient for various types of surfaces. However, for this research, the most unfavorable friction coefficient is considered. Friction is applied as point load at the corner of each floor.

$$F_{w,friction} = C_{fr} * q_P(z) * A_{fr} = 0.04 * 1.45 * 40.2 * 3$$
$$= 7{,}0\ kN$$

Wind friction at the roof according to NEN-EN 1991-1-4:2005 is encountered as well.

$$q_{w,com+suc} + q_{roof} = C_S C_d * C_{pe} * q_P(z) * \frac{1}{2} h + f_r * q_P(z) \frac{1}{3} l \rightarrow$$
$$\rightarrow\ 0.95 * 1.4 * 1.45 * \frac{3}{2} + 0.04 * 1.45 * \frac{40.2}{3} = 3.7 \frac{kN}{m}$$

Figure 56 presents the van der Valk hotel's finite element (3D) model with the applied wind load in a Y-direction. The software determines the dead load based on the element's thickness. The dimension of the concrete walls, columns, and the thickness of the glass inclusive live load is also inserted, as discussed in paragraph 5.6 and determined in Table 8.

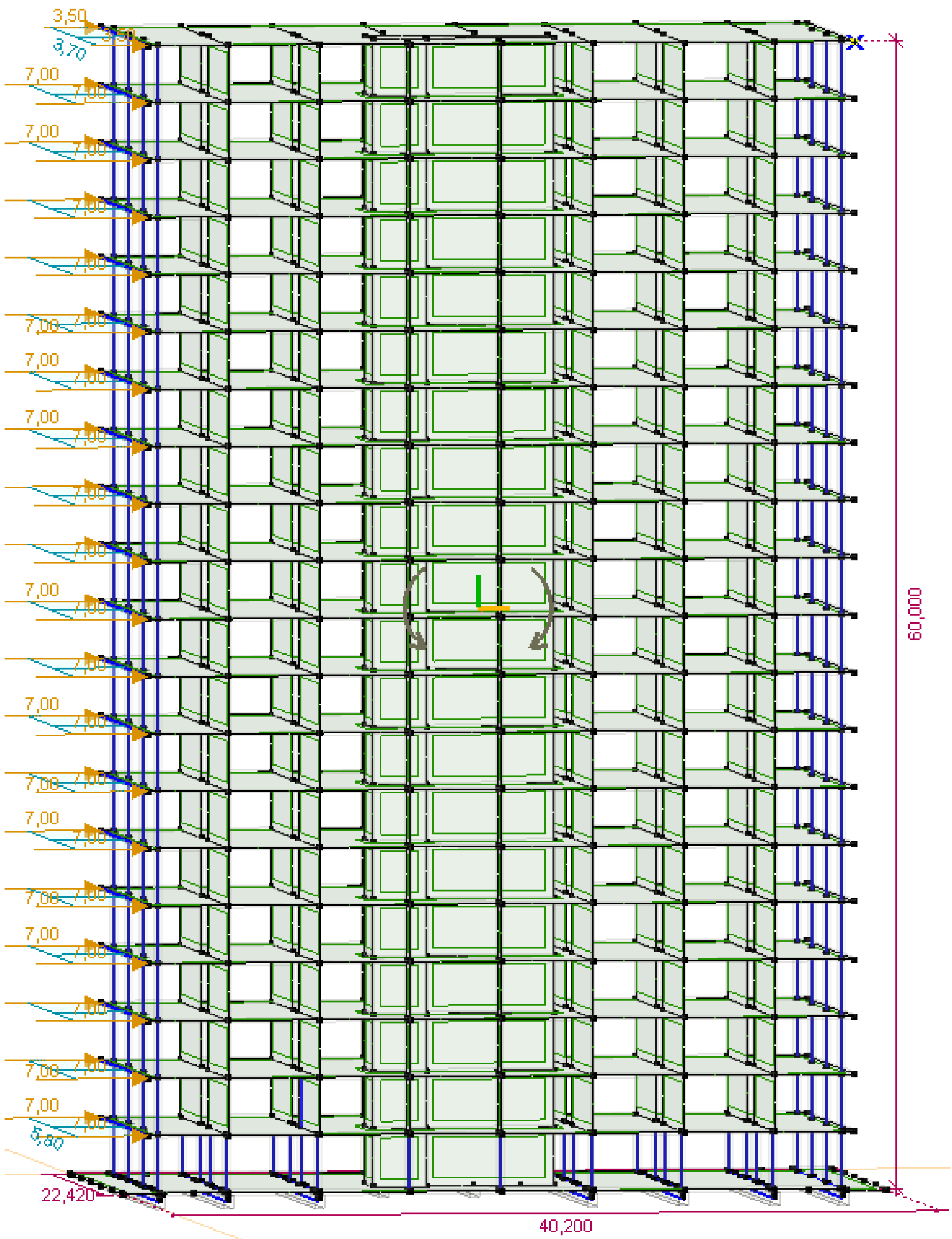


Figure 56: FE-model of van der Valk

5.7.2 Reaction forces on foundation piles

Figure 57 is obtained after the completion of the finite element model. The reaction forces are determined, and there is tension in the foundation piles. The location of the piles with tension is circled and located between canvas G-H, according to the expectation. These piles are positioned under the lift shaft resisting the vertical and horizontal (wind) loads.

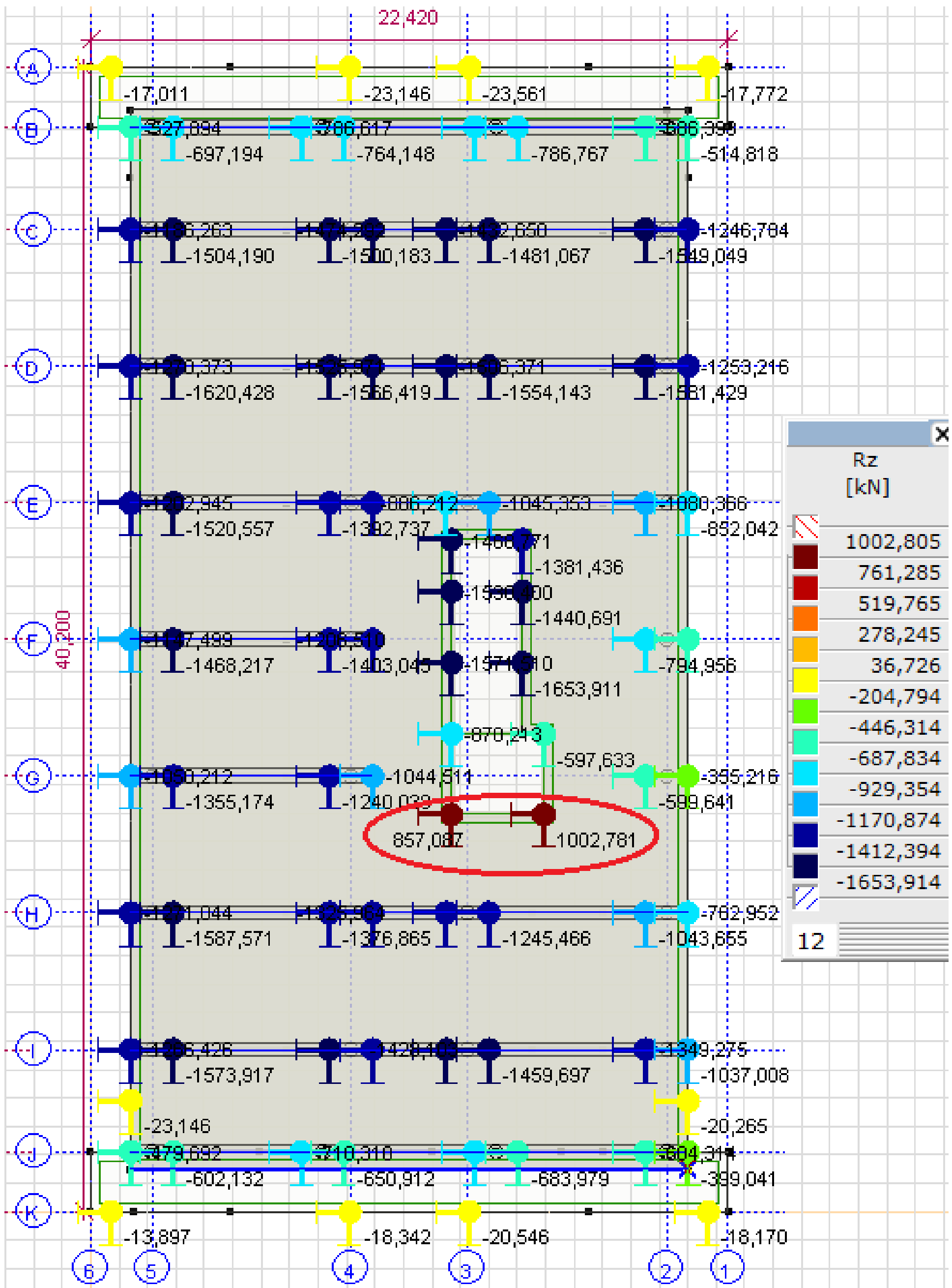


Figure 57: tension in foundation piles

5.7.3 Deformation

The layout and number of stability elements in X- and Y-directions at the ground level are discussed in paragraph 5.2 and presented in Figure 44. The building is stabilized in a Y-direction, mainly by a lift shaft, confirmed through the FE model presented in Figure 58. After activating the glass façade, the deformation will be studied again to underline the effect of the glass façade on horizontal deformation. The vertical deformation of the slabs also conforms to expectations. Due to the thin cross-section of the concrete columns, their contribution to the reduction of deformation is minimal.

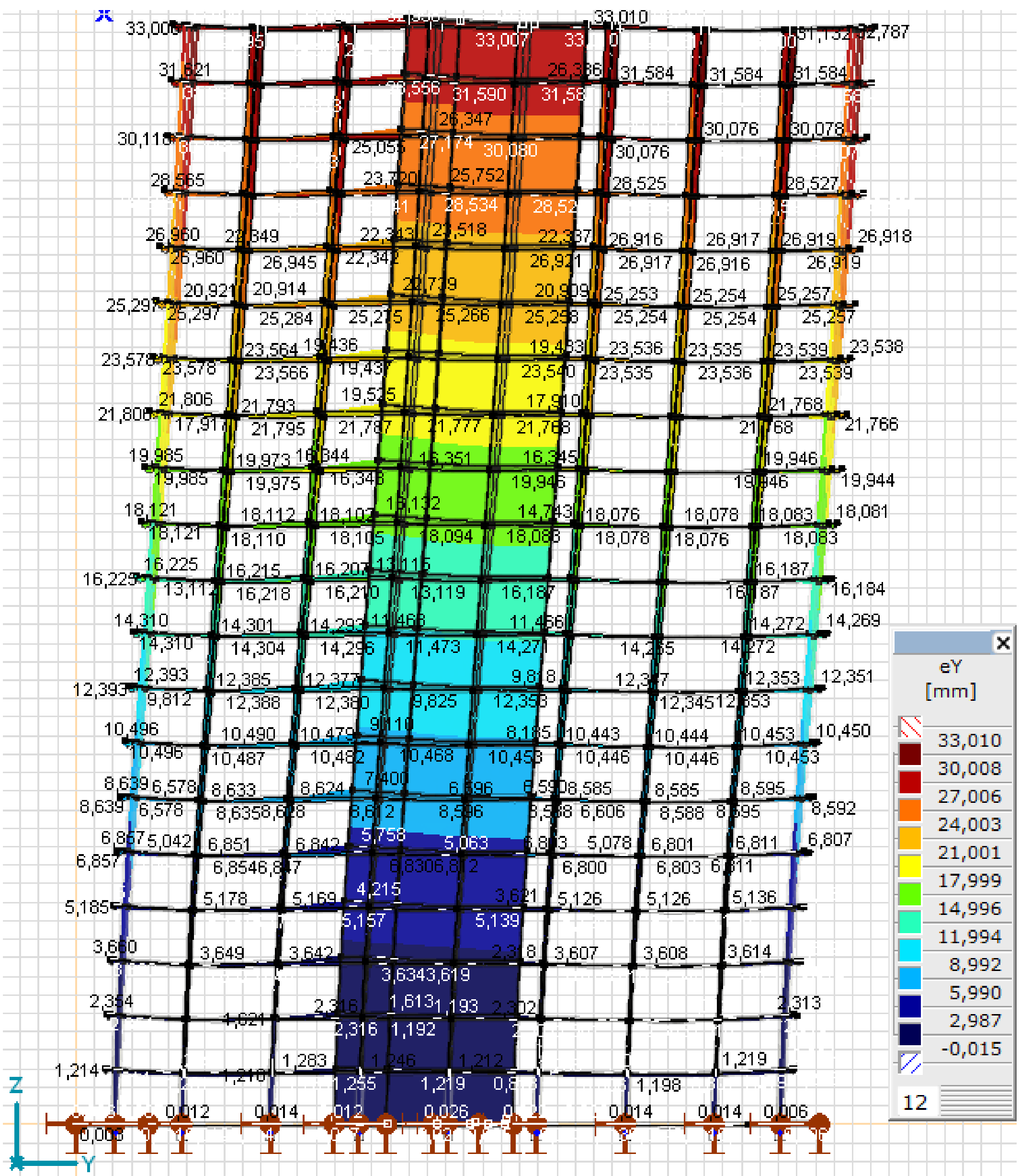


Figure 58: deformation of v/d Valk in the Y-direction

5.8 ACTIVATION OF GLASS FAÇADE

Activation of the glass facade as a stability element is the primary goal of this research. After studying the structure of van der Valk from various perspectives and discussing the tension in the foundations and deformation of the structure, it is now time to add/activate the glass façade as a stability element.

Figure 59 demonstrates the exact location and length of the glass façade. The glass façade between canvas B-C, H-I, and I-J is sufficient to minimize tension in the foundation. The position required length, and thickness of the glass façade is determined after various trials and errors. The aim was to eliminate/minimize tension at the foundation, which was a challenge. To prevent torsion of the building, the glass façade parallel to the lift shaft on canvases 2 and 5 is activated. The remaining glass façade between canvas C-H is not used as a structural or stability element. These façades could be used to enable an opening for the entrance or windows. The glass elements used for stability are unsuitable for any opening and are rigidly connected to the floors, which will be discussed later.

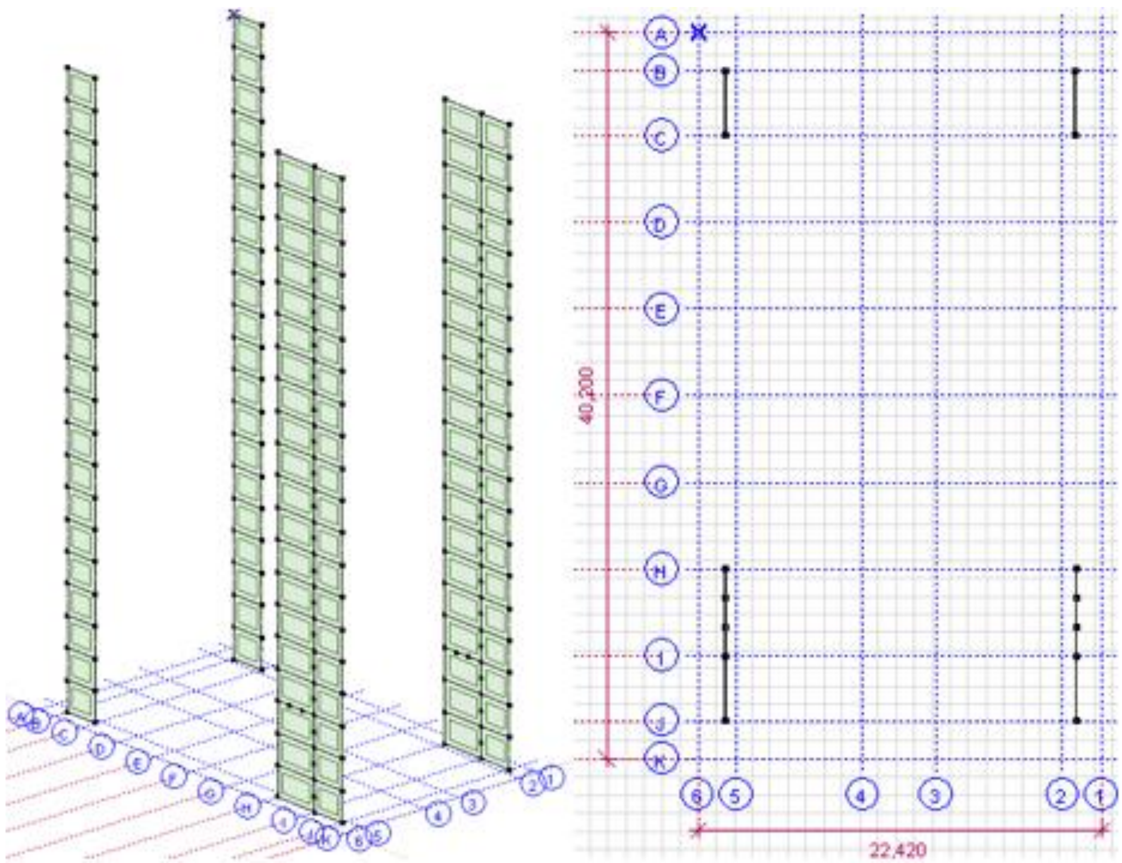


Figure 59: position of glass façade

5.8.1 Reaction forces after activation of the glass facade

The purpose of activating the glass façade in this design is to prevent tension in the foundation, make additional measurements redundant, save costs, and prove that glass facades are suitable for application as load-bearing and stability elements. Reaction forces and tension in the foundation piles are already discussed in paragraph 5.7.2. Figure 60 presents the reaction forces in the foundation piles after activating the glass façade discussed in paragraph 5.8. Comparing the two layouts, one can conclude that tension in the foundation piles is neglectable low (6.6 vs. 1002,8 kN).

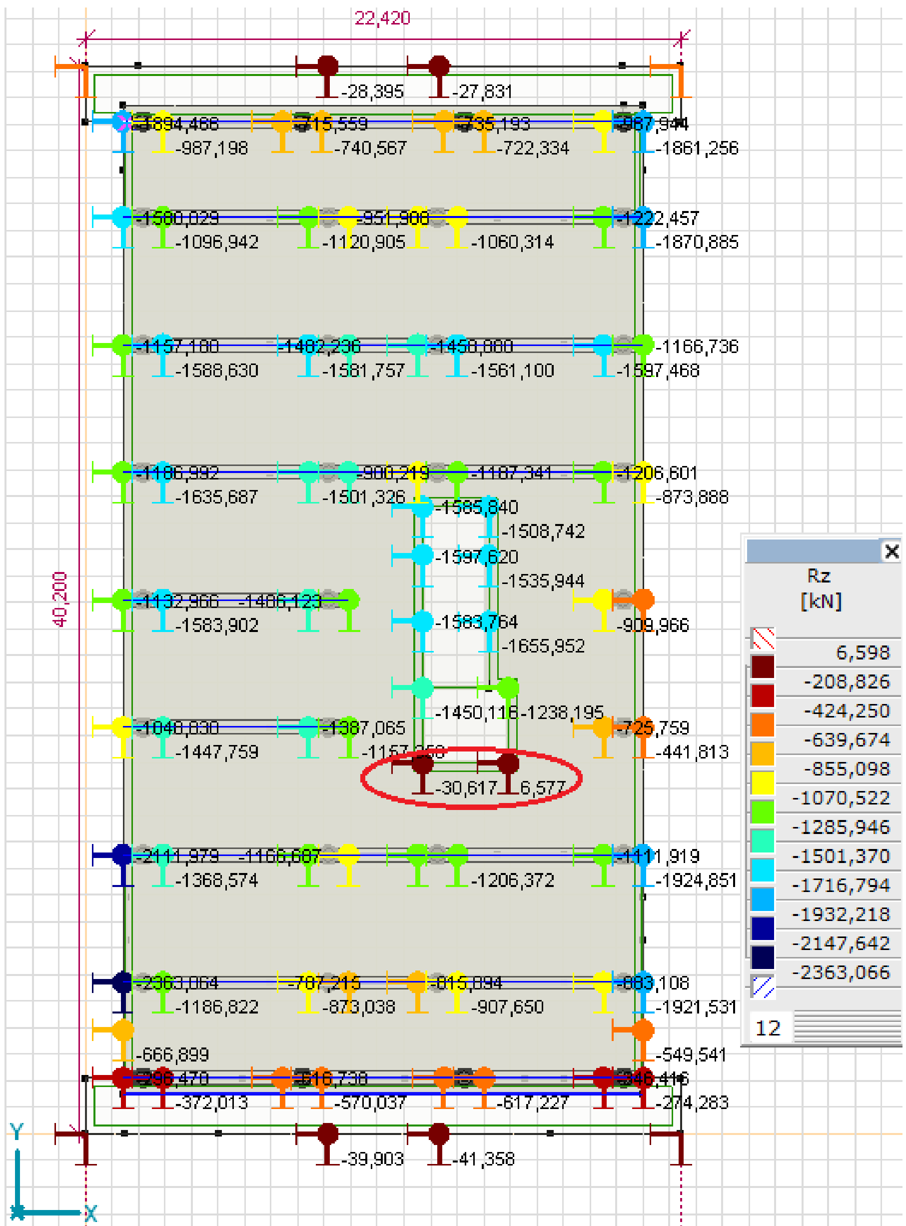


Figure 60: reaction forces of the foundation piles after activation of the glass facade

5.8.2 Deformation with glass facade

Figure 61 demonstrates the horizontal deformation of the building after activating the glass façade. Comparing this model with Figure 58, one can conclude that deformation in the Y-direction is reduced by about 30%. The structure of van der Valk has become more rigid. Since glass is a brittle material, the magnitude of the horizontal deformation should be limited and resistible. The immensity of the deformation per glass façade must be analyzed to prevent sudden failure. Glass is not as elastic as a timber frame; therefore, the deformation per glass pane should be accommodated through a timber frame connection. The following paragraphs will study stress distribution and horizontal deformation per glass pane.

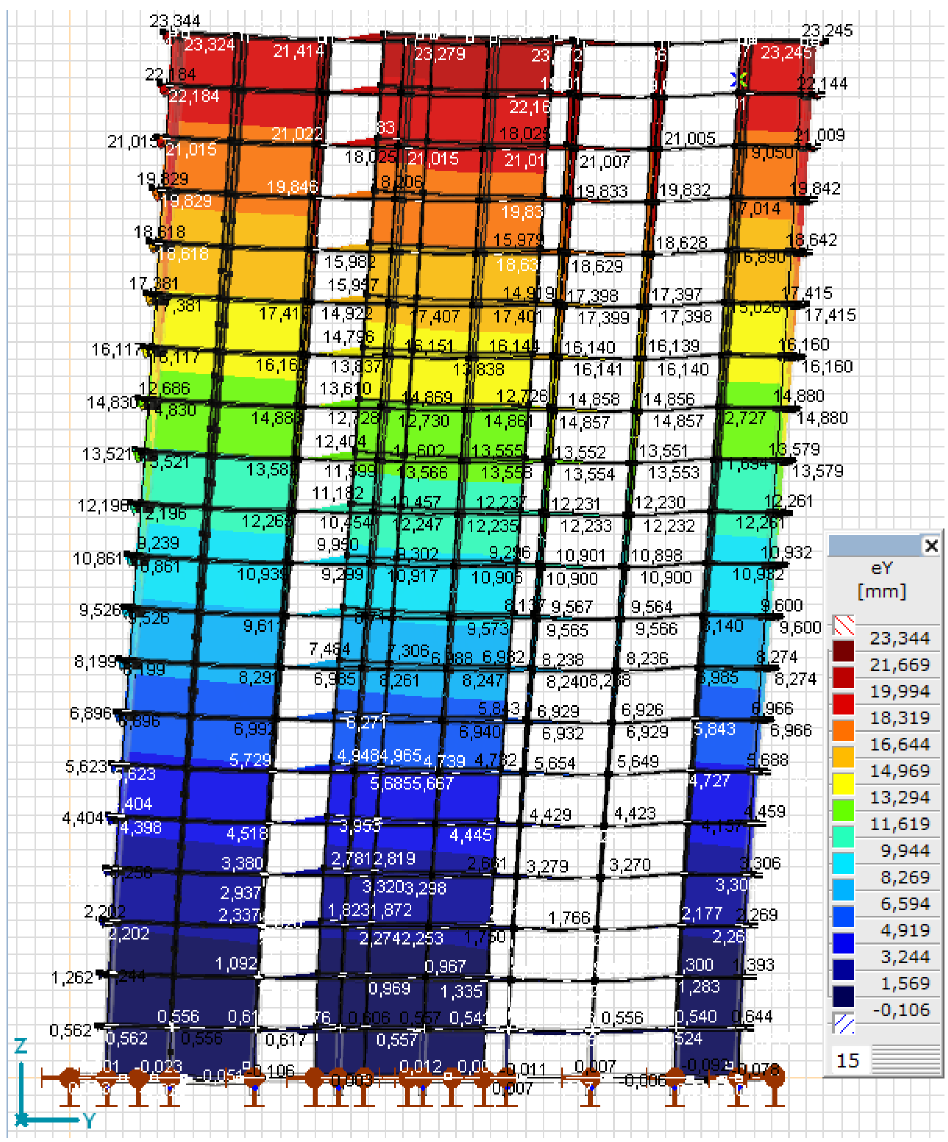


Figure 61: deflection of v/d Valk after activation of glass facade as the stability element

5.9 STRESS DISTRIBUTION IN GLASS FAÇADE

So far, the tension in the foundation piles and horizontal deformation with and without the glass façade has been discussed. However, with such a large structure and massive dead and live load and the horizontal wind load, the stress distribution in the glass façade is also very important to be studied. To understand how the stress on the glass pane is distributed and to emphasize the contribution of the timber frame around the glass façade, a new finite element design for the timber-glass façade has been developed. The decisive glass pane to study is the one at the ground level (canvas B-C, H-I & I-J). These are loaded vertically and horizontally.

To examine and determine the loadbearing capacity of the timber-glass facades, the weights will be applied as concentrated point loads, which are extensively discussed in paragraph 3.2. Different models are designed with the same immensity of loads to understand the contribution of the timber frame around the glass façade. The main goal is to underline and prove that timber glass can resist combined loads and is suitable for stabilizing buildings. A thicker glass façade or more extensive timber frame cross section resists higher loads. A rough calculation of the loads is presented in Table 11. The effort is to calculate the load as accurately as possible. However, the load will differ for various projects and is not the triggering factor.

On timber glass facade									
	qG	qQ	w	l	h	n	FG	FQ	
roof	7,3	-	1,2	1,5	-	1	13	-	kN
	-	1,0	1,2	1,5	-	1	-	2	kN
timber-glass facade	1,0	-	-	1,5	3,0	19	86	-	kN
floors	6,2	-	1,2	1,5	-	19	280	-	kN
	-	3,3	1,2	1,5	-	19	-	113	kN
							311	115	kN

Table 11: combined load on timber-glass facade

$$F_{wind} = \frac{(C_s C_d * C_{pe} * q_p(z) * h * b * k_{glass})}{n}$$

$$\rightarrow \frac{0.95 * 1{,}4 * 1{,}48 * 60 * 19.65 * \left(\frac{4.47}{19.65}\right)}{3} = 172\ kN$$

- n number of glass pane to resist the load in the same direction
- k_{glass} load division based on bending stiffness of glass presented in Figure 54
- C the remaining coefficients are discussed in paragraph 2.2.2

Figure 62 demonstrates the glass façade at the ground level between canvas H, I-5, with a length of 4,8 m and a thickness of 60 mm. This glass pane is considered the leading element and was designed with the FE program to investigate stress distribution in the glass facade further.

Figure 62 is designed without a timber frame around the glass façade to analyze the stress distribution due to concentrated death, live, and wind loads. The concentrated loads are presented on the left, and the stress climax's outcome is provided on the right hand. By analyzing the applied load and the stress distribution, one can conclude that the magnitude of the stress in the glass pane is above the maximum stress resistance capacity of the applied laminated glass. The maximum bending tensile strength for laminated glass is calculated in paragraph 3.4.1 and is equal to 27.7 N/mm^2, lower than the 33,4 N/m^2 occurred stress level in the glass façade.

There are various possibilities to regulate and reduce the stress climax beneath the maximum bending tensile strength of the glass façade. Increasing the glass thickness is the most obvious solution. However, this will increase the total weight of the structure again and the costs. By studying the FE model carefully, one can realize that the stresses are also very local due to applied local loads. The average stress over the glass façade is below 10 N/mm^2. Therefore, applying the timber frame around the glass façade might help distribute the stress better over the glass pane.

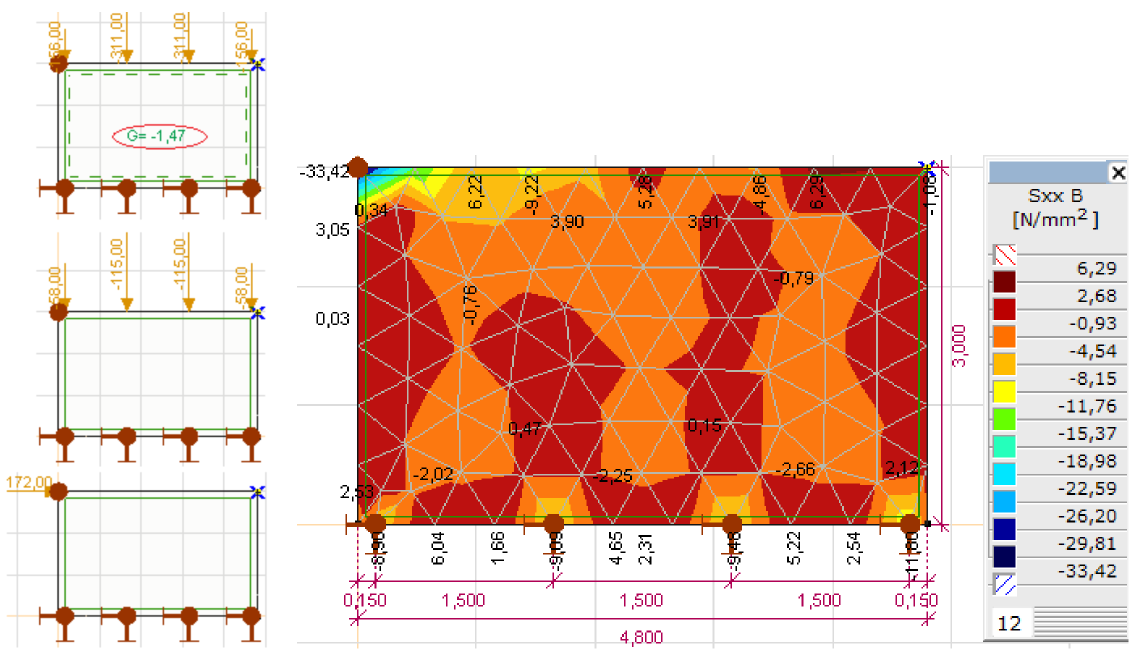


Figure 62: stress distribution in the glass pane

5.9.1 CONTRIBUTION OF TIMBER FRAME ON THE DISTRIBUTION OF STRESS ON THE GLASS

The timber around the glass façades and windows generally connects the glass with the surrounding structures. However, timber is an excellent structural material with various application fields in the construction world. Frames could be added to prevent highly concentrated stresses or singularities in glass facades under point loads. The goal is not to spread the stresses over a more considerable distance but to prevent highly concentrated peaks. The dimension and source of the timber to introduce the point load on the glass façade will be discussed and determined in the next paragraph.

Figure 63 demonstrates an overview of the timber-glass façade. The thickness of the glass is equivalent to the previous model of Figure 62. The dead load of the glass façade is linearly related to the thickness of the glass and determined by FE-program, which is emphasized in both models with a red ellipse (1,47 kN/m^2). The difference in this model is the timber frame around the glass façade represented by blue lines on the left model. The remaining parameters are identical. Analyzing the stress distribution in Figure 63, one can conclude that stress climax is reduced by about 10%. This is precise enough to fulfill the strength requirement.

$$\sigma_{Ed} \leq f_{t,m,u,d}$$

The stresses could even be decreased by applying a more extensive cross-section of the timber frame or increasing the thickness of the laminated glass façade. By comparing the two models, it is visible that the magnitude of the stress in this model is still concentrated under the point loads, at constraints, and around the timber frame with a relatively lower climax, which is in line with the expectation. However, comparably, the middle surface of the glass façade released the stress.

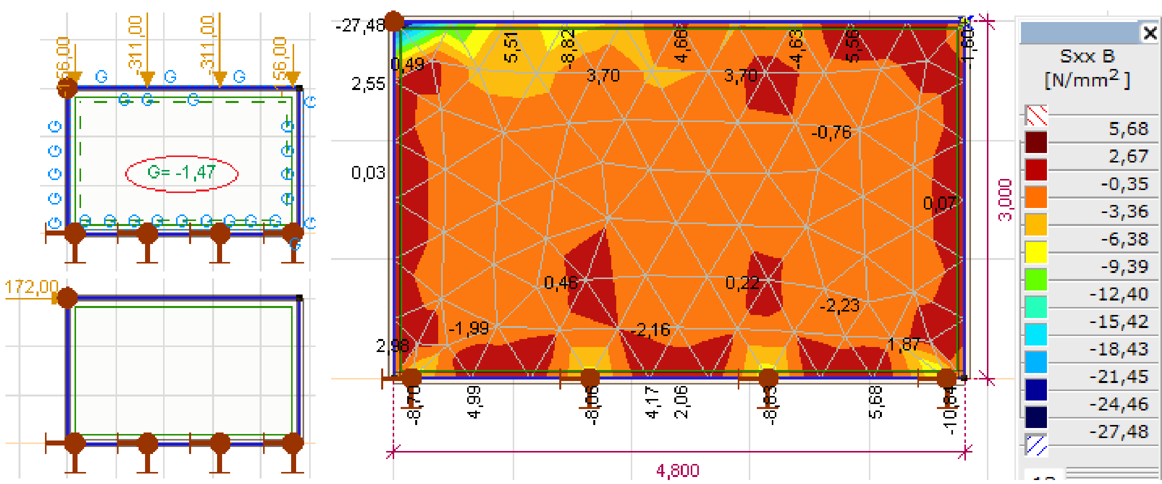


Figure 63: timber frame around glass façade

5.9.2 TIMBER FRAME & CONNECTION

The timber frame around the glass façade is designed with a high wood strength class D70 as mentioned in paragraph 3.3.2. The advantage and contributions of the timber frame around the glass façade are also discussed in paragraph 5.9.1. This paragraph aims to draw detailed designs to clarify the load transfer from the main structure through the timber frame to the glass façade.

The first design presented in Figure 64 demonstrates a longitudinal cross-section of a timber-glass façade with the concrete slab. The left part of the design represents an application of the line load on the glass façade where a full connection between timber and glass façade is visible. The right part of the design represents point load transfer with a limited surface connection between the timber frame and glass façade. The colored arrows represent wind load transfer through shear stresses at the connection.

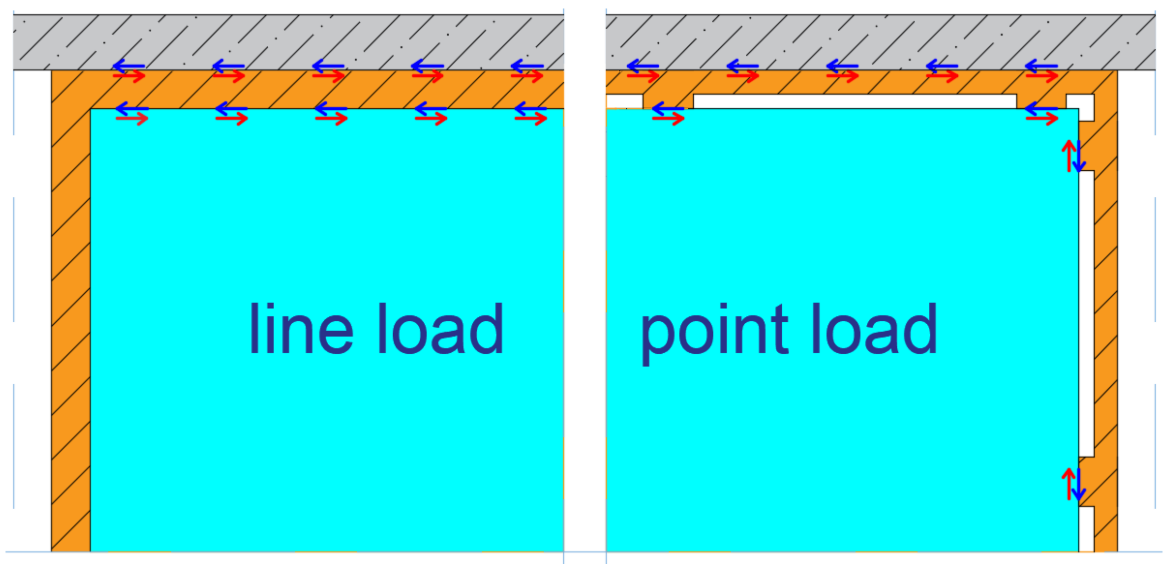


Figure 64: wind load transfer to the timber-glass facade as shear stress

Figure 64 confirms the importance of an appropriate connection type to transfer the load between various structural components. With this information in mind, the detailed connection design to introduce the point load on the glass façade is designed and presented in Figure 65. First, to accommodate concentrated load transfer, only a limited timber surface is connected to the glass façade. The area between the contact points is defined with sufficient gap to encounter the imparity of the glass and displacement of the timber frame. This area can be filled with a flexible material like rubber, PUR, etc., to prevent dust, noise, and moisture entrance, but allow small deformation. Glued connections are applied to connect the glass with a timber frame.

The maximum horizontal displacement of the building should not exceed 120 mm ($u = \frac{1}{500}h$). Therefore, it's essential to design connections with sufficient stiffness. To realize this, a steel plate is needed to integrate into the timber frame. The stiffness of the bolted timber steel to the concrete slab is manually calculated and added under the appendix A.

The surface area at the connection between the glass façade and the timber frame is related to the maximum shear resistance capacity of the Silkaflex-252 glue. The shear resistance capacity of Silkaflex-252 is already discussed under literature study executed by Kisa (2019) and Huveners (2009). The T-shape steel plate is designed at the connection between the timber frame and concrete slab to transfer the shear force mutually.

Figure 65 demonstrates an overview of the updated detailed design of the connection between different materials. To allow concentrated load transfer from the timber frame, the contact area of the timber with the glass is limited. The required length is determined by the magnitude of the shear force to be transferred from timber to glass.

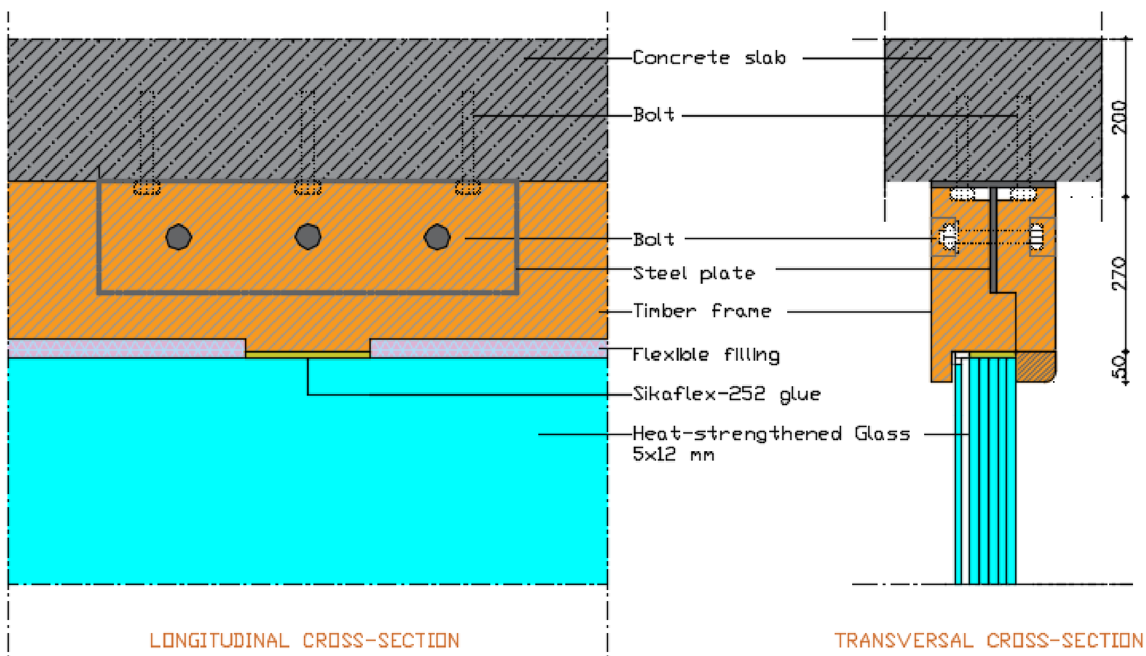


Figure 65: longitudinal and transversal cross-section of the timber-glass connection

5.10 SAFETY AT FAILURE & RESIDUAL CAPACITY OF GLASS

The application of glass windows to allow light entrance into buildings has existed for centuries. The functions of the glass pane were quite essential, and the variety of the glass material was limited. Nevertheless, glass material has developed enormously in the last decades, and its application field and sizes have also increased. Modern glass elements have relatively higher thermal resistance and are equipped with better sound isolation properties. Many global scholars proved the load-bearing capacity of the glass material through several destructive tests, which enabled the design of the glass elements with the invention of modern state-of-the-art programs.

Glass material poses high compression strength, making it a suitable transparent and elegant material to be applied as a load-bearing member. However, its ductile behavior and impressionability to cracks under tension and bending stresses must be considered while designing. Another critical aspect of the load-bearing glass member is safety at failure and the residual load-bearing capacity. The glass members' total failure should be minimized to ensure the end user's safety. The designer should also minimize harm to the users from failure due to large particles of the glass element with a suitable glass type. Improvement in the properties of the glass types and lamination of the suitable glass types can enlarge the safety and ensure sufficient residual capacity at the failure to prevent total failure (Figure 66).

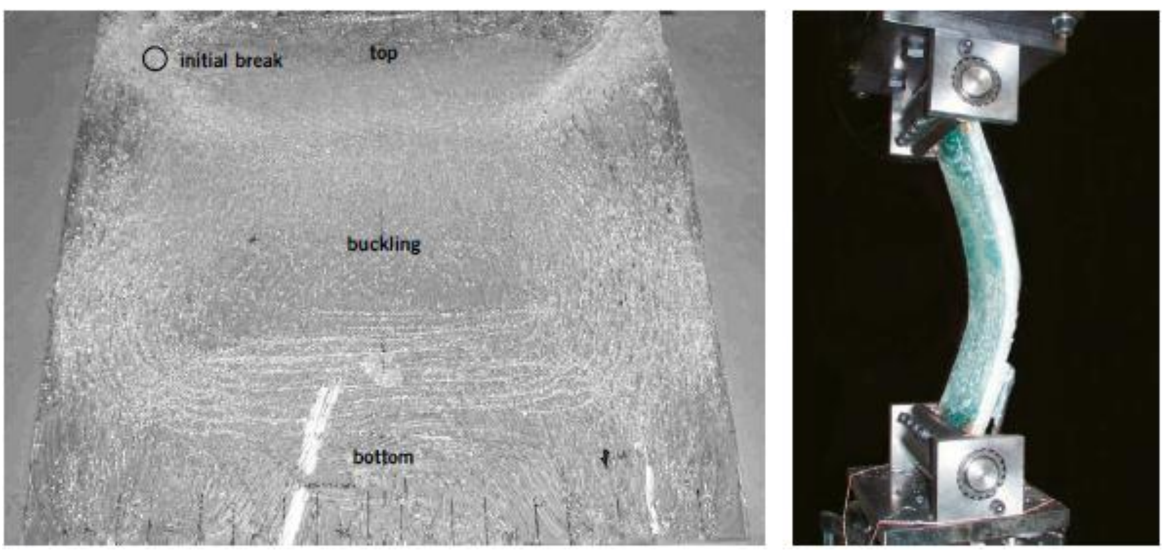


Figure 66: residual capacity of glass and failure in very small particles to enlarge safety (image, Jan Wurm)

To ensure the safety of the users, the designed laminated glass must break into small particles to prevent/minimize the damage, which brings us to tempered glass. While designing, the designer should also consider that not all the layers break simultaneously and size to prevent local failure due to local stresses. Therefore, it is essential to apply the leaves with heath-strength glass. Heath-strength glass breaks into relatively larger particles, which enables stress distribution over a substantial area of the laminated panel. This will distribute the stresses and activate the surface of the glass pane. Special attention is required to the connections. The connected surface should be smooth, and the stress distribution must be equal to prevent the local failure of glass. The edges of the glass panels are quite vulnerable and have relatively more minor residual stress resistance. Prestressing of the thermal glass occurs due to relatively fast cooling. Where tension occurs at the surface layer and compression in the center part of the glass. Cooling at the edges is slightly different, with relatively close temperature differences. Therefore, the residual load-bearing capacity of the edges is lower and sufficient distance from the edges is required for drilling.

The most desired glass pane is laminated from various types of glass. To ensure the user's safety and provide sufficient residual capacity, the timber-glass facade could be designed in heath-strength glass for the leaves and tempered glass for the inner layer. This will enable the timber-glass façade to resist stresses and provide safety. The exact thickness of the tempered glass is related to the magnitude of the applied load, which will be discussed and determined under manual calculation and FE analysis.

5.10.1 Fire safety

Building fire safety is one of the critical aspects to consider while designing. Fire safety measurements and requirements for buildings in the Netherlands are regulated and defined in the Eurocode under fire safety (NEN-6069, 2019) and the national building decree "bouwbesluit." The goal of this requirement is to encourage the designer to take sufficient precautions during the design stage so that the buildings are enforced with fire safety measurements. These measures should enable the user to leave the building safely and on time when a fire breaks out. Figure 67 presents the minimum required time in minutes for various buildings during an evacuation of a fire.

There are various technics to make construction materials fire-resistant to fulfill fire safety regulations. For example, steel structures could be equipped with a coating layer or gypsum plates to delay the fire and protect the steel structure. The dimensions of the steel structure could also be overdesigned to provide extra time before collapsing due to melting.

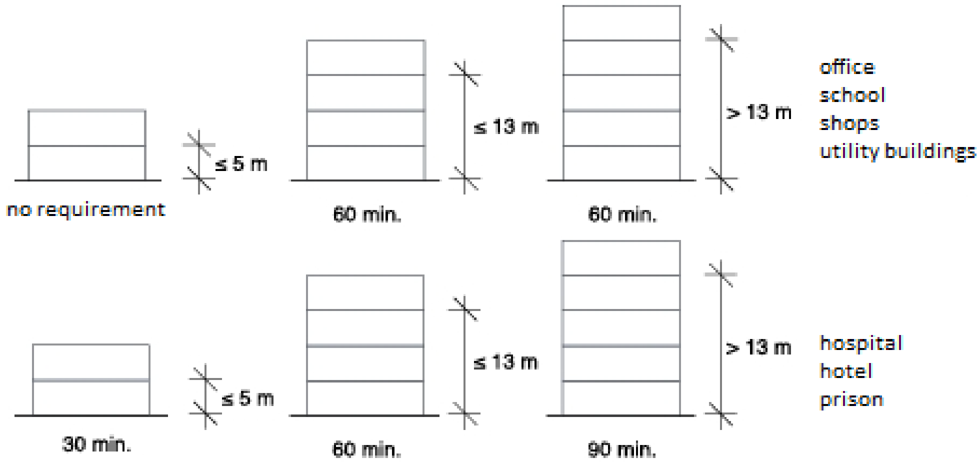


Figure 67: fire safety of the buildings in correlation with its height and function expressed in minutes

Fire safety of structures and construction materials is an important topic. This report focuses on evaluating the application of the timber-glass façade as a stability element, which covers the requirements. Although the fire resistance per country in the European norm differs, the Dutch and French national building decree requires doors and façades to fulfill the EW30 and EW60 (van Dijk, 2011).

E = Flame tightness; should not pass through the construction for a specific time

W= Heat radiation; measured at a 1-meter distance from the structure should not exceed 15 kW/m2.

Due to the limited application of the glass as a structural element, the fire safety of the glass is not clarified and documented very well. Therefore, additional research is required to assess whether fire resistant coating layer is required.

5.11 CONCLUSION

The main goal of this chapter was to prevent tension in the foundation piles by activating the available timber-glass façades. The existing building is equipped with additional measurements to compensate for the tension in the foundation piles without considering the available glass façade as a stability element. Therefore, discussing the original design and the applied loads on the structure was essential.

Before activating the timber-glass facades as a stability element, the location of the existing load-bearing and stability concrete walls are identified, and their contribution toward stabilizing the building is discussed. The suitable, load combination and division of the vertical load on the stability elements are also studied to encounter this for possible FEM analysis and manual calculation. The correlation is also highlighted by determining the bending stiffness of the concrete wall and the glass façade. Before starting with any calculation, one can conclude, based on obtained bending stiffnesses, that the timber-glass façade has sufficient bending stiffness strength to stabilize the building.

To emphasize the effect of the timber-glass façade as a load-bearing and stability element in the design of the van der Valk hotel, the tension in the foundation piles of the original design had to be identified. This is confirmed through manual calculation in paragraph 5.5.3. With this information, the building is designed with the FE program to precise the outcome. The tension in the foundation piles is also confirmed in paragraph 5.7.2 by analyzing the finite element model. The next challenge was to prove that timber-glass facades are suitable for reducing/minimizing the tension in the foundations.

After trial and error, the required number and location of the timber-glass facades are determined in paragraph 5.8 to reduce/neglect the tension in the foundation piles. Reduction of the horizontal deflection due to the activation of glass façades as the stability element is also highlighted. The chapter is finalized by designing a separate FE model for the timber-glass façade to study the stress distribution over the glass façade and to determine the required thickness of the glass façade at the ground level. This final case study of this research confirms that timber-glass facades are suitable for extended buildings and could be applied to new midsize and large buildings. Timber-glass facades are suitable load-bearing elements and can also resist horizontal wind loads. However, additional research is required to investigate the replacement of the glass at failure and detail designing with the surrounding structure.

6 Discussion

The application of timber-glass façade is extensively discussed in this report. The three selected projects demonstrated the appropriateness of the timber-glass façades as load-bearing and stability elements. In this report, the compression strength of the glass material is emphasized and purposed to apply glass façade as a structural component to distribute the horizontal and normal forces nicely over the foundation piles. However, the vulnerability of the glass panes and the drilling of glass, especially at the edges, is left out of the context of this research.

The connection of the glass panes, where a high magnitude of the combined shear and tension is discussed very briefly. Glass is a very vulnerable material and challenging to design an appropriate connection where a high magnitude of shear stress can be transferred. This makes timber-glass façade less suitable for larger projects with relatively high combined loads' immensity. However, expansion and midsize buildings with limited shear stresses can be very well executed with traditional bolt and glue connections. An important measure also to be considered is the bending stiffness losses due to the type of connection. This can affect the load division and appropriateness of the timber-glass façade as a stability element. Unexpected bending stiffness losses can cause, e.g., horizontal deflection of the structure with possible cracks.

The results of this research were to investigate the suitability of glass as a load-bearing and stability element for different types and scales of buildings. A critical aspect requiring attention is also the constructability and build sequence. It is not easy to apply glass already in an early stage together with the remaining structural members to stabilize the building. Because the remaining stability and load-bearing members generally get a finishing layer afterward, this is impossible for a timber-glass façade, which contributes to the design's aesthetic and look and is very vulnerable. However, during the designing stage can be decided to encounter timber-glass façade to participate only in resistance of variable loads, which is very common in the bridge industry. Timber-glass façade can be installed at a later stage. The opening for the glass façade can be stabilized during construction, e.g., by diagonals.

The vulnerability of glass is discussed extensively. The melting point of the glass is relatively higher. However, additional investigation is required to map the behavior of structural glass at the fire. To create a homogeneous and strong structure, the required thickness of the glass façade could be designed in heat-strengthened glass. Heat-strengthened glass provide also the sufficient residual capacity to prevent the total collapse of the façade at failure.

Despite various challenges and limitations of the glass material, this research can contribute to further investigation to apply glass panes as a structural component in buildings. With the anticipation of the limitation of the glass material, the structural designer is better prepared to apply glass material as a structural element where possible. This can be the start of a new structural material already available in every building. Using it as a structural member, the construction industry can save costs by making additional structural elements redundant, reducing emissions, and creating excellent, transparent designs.

7 Conclusions

This research investigates the suitability of structural glass as a stability element in various types of buildings. Three different and independent real projects are evaluated to assess the appropriateness of the timber-glass façade from different perspectives to answer the main research question, of whether buildings can be stabilized by timber-glass façade. Based on the numerical calculation, different 2D and finite element calculations confirmed that timber-glass façade could be applied to stabilize the buildings. However, the proper detailed design of the connections and the relatively lower bending tensile strength of the glass must also be considered.

Literature study

The history and evolution of structural glass including the variety in the type of glass and the corresponding connections are discussed in the literature study. Among others, A. de Groot and M. Kisa investigated the possibilities to strengthen the existing buildings in the province of Groningen by the application of structural glass against seismic loading. They have enforced their findings by executing lab research to study the behavior of structural glass undergoing mutual loadings. The glass pane was glued to the timber frame to also study the strength of the glued connections. The appropriateness of the glass pane glued to the timber frame was in line with the finding of Huveners Ph.D. research. The results of this research and other scholars provided sufficient confidence to continue investigating the appropriateness of structural glass as a load-bearing and stability element for existing and new buildings.

Based on the literature study and assembled information is concluded that heat-strengthened glass glued to the timber frame and bolted connection of the timber frame to the concrete slab can be designed for further assessment in this research. Silkaflex252 glued connections are suitable to transfer the shear stress and to create moisture, noise, and airtightness to fulfill the building physics requirements. Bolted connections are appropriate to connect the timber frame with the concrete slab which minimizes the bending stiffness losses of the stability elements.

Existing stability techniques

The main objective of studying the existing technics was to assess their appropriateness in designing a hybrid system where timber glass is included as load bearing and stability element. After reviewing the bending stiffness of different materials and their deformation behavior, it is concluded that a hybrid system combining glass with steel portals is very suitable. Timber glass façade can provide stiffness to the hybrid system to fulfill the displacement requirement. Steel columns are pretty slender and can deform horizontally once loaded by the wind. Therefore, the additional requirement (diagonals, thicker cross-section, etc.) are needed to fulfill the maximum allowed deflection of 1/500*h. The essence of the hybrid system is to combine the strength of different construction materials with available glass in a building to reduce general costs and construction emissions and create new intelligent designs with glass as a structural element.

Application field of timber-glass façade

Based on the mechanical properties of the glass material and its application in practice as load bearing element can be concluded that glass is a suitable structural element. Glass has outstanding compression capacity and can resist combined loads. Insulated glass units (IGU) with various thicknesses can be glued together to realize the desired thickness for stabilizing existing buildings, expanding buildings, and creating new designs. IGUs are designed to improve energy efficiency, reduce noise transmission, and enhance thermal performance. They can also reduce condensation and improve indoor comfort. Besides their structural performances, IGUs can be customized with different glass coatings, tints, and patterns to meet specific design requirements. The limitation of the glass is also considered while identifying its application field. Glass is very venerable to cracks, and its bending tensile strength about its compression strength is minimal. Designing of appropriate connection to introduce load on glass is also very complicated and limited due to its bending tensile strength. To determine the application field of the timber glass façades, three case studies are evaluated for further analysis.

Case study 1

Stabilizing the buildings with timber-glass facades while expanding is an opportunity to integrate the timber-glass façade into the design. Integration of the timber-glass façade analyzed under case study 1 approved its appropriateness and contribution in linked houses. By activation of the available glass façade, additional stability elements were redundant. The manual calculation demonstrated the glass's compression and bending tensile strength. Combining timber-frame with glass pane is the best of both materials put forward. This case study demonstrated that timber-glass façade could stabilize traditional linked houses.

Case study 2

Reducing CO2 emissions is an important aspect that an engineer must consider during design and decision-making. In the Netherlands, even far-reaching measurements are required regarding the CO2 emission/footprint to receive construction permission. Therefore, integrating different construction materials is crucial to come up with intelligent designs to reduce the magnitude of pollution due to the construction of buildings. The application of the timber-glass façade discussed in case study 2 demonstrates the appropriateness of glass combined with steel structure in a hybrid system. Applying a timber-glass façade as a load-bearing and stability system can significantly reduce the cross-section of the steel portal around the glass. To emphasize the contribution of the timber-glass façade in a hybrid system, the cross-section of the steel portals is with and without the timber-glass façade structurally assessed. Application of the finite element program confirmed that timber-glass façade elements are suitable to be applied as load-bearing and stability elements and be combined with existing stability techniques. By carefully analyzing the three case studies, one can conclude that timber-glass façade is available in all three projects. However, its behavior, contribution, and limitation regarding the existing techniques are elaborated under case study two.

Case study 3

The focus of the last case study was to investigate the appropriateness of the timber-glass façade from a broader perspective. The design of a high-rise building is carefully selected to demonstrate the load transfer from the top to the foundation through the timber-glass façade and to study the behavior of glass, its contribution, and its limitation. Due to a limited number of stability elements, tension occurred in the foundation piles of the existing design. The appearance of tension in the foundation due to horizontal wind load is confirmed by manual calculation and as with the FE program.

To study the behavior of the building under combined load, a selected number of available glass façades are activated, and the finite element model is simulated for further analysis. With the knowledge of the structural theory and some trial and error, the required number of glass panes are activated to neglect tension in the foundation piles. This case study again confirms the contribution and suitability of the timber-glass façade as a load-bearing and stability element in high-rise buildings.

Based on the content of this research and the discussed projects extended with manual calculation and finite element analysis can be concluded that timber-glass façade is suitable to be applied as load bearing and stability element. However, it should be also mentioned, that involvement of a high magnitude of forces at the connections requires additional attention to detail designing. Glass is very vulnerable at connections, and transferring shear forces and tension is quite critical. Stiffness losses due to connection must be also examined to assure proper load division between various stability elements.

8 Recommendation

This research focused on evaluating the suitability of the timber-glass façade as a stability and load-bearing element. Therefore, load transfer to the glass pane and the behavior of the glass under combined load is discussed extensively. Attention is also paid to applying the stability element in different types of buildings. However, there are still opportunities to evaluate glass as the main stability element without the involvement of any other stability material. The focus then will be on how to deal with failure. The residual capacity of the glass is discussed in this research, but another measurement could also be considered by applying glass as a single stability element. Think of the thicker dimension of the glass panes, increasing the number of stability elements than required, or protecting the glass pane with other mediums to prevent accidental damage and guarantee the safety of the users until the evacuation.

Connections are also a fascinating and crucial topic to be investigated. Connections are expected to transfer the combined loads. Glass can resist compression forces very well compared to tension. Therefore, paying extra attention to the connections loaded on tension and shear forces is essential. The general theory of connection and connectivity of glass to the timber and the surrounding structures is discussed based on existing literature. However, the magnitude of the forces can increase once the glass is applied as the main stability element. Therefore, the glued connection's resistance capacity and the glue's required thickness must be investigated to guarantee safe load transfer.

Drilling in glass requires ultimate caution to prevent failure. Glass is also weaker at the edges than the remaining parts. Stability elements are required to resist compression, tension, and shear forces. Therefore, additional research is required to investigate the drilling of holes for the bolted connection at the edges of the glass pane, which can subsequently transfer the combined forces.

Eventually, there is also the requirement to investigate the fire safety, explosion, and accidental load-bearing capacity of the glass to demonstrate the appropriateness of the glass as a stability element. This could be combined with the residual capacity of the glass to discuss the safety of the glass pane as a load-bearing and stability element.

I. List of Figures

II. List of tables

III. References

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IV. Appendixes

A. Bending stiffness of the connection:

Below is the bending stiffness of the bolted connection of the timber frame to the concrete slab is determined.

$$k_{ser} = \frac{1}{23} * \rho_{mean}^{1,5} * d \rightarrow \frac{1}{23} * 520^{1,5} * 16 = 8249 \frac{N}{mm}$$

$$k_c = n_{shear\ face} * n_{dowel} * k_{ser} = 2 * 2 * 8249 = 32996 \frac{N}{mm}$$

$$k_{steel\ plate} = \frac{EA}{L} \rightarrow 210000 * 10 * \frac{200}{600} = 700000$$

$$\frac{1}{k_{tot}} = \frac{1}{k_s} + \frac{1}{k_c} \rightarrow k_{tot} = \frac{700000 * 32996}{700000 + 32996} = 31510 \frac{N}{mm}$$

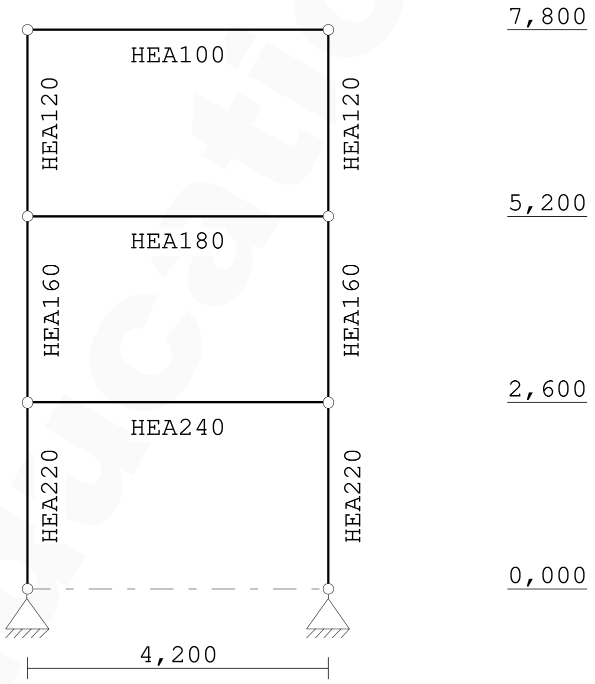
$$k_r = a^2 * k_r \rightarrow 100^2 * \frac{31510}{10^6} = 315\ kNm/rad$$

```
Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal
Struct. eng..: A.P. Azamy
Units........: kN;m;rad  (unless otherwise stated)
Date.........: 08/08/2022
File.........: D:\TU-2019-2020\Afstuderen feb 20-21\Afstuderen BE\Thesis
               report\Technosoft\Portaal Maassluis inc. wind.rww

Load width..........:   1.000
Calculation model.: 2nd-order-elastic
Theories for structural analysis:
1) Load cases:
   Linear elastic theory
2) Ultimate limit state:
   Geometrical non-linear all bars.
   Physical linear all bars.
3) Serviceability limit state:
   Geometrical non-linear all bars.
   Physical linear all bars.

Max. number of iterations.....: 50
Max. part length columns/walls:  0.500 Max. part length beams/floors:  0.500
Max. X-displacement in ULS....:  0.500 Max. Z-displacement in ULS...:  0.250

Favorable effect of the permanent load is automatic processed.
```

Applied standards according to Eurocode with Dutch NA

Loads	NEN-EN 1990:2002	C2:2010,A1:2019	NB:2019(nl)
	NEN-EN 1991-1-1:2002	C1/C11:2019	NB:2019(nl)
Steel	NEN-EN 1993-1-1:2006	C2:2011,A1:2016	NB:2016(nl)

GEOMETRY



Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

GRID LINES

No.	Name	X	Z-min	Z-max
1		0.000	0.000	7.800
2		4.200	0.000	7.800

LEVELS

No.	Z	X-min	X-max
1	0.000	0.000	4.200
2	2.600	0.000	4.200
3	5.200	0.000	4.200
4	7.800	0.000	4.200

MATERIALS

Mt	Quality	E-modulus[N/mm2]	S.W.	Pois.	Exp. coeff.
1	S235	210000	78.5	0.30	1.2000e-05

SECTIONS [mm]

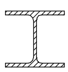
Sect.	Description	Material	Area	Inertia	Formf.
1	HEA120	1:S235	2.5340e+03	6.0600e+06	0.00
2	HEA160	1:S235	3.8800e+03	1.6730e+07	0.00
3	HEA220	1:S235	6.4300e+03	5.4100e+07	0.00
4	HEA240	1:S235	7.6800e+03	7.7630e+07	0.00
5	HEA180	1:S235	4.5300e+03	2.5100e+07	0.00
6	HEA100	1:S235	2.1240e+03	3.4900e+06	0.00

SECTIONS contd. [mm]

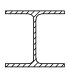
Sect.	Bar type	Width	Height	e	Type	w1	h1	w2	h2
1	0:Normal	120	114	57.0					
2	0:Normal	160	152	76.0					
3	0:Normal	220	210	105.0					
4	0:Normal	240	230	115.0					
5	0:Normal	180	171	85.5					
6	0:Normal	100	96	48.0					

SECTIONFORMS [mm]

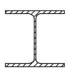
1 HEA120



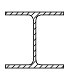
2 HEA160



3 HEA220



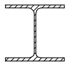
4 HEA240



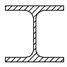
Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

SECTIONFORMS [mm]

5 HEA180



6 HEA100



NODES

Node	X	Z	Node	X	Z
1	0.000	0.000	6	4.200	5.200
2	0.000	7.800	7	0.000	2.600
3	4.200	0.000	8	4.200	2.600
4	4.200	7.800			
5	0.000	5.200			

BARS

Bar	Ni	Nj	Section	Joint.i	Joint.j	Length	Rem.
1	1	7	3:HEA220	ASM	ASM	2.600	
2	3	8	3:HEA220	ASM	ASM	2.600	
3	2	4	6:HEA100	ASM	ASM	4.200	
4	5	2	1:HEA120	ASM	ASM	2.600	
5	5	6	5:HEA180	ASM	ASM	4.200	
6	6	4	1:HEA120	ASM	ASM	2.600	
7	7	5	2:HEA160	ASM	ASM	2.600	
8	7	8	4:HEA240	ASM	ASM	4.200	
9	8	6	2:HEA160	ASM	ASM	2.600	

FIXED SUPPORTS

No.	node	Code XZR 1=fixed 0=free	Angle
1	1	110	0.00
2	3	110	0.00

LOAD GENERATION GENERAL

Safety class ....: 1 Reference period.......: 50
Building depth..............: 10.00 Building height........: 7.80
Level adjacent terrain.......: 0.00 S.w. sep. walls [kN/m2]: 1.20

LOAD CASES

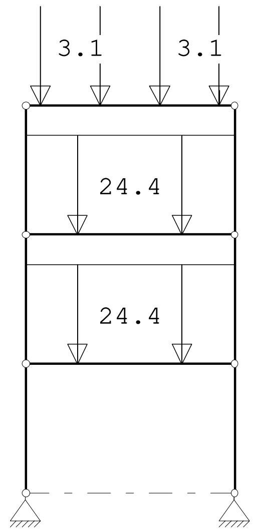
LCa	Description		Type
1	Permanent loads	SWZ=-1.00	1 Permanent load
2	Variable loads		2 Var. load pers. etc. (q_k)

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

LOADINGS

LCa:1 Permanent loads

Selfweight of all bars. Direction:↓



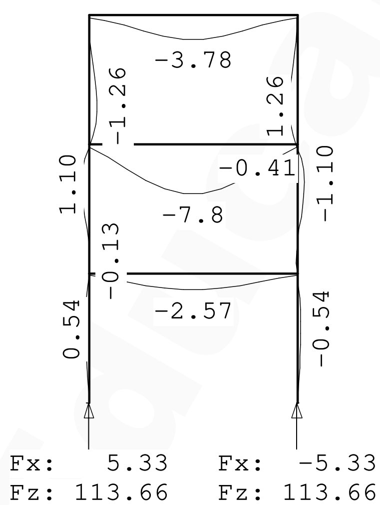
BAR LOADS

LCa:1 Permanent loads

Bar	Typ	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
8	5:QZGlobal	-24.40	-24.40	0.000	0.000			
5	5:QZGlobal	-24.40	-24.40	0.000	0.000			
3	10:PZProjected	-3.10		0.300				
3	10:PZProjected	-3.10		1.500				
3	10:PZProjected	-3.10		2.700				
3	10:PZProjected	-3.10		3.900				

TRANSLATIONS

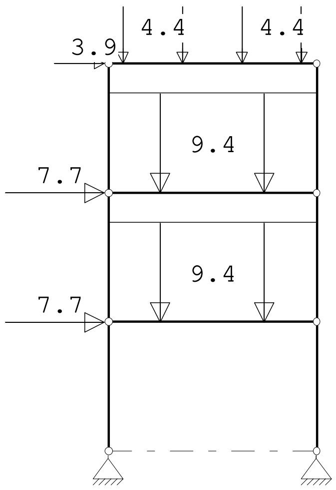
1st order [mm] LCa:1 Permanent loads



Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

LOADINGS

LCa:2 Variable loads



NODE LOADS

LCa:2 Variable loads

Load	Node	Direction	value	ψ_0	ψ_1	ψ_2
1	7	X	7.700	0.40	0.50	0.30
2	5	X	7.700	0.40	0.50	0.30
3	2	X	3.900	0.40	0.50	0.30

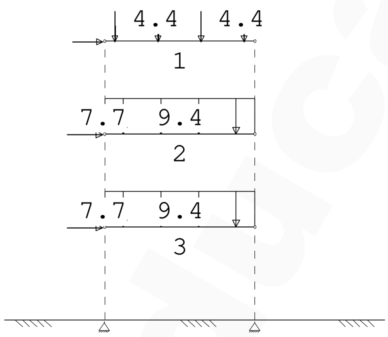
BAR LOADS

LCa:2 Variable loads

Bar	Typ	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
8	5:QZGlobal	-9.40	-9.40	0.000	0.000	0.40	0.50	0.30
5	5:QZGlobal	-9.40	-9.40	0.000	0.000	0.40	0.50	0.30
3	10:PZProjected	-4.40		0.300		0.40	0.50	0.30
3	10:PZProjected	-4.40		1.500		0.40	0.50	0.30
3	10:PZProjected	-4.40		2.700		0.40	0.50	0.30
3	10:PZProjected	-4.40		3.900		0.40	0.50	0.30

SITUATIONS LOADED/UNLOADED

LCa:2 Variable loads



Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

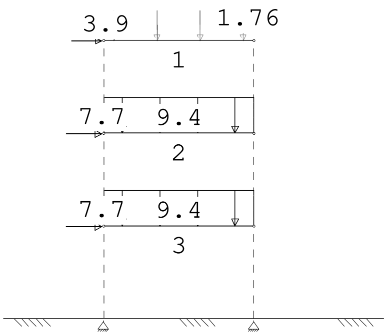
SITUATIONS LOADED/UNLOADED

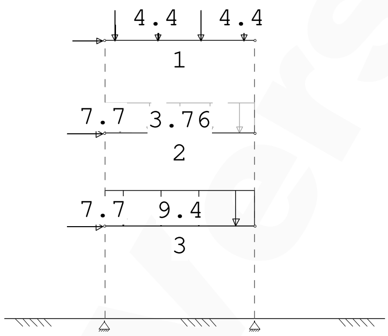
Loadtype: q_k

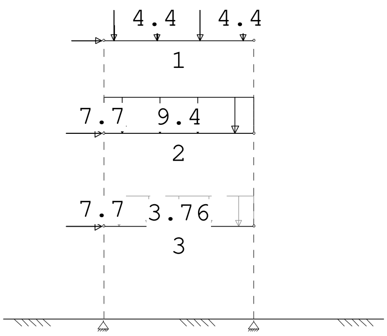
No.	Loaded fields	Unloaded fields
1	1-3	

SITUATIONS FLOOR EXTREME

LCa:2 Variable loads







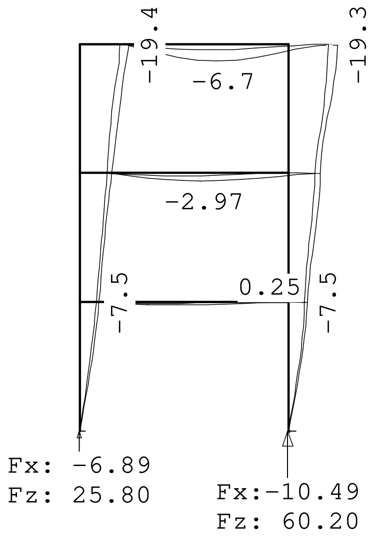
SITUATIONS FLOOR EXTREME

Loadtype: q_k

No	Floor with Extreem loads	Floor with Momentary loads
1	1,2	3
2	1,3	2
3	2,3	1

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

TRANSLATIONS 1st order [mm] LCa:2 Variable loads



REACTIONS 1st order

Nd.	L.C.	X-min	X-max	Z-min	Z-max	M-min	M-max
1	1	5.33		113.66			
1	2	-6.89	-4.80	17.75	25.80		
3	1	-5.33		113.66			
3	2	-10.49	-7.79	52.26	60.20		

CALCULATION STATUS Reinf. check

L.C.	Iteration	Status
1	3	Accuracy reached
2	3	Accuracy reached
3	3	Accuracy reached
4	3	Accuracy reached
5	3	Accuracy reached
6	3	Accuracy reached
7	3	Accuracy reached
8	3	Accuracy reached
9	3	Accuracy reached
10	3	Accuracy reached
11	3	Accuracy reached
12	3	Accuracy reached

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

LOAD COMBINATIONS

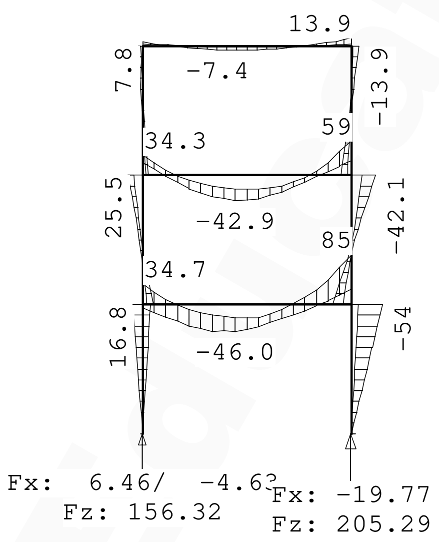
BC	Type					
1	Fund.	1.22 $G_{k,1}$				
2	Fund.	0.90 $G_{k,1}$				
3	Fund.	1.22 $G_{k,1}$	+	1.35	ψ_0	$Q_{k,2}$
4	Fund.	1.08 $G_{k,1}$	+	1.35		$Q_{k,2}$
5	Fund.	0.90 $G_{k,1}$	+	1.35		$Q_{k,2}$
6	Fund.	0.90 $G_{k,1}$	+	1.35	ψ_0	$Q_{k,2}$
7	Char.	1.00 $G_{k,1}$	+	1.00		$Q_{k,2}$
8	Quas.	1.00 $G_{k,1}$				
9	Quas.	1.00 $G_{k,1}$	+	1.00	ψ_2	$Q_{k,2}$
10	Freq.	1.00 $G_{k,1}$				
11	Freq.	1.00 $G_{k,1}$	+	1.00	ψ_1	$Q_{k,2}$
12	Perm.	1.00 $G_{k,1}$				

FAVOURABLE PARTS OF PERMANENT ACTION

LCo	Beams with favourable parts of permanent action
1	No beams
2	All beams the factor:0.90
3	No beams
4	No beams
5	All beams the factor:0.90
6	All beams the factor:0.90

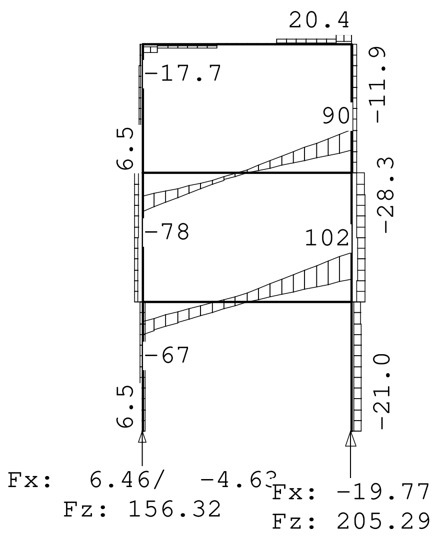
CONTOUR OF THE FUNDAMENTAL COMBINATIONS

MOMENTS 2nd order Fundamental combination

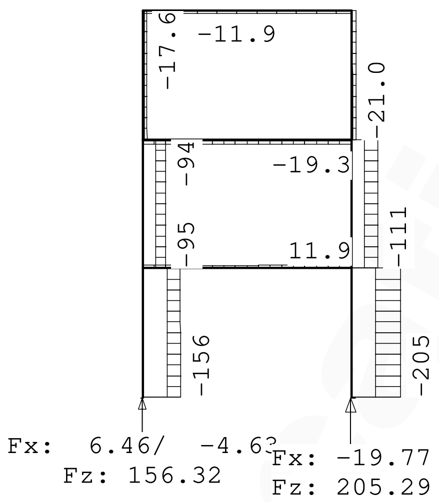


Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

SHEAR FORCES 2nd order Fundamental combination



AXIAL FORCES 2nd order Fundamental combination



BAR FORCES 2nd order Fundamental combination

			AXi/AXj				SZi/SZj				MYi/MYj			
Bar	Nd.	Pos.	Min	LC	Max	LC	Min	LC	Max	LC	Min	LC	Max	LC
1	1		-156.31	4	-102.30	2	-5.16	5	6.54	1	0.00	1	0.00	1
1		1.733	-155.37	4	-101.51	2	-5.09	5	6.45	1	-8.89	5	11.28	1
1		1.733	-155.37	4	-101.51	2	-5.05	5	6.40	1	-8.89	5	11.28	1
1	7		-154.89	4	-101.12	2	-5.00	5	6.32	1	-13.25	5	16.79	1
2	3		-205.16	4	-102.30	2	-21.04	4	-4.84	2	0.00	1	0.00	1
2	8		-203.85	4	-101.12	2	-19.98	4	-4.71	2	-53.63	4	-12.45	2

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

BAR FORCES 2nd order Fundamental combination

			AXi/AXj				SZi/SZj				MYi/MYj			
Bar	Nd.	Pos.	Min	LC	Max	LC	Min	LC	Max	LC	Min	LC	Max	LC
3	2		-11.75	4	-3.79	2	-17.68	4	-5.90	2	3.59	2	7.85	4
3		0.578	-11.81	4	-3.79	2	-8.30	4	-3.03	2	0.00	5	1.33	1
3		0.907	-11.81	4	-3.78	2	-8.23	4	-2.98	2	-2.53	4	0.00	1
3		1.500	-11.83	4	-3.79	2	-8.10	4	-2.89	2	-7.37	4	-1.75	2
3		1.500	-11.88	4	-3.79	2	-0.13	4	1.32	2	-7.37	4	-1.75	2
3		2.300	-11.87	4	-3.79	2	0.03	2	1.49	4	-6.31	4	-1.77	2
3		2.300	-11.87	4	-3.79	2	0.03	2	1.53	4	-6.31	4	-1.77	2
3		3.159	-11.79	4	-3.78	2	2.96	2	11.01	4	-0.96	4	0.00	5
3		3.298	-11.79	4	-3.78	2	2.98	2	11.04	4	0.00	2	1.11	4
3	4		-11.82	4	-3.79	2	5.90	2	20.42	4	3.59	2	13.89	4
4	5		-18.18	4	-6.37	2	3.78	2	6.50	4	-10.04	4	-5.65	5
4		1.152	-17.92	4	-6.15	2	3.79	2	6.56	4	-3.09	4	0.00	5
4		1.678	-17.81	4	-6.06	2	3.80	2	6.57	4	0.00	2	2.63	4
4		2.166	-17.71	4	-5.97	2	3.80	2	6.56	4	1.94	2	5.36	4
4	2		-17.62	4	-5.89	2	3.79	2	6.53	4	3.59	2	7.85	4
5	5		-19.20	4	-8.38	2	-78.05	4	-46.82	2	21.41	5	34.33	4
5		0.433	-19.33	4	-8.42	2	-60.99	4	-37.18	2	0.00	5	9.61	1
5		0.660	-19.38	4	-8.42	2	-52.02	4	-32.11	2	-10.30	4	0.00	2
5		1.867	-19.79	4	-8.55	2	-7.05	1	-1.39	5	-42.86	4	-23.44	2
5		1.867	-19.81	4	-8.56	2	-7.02	1	-1.36	5	-42.86	4	-23.44	2
5		2.333	-19.79	4	-8.56	2	5.20	2	16.00	4	-40.34	4	-23.44	2
5		2.333	-19.74	4	-8.55	2	5.22	2	16.05	4	-40.34	4	-23.44	2
5		3.349	-19.40	4	-8.44	2	27.86	2	56.14	4	-8.51	1	0.00	5
5		3.563	-19.33	4	-8.42	2	32.64	2	64.60	4	0.00	1	11.13	5
5	6		-19.29	4	-8.38	2	46.82	2	89.67	4	25.14	2	59.02	4
6	6		-20.96	4	-6.37	2	-11.86	4	-3.78	2	6.27	2	17.18	4
6		1.356	-20.59	4	-6.11	2	-12.00	4	-3.80	2	0.00	5	1.86	4
6		1.681	-20.52	4	-6.06	2	-12.00	4	-3.80	2	-3.40	4	0.00	2
6	4		-20.37	4	-5.89	2	-11.92	4	-3.79	2	-13.89	4	-3.59	2
7	7		-94.97	4	-53.86	2	9.66	5	16.65	1	-17.89	1	-7.32	5
7		0.756	-94.71	4	-53.65	2	9.69	5	16.75	1	-5.27	1	0.00	5
7		1.087	-94.60	4	-53.55	2	9.68	5	16.78	1	0.00	5	3.26	4
7		2.167	-94.27	4	-53.27	2	9.53	5	16.64	1	11.29	5	18.32	1
7		2.167	-94.30	4	-53.29	2	9.39	5	16.48	1	11.29	5	18.32	1
7	5		-94.16	4	-53.17	2	9.39	5	16.48	1	15.77	5	25.46	1
8	7		2.16	5	11.99	4	-67.30	4	-39.73	5	-5.93	5	34.68	1
8		0.669	2.07	5	11.91	4	-43.45	1	-21.28	5	-27.79	4	0.00	2
8		1.400	2.00	5	11.86	4	-21.25	1	-1.13	5	-45.75	4	-18.40	2
8		1.400	1.99	5	11.86	4	-21.25	1	-1.14	5	-45.75	4	-18.40	2
8		1.633	1.98	5	11.85	4	-14.17	1	5.29	5	-45.66	4	-20.85	2
8		1.867	1.98	5	11.85	4	-7.08	1	11.73	5	-46.04	4	-23.30	2
8		1.867	1.98	5	11.86	4	-7.08	1	11.73	5	-46.04	4	-23.30	2
8		2.333	2.00	5	11.87	4	5.25	2	28.17	4	-38.55	4	-23.30	2
8		2.333	2.02	5	11.89	4	5.25	2	28.17	4	-38.55	4	-23.30	2
8		3.008	2.09	5	11.94	4	20.42	2	54.93	4	-18.96	1	0.00	5
8		3.551	2.12	5	11.96	4	32.64	2	76.49	4	0.00	1	28.31	5
8	8		2.07	5	11.89	4	47.25	2	102.27	4	25.69	2	85.00	4

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

BAR FORCES 2nd order Fundamental combination

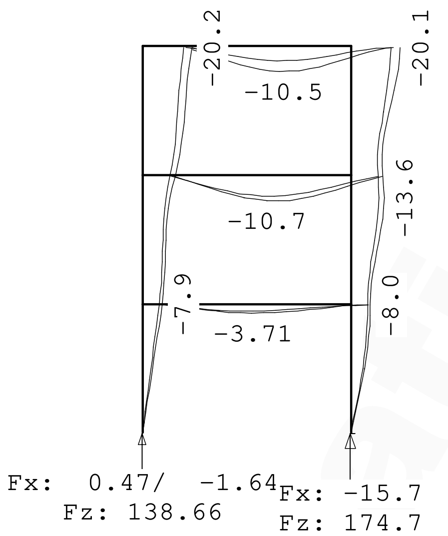
			AXi/AXj				SZi/SZj				MYi/MYj			
Bar	Nd.	Pos.	Min	LC	Max	LC	Min	LC	Max	LC	Min	LC	Max	LC
9	8		-111.33	4	-53.86	2	-28.23	4	-12.34	2	13.25	2	31.82	4
9		1.004	-110.90	4	-53.57	2	-28.59	4	-12.41	2	0.00	2	4.22	4
9		1.195	-110.84	4	-53.52	2	-28.59	4	-12.41	2	-3.54	4	0.00	5
9		2.167	-110.60	4	-53.27	2	-28.28	4	-12.34	2	-30.01	4	-13.56	2
9		2.167	-110.70	4	-53.29	2	-27.88	4	-12.25	2	-30.01	4	-13.56	2
9	6		-110.56	4	-53.17	2	-27.88	4	-12.25	2	-42.06	4	-18.87	2

REACTIONS 2nd order Fundamental combination

Nd.	X-min	X-max	Z-min	Z-max	M-min	M-max
1	-4.63	6.46	102.30	156.32		
3	-19.77	-4.79	102.30	205.29		

CONTOUR OF THE CHARACTERISTIC COMBINATIONS

TRANSLATIONS 2nd order [mm] Characteristic combination



Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

GENERAL DATA

Stability:	Classification of structure as whole:	Sway
Deflection and translation:		
	No. of storeys:	1
	Type of building	Other
	Permiss. horiz. trans. of whole building:	h/300
	Lowest outside wall height [m]:	0.0

SECTION/MATERIAL

S/M No.	Section name	Yield str. [N/mm^2]	Manufac. method	Min. x-sect. class
1	HEA120	235	Rolled	1
2	HEA160	235	Rolled	1
3	HEA220	235	Rolled	1
4	HEA240	235	Rolled	1
5	HEA180	235	Rolled	1
6	HEA100	235	Rolled	1

Partial safety factors:
Gamma M;0 : 1.00 Gamma M;1 : 1.00

BUCKLING STABILITY

Bar	l_{sys} [m]	Classif. y strong axis	$l_{buckl;y}$ [m]	Suppl. pend. y [kN]	Classif. z weak axis	$l_{buckl;z}$ [m]	Suppl. pend. z [kN]
1	2.600	Sway	2nd orde		Non-sway	2.600	0.0
2	2.600	Sway	2nd orde		Non-sway	2.600	0.0
3	4.200	Sway	2nd orde		Non-sway	4.200	0.0
4	2.600	Sway	2nd orde		Non-sway	2.600	0.0
5	4.200	Sway	2nd orde		Non-sway	4.200	0.0
6	2.600	Sway	2nd orde		Non-sway	2.600	0.0
7	2.600	Sway	2nd orde		Non-sway	2.600	0.0
8	4.200	Sway	2nd orde		Non-sway	4.200	0.0
9	2.600	Sway	2nd orde		Non-sway	2.600	0.0

LATERAL-TORSIONAL BUCKLING

Bar	Action point		l fork [m]	Lateral restraint distances [m]
1	1.0*h	upper:	2.60	2.600
		lower:	2.60	2.600
2	0.0*h	upper:	2.60	2.600
		lower:	2.60	2.600
3	1.0*h	upper:	4.20	3*1,4
		lower:	4.20	4,2
4	1.0*h	upper:	2.60	2.600
		lower:	2.60	2.600
5	1.0*h	upper:	4.20	3*1,4
		lower:	4.20	4.200
6	0.0*h	upper:	2.60	2.600
		lower:	2.60	2.600
7	1.0*h	upper:	2.60	2.600
		lower:	2.60	2.600

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

LATERAL-TORSIONAL BUCKLING

Bar	Action point		l fork [m]	Lateral restraint distances [m]
8	1.0*h	upper:	4.20	3*1,4
		lower:	4.20	4.200
9	0.0*h	upper:	2.60	2.600
		lower:	2.60	2.600

CHECK OF STRESSES

Bar	S/M No.	LCo	Sit	Cl	Loc.	Stand.	Article	Formula	Highest check U.C.	[N/mm²]	Rem.
1	3	1	1	1	Bar	EN3-1-1	6.3.3	(6.61)	0.209	49	46,47
2	3	4	1	1	Bar	EN3-1-1	6.3.3	(6.61)	0.507	119	46,47
3	6	4	3	1	Bar	EN3-1-1	6.3.3	(6.62)	0.802	189	46
4	1	4	1	1	Bar	EN3-1-1	6.3.3	(6.62)	0.384	90	47
5	5	4	3	1	End	EN3-1-1	6.2.10	(6.45+6.31y)	0.773	182	46
6	1	4	3	1	Bar	EN3-1-1	6.3.3	(6.62)	0.650	153	46,47
7	2	3	1	1	Bar	EN3-1-1	6.3.3	(6.61)	0.501	118	46,47
8	4	4	1	1	End	EN3-1-1	6.2.10	(6.45+6.31y)	0.486	114	46
9	2	4	3	1	Bar	EN3-1-1	6.3.3	(6.61)	0.806	189	46,47

Remarks:
[46] An equivalent Q-load has been calculated for the purposes of LTB.
[47] In case of a variable axial force, the largest compressive stress is taken.

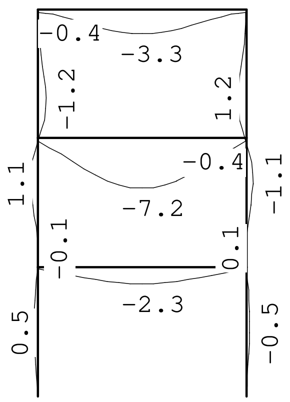
CHECK FOR DEFLECTION

Bar	Type	Det	Length [m]	Cantil I	Cantil J	Camb [mm]	u_{tot} [mm]	LCo	Sit		u [mm]	Permissible [mm]	*l
3	Roof	df	4.20	N	N	0.0	-9.9	7	2	End	-9.9	-16.8	0.004
		df						7	2	Suppl	-6.6	-16.8	0.004
5	Floor	df	4.20	N	N	0.0	-10.1	7	1	End	-10.1	±16.8	0.004
		df						7	1	Suppl	-2.8	±12.6	0.003
8	Floor	df	4.20	N	N	0.0	-3.4	7	2	End	-3.4	±16.8	0.004
		df						7	2	Suppl	-1.1	±12.6	0.003

Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

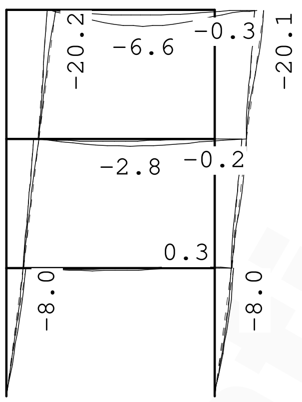
DEFLECTION w1

Permanent combination



DEFLECTION Wadd

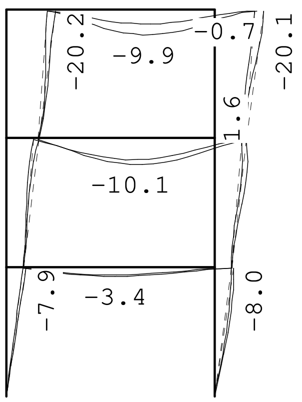
Characteristic combination



Project......: Stabilization of buildings by timber-glass facade
Part.........: Steel portal

DEFLECTION Wmax

Characteristic combination



DEFLECTION

Characteristic combination

No.	Bars	side	position [m]	l_{rep} [mm]	w_1 [mm]	w_2 [mm]	w_{add} [mm]	w_{add} [lrep/]	w_{tot} [mm]	w_c [mm]	w_{max} [mm]	w_{max} [lrep/]
7	3	Neg.	1.900	4200	-3.3		-6.6	639	-9.9		-9.9	426
8	5	Neg.	1.867	4200	-7.2		-2.8	1475	-10.1		-10.1	417
9	8	Neg.	1.400	4200	-2.0		-1.2	3571	-3.1		-3.1	1336

Technosoft Liggers release 6.74 3 feb 2023
Project......: Load division
Component....: Stabilization of buildings by timber-glass facade
Struct. eng..: A.P. Azamy
Units........: kN/m/rad
Date.........: 18/04/2022
File.........: D:\TU-2019-2020\Afstuderen feb 20-21\Afstuderen BE\Thesis report\Technosoft\Windload division.dlw

Applied standards according to Eurocode with Dutch NA

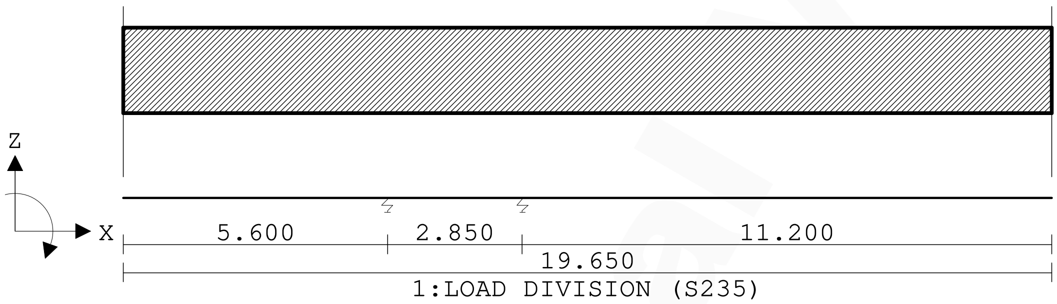
Loads	NEN-EN 1990:2002	C2:2010,A1:2019	NB:2019(nl)
	NEN-EN 1991-1-1:2002	C1/C11:2019	NB:2019(nl)

BEAM:1

Section : LOAD DIVISION

GEOMETRY

Beam:1



FIELD LENGTHS

Beam:1

Field	From	To	Length
1	0.000	5.600	5.600
2	5.600	8.450	2.850
3	8.450	19.650	11.200

MATERIALS

Mt	Quality	E-modulus[N/mm2]	S.W.	Pois.	Exp. coeff.
1	S235	210000	78.5	0.30	1.2000e-05

SECTIONS [mm]

Sect.	Description	Material	Area	Inertia	Formf.
1	LOAD DIVISION	1:S235	1.9750e+04	8.6980e+08	0.00

SECTIONS contd. [mm]

Sect.	Bar type	Width	Height	e	Type	w1	h1	w2	h2
1	0:Normal	300	490	245.0					

SPRINGS

Beam:1

Spring	Supp.	Direction	Spring const.	Type	Bottom limit	Top limit
1	1	2:Z-transl.	3.780e+08	Normal	-1.000e+10	1.000e+10
2	2	2:Z-transl.	3.780e+08	Normal	-1.000e+10	1.000e+10

Project......: Load division
Component....: Stabilization of buildings by timber-glass facade

LOAD CASES

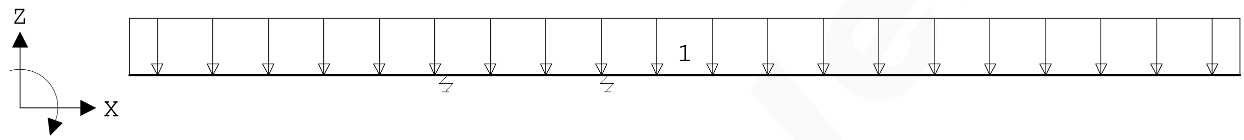
L.C.	Description	Loaded/unloaded	ψ_0	ψ_1	ψ_2	s.w.
1	Permanent	2:Permanent EN1991				0.00

LOAD CASES

LCa	Description	Type
1	Permanent	1 Permanent load

FIELD LOADS

Beam:1 LCa:1 Permanent



FIELD LOADS

Beam:1 LCa:1 Permanent

Load Ref.	Type	Description	q1/p/m	q2	psi	Dist.	Length
1	1:q-load		-1.000	-1.000		0.000	19.650

LOAD COMBINATIONS

LCo	Type	LCa	Gen.	Factor	LCa	Gen.	Factor	LCa	Gen.	Factor	LCa	Gen.	Factor
1	Fund.	1	Perm	1.49									
2	Fund.	1	Perm	1.32									
3	Fund.	1	Perm	0.90									
4	Char.	1	Perm	1.00									
5	Freq.	1	Perm	1.00									
6	Quas.	1	Perm	1.00									
7	Perm.	1	Perm	1.00									

FAVOURABLE PARTS OF PERMANENT ACTION

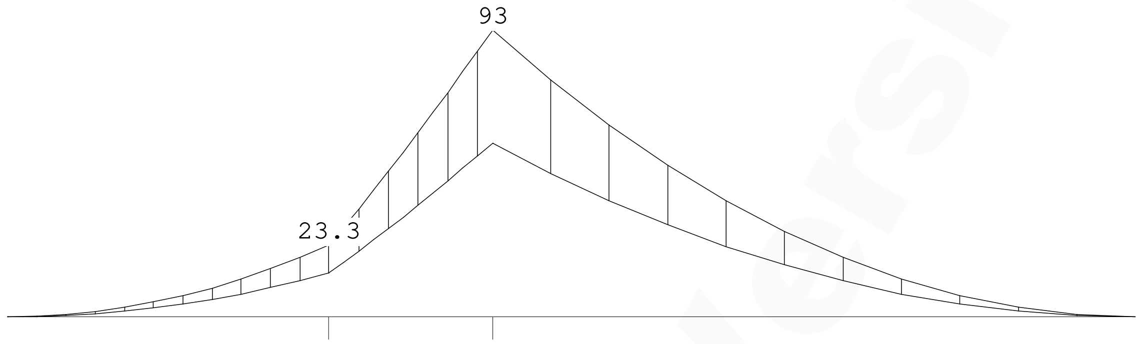
LCo	Fields with favourable parts of permanent action
1	No beams
2	No beams
3	All fields the factor:0.90

Project......: Load division
Component....: Stabilization of buildings by timber-glass facade

CONTOUR OF THE FUNDAMENTAL COMBINATIONS

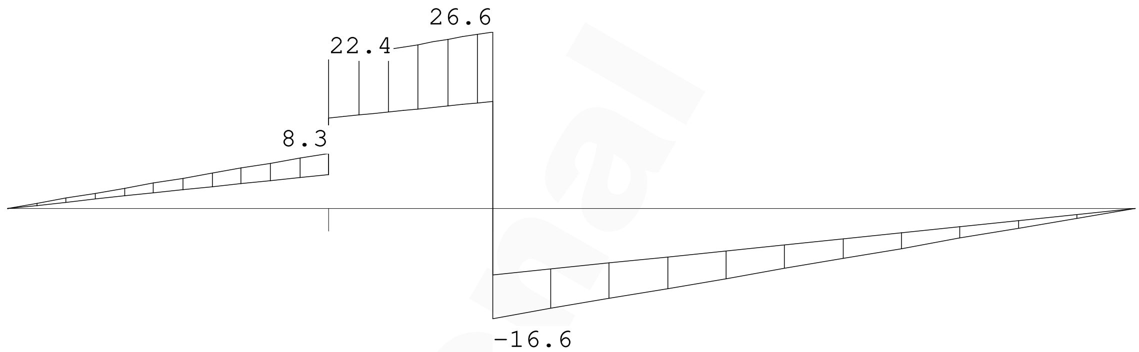
MOMENTS

Beam:1 Fundamental combination



SHEAR FORCES

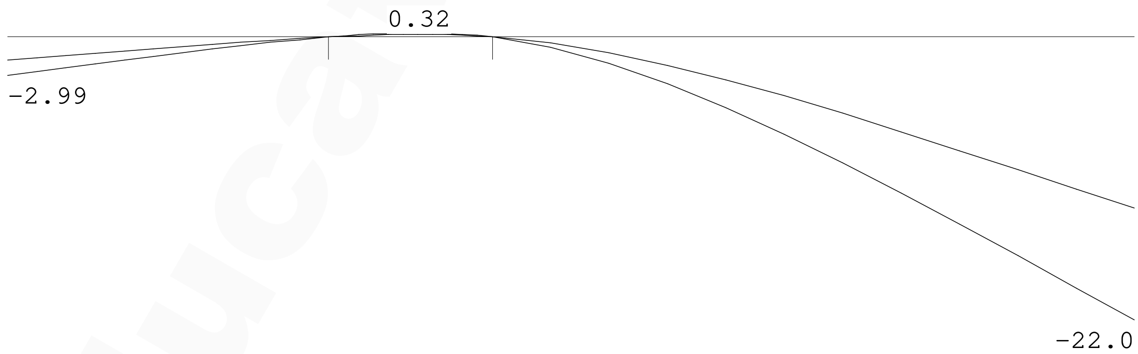
Beam:1 Fundamental combination



Fmin:-14.1 26.2
Fmax:-8.5 43.3

TRANSLATIONS [mm]

Beam:1 Fundamental combination



REACTIONS

Beam:1 Fundamental combination

Supp	Fmin	Fmax	Mmin	Mmax
1	-14.08	-8.53	0.00	0.00
2	26.22	43.26	0.00	0.00

Technosoft Liggers release 6.74 3 feb 2023

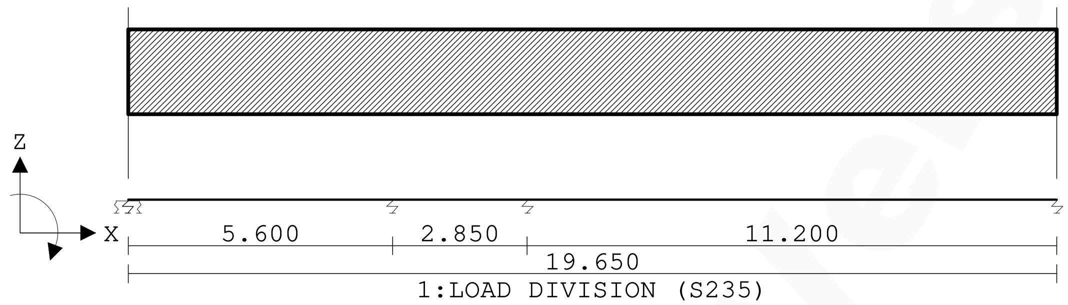
Project......: Load division
Component....: Stabilization of buildings by timber-glass facade

BEAM:2

Section : LOAD DIVISION

GEOMETRY

Beam:2



FIELD LENGTHS

Beam:2

Field	From	To	Length
1	0.000	5.600	5.600
2	5.600	8.450	2.850
3	8.450	19.650	11.200

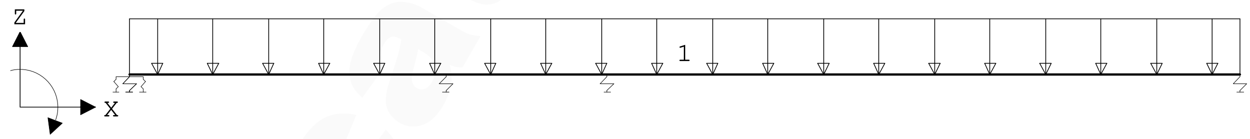
SPRINGS

Beam:2

Spring	Supp.	Direction	Spring const.	Type	Bottom limit	Top limit
1	2	2:Z-transl.	3.870e+08	Normal	-1.000e+10	1.000e+10
2	3	2:Z-transl.	3.870e+08	Normal	-1.000e+10	1.000e+10
3	4	2:Z-transl.	1.630e+07	Normal	-1.000e+10	1.000e+10
4	1	3:Rotation	1.000e+03	Normal	-1.000e+10	1.000e+10
5	1	2:Z-transl.	1.630e+07	Normal	-1.000e+10	1.000e+10

FIELD LOADS

Beam:2 LCa:1 Permanent



FIELD LOADS

Beam:2 LCa:1 Permanent

Load Ref.	Type	Description	q1/p/m	q2	psi	Dist.	Length
1	1:q-load		-1.000	-1.000		0.000	19.650

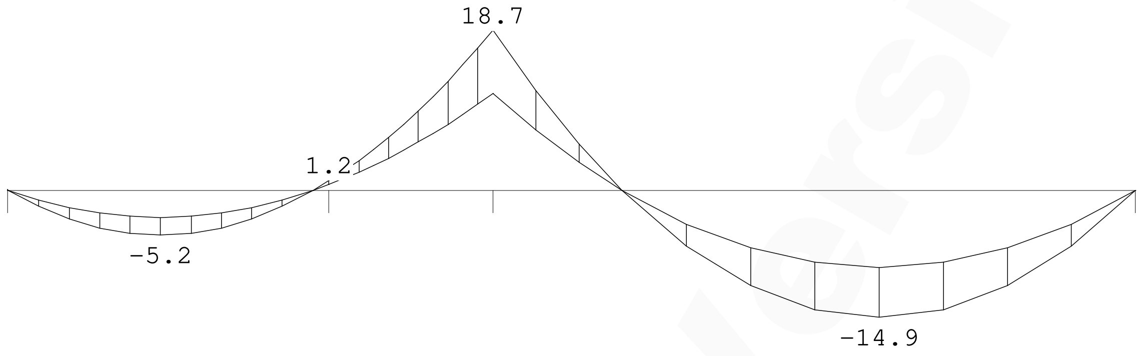
Technosoft Liggers release 6.74 3 feb 2023

Project......: Load division
Component....: Stabilization of buildings by timber-glass facade

CONTOUR OF THE FUNDAMENTAL COMBINATIONS

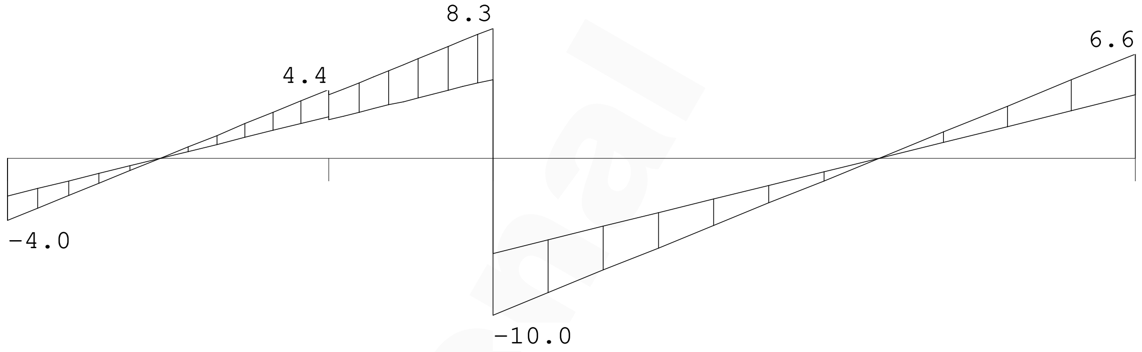
MOMENTS

Beam:2 Fundamental combination



SHEAR FORCES

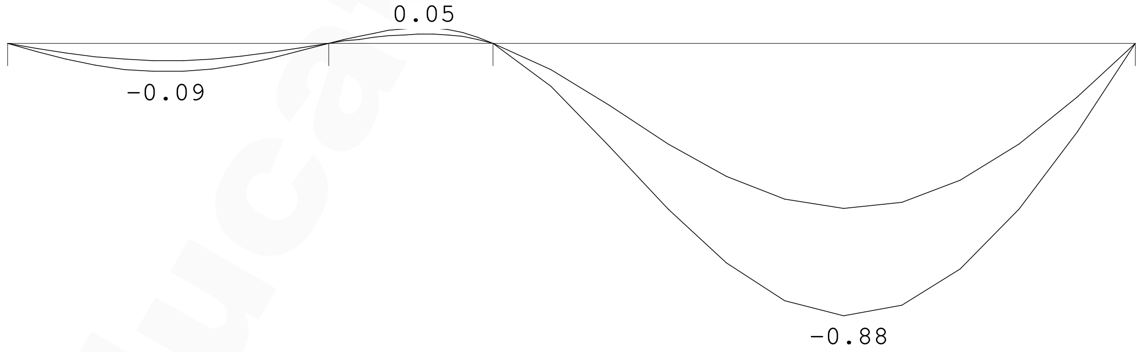
Beam:2 Fundamental combination



Fmin:2.40 0.19 11.1 4.03
Fmax:3.95 0.32 18.3 6.6

TRANSLATIONS [mm]

Beam:2 Fundamental combination



REACTIONS

Beam:2 Fundamental combination

Supp	Fmin	Fmax	Mmin	Mmax
1	2.40	3.95	-0.05	-0.03
2	0.19	0.32	0.00	0.00
3	11.07	18.27	0.00	0.00
4	4.03	6.64	0.00	0.00

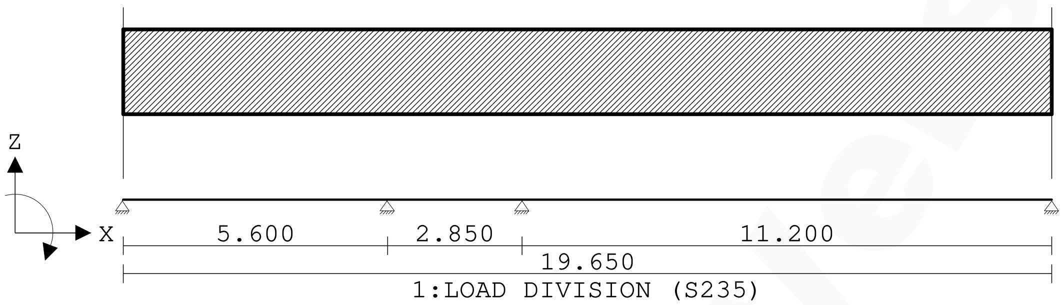
Project......: Load division
Component....: Stabilization of buildings by timber-glass facade

BEAM:3

Section : LOAD DIVISION

GEOMETRY

Beam:3



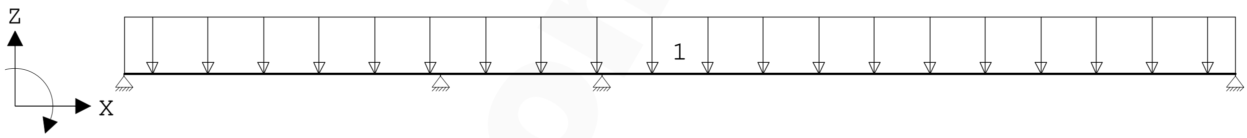
FIELD LENGTHS

Beam:3

Field	From	To	Length
1	0.000	5.600	5.600
2	5.600	8.450	2.850
3	8.450	19.650	11.200

FIELD LOADS

Beam:3 LCa:1 Permanent



FIELD LOADS

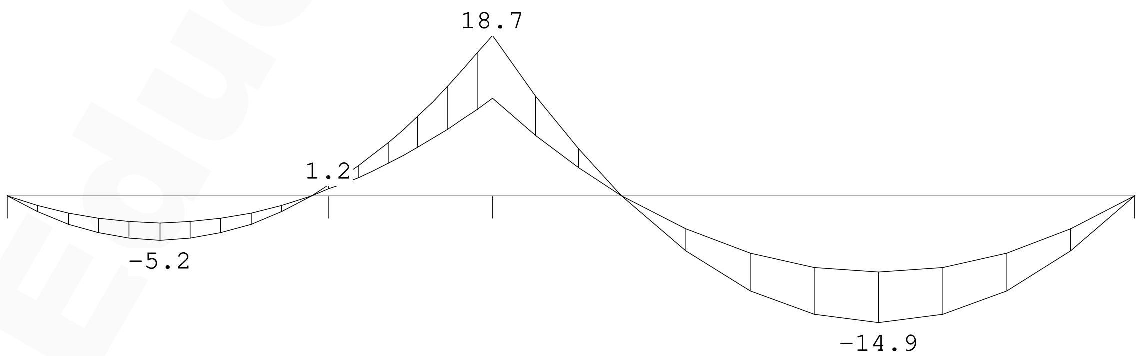
Beam:3 LCa:1 Permanent

Load Ref.	Type	Description	q1/p/m	q2	psi	Dist.	Length
1	1:q-load		-1.000	-1.000		0.000	19.650

CONTOUR OF THE FUNDAMENTAL COMBINATIONS

MOMENTS

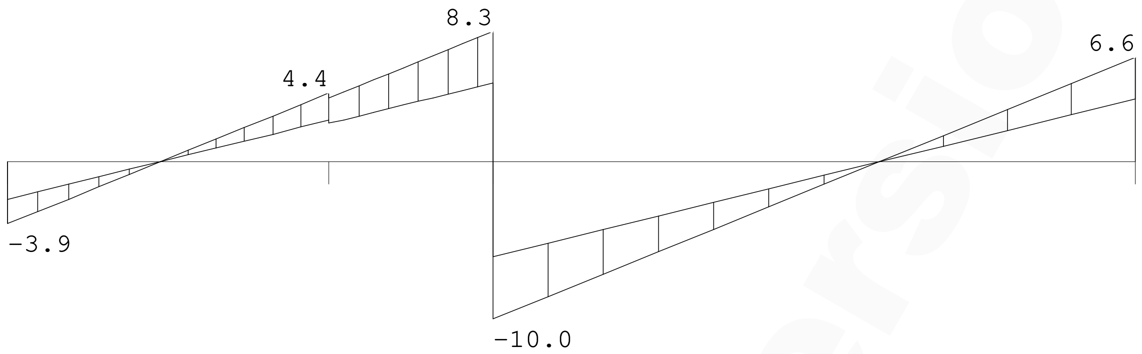
Beam:3 Fundamental combination



Project......: Load division
Component....: Stabilization of buildings by timber-glass facade

SHEAR FORCES

Beam:3 Fundamental combination



Fmin:2.39 0.20 11.1 4.03
Fmax:3.94 0.33 18.3 6.6

TRANSLATIONS [mm]

Beam:3 Fundamental combination

REACTIONS

Beam:3 Fundamental combination

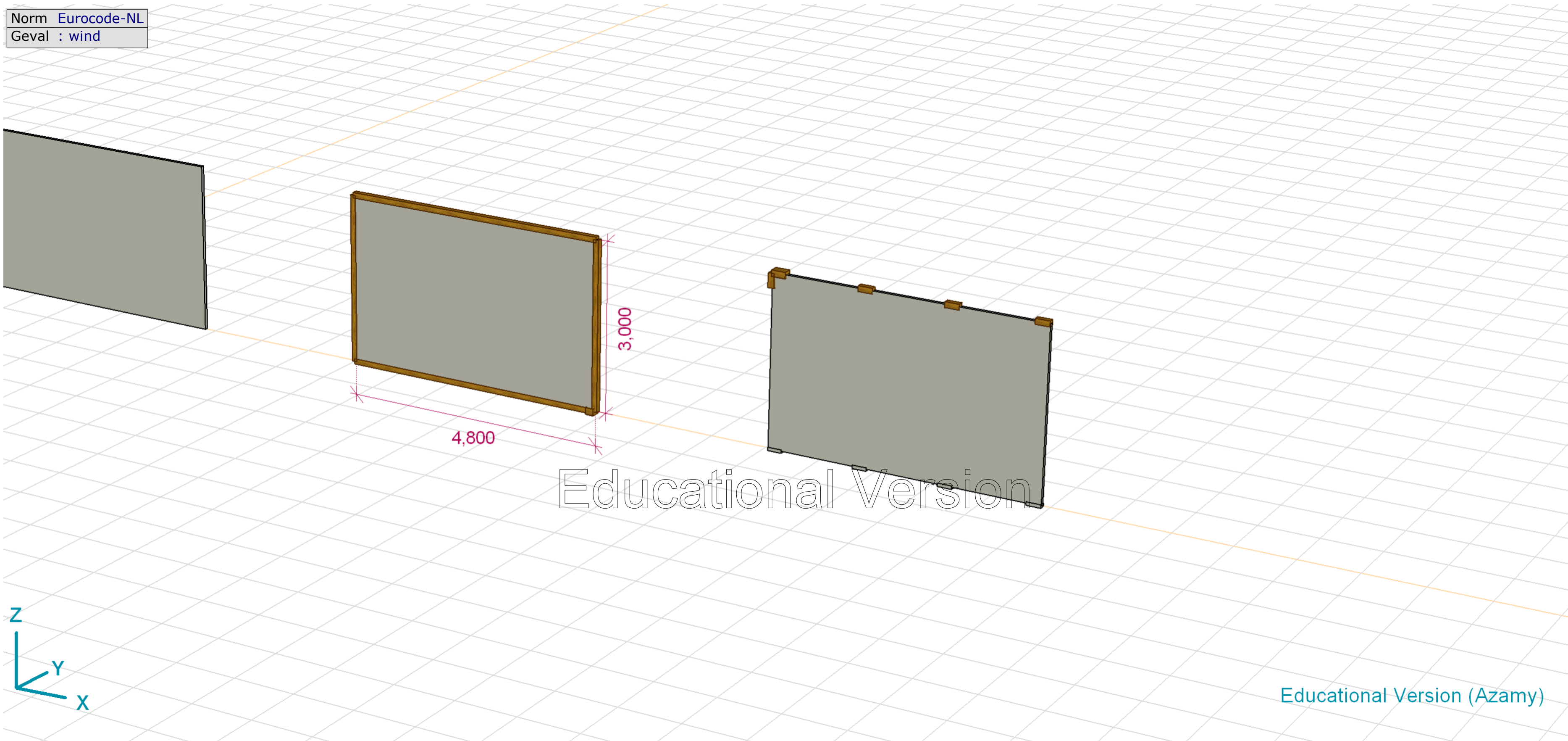
Supp	Fmin	Fmax	Mmin	Mmax
1	2.39	3.94	0.00	0.00
2	0.20	0.33	0.00	0.00
3	11.07	18.26	0.00	0.00
4	4.03	6.64	0.00	0.00

Project:

Constructeur:

AxisVM X6 R2j-hf1 · Geregistreerd aan Azamy
Glass facade+timber-3-1.axs

Rapport



Rapport Overzicht

Modelgegevens

Materialen

	Naam	*Type*	*Nationale norm*	*Materiaalnorm*	*Model*	*E_x [N/mm^2]*	*E_y [N/mm^2]*	*ν*	*α_T [1/°C]*	*ρ [kg/m^3]*	*Materiaal kleur*	*Contour kleur*	*Structuur*
1	Glass	Ander	Eurocode-NL		Lineair	70000	70000	0,23	7,7E-6	2500			–
2	C24	Hout	Eurocode-NL	EN 338:2009	Lineair	11000	370	0,20	8E-6	420			Wood 1
3	D50	Hout	Eurocode-NL	EN 338:2009	Lineair	14000	930	0,20	4E-6	750			Corn

	Naam	*P_1*	*P_2*	*P_3*	*P_4*	*P_5*	*P_6*	*P_7*
1	Glass							
2	C24	Zacht	$E_{0.05}$[N/mm^2] = 7400	G_{mean}[N/mm^2] = 690	f_{mk}[N/mm^2] = 24,00	f_{t0k}[N/mm^2] = 14,00	f_{t90k}[N/mm^2] = 0,40	f_{c0k}[N/mm^2] = 21,00
3	D50	Hard	$E_{0.05}$[N/mm^2] = 11800	G_{mean}[N/mm^2] = 880	f_{mk}[N/mm^2] = 50,00	f_{t0k}[N/mm^2] = 30,00	f_{t90k}[N/mm^2] = 0,60	f_{c0k}[N/mm^2] = 29,00

	Naam	*P_8*	*P_9*	*P_{10}*	*P_{11}*	*P_{12}*	*P_{13}*	*P_{14}*
1	Glass							
2	C24	f_{c90k}[N/mm^2] = 2,50	f_{vk}[N/mm^2] = 4,00	k_{cr} = 1,00				
3	D50	f_{c90k}[N/mm^2] = 9,30	f_{vk}[N/mm^2] = 4,00	k_{cr} = 1,00				

Naam: Materiaalnaam; **Type:** Type materiaal; **Model:** Materiaal model; **E_x:** Elasticiteitsmodulus in lokale x richting; **E_y:** Elasticiteitsmodulus in lokale y richting; **ν:** Poisson's verhouding; **α_T:** Warmteuitzettingscoëfficiënt; **ρ:** Dichtheid; **Materiaal kleur:** Materiaalkleur; **Contour kleur:** Contourkleur; **P_1, P_2, P_3, P_4, P_5, P_6, P_7, P_8, P_9, P_{10}, P_{11}, P_{12}, P_{13}, P_{14}:** Ontwerpparameter;

Veereigenschappen

	Naam	*Type*	*Vrijheidsgraden*	*Model*	*K*	*K_V*	*P_1*
1	Verend - translatie	N-N	translatie	Lineair	1E+0 kN/m	1E+0 kN/m	—
2	Vast - translatie	N-N	translatie	Lineair	1E+10 kN/m	1E+10 kN/m	—
3	Verend - rotatie	N-N	rotatie	Lineair	1E+0 kNm/rad	1E+0 kNm/rad	—
4	Vast - rotatie	N-N	rotatie	Lineair	1E+10 kNm/rad	1E+10 kNm/rad	—
5	Compleet - indirect	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = -1
6	Totaal - direct	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = 1
7	Vast	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = 0
8	Lineair 1E+7 kN/m	N-N	translatie	Lineair	1E+7 kN/m	1E+7 kN/m	—
9	Lineair 1E+8 kN/m	N-N	translatie	Lineair	1E+8 kN/m	1E+8 kN/m	—
10	Compleet - indirect_1	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = -1
11	Totaal - direct_1	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = 1
12	Vast_1	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = 0
13	Compleet - indirect_2	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = -1
14	Totaal - direct_2	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = 1
15	Vast_2	Kromrtrekken aansluiting	Kromtrekken	Lineair	—	—	WF = 0

Naam: Naam van de veereigenschappen; **Model:** Materiaal model; **K:** Initiële stijfheid; **K_V:** Trillingsstijfheid; **P_1:** Parameter;

Belastinggevallen

	Naam	*Groep*	*Groepstype*
1	DL	Dead load	Permanent
2	LL	Live load	Veranderlijk
3	wind	Wind	Veranderlijk

Naam: Naam belastinggeval; **Groep:** Belastinggroep; **Groepstype:** Belastinggroep type;

Gebruiker gedefinieerde belastingcombinaties uit belastinggevallen

	Naam	*Type*	*DL (Dead load)*	*LL (Live load)*	*wind (Wind)*	*Commentaar*
1	Co #1	UGT (a, b)	1,50	0	0	
2	Co #2	UGT (a, b)	1,50	0,83	0	
3	Co #3	UGT (a, b)	1,50	0,83	0,83	
4	Co #4	UGT (a, b)	1,50	0	0,83	
5	Co #5	UGT (a, b)	0,90	1,65	0	
6	Co #6	UGT (a, b)	0,90	1,65	0,83	
7	Co #7	UGT (a, b)	0,90	0	1,65	
8	Co #8	UGT (a, b)	0,90	0,83	1,65	
9	Co #9	UGT (a, b)	1,30	1,65	0	
10	Co #10	UGT (a, b)	1,30	1,65	0,83	
11	Co #11	UGT (a, b)	1,30	0	1,65	
12	Co #12	UGT (a, b)	1,30	0,83	1,65	
13	Co #13	BGT Karakteristiek	1,00	0	0	
14	Co #14	BGT Karakteristiek	1,00	1,00	0	
15	Co #15	BGT Karakteristiek	1,00	1,00	0,50	
16	Co #16	BGT Karakteristiek	1,00	0	1,00	
17	Co #17	BGT Karakteristiek	1,00	0,50	1,00	
18	Co #18	BGT Quasi-blijvend	1,00	0	0	
19	Co #19	BGT Quasi-blijvend	1,00	0,30	0	
20	Co #20	BGT Quasi-blijvend	1,00	0	0,30	
21	Co #21	BGT Quasi-blijvend	1,00	0,30	0,30	

Naam: Naam belastingcombinatie; **Type:** Type belastingcombinatie; **DL (Dead load), LL (Live load), wind (Wind):** Factor;

Domeinen

	Element type	*Materiaal*	*Ref$_x$*	*Ref$_z$*	*Dikte [mm]*	*k,buiging []*	*k,torsie []*	*k,afschuiving []*	*Oppervlakte [m^2]*	*Gat*	*Mesh*
1	Schaal	Glass	Auto	Auto	60				14,400	−	✓
2	Schaal	Glass	Auto	Auto	60				14,400	−	✓
3	Schaal	Glass	Auto	Auto	60				14,400	−	✓

Element type: Plaatelement type; **Ref$_x$:** Referentie voor lokale X-richting; **Ref$_z$:** Referentie voor lokale Z-richting; **k,buiging:** Buigsterkte coefficient; **k,torsie:** Torsiesterkte coefficient; **k,afschuiving:** Dwarskrachtsterkte coefficient; **Oppervlakte:** Domein oppervlak; **Gat:** Aantal gaten in domein; **Mesh:** Gegenereerde mesh;

Knoopopleggingen

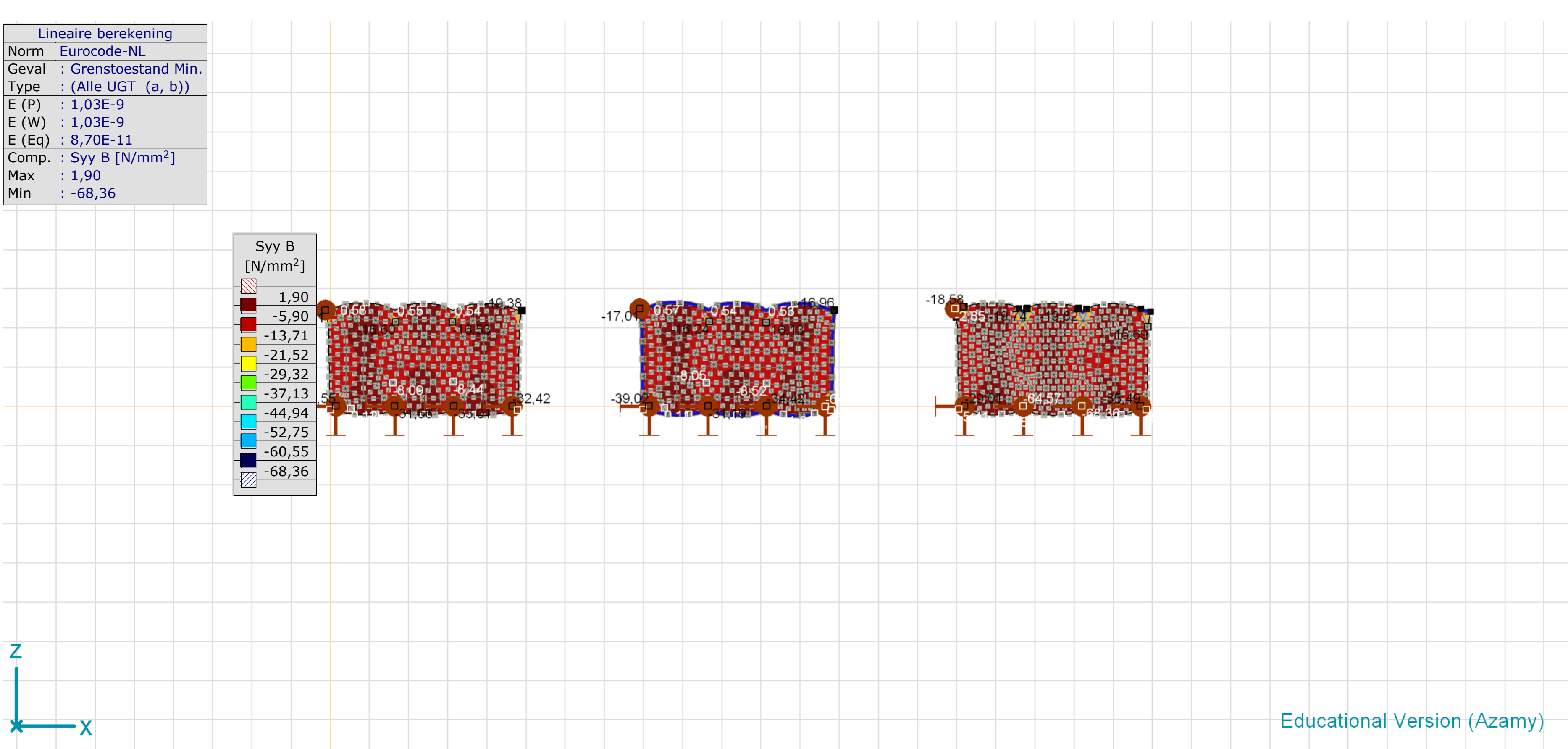
	Knoop	*X [m]*	*Y [m]*	*Z [m]*	*Type*	*Naam$_x$*	*K$_x$ [kN/m]*	*Naam$_y$*	*K$_z$ [kN/m]*
1	25	0	0	3,000	Glob.	—	0	Vast - translatie	0
2	7	8,000	0	3,000	Glob.	—	0	Vast - translatie	0
3	32	9,650	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
4	33	11,150	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
5	28	1,650	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
6	29	3,150	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
7	34	0,150	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
8	35	4,650	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
9	41	8,150	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
10	30	12,650	0	0	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+8
11	47	16,057	0	2,998	Glob.	—	0	Vast - translatie	0
12	57	17,707	0	−0,002	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
13	60	19,207	0	−0,002	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
14	44	16,207	0	−0,002	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10
15	46	20,707	0	−0,002	Glob.	Vast - translatie	1E+10	Vast - translatie	1E+10

Knoop: Ondersteunde knoop; **Type:** Opleggingstype; **K$_x$, K$_z$:** Initiële stijfheid;

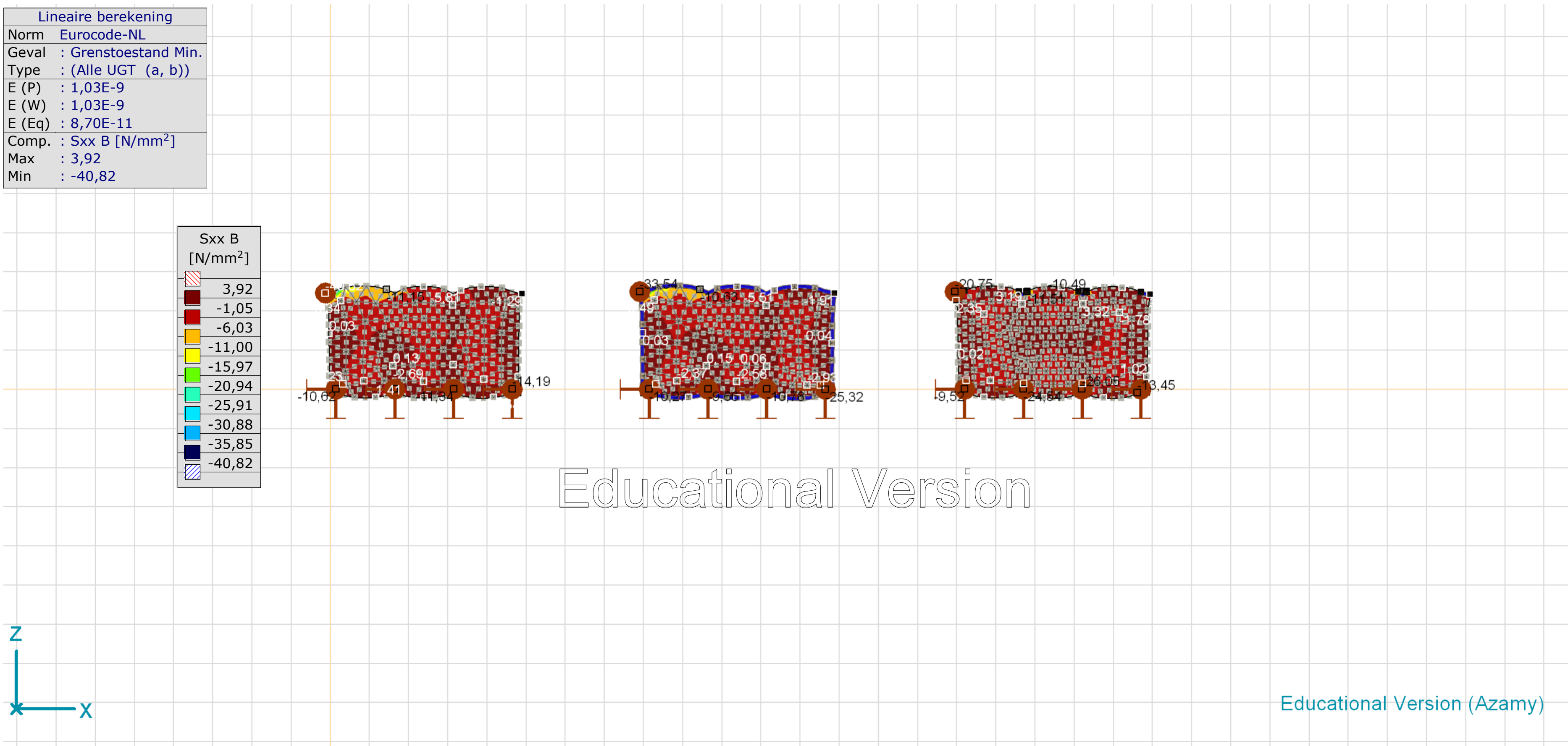
Gewicht per materiaal

	Materiaalnaam	*Σ G [kg]*
1	Glass	6494,160
2	D50	120,704
	Totaal	**6614,864**

Σ G: Totale massa;



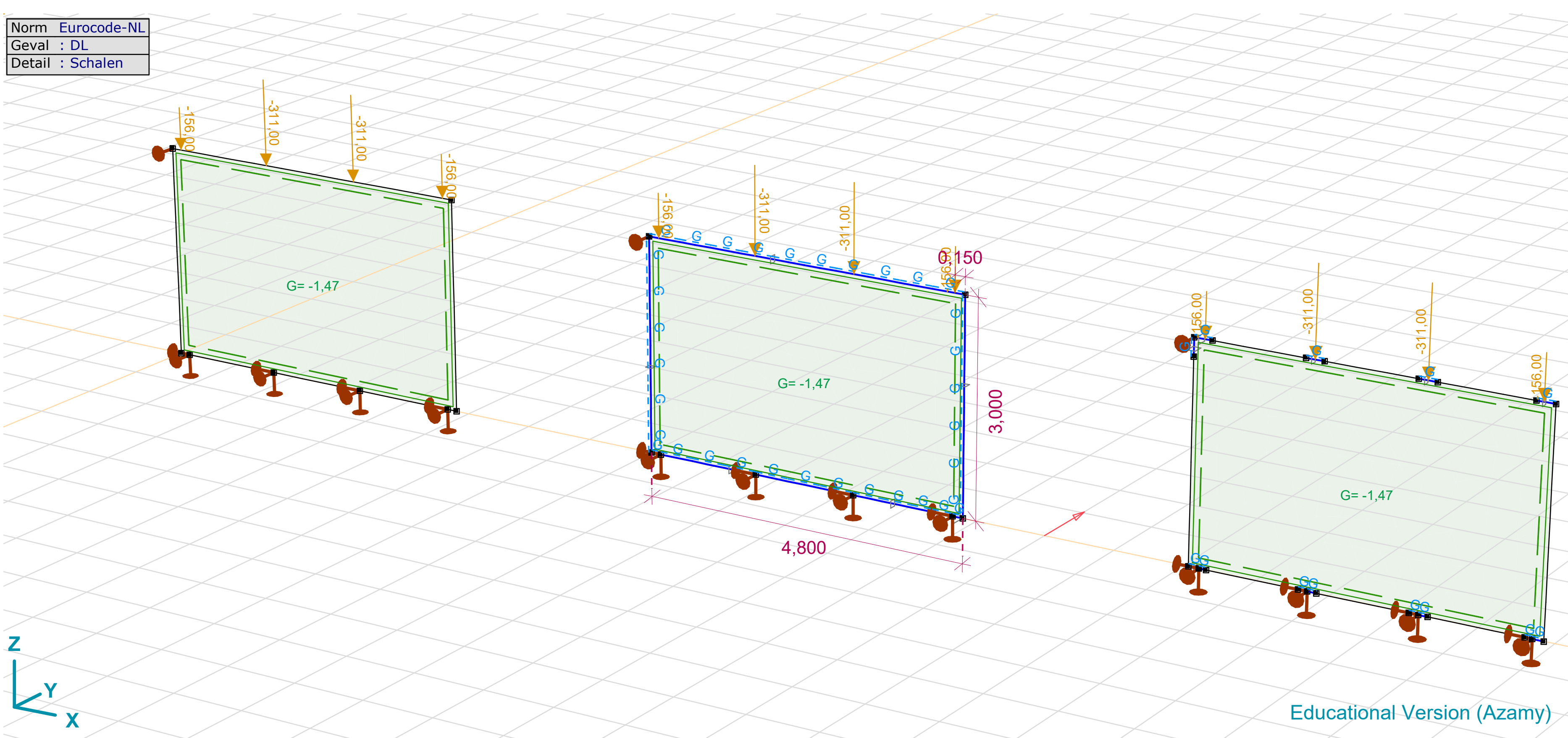
Syy B



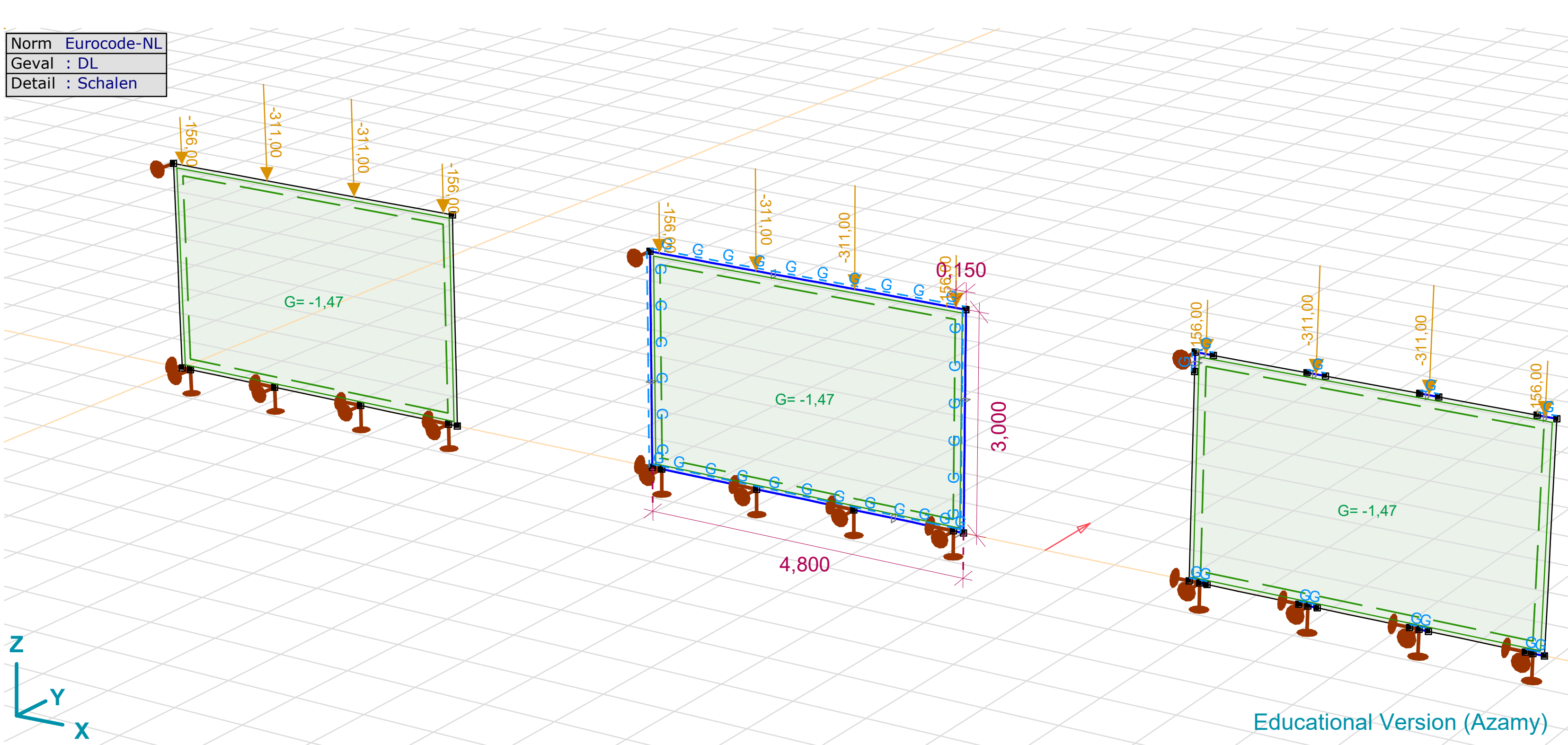
Vooraanzicht

Logische onderdelen

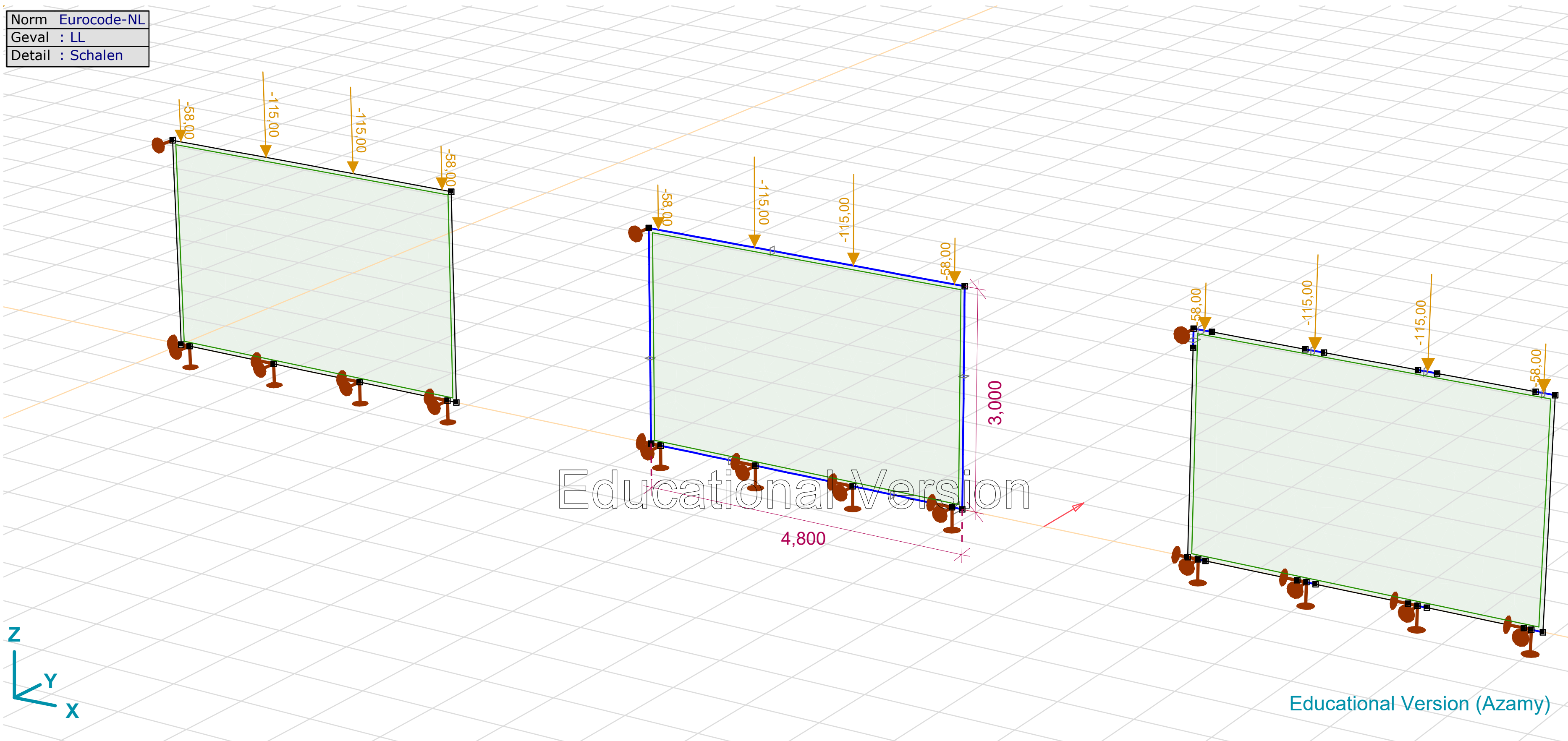
Domeinen



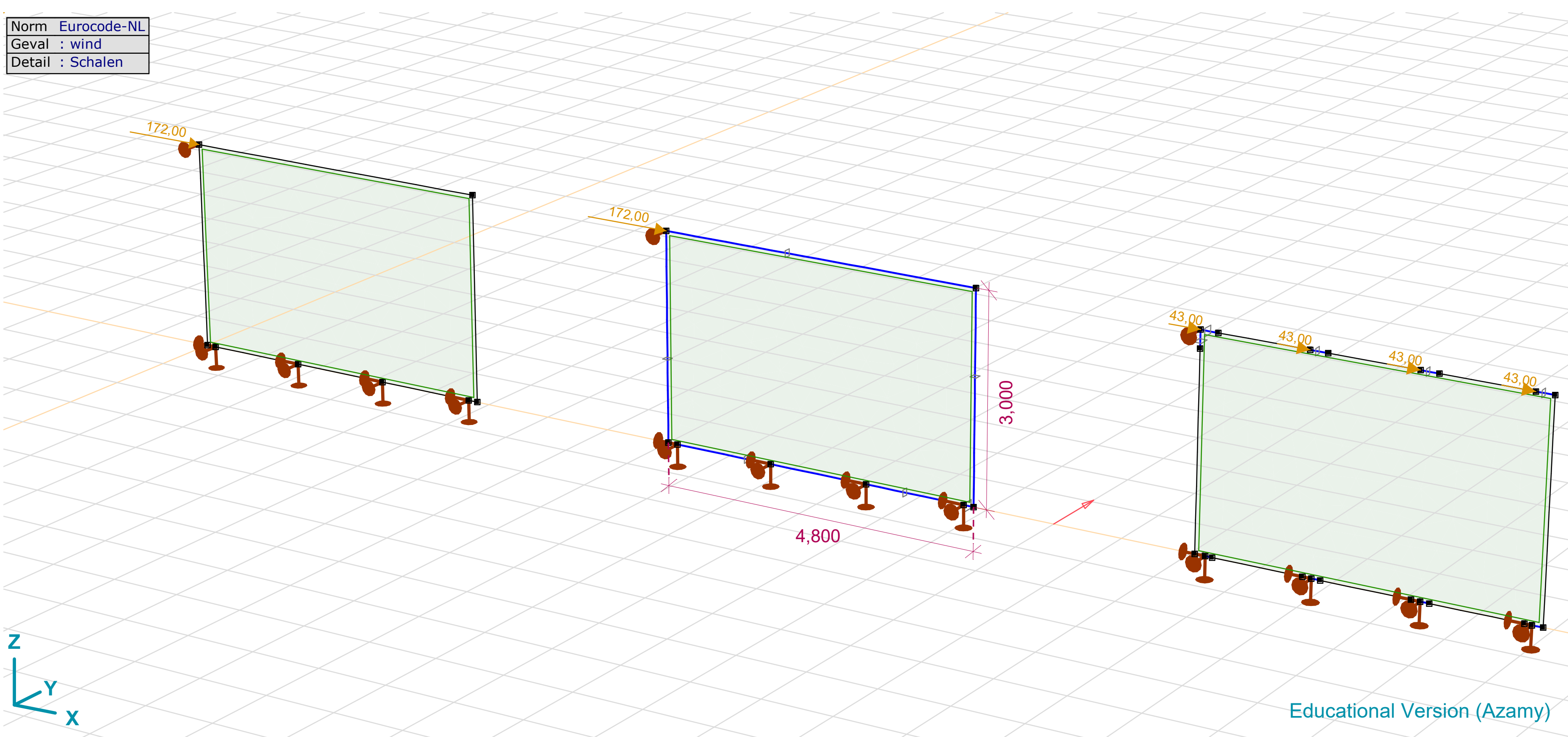
Rapport Domeinen



Rapport Domeinen, DL



Rapport Domeinen, LL



Rapport Domeinen, wind

Lineaire statische berekening

Interne krachten

Interne krachten knoopoplegging

Omhullende Min,Max

Alle UGT

Interne krachten knoopoplegging [Lineair, Omhullende (Alle UGT)]

Ext.	Knoop	X [m]	Y [m]	Z [m]	Type	C	min. max.	Geval	Rz [kN]
6	29	3,150	0	0	Glob.	Rz	min	Co #10	**−558,545**
9	41	8,150	0	0	Glob.		max	Co #7	**4,400**

Knoop: Ondersteunde knoop; **Type:** Opleggingstype; **C:** Extreme component; **min. max.:** Extreme type; **Geval:** Belastinggeval van de extreme; **Rz:** Z-component opleggingsreactiekracht;