



THE TIMBER SKELETON

Redesigning the roof structure of Notre Dame de Paris

How can historical Gothic construction principles, like integrated support structures and utilization during construction, be combined with modern timber production processes to design a new roof structure for Notre Dame de Paris?

NOUT BARENTSEN

ABSTRACT

Key words: Notre Dame de Paris, The Forest, Gothic architecture, Master Builders, structural expression, laminated veneer lumber, openwork

On April 15th, 2019 a devastating fire burned down Notre Dame de Paris and left the cathedral without its central spire and ancient gable roof, better known as *The Forest*. The required reconstruction initiated a global debate between architects, engineers, and other stakeholders. The discussion was put to an end by the French authorities, who proposed an identical reconstruction within a timeframe of 5 years. The decision forced the construction crew into a difficult job of finding identical construction materials, eventually leading to environmental and humanitarian suffering in Ghana, where a tropical underwater forest was felled. Besides, the time pressure put the quality of the reconstruction at risk.

In the presented thesis, an alternative design for Notre Dame's roof is proposed. First, the authorities' reconstruction plans are analyzed to see which practical challenges will have to be tackled. An alternative reconstruction strategy is then developed for the choir and nave of the cathedral. Inspiration is drawn from Notre Dame's initial construction in the 12th and 13th centuries, during which time pressure also played a key role in the design. The Master Builders' clever construction strategies like *integrated support structures* and *utilization during construction* are indispensable parts of Gothic engineering and are used to tackle current problems.

The proposed alternative construction process consists out of three phases: Fast Fix, Field Factory, and Former Finishing. The resulting structure, *The Timber Skeleton* – named after Professor Jacques Heyman's book: *The Stone Skeleton* -, consists out of stabilizing cassettes (Fast Fix), expandable roof segments (Field Factory), covered by a roof cladding with the same external appearance as before the fire (Former Finishing). Notre Dame's attic is imagined to function as a publicly accessible workshop, dedicated to the constant maintenance of the cathedral and all its Gothic ornamentation. As an alternative to the difficulties the authorities are facing in finding proper timber, modern construction materials and prefabrication strategies are investigated. They are utilized to facilitate a relatively quick construction without obstructing other on-site restoration works. Laminated veneer lumber with cross-ply is used to design openworked members with structural expression.

For the particular geometry of the rafters, the boundaries of current analytical engineering tools have been explored. Different verification strategies have been assessed and compared to analyze the structural functioning and feasibility of *The Timber Skeleton*. A *conventional stress analysis* is eventually used for the structural verifications of *The Timber Skeleton* and its most important joints. Recommendations are given for further development of the design, in which mainly the moment rigid connections of the rafters will require numerical and physical testing. A Master Builder's mentality of prototyping and trial and error will be needed to realize the proposed design.

The research shows how an alternative reconstruction strategy for Notre Dame de Paris could result in a good functioning structural design that pays homage to Gothic engineering principles. The design functions as an innovative example for how modern production processes could help to arrive at a contemporary interpretation of *The Forest* that functionally and structurally better fulfills the contemporary requirements.

ACKNOWLEDGMENTS

The design for Notre Dame de Paris' roof structure is the final project of my studies Structural Design / Building Engineering at the faculty of Civil Engineering and Geosciences at the Technical University Delft. The fire of 2019 created a unique opportunity for me, educated as both an engineer and architect. A challenge in which I could combine my interests in civil engineering and architecture and project my creativity and technical knowledge on this important Gothic masterpiece.

The cathedral had my special interest as I find Gothic buildings one of the prime examples of harmony between architecture and engineering, between form and structure. Notre Dame was therefore a great example to learn from. Besides, working with existing buildings and timber structures is becoming more important in modern engineering, in which sustainability considerations are challenging the established construction industry. The possibility of working with an ancient building and innovative timber structures was therefore a great way to increase my knowledge on these specific topics.

The development of the thesis had many challenges which I couldn't have tackled without others. Foremost, I would like to thank the members of my graduation committee Dr. ir. Karel Terwel, Dr.-Ing. Marcel Bilow, and Ir. Chris Noteboom, who have supported me in the research process. Their enthusiasm and criticism during the project have been of great help, especially during the periods that the COVID-19 pandemic forced everybody into their own little home bubble. I am especially grateful to Chris, who actively reached out at moments in which I was stuck in the complex research and design process.

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1. INTRODUCTION

The following chapter contains a brief introduction to the project: a description of the events that occurred during the fire on April 15th, 2019 (Figure 1), the resulting damage on Notre Dame de Paris, how this could have happened, and the debate around the reconstruction of the cathedral. Within the scope of the research, the fire and its development will only be explained briefly and the emphasis will be put on the resulting damage and the necessary reconstruction works. The chapter concludes with the research outline, a framework that will be used to develop a reconstruction strategy, a design, and its structural verification.

Figure 1 Smoke and flames rise during a fire at the landmark Notre-Dame Cathedral on April 15, 2019 (Stephane Rochon Vollet, 2019)



0.1 THE FIRE

April 15th, 2019 is a day that many Parisians will never forget when a devastating fire swept through the Notre Dame in the center of Paris and marked the cathedral forever. The Gothic masterpiece is the most beloved and visited monument in the world and is considered by many the heart and soul of Paris (The Local France, 2019).

As one might expect, the Notre Dame de Paris was supervised 24/7 and the dramatic events could therefore only happen due to a chain of human errors. Since the exact cause of the fire is still under investigation, the authorities have been reserved in providing publicly accessible information. Luckily the fire itself has been live recorded by both professionals and amateurs. In the aftermath, only a restricted amount of people were allowed to enter and capture the damage and reconstruction works with their cameras. These images, together with the detective-like research done by many journalists, results in a body of knowledge that offers a global overview of the fire and damage.

The ignition is very likely to be caused by temporary renovation works that took place on the central spire, where on April 15th in an early stage smoke and fire got detected (Figure 2). The required quick action during the growth phase of the fire failed due to a too complex fire detection system. Fuelled by the highly flammable carpentry in the attic of the Notre Dame, described by some as a tinderbox (Buchanan et al., 2019), the fire could spread rapidly from the central spire towards the four sides of the cruciform cathedral (Figure 3).

When the firefighters arrived more than half an hour after the first detection signals were received, the fire was almost impossible to extinguish. Firefighters entered the cathedral where they risked their lives in an attempt to control the flames and to prevent the fire from spreading to the two bell towers on the west side of the cathedral. The dramatic collapse of the central spire was the highlight of the inferno and it's been called a miracle that nobody died due to the fireball that was generated (Figure 4) (Peltier et al., 2019). The collapse of the central spire meant that also the vulnerable interior of Notre Dame got exposed to the flames. Thanks to the members of Paris's fire brigade the cathedral didn't completely collapse (Figure 5) (Peltier et al., 2019).



Figure 2 Within minutes flames were visible around Notre Dame's central spire (JBFAurora, 2019).



Figure 4 The fully developed fire causing the spire to collapse on top of the nave (NBC News, 2019).

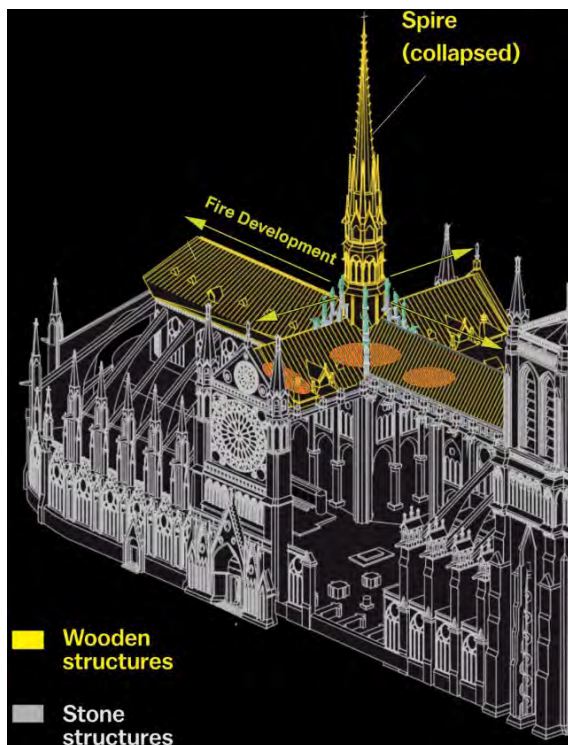


Figure 3 The fire spread rapidly to all four sides of the cathedrals attic (Vox, 2019).



Figure 5 The fire brigade could only control the fire after the complete carpentry roof burned down (New York Times, 2019).

0.2 THE DAMAGE

Using the brief analysis of the order of events in the previous section, a first impression of the caused damage can be portrayed. The presented footage, combined with publications and interviews with Paolo Vannucci, professor of mechanics at the faculty of Sciences at the University of Versailles, generate a fairly accurate overview of the fire's consequences. Attention will be given to the damage of Notre Dame's primary structure, even though the cathedral's vulnerable ornamentation: statues, paintings and stained glass windows, got heavily damaged too.

After the ignition at Notre Dame's central spire, the fire spread rapidly and within half an hour it burned down almost the complete roof structure, which consisted of more than 1000 massive wooden oak beams (Figure 6) Vannucci, who already investigated Notre Dame prior to the fire (Vannucci et al., 2019a), demonstrated the structural importance of the burned-down wooden gable roof and vaulted ceiling and argued that the structural integrity of Notre Dame could no longer be guaranteed (Vannucci et al., 2019b). The cathedral's existence got threatened even more by a large scaffolding structure that was used during the reconstruction works on the central spire, and that after the fire became dangerously unstable (Figure 7).

Further, Vannucci stated that the fire heated the limestone walls in the attic to a critical temperature. The subsequent cooling of the walls during the extinguishing of the fire caused significant carbonation of the limestone. It resulted in a decrease in the strength and stiffness of the limestone structure in the attic of the cathedral (Vannucci, 2020). The brick vaults, located directly underneath the attic, were directly exposed to the fire. Luckily a thick layer of mortar protected them from potential damage caused by high temperatures (Vannucci, 2020). It's generally accepted that the Gothic vaults indeed divide a cathedral into different fire compartments separating the cathedral's wooden roof structure and interior.

The protection of the Notre Dame's vulnerable interior by the vaults functioned perfectly until the central spire collapsed on top of the nave and destroyed the ceiling at two places, marked yellow in Figure 6 and visible from inside in Figure 8. A third hole in the vaulted ceiling can be observed in the north part of the transept, marked red in Figure 6.

The exact reason for this damage is still unknown. The holes in the ceiling of the cathedral meant that the interior of Notre Dame got subjected to flames and heat. As a result, the limestone columns on the ground floor got severely deteriorated, as shown in Figure 9. Moreover, a part of the 460 ton weighing lead roof covering fell into the cathedral and turned the interior into a poisoned environment (Peltier, et al., 2019). The entire interior and all stained-glass windows need to be cleaned and many Gothic statues require restoration.

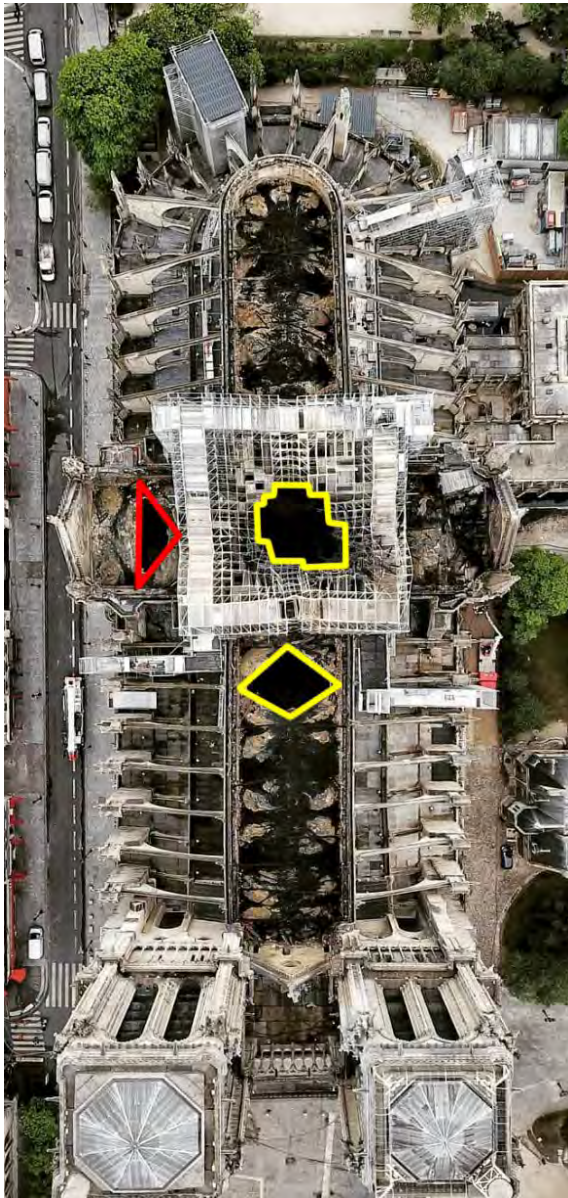


Figure 6 Topview of Notre Dame de Paris showing the scaffolding which surrounded the collapsed central spire, that left two holes in the vaulted ceiling of the nave marked in yellow. The reason for the existence of the third hole, marked red, is still unknown (R



Figure 7 Unstable, partly melted aluminum scaffolding structure (New York Times, 2020).

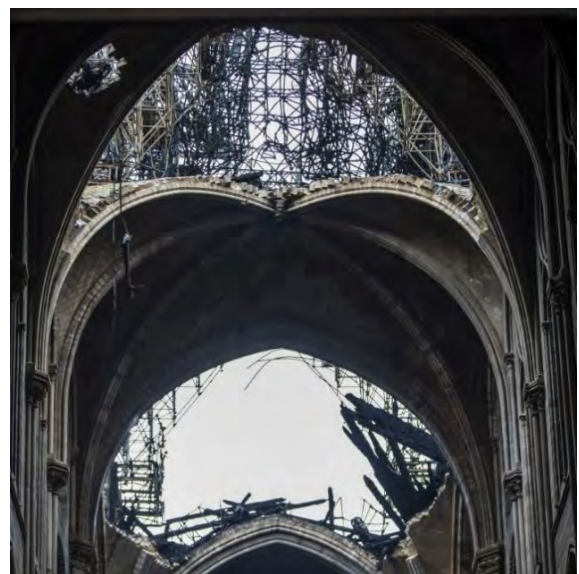


Figure 8 Two holes in the vaulted ceiling above the central aisle (New York Times, 2020).



Figure 9 Burned oak beams fell through the vaults and damaged the interior (New York Times, 2020).

0.3 THE RECONSTRUCTION UNDER DEBATE

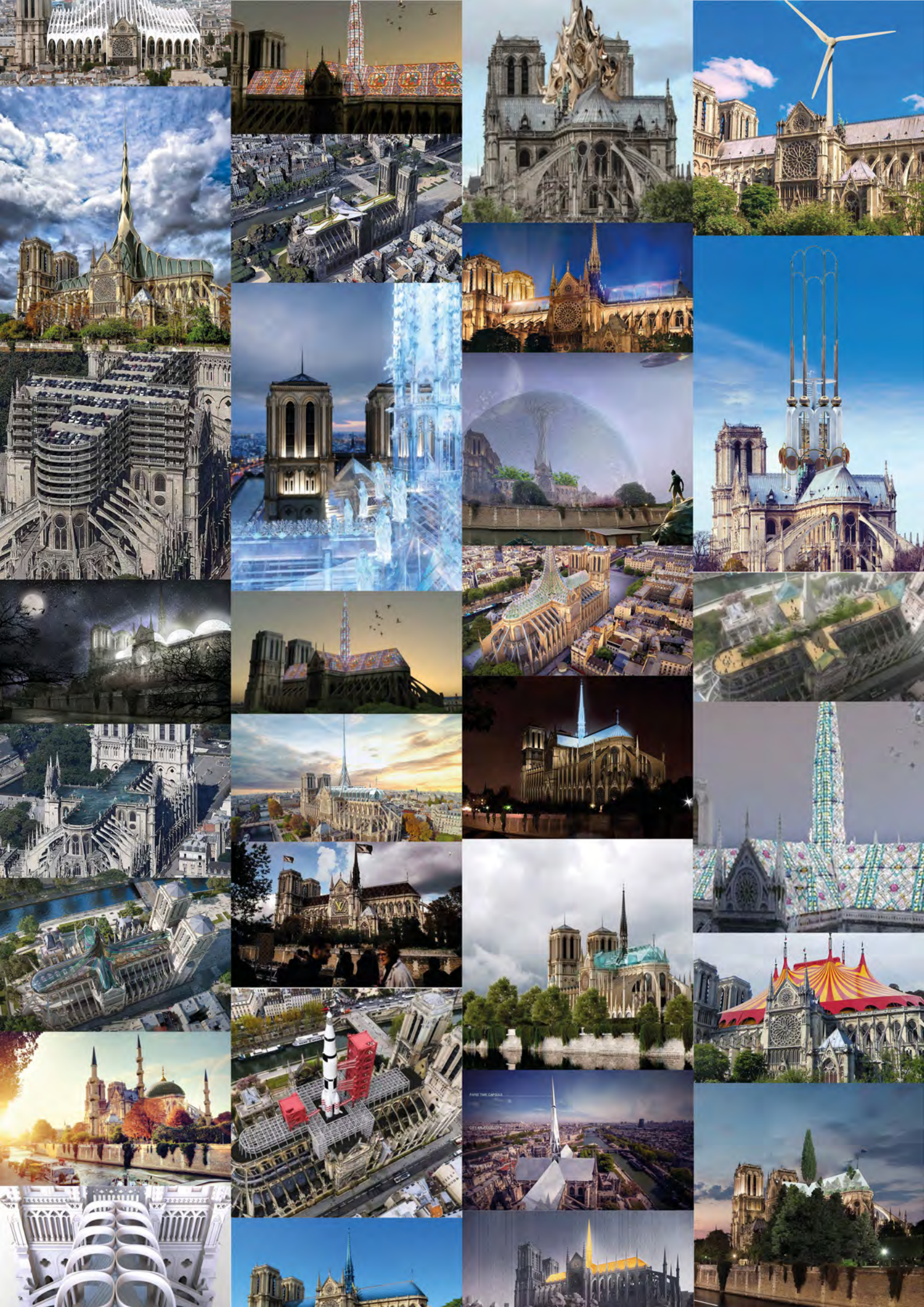
Design proposals by architects from all over the world fuelled an international debate on the reconstruction of Notre Dame de Paris (Figure 10). The discussion on the restoration of monuments, and the Notre Dame, in particular, is a re-emerging phenomenon.

The most recent significant restoration of Notre Dame dates back to the 19th century when the renowned architect and historian Eugene-Emmanuel Viollet-le-Duc executed his highly disputed modifications. Forced by the big leap in time between the 12th and 19th century, and the lack of medieval records, Viollet-le-Duc was arguably the first to approach the medieval cathedrals through their structural logic and construction methods, instead of their pictorial appearance (Powell, 2011). During his career, Viollet-le-Duc was highly criticized for his analyses and interpretation of history. Modern analytical tools enable us to argue that many of his technical assumptions and hypotheses were incomplete or wrong. At Notre Dame though, most of Viollet-le-Duc's 19th-century interventions are nowadays considered to be "original". The central spire and demonic statues, so-called gargoyles, have become part of the cathedral's identity.

The debate around what to conserve, restore or reinterpret is a complex and re-emerging discussion. When on the day of the fire French president Macron announced to rebuild Notre Dame "*more beautiful than ever before*", an international debate emerged, and architects and engineers from all over the world made proposals for the cathedral's reconstruction (Figure 10). A few months after the fire of 2019, the discussion was abruptly put to an end by French president Macron, who decided that Notre Dame would be restored as it was before the fire. While Viollet-le-Duc justified his "Gothic" designs through historical analyses and a fair dose of fantasy, it was time-pressure that led Macron to his decision to restore Notre Dame exactly as it was, as captured in his quote: "not delaying the reconstruction and making it complicated - things have to be cleared up quickly" (BBC, 2020).

After the authorities announced their wishes to quickly rebuild Notre Dame before the Olympics of 2024, many heritage experts warned for the problems of a fast identical reconstruction and stated that restoring the cathedral decently within five years would be unfeasible (McGivern & Kenney, 2019). Moreover, according to some experts, it is impossible to rebuild the original timber roof structure at all. The former wooden roof was built out of more than 1000 oak trees, each of them around 400 years old. Nowadays this material is simply not available (Smith, 2019). Although Notre Dame's exterior physical appearance can be restored, the cathedral will never be the same.

Figure 10 Proposals for Notre Dame's reconstruction, ranging from a parking lot to a stained-glass roof.



0.4 RESEARCH FRAMEWORK

Research Outline

The problem description in the previous sections contains only a brief introduction to the rich history, current debate, and complex reconstruction of Notre Dame de Paris. This thesis presents a new design for the cathedral's roof structure, which pursues to be a complete and integrated proposal, strongly rooted in historical and technical research. Due to the reconstruction work's diversity and complexity, the focus will be narrowed to the repetitive roof structure on top of the nave and choir, which together form the main axis of the cathedral.

During the research, it became clear that the conservative wish of the authorities to reconstruct an identical roof structure within the given timeframe of 5 years, led them into complex situations that are unnecessarily harmful to people and planet. An alternative reconstruction strategy is therefore developed that draws inspiration from Gothic engineering history while it allows technological progress to tackle current challenges. More specifically, the concepts of *integrated support structures* and *utilization during construction* will be the basis of a design that strives to be an homage to *The Forest* and that captures the structural expression of Gothic architecture. At the same time, the functioning of Notre Dame's attic is addressed and explored to let the cathedral not merely be a church and monument, but also a building that actively shares its rich history. The following main research question is formulated:

How can historical Gothic construction principles, like integrated support structures and utilization during construction, be combined with modern timber production processes to design a new roof structure for Notre Dame de Paris?

The main research question can be broken down into sub-questions, which are divided into three parts.

Part A: Focusing on Notre Dame's historical context and challenges in the current reconstruction.

- Which lessons can be learned from Gothic - and in particular Notre Dame's - engineering history?
- Who were the Gothic builders and which construction strategies enabled the erection of cathedrals?
- What can be learned from the reconstruction process executed by the authorities?

Part B: Proposal for an alternative reconstruction strategy and the development into a design, assessing the materialization, assembly, and project-specific.

- Which alternative construction strategy can be developed and what is its program of requirements during the different building phases?
- Which materialization, manufacturing methods, assembling strategies, and joint types can be utilized?
- How can the construction strategy be translated in a global design, and what are its execution challenges and point of attention for the structural verification?

Part C: Structural verification and feasibility of the design.

- What is the structural behavior of *The Timber Skeleton*, and what are its critical points?
- Which verification strategies can be used to check the stresses in the Timber Skeleton's rafters?
- What is the design's feasibility, which alterations are advisable, and which conclusions can be drawn?

HOW CAN HISTORICAL GOTHIC CONSTRUCTION PRINCIPLES BE COMBINED WITH MODERN TIMBER PRODUCTION PROCESSES TO DESIGN A NEW ROOF STRUCTURE FOR NOTRE DAME DE PARIS?

PART A

NOTRE DAME IN ITS HISTORICAL CONTEXT (Ch.1)
GOTHIC
NOTRE DAME DE PARIS
THE FOREST

MEDIEVAL HIGH TECH (Ch.2)
A MASTER BUILDER'S SCIENTIA
CONSTRUCTING A CATHEDRAL

RECONSTRUCTION BY AUTHORITIES (Ch.3)
STABILISING NOTRE DAME
DEBRIS CLEANING
RECONSTRUCTING THE FOREST
A CONSTRUCTION SITE IN PARIS' HEART

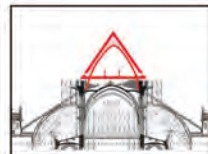
HISTORIC RESEARCH
LITERATURE STUDIES,
ANALYSES AND INTERVIEWS

PART B

CONSTRUCTION STRATEGY (Ch.4)
LEARNING FROM THE MASTER BUILDERS
PROPOSAL CONSTRUCTION STRATEGY



FAST FIX



FIELD FACTORY



FORMER FINISHING

PROGRAMME OF REQUIREMENTS PER PHASE

HIGH TECH TIMBER (Ch.5)
ENGINEERED TIMBER (LVL)
MANUFACTURING BAUBUCHE
LVL PREFABRICATION AND CONNECTIONS

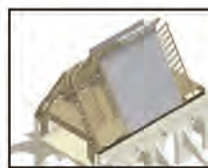
GLOBAL DESIGN AND MANUFACTURING
OF THE TIMBER SKELETON (CH.6)



FAST FIX



FIELD FACTORY



FORMER FINISHING

CONCEPT DEVELOPMENT AND DESIGN
DESIGNING USING SKETCHES, PHYSICAL AND DIGITAL
MODELLING, CALCULATIONS AND MATERIAL RESEARCH

PART C

STRUCTURAL VERIFICATION (Ch.7)
STRUCTURAL DESIGN WITH LVL
LOADS AND LOAD-COMBINATIONS
STRUCTURAL ANALYSES GUTTERING WALL
FAST FIX
FIELD FACTORY

JOINT DESIGN (Ch.8)
ROOF RIDGE HINGE
INTERMEDIATE MEMBERS
KINEMATICAL JOINTS PURLINS

DISCUSSION AND RECOMMENDATIONS (Ch.9)

CONCLUSIONS AND REFLECTION (Ch.10 & Ch.11)

STRUCTURAL VERIFICATION
3D COMPUTER MODELS IN SCIA
ENGINEER, HAND CALCULATIONS,

Methodology

The structure of the research can be divided into three phases as can be seen in the scheme on the previous page. The thesis has been composed with the belief that image is as important as text, and that both can capture particular aspects of the research and design process. This is reflected through the extensive use of photos and illustrations to clarify and enrich the research.

Part A covers (recent-) historical research during which literature studies and interviews are used to develop an understanding of Notre Dame with its ingenious timber roof structure (Ch.1), and its designers (Ch.2). The records of design and construction of most of the Gothic cathedrals from the 12th and 13th centuries are almost non-existent. The cathedrals themselves are therefore the most significant sources of information and supported by new analysis methods (Figure 11) archaeologists, engineers and architects are unraveling their construction methods (Sandron & Tallon, 2013). The COVID-19 pandemic made site visits difficult, leading to a high dependency on photos and videos made by authorities, journalists and other third parties. Part A ends with Notre Dame's recent history and current reality – in august 2021 - by describing the reconstruction steps executed by the authorities (Ch.3).

Part B is used to develop a new design for Notre Dame's roof. First, the conclusions and interpretations of Part A are used to develop an alternative reconstruction strategy (Ch.4). To deal with the practical and time-dependent challenges in the production and assembly, research is presented about the possibilities of modern timber structures (Ch.5). Accompanied by analyses, observations, and interviews with manufacturers, knowledge of the chosen building materials is generated and translated into a program of requirements and relevant constructional boundary conditions. Part B is concluded with a description of the final design, its premises, and assembly procedures (Ch.6). The design has been organically developed through sketches, physical modeling, hand calculations, and material research. The final result is presented through technical drawings and a physical model.

Part C will be completely devoted to the structural verification of the proposed design (Ch.7). Calculations are mainly based on Scia Engineer 3D computer models which are accompanied by hand calculations in Microsoft Excel. The design of the timber structure's rafters appeared to be challenging. A large appendix is therefore added in which different verification methods are explained. The *conventional stress analysis* has been selected to design *The Timber Skeleton* as it turned out to be the most appropriate verification method. Next, a structural analysis of respectively the hinging joint in the roof's ridge and the connection within the rafters is presented (Ch.8). Part C ends with a few chapters in which the structural verification is discussed and recommendations are being made to improve the design. The historical research, development of the design, and structural design provide much new insight. Hence, the last part of the thesis will conclude with a few conclusions and a reflection upon the entire research. Although maybe the best answer to the main research question is the design itself.

During the development of the design, the relationship between engineering and architecture, between structure and form, has continuously been questioned. The aesthetics of structures has been translated into a wish for structural expression in the design, for which the following description by Tzonis (2004) will be used: *"Division of mechanical labor between the various components of a structure results in the articulation of individual elements, and distribution of material within each element results in an elaboration of the profile of the structural elements"* (Tzonis, 2004). From Part A it will become clear that both the *articulation of individual elements* - by the concentration of forces - and the *elaboration of the profile* - by removing unnecessary material – can be considered important Gothic design principles. They also have been interpreted and applied by numerous contemporary engineers and architects, which frequently led to distinctive exposed structures. The white steel structure of Centre Pompidou, located in the direct vicinity of Notre Dame de Paris, is a fine example of what can be considered as structural expression (Figure 12).

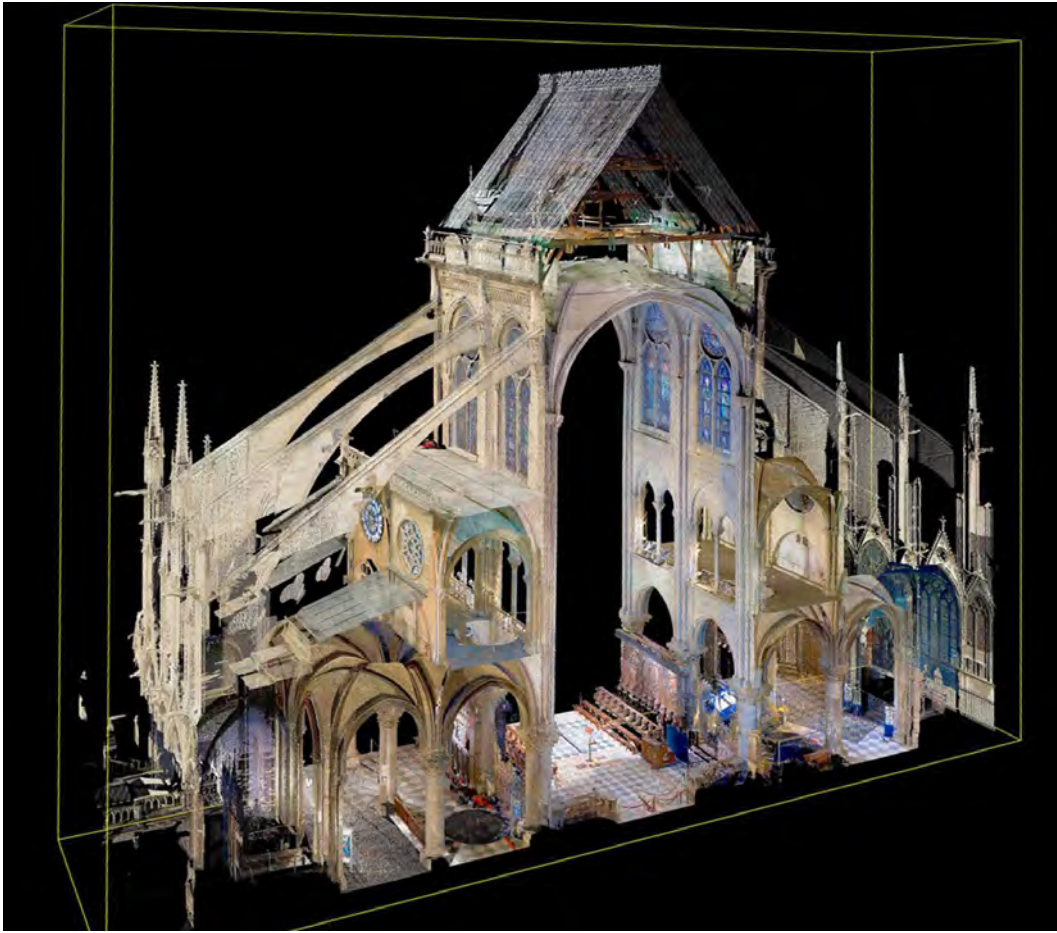


Figure 11 Andrew Tallon's 3D scan of Notre Dame's transversal section (Tallon, 2014)



Relevance Of A Unique Case Study

The presented research and design draw inspiration from, and projects upon, Notre Dame de Paris. The uniqueness of the cathedral as a case study delivers highly specific knowledge which is both a strength and a weakness considering the relevance of the research. The research's broader relevance lies mainly in the study of ancient engineering and the argumentation of appropriate construction methods in dealing with ancient monuments. Gothic engineering history is poorly documented a therefore subject for a vivid discussion between archeologists, architects and engineers. Modern technology has enabled new insights into medieval construction methods which can provide solutions to challenges in the building industry nowadays. With the vast amount of monuments and other important symbolic buildings worldwide, reconstruction and maintenance will become increasingly more important. Also from a sustainability point of view, dealing with existing buildings will become more and more relevant in the building industry.

I believe that beauty in the building environment arises partly from a harmonious convergence in the design; a building should be more than a summation of solutions and decisions. This endeavor is becoming increasingly challenging due to a building industry that has become more and more specialized. The gap between structure and design has been widening already since the Renaissance, resulting in a separation between architecture and engineering. It created the possibility of a scientific approach to structural problems resulting in safer and more economic structures. It became possible to design buildings from two different perspectives, with an emphasis on structure or aesthetics. But the fragmentation of disciplines also frequently results in conflicting interests which require new integrated approaches to designing (Blockley, 2014). The presented project is an example of such an integrated design method, in which aesthetical, structural, and execution requirements are united in one design.

The rise of Gothic architecture in the 12th and 13th centuries was a result of a vast increase in technical knowledge and skills. Key in this development was the tight interrelationship between the different domains Art, Science, Engineering, and Design. This interrelationship, as Neri Oxman describes it in her Krebs Cycle of Creativity (Figure 13), could be seen as creative energy: the output of one domain becomes the input of the others. The convergence of the different domains in Gothic architecture makes the cathedrals a great example and inspiration for collaboration. The relevance of Gothic architecture and Notre Dame de Paris for the modern construction industry lay therefore in the harmony between the different domains, as a result of the *Master Builder design approach*.

Figure 12 Gothic gargoyles of Notre Dame de Paris, overlooking Centre Pompidou by Renzo Piano, Richard Rogers, in collaboration with Arup

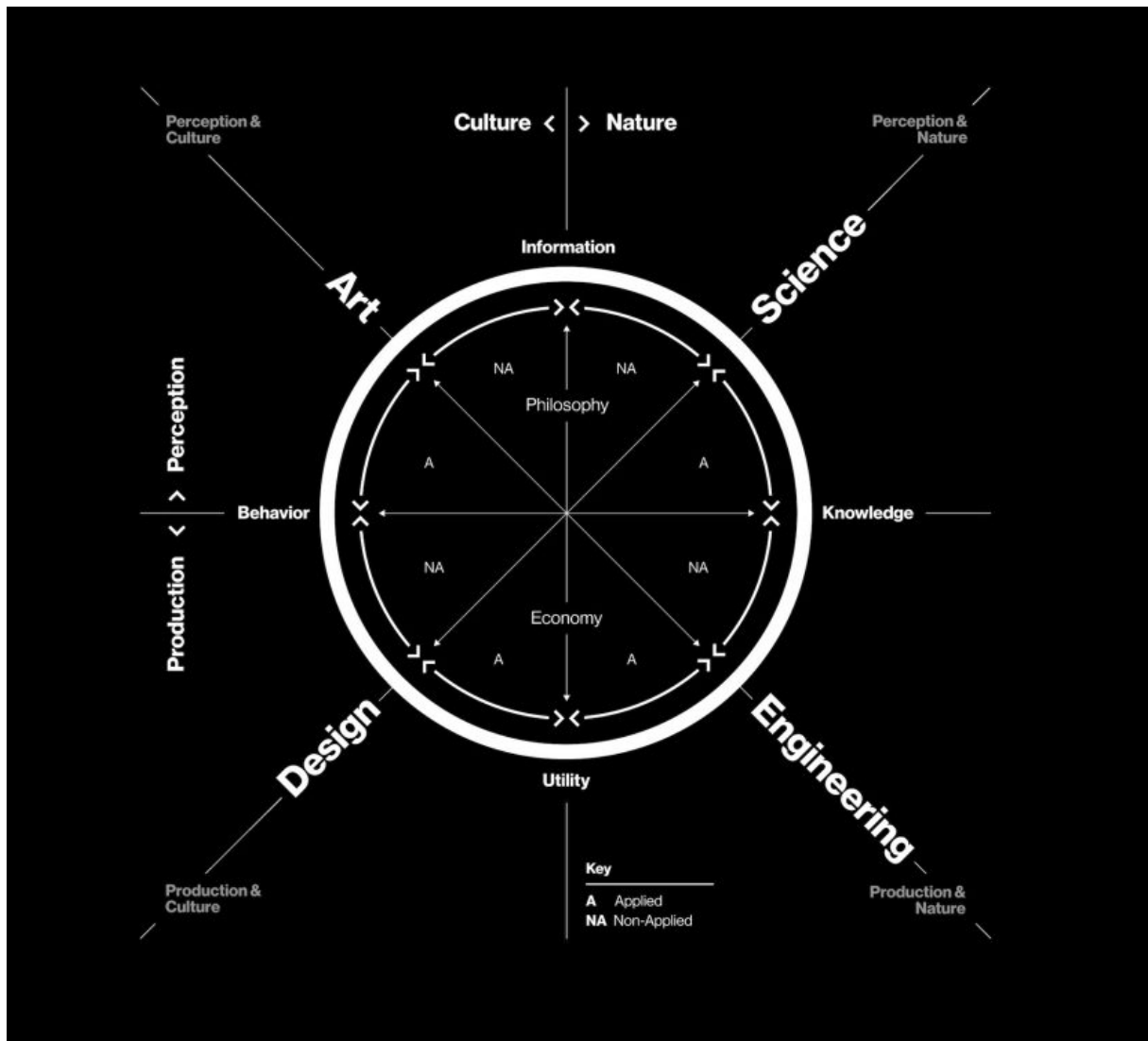


Figure 13 The Krebs Cycle Of Creativity, by designer, architect, and MIT Media Lab faculty member Neri Oxman, in the *Journal of Design and Science (JoDS)* (2016).

“If you want to be original, start at the origin.”

Sjoerd Soeters (2020)

1. NOTRE DAME DE PARIS IN ITS HISTORICAL CONTEXT

To recognize the challenges and opportunities in reconstructing the roof of Notre Dame, one should first understand the cathedral's unique place in history during the Gothic era. This chapter will start by describing the social and technical developments that facilitated the erection of such complex stone structures. Without belittling the religious motivation, the Gothic cathedrals will be approached as a product of a non-religious intellectual movement, with a primary focus on technological reasoning. Subsequently, the position of Notre Dame de Paris within the Gothic movement and the former timber roof structure of the cathedral, *The Forest*, will be discussed.



1.1. GOTHIC ENGINEERING

Gothic's Societal Conditions

The exact starting point of Gothic architecture is still under debate. It can be generally stated though that a few buildings erected in the north of France between 1140 and 1160 are the first relevant Gothic examples since all the characteristic features of the style are present in their structure (Pevsner, 1963). Although Gothic cathedrals were built until the 16th century, Jacques Heyman in his renowned book *The Stone Skeleton* stated that the collapse of the Beauvais cathedral in 1284 marked the end of the greatest period of cathedral building, “after which work never achieved the structural quality of that of the thirteenth century” (Heyman, 1966).

These about 140 years are generally accepted to be the Early Gothic period, a time in which society in western Europe changed drastically. Medieval cities grew rapidly, new social classes started to emerge, while the Catholic church, already the dominant power in Europe since the fall of the Roman Empire, controlled a big part of everyday life. In this context, taxes and resources of wealthy merchants were gathered into campaigns to construct buildings to embody religious power and to create new extravagant places of worship, cathedrals. The social and religious communities in different cities in Northern France competed over power and prestige, which initiated a fast increase in the size and height of cathedrals (Figure 14).

The realization of a cathedral required a lot of resources and extraordinary skills. The latter was often possessed by Master Builders who had *Scientia*, a body of knowledge that was empirically obtained through observations of existing buildings and experience in designing complex structures (Huerta, 2012). The master-builder led a team of skilled workforce, probably the first highly skilled and well-paid laborers erecting monuments in history (Mark and de Sampaio Nunes, 1982). Still, it is argued that material costs easily exceeded labor costs and that necessary cutbacks often resulted in a design that required less material and not necessarily less construction time (Fitchen, 1981).

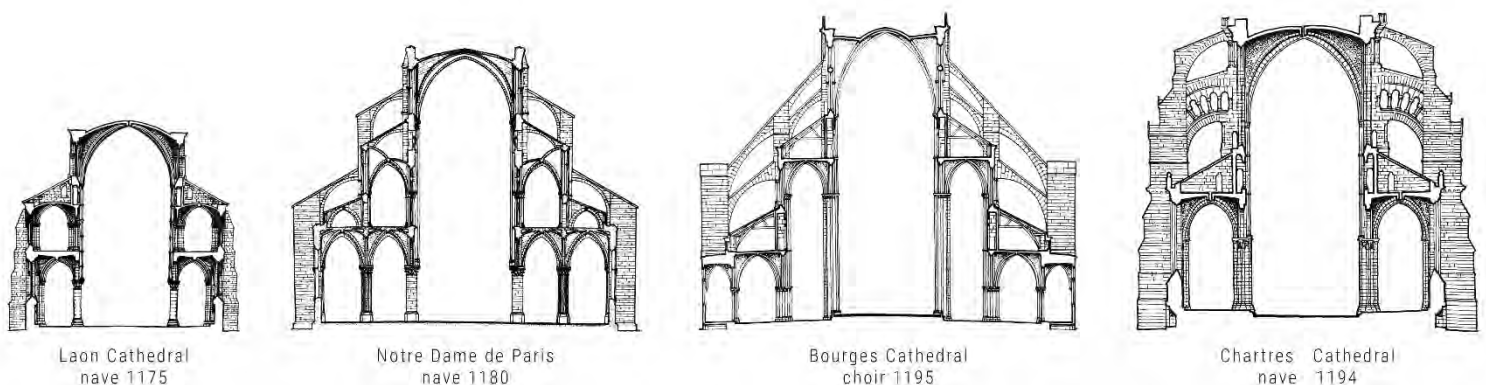


Figure 14 Evolution of cathedrals in the Early Gothic (12th and 13th century) in Northern France (Heyman, 1997).

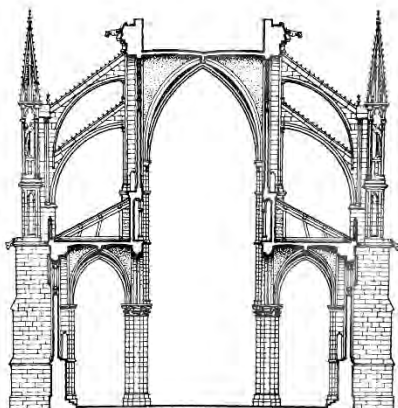
High Light

The wish to build cathedrals, which were significantly bigger structures than mankind had ever built before, was accompanied by a range of technical challenges. The aesthetic ambition to have places of worship bathed in daylight, made the construction of a cathedral even more complicated, since preceding examples in history, as the dark Roman churches, couldn't be used as an archetype. Moreover, the massive Roman building style would have been too heavy for the foundations of the much bigger medieval cathedrals. The combination of technological challenges and aesthetic wishes, forced the Master Builders to be creative, resulting in a range of inventions that are nowadays seen as the characteristics of Gothic architecture.

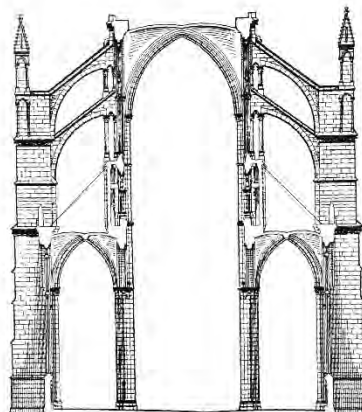
Force Flows

By looking at the path of forces through a Gothic stone structure, an initial overview of the structural challenges and their solutions can be generated. One of the leading design principles of the medieval builders was to concentrate the vertical forces that act in the structure. By doing so, the structure could be executed significantly lighter and parts of the walls could be left out and substituted by stained glass windows. Relatively big spans were created by using vaulted ceilings, while their self-weight induced vertical forces could be concentrated at specific points, where columns carried the vertical forces down to the foundation.

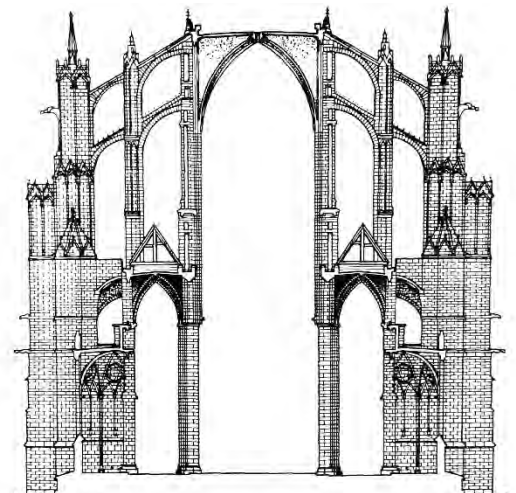
What was of bigger concern for the Master Builders were the horizontal forces acting in a cathedral. The Master Builders were aware of, and experienced with, the horizontal thrust forces which originated from a vaulted ceiling's geometry. The significantly increased heights of the cathedrals though resulted in unprecedented lateral forces, induced by higher wind speeds (Mark, 1990). As mentioned before, using thick walls to create lateral stability wasn't an option and more localized lateral supports, in the form of buttresses, would block out too much sun- and daylight. This problem led to one of the most striking inventions of the Gothic era: the flying buttress. The flying buttress evolved as the cathedral's heights and widths increased and Master Builders became more familiar with the pros and cons certain configurations had for structural performance and maintenance (Figure 14).



Reims Cathedral
nave 1210



Amiens Cathedral
nave 1220



Beauvais Cathedral
choir 1225

Medieval Masonry

The structural design and integrity of the Gothic structures were heavily influenced by the material used: granite, limestone, and sandstone, which were favored due to their general availability, relatively easy workability, and robustness. Heyman (1966) argues through calculation on a prismatic sandstone column loaded by self-weight, that compressive forces in cathedrals have such low values that the strength of a stone structure is negligible. The different stone materials used are very strong in compression but are relatively vulnerable to tension forces. Moreover, the tensile strength of the pre-modern mortars that were usually applied in medieval masonry, were even two orders of magnitude smaller than the compressive strength of the stones (Mark, 1990). A cathedral's stone structure can therefore be assumed to have plenty enough compressive strength and no tension strength (Heyman, 1966), (Stefanou et al., 2015), (Vannucci et al., 2019).

The low nominal stress in a cathedral's stone skeleton is a necessary condition for its survival through the centuries, it is not sufficient though. It is the shape of the stone skeleton that's governing the stability of a cathedral (Heyman, 1998). In an inappropriate geometry, the line of thrust can enter outside the middle third of the cross-section, leading to tension forces inside the vulnerable masonry. Although the failure of the structure only occurs if the thrust line ends up outside the entire cross-section, minor tension forces can already have a considerable influence on the durability of a stone structure. Robert Mark (1982) for example, showed how local tension forces in Amiens cathedral caused microcracks in the mortar resulting in high maintenance costs (Mark and de Sampaio Nunes, 1982).

The Master Builders understood the importance of avoiding these small tension forces in stone structures but were not always able to prevent them from occurring. A strategy to avoid tension forces in the masonry was to add weight (material) on top of the structure, leading to the invention of the pinnacle: often richly decorated pointed spires on top of buttresses. Much has been written about the structural purpose of pinnacles, which are generally considered to change the line of thrust's shape. Mark noted however that many pinnacles are positioned on the very edge of the buttresses which doesn't make sense considering the line of thrust. Instead, Mark investigated how the weight of pinnacles could act as a prestressing load. At Amiens Cathedral, he proved how the pinnacles would prestress the masonry to prevent tension forces to deteriorate the structure (Figure 15).

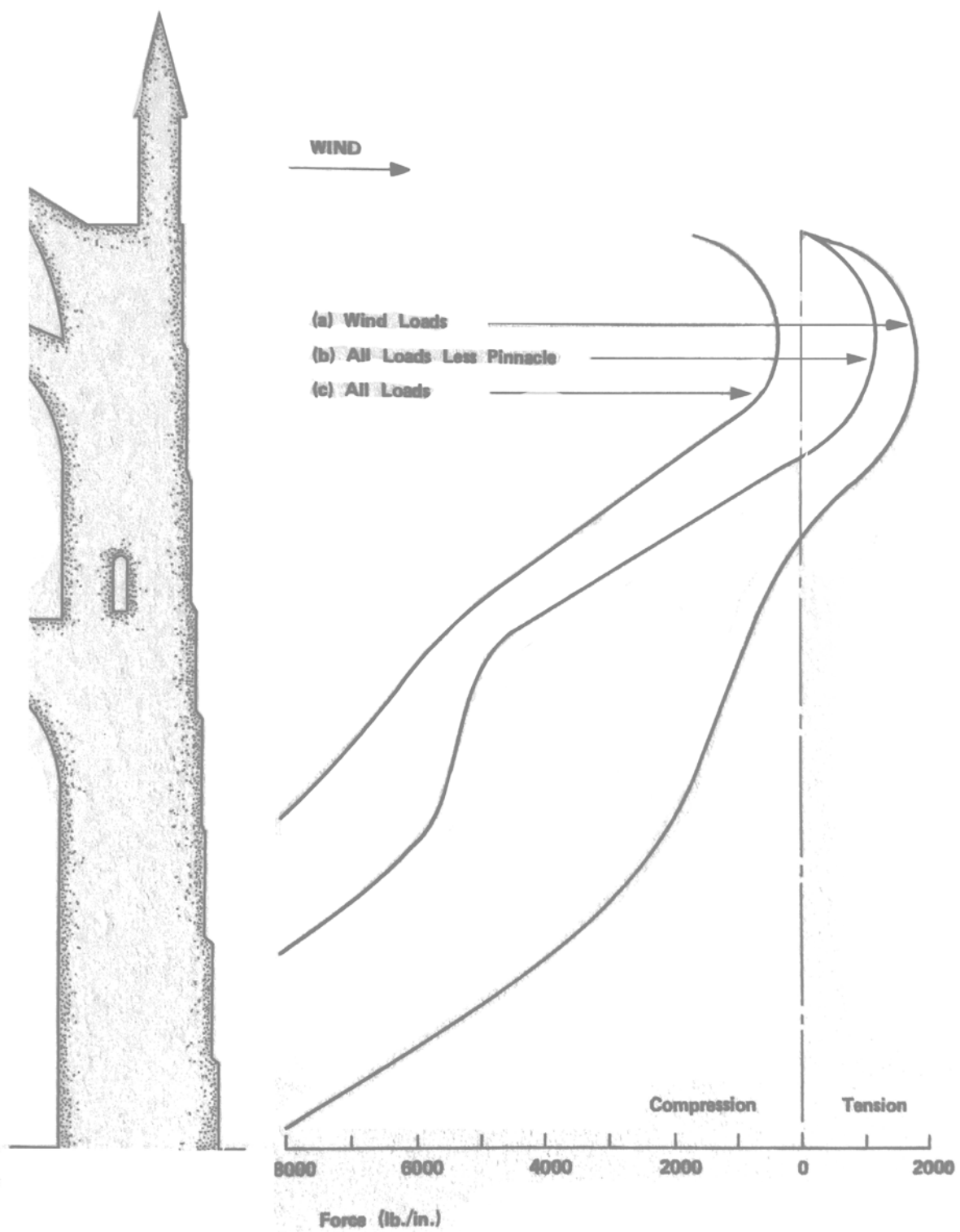


Figure 15 Robert Mark's analysis of Amiens Cathedral subjected to wind load with only wind load, all loads without pinnacles, and all loads with pinnacles (Mark 1984).

1.2. NOTRE DAME'S STONE SKELETON

Constructing The Cross

Notre Dame de Paris certainly fulfilled an important role within the Early Gothic era in Northern France. During its first 200 years, one could say that the cathedral was subject to a process of trial and error and constant improvements which resulted in minor and major alterations. Although the cathedral has always been an important symbol of Paris, Notre Dame's vulnerable stone skeleton has been in constant threat of neglect.

Starting around 1160, the Bishop of Paris initiated the construction of a cathedral at Île de la Cité, at the very heart of Paris. The proximity of the Seine river guaranteed an easy supply of building materials and the existing religious buildings at the site could be demolished and their foundation appropriated. The construction started at the east end of the building site, where the choir was built within approximately 20 years. Around 1180 the cathedral was already utilized for religious gatherings while construction on the nave continued until around 1220. The main cruciform geometry of Notre Dame got completed in 1250, when the cathedral's western towers were finished (Sandron and Tallon, 2013).

Right after the main body of the cathedral was built additions and modifications started to take place, changing the appearance of Notre Dame significantly. Powerful people left their mark on the religious building by adding statues or even whole chapels. Moreover, the Notre Dame evolved in coherence with the Gothic building style into a cathedral with an increasing amount of decoration.

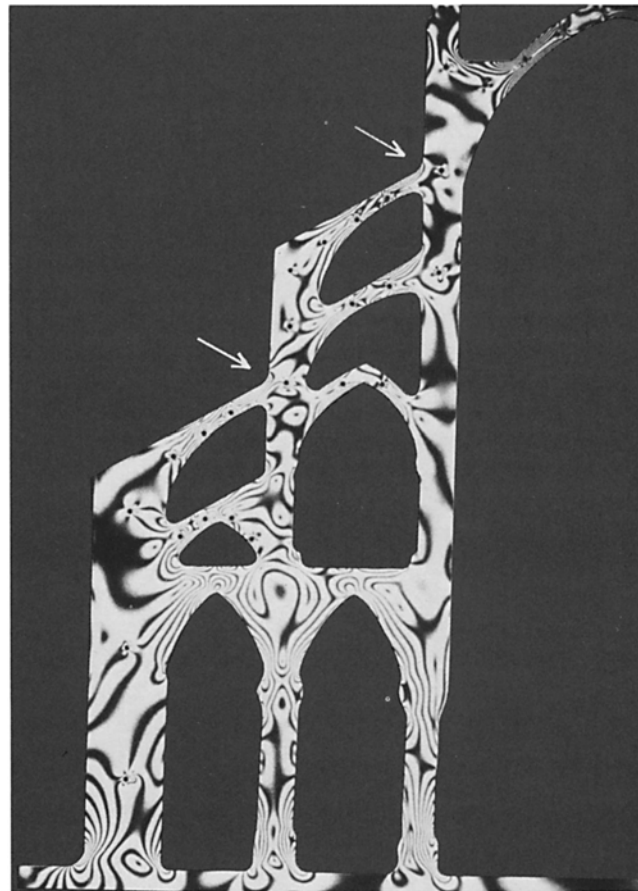


Figure 16 Epoxy scale model 1:100 of typical bay "frame" of Notre Dame's 13th century structure, (Mark, 1984).

Structural Improvements

Structurally speaking the Notre Dame was subjected to some significant changes in its stone skeleton. This can be explained using the cross-sections in different stages of time. Clark & Mark (1984) reconstructed these sections and developed their analyses through thorough research using both archaeological evidence and modern structural analysis methods (Clark and Mark, 1984). The sections in Figure 17 show the original configuration of the choir (1163) and nave (1175), the nave's structure after the reconstruction in the 13th century (1230), and the nave section after the modifications of Viollet-le-Duc in the 19th century (1850).

As shown above in Figure 14, Notre Dame was one of the first Gothic cathedrals ever to be constructed and Master Builders had still limited experience. This was especially problematic at Notre Dame due to the vast increase in scale and complexity of the structure compared to those preceding it. Notre Dame was designed with a ceiling height of 34 meters, which was 25% higher than the highest church built up to that moment. Additionally, the cathedral was significantly wider due to its five aisled floorplan.

The 13th-century alterations were arguably necessary because of insufficient daylight entering the interior of Notre Dame and due to structural problems of its stone skeleton, resulting in high maintenance costs. Experiments with epoxy models (Clark and Mark, 1984) revealed possible vulnerable spots in Notre Dame's stone skeleton where significant tensile stresses occurred during high wind speeds (Figure 16). The solution was to improve the lateral stability of the cathedral by increasing the size of the flying buttresses and letting them attach higher to the clerestory wall (1230 nave in Figure 17).

The 19th-century reconstruction works were necessary due to the cathedral's bad state after the French revolutions and years of neglect. The badly maintained structure got beautifully captured in Victor Hugo's book: *Hunchback of Notre Dame*, which marked a shift in the appreciation for the cathedral. Viollet-le-Duc's restorations and additions to Notre Dame de Paris in the 1840's survived the two world wars and became part of the DNA of the cathedral. If one would take a close look at the flying buttresses built by Viollet-le-Duc, one could see that their geometry was modified and that, again, their connection with the clerestory wall was moved up (1850 nave in Figure 17).

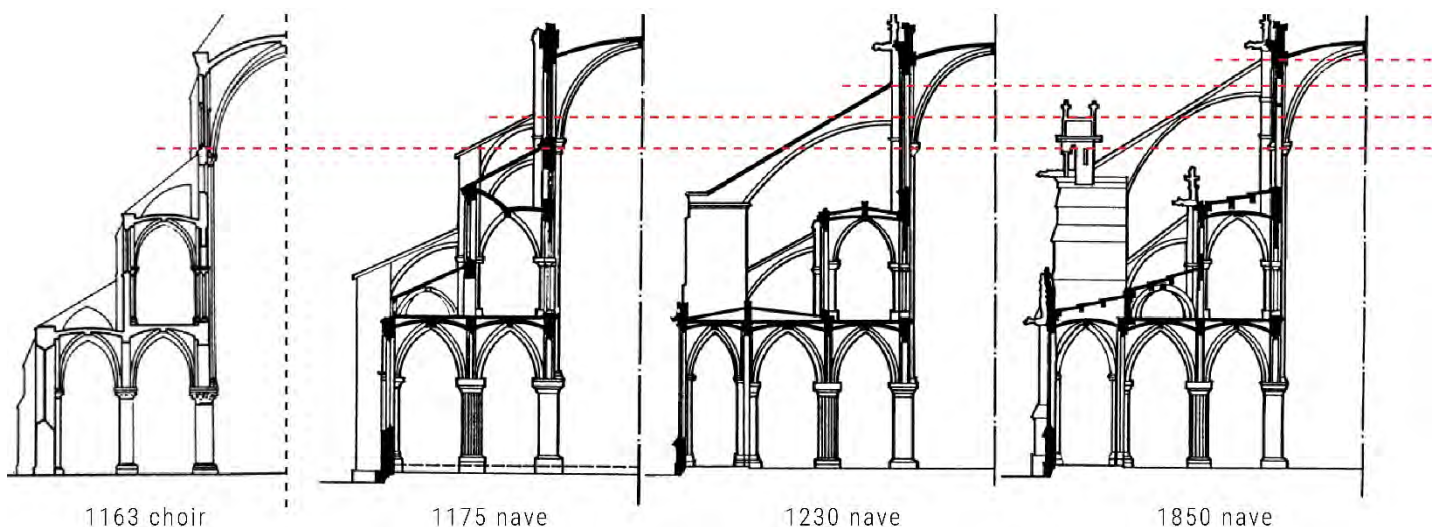


Figure 17 Nave sections throughout the centuries. The red dotted lines show the increased height where the flying buttresses connect to the clerestory wall (Clark and Mark, 1984).

Stability System

In every modification of Notre Dame's stone skeleton, one can notice the focus on the clerestory wall. Notre Dame's clerestory wall from before the fire in 2019, is analyzed in Figure 18. Considering only dead load, the triangular timber roof structure only exerts a vertical force on the walls. Just like the pinnacles at Amiens cathedral, this vertical load can help to stabilize the masonry wall by functioning as a prestressing force. Lateral stability is further guaranteed by the stone vaults and the flying buttresses. The reaction force at the base of the clerestory wall could be assumed to be only vertical.

If besides dead load one considers also external horizontal loads, it becomes clear which parts of Notre Dame's stone skeleton are vulnerable. As mentioned in the previous chapter, a Gothic stone structure and especially the mortar joints are highly sensitive to tension forces. Vannucci investigated the failure mechanism of the nave when subjected to wind loads (Vannucci et al., 2019a). The presence of the timber roof structure - its vertical load on the clerestory wall and its functioning as a connecting element between both walls - was left out of the analyses. One can observe in Figure 19 (left) how local tension forces on the windward side cause cracking in the flying buttress and at the base of the clerestory wall. Eventually, the thin masonry vaults break, the stone skeleton at the leeward side fails and the complete nave collapses at a wind speed of 222 km/h.

In 2018, a 30-year long restoration plan was proposed to maintain Notre Dame's stone structure, roof, and central spire, which again had fallen into neglect. The most significant damage to the stone skeleton was the collapse of parts of the vaulted ceiling. The right image in Figure 19 shows the research of Vannucci after the devastating fire of 2019. It becomes clear that failure occurs much faster since the windward side of the cathedral can only rely on the tensile strength of its flying buttresses. Vannucci's model collapses at a wind speed of 90 km/h, proving a significant decrease of almost 60 % in strength compared to the structure before the fire. The Notre Dame de Paris was therefore after the fire in a critical state and fast action was needed to save the most famous monument of Paris from further collapsing. Afterward, the timber roof has to be rebuild, a part of the cathedral with its own particular story as will be described in the next section.

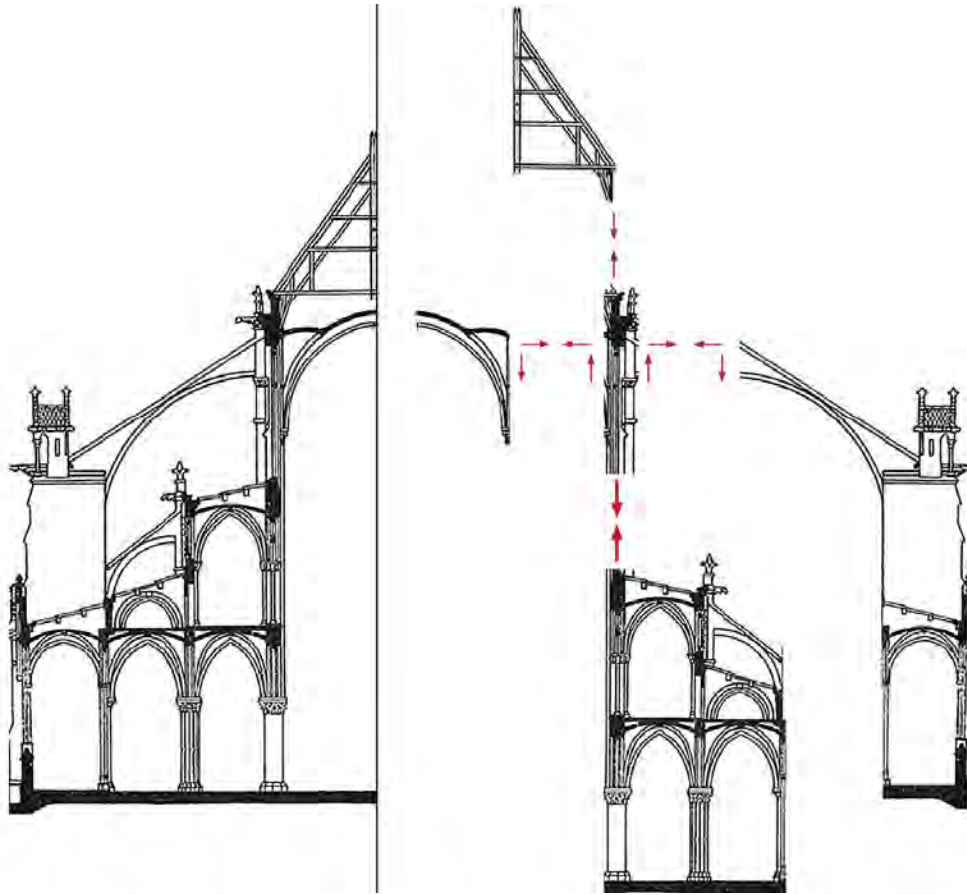


Figure 18 Left: Cross section of Notre Dame de Paris of before the fire of 2019. Right: Simplified structural functioning of the building elements subjected to self-weight only (!), with the clerestory wall in the centre.

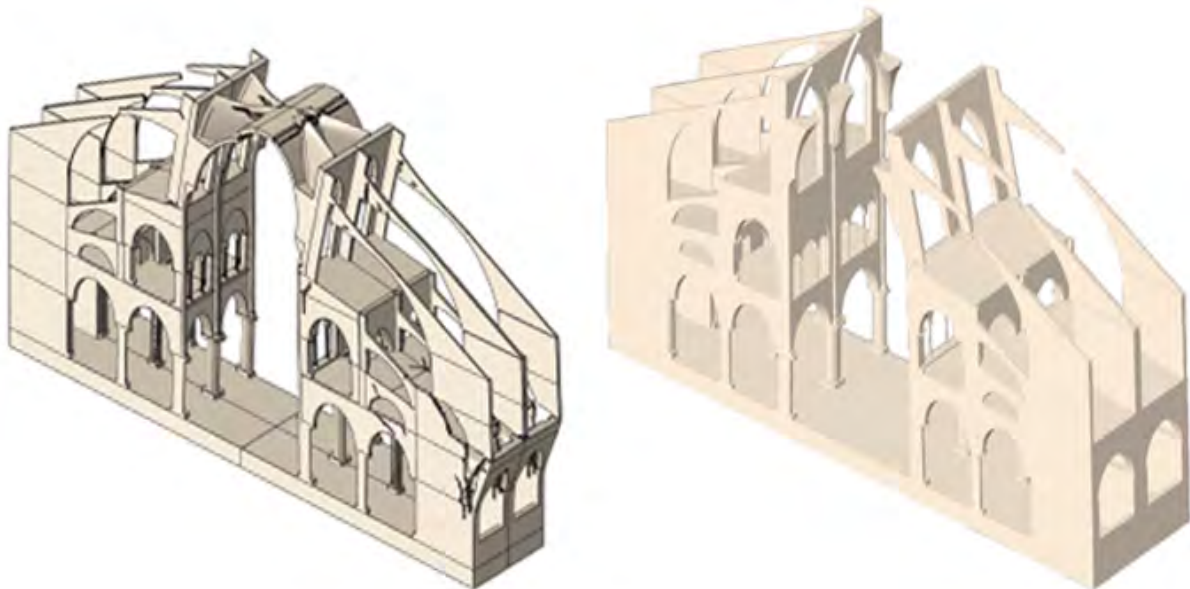


Figure 19 Left: damage of the structure and formation of the collapse mechanism at a wind speed of 222 km/h (Vannucci et al., 2019) Right: Notre Dame without the collapsed vault, collapse mechanism occurs at 90 km/h, (Vannucci, 2019). Notice that in both images the windward and leeward sides are mirrored.

1.3. THE FOREST

Creative Carpenters

The attic of Notre Dame de Paris is one of the most secluded spaces of the cathedral and a place of imagination and fictive stories like Victor Hugo's *Hunchback of the Notre Dame*. Though, the structure that makes up the attic space, has an interesting non-fiction story to tell too.

Figure 20 shows on the left a section of a typical wooden roof of a Gothic cathedral. It's likely that Notre Dame initially had a similar roof structure (Aubert and Vitry, 1919). The roof of Notre Dame's choir was for sure finished before 1182, but there is evidence that this initial roof already had to be rebuilt around 50 years later when a thief caused a fire while stealing a chandelier (Sandron and Tallon, 2013).

The 13th-century reconstruction of the roof is shown on the right side of Figure 20. There are two important modifications visible compared to the initial roof structure. The first modification is the height of the clerestory wall, which was made 2.7 meters higher through the addition of a so-called guttering wall (Vannucci, 2020). The second difference is that the wooden roof structure got a completely different mechanical scheme caused by the addition of a tie beam located at the bottom of the frames, which changes the whole structure into a self-equilibrated system. If these radical changes were initiated by aesthetics (Sandron & Tallon, 2013) or structural problems (Clark & Mark, 1984) is unknown and the causality of both changes remains uncertain.

The rebuilt wooden roof structure of the 13th century survived for around 800 years until the fire in 2019. The entire roof was 115.6 meters long, 13 meters wide, and 9.75 meters high. Due to its impressive wooden structure, built out of 1000-1300 oak trees with a diameter of 25-30 cm (Vannucci, 2020), the attic earned its nickname: *The Forest*. The origins of the oak trees remain questionable, but historical records mention that the timber for some other cathedrals was provided by the king or other powerful people. The first qualitative descriptions of *The Forest* date back to the 19th century and provide us with global analyses and a series of structural hypotheses and concepts (Viollet-le-Duc, 1854) (Choisy, 1899). Quantitative analysis and verifications of these hypotheses have never been provided, until after the fire of 2019. A few of the recent findings and their accompanying problems are important to mention.

Weaknesses To Wind

Vannucci (2020) described and investigated the three-dimensional properties of the wooden structure in Notre Dame's attic. It's suggested that the carpenters were restricted by the available materials and costs and couldn't position the wooden roof frames in the same rhythm as the flying buttresses (Figure 21). Wooden frames were therefore also placed in-between the main-frames, which made some experts believe that the guttering walls were loaded relatively evenly. Recent research proves that this is not the case and that small differences between the main-frames and the in-between frames resulted in a complex three-dimensional interaction. Through this complex wooden spaceframe, the vertical forces were guided towards the primary frames (Figure 22) (Vannucci, 2020).

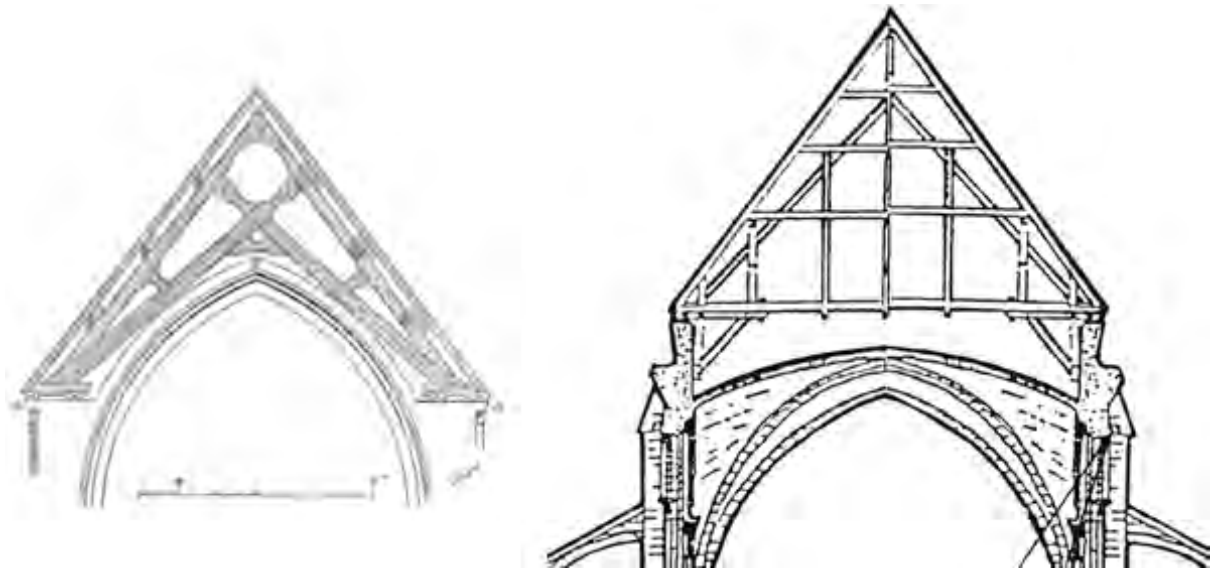


Figure 20 Left: Structural scheme by Viollet-le-Duc (1854), showing a typical wooden roof structure – without tie beam -. It's assumed that Notre Dame de Paris had a comparable roof around 1170. Right: scheme of a typical wooden frame of the nave's structure.

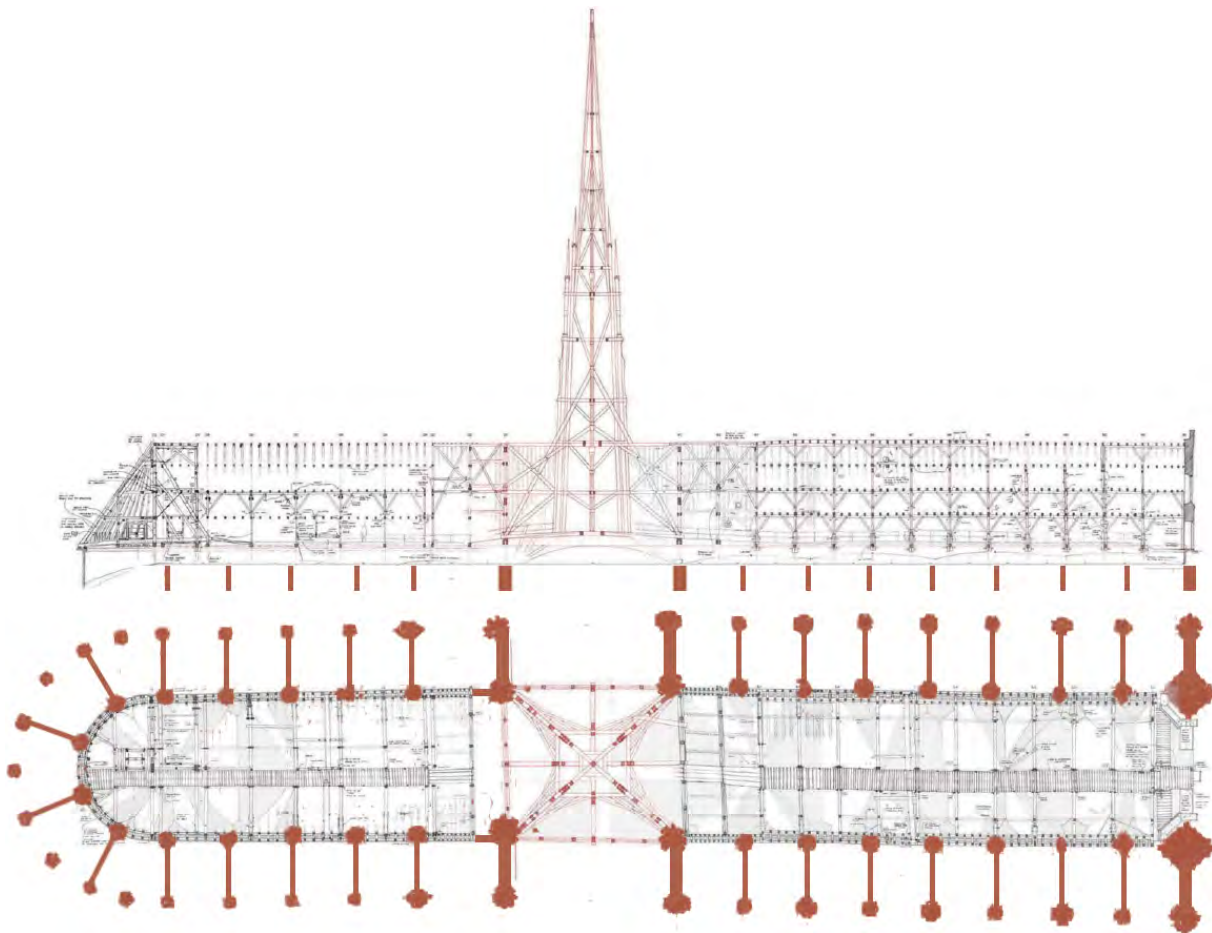


Figure 21 Longitudinal section of Notre Dame de Paris displaying that the timber roof frames are not following the rhythm of the underlying stone skeleton (base from Rémi Fromont et Cédric Trentesaux, 2020).

Furthermore, the 2.7 meters increased height caused some problems with the relatively slender limestone guttering walls. Robert Mark's analysis of the pinnacles at the cathedral in Amiens shows the importance of vertical loads acting on the stone skeleton, which could 'prestress' the structure to prevent tension forces to occur. During the initial construction of *The Forest*, the Master Builder's design was however governed by available material and economic reasons. The timber roof was therefore built with members of small diameter, resulting in a relatively lightweight structure. In combination with the raised wall, it caused the limestone walls to be sensitive to wind-induced horizontal forces.

At Notre Dame de Paris, the solution of the Master Builders was the application of ingenious connections between the timber structure and the limestone walls. The vertical forces were mainly introduced at the very top part of the guttering wall, 'prestressing' it, and the horizontal forces were transferred down towards a point where the flying buttresses could provide lateral stability. Structural analyses (Figure 22) show how *The Forest* would split the vertical and horizontal forces and lead them to different parts of the stone skeleton (Vannucci, 2020). Most of the vertical load is transferred to the top of the guttering walls. The lower positioned supports are subjected to a small portion of the total vertical load, which can only be explained due to a low vertical stiffness of the joints of the corbels. The Notre Dame timber structure's support condition is schematized in Figure 24, displaying the compression-only supports.

One can observe that the resistance against horizontal wind forces is completely dependent on the corbel on the leeward side of the structure. This is probably the result of the connection between the timber frame and guttering wall, which doesn't allow horizontal tension forces to be transferred (Figure 22). Other cathedrals with a comparable height difference between the timber roof structure and flying buttresses, like the cathedral of Beauvais, contain completely different solutions to the lateral stability issue. At Beauvais for example, multiple flying buttresses above each other are integrated into the stone skeleton to provide lateral stability for either the vaults or roof.

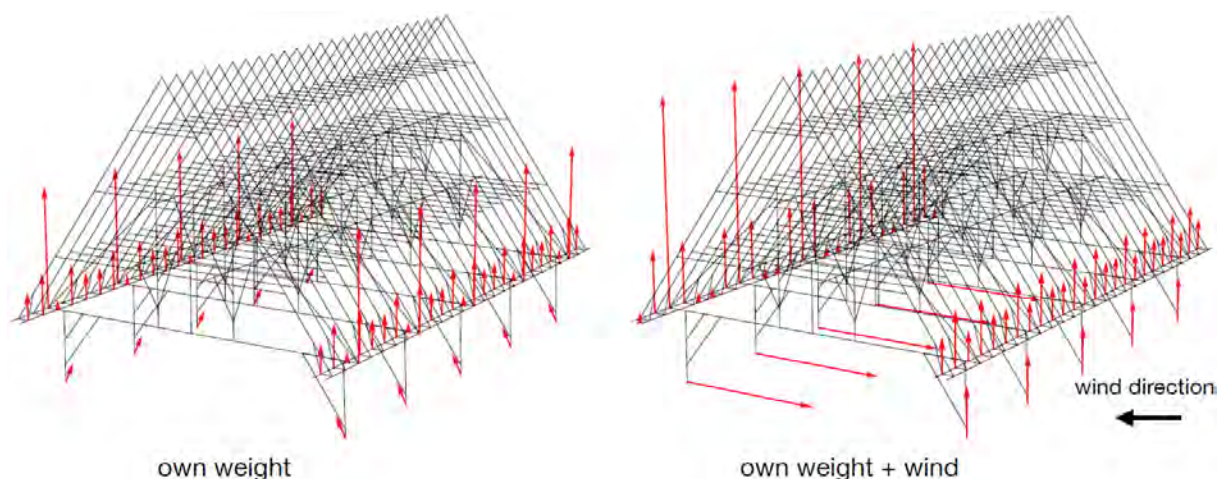


Figure 22 Structural Analyses in 3D of Notre Dame's former roof structure, by Vannucci (2020). The corbels take up only a small portion of the vertical load, but a large portion of the horizontal load.



Figure 23 Photo of Notre Dame's attic: In blue, the lower positioned connection between corbels and the stone skeleton of the cathedral. In red, the position where the flying buttress meets the clerestory wall on the outside.

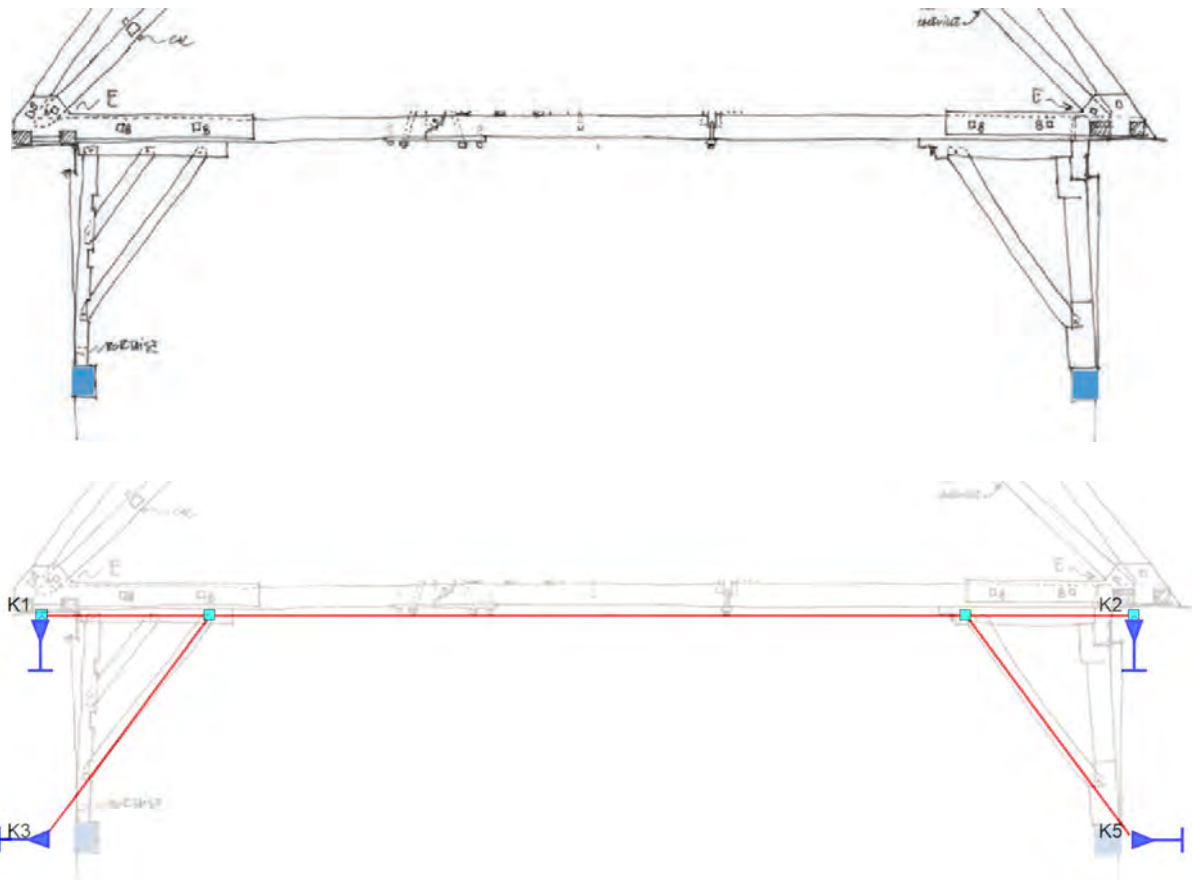


Figure 24 Above: sketch by Metalocus (2020) of the former timber structure. Bottom: Idealised mechanical scheme of the guttering wall translated into structural boundary conditions with compression-only supports.

1.4. CONCLUSIONS

Which lessons can be learned from Gothic - and in particular Notre Dame's - engineering history?

The design strategy to construct the, for that time, huge Gothic cathedrals was to concentrate forces acting in the stone skeleton, which allowed masonry walls to be substituted by big windows and resulted in a characteristic building style. The stone skeleton's structural functioning is governed by the weak tensile strength of the masonry, and more specifically the mortar. The structural performance is therefore highly dependent on stability achieved by the geometry - self-weight - of the building elements themselves. Since one doesn't have to be concerned about compression forces, tension forces can be averted by adding weight - material – on top of the structure. Common examples like pinnacles can have a prestressing effect on the masonry underneath.

Notre Dame de Paris is one of the first and most beloved Gothic cathedrals and played a fundamental role in the development of the Gothic structural system, characterized by flying buttresses. While Gothic engineers were learning the pros and cons of such a structural scheme, Notre Dame's stone skeleton underwent an evolution, with the nave's transversal structural system as a distinctive example; the development of the flying buttress and the gradual elevation of its connection to the clerestory wall. Throughout history, Notre Dame has been subject to periods of neglect. The fire, during the necessary maintenance in 2018, damaged the vaulted ceiling and deteriorated the limestone skeleton, resulting in a discontinuity in the structural system and therefore vulnerability to lateral (wind) forces.

The design of Notre Dame's former timber roof structure, *The Forest*, was governed by economic reasoning, which resulted in a timber structure that wasn't aligned with the stone skeleton of the cathedral. The roof was a relatively lightweight structure that relied on its functionality through a complex three-dimensional interaction between the numerous trusses (Figure 25). Ingenious connections were made between the timber trusses and the guttering walls with the primary aim to lead horizontal forces down towards the flying buttresses and to prevent tensile reaction forces in the limestone.



Figure 25 Complex three-dimensional configuration of the carpentry in Notre Dame's attic. Photo is taken from the walkway inside the attic, the Catwalk (Notre Dame de Paris, 2018).

2. MEDIEVAL HIGH TECH

Parallel to the evolution of the Gothic cathedrals, without a doubt, there was a corresponding development of technology and construction methods to build these fabulous structures. This chapter will first discuss the main designers of the Gothic buildings, the so-called Master Builders, who were due to nonexistent mechanics theory dependent on particular design and validation methods. Subsequently, their ingenious construction methods as solutions to practical problems are described.



2.1. A MASTER BUILDER'S SCIENTIA

Rules Of Thumb

The Master Builders, sometimes referred to as master masons, are generally mysterious figures, who are hardly mentioned in historical records. They led a building process executed by hundreds of construction workers and possessed the skills, experience and courage to make design decisions considering a broad range of technical and aesthetical aspects ranging from foundations to symbolism of ornamentation (BBC, 2008).

As the first preceding ideas about strains and stresses in materials only appeared in Galileo's work in the 17th century, the Master Builders in medieval times couldn't rely on mechanics theory (Heyman, 1998). Even the Imperial or Metric System didn't exist as a standard for lengths across a certain region (Akande, 2017). Instead, the Master Builders' designs followed geometrical rules using simple geometrical shapes like the square, circle and triangle. Their symmetrical and proportional characteristics were responsible for *the harmony of creation*. But more than merely *divine* aesthetical rules, the proportioning was also important for the stone skeleton's structural performance, as thickness and span would also be captured in rules of thumb (BBC, 2008).

Supported by contemporary analytical tools to calculate and describe strains and stresses in structures, one can observe that many cathedrals are heavily over-dimensioned (Heyman, 1997). Cathedrals were designed with the stability of the structure as a primary concern. The geometrical stability of a stone skeleton could be relatively easily investigated and altered in small-scale models. Additionally, these scale models were used to communicate with the clients and to give instructions to the builders (Fitchen, 1981).

A Culture Of Copying

Luckily, Master Builders acknowledged that small-scale models couldn't predict exactly how a full-scale cathedral would behave. Therefore, existing churches and cathedrals were important references from which the main dimensions could be copied to function as valuable guidelines. By simply observing and measuring, an existing structure could be captured in geometrical rules which could be used to design new cathedrals. We can only speculate about the interrelationship of Master Builders, but for sure the design of the numerous Gothic cathedrals in Northern France must have influenced each other. Additionally, the cathedrals themselves had a repetitive layout and were constructed bay by bay. By utilizing this building order each bay could function as a prototype for the next one to be constructed.

Due to the sometimes centuries-long construction time, a Gothic cathedral had often multiple consecutive Master Builders who used the latest medieval aesthetics and knowledge. It is suggested that Notre Dame had five Master Builders during its construction in the 12th century, from which only the last one is known, Jean de Chelles (Bruzellius, 1987). Also in the centuries to follow, many - necessary or fashionable - modifications were made to the cathedral's stone skeleton, which is still clearly visible nowadays. The differences between Notre Dame's nave and choir can mainly be attributed to Viollet-le-Duc's 19th-century modifications. The harmonious interior of Notre Dame hides the fact that the external stone skeleton has a completely different geometry (Figure 26).

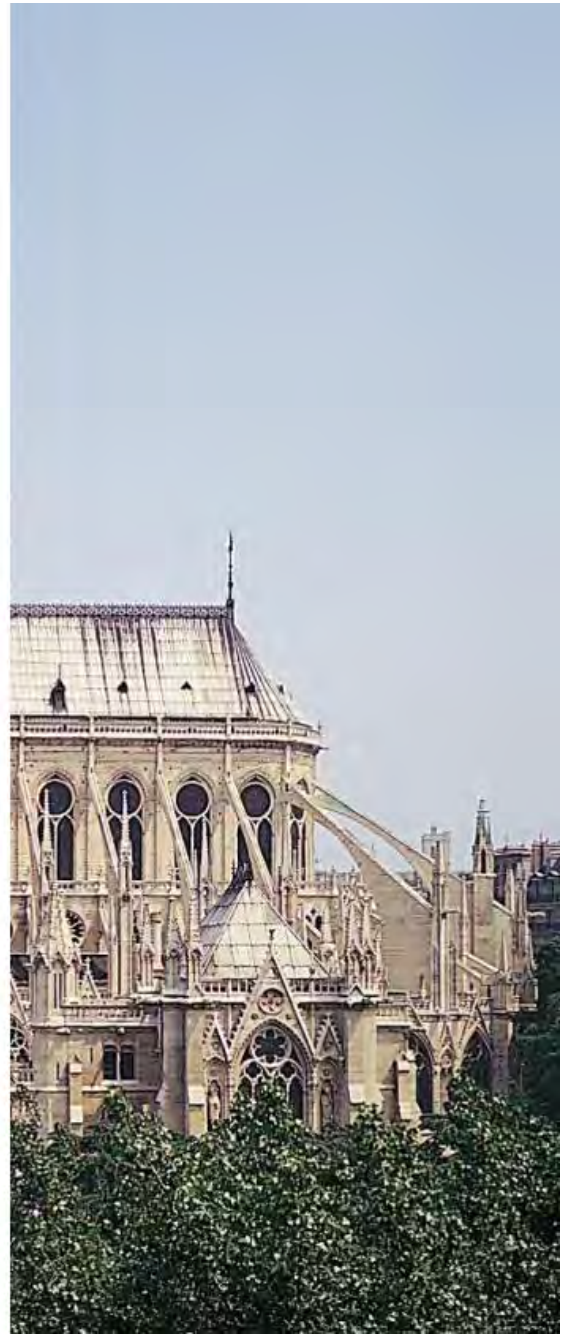


Figure 26 Notre Dame de Paris seen from the south. The transept is taken away to reveal the different systems of flying buttresses of the choir and the nave (Notre Dame de Paris, 2003).

2.2. CONSTRUCTING A CATHEDRAL

The stability of Gothic cathedrals relied on equilibrium within the complete stone structure. The application of stone arches and vaults meant that during construction a cathedral was depending on temporary support structures, like shoring and centering, to provide stability. Since materials were the largest expense in the building process of a cathedral, both the final permanent structure and the temporary support structures, had to be designed efficiently.

A rough idea of a cathedral's building process is provided in Figure 27. The construction steps can simply be approached as a repetition of stone columns, arches, and vaults. Due to the low tensile strength of medieval masonry, each wall and column on the ground floor had a relatively low slenderness ratio. As the columns would increase in height, their stability could be increased by connecting them with stone arches and vaults. This system could be repeated relatively easily horizontally aligned with a cathedral's cross-shaped floorplan. Vertically the structural elements had to be built with less material to limit the weight on the structure situated below. Finally, the ground floor's central aisle was covered by a gable timber roof to provide weather tightness, after which a thin brick vault was erected directly underneath. The vaults had multiple functions: they completed the continuous structural system by equilibrating the horizontal thrust forces of the flying buttresses, they functioned as a fire barrier between the flammable timber roof and valuable interior and were considered to be an aesthetically pleasing ceiling.

Integrating Support Structures

To save material and therefore money and time, the Master Builders developed strategies to integrate support structures within the fabric of the cathedral. The ribbed cross vaults, one of the most discussed Gothic structural inventions, was for example developed as a clever solution to stiffen the temporary timber centering of the brick vaults. Timber has relatively low stiffness, which became a problematic characteristic during the gradual construction of the brick vaults. The weight of the heavy masonry could cause significant deflections of the temporary centering resulting in geometrical inaccuracies during the vaults' subsequent construction steps. By first integrating two stone arches (ribs) within the timber support structure, the centering became a lot stiffer. The brick vaults could be erected on top of this stiffened support structure and after completion, the stone ribs remained visible from the interior of the cathedral (Figure 28).

The erection of cathedrals could last for centuries, while the need for new places of worship was urgent and resulted in a considerable amount of time pressure. Special strategies were used to utilize cathedrals already during construction. It was first of all important to quickly free the ground floor from any support structures and to provide weather-tightness to the building. The former was accomplished by attaching scaffolding and support structures to the stone skeleton, which allowed for the removal of structural members on the ground floor. The latter requirement was tackled by applying, and taking profit from, a bay-by-bay building order. As can be seen in Figure 27, the finished choir of a cathedral could already be used for religious services, while the nave was still under construction. A temporary brick wall could separate the church services from the construction activities that were still going on in other parts of the structure (Fitchen, 1981).



Figure 27 Reconstruction of the building activities during the initial construction process of Notre Dame (Andrew Tallon, 2013). (As described in the previous chapter this image isn't accurate, since the Notre Dame initial construction contained a completely different flying buttress system.)



Figure 28 Ribs in vaulted ceiling visible at Notre Dame de Paris (Carlos Delgado, 2012).

Within this same concept of integrating support structures in the eventual fabric of a cathedral, one can highlight the phenomenon of incorporated scaffolding. As described by Viollet-le-Duc (1854), cathedrals like the Notre Dame de Paris contained a system of integrated stairs, galleries, and service passageways both on the inside and outside of the structure. These paths were used during construction to improve the communication between workers and to transport people and small amounts of materials. In the cross-section of Bourges cathedral's nave, one can observe the corridors integrated within the stone skeleton (red in [Figure 29](#)). After completion of the cathedral, these passageways were used to monitor the structure and to execute the inevitable maintenance. Just like the ribbed vaults, the integrated system of passageways is characteristic of Gothic structures, as stated by Fitchen (1961) “... the deliberate planning and the multiplication of these service passageways are uniquely Gothic developments.”

At the slender top part of the cathedral, the scaffolding couldn't be incorporated within the structure. The Master Builders applied therefore a putlog scaffolding system that hung from the stone walls of the cathedral and contained wooden beams that pierced the masonry walls (Fitchen, 1981). These scars, which are still visible in the clerestory walls of Notre Dame de Paris, didn't prevent the Master Builders from applying the putlog system, functionality was favored over a perfectly finished wall.

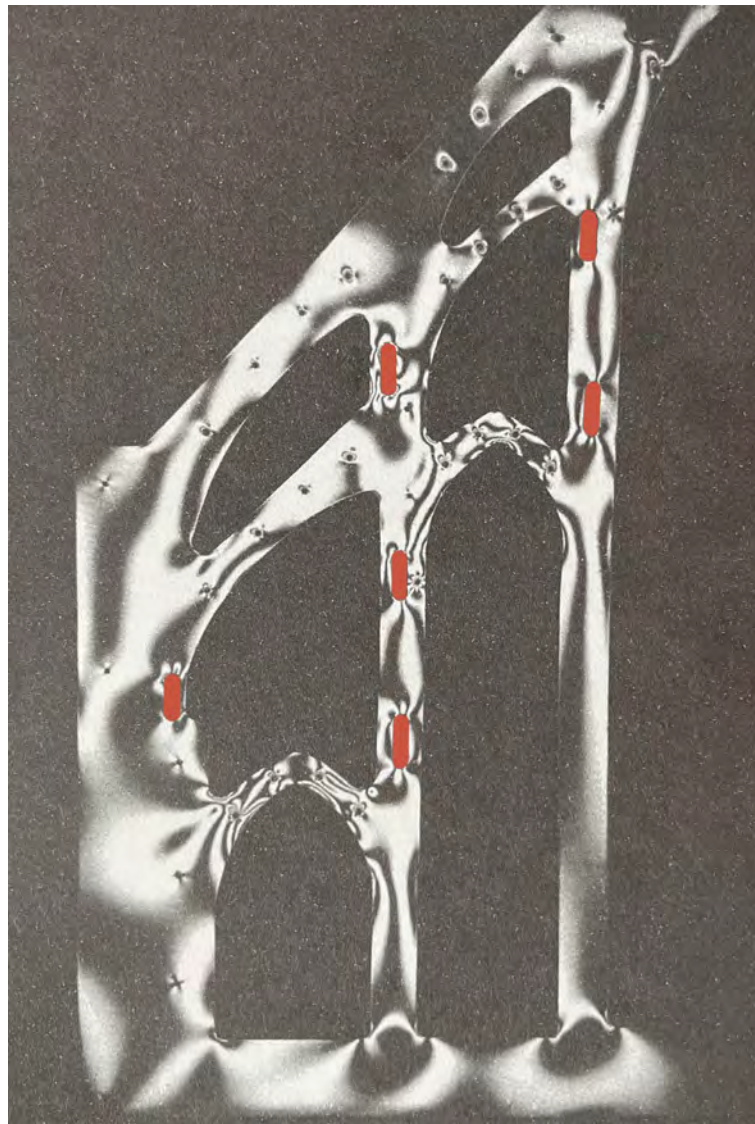


Figure 29 Analysis of the nave of Bourges cathedral and the influence of passages (red) in the structural performance of the stone skeleton (base by Mark, 1984).

2.3. CONCLUSIONS

Who were the Gothic builders and which construction strategies enabled the erection of cathedrals?

Without a doubt, the inventive and creative Master Builders possessed a broad range of technical and execution knowledge ranging from masonry foundations to symbolic ornamentation. The design process of a cathedral was dependent on specific design processes during which scale models, trial and error, prototyping, and building order, all played a crucial role.

The Gothic construction methods were due to the lofty design and use of masonry highly dependent on support structures, which were often ingeniously integrated within the cathedral's fabric. Time pressure, technical challenges and material expenses, played a key role in the construction process resulting in inventions like the bay-by-bay building order, ribbed vaulted ceilings and an ingeniously integrated communication and transportation systems.

3. AN IDENTICAL RECONSTRUCTION

The extraordinary skills and knowledge of the crew currently reconstructing Notre Dame de Paris can be used to gain valuable information. Getting in touch with team members has unfortunately proven difficult, due to the ongoing investigation of the fire. Hence, through online published drawings and photos, an analysis of the reconstruction is assembled and presented in this chapter. To limit the vast amount of required reconstruction works, special focus has been given to the nave and choir, which together form the main axis of the cathedral. The most important excluded building parts of the research are the central spire and the large scaffolding structure that surrounded it.

Figure 30 Reconstruction crew member hovering over vaulted ceiling to remove debris (Zachmann, 2020).



3.1. STABILISING NOTRE DAME

Vaults - Flying Buttress Instability

The instability of Notre Dame's stone skeleton due to the collapsed vaults and deteriorated masonry had resulted in a situation in which the complete cathedral was in danger of collapsing. The authorities' first big challenge was to save Notre Dame from total collapse by quickly stabilizing the structure. The working conditions got significantly hampered due to the highly toxic lead cloud which had covered large parts of the cathedral's interior and exterior.

An easy solution to stabilize the cathedral would have been to substitute the collapsed vaults with a temporary propping structure. But to prevent construction workers to be exposed to the risks of a total collapse and lead contamination, the authorities developed a strategy to stabilize the cathedral from the outside. By placing timber support structures underneath the flying buttresses, their inward thrust forces got averted. Figure 31 shows the intervention in a schematic cross-section of the cathedral's nave (left) and a photo which captures how the prefabricated timber structures were connected to the flying buttresses and the clerestory wall using respectively ordinary wooden planks and steel profiles with large slotted holes - to allow on-site adjustments - (right).

Rain Induced Deterioration And Instability Of Guttering Walls

Right after the fire, Notre Dame was in desperate need of weatherproofing the vulnerable stone structure. The destruction of the roof and vaults left the limestone and a section of the interior unprotected. The rain kept deteriorating the cathedral and caused a steady increase in the weight of the porous limestone and hence growing instability issues (Lesté-Lasserre, 2020). The burned-down timber roof structure used to connect the two guttering walls on either side of the central aisle and helped to prestress the limestone masonry. With the missing roof resting on top of the guttering walls, they became vulnerable to be pushed over by strong winds (Vannucci et al., 2019b).

The authorities' response to this critical situation was to place prefabricated timber cassettes on top of the guttering walls. Figure 33 displays how the prefabricated timber frame cassettes, measuring 4 meters in width and 15 meters in length, were delivered on-site and hoisted in place at once. Their position followed the rhythm of Notre Dame's bays – 5.7 meters in wide - and the primary beams were placed above the flying buttresses. The glued laminated timber members were interconnected with straight-forward connections made out of preformed steel plates and metal fasteners. Online construction drawings (Figure 32, left) show diagonal members positioned on both ends of each cassette to provide in-plane stability. Further research though, showed that the diagonal members were positioned more inwards (Figure 33, left). Weatherproofing was achieved by a fabric cover with a small lateral slope to guide rainwater to the small aqueducts that are integrated into the flying buttresses.

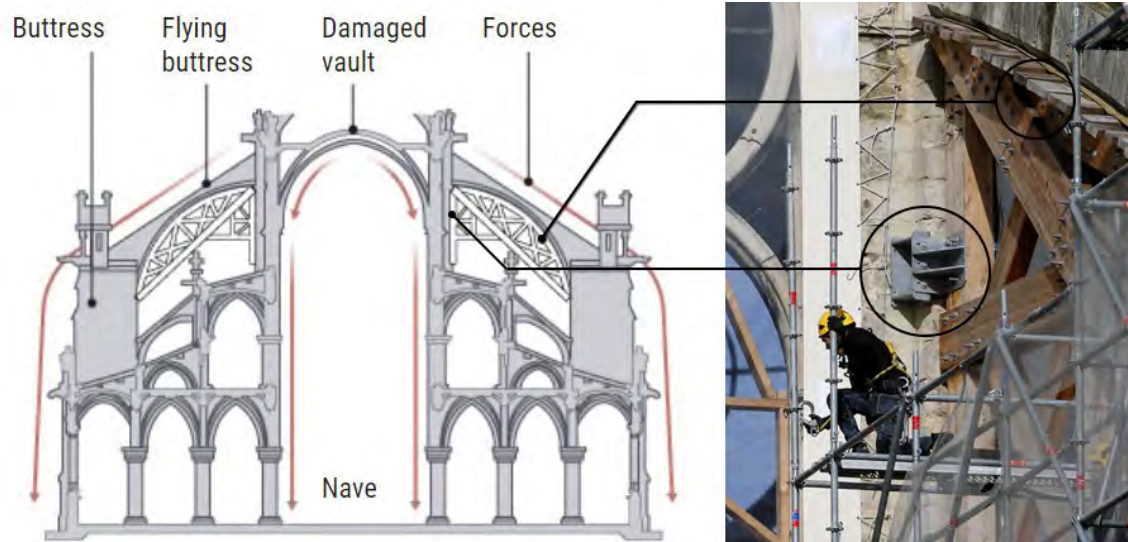


Figure 31 Left: Schematic cross-section of Notre Dame's nave, showing the position of the temporary wood structures underneath the flying buttresses. Right: Photo capturing the moment of placement (Zachmann, 2020).



Figure 32 Left: Overview of prefabricated Glulam cassettes positioned on top of the guttering wall of the choir. Right: photo showing moment of placement of cassette (Zachmann, 2020).

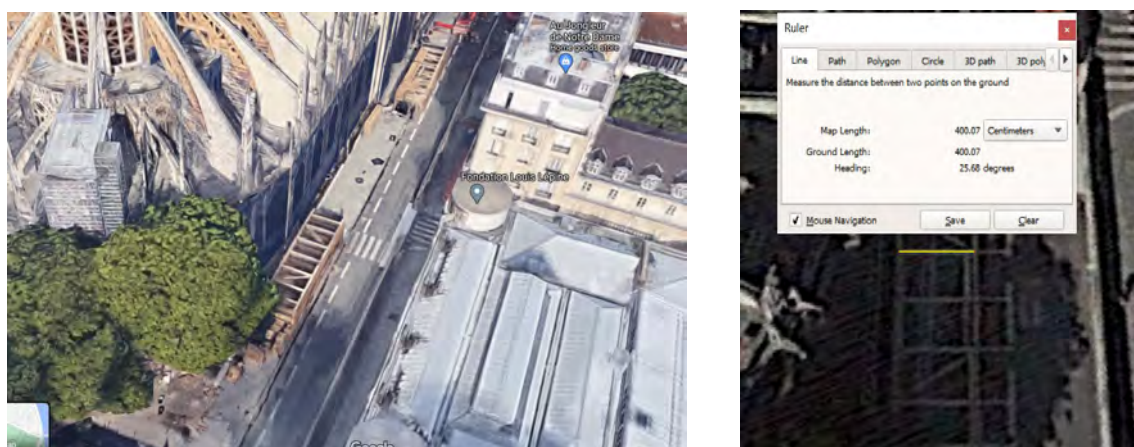


Figure 33 Prefabricated Glulam cassettes measuring 4 meters in width and 15 meters in length (Google Earth, imagery date 22-08-2019)

3.2. DEBRIS CLEANING WITH AN ARCHEOLOGIST'S BRUSH

The fire led to the collapse of a significant part of Notre Dame's primary structure, but also damaged many of the Gothic statues and covered the glass-stained windows and interior elements with a dirty toxic lead layer. Restoration and cleaning will take years of highly specialized labor, according to people involved a beautiful opportunity to see the beautiful Gothic ornaments from up close (Marlowe, 2020). Restoring the ancient ornamentation is taking place in workshops outside Notre Dame de Paris and out of the public eye.

The debris of the burned-down timber roof is treated as a precious relic. The charred fragments of *the Forest* could reveal clues about the cause and development of the fire, which is important in the ongoing investigation. Archeologists and climate scientists are also interested in the ancient oak wood which can reveal how the climate 1600 years ago used to be (Ball, 2020).

Notre Dame's burned-down roof resulted in a unique archeologic opportunity, but at the same time delayed the reconstruction. Figure 34 shows how each element of the former timber structure was labeled and investigated before it could be taken away. Since the vaults couldn't carry the extra weight of construction workers (Lesté-Lasserre, 2020), they got suspended from the timber cassettes that were placed on top of the guttering walls. A temporary tent was built up out of aluminum members to protect the vulnerable debris and construction workers from weather conditions (Figure 35).

3.3. RECONSTRUCTING *THE FOREST*

To eventually reconstruct *the Forest*, the authorities will erect a big temporary construction tent to climatize the area where the roof will be rebuilt. The construction tent contains an integrated crane to hoist heavy materials and equipment and consists out of aluminum elements that are covered with a canvas to block rain and wind. It is still unclear how the authorities will construct the timber roof identical to the one that burned down since the required oak trees are not easily available. Part of the authorities' solution is an underwater forest in Ghana (Figure 36). Although the wood seems to have exceptional physical and mechanical properties, the harvesting of the trees seems to disrupt the ecosystem in Lake Volta. A "sharp decline" in the number of fish currently threatens the lifeline of around 300.000 fishing families (Akinyemi, 2019). It exemplifies the difficulty that authorities are facing in reconstructing Notre Dame's roof with identical timber. To reconstruct the exterior appearance of the former roof, the authorities will eventually cover the structure with the same lead roof cladding and use the same Gothic ornaments which originated from Viollet-le-Duc's 19th-century additions to the cathedral.



Figure 34 Hanging workers from the ceiling, hovering above the labeled debris (Zachmann, 2020).



Figure 35 Temporary tent to protect construction workers and debris from rain and sun (Zachmann, 2020).



Figure 36 Underwater Forest Ghana being harvested and transported to Paris (Ageke, 2021).

3.4 NOTRE DAME'S CONSTRUCTION SITE

On-site Handling

The construction site of Notre Dame has been analyzed to get grip on the limitations the site and the machines give. It's assumed that the construction vehicles used by the authorities are the ones that should be used for the new design. The Potain MDT 809 tower crane has been present at Notre Dame de Paris since the reconstruction works on the central spire started in 2018 (Figure 37). The crane has a maximum carrying capacity of 25000 kg. The lever arm that's allowed to carry this load though has a maximum of 30 meters. To cover the complete Latin cross-shaped floorplan of Notre Dame de Paris, it's assumed that the crane slightly changes its position and uses a lever arm of a maximum of 60 meters. The resulting load carrying capacity is therefore 13800 kg, as can be seen in *Figure 38*.

Transport

The maximum sub-assembly size is generally dependent on handling and road transport limitations. Each country has specific regulations concerning the maximum dimensions of road transport. For Notre Dame de Paris the maximum dimensions are assumed to be: width <4 m, height <3.75 m, length: <18.5 m. Special transport for exceptional sizes can be arranged but is often limited to special (night) hours (ACEA, 2015). With the presence of the Seine next to Notre Dame de Paris, water transport can be an alternative that allows for considerably larger transportations dimensions. A limiting factor though is the accessibility from the factory to the major waterways. The maximum weight a truck including transport can weigh in the European Union is 40.000 kg. Including the weight of the truck itself, which is generally around 40% of the total mass, the weight of the transport is assumed to be limited to 24.000 kg (ACEA, 2015).

After the geometrical alterations that resulted from the structural verification in Chapter 8, the weight of one sub-assembly of the Field Factory structure was 9200 kg, which, with the conservative estimation that connections add an extra 10% of weight, means that the design wasn't governed by the maximum load-carrying capacity of the construction crane.



Figure 37 Potain MDT 809 Tower Crane at Notre Dame de Paris (Zachmann, 2020)

CRANE AT CONSTRUCTION SITE

Potain MDT 809:
 with leverarm max. 30 meters: 25000 kg
 with leverarm of 60 meters: 13800 kg

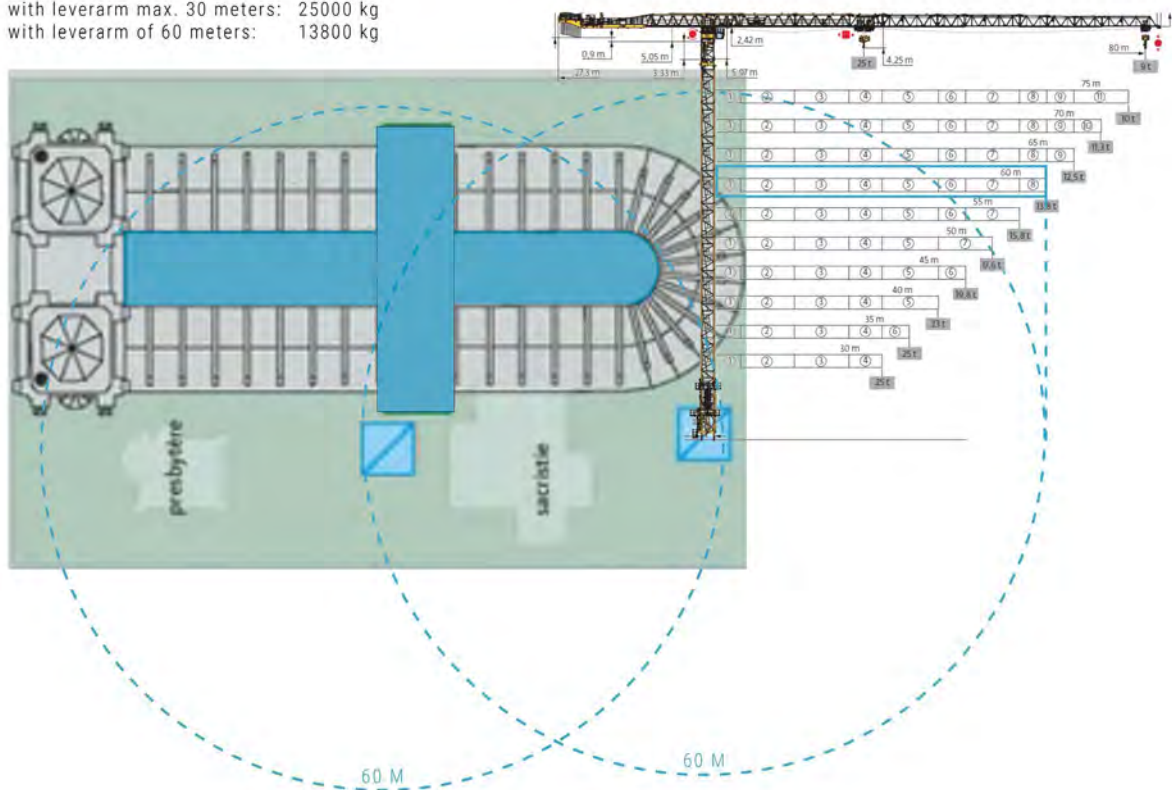


Figure 38 Construction site of Notre Dame de Paris with the Potain MDT 809

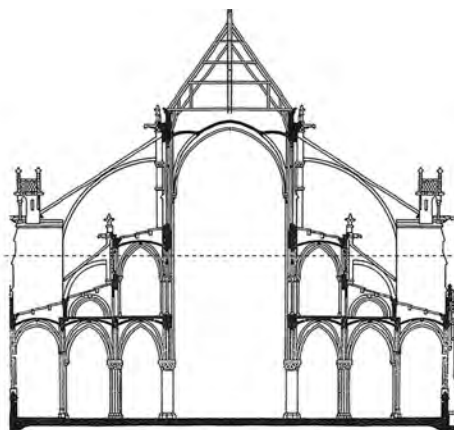
3.3 CONCLUSIONS

What can be learned from the reconstruction process executed by the authorities?

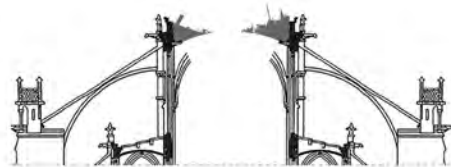
In summary, one can observe that the authorities will utilize three different temporary structures to reconstruct the top part of Notre Dame. First, a stabilizing cover is urgently needed to avoid rain deteriorating the masonry and to stabilize the guttering walls. Construction workers can be suspended from this covering structure allowing the removal of debris without standing on top of the vulnerable stone vaults. Second, a temporary construction tent is placed to create a comfortable working place for archaeologists and construction workers to analyze the debris and to execute the first repairs to the masonry. And a third, a big temporary construction tent, that facilitates the rebuilding of the burned down timber roof structure.

When examining the authorities' reconstruction strategies, two different phases should be identified: one before and after Notre Dame's stone skeleton is stabilized. The former can be characterized as a quick fix during which safety of the staff and speed of construction can be assigned as key requirements in the decision-making. Prefabrication is utilized to maximize construction speed and to limit the amount of labor on top of the unstable cathedral. The latter phase can be referred to as a conventional construction process governed by the result: the identical reconstruction of the roof. Demountable aluminum support structures, commonly applied as scaffolding structures, are used to facilitate a traditional stick-built carpentry building process.

The cathedral's construction site contains large construction machinery and is relatively easily available by road and water transport.



1300



FIRE

- timber roof burned down completely
- the vaulted ceiling partly collapsed
- result: unstable structure without protection against rain



STABILISE + WEATHERTIGHTNESS

- save cathedral from collapsing!
- make cathedral quickly weathertight
- result: stabilising structures (glulam) and raincover (fabric)



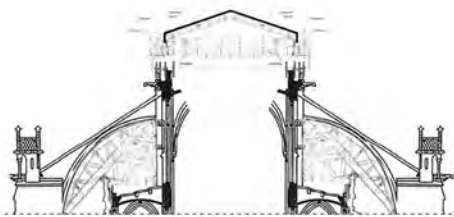
former timber roof structure: The Forest



burned down timber debris on top of vaults

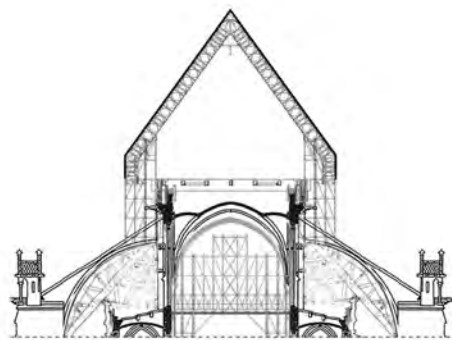


temporary cassettes on top of clerestory wall



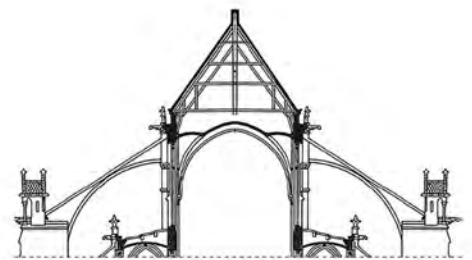
PREPARATORY WORKS

- remove debris of former timber roof
- adding aluminium structure as cover for construction workers



RECONSTRUCTION

- reconstructing vaults
- reconstructing roof
- aluminium construction tent with integrated crane



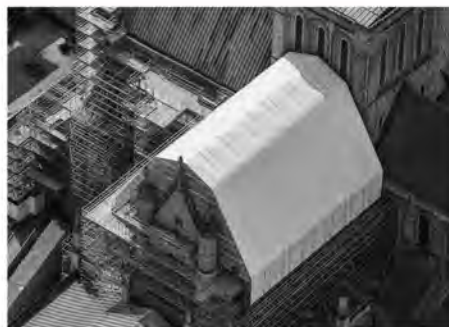
NEW (OLD) ROOF

NOW

2024+



temporary aluminium roof on top of glulam structure



*WINCHESTER CATHEDRAL existing roof (right) covered by aluminium construction tent



new (old) roof timber roof structure and cladding

4. RECONSTRUCTION STRATEGY AND DESIGN

The following chapter will start with describing an alternative approach to reconstruct the roof of Notre Dame de Paris. A reconstruction approach will be defined, after which a design strategy and its associated construction steps will be explained. A program of requirements is then outlined to specify the different functional, structural, execution, and aesthetical demands per phase. These are also based upon observations from Part A and will be used in the following chapter to evaluate different materials and assembly strategies.

Figure 39 Illustration of Notre Dame during the Field Factory phase, during which the gable roof geometry is restored and the roof is cladded with a translucent canvas.



4.1. LEARNING FROM THE MASTER BUILDERS

Ancient Alternatives

The authorities announced that the Notre Dame de Paris will be rebuilt as it was before the fire of April 2019. The required long-lasting building process becomes problematic due to time pressure, which seems to play a fundamental role in decision-making by the authorities. It is relevant to question if the restoration of such an important monument should be subjected to the combination of such time pressure, delivering a renovation of uncertain quality and durability according to heritage experts (Gombault, 2020). History could provide an alternative.

Already eight centuries ago, time pressure had a major impact on the design and construction of Notre Dame de Paris. The Gothic Master Builders had to be - partly forced by a long-lasting building period - creative in allowing the cathedral to be used during construction and the utilized support structures. The developed strategies, such as ribbed vaults, bay by bay construction order and built-in passageways, all had a significant impact on the appearance of the cathedral. The Master Builder's ingenious integrated support structures and their building order are therefore a fundamental part of the DNA of Notre Dame.

Inspired by these ancient ideas, that resulted in efficient material use, increased construction speed and the possibility of using the Notre Dame during its initial construction, a design strategy can be developed that deals with the current challenges. A strategy that does not result in an exact copy of the burned-down cathedral, but that, just like the Master Builders did in the 12th century, embraces technology and inventive construction methods to tackle practical issues, without compromises on durability and beauty.

Tackling Time Pressure

The imposed time pressure should be tackled from a broad perspective, both responding to the use of the cathedral and its external appearance. The proposal should investigate if the construction crew or visitors can enter Notre Dame's attic space before the entire cathedral with all its Gothic ornamentation is finished. A contemporary example of this idea is the Sagrada Familia in Barcelona, which is, although scarcely used for religious events, open for visitors while construction is still taking place. Additionally, the workshop inside Gaudi's masterpiece is turned into an experience for visitors, who can observe the required craftsmanship from up close (Figure 40).

Besides the role of Notre Dame as a place of worship and as a touristic attraction, the presence of the cathedral as a symbol of Paris is also a matter of external appearance. The by the authorities imposed time pressure doesn't only relate to the use of the building, but also its physical manifestation. The aesthetical quality of the roof during the rebuilding process should therefore be a point of attention. Even without the original cladding and all its ornamentation, the roof's geometrical presence should be quickly restored. Inspiration is drawn from Christo's Pont Neuf Wrapped (Figure 41), transforming the famous bridge at Île de la Cité from an architectural object, into "*an object of inspiration for artists, to an art object.*" (Christo, 1985).

In summary, the design should use the concept of integrating support structures: the incorporation of the structures which are needed during the construction. To limit the construction time, an attempt should be made to use every piece of (support-) structure that's brought to the building site, resulting in efficient use of materials and labor. Notre Dame's attic is imagined to accommodate crew and visitors while other parts of the cathedral are still being restored. At the same time, the external presence of Notre Dame as a symbol should be re-established in an early stage of the reconstruction.



Figure 40 Workshop inside Sagrada Familia showing the construction process to visitors (Morgan-Mar, 2014).



Figure 41 Pont-Neuf Wrapped by Christo, wrapping the famous bridge in canvas for two weeks (Christo, 1985).

4.2. PROPOSED DESIGN STRATEGY

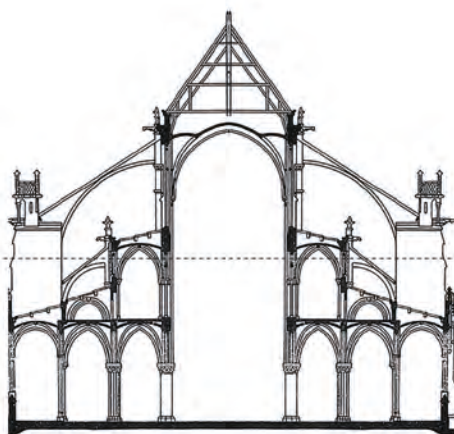
Design Strategy

The presented design proposal, named *The Timber Skeleton*, partly ignores the re-emerging debate on the aesthetical restoration of the cathedral, but approaches the roof's new design in the same practical, technical, and innovative manner as the Master Builders did in the 13th century. Notre Dame's former wooden roof structure could rightfully be called a manifestation of medieval high tech and the proposed design will be a tribute to, *The Forest* and will be built out of engineered timber. However, the function – or non-function - of the attic space and the inconsistency between the timber structure and stone skeleton will be reconsidered.

The strategy seeks to deal with the time pressure imposed by the authorities, by exploring the boundaries between temporary and permanent structures. If one would reason with the spirit of a Master Builder, clever construction methods should be used to deal with time pressure. The proposal investigates and showcases the use of support structures for the reconstruction of Notre Dame de Paris while exploiting their presence as part of the final design. The proposal comprises three steps which are explained and captured in a scheme at the bottom of this page.

Fast Fix

The need for saving Notre Dame from collapsing and further deterioration is evident. The first step will therefore be a Fast Fix structure with the same functional requirements as the one installed by the authorities: make the cathedral weathertight, stabilize the guttering walls, and allow construction workers to reach the vaulted ceiling to remove debris. For its construction, the Fast Fix structure has time-pressure as governing criteria. Each step in its manufacturing, assembly, transportation and installation will be optimized for production speed. At the same time, the geometry and position of the Fast Fix structure will allow the integration of this support structure into the final design. The Fast Fix cassettes will function as the floor of the attic space and as tie-beams for the final structure.



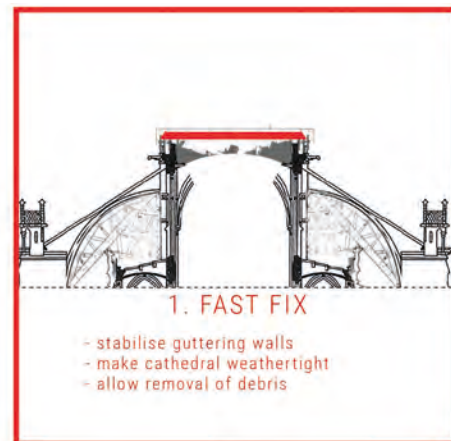
1300



FIRE

- timber roof burned down completely
- the vaulted ceiling partly collapsed
- result: unstable structure without protection against rain

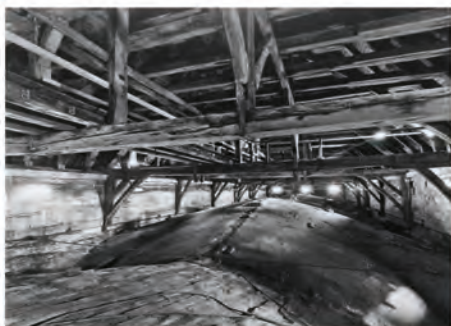
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1. FAST FIX

- stabilise guttering walls
- make cathedral weathertight
- allow removal of debris

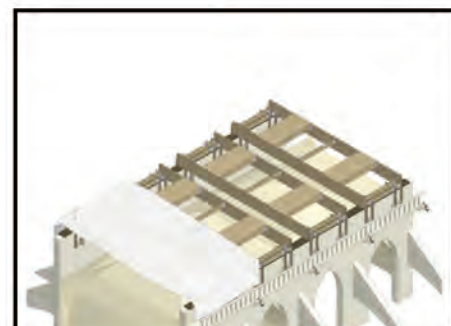
AS FAST AS POSSIBLE!



former timber roof structure: The Forest



burned down timber debris on top of vaults



LVL cassettes on top of clerestory wall with fabric raincover

Field Factory

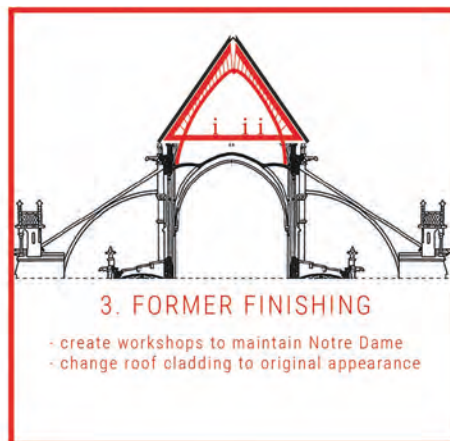
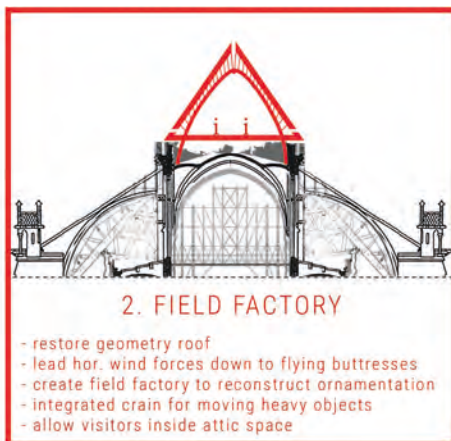
In step two, a Field Factory will be placed on top of the cathedral. Its external geometrical shape is predetermined by the objective of restoring Notre Dame's former gable roof geometry. The roof's finishing initially will consist out of a translucent canvas. The obtained former geometrical shape of Notre Dame's roof will require an immediate answer to the horizontal wind forces that act on the gabled roof structure. The proposed solution is based upon *the Forest*, which splits the vertical and horizontal load paths, and leads the latter down towards the flying buttresses.

The resulting indoor space will function as a publicly visible and visitable workshop, where the crew can work on restoring the cathedral and all its Gothic ornamentation. A crane will be integrated into the structure to lift and transport heavy statues or other building parts. Furthermore, the integration of the Fast Fix cassette tie-beams, will result in a triangular self-equilibrated system. The governing boundary conditions for the Field Factory structure are the limited on-site assembly time and that its placement will not rely on temporary support structures. Due to these demands, prefabrication will be investigated with a specific focus on the possibility of expandable structures.

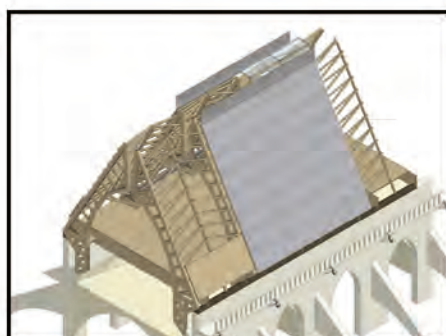
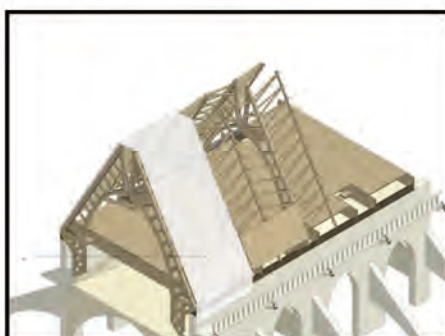
Former Finishing

The last step of the proposed reconstruction strategy will be concerned with completing the roof's external appearance and hence is called: Former Finishing. As desired by the authorities, Notre Dame's external look will be identical to the one before the fire. This condition is slightly modified into the requirement that the roof will look identical from street level. As will be clarified, this allows the creation of a comfortable indoor climate inside the attic space, with daylight entering and the possibility of natural ventilation.

The Field Factory inside the attic space will be turned into a permanent workshop where people can work on the constantly required maintenance of Notre Dame de Paris. The workshop can be seen as a statement to prevent the reoccurring neglect of the cathedral from happening in the future.



6-9 MONTHS



4.3. PROGRAMME OF REQUIREMENTS

General Aims And Boundary Conditions

- **Functional requirements:** The new roof should provide facilities for the construction crew to help in the reconstruction while contributing to a quick reopening of – parts of the – Notre Dame de Paris.
- **Manufacturing and assembly:** The construction of *The Timber Skeleton* shouldn't obstruct other reconstruction activities. Maximum transportation dimensions: width <4 m, height <3.75 m, length: <18.5m. Maximum weight carried by construction crane: 13800 kg.
- **Structural performance:** Improve structural functioning of the roof by aligning it with the stone skeleton of the cathedral so support reactions are guided towards the lateral stability system of the cathedral. No tension reaction forces can be exerted on the masonry structure of the cathedral.
- **Aesthetical:** From the street, one should hardly be able to see the difference between the appearance of the burned-down roof and the new one.

Fast Fix

- **Functional requirements:** Stabilize the stone skeleton of Notre Dame and provide weatherproofing as fast as possible. Function as working platforms for debris cleaning from which the crew can be suspended towards the vaults. Space between the beams to remove large pieces of debris with a crane.
- **Manufacturing and assembly:** Design should be optimized for manufacturing- and assembly speed. The structure should be prefabricated to limit on-site activities which bring the crew members in danger.
- **Structural performance:** Overdimensioned to eventually function as the attic's floor and tie-beam for the roof structure. Durable building materials that enable integration in the final roof design.
- **Dimensions and aesthetics:** Dimensions should allow for integration in the final design. Further, no aesthetical wishes are important when saving Notre Dame de Paris from collapsing!

Field Factory

- **Functional requirements:** Restore geometry of the former roof. Create a Field Factory inside the attic; a space that can be used by the construction crew to restore gothic ornamentation. A crane will be integrated into the structure to hoist heavy statues. Visitors will be allowed to witness the reconstruction.
- **Manufacturing and assembly:** Prefabricated to limit on-site building activities. Manufacturing and assembly should allow for completion within one year.
- **Structural performance:** Guide horizontal wind forces down to flying buttress on leeward side structure. The quest for structural expression requires strong building materials that allow the concentration of forces in slender cross-sections.
- **Aesthetical:** The external appearance will restore the gable roof geometry. The structure should consist out of an expressive structure with *the articulation of individual elements* and *elaboration of the profiles of the structural elements*" (Tzonis, 2004) that's visible inside the attic space.

Former Finishing

- **Functional requirements:** The attic is turned into a visitable workshop that's dedicated to the constant maintenance of the cathedral's Gothic ornaments. The attic space, therefore, needs thermal comfort, with sufficient daylight and thermal comfort.
- **Manufacturing and assembly:** Prefabricated roof panels are placed, after which necessary on-site additions like ornamentation can take place.
- **Structural performance:** The roof cladding can help to stabilize the roof. Furthermore, the structure should be durable to exist for eternity without little maintenance.

5. HIGH TECH TIMBER

With the wish for an expressive timber structure as an homage to *The Forest*, this chapter will first describe the choice for Laminated Veneer Lumber as a construction material. Quick availability requirements during the Fast Fix phase led to a hardwood LVL producer Pollmeier, which produces a product named Baubuche. The hardwood LVL panels also provide great potential to design an expressive Field Factory structure. Information is gathered by online research and interviews with Pollmeier employees. Furthermore, prefabrication strategies and joint types are discussed.



5.1. THE RISE OF ENGINEERED TIMBER

The Rise of Timber

The sustainable properties of wood, both in its embodied energy and renewability, launched a new era of timber construction (Hart and Pomponi, 2020). Stimulated by legislation, building envelopes and the efficiency of energy-using devices improved significantly. The next steps in lowering the building industry's impact on climate change should be focused on material scarcity and minimizing a building's embodied energy, where a lot of energy-saving potential can still be gained. Acknowledgment of the transition from an energy problem to a material problem has made timber the construction material of the future (Rau and Oberhuber, 2016). Nowadays, a broad range of timber construction materials is readily available, although round wood and sawn timber are limited by the size of a tree, which makes their application for Notre Dame's roof structure doubtful. The excursion of the French authorities to Ghana exemplifies the difficulties to obtain timber of large dimensions.

Timber's role in bigger and more complex structures like Notre Dame is partly enabled by the invention of engineered timber products like cross-laminated timber (CLT), glue-laminated timber (Glulam), and laminated veneer lumber (LVL) (Figure 43). Their manufacturing processes slightly increase the material's embodied energy, but liberates timber from the geometrical limitations of a tree, giving rise to a wide range of freeform structures, from which the structural member sizes are only limited by machining and transportation, and their form only restrained by imagination. The geometrical freedom and improved mechanical properties of engineered timber products enable the design of expressive structures. Just as in Gothic architecture, the concentration of forces into small cross-sections, allows a structure to convey the load it carries (Figure 44).



Figure 43 LVL, GLULAM, and CLT (unknown).

Eliminating Timber's Deficiencies

The natural growth of a tree causes timber to be an anisotropic building material, meaning that it has unequal behavior in different directions. Wood grains - microfibers running parallel to the axis of a tree - result in better mechanical performance and increased deformation sensitivity, in the longitudinal direction. A tree's branches and other natural irregularities lead to imperfections that cause timber to be an inhomogeneous material, from which the structural performance is limited by local weaknesses. Timber's structural application is further hampered by the hygroscopicity, the reaction to the moisture content in the air, which mainly affects its deformation behavior. Naturally, trees contain a certain moisture content which after logging typically decreases through evaporation, leading to increased strength performance and shrinkage. The former can be seen as a beneficial effect, the latter can lead to unfavorable stresses or cracks.

The overarching principle of engineered timber products is to deconstruct tree logs into smaller elements, boards or veneers, and subsequently glue them together. The manufacturing process ensures monitoring of timber's deficiencies caused by anisotropy, imperfections and moisture, and provides the possibility to improve the mechanical properties and deformation behavior. Firstly, the anisotropic behavior of the timber can be consciously manufactured through the rearrangement of the grain directions in the boards or veneers. Secondly, the production process allows the removal or redistribution of timber's natural imperfections, leading to structural members with better homogeneous properties. And thirdly, the deconstruction of the tree into boards or veneers allows for a fast and monitored drying process. Hence, the manufacturing speed of engineered timber can be increased and the produced structural members have better dimensional stability.

Respecting Wood's Limitations

Some properties of wood's natural molecular structure remain important in the handling of, and designing with, engineered timber. Timber's relatively low stiffness compared to other construction materials remains frequently governing in the structural design process. This becomes even more pertinent if one includes the characteristic time-dependent deformation behavior of wood. Long-term deformations as a result of loads acting on the material, better known as creep, forces one to consider both the load duration and long-term deformation behavior. In the long run, wood's sensitivity to water can lead to the growth of mould. Protection against water is therefore key, which can only be ensured by protecting the material from rain or by applying special coatings. Finally, the durability of timber structures can only be guaranteed if the possibility of a fire is taken into account. This can be done by passive measures like fire compartmentation and material's fire resistance, or through active systems like sprinklers.

Timber Products Comparison

Practical, structural, and aesthetical criteria of different timber products are compared in Table 1 to see which options are available for *The Timber Skeleton*. The Fast Fix structure requires quick availability, while its integration in the final design becomes easier when the members are dimensionally stable and have great strength properties that limit the cross-section sizes. Great strength properties are also considered beneficial for the entire *Timber Skeleton*, as it allows the concentration of forces and easier elaboration of the structural members' profiles. Besides, the wish for an exposed expressive timber structure without finishing results in the criteria for a beautiful surface.

Laminated Veneer Lumber has been chosen as a building material due to its great strength and stability properties and beautiful surface. The quick availability of structural LVL members can be a concern and will be addressed in the next section.

LVL Design Considerations

LVL consists out of veneers of a few millimeters thick, which are obtained by peeling away the layers of a tree. This production process makes it possible to also use trees with small diameters. The veneers are then glued together and compressed to form LVL panels, which can be used directly as wall panels or floorboards. Panels can be sawn and glued together to create structural members with specific dimensions.

The mechanical and physical properties depend on the veneer's wood type and the exact production process. Table 2 shows that the strength properties of softwood LVL (Kerto-S) are considerably lower than hardwood LVL (Baubuche-S). The wood grain direction of the veneers in LVL panels also has a significant impact on the anisotropic strength properties. A quick quantitative comparison between Baubuche-S and Baubuche-Q (Table 2) shows that Baubuche-S' strength perpendicular to the grain direction in compression and tension are respectively 4x and 40x lower than the strength parallel to the grain. It displays timber's vulnerability for stresses perpendicular to the grain, which can be dealt with by designing the structure in such a way that stresses perpendicular to the grain are kept to a minimum or by making the material more isotropic. By adding veneers in the perpendicular direction, so-called cross-ply, LVL's anisotropic properties can be decreased significantly. Baubuche-Q (Table 2) contains around 14% cross-ply which result in slightly lower strength values in the parallel direction and a significant increase of the perpendicular strength properties.

Easy workability of LVL allows building specific structures like trusses or arch- or dome-shaped systems, which limit the bending moments in the material. Where the design of a structure requires common member sizes, stiffness can be increased by optimizing the cross-sections. Just as is common for steel structures, extra material can be located in flanges at the top and bottom of the cross-section. The web can be kept thin to minimize the self-weight or to allow holes to be made for ducts. The resulting cross-sections, like Finnjoist I-profiles, contain excellent stiffness and strength compared to their weight (Hermelin, 2006).

By using new technology in the production process, engineers and architects can optimize the structural systems and cross-sections. Maggie's care center in Manchester is a beautiful example of the structural expression that can be achieved by optimizing an I-profile section with advanced production methods (Figure 44). Moment forces are resisted through the cross sections' flanges, while the shear capacity is made beautifully visible by diagonals in the web (Figure 44 right).

Table 1 Assessment of practical, structural, and aesthetical criteria for different timber products.

ASSESMENT CRITERIA TIMBER PRODUCTS	QUICK AVAILABILITY OF THE REQUIRED MEMBER SIZES	SUSTAINABILITY	STRENGTH PROPERTIES	DIMENSIONAL STABILITY	SURFACE AESTHETICS
ROUND WOOD	-	++	-	-	-
SAWN TIMBER	-	++	--	-	+/-
GLULAM	++	+	+	+	+
CLT	+	+	+/-	++	+
LVL	+/-	+	++	++	+

Table 2 Strength properties of LVL: Kerto-S, Baubuche-S, and Baubuche-Q (Pollmeier, 2020)

Property			Kerto-S	Baubuche-S	Baubuche-Q
Bending strength	$f_{m,g,k}$	N/mm^2	44	75 ^a	59
Tensile strength	$f_{t,0,g,k}$	N/mm^2	35	60	49
	$f_{t,90,g,k}$	N/mm^2	0.8	1.5	8
Compression strength	$f_{c,0,g,k}$	N/mm^2	35	57.5	62
	$f_{c,90,g,k}$	N/mm^2	6.0	14	22
Shear strength	$f_{v,g,k}$	N/mm^2	4.1	8.0	7.8
Modulus of elasticity	$E_{0,g,mean}$	N/mm^2	13800	16800	12800
	$E_{0,g,05}$	N/mm^2	11600	14900	11800
	$E_{90,g,mean}$	N/mm^2	350	470	800
Shear-modulus	$G_{g,mean}$	N/mm^2	600	760	820
Density	$\rho_{g,k}$	kg/m^3	480	730	730
	$\rho_{g,mean}$	kg/m^3	510	800	800

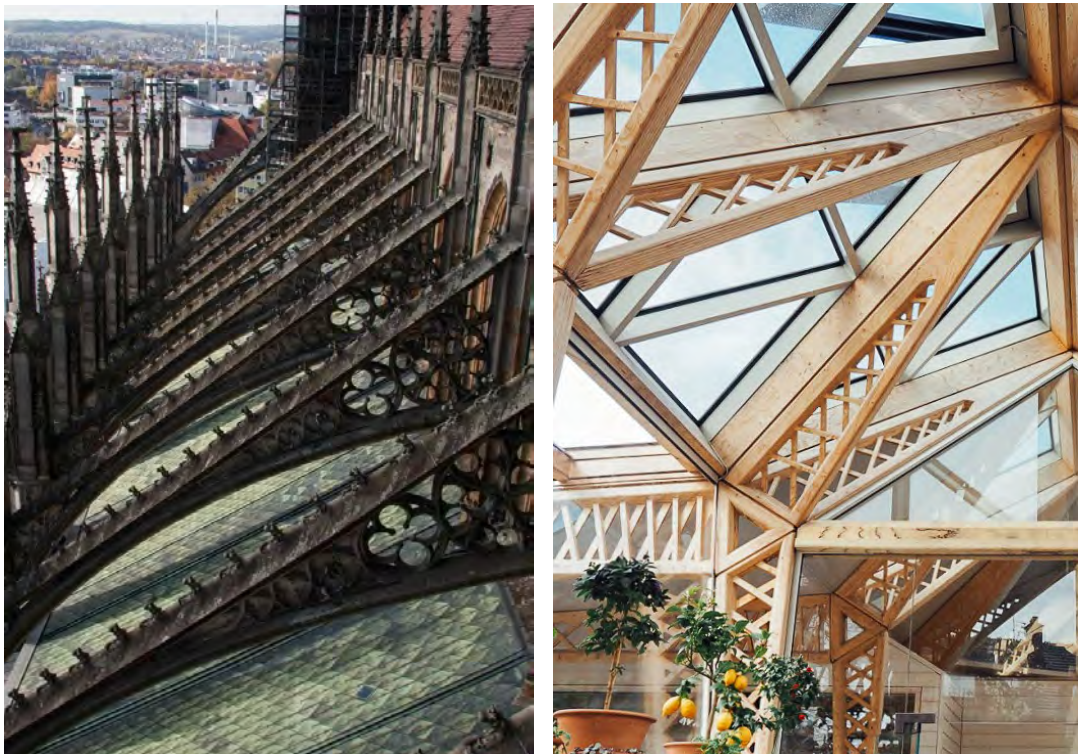


Figure 44 LVL structure (left) & Concentration of forces in openworked Flying buttresses resulting in elaboration of the profiles, in Gothic architecture, often combined with ornamentation (right)

5.2. A FAST FIX STRUCTURE

Baubuche

To design an expressive timber structure and to deal with the time pressure that is imposed by the design strategy, it is decided to use LVL from Pollmeier. The factory, located relatively close to Paris (Figure 45), produces LVL products, called Baubuche, which are produced from European beech, a hardwood species that gives the structural members superior strength and stiffness and an appearance that is also appreciated for its aesthetical quality.

Pollmeier manufactures BauBuche on request and the production process of a large project is around 4 weeks (Hassan, 2020). Locally sourced beech is debarked, peeled into veneers, dried, glued, and compressed together. The production process is organized such that the direction of the veneers can be altered according to the wishes of the client. Structural members with a constant grain orientation can have great structural properties in a specific direction (Baubuche-S), while alternating grain directions can result in uniform deformation behavior and beneficial spreading of stresses (Baubuche-Q). The continuous process of both laminating and compressing makes it possible to manufacture structural elements with infinite length. However, the length of Baubuche panels is a maximum of 18 meters due to handling purposes. The machines produce standard panels with a width of 1820 mm and a thickness of 40 mm, - on request other thicknesses can be produced -. The resulting panels can be seen as the source material from which BauBuche structural elements can be made.

Machining And Finishing

Subsequently to the production of Baubuche boards with a specified amount of cross-ply, the panels have to be modified according to the demands of a project. The manufacturing process, the gluing and compressing of the timber fibers, combined with the use of heartwood results in a product with a relatively high density. The workability of Baubuche is therefore difficult compared to regular softwood LVL. Typically the LVL boards are sawn into the demanded member sizes, or, in the case of complex geometries, milled by a computer-numerical-controlled (CNC) machine into the desired shape.

Typically, it takes 2 weeks of machining to turn the basic BauBuche panels into structural members with the right size and shape (Hassan, 2020). The LVL components can be put together to compose longer, wider, or thicker structural members. Assembling can be done by gluing components on top of each other or by connecting them with nails, screws, or bolts. To ensure stability, the latter method requires a well-considered placement of the connectors, especially when beams are loaded sideways (Marika and Raute, 2019). Generally, the maximum height of structural members is 1360 mm and the thickness is limited to 500 mm due to handling limitations (Pollmeier, 2020).

Reworking can be desirable if the project at hand demands specific surface or edge finishing, fire resistance, or moisture protection. Sanding can be desired to take off sharp edges or to change the aesthetics of Baubuche's surface. When required, the structural members can be finished with a fire retardant coating to increase their fire resistance. If the LVL gets exposed to moisture due to for example transportation, there is also a possibility to apply a moisture-protective coating, which guarantees a 12-week long water resistance. Packaging of the structural members and transportation from the Pollmeier factory to Paris takes around 2 weeks for a project the size of Notre Dame's roof (Hassan, 2020).



Figure 45 Factories of LVL in Europe with 1-day transportation circles to Paris.

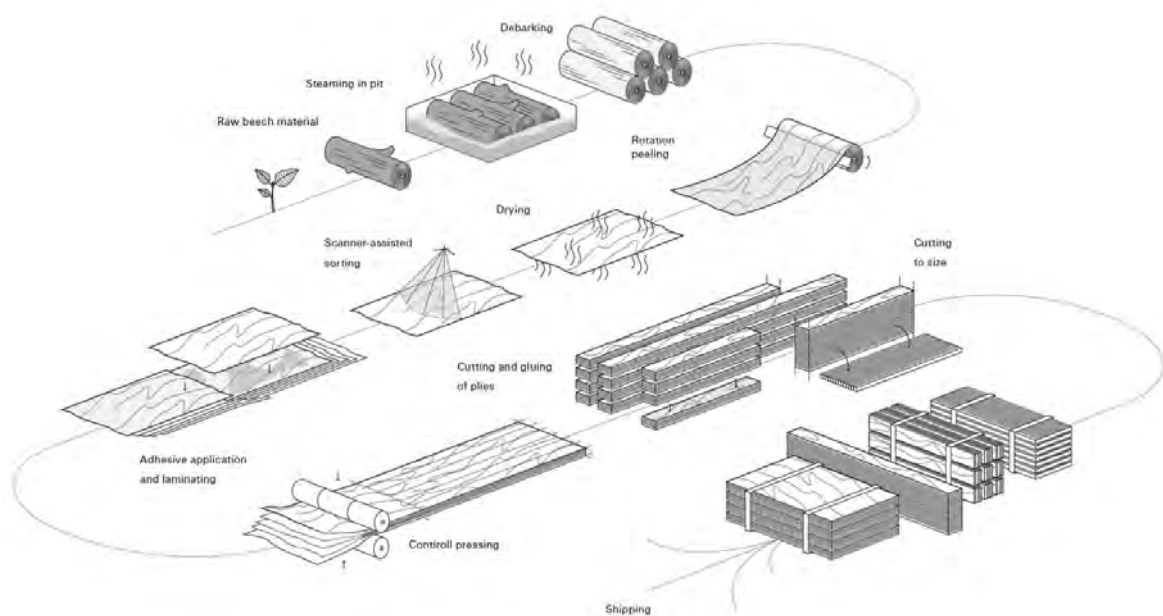


Figure 46 Production process of BauBuche (Pollmeier, 2020).

5.3. LVL PREFABRICATION

Prefabrication

Fuelled by the increased size and complexity of buildings, limited space at high-density construction sites, and time and cost efficiency, prefabrication has become an indispensable factor in the modern building industry. Prefabrication of steel members already flourished during the 19th-century industrial revolution, with the Eiffel Tower as an early example. And precasting of concrete became mainstream in the 20th-century, driven by a combination of Fordism and the prefab-housing boom after the first two World Wars. Timber's significantly lower density compared to steel and concrete, allows the off-site fabrication of large sub-assemblies. It led to new modular design strategies with unprecedented erection speed, for example employed during the construction of Hotel Jakarta in Amsterdam (Figure 47).

Considerations And Limitations For Prefabricating LVL

The large production machines that produce the LVL boards require indoor climatic conditions, and regulated temperature and moisture levels are used in pursuing a construction material with good physical and mechanical properties (Pollmeier, 2020). The LVL boards are due to their limited thickness (40-60 mm) sometimes unusable for immediate use in primary structures. Frequently they are therefore assembled to form sub-assemblies like floor cassettes, or frame walls.

The design of prefabricated LVL sub-assemblies is governed by their size rather than their weight. Other considerations for prefabricated LVL products revolve around the strength, stability and robustness during the transfer from factory to the construction site. Sub-assemblies frequently contain primary structural members located in their outer edges. They guarantee robustness during transportation and frequently fulfill a structural role in the finished building. A characteristic doubling of the structural members can occur as shown in Figure 48.



Figure 47 Hotel Jakarta, Amsterdam. Stacking of completely insulated and furnished hotel rooms (Search, 2019).



Figure 48 Macquarie University clearly showing the doubling of the primary beams (Arup, 2018).

Machine- x Man-Made

The choice to place a prefabricated LVL roof on top of the stone structure of Notre Dame de Paris comes with a few benefits and challenges. The main advantage of prefabricating the roof is the minimal on-site building activity, which means that the roof's reconstruction is neither obstructing nor obstructed by, all the other reconstruction works taking place in and around the cathedral. The limited on-site operation also helps to limit the construction site risks for the crew, which is especially necessary during the first stages of the reconstruction when the cathedral is still unstable.

An important factor for the success and efficiency of prefabrication strategies is the design's grid and the division of the structure into sub-assemblies. Challenges are therefore expected in the alignment of a prefabricated LVL structure, geometrically bounded to transport and handling measurements, and Notre Dame's stone skeleton, from which the guttering walls are positions 13 meters apart, and each bay measures 5.7 meters. BauBuche's on-site modification limitations, combined with the relatively irregular masonry of Notre Dame, require strategies to be developed to connect the ancient masonry and LVL timber members. Options are to be found in a tailor-made manufacturing process or by creating adaptability in the joints. The former method can make use of laser scans (Sandron and Tallon, 2013) and CNC machining to arrive at a perfect fit. The latter will be further explained in the following section.

Prefabricating Notre Dame's Roof

Ribbed slabs or cassettes are frequently used to prefabricate horizontal panels (Figure 49). The functional requirement of the Fast Fix structure to allow construction workers to reach the debris on top of the vaults though limits the possible use of horizontal panels at the bottom and top of the prefabricated slabs, which are normally applied to increase the stiffness of the sub-assembly and to deal with LVL boards' buckling sensitivity. Instead, stiffness and stability of the structural members of the Fast Fix structure have to be guaranteed by increasing by strategically positioning secondary members or by increasing the cross-sections.

For the Field Factory's structure, different prefabrication strategies can be utilized. Figure 50 shows some typical sub-assemblies of gable roofs, which have their specific benefits and drawbacks considering the number of joints, required support structures, and transversal and longitudinal stability. The hinging gable roof (Figure 50, right) is of particular interest for Notre Dame's roof structure. The construction strategy, mainly adopted for mass housing in the Netherlands, limits on-site modifications and contains joints that are easily accessible. The use of kinematical connections allows prefabrication which is less restricted to handling and transportation sizes. The concept of using expandable roof segments will be further investigated in the following section.

The last step of the proposed reconstruction strategy, applying the Former Finishing on the roof, can also be executed with prefabricated roof panels. The lead cladding and Gothic ornamentation can be produced easier off-site or inside the Field Factory of Notre Dame's attic.



Figure 49 Production of prefabricated LVL cassettes (Stora Enso, 2016).

TRUSS		SINGLE PANEL		HINGING DOUBLE PANEL (klapdak)	
number of connections	+/-	number of connections	+	number connections	+++
transversal stability	+	transversal stability	--	transversal stability	+++
longitudinal stability	--	longitudinal stability	+/-	longitudinal stability	++
required support struct.	-	required support struct.	--	required support struct.	++





Figure 50 Typical sub-assemblies of a gabled roof structure, from left to right: prefab frames, prefab roof plates, and hinging roof segments.

5.4. LVL CONNECTIONS

Connection Considerations

Connections fulfill a crucial role in the structural behavior of timber structures (Descamps et al., 2014). Due to timber's relatively low strength and stiffness, connections display significant joint deformation. The resulting large deflections are frequently governing the structural design (Aicher, Garrecht, & Reinhardt, 2014). A timber joint's deformation margins have also benefits. They allow the joint to act in a ductile manner and provide the possibility of redistributing forces in the structure, which can be key when dealing with thermal and temporal deformation behavior. These structural considerations are only one part of designing connections, which is further heavily influenced by costs, construction time, robustness, and aesthetics.

Carpentry Joints

Depending on the nature of the carried forces, a wide range of carpentry joints can be built. The structural functioning of carpentry joints depends on the size and direction of the connecting surfaces between the structural members (Figure 51). Due to a carpentry joint's geometrical dependence, accurate machining is required to guarantee a proper load transfer. Laminated Veneer Lumber has a relatively high stiffness compared to other timber products and Baubuche joints are therefore even more dependent on accurate machining for proper structural functioning of joints. The exact geometry of the LVL members is preferably known beforehand since the required cut-outs and holes can be accurately made by machines in the factory. Technological developments like CNC machining, have increased the possibilities and accuracy of carpentry joints. The automation of the production processes helped to significantly drop the price of carpentry joints, initiating a new era of complex timber structures (Weinand, 2016).

Joints With Metal Fasteners

Where carpentry connections are inadequate for aesthetical, structural, or execution reasons, the use of metal fasteners can be an outcome. They can be divided into two main groups based on their transfer mechanism (Blaß and Sandhaas, 2017). A "surface-type" group of fasteners, such as toothed-plate and punched metal plate fasteners will not be considered for they are not fitting the aesthetically wishes of the proposed design for Notre Dame de Paris. The second group, "dowel-type" fasteners, consists out of a wide range of connection types: nails, staples, bolts, dowels, and screws.

Typically, these joints generate shear and embedment stresses in the timber and bending and tensile stresses in the fasteners. Different failure modes can be considered depending on the geometry of the applied materials (Figure 53). The area directly around the timber joints, which can be subject to high concentrated stress peaks, is often the weakest point of the connection. Detailing is therefore crucial in achieving the desired structural behavior. The robustness of engineered timber, and especially LVL that contains cross-ply, make these types of joints less prone to splitting and allows fasteners to be located close to the edge.

Dowel-type fasteners can be combined with preformed steel parts, which have many benefits, among them: variety in shape and application, speed of construction, and costs. The option of recessing the steel plates inside the timber elements results in a non-visible steel connection, an efficient load transfer, and protection against fire (Figure 52). The theory around the calculations of this joint evolve around the Johansen models (1949) and their failure mechanisms that depend generally on the joint geometry, the embedment strength of the LVL, and the plastic bending moment of the dowel.

For the roof ridge joint, the failure mechanisms taken under consideration are f, g, and h – which have similar failure mechanisms as respectively c, d, and e - (Figure 53).



Figure 51 Wind and Rain Bridge, connections without any mechanical fasteners (Holohan, 2018).



Figure 52 Recessed steel plate connection in Baubuche (Pollmeier, 2020).

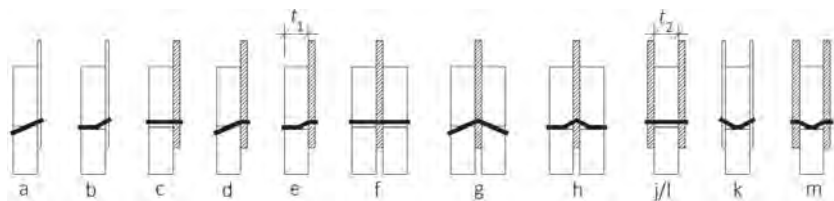


Figure 53 Johansen failure modes (Blaß and Sandhaas, 2017).

Connections For Notre Dame

The proposed design strategy requires the use of a few specific connection types to allow a high production speed, joints between LVL and ancient masonry, and the prefabrication of expandable sub-assemblies.

Fast Fix

The fastest way to make joints between BauBuche members is with preformed steel parts like joist hangers or corner profiles (Figure 54, left). They are often used in non-visible areas since they have poor aesthetical quality. Predrilling of the fasteners requires some extra time but results in an accurate structure that is easy to assemble. Considering the structural functioning of the joints, one should give extra attention to the eccentricities and possible tension forces perpendicular to the grain (Hassan, 2020).

The connections between the new timber roof structure and Notre Dame's stone skeleton are governed by the wish to have limited modifications to the ancient monument and on-site adjustability to deal with the geometrical and structural irregularities of the masonry. The former means that moment- and tension forces can not be transferred to the stone structure and that connections rely mainly on compression. Compression in joints is largely dependent on a structural system's geometry and should therefore be considered in an early stage of the design. An alternative strategy used by the authorities is the application of clamping joints, which can also tackle problems with the geometrical irregularities of masonry (Figure 54, right). The use of slotted hole connections allows on-site modifications without much effort. They should be applied such that they can be removed after the design is finished. To take the structural irregularity of the stone skeleton into account it is important to vertically align both guttering walls and to prevent large force concentrations. Strategies can be applied like applying intermediate layers or spreading forces over multiple structural members as executed by the authorities. Their necessity will become clear during the structural verifications.

Kinematical Joints

To produce prefabricated expandable structures kinematical joints will be utilized. To keep the connections relatively simple, only revolution joints will be considered, better known as hinges. These types of joints allow the rotation of two elements in a single plane and are frequently used to create statically determinate structures. The nature of hinge joints, to consist out of a single often cylindrical, element, results in largely concentrated stresses. For timber structures, this can be problematic due to their limited embedment strength and vulnerability to fail in a brittle manner. In the case of large structures, engineers use steel as a solution, which is stronger and provides more ductility. Unfortunately, this frequently results in relatively bulky connections which are aesthetically dominated by large steel elements.

A strategy is therefore proposed to spread the loads over many small members, which results in modest forces acting in the connections and provides the opportunity to use rather simple hinging connections (Figure 55). Grid shell joints are taken as an important reference (Figure 56). The strategy will result in more joints in the structure, but which are significantly easier to construct. Envisioned are joints that minimize the use and visibility of steel.



Figure 54 Joint types used by authorities: left: Joist hangers used to connect members of cassettes. right: Steel clamping connections located underneath flying buttresses.

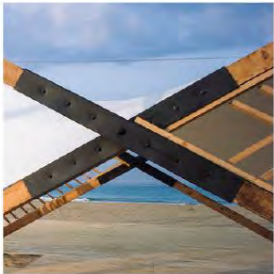
AMOUNT OF JOINTS	FORCE TRANSFERRED	EXECUTION EFFORT
 few	 high	 high
 many	 low	 low

Figure 55 Two (partly) kinematical joint strategies.

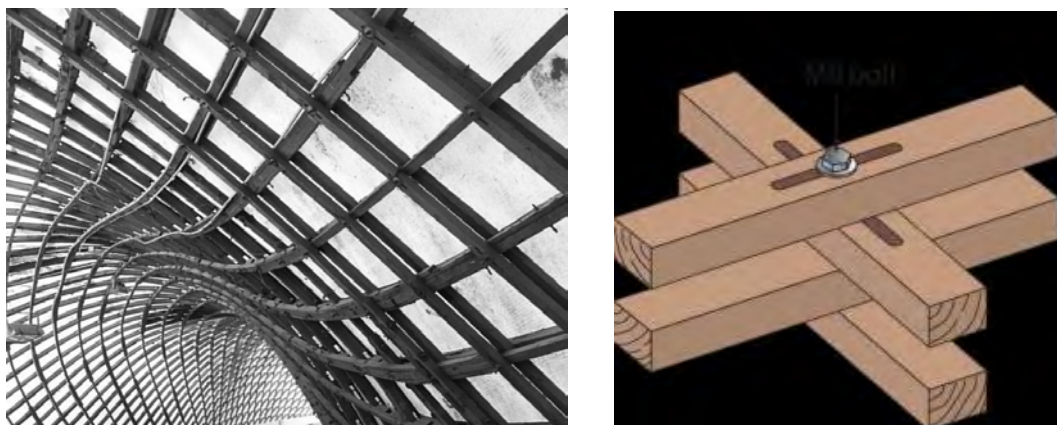


Figure 56 Mannheim Multihalle by Frei Otto, displaying simple joints with limited use of steel, using a single bolt.

5.5. CONCLUSIONS

Which materialization, manufacturing methods, assembling strategies, and joint types can be utilized?

Engineered timber products help to deal with timber's natural deficiencies and create stronger, stiffer, and more reliable construction materials. The geometrical freedom, excellent weight to strength ratio, dimensional stability, and speed of production, make LVL an appropriate building material to reconstruct the roof of Notre Dame de Paris.

When designing with LVL one should take into account:

- Different material properties depending on direction, with special attention to tensile weakness perpendicular to the grain. These can change according to the number of cross-ply.
- Time-dependent behavior depending on the load-duration.
- Sensitivity for climatical conditions: temperature and moisture.

Fast Fix

A fast production strategy is proposed using BauBuche S boards which are stored in Pollmeier's warehouse. They have the following dimensions: 18 meters long, 1840 millimeters wide, and 40 millimeters thick. Using these elements to construct the Fast Fix structure saves at least 4 weeks of production time and in reality much more since there is a waiting list for the manufacturing process. Limited tooling, mainly sawing, will be used to manufacture structural members with the correct sizes. Due to the limited thickness of the material (40 mm), multiple panels will be bolted together to form the thicker primary beams. Steel joist hangers and metal fasteners are good options to quickly create proper connections between the secondary beams and the primary beams. A moisture protective coating will be applied, which will protect the cassettes for up to 12 weeks. Tooling and assembly of the cassettes are estimated to take 2 weeks after which packaging and transportation to Paris would take another 2 weeks. The total production time is therefore estimated to be around one month.

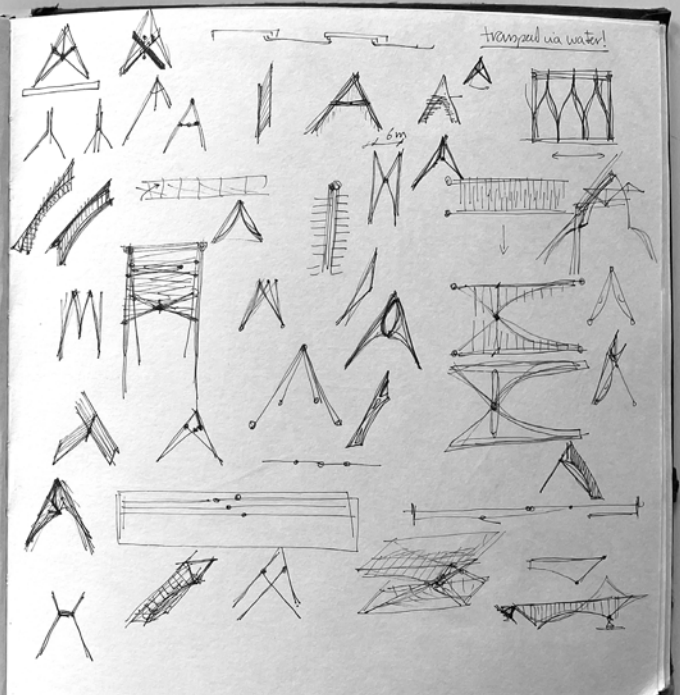
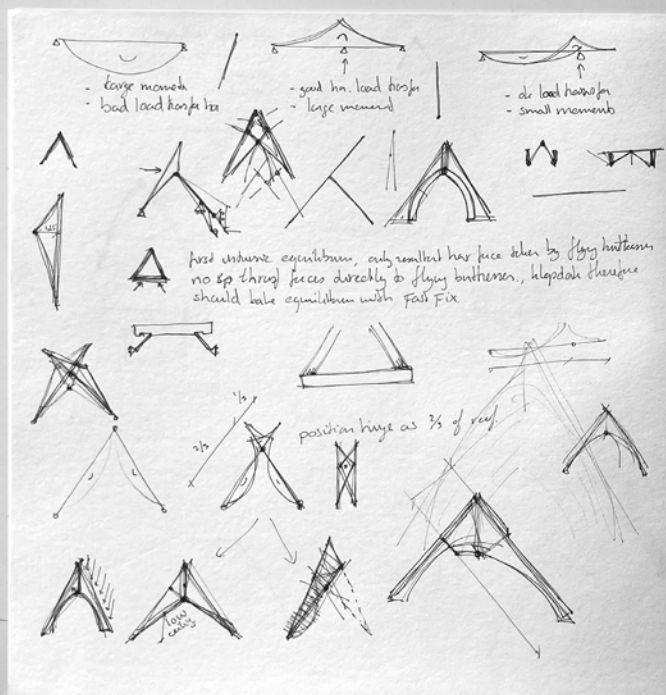
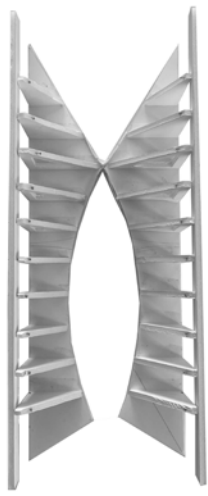
Field Factory

For the production of the Field Factory structure, time isn't the governing design criteria. The production of BauBuche panels, therefore, allows for more conscious decisions considering the number of cross-ply and the geometry of the primary structural members. CNC machining will be employed to guarantee speed and precision while manufacturing an expressive structure. Maggie's care center is an important reference for the manufacturing process. A sub-assembly's geometry is limited by transport dimensions and weight: width <4 m, height <3.75 m, length: <16.5 m, and a maximum weight of 13.8000 kg – due to the crane at the construction site. To align the new roof structure with the existing masonry skeleton of Notre Dame large expandable prefabricated sub-assemblies with kinetical joints will be manufactured. The joints are envisioned to be relatively simple with the grid shell joints as the main reference.

6. GLOBAL DESIGN AND MANUFACTURING

This chapter describes the global design of Notre Dame's new timber structure, *The Timber Skeleton*. Model making and sketching, as is displayed on the adjacent page, were mixed with calculations and literature research - presented in the previous chapter -. The sketches on the right only show a single page of the long and complex design process of combining practical challenges and aesthetical wishes. Practical design aspects like accessibility and building physics can only be harmoniously integrated if they are taken into consideration at an early stage of the design process. The last section of this chapter will therefore be used to specify these kinds of practical design issues, which are portrayed through the envisioned experience of visitors. The chapter is accompanied by a set of technical drawings and impressions that capture the assembly and design premises.

Figure 57 Top: Physical model of expandable Field Factory sub-assemblies in transversal section. Bottom: pages from notebook displaying sections



6.1. FAST FIX

Prefabricated Cassettes

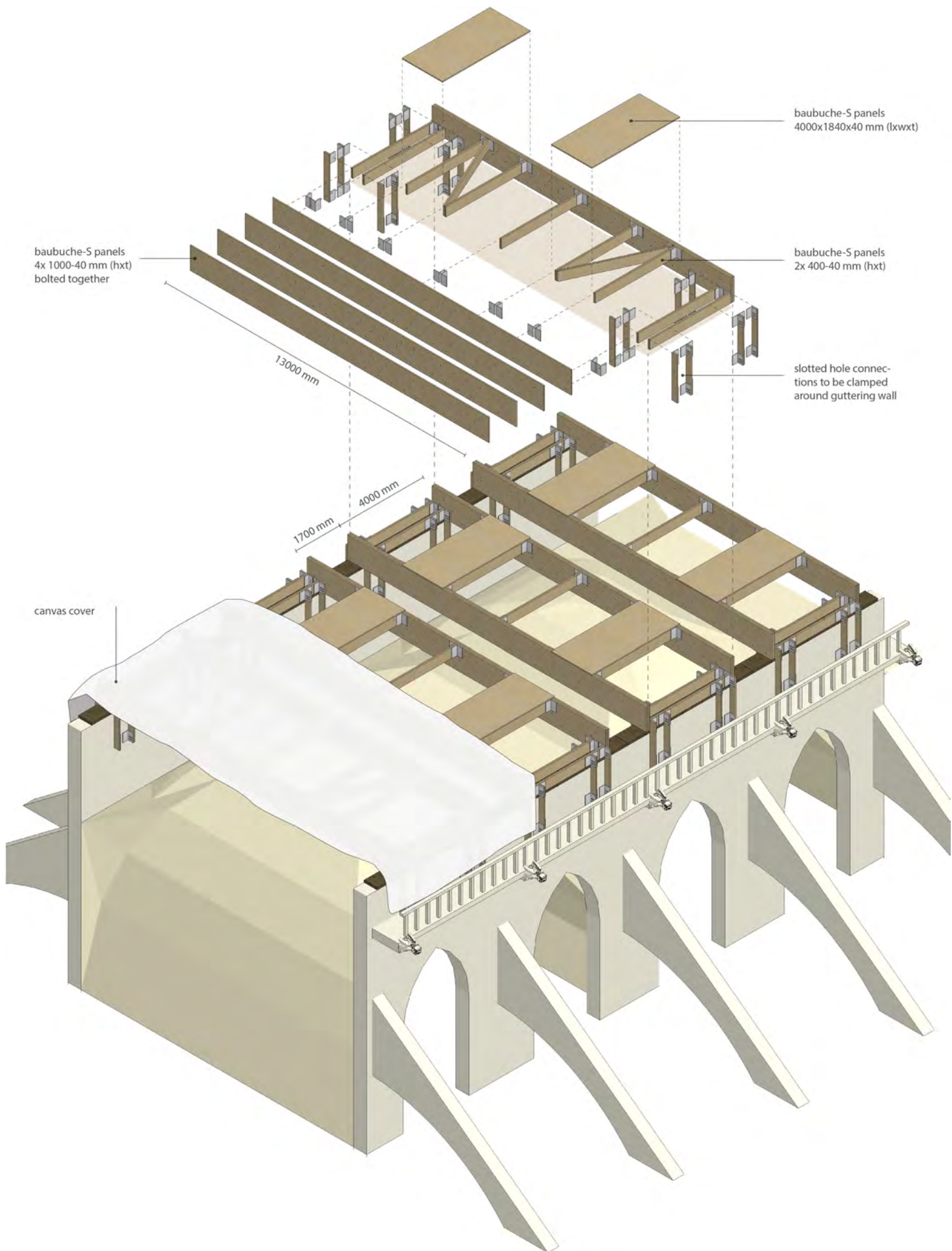
The structural design of the prefabricated cassettes for the FAST FIX structure is governed by the speed of fabrication and is limited by transportation sizes. To facilitate quick production and safe placement strategies have been discussed above. BauBuche S boards will be used since they are the only LVL panels that are readily available. After tooling they have the following dimensions: 13 meters long, 1000 millimeters high, and 40 millimeters thick. The cassettes are 4 meters wide due to transportation limitations and their primary beams will be positioned above the flying buttresses.

Primary Beams

Due to the limited thickness of the BauBuche S boards, multiple panels will be assembled to form thicker primary beams. Different connecting strategies can be used, but partly threaded screws are considered to have the fastest assembly time and best structural performance. Alternatives like nails won't clamp the LVL boards together and bolts would require predrilling of holes and multiple steps in the assembly. To avoid predrilling, special partly threaded screws are used: Schmid Schrauben Hainfeld (ETA 12/0373). These fasteners have high axial strength and a special head avoids the veneers from splitting. In this way, a proper clamping of the individual panels can be guaranteed. A suitable fastening pattern will be applied, where for every 600 mm length of the beam 3 screws are positioned diagonally (Pollmeier, 2020).

Secondary Beams And Horizontal Panels

Steel joist hangers and screws will be used to interconnect the primary beams with secondary structural members. The connections are located at the top part of the primary beams to tackle stability problems which can result in for example lateral-torsional buckling. With the same intention, horizontal boards are positioned on top of the secondary beams. Diagonals are added to stabilize the cassettes, while the horizontal boards provide the possibility for the construction crew to walk on top of the cassettes.



6.2. FIELD FACTORY

Primary Structure: Rafters

Just as in Gothic cathedrals, the structural and aesthetical requirements are envisioned to come together harmoniously in an expressive design. The starting point of the structural design is a standard hinging gable roof segment consisting of two diagonal members, for gable roof's so-called rafters, interconnected with a prefabricated hinge and supported by two pinned supports (Figure 58.1 & Figure 58.2). An optimization in the structural scheme is sought by considering the main loads acting on the rafters.

The triangular shape of the roof's structure contains good resistance against the Field Factory's crane, as the point load results only in axial forces in the rafters, shown in Figure 58.1. Stresses and displacements due to the integrated crane are therefore expected to be limited, although the compression stresses should be taken into account for the other load cases.

It is expected that the structural design of the rafters will be governed by the deflections that are mainly the result of wind-induced moment forces. The initial triangular structural scheme shows poor performance against forces perpendicular to the roof's surface – assumed to be the resultant of wind-loads and self-weight -, which results in an unfavorable moment distribution and its associated deflections (Figure 58.2).

The functional and structural boundary conditions of Notre Dame's roof offer little design freedom. A strategic position of the prefabricated hinge at the roof's ridge though can provide a significant reduction in the moment forces. If this hinge is schematized as roll support, and the roof's surface is considered to be loaded by an equally distributed load perpendicular to the roof plane, the mechanical scheme turns into Figure 58.3. With the positioning of the hinge on $0.3 \cdot L$ of the complete length L , the maximum moment force can ideally be lowered by 66%.

In the pursuit of structural expression and suitable aesthetics for Notre Dame's attic space, this improved mechanical scheme is crystallized in the shape of the rafter's cross-section, resulting in a characteristic Gothic pointed arch shape (Figure 58.4). It helps to create a spacious attic space, making it easier to hoist and transport gothic ornamentation elements. Although this cross-section provides great resistance against the hogging moment at the roof's ridge, further validation and optimization are expected to be needed to verify the deflections induced by the sagging moment.

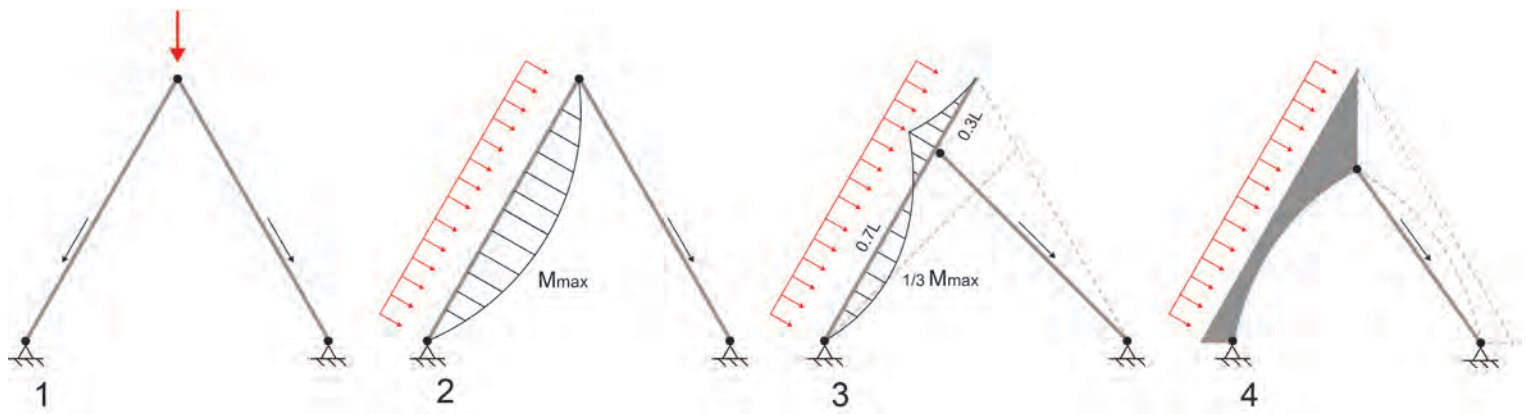
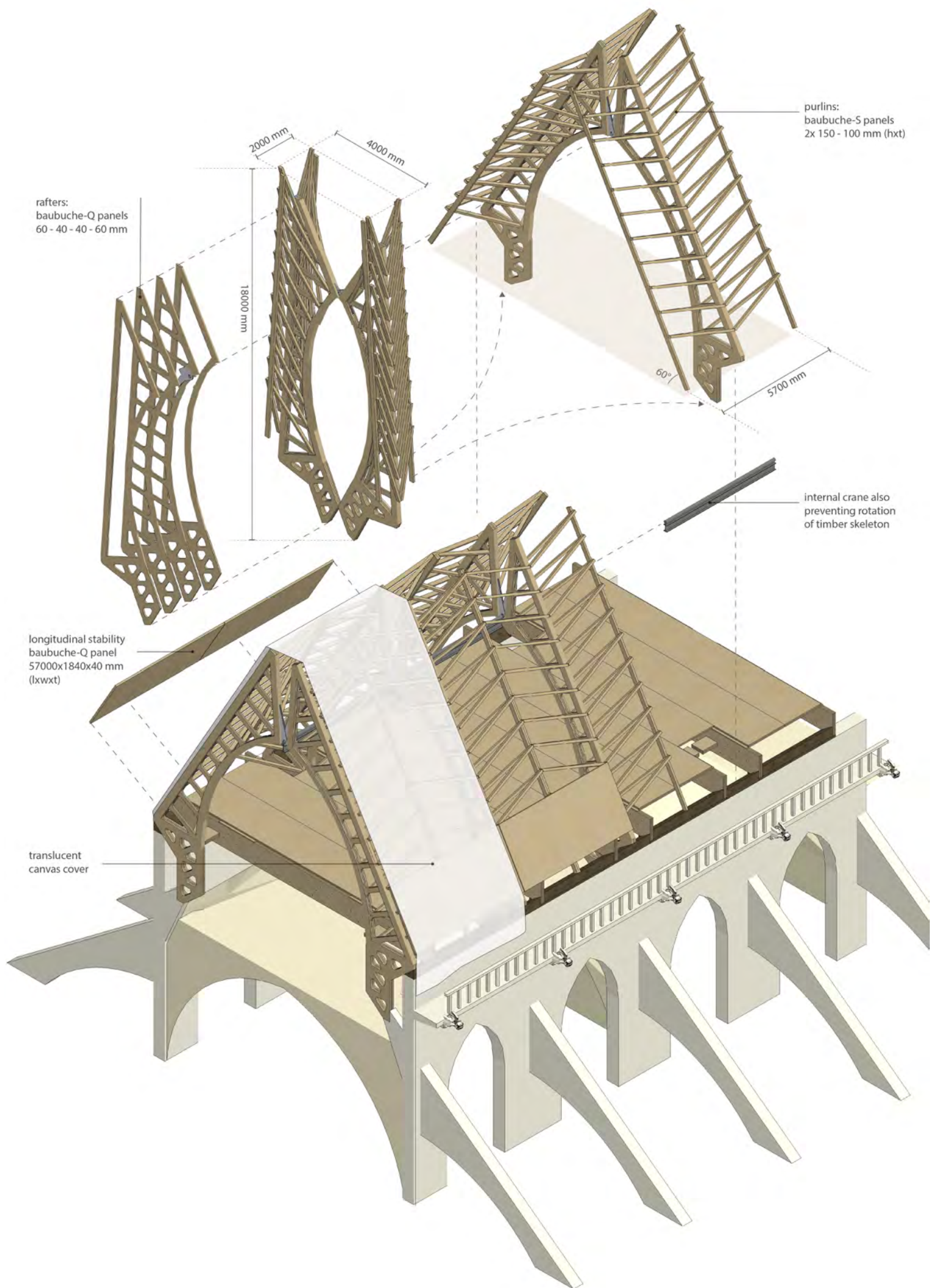


Figure 58 Evolution of Notre Dame roof's mechanical scheme.



Integrating Fast Fix Cassettes

To counteract the horizontal thrust forces at the base of the gable roof, the rafters will be connected to the Fast Fix cassettes which in this way will be utilized as tie-beams (Figure 59). Validation needs to prove if the primary beams of the Fast Fix structure have sufficient residual tensile strength to resist the thrust forces, and if not, reinforcement or adjustment of the cassettes will be required.

It is expected that, like the former roof structure of Notre Dame, the triangular self-equilibrated roof structure can't be horizontally supported by the guttering walls. To lead the wind-induced horizontal forces down to the flying buttresses it has opted to position the rafters between the Fast Fix beams so they can be extended down until the top part of the vaults. Further structural analyses and validation have to prove if this strategy works properly.

Primary Structure: Purlins

In the proposed prefabrication of the gable roof structure, 5.7 meters wide roof segments are unfeasible due to the maximum road transportation measurements. The expandability of regular hinging gable roof segments is taken as a starting concept to tackle this problem while taking prefabrication to the absolute limit and profit from all its benefits.

To limit the practical execution, preference is given to simple hinging connections with limited (visible) steel parts, which is achieved by spreading the forces over a multitude of members. The expansion of the gable roof segments' is captured in transversal (top) and longitudinal (bottom) elevation in Figure 60. It has opted to use a single purposeful kinetic motion of the purlins.

The purlins are used to tackle problems with buckling of the rafters, from which both the top and bottom flange have to be stabilized. The purlins are not merely used as structural members but are at the same time used for aesthetical purposes in designing a reinterpretation of *The Forest*. The purlins' cross-sections relate to the arch-shaped rafters resulting in a gradual shift in angle. The linear structural members together create a ruled surface that can be seen as a ceiling and capture movement in the structure. The physical model at the top of Figure 57, displays a design in which the purlins are positioned horizontally, while the drawing at the bottom of the adjacent page shows the purlins at an angle with the intention to decrease the bending moments parallel to the roof's plane and instead take profit of the axial strength of the members.

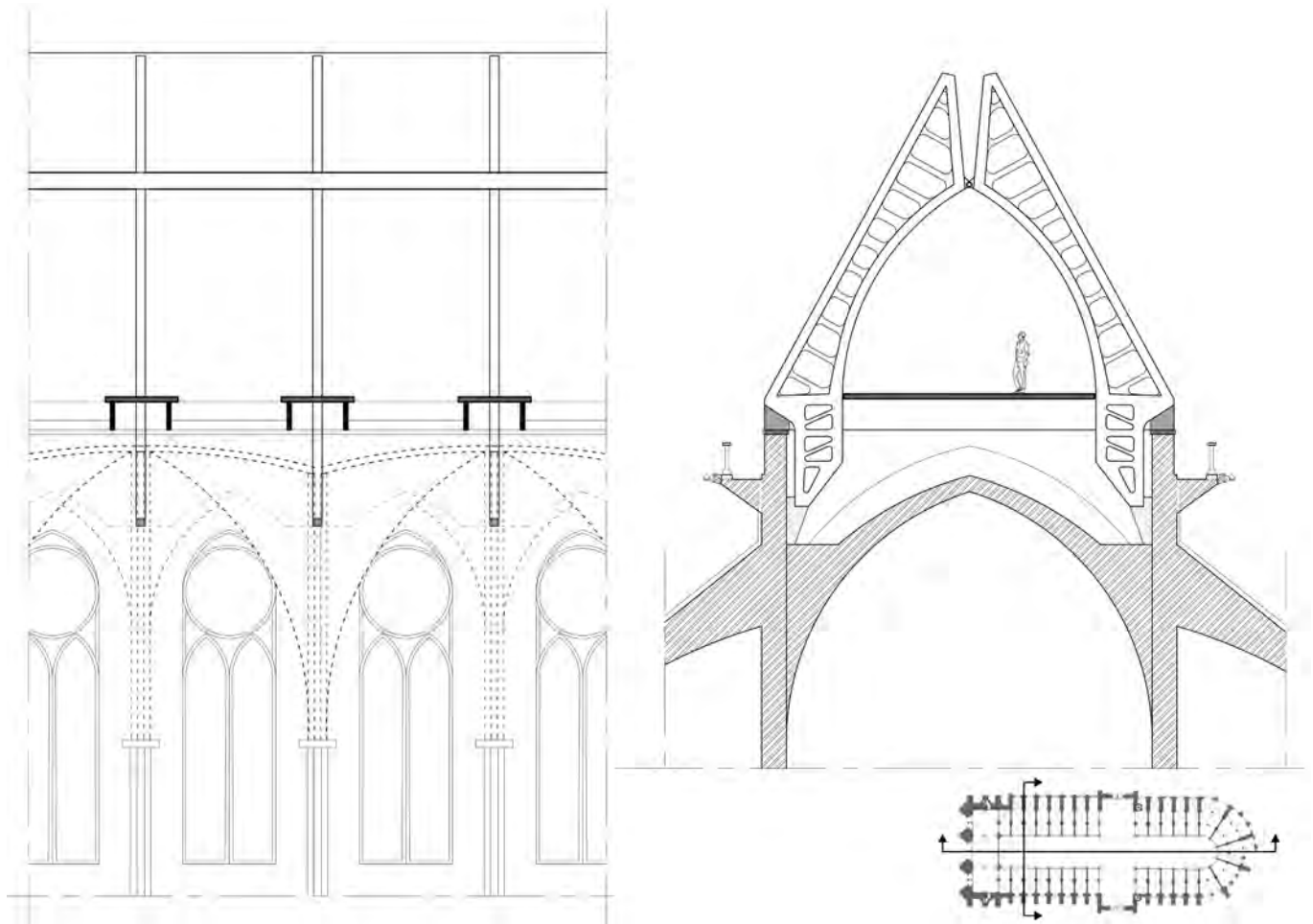


Figure 59 Left: longitudinal section. Right: transversal section of Notre Dame displaying the Fast Fix cassettes working together as tie-beams and the extended rafters to provide horizontal support near the flying buttresses.

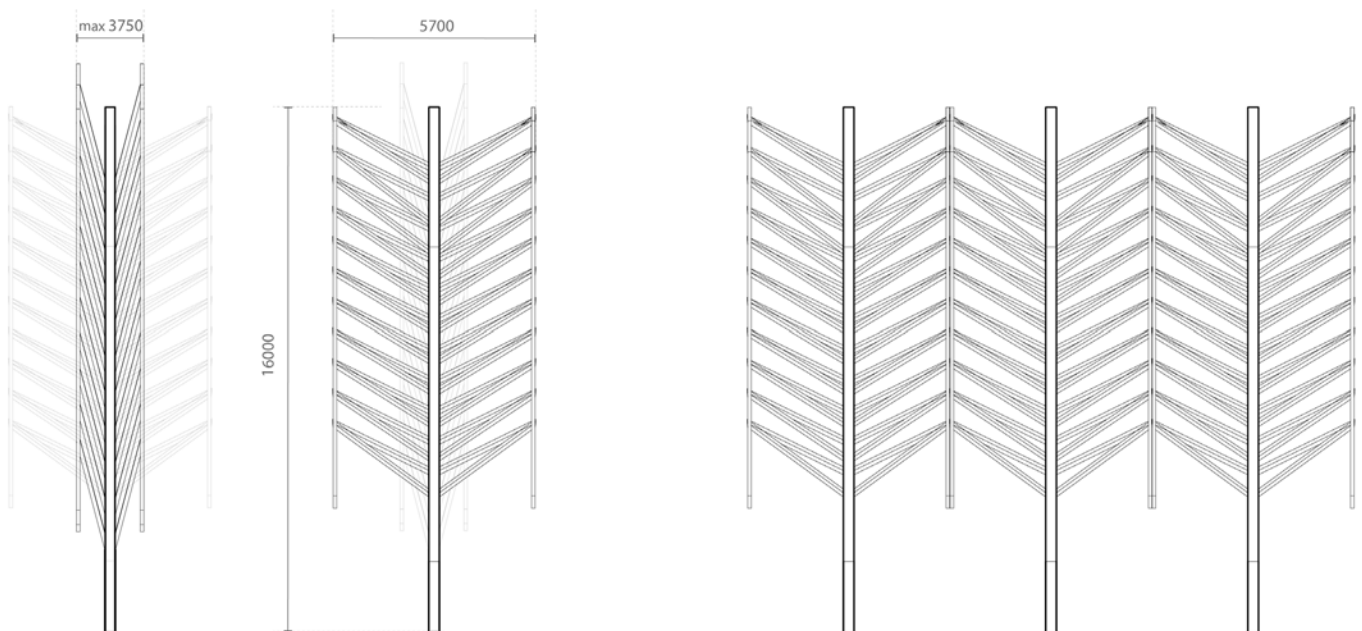


Figure 60 Expanding of the gable roof segments in transversal (top) and longitudinal (bottom) elevation. The physical model at the top shows the initial design with horizontal purlins. Bottom drawings shows purlins at an angle decreasing bending moments.

Temporary Longitudinal Stability

The expandability of the roof segments requires extra measurements to guarantee temporary longitudinal stability. Notre Dame's former roof structure relied for this on the roof's paneling, central spire, and the choir. Since the latter two are out of the focus of the project, alternatives will be proposed which are schematically captured at the bottom of this page.

A few bracing options can be considered that provide stability using diagonal members (Figure 61 A, B & C). Option A is the most straightforward approach, using steel tension rods resulting in a great structural performance with little bending moments. Alternative B provides great aesthetics since all the bracing is hidden under the attic's floor. The structural performance though is relatively poor, with large bending moments occurring in the rafters. C explores the possibility of elongating a few purlins to become bracing. The good structural performance is accompanied by extra construction time and structural members which will be difficult to be positioned. Option D is proposed with the idea to both create longitudinal stability by reinforcing the integrated crane of the Field Factory. For alternative E, a single horizontal member or part of the roof's eventual paneling is placed to create bracing. F is an out-of-the-box idea in which the temporary canvas, covering the attic, functions as bracing. Research has been done on fabric that can resist high tension forces, but for the scope of the thesis, this option is not further investigated.

Option E has been favorited for its strong relationship with the project's concept of integrating every piece of structure used during the construction process. The roof paneling can easily be integrated into the eventual Former Finishing of the project. Besides, the paneling functions as a guardrail preventing crew members or visitors to reach the relatively vulnerable temporary canvas cladding of the Field Factory. Its structural performance is not great but will be investigated. while A is taken as a reliable alternative.

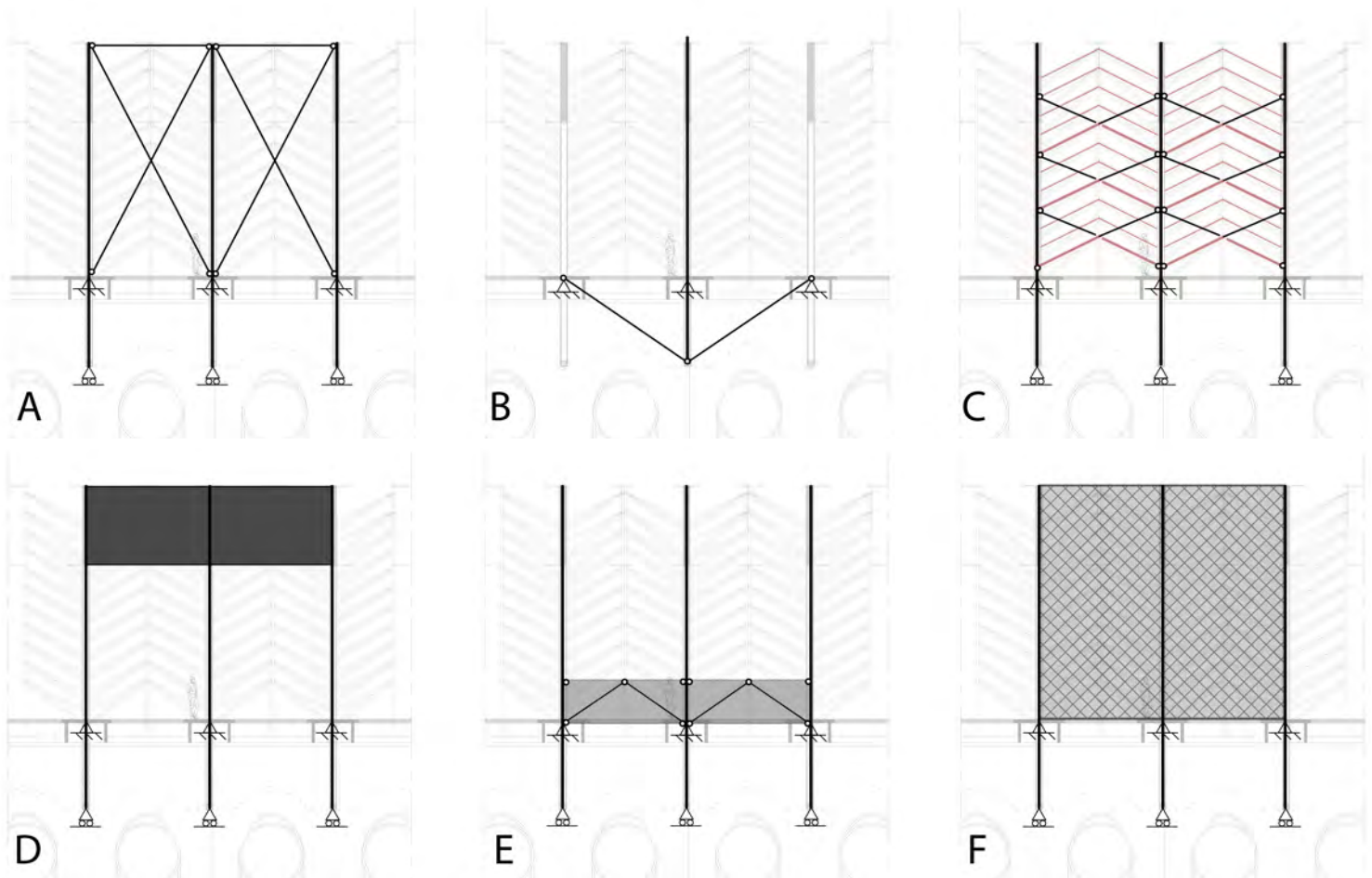
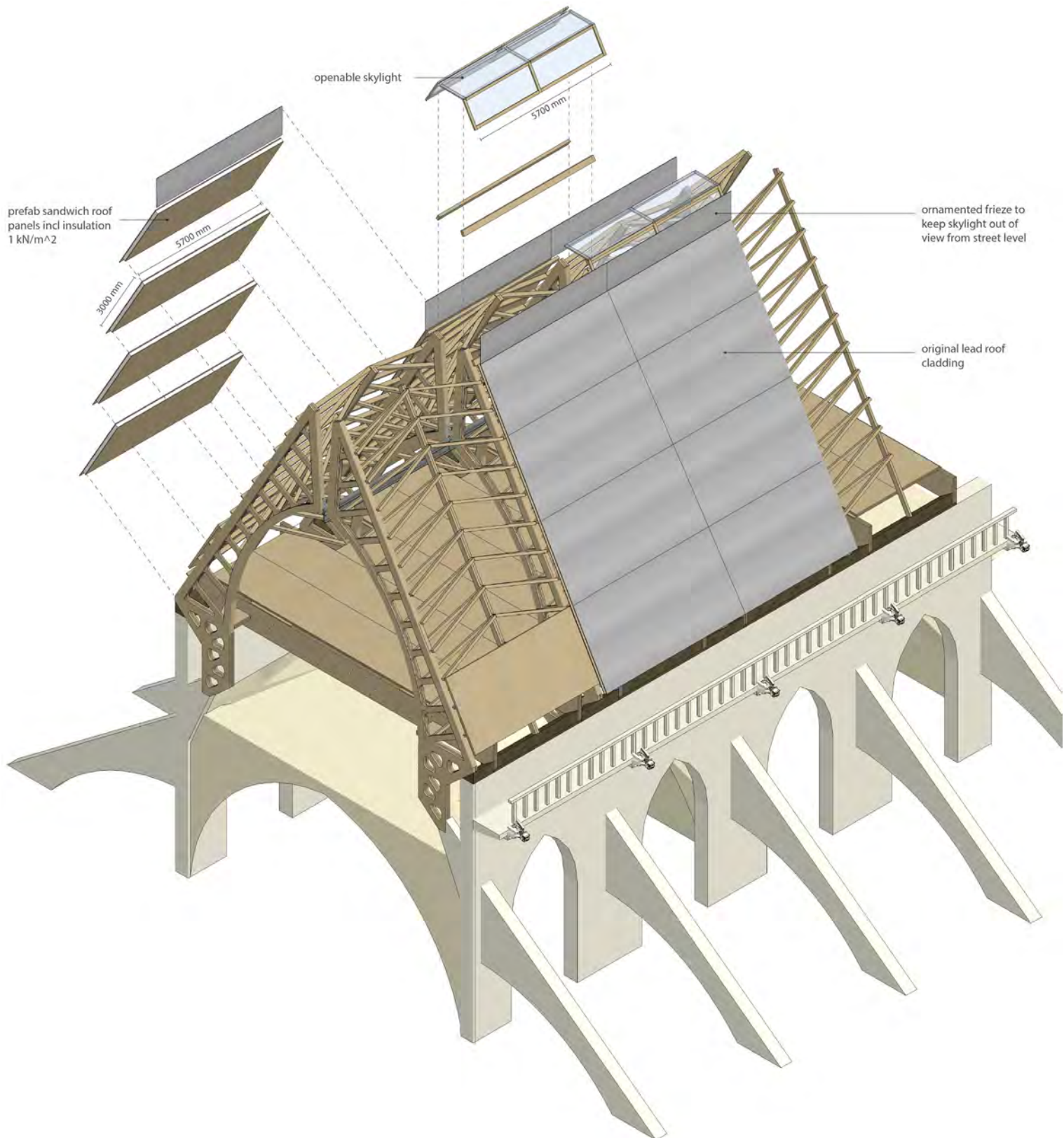


Figure 61 Options to create temporary longitudinal stability in Notre Dame's roof.

Former Finishing

The final cladding consist out of prefabricated insulating panels that are mounted on the Field Factory's timber structure. In the roof ridge an openable skylight will enable natural ventilation and some daylight to enter the attic. The following page explains the final design in more detail.



NOTRE DAME'S PUBLIC WORKSHOP

Routing

To enter Notre Dame's attic space, four ancient staircases will be used that are integrated within the stone structure of the cathedral (Figure 62). The two spiral staircases on the western side of the cathedral (Figure 62, 1) have already been used by visitors, who can buy tickets to climb on top of Notre Dame's bell towers. A wall, that is assumed to have significant fire resistance, is used to separate the bell towers from the attic space, which can be entered through three regular size doors (Figure 62, 2). After visiting the Field Factory workshops, visitors can exit the attic through the two staircases in the transept Figure 62, 3). The four staircases are strategically located to provide different escape routes. Their size though is limited, and therefore a limited amount of visitors will be allowed to enter the bell towers and attic space. In this way, evacuation can take place within an acceptable time frame.

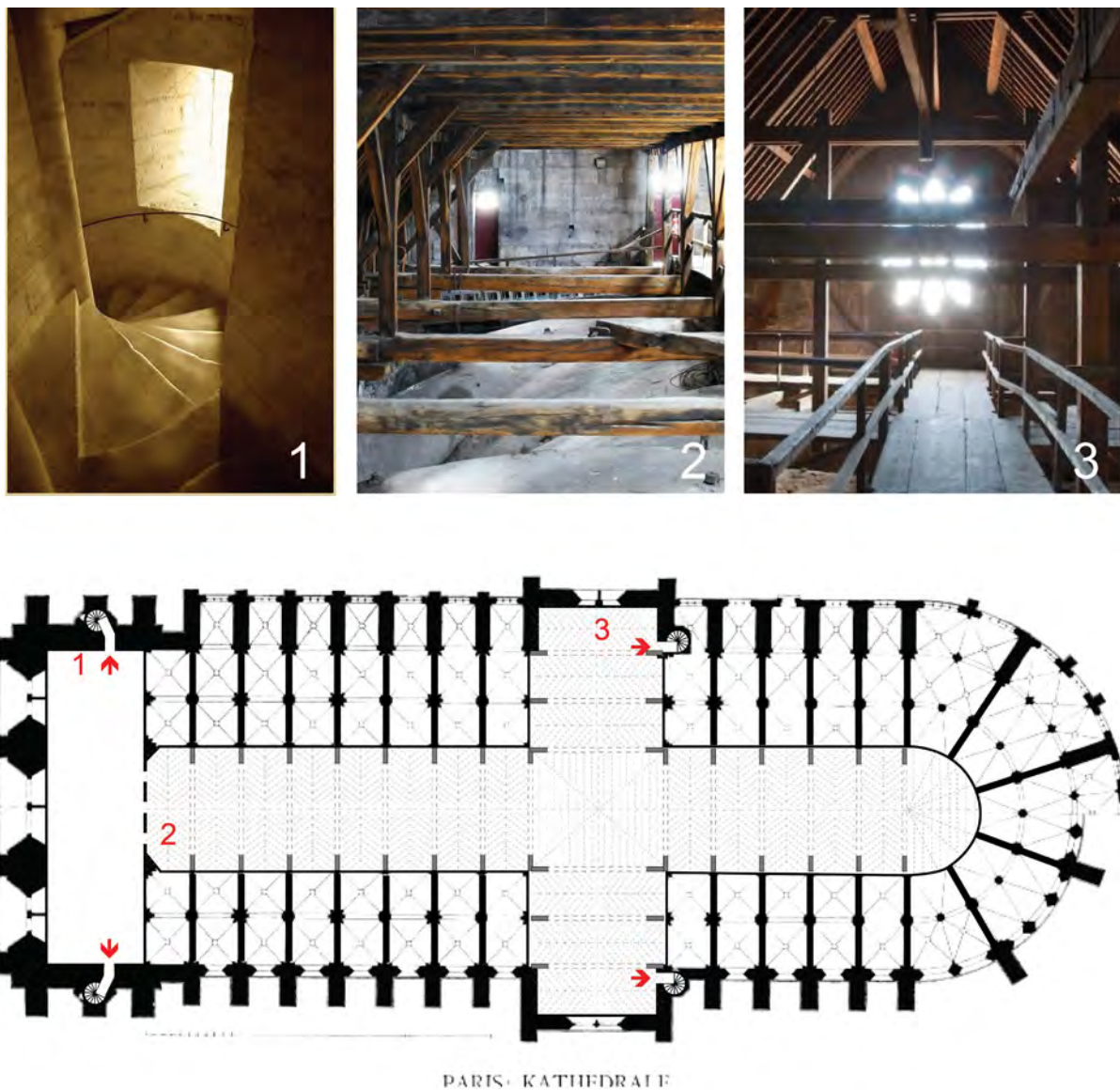


Figure 62 Floorplan of Notre Dame's attic accompanied by photo's of locations in the attic.

Indoor Climate

To create a comfortable indoor climate inside Notre Dame's attic a few guidelines are proposed. Inspired by Louis Kahn's design for the Kimbel Art Museum (Figure 63), all mechanical devices that are assumed to have a negative aesthetic effect are located in a single technical node at the tip of the roof (1.). Natural airflow is made possible by openings in the top part of the gable roof. Natural ventilation can be supported by mechanical ventilation, if necessary, which is strategically positioned to limit the effect on the aesthetics. Daylight can enter the attic space through skylights in the roof ridge, hidden from street view by the frieze (Figure 65) (2.). Supported by artificial light, hidden from sight, this should create proper light conditions for the workshops and visitors. Rainwater drainage makes use of the gargoyles positioned at the roof's edge and on the bottom of the flying buttresses, that since their initial construction already contain integrated gutters (Figure 64) (3.).



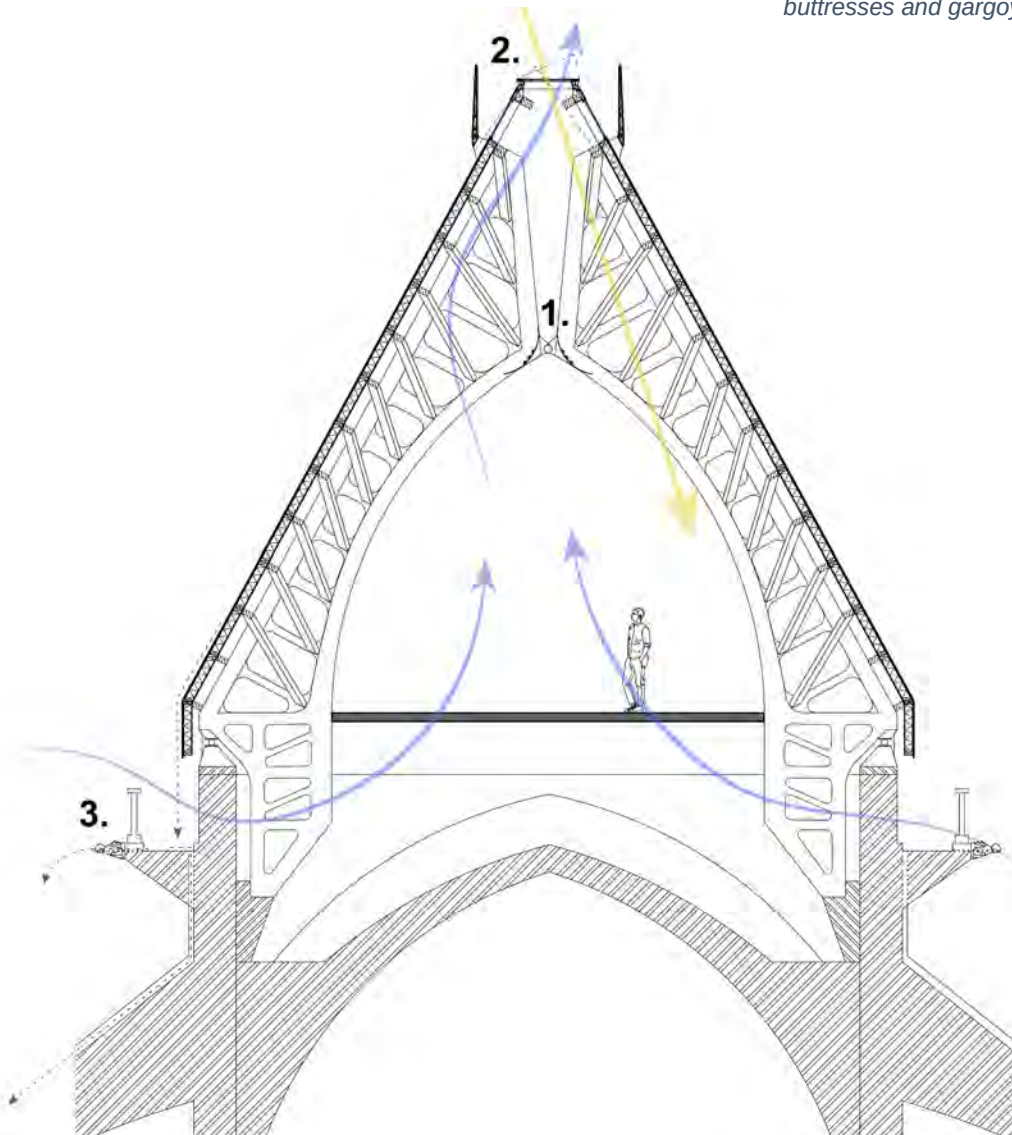
Figure 63 Louis Kahn's Kimbel art Museum.



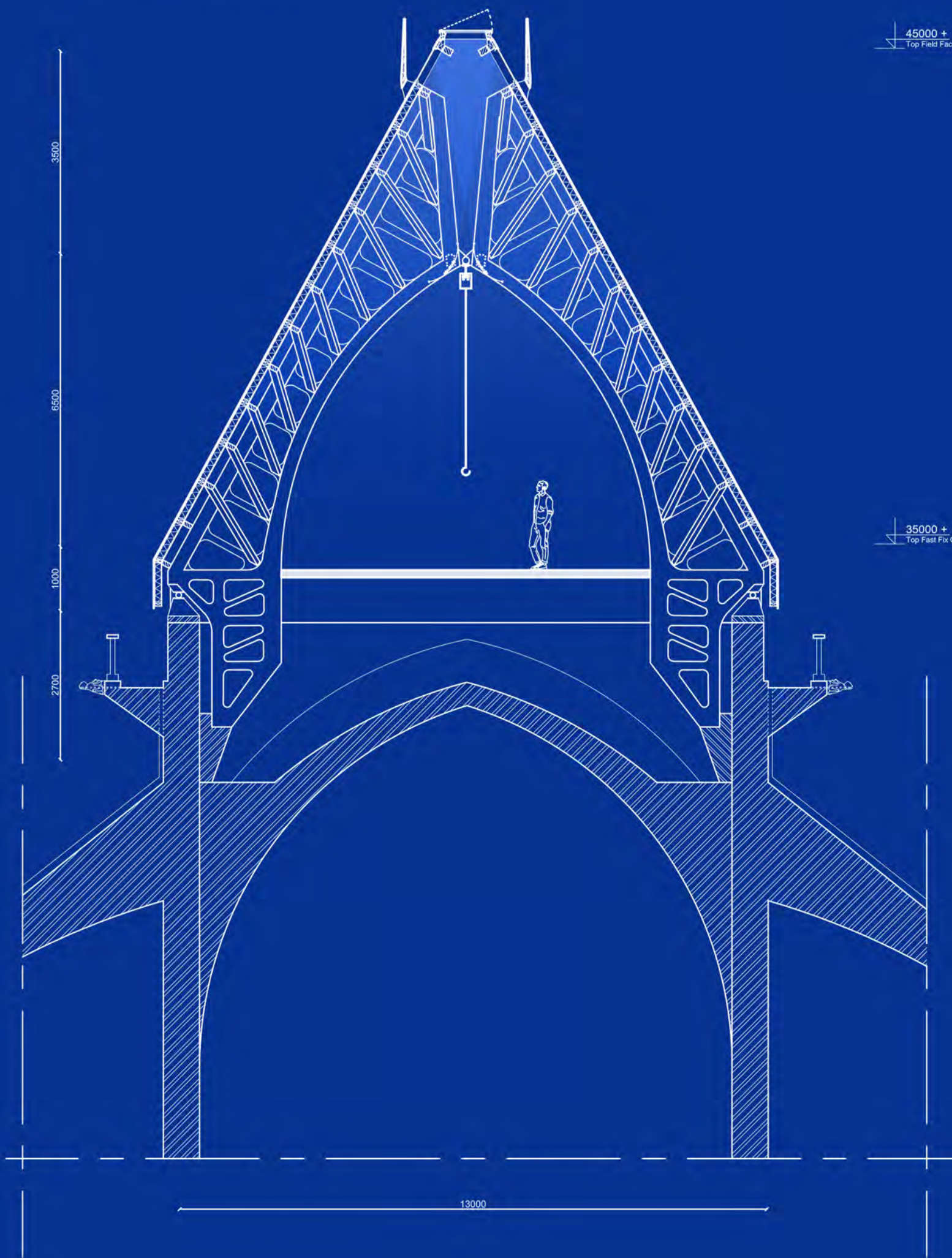
Figure 65 Roof ridge's frieze.



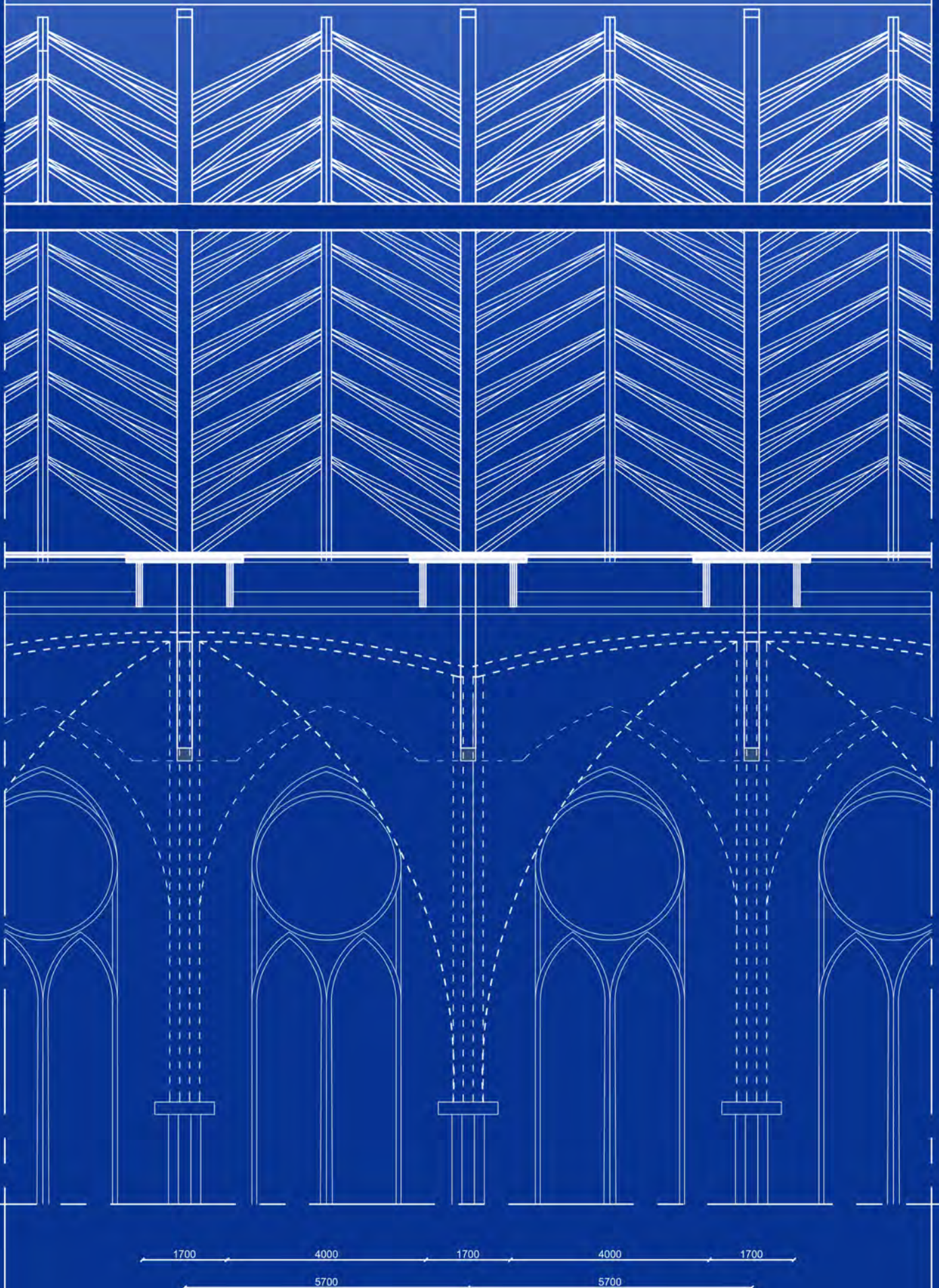
Figure 64 Integrated gutters inside flying buttresses and gargoyles at Notre Dame



Transversal section



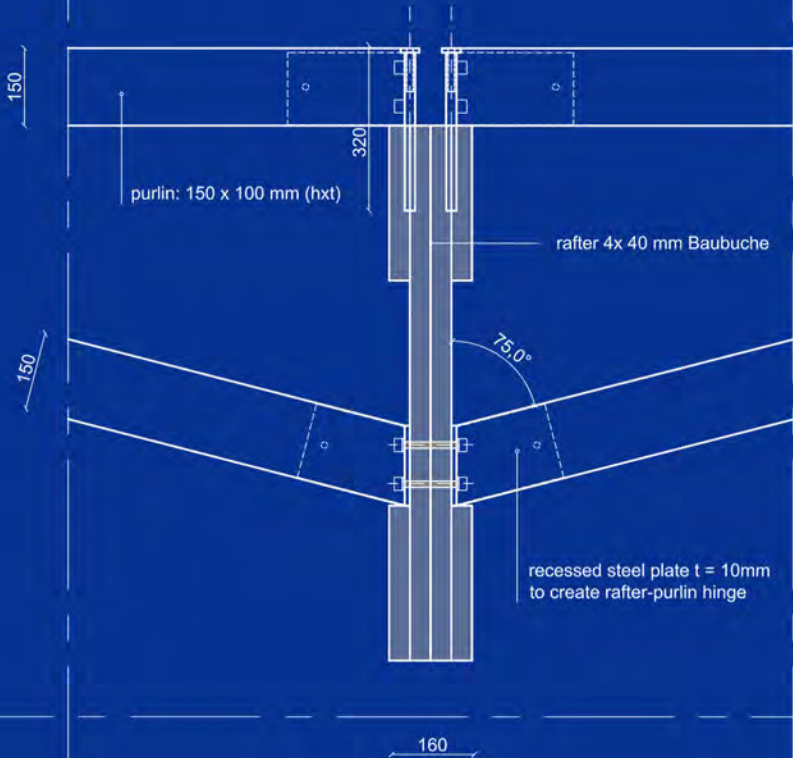
Longitudinal section



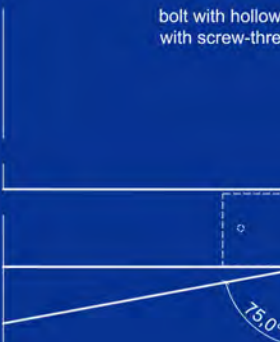
Principle Details:

section perpendicular to roof surface

rafter - purlins

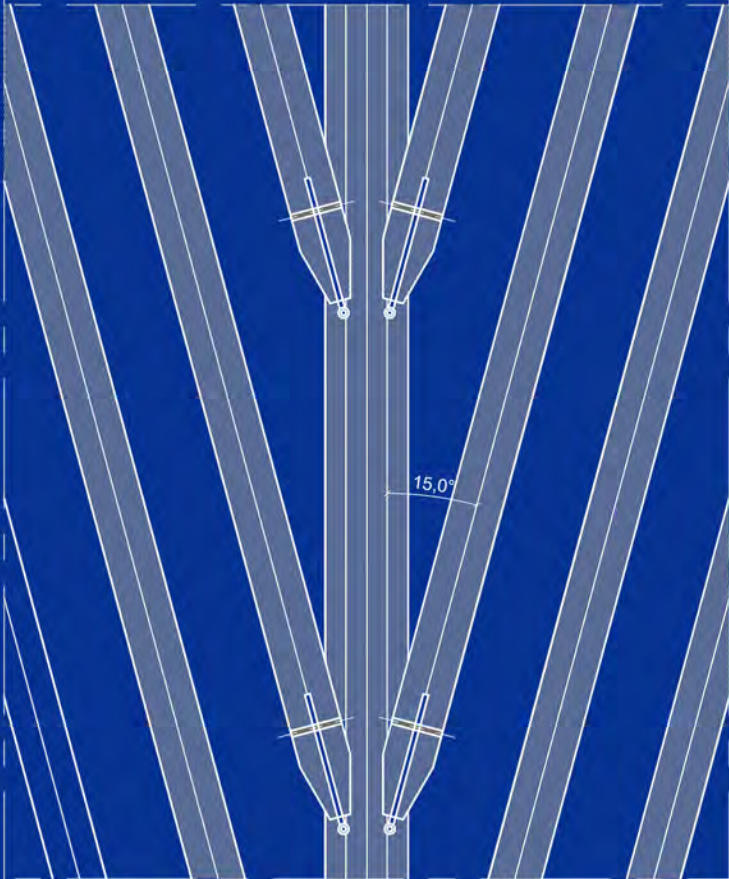


purlin - purlin



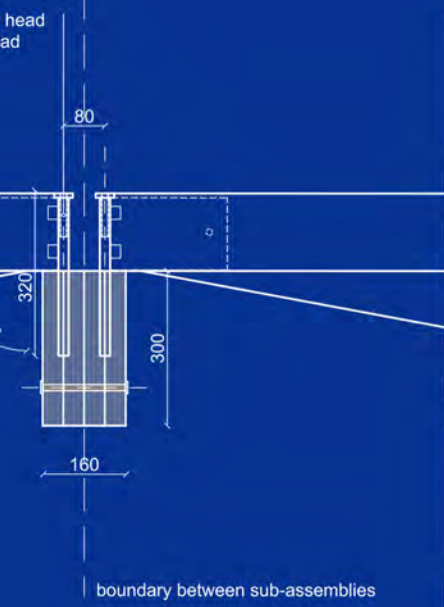
section parallel to roof surface

rafter - purlins (folded)

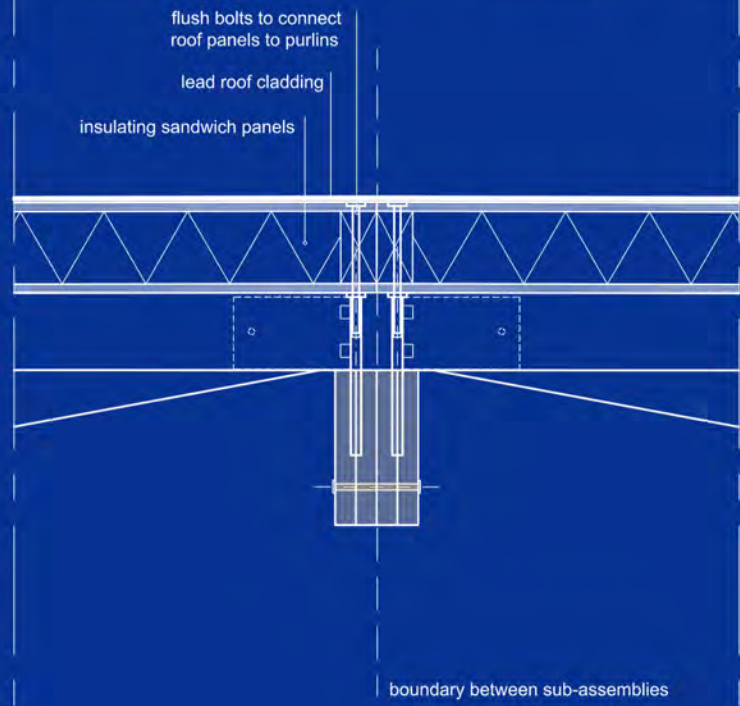


rafter - purlins

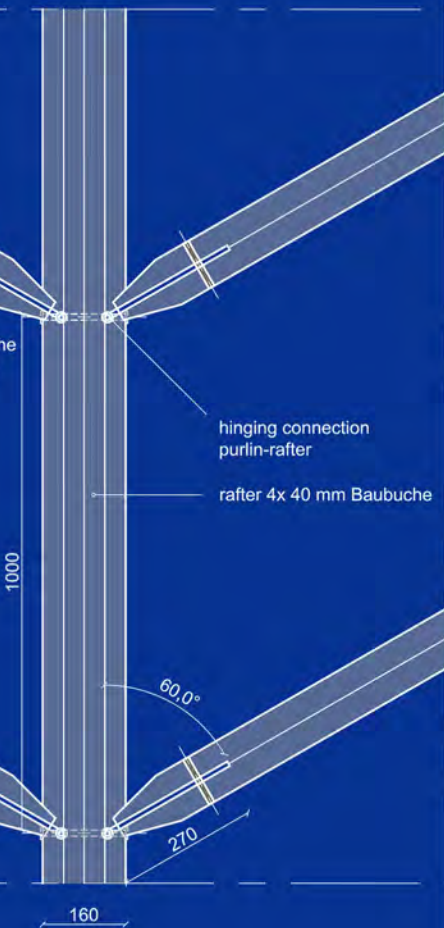




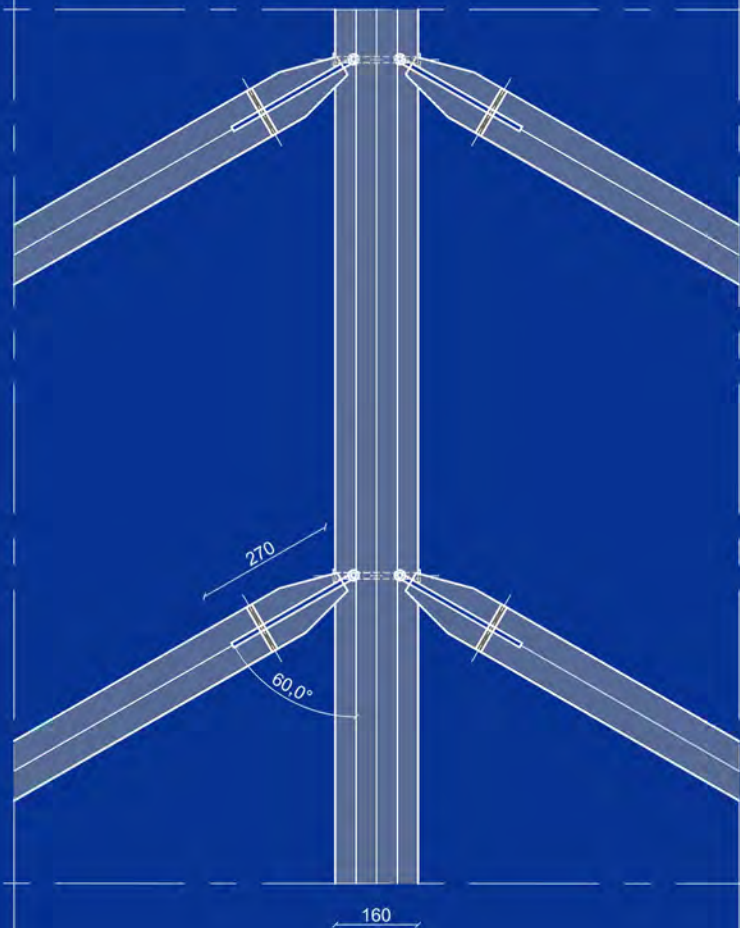
purlin - purlin with former finishing



(unfolded)



purlin - purlin (unfolded)



7. STRUCTURAL VERIFICATIONS

This chapter presents the structural verification of the design described in the previous chapter. Loads and support conditions are analyzed and followed by structural analyses of respectively the Fast Fix structure and the Field Factory structure. For the structural analyses, different computer models are utilized, from which the modeling assumptions will be discussed. In the chapter's Appendix, different verification methods are discussed. This chapter only captures the applied verification methods used for the design.

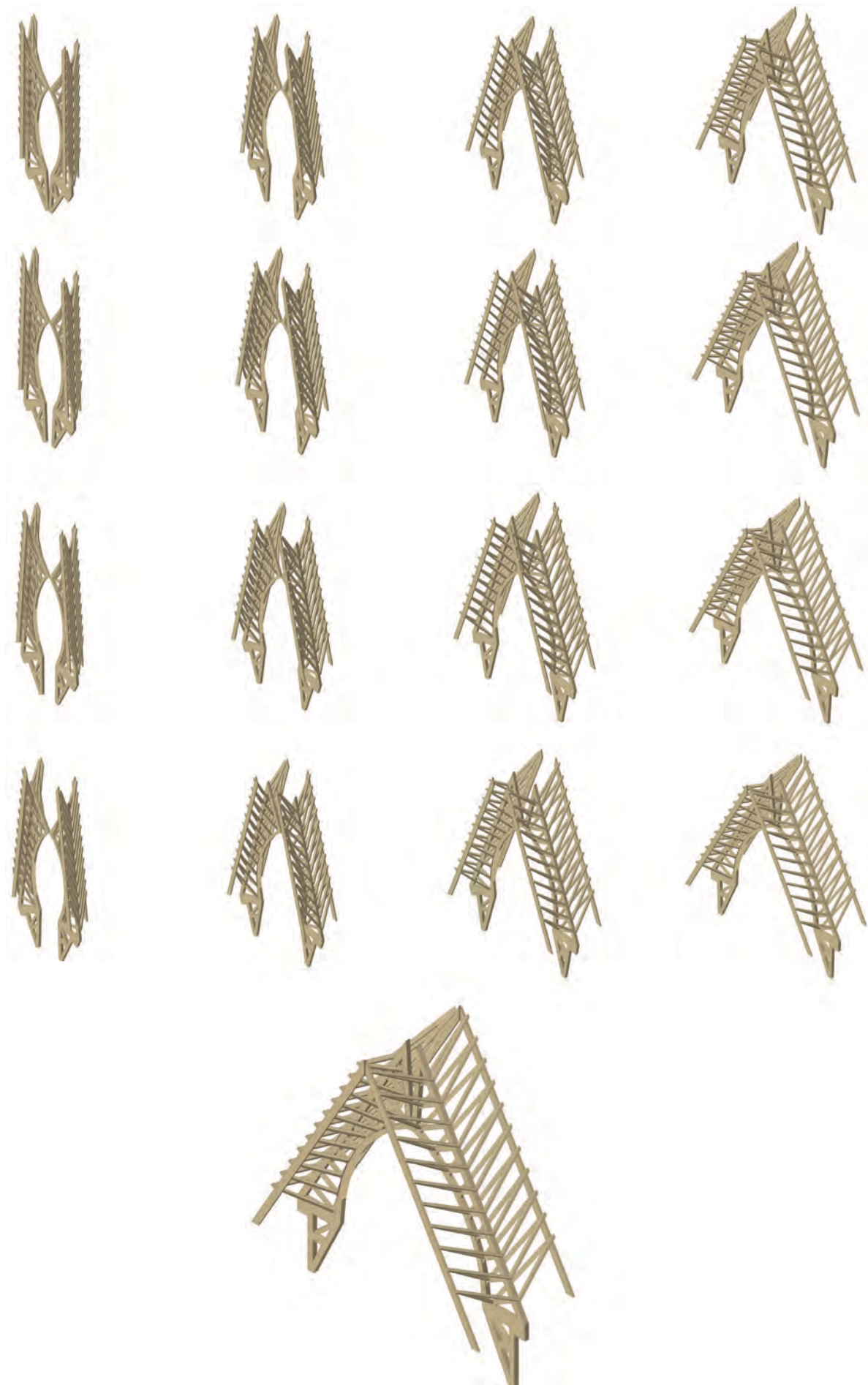


Figure 66 The expanding of a Field Factory's sub-assembly

7.1. STRUCTURAL DESIGN WITH LVL

Reliability And Durability

The importance of reliability of the structural design is depending on its application. This is translated into numbers through consequence classes. The public accessibility of Notre Dame's attic, in combination with the importance of the cathedral, makes the structure fall under consequence class 3 (**CC3**), meaning that failure of the structure is considered to have a high economic and social impact.

To guarantee the durability of a timber structure five service classes are specified which are mainly related to the moisture content in the air. The new roof structure of Notre Dame de Paris will be considered to fall under **service class 2**, which is used when timber structures are protected against weathering but are applied in buildings that are not completely enclosed. The moisture content of the timber typically tends to be below 20%. The nature of the Notre Dame de Paris to be a building that's assumed to stay present in the centre of Paris, requires special attention to be taken to protect the timber from any deterioration.

Hygroscopic And Time-Dependent Behavior

Structural properties of laminated veneer lumber have to be corrected for the material's hygroscopic and time-dependent behavior. The structural design values are therefore modified with the coefficients: k_{mod} and k_{def} . The former is used in the ultimate limit state (**ULS**) and depends on the expected moisture content of the material (**service class**), and due to timber's time-dependent behavior for load-duration (**load-duration class**). The latter is used to correct for timber's time-dependent deformation behavior, creep, when calculating long-term displacements in the serviceability limit state (**SLS**). All values for Baubuche, which can only be used in service classes 1 and 2, are presented on the adjacent page.

Material's Reliability

The reliability of a structure's material depends on variations in the material properties and is partly based upon physical experiments. This should be accounted for when designed with laminated veneer lumber, and in this case BauBuche specifically. Baubuche's natural variations, which possibly have a negative influence on the material's resistance, are corrected by applying a material partial safety factor γ_M , which can have different values depending on a transient or permanent design situation. Physical experiments have been done with Baubuche samples with a certain geometry. The characteristic strength values of BauBuche S and Q boards should therefore be corrected depending on their geometry. On the adjacent page, formulas are presented that for service class 2 will be used to modify bending (k_h) and tensile (k_t) strength. Characteristic and design values of BauBuche S and Q boards are provided in the Appendix.

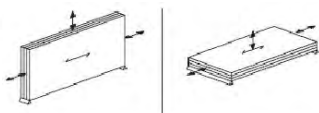
Stresses Perpendicular To The Main Grain Direction

The strength of Baubuche is heavily influenced by the grain direction of the structural member under consideration. Figure 67 displays the significant decrease of Baubuche-Q's compression tension and bending strength. Even with the presence of cross-plyes – around 15% -, it can be concluded that LVL is very sensitive for a deviating grain angle. The derivation of the correction factors for compression ($k_{\alpha,c}$), tension ($k_{\alpha,t}$), and bending ($k_{\alpha,m}$) are presented in the appendix.

Table 3 Modification factor k_{mod} and deformation factor k_{def} for BauBuche

service class	k_{mod}					k_{def}
	Class of the load duration					
	continuous	long	medium	short	very short	
1	0.60	0.70	0.80	0.90	1.10	0.60
2	0.60	0.70	0.80	0.90	1.10	0.80

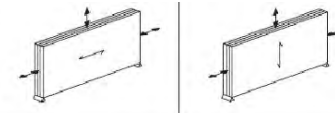
Characteristic values
for Board BauBuche S with
a nominal thickness
of 21 to 66 mm in N/mm²



Characteristic strength values in N/mm ²			
Bending	$f_{m,k}$	(64.9) – 75.0 ^{a)}	80.0
Tension	$f_{t,o,k}$	(51.7) – 60.0 ^{b)} – (66.0)	
	$f_{t,90,edge,k}$	1.5	
Compression	$f_{c,o,k}$	57.5 ^{c)} – (69.0)	
	$f_{c,90,edge,k}$	11.7 ^{c)} – (14.0)	
	$f_{c,90,flat,k}$	10.0 ^{c)} – (12.0)	
Shear	$f_{v,k}$	8.0	

Stiffness values in N/mm ²			
Modulus of elasticity	$E_{o,mean}$	16,800	
	$E_{o,05}$	14,900	
	$E_{90,mean}$	470	
	$E_{90,05}$	400	
Shear modulus	G_{mean}	760	850
	G_{05}	630	760

Characteristic values
for Board BauBuche Q
with nominal thickness
27 mm ≤ B ≤ 66 mm in N/mm²



Characteristic strength values in N/mm ²			
Bending	$f_{m,o,k}$	(51.1) – 59.0 ^{a)}	
	$f_{m,90,k}$		(7.8) – 9.0 ^{a)}
Tension	$f_{t,o,k}$	(42.3) – 49.0 ^{b)} – (53.9)	
	$f_{t,90,edge,k}$	8.0	8.0
Compression	$f_{c,o,k}$	62.0 ^{c)} – (74.4)	
	$f_{c,90,edge,k}$	22.0 ^{c)} – (26.4)	22.0 ^{c)} – (26.4)
	$f_{c,90,flat,k}$		
Shear	$f_{v,k}$	7.8	7.8
Rolling shear	$f_{vR,k}$		

Stiffness values in N/mm ²			
Modulus of elasticity	$E_{o,mean}$	12,800	
	$E_{o,05}$	11,800	
	$E_{90,edge,mean}$	2,000	2,000
	$E_{90,edge,05}$	1,800	1,800
	$E_{90,flat,mean}$		
	$E_{90,flat,05}$		
Shear modulus	G_{mean}	820	820
	G_{05}	540	540

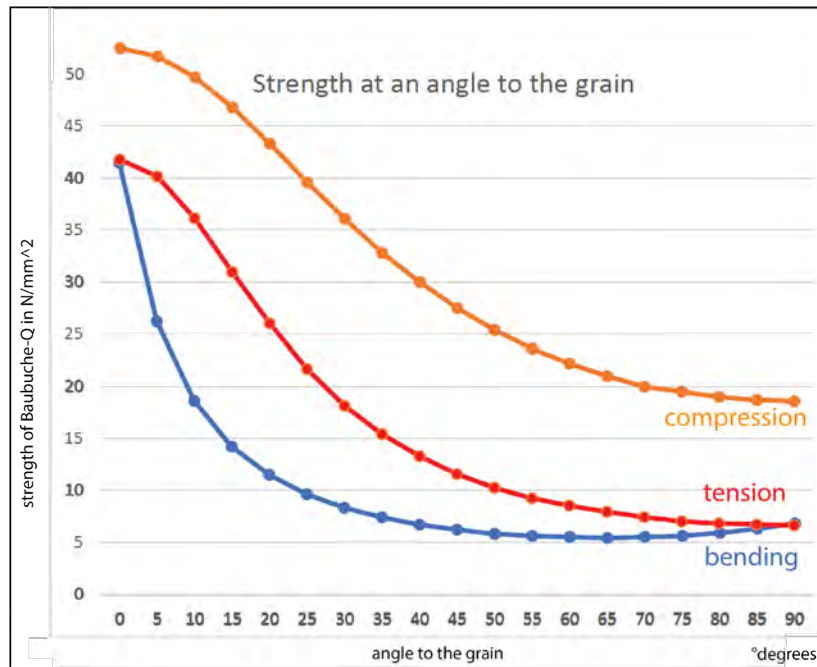


Figure 67 Reductions of Baubuche-Q's compression (orange), tension (red) and bending (blue) strength, determined by LVL suppliers Pollmeier and Stora Enso (2019).

LOADS AND NON-LINEAR LOAD-COMBINATIONS

Loads

The different loads will be discussed according to their load-duration class for the different phases of the project. Continuous loads are permanently present, like self-weight of the primary structure, secondary structure, and the architectural finishing. BauBuche's density results in a relatively high self-weight of **730 kg/m³**, the self-weight of the connections have not been considered. The cladding of the roof structure during the Fast Fix phase and the Field Factory consist out of a canvas cover from which its weight will be neglected. The self-weight of the Former Finishing roof cladding is estimated at **1.0 kN/m²** and will be considered to act on the timber gable roof structure.

Medium-term loads are typically imposed live-loads, construction workers and visitors, and dependent on the category of use. For Notre Dame attic's floor category C3 is assigned, as it is approached as a congregation area without obstacles. An imposed load of **q_k = 5.0 kN/m²** and **Q_k = 4.0 kN** is advised in the Eurocode. The final gable roof is only accessible for maintenance and therefore category H is assigned, resulting in an advised imposed load of **q_k = 0.4 kN/m²** and **Q_k = 1.0 kN**. In combination with wind or snow loads though, the maintenance loads don't have to be included in the load combinations. Since the wind loads described below are significantly higher, the maintenance loads are not included in the calculations.

Snow is a common short-term load of **0.7 kN/m²**, which will only be considered to act during the transient phase on top of the Fast Fix structure. The Field Factory's equilateral triangular-shaped roof results in a roof slope of 60° degrees, which according to the Eurocode and National annex means that snow loads don't have to be considered. The integrated crane in the FieldFactory is estimated to be able to lift a statue of a maximum of 1000 kg (point-load of **q = 10 kN**).

Instantaneous loads are limited to wind-loads which can both act horizontal, upwards (suction), and downwards (pressure). NEN-EN 1991-1-4 is used to find the loads acting on the structure. The basic wind pressure is assumed to be similar to wind region III in the Netherlands, yielding a basic wind speed of **v_b = 24.5 m/s** which in the center of Paris on 35 meters and 45 meters height results in a peak velocity pressure of respectively **q_p(35) = 0.773 kN/m²** for the Fast Fix structure and **q_p(45) = 0.847 kN/m²** for the Field Factory. The wind-load cases with internal underpressure **c_{pi} = -0.3** and overpressure **c_{pi} = +0.2** are provided in the Appendix. On the adjacent page, the governing wind load cases are displayed. For the Fast Fix structure, this yields the overpressure wind-load case (Figure 68, left) and for the Field Factory, the load case with underpressure has been governing for the design (Figure 68, right)

Load Combinations

The safety factors associated with the ULS are **γ_G = 1.35** (or $\zeta\gamma_{Gj,sup} = 0.85 \times 1.35 = 1.15$) for permanent unfavorable loading, **γ_G = 0.9** for permanent favorable loads, **γ_G = 1.5** for all variable loads. For the SLS load combinations, all safety factors are **γ = 1.0**.

Determining the governing load-combinations acting on Notre Dame's new roof structure is a relatively extensive task. As already discussed is the load-duration key for a timber structure's resistance. Complexity is further increased by different load factors to be applied to the leading- (largest) and non-leading variable loads. As will be further discussed below, the compression-only support conditions as a result of Notre Dame's limestone structure, required a non-linear structural analysis during which supports can become inactive depending on the load combination. The non-linear load combinations have been implemented manually and can be found in the engineering report in the appendix.

FLAT ROOFS $\alpha < 5^\circ$

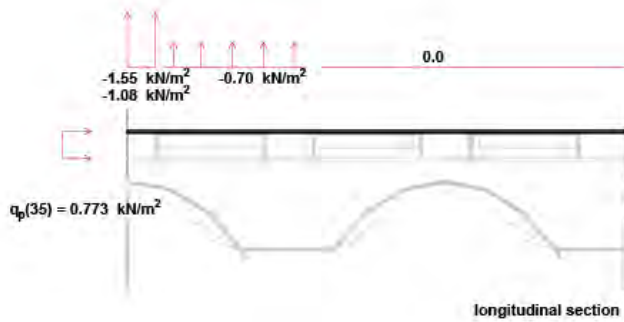
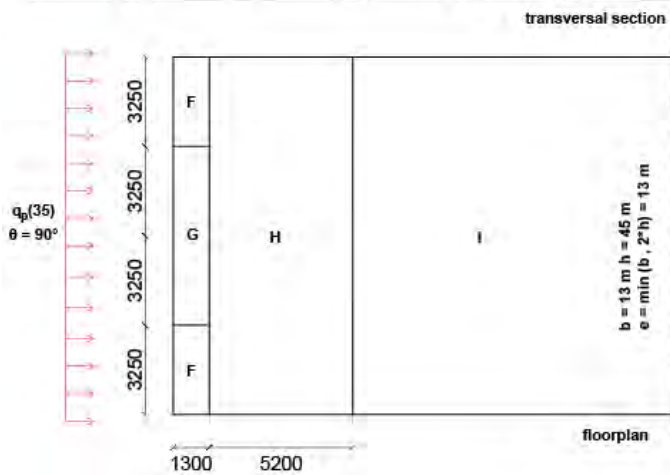
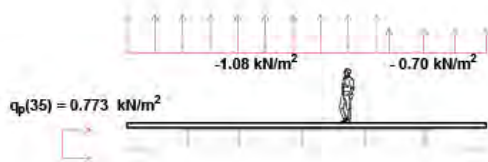
Load case with overpressure $c_{pi} = +0.2$

Zone F: $c_{f,pe} (-1.8) - c_{pi} (+0.2) = -2.0$

Zone G: $c_{g,pe} (-1.2) - c_{pi} (+0.2) = -1.4$

Zone H: $c_{h,pe} (-0.7) - c_{pi} (+0.2) = -0.9$

Zone I: $c_{i,pe} (+0.2) - c_{pi} (+0.2) = 0.0$



GABLE ROOFS $\alpha = 60^\circ$

Load case with underpressure $c_{pi} = -0.3$

Zone F: $c_{f,pe} (+0.7) - c_{pi} (-0.3) = 1.0$

Zone G: $c_{g,pe} (+0.7) - c_{pi} (-0.3) = 1.0$

Zone H: $c_{h,pe} (+0.7) - c_{pi} (-0.3) = 1.0$

Zone I: $c_{i,pe} (-0.2) - c_{pi} (-0.3) = 0.1$

Zone j: $c_{j,pe} (-0.3) - c_{pi} (-0.3) = 0.0$

$q_p(45) = 0.847 \text{ kN/m}^2$

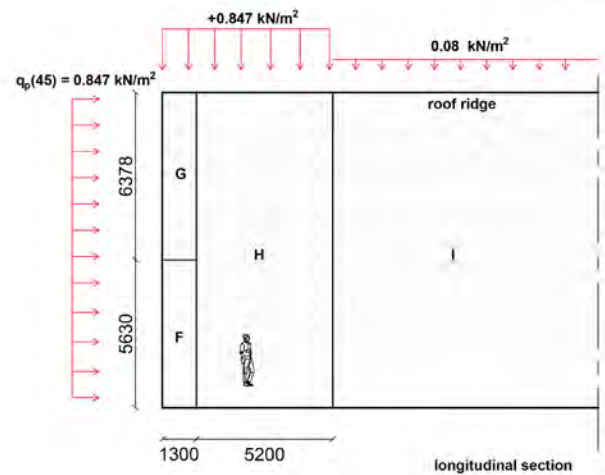
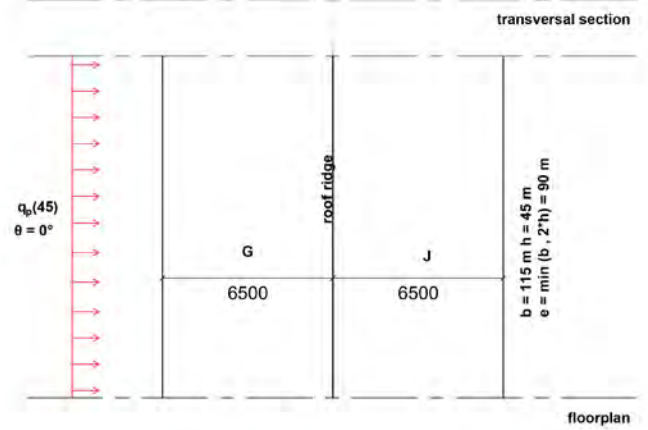
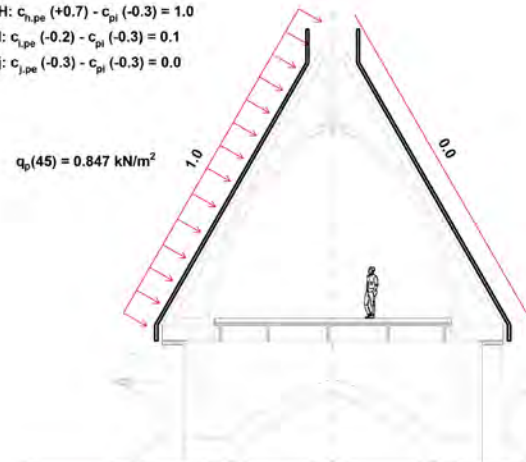


Figure 68 Governing wind loads acting on the Fast Fix cassettes (left) and Field Factory's gable roof structure (right).

7.3. STRUCTURAL ANALYSIS GUTTERING WALL

Geometry And Materiality

The new roof of Notre Dame de Paris will be supported by the stone skeleton of the cathedral. It is assumed that the stability of the guttering walls, the slender top part of the cathedral, will be critical for the roof's support conditions. The guttering walls are 600 millimeters thick (Vannucci, 2020), and with a freestanding height of 2.7 meters - as a result of the 13th-century modifications -. Different reasons caused the structural properties of Notre Dame's stone skeleton to be far from uniform. The stone skeleton of Notre Dame de Paris was built using limestone from quarries from the region of Paris, better known as Lutetian limestone, which was extracted from different quarries and each quarry contained limestone from different qualities. Additionally, modification to the cathedral since its initial construction increased the inhomogeneity of the masonry and the mortar.

The Lutetian limestone used for the Notre Dame is assumed to have a density of 2400 kg/m³ and compressive strength of $\sigma_c = 40 \text{ N/mm}^2$ (Vannucci et al., 2019b), far greater than the maximum compressive stresses occurring in the structure before the fire, which are of the order 4 – 6 N/mm². The new roof structure is expected to lead to maximum stress concentrations of 2 N/mm² and therefore no further strength verification for the guttering walls will be presented.

Decarbonization

The structural performance of the guttering walls is analyzed, taking into account the possible deterioration of the material after the fire. When limestone gets subjected to temperatures above 900 degrees Celsius, the material starts to decarbonate, resulting in a powdery substance and total loss of resistance. The temperatures reached during the fire at Notre Dame, have been traced back using timber debris (Deldicque and Rouzaud, 2020). In Figure 69 the maximum temperatures are projected on the cathedral's floorplan, which shows that temperatures easily crossed the 900 degrees Celsius line. Consequently, large cracks could have occurred between the healthy and affected limestone.

Generally speaking, the depth of decarbonization due to a fire is seldom more than 20 mm, but exceptions can be found in the literature. During the fire at the York Minster in 1984 for example, structural properties of the limestone structure were altered up to 50 mm depth (Chakrabarti et al., 1996). For Notre Dame de Paris this worst-case scenario is assumed and the structural resistance of the guttering wall should therefore be corrected. This is done by reducing the effective width of the wall from $t_{ini} = 600 \text{ mm}$ to a corrected width of $t_{cor} = 500 \text{ mm}$.

Stability Of Guttering Walls

Hand calculations are presented to check the stability of a free-standing guttering wall and a Fast Fix cassette positioned on top (Figure 70). The overturning moment at the base of the guttering walls is considered to be governing for the stability of the limestone walls. Conservative assumptions are taken by schematizing the cassettes in such a way that its presence only contributes to the wind-induced overturning moment, while its self-weight is neglected. The self-weight of the limestone masonry results in the resisting moment of $(31,8\text{kN} * 0.25\text{m}) = 7.95\text{kNm}$, which is sufficient to counteract an overturning moment of $(2.7\text{kN} * 1.85\text{m}) = 5.01 \text{ kNm}$. No problems are therefore expected with the reaction forces of the Fast Fix structure.

Through a similar check, it can be proven that the guttering walls can't provide horizontal support to the Field Factory structure. If a pinned support would be positioned on the top of the wall, a horizontal reaction force due to the wind load (LC.3) of around 34 kN can be expected, which will lead to a total overturning moment of $(34\text{kN} * 3.7\text{m} + 5 \text{ kNm}) = 130 \text{ kNm}$. Even the maximum vertical reaction force of the Field Factory structure of 180 kN - resulting in a resisting moment of **52.95 kNm** - can not prevent the guttering wall from overturning. As expected, horizontal roll support needs to be positioned on top of the guttering wall, and the horizontal reaction forces have to be guided down where the flying buttresses and vaults can provide lateral support.

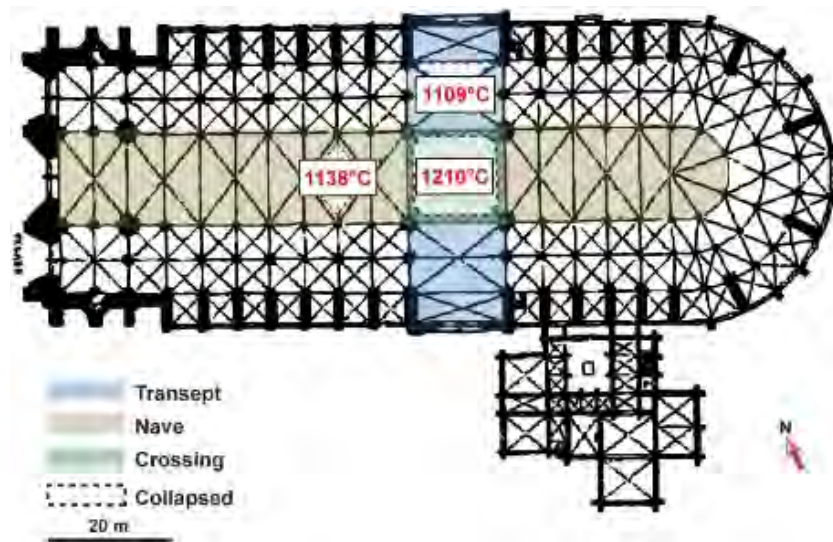


Figure 69 Temperatures reached during fire at Notre Dame de Paris at April 15th, 2019 (Deldicque and Rouzaud, 2020)

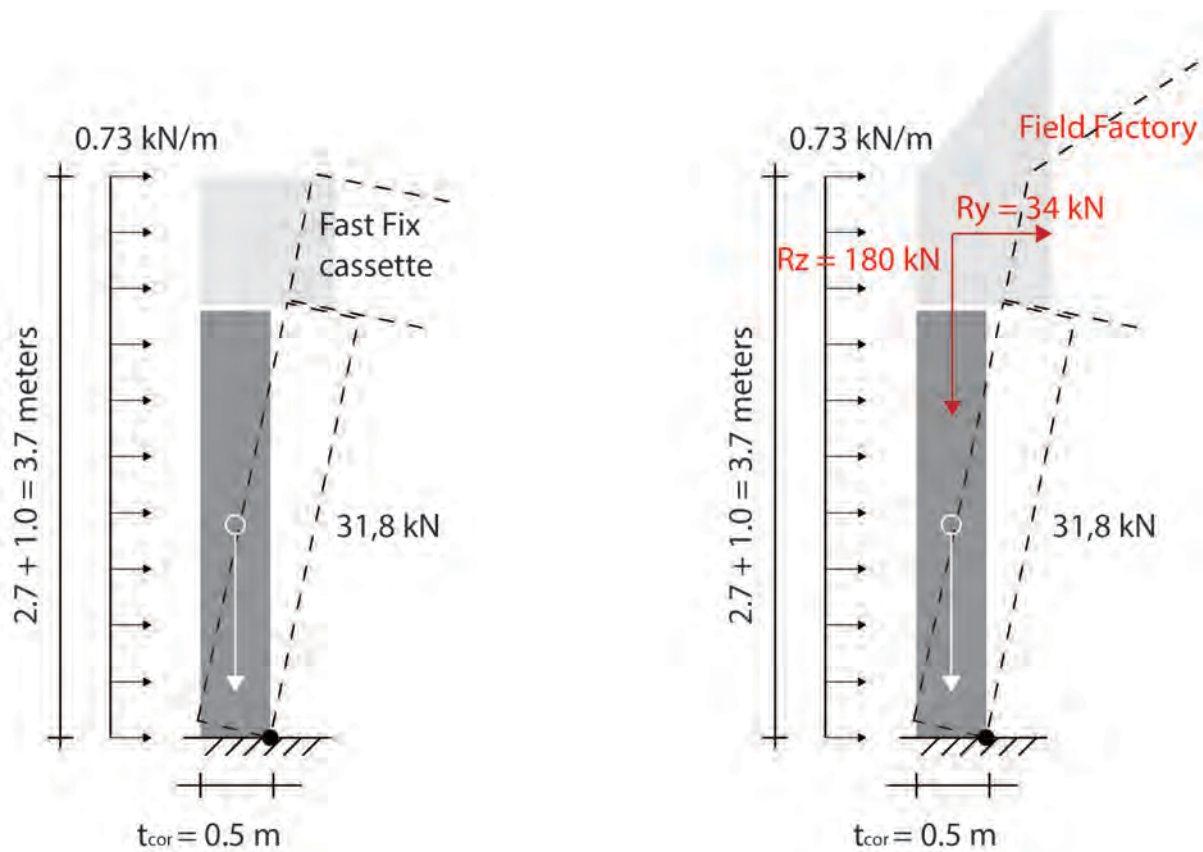


Figure 70 Calculation scheme overturning moment guttering wall for the Fast Fix cassettes (left) and the Field Factory structure (right), in case horizontal displacements would be prevented.

7.4. CASSETTES AS FAST FIX AND FLOOR STRUCTURE FOR ATTIC

The modeling of the Field Factory in SCIA Engineer is done with a few simplifications and assumptions. The connections of the cassettes and the limestone guttering walls of Notre Dame are modeled such that the primary beams are simply supported. The primary beams ($t \times h = 160 \text{ mm} \times 1000 \text{ mm}$) consisting out of 4 x 40 mm thick Baubuche S boards bolted together, are modeled as solid LVL beams. Secondary beams are positioned at the top of the primary beams, preventing stability problems like lateral-torsional buckling and tension stresses perpendicular to the grain. In-plane stability is guaranteed by two diagonal members. The horizontal floor panels are not modeled, although their self-weight is included through a permanent load case. The floor panel's additional stiffness to the cassette structure is not taken into account and it is expected that displacements in practice will be slightly smaller than the model displays.

The Fast Fix cassettes are over-dimensioned considering the forces acting on the structure during the stabilizing phase of the reconstruction. The load combination of high wind speeds and without any live loads result in an uplifting support force of $R_z = +17.73 \text{ kN}$ at the windward side of the cathedral. Therefore extra attention has to be given to the design of the connections between the cassettes and the masonry of the guttering walls. As a solution, the temporary clamping joints will exert their force on the vertical position of the masonry where the self-weight of the guttering wall will counteract the vertical support force, which is estimated at a height of at least one meter from the top of the guttering wall.

Since the cassettes will also function as the floor structure of Notre Dame's attic, there will be a considerably higher live load acting on the structure. At the same time, the uplifting wind force and possible snow load are no longer present. As shown in the Appendix of this chapter, no problems are expected with the strength of the LVL cassettes and the reaction forces exerted on the guttering wall. The displacements, including the long-term creep effects, will lead to considerable displacements. The total final deformation will lead to a displacement of $u_z = -40$ mm, and the associated unity check of $UC = 0.94$ (Figure 71). Precamber is not considered as its application would increase the production time.

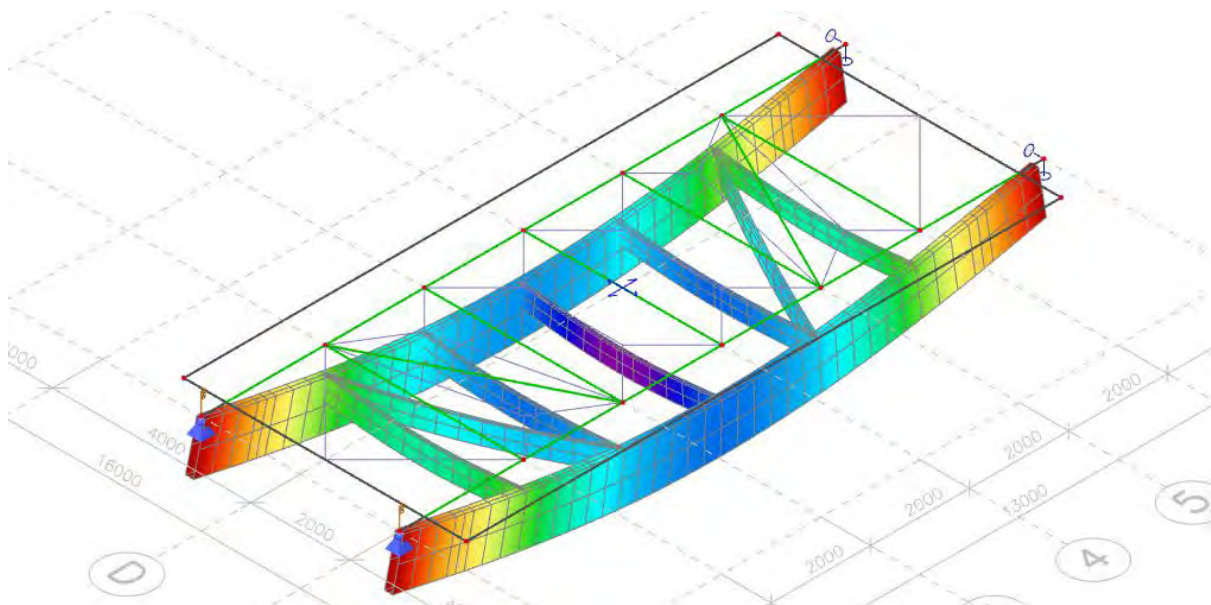


Figure 71 Deflection of the cassettes when exposed to the ULS load combination of Notre Dame's attic. The governing displacement is expected at the middle of the primary beams as a result of creep effects.

7.5. FIELD FACTORY

For the relatively complex LVL structure, two models have been developed to limit the calculation time. With the proper boundary conditions, the repetitiveness and symmetry of the structure are exploited to arrive at the models shown in (Figure 72). The longitudinal model contains a repetition of 5 bays, from which two bays contain a roof panel to provide longitudinal stability. For the transversal model, a single bay was sufficient to determine the forces and deflections acting in the structure.

The different parts of the model will be discussed separately, starting with the longitudinal model which appeared only important to investigate the stability of the structure. Next, the transversal model is discussed, which proved to be governing the deflections and member forces.

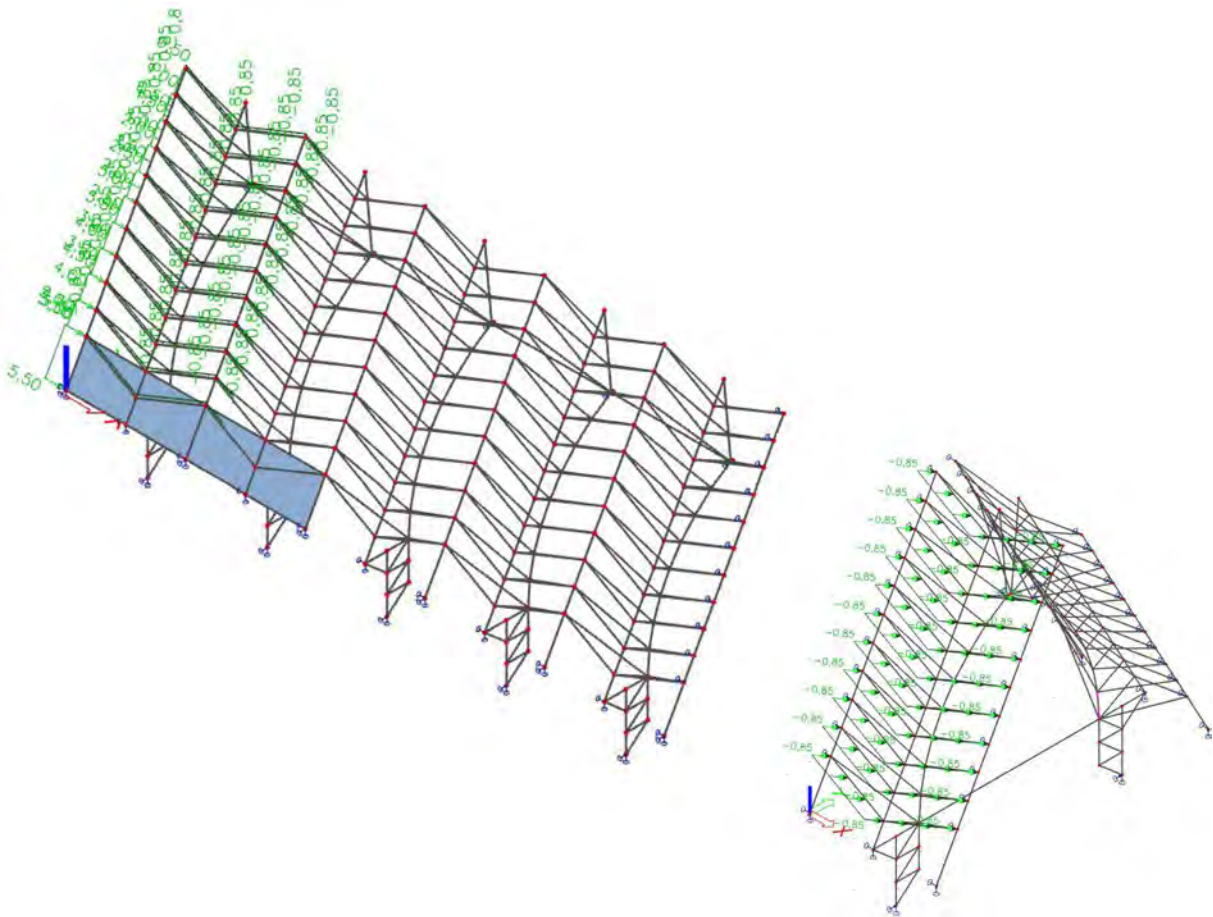


Figure 72 Scia Engineer models. Left: longitudinal model from which two bays contain panels to guarantee stability. Right: transversal model from which the tie-beam is modeled as two primary beams from the cassettes.

Longitudinal Model

The longitudinal computer model represents the moment during which the transversal section is completely exposed to the wind. The resulting deflections and members forces are not governing for the design and will be discussed below for the transversal computer model. As stated in the previous chapter, temporary longitudinal stability is created by placing panels at the base of the rafters. Additionally, the computer model reveals that the Field Factory's crane - the horizontal members colored red - helps to prevent the rafters from rotating around their own axis (R_x0° , R_y60° , R_z0°). In reality, the stability in the longitudinal direction will be further improved by connecting the timber structure to the stone structure of Notre Dame and due to the cross-shaped floor plan of the cathedral.

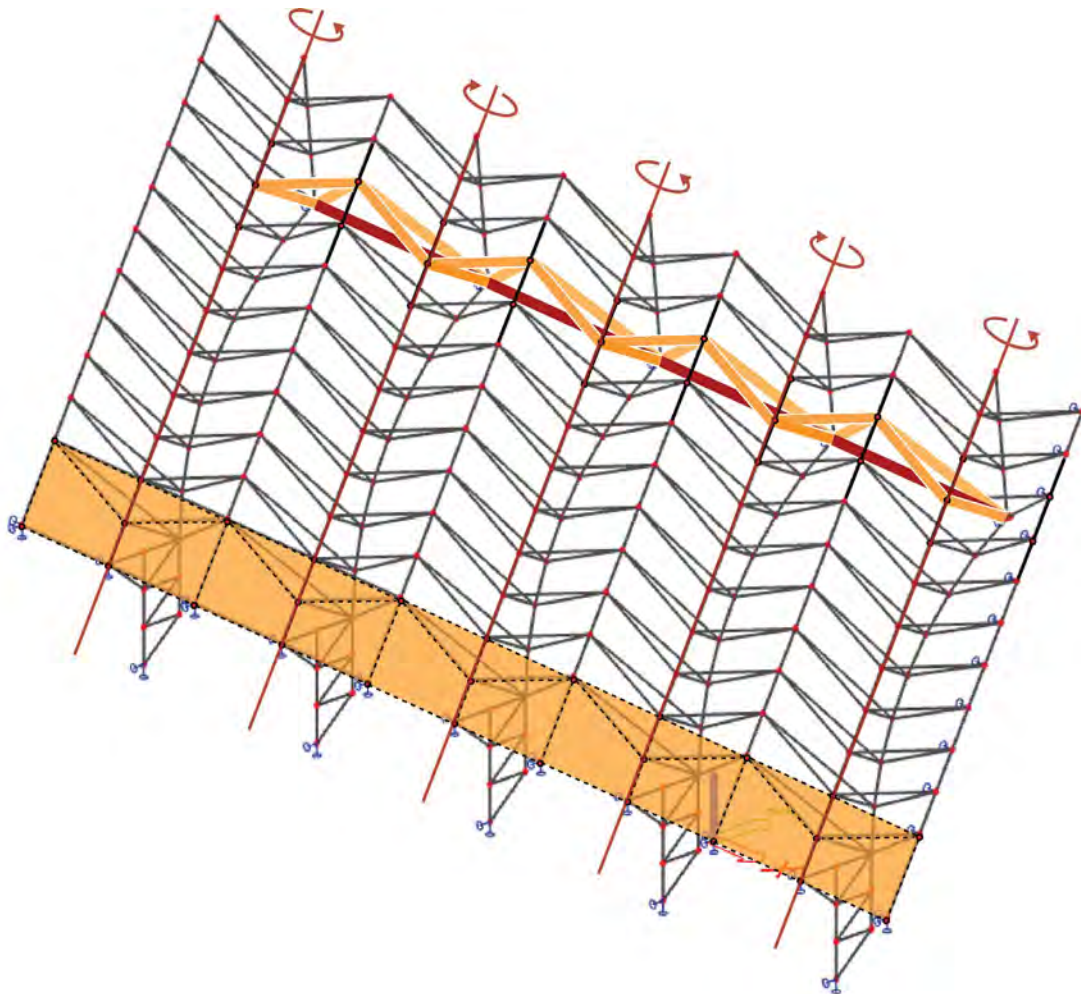


Figure 73 Longitudinal model displaying the panels at the base of the rafters for lateral stability. The integrated crane (red) creating triangles at the roof's ridge that prevent the rafters from rotating around their own axes.

Purlins In Transversal Model

The connections between the rafters and purlins are both in the engineering model and in practice perfectly hinging to allow expandability of the structure. The top parts of the purlins are governed by the deflections as a result of self-weight and wind forces in the transversal direction. Since the angle of Notre Dame's roof has a relatively steep angle of 60° degrees the purlins need to resist both deflections parallel en perpendicular to the roof's plane. The resulting, almost rectangular, cross-sections are (t x h) 100 mm x 150 mm.

Modelling Of Rafters

The modeling and design of the rafters appeared to be a challenge, governing the deflections and sensitivity for local stresses as a result of the rather uncommon geometry and production process. CNC'ed out of Baubuche Q boards, strength and stiffness performance depends on the load angle (Figure 74). The Baubuche handbooks provide strength and stiffness for the application parallel and perpendicular to the main grain direction. Figure 74 shows in orange the members which are assumed to be parallel, and red which are assumed perpendicular to the grain direction. This assumption becomes questionable where the bottom part of the rafter slowly curves away until it reaches an angle up to 35° . For these members, extra verifications are presented in the Engineering Report, section 6.4, which takes the stresses at an angle to the grain into account. The part below the floor marked light orange in Figure 74, will not be further analyzed, although the reaction forces acting on the stone structure will be assessed.

The support conditions of the rafters have been extensively investigated. Since the floor cassettes are governed by long-term deflections they have the redundant load-carrying capacity to function as a tie-beam for the Field Factory's roof. The tie-beam is modeled as a hinging member, being able to transfer an axial force without resisting rotation of the rafters. While the concept of different load paths for horizontal (K3 and K4) and vertical forces (K1 and K2) was proposed above, it was found that the geometry of the rafters results in an important load path in the bottom members. It was therefore decided to let all supports be able to carry vertical forces. The resulting support condition consists out of two horizontal roll supports (K1 and K2) and two pinned supports (K3 and K4).

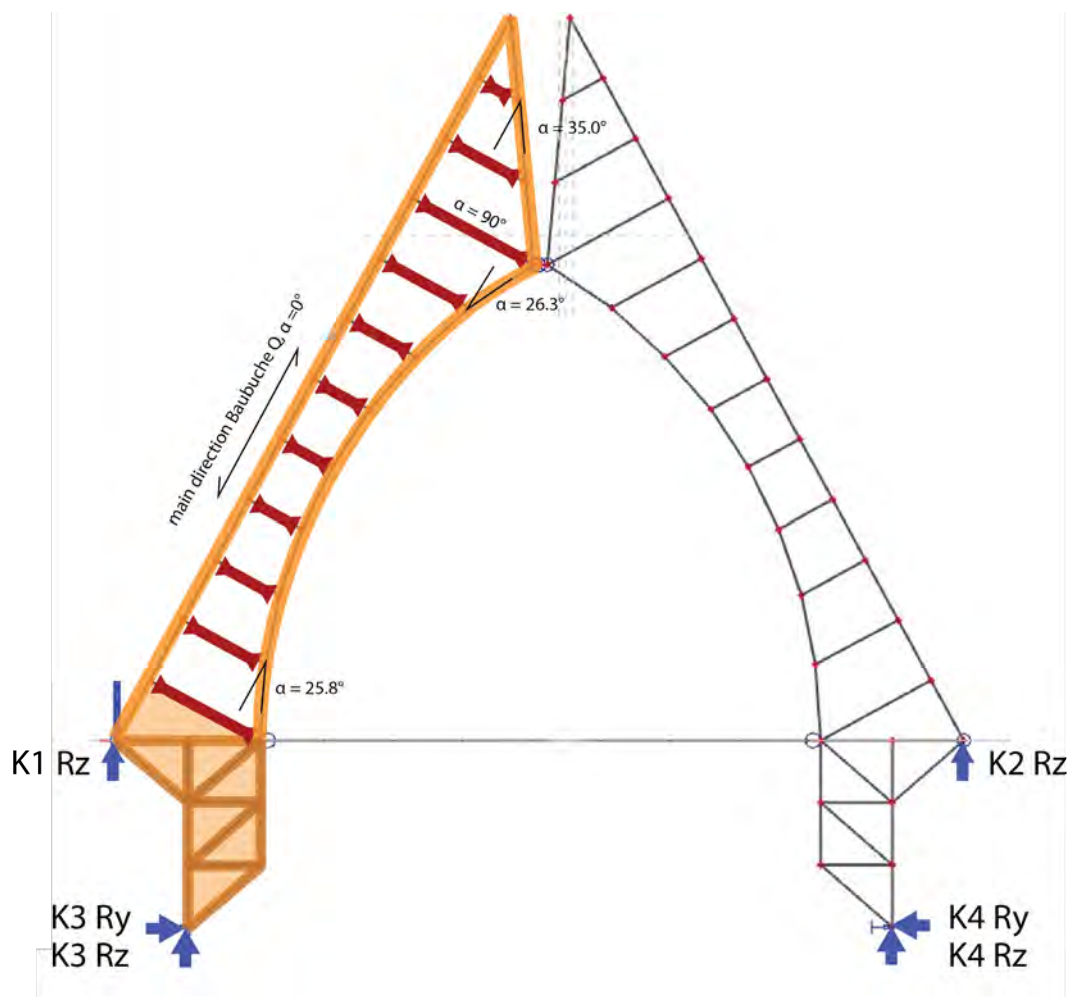


Figure 74 Transversal section of the computer model of Field Factory structure with different member angles.

The Rafters' Support Conditions

The reaction forces as a result of the different load cases are qualitatively displayed in Figure 75 and quantitatively presented in the Appendix. In the non-linear analysis, it was found that the wind load only (LC.3) doesn't make external equilibrium as only two supports (K2 Rz and K4 Ry) are subjected to compression. The wind load is therefore modeled together with the self-weight of the rafters (LC.3 + LC.1) resulting in the support condition displayed in Figure 75 (left-bottom). The wind load (LC. 3) turns the structure into an external statically determined structure in which all horizontal support is provided by a compressive resisting force through the flying buttress at the leeward side of the roof.

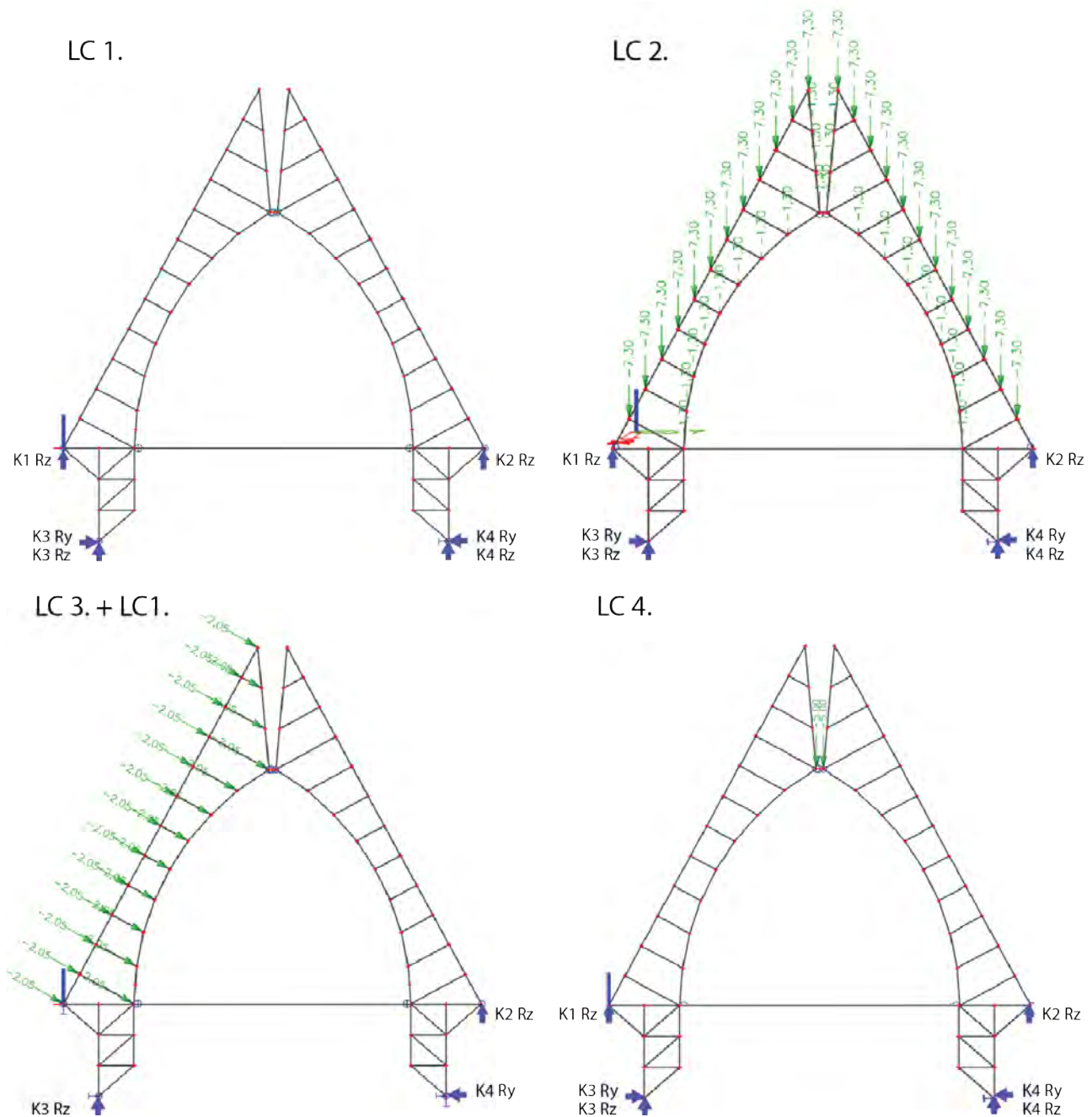


Figure 75 The qualitative support conditions depending on the load case. With blue arrows, the active (compression-only) supports are displayed.

Deflection Of Rafters

The deflection analysis appeared not to be the governing in the structural verification. The deflections therefore relatively easily fulfill the maximum allowable deformation requirements. For the deflection analysis, a few considerations have to be mentioned. First, the connections between the members are modeled completely rigidly, but it is conservatively assumed that the joints will be subjected to a certain amount of rotation. For Vierendeel beams generally, a stiffness reduction of 15% is advised. In the case of Notre Dame's rafters, this stiffness reduction is implemented by multiplying the deformations with a factor $\times 1.2$. Second, long-term deformation behavior has been calculated manually (Engineering Report, section 4.3). Linear elastic deflections were taken from the Scia Engineer model, after which creep effects got included in the verifications. They didn't appear critical as the governing wind load is instantaneous not causing any long-term deflections.

The deflection values of the different load combinations can be found in this chapter's appendix. The short-term and long-term deflections due to the self-weight of the rafter (LC.1), the cladding (LC.2), and the crane (LC.4) are very minimal. The wind however causes large deflections in both the windward and leeward rafter. Figure 76 shows the governing initial elastic deflections of the leeward side's rafter, being subjected to SLS 1.2: self-weight of the rafter (LC.1) and the wind load (LC.3). The rafter's deflections are corrected for the global translation of the structure and are projected perpendicular to the roof surface. When the stiffness reduction of a Vierendeel beam is included, the deflections increase up to $U_{\max} = (16.3 \text{ mm} \times 1.2) 19.6 \text{ mm}$, which over a span of 9 meters – marked with a dotted line – leads to a unity check of **UC = 0.65**.

Two things can be observed from the deflection analyses on the rafters. Load cases 1, 2, and 4 don't cause large deformations in the structure. The position of the hinge in the roof's ridge and the particular design of the rafters cause the tip of the roof to move inwards, consequently pushing outwards the middle of the rafters. While this has a favorable effect on the deflections in the windward rafter, it reinforces the deflections on the leeward side. Secondly, one can observe that the deflections are almost completely the result of the wind force acting on the structure. Interestingly, the SLS load combination assessed and displayed in Figure 76 (right) contains only wind loads acting on the left roof surface. A large portion of the deflection in the leeward rafter can therefore only be the result of a reaction force that's imposed by the left rafter.

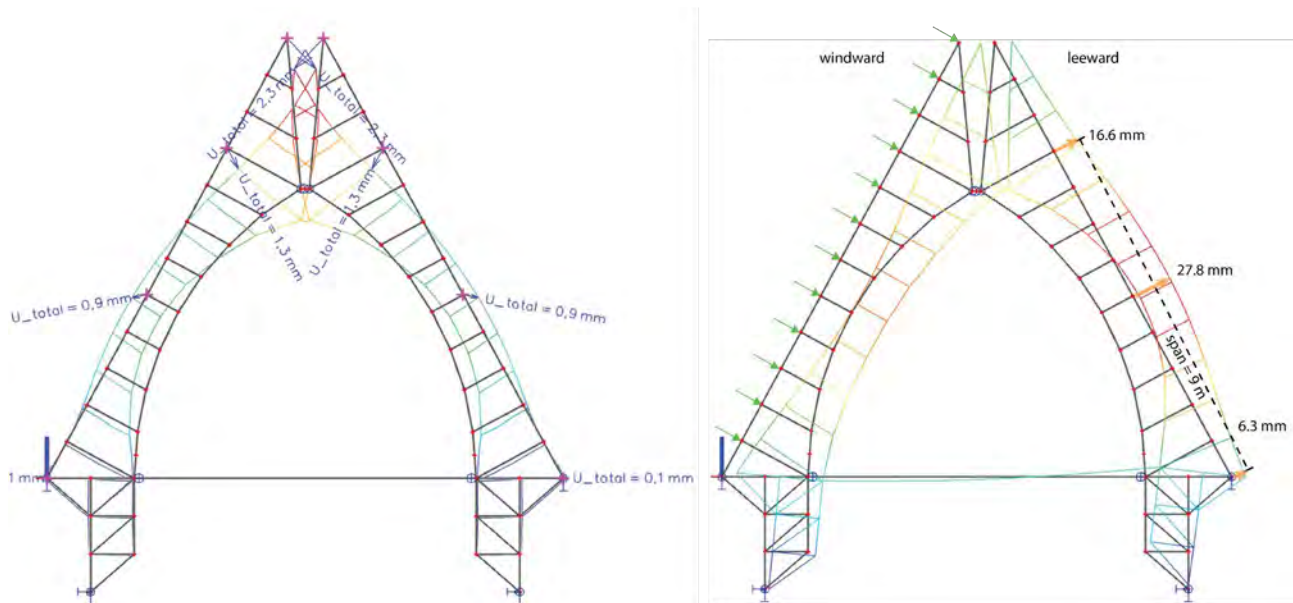


Figure 76 Transversal section of the Field Factory's rafters, displaying the SLS deflections. Left: SLS 2.1: the self-weight deflections (LC.1) + (LC.2). Right: SLS 1.2: rafter's self-weight (LC.1) + wind load (LC.3).

Structural Verification Rafters

For the structural design of the rafters of Notre Dame's timber structure, research has been done on different analytical verification strategies. They are presented and discussed in the appendix of the chapter. In this section, the *conventional stress analysis (CSA)* is used to check the forces in the rafters. The CSA appeared to be the governing and most proven verification method. The rafters are schematized as Vierendeel trusses consisting out of individual members with moment rigid connections. The most important findings are presented below while the full analysis is presented in the engineering report attached as an appendix.

The members of the rafter's Vierendeel truss model are divided into three groups for analyzing purposes. The top members, bottom members and intermediate members. The top and bottom members consist out of four layers of 40 mm Baubuche-Q panels resulting in a cross-section of 160 mm thick. Only the two inner panels are CNC'd in such a way that they create intermediate members, which therefore are (2 x 40 mm) 80 mm thick. The cross-sections of the resulting rafter can be seen on the right side of Figure 79 on the adjacent page. The height of each member was initially kept adaptable in case stresses reached critical values.

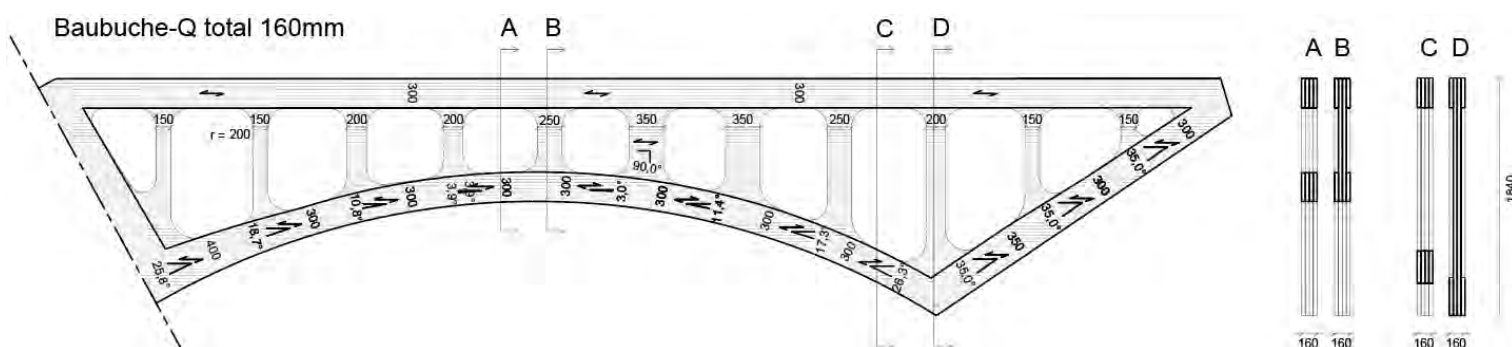
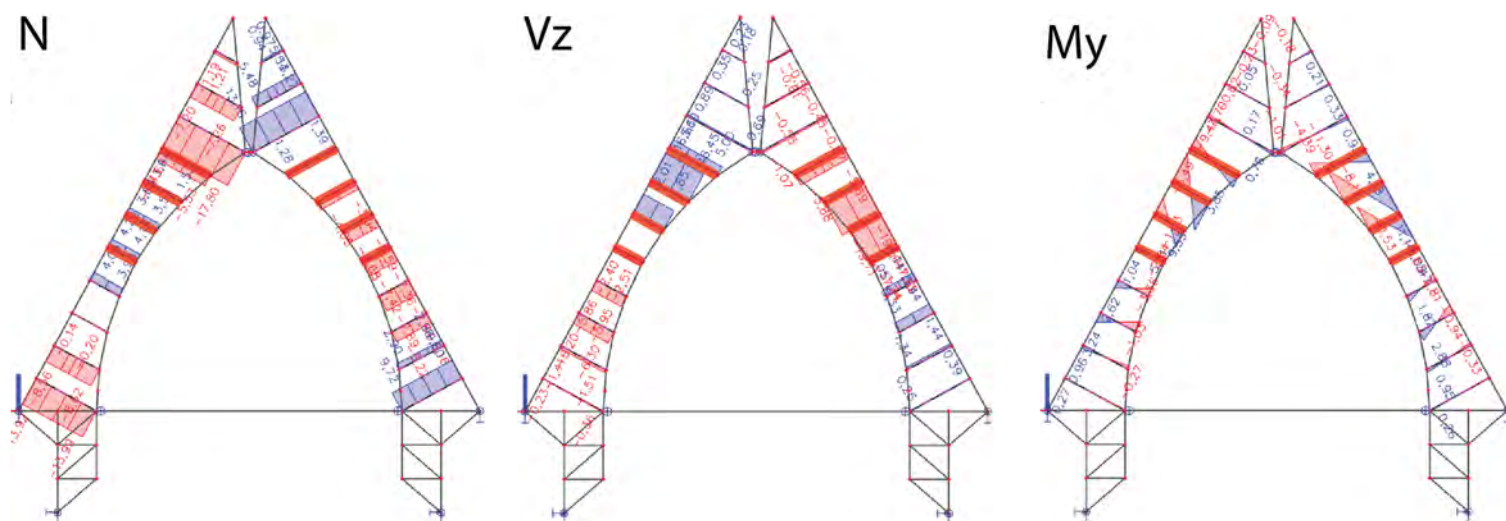
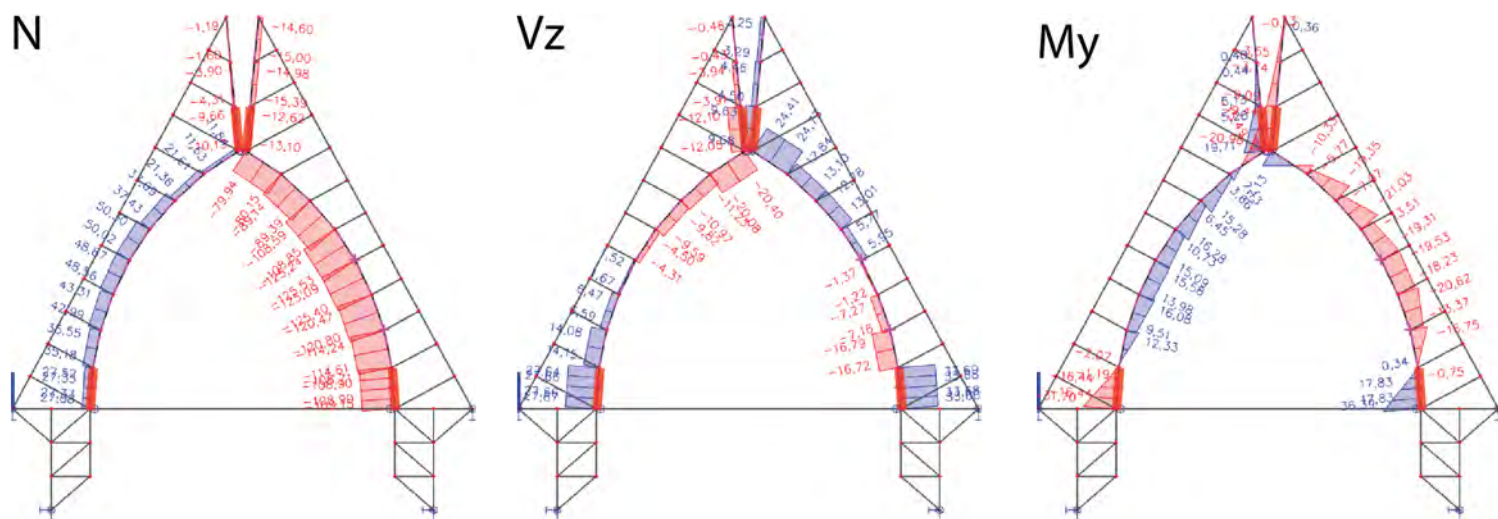
A design exercise was done to let the rafter fulfill its structural requirements while keeping structural expression in its appearance. The Vierendeel truss is an internally indeterminate structure, which means that any alteration in the geometry (stiffness) of individual members changes the forces in the entire structure. Each modification in the rafter's geometry, therefore, required an overall analysis of the structure. The resulting rafter geometry is depicted in Figure 79.

Top And Bottom Members

The governing load combination for the design of the top and bottom members is the ULS 1.8 (Engineering Report, Chapter 6). The stresses in the rafter's top members ($h \times t = 300 \times 160$ mm) are relatively low. The parallel direction of the members relative to the main grain directions results in a good structural performance. For the bottom members, the member forces are displayed in Figure 77. The bottom members deviate from the main grain direction leading to a quick reduction of the material strength. A grain angle of 3° degrees in LB5 and RB5 for example, doesn't lead to any reduction in the compression strength of the LVL, the bending strength however decreases significantly: -29%. Alterations of the bottom members' geometry were required at the lower part of the structure where the 25.8° degree angle to the grain caused an -26% compression strength reduction and an -80% bending strength reduction. Also at the tip of the rafter where the angle with the grain reaches a value of 35.0°, a larger cross-section height was required.

Intermediate Members

The intermediate members show relatively the largest geometrical variety. Different load combinations appeared to be governing for different intermediate members. There can not be assigned one single load combination which was governing for all the intermediate members. The member forces of ULS 1.3, which was governing for a few members, are shown in Figure 78. The analysis of all the different non-linear load combinations is provided in the Appendix (Engineering Report, Section 5.5). The centrally located intermediate members (R/LM5, R/LM6, R/LM7, R/LM8) required larger cross-sections than the initially opted 200 mm. This led them, due to the structure's internal indeterminacy, to take up higher moment forces. The verification-design loop eventually resulted in significantly larger cross-sections, up to 350 mm high. This increase in cross-section made the structure less prone to deflections as presented earlier in this chapter. This provided the freedom to change the geometry of the intermediate members at the bottom and top of the rafters resulting in a gradual change of cross-sections, ranging from 150 mm to 350 mm (Figure 79).



8. JOINT DESIGN

8.1. HINGING ROOF RIDGE JOINT

The hinging joint in the roof ridge consists out of a steel fork connection with recessed steel plates. Steel dowels are used to connect the steel plates and the LVL panels, resulting in a joint typically classified as a *connection with laterally loaded fasteners*.

The geometry of the joint is displayed in Figure 80 and the calculations are provided in the appendix of chapter 9. For the verification, the crane load case (LC.4) has been projected separately. The governing load case for the joint design is the ULS 1.4 resulting in a relatively high moment due to the eccentricity of the hinge. To allow proper structural and ductile behavior of the dowels, conservative edge and in-between spacing has been applied of relatively 84 mm and 60 mm. The high embedding strength of the Baubuche Q panels – even when considering an unfavorable load angle – caused Johansen failure mode H to be governing in the joint design, resulting in a load-carrying capacity per dowel of **29.3 kN**. Since the splitting risk in LVL with cross-plies is reduced, the effective number of fasteners is assumed to be equal to the actual amount of fasteners ($n = n_{ef}$).

The normal and shear forces are uniformly distributed over all fasteners, and the distribution of bending moment assumes elastic distribution over all fasteners. The total load per fastener can be determined by vectorial summation, displayed at the bottom of Figure 80. The force direction compared to the main grain angle direction of the LVL only has minor influence – a maximum embedding strength loss of 7.4% –, which yields that the coinciding direction of the normal, shear, and moment forces is decisive in determining the critical dowel. The governing dowel is positioned at the right bottom and subjected to a total load of **22.1 kN**. The structural performance of the joint is therefore considered sufficient.

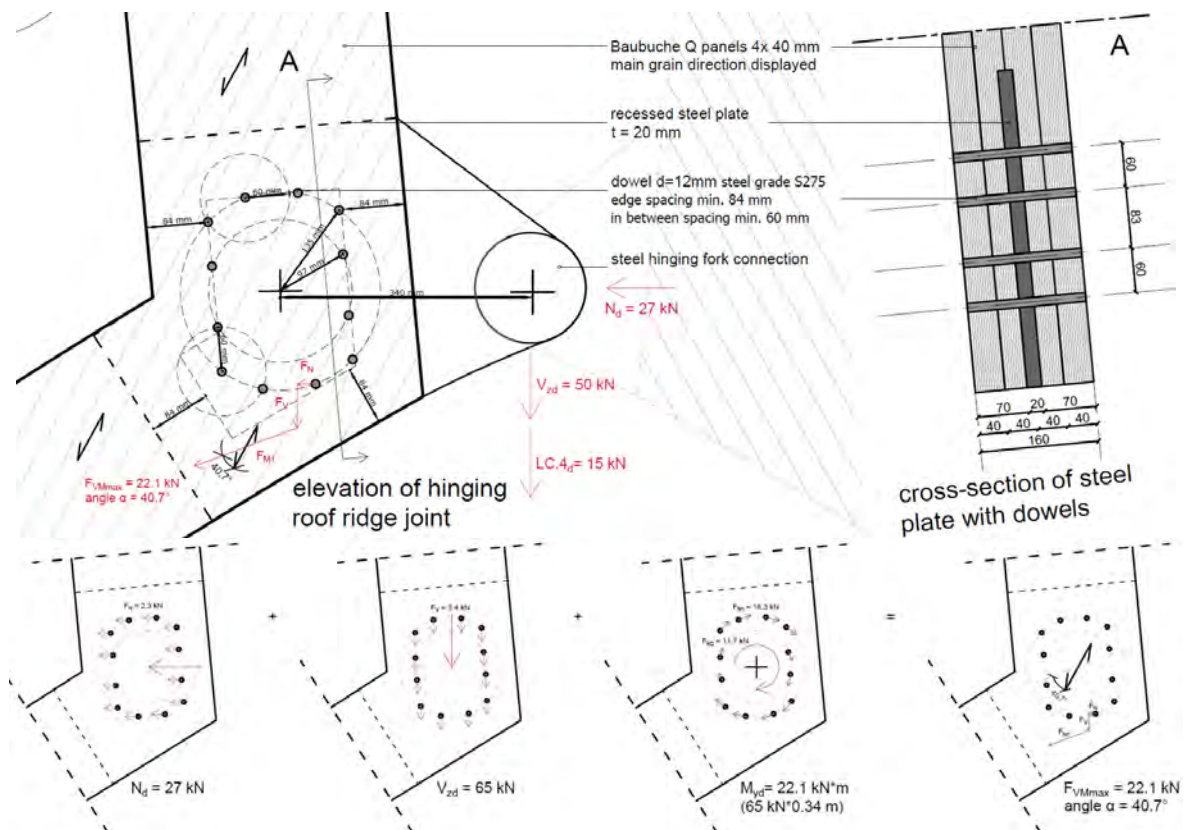


Figure 80 Top: geometry of hinging roof ridge connection with steel dowels $d=12$ mm. Bottom: forces on dowels from left to right: normal, shear, moment and total forces.

8.2. CONNECTIONS INTERMEDIATE MEMBERS

The connections between the intermediate members of the rafters have been proven difficult to verify with analytical design equations. The research presented in the appendix shows a wide variety of design models from which the typically known model of a *beam with large rectangular openings* is considered to fit best to the design of Notre Dame's rafters. The lack of knowledge and consensus is reflected in the fact that there are no analytical verification rules for the design of large openings (> 50 mm) in LVL beams given in de Eurocode EN 1995-1-1. In this section, the design of the intermediate members is presented using the verification equations from the National Annex (NA) of Austria, which are also used by LVL suppliers like Stora Enso and Metsä Wood.

The model and verification focus on the presence of stresses perpendicular to the grain. Due to the stress peaks that normally occur in the vicinity of the opening's corner, the local stresses are translated into a force perpendicular to the main grain direction ($F_{t,90}$) by means of the integration of stresses. This force is then compared to a resisting force at a presumed crack location ($R_{t,90}$).

Member RM7 subjected to load-combination ULS 1.8, displayed in Figure 81, appeared to be the governing intermediate member in the Scia Engineer model. The axial (compression) and shear forces are negligible, while the moment forces lead to tensile stresses perpendicular to the grain. The moment forces reach their highest values at the joints with the top and bottom members of the rafter. As a conservative approach, these forces are projected where the cross-section of the intermediate member becomes narrower. The presence of cross-plyies and the rounded corners - radius of 200 mm - significantly decrease the stress peaks that would occur in the vicinity of rectangular openings with sharp corners. A linear stress distribution is therefore assumed suitable for the verification, which for the critical crack leads to $F_{t,90} = 27.9$ kN.

Due to the relatively complex geometry of the rafters, the exact crack location can't be determined beforehand. The resistance $R_{t,90}$ of four possible cracks is therefore evaluated. The tensile force resistance $R_{t,90}$ is derived by multiplying the resistant area ($R_{t,90,A} = 0.5 \cdot b \cdot l_{t,90}$) with the material strength perpendicular to the grain ($f_{t,90}$). It is then corrected with a Weibull-type size-dependent degradation in strength for LVL with cross-plyies ($k_{t,90}$). For the length of the crack ($l_{t,90}$), the region is taken where the moment force causes tension perpendicular to the grain. The governing crack, also displayed in Figure 81, has an $l_{t,90} = 175$ mm resulting in a tensile force resistance of $R_{t,90} = 47.4$ kN, leading to a unity check of $UC = 0.59$. The dimensions of the connections between the intermediate members and the bottom- and top members of the rafters are therefore considered sufficient.

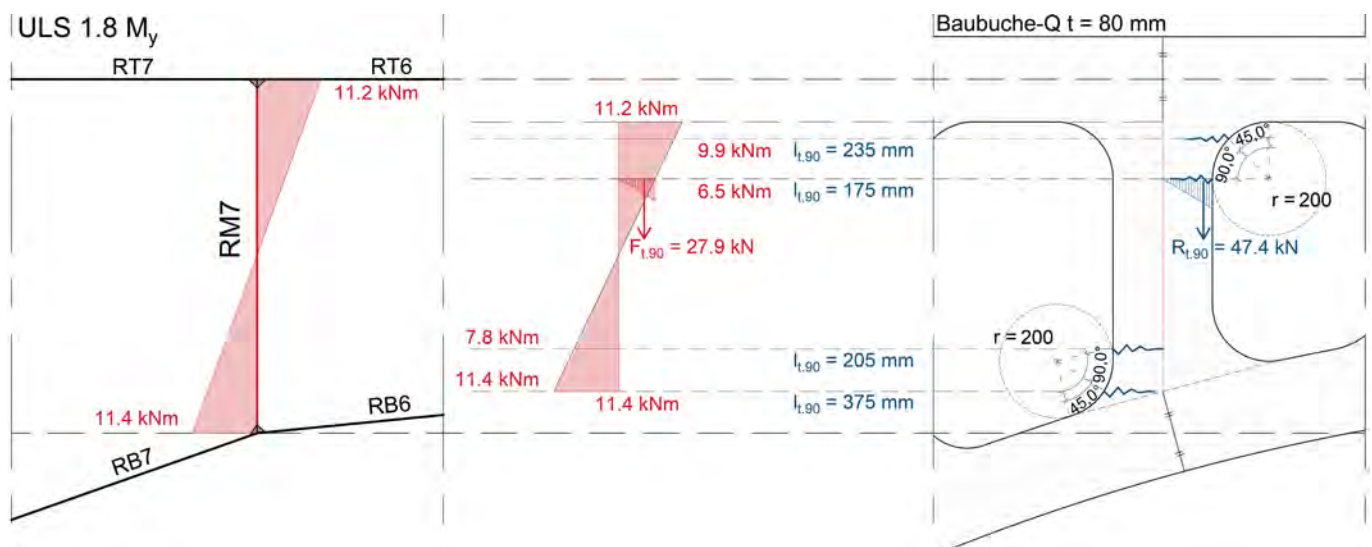


Figure 81 (left) Intermediate member RM7 subjected to ULS 1.8. (middle) Forces projected on the narrow part of RM7 with moment force leading to $F_{t,90}$. (right) Possible crack locations and the governing crack resistance $R_{t,90}$.

8.3. DEFLECTIONS ROOF RIDGE JOINT

The deflections of the rooftop subjected to the SLS load combinations are presented in this section. The horizontal window frames, displayed on the adjacent page, have to be designed in such a way that displacements can take place freely. The required free movement in the tip of the roof structure is dependent on the relative displacement of the left and right rafter. Vertically this is governed by SLS 1.2 in which a deflection difference of 2.4 mm has to be enabled. Horizontally the tips of the rafters need to be able to freely move 6.6 mm apart and 3.2 mm towards each other. It is decided to let the movement take place in one joint which requires a free movement of 9.8 mm horizontally and 2.4 mm vertically, which is accomplished by allowing the deflections to take place in the connection between the rafter and window frames.

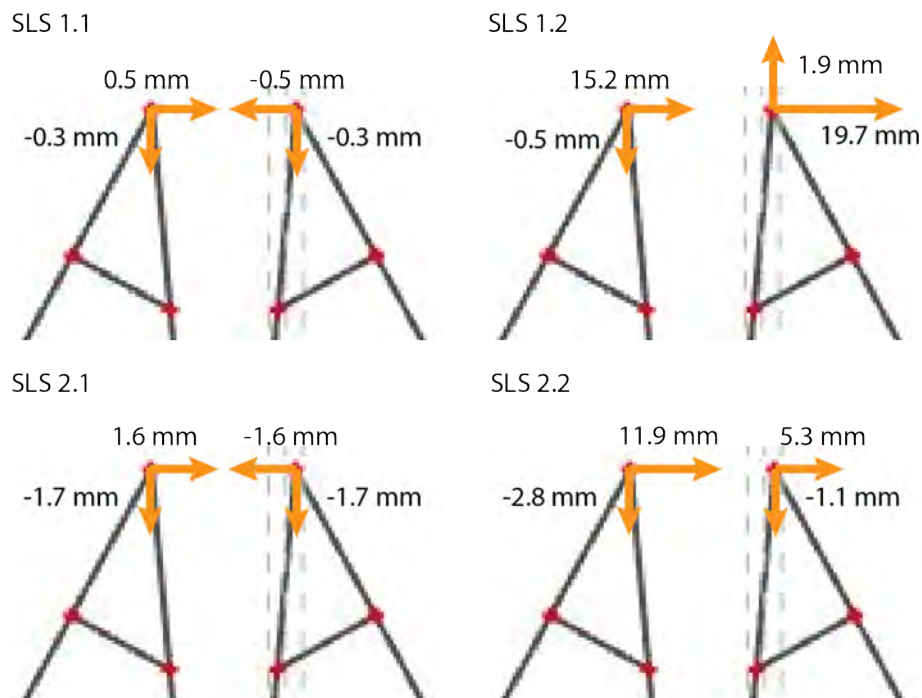


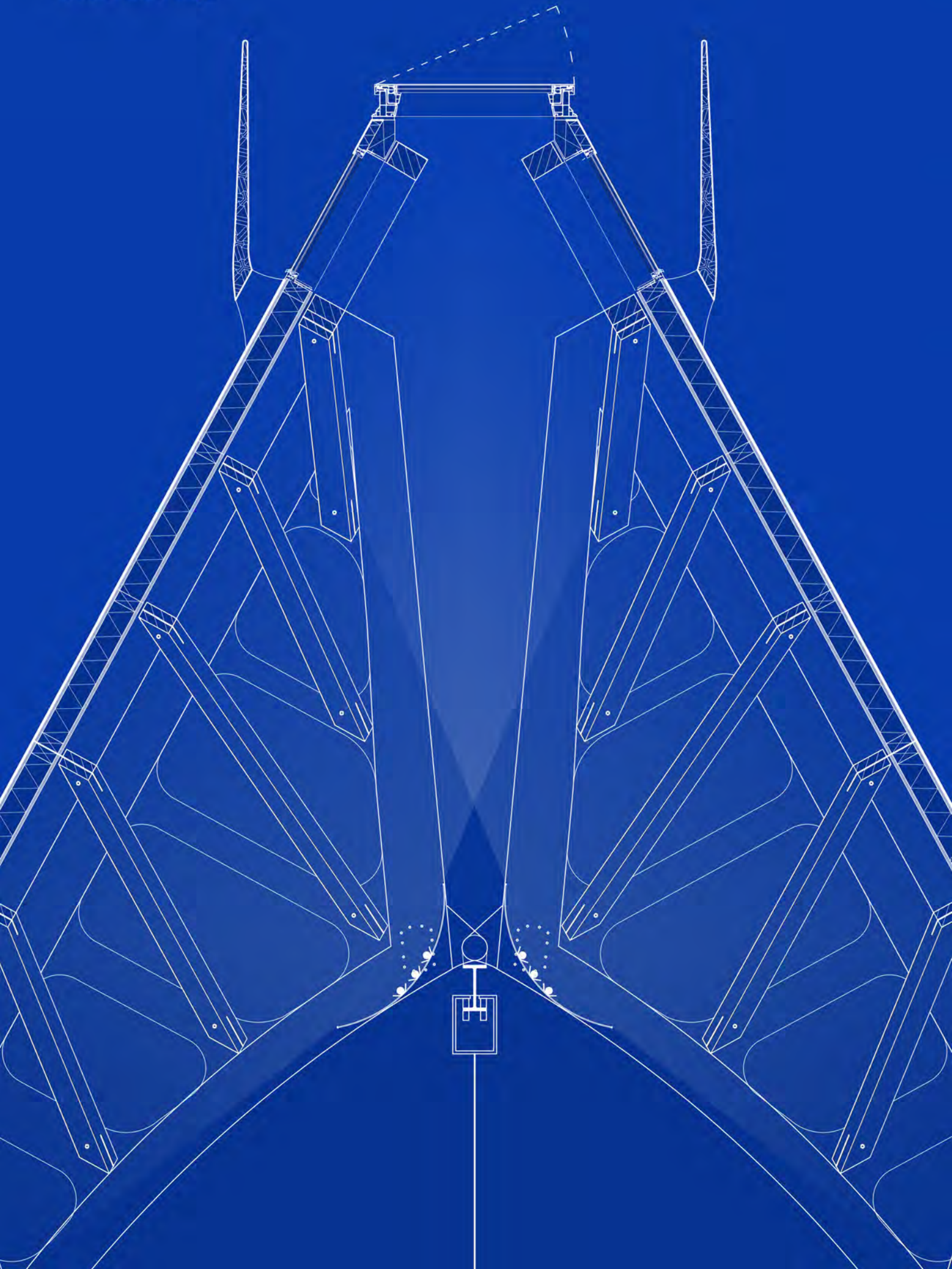
Figure 82 Deflections of the top of the rafters subjected to the SLS load cases.

8.4. FORMER FINISHING

In the proposed design, the joints of the roof panels are designed such that they help to stabilize the structure in the longitudinal direction, but that they don't contribute to the rafter's structural behavior in a transversal direction. By doing so, the connections between the rafters and the roof panels can be kept relatively simple since they only have to resist the modest forces in the longitudinal direction.

Figure 83 Detail roof ridge joint displaying the different joint designs performed in this chapter.

Detail Roof Ridge



9. DISCUSSION AND RECOMMENDATIONS

What is the design's feasibility, which alterations are advisable, and which conclusions can be drawn?

The presented design and structural verifications will be reviewed in this chapter. Where the design could be further enhanced, recommendations are presented. The chapter can roughly be divided into three parts. The first part discusses the limitations of the structural model and which future alterations and analyses are recommended. Part two revolves around the particular geometry of *The Timber Skeleton* and its associated structural performance. The last section discusses possible alterations in the material, more specifically on the lay-up of the cross-laminated LVL panels.

Model Input & CSA Analysis

Firstly, the complexity of the existing cathedral will require more detailed information as input for the structural model. It is recommended to precisely model the wind loads acting on the roof structure, especially since they appeared to be governing the deflections and forces in the structure. On-site or wind tunnel testing is advised to precisely model the influence of the buildings in the direct surrounding of the cathedral, as well as Notre Dame's bell towers and the Gothic ornamentation. The structural model of the transversal section in this thesis is a basic timber structure unit on top of a uniform limestone structure. The complex and damaged geometry of the cathedral, with its cross-shaped plan with stone façades at the heads of the transept and a curved roof shape at the end of the choir, will require tailor-made design changes and verifications. Also, the inhomogeneity of ancient masonry, resulting in local vulnerable spots, will need extensive analyses and if necessary repair.

Secondly, the internal structural indeterminacy of the rafters made the force distribution dependent on the stiffness of the individual members. It is recommended to model the stiffness properties of the members more accurately. Mainly the bottom members with deviating grain angles are expected to have varying stiffness properties. Furthermore, the behavior of the joints between intermediate members and top and bottom members in the rafters will need further research and numerical and physical testing.

The complex geometry of the rafters comes with many uncertainties for which till today no obvious standard analytical verification methods are available. The use of a Vierendeel truss as a schematical structural model provided the opportunity to subdivide the rafters into smaller elements, which gave more insight into the local forces in the rafters. Besides resulting in the most conservative results, the CSA is assumed to be an appropriate verification method since the rounded corners of the rectangular openings and the cross-ply will lead to more linearly distributed stresses. The exact behavior of the connections between the individual members remains a subject of debate. Alternative verification methods like the ones based on fracture mechanics show promising results. These methods however have a narrow focus on a specific geometrical range, which in combination with their almost non-existing practical application would make their use questionable.

If future verification of *The Timber Skeleton* will also be based on CSA, it is recommended to create a parametric model of the LVL Vierendeel truss. The quick automated stiffness alterations and verifications of a parametric model could provide the opportunity to relatively easily optimize the structural behavior of the rafters.

Geometrical Considerations

It can be concluded that the requirements for the support conditions, only being subjected to compression forces, were fulfilled. The concept of guiding the wind forces from the windward side of the roof down to the flying buttress at the leeward side of the cathedral turned out to fit the design of *the Timber Skeleton*. The Fast Fix cassettes play - as compressive members - an important role in leading the forces from the windward rafter to the leeward side. Also when the wind is absent, the Fast Fix cassettes – as tension members – play a modest role in the global structural behavior. It is therefore recommended to cautiously design the connection between the Field Factory's structure and the Fast Fix cassettes.

In the preliminary design phase, a rough structural analysis combined with the aesthetical wish of an open attic space resulted in the particular curved bottom shape of the rafters, together forming a Gothic pointed arch. The pointed arch furthermore allows for easy transport of large objects using the integrated crane. The resulting geometry has been leading in the subsequent design steps but caused some difficulties in the structural verifications. The rafter's geometry performed well at the windward side of the roof structure. On the leeward side, however, the curved bottom member resulted in a relatively inefficient load path that didn't prevent a global hogging moment to occur. If the structural verification would have been leading in the design, a more efficient load path could have been created by straightening the bottom part of the rafters. Besides, a straight bottom member could have prevented the large grain angles which in the current design appeared to reach critical values.

The decision to create openworked rafters in the quest for structural expression significantly increased the complexity of the structural analysis. The elaboration of the rafter's profile resulted in an aesthetically pleasing result, but the available analytical knowledge to design such big rectangular openings – with rounded corners - appeared limited. From a structural point of view, alternative solutions have been evaluated. A possible alternative could have been the addition of diagonal members, making the structure stiffer and decreasing the moment forces in the intermediate members. Maybe the biggest advantage of adding intermediate diagonals would be the increased predictability. From a purely material standpoint, the next section will show how the approximately 45° degrees angle can be considered to be the least efficient application to resist bending forces. However, the adjusted structural scheme with diagonals would limit the moment forces in the structure, turning it into a regular truss mainly subjected to axial forces.

If the rectangular openings and the resulting Vierendeel truss geometry are assumed to be an important starting point for the design – which was the case for *The Timber Skeleton* -, the production process of CNC'ing cross-laminated LVL panels can be seen as an interesting production process to create moment rigid connections. Its production and assembling process could have many benefits over regular moment rigid connections with mechanical fasteners. Numerical and physical testing however required to get more grip on the exact stress distribution in these types of joints.

Material: Cross-ply Ratios

The insertion of cross-ply during the production process of LVL provides the possibility to alter the material's strength and stiffness properties and creates the opportunity to optimize these properties depending on the design. For the Vierendeel truss design of Notre Dame's rafters, the bending strength properties are of special interest as they appeared governing in the verification. *Figure 84* captures the bending strength of three different cross-laminated LVL lay-ups applied in different angles to the main grain direction. The standard Baubuche-Q lay-up with 14% cross-ply, used for *The Timber Skeleton* of Notre Dame de Paris, an alternative Baubuche-Q lay-up with 28% cross-ply, utilized for the International House Sydney (Aicher and Tapia, 2018), and an imaginary Baubuche-Q lay-up with 50% cross-ply (the asymmetrical lay-up of cross-ply presented, in reality, would not be desirable). The bending strength values are derived using an equation by LVL supplier Stora Enso (2020).

By applying the information from *Figure 84* to the design, a few observations and design recommendations can be derived. First of all, it can be observed that all lay-ups display a significant decrease (>50%) in bending strength within the range of 15° - 75°. The imaginary 50-50 veneer configuration reveals that the limited shear strength, which doesn't differ per lay-up, results in the lowest bending strength values at a grain angle of 45°. As can be expected, a higher cross-ply ratio increases the bending strength perpendicular to the grain.

The top members of the rafters are perfectly orientated with an assumed 0° angle to the grain. To exploit their strength and stiffness properties it is therefore recommended to not increase the number of cross-ply in the top members. This becomes even more relevant in an internally indeterminate structure like a Vierendeel truss, where a lower stiffness of the top members would increase the forces in the members with a less efficient grain angle. The rafter's critical bottom members have a grain angle between 25° and 35°. *Figure 84* shows clearly that for this angle range the different lay-ups show little bending strength difference. Changing the number of cross-ply for the bottom members is therefore assumed to not improve the structural performance significantly. For the intermediate members, with a grain angle of 90°, the application of more cross-ply can be considered to be a relevant design improvement, enabling significant cross-section reductions. However, the low bending strength values for angles around 45° remain important as they can play a role in the rigid connection between the intermediate members and the top and bottom members where the stresses have to be diverted.

The manufacturing process of LVL doesn't allow the application of different cross-ply ratios within a single panel. However, the proposed assembling process of Notre Dame's rafters, consisting out of four LVL panels glued on top of each other, allows the application of different cross-ply ratios for the inner panels (intermediate members only) and outer panels. Although the intermediate members could benefit from more cross-ply, the bottom members would not. Besides, the increased amount of cross-ply will have a negative influence on the structural performance of the top members. The unequal cross-ply ratios will lead to different stiffness properties which require extra verification steps to find out what their shared structural performance is; if normal and shear stresses are distributed homogeneous and if high local stresses can be expected in the adhesives between the panels.

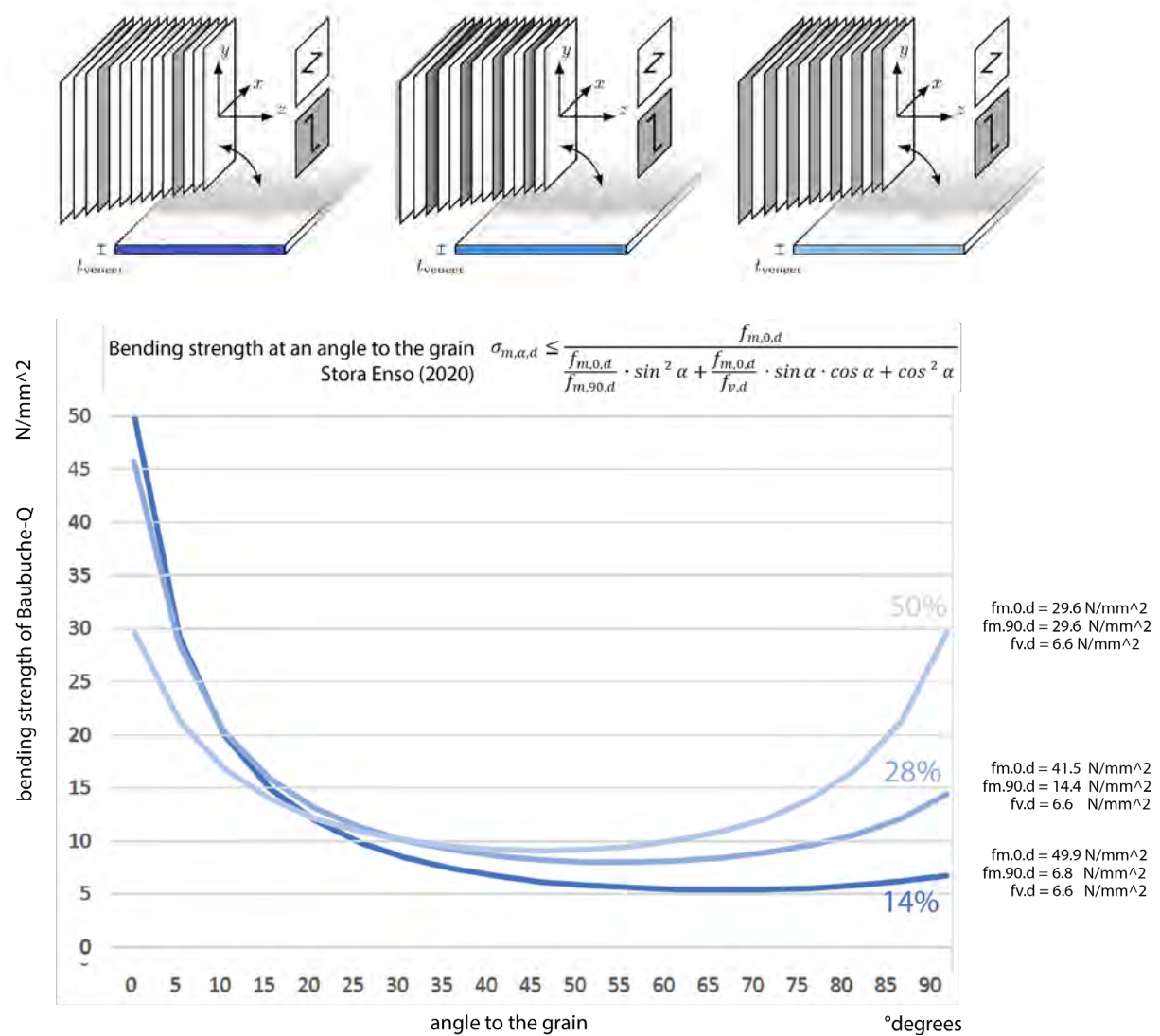


Figure 84 Bending strength at an angle to the grain: Standard Baubuche-Q panels with 14% cross-plyes (dark blue), Baubuche-Q panels with 28% cross-plyes (blue), and (imaginary) Baubuche-Q lay-up with 50% cross-plyes.

10. CONCLUSIONS

How can historical Gothic construction principles, like integrated support structures and utilization during construction, be combined with modern timber production processes to design a new roof structure for Notre Dame de Paris?

A Gothic Inspired Reconstruction Strategy

The research presented in the current thesis shows how an alternative reconstruction strategy for Notre Dame de Paris could result in a functional design that pays homage to the Gothic engineering principles of integrating support structures and utilization during construction. The Fast Fix cassettes don't only help to stabilize the cathedral, but also function as the floor structure of the attic and as a tie-beam in the structural model of *The Timber Skeleton*. Furthermore, the attic's temporary function as Field Factory will stimulate the reconstruction works, while the eventual function as an ornamentation workshop will help to prevent deterioration in the future. Functionally, it can therefore be concluded that the new design provides a suitable answer to the research question.

The appropriation of Gothic engineering and construction concepts was used to tackle the time pressure that's projected on the reconstruction of the cathedral. For that sake, one can be critical of the proposed design of *The Timber Skeleton* which due to its complex geometry in reality will require much numerical and physical testing. The extra time this will take counteracts the wish for a quick reconstruction and hamper the function of the attic space as a Field Factory.

Furthermore, the expandable sub-assemblies will have to deal with many execution requirements and be subjected to different temporary load conditions which are not addressed in this thesis. A design that wouldn't have taken prefabrication to such an extreme, and would merely be focussed on practicalities like easy manufacturing and assembly could have overcome the need for expandability. The proposed design in that sense could be seen as a provoking attempt to take prefabrication to an extreme to absolutely minimize on-site construction activities. Besides, the visible hinging connections in the final design are appreciated for their aesthetics as remains of the construction and assembly process, similar to the ribs of the vaulted ceilings.

A Contemporary Timber Structure On Top Of An Ancient Monument

The design of *The Timber Skeleton*, based on the use of expandable sub-assemblies resulted in a timber structure with the desired structural expression as an homage to *The Forest*. This aesthetical wish didn't result in the most economical design, which became clear in the structural verification of the rafters. The design however can be called innovative, or at least progressive in that it touches - and maybe even crossed - the line of the current knowledge and verification methods for LVL structures. A Master Builder's mentality of prototyping and trial and error will be needed to realize the proposed design.

The characteristic fast repetitiveness of prefabrication cannot be utilized as easily as for an entirely prefabricated structure, since the geometry of Notre Dame's limestone structure is highly irregular. The use of computerized production processes that prefabrication enables, however, can be applied to realize a tailor-made fit between the new roof structure and the ancient stone skeleton. The resulting geometrical variety of the new roof structure will require extensive structural analyses, for which modern computer technology can be a solution. Interestingly, the precise measuring - with 3D scanners (Sandron and Tallon, 2013) - and manufacturing has been adopted by the authorities, but until now it's only been used for the design of the stabilizing structures. The authorities' conservative approach to not apply these modern technologies for the reconstruction of the actual roof can at least be called remarkable (Figure 85). The proposed design shows how modern production processes could help to arrive at a modern interpretation of *The Forest* without neglecting the cathedral's aesthetical and symbolic value.



Figure 85 Contrasting manufacturing and assembly strategies are adopted by the authorities. Top: modern timber structure to stabilize vaults. Bottom: medieval techniques utilized to reconstruct example truss (Zachmann, 2020).

11. REFLECTION

From the very start, the presented research has been executed with a broad view and a versatile methodology, leading the design into unexpected territory. The reconstruction strategy and the design of *The Timber Skeleton* structure are original, but therefore also unpredictable, something that became troubling during the structural analyses of the rafters.

From the start, the reconstruction plans of the Frech authorities have been observed and compared with the research. It has been inspiring to follow thousands of restoration laborers working every day on the restoration of Notre Dame. It quickly became evident that this thesis could only capture a small part of the cathedral and the complexity of the reconstruction. But as time passed, it became also clear that the authorities found themselves in a complex situation, eventually leading to the deforestation of a rare underwater forest in Ghana. It fuelled the quest to show what an alternative design approach could result in.

It has been challenging to balance between limiting the scope of the research and design, without losing grip on the reality of the complexity of Notre Dame's reconstruction. By excluding the unstable, half-melted, scaffolding structure and the poisonous construction site the project found its design freedom. The proposed reconstruction strategy though will have to face many more restrictions in reality.

One of the contributions of this research is that it questions the conservative way in which some deal with the monuments in the building environment. As explained in the thesis, the debate around the maintenance and reconstruction of old buildings is a reoccurring phenomenon in which there is no clear right or wrong. One should be critical however on the strange event of cutting down an underwater forest in Ghana to rebuilt a cathedral's roof in Paris. Furthermore, the design tries to demonstrate, just like the Gothic cathedrals, how technical and practical challenges can be combined with aesthetical wishes and crystallized in a beautiful structure. The design process has been difficult to capture in the thesis as many different design methods were used, ranging from sketching, physical and digital modeling, hand calculations and material research. At some point in the research process, it was decided to make a clear cut between the aesthetical design and the structural verification. Redesigning the roof structure from scratch with the knowledge gained during the research would be an interesting challenge.

The "architectural" design decisions forced the structural verification to the boundaries of what is currently known. As the timber industry is currently booming, experienced specialists were difficult to reach. Literature and common sense were used as ingredients to choose the proper verification strategies. Although literature suggested alternative verification theories like fracture mechanics, its limited practical application and physical tests became problematic. Therefore the *conventional stress analysis* was utilized. The research into alternative verification methods for LVL structures, attached in the Appendix, was used to get more insight into the possible critical aspects of the structure. The gained knowledge will for sure be useful in future projects.

APPENDICES CHAPTER 7

- Verification methods for LVL beams and trusses (Field Factory).
- Wind loads according to EN 1991-1-4:2005 + wind loads on Notre Dame.

APPENDICES CHAPTER 8

- Engineering reports Fast Fix and Field Factory.

APPENDICES CHAPTER 9

- Roof ridge joint calculation.

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The forces and geometry of the rafters used in this appendix are from the initial design. They, therefore, differ from the final structural analysis presented in the main body of the thesis and the engineering reports.

VERIFICATION METHODS FOR NOTRE DAME'S RAFTERS

The design of Notre Dame's rafters is rather unusual for a timber structure. The analyses in this thesis remain analytical, although much of the used research is based on numerical and physical tests. In the presented chapter different analytical verification methods will be used to check the feasibility of the rafters' design.

For analyzing purposes, the rafters will be schematized in two different ways. The first schematical model assumes that the rafters are single beams with large rectangular openings. The increasing amount of timber highrise buildings has initiated interest in the possibility of holes in beams for building services. Large openings in timber beams are therefore nowadays reoccurring design cases and are subject to much research and debate (Dröscher et al., 2016). Although the rafters of Notre Dame don't meet some of the geometrical requirements to apply the design equations, they will help to reveal the vulnerable spots around the rafter's openings and provide first estimations of the stresses. In the second engineering model, the rafter will be schematized as a Vierendeel truss consisting out of individual members with moment rigid connections, for which two verification methods will be addressed.

Figure 86 Hess Timber beams for International House Sydney: multiple rectangular openings close to each other. The beam is built up out of glulam beams with cross-laminated LVL panels in-between.



BACKGROUND TIMBER BEAM WITH OPENINGS

Interest without consensus

To the present day, there has not been a completely accepted design method for beams with large openings. The strength analyses have been approached through different – sometimes combined - mechanical theories, with different interpretations and assumptions, resulting in a large range of outcomes. The theory often shows discrepancies with the numerical and experimental results (*Figure 87*). The lack of knowledge and consensus is reflected in the fact that there are no analytical verification rules for the design of large openings (> 50 mm) in timber beams given in de Eurocode EN 1995-1-1.

The national annexes of Germany and Austria (DIN EN 1995-1-1/NA) give some guidelines in the handling of openings in LVL beams in service class 1 and 2 conditions, based on a semi-empirical approach done by Kolb and Epple (1985). Furthermore, researchers and companies propose different design methods depending on the applied theory, numerical analyses, and physical tests. The physical testing proved that the design equations in the codes/handbooks are very sensitive for the geometry of beams and the opening's position, size and geometry. Structural members, with a from the standard deviating geometry, therefore, deserve caution, which is clearly the case for Notre Dame's rafter.

Much of the research has been focusing on glulam beams of common sizes subjected to shear- and moment forces. Few authors cast doubt on the application of similar design checks on other engineered timber products like LVL, since the microscale timber fibers demonstrate comparable failure behavior. A significant difference between glulam beams and the Baubuche Q panels used for the rafters are the cross plies which significantly increase the strength in the perpendicular direction and help to reduce the brittle behavior (Aicher and Tapia, 2018).

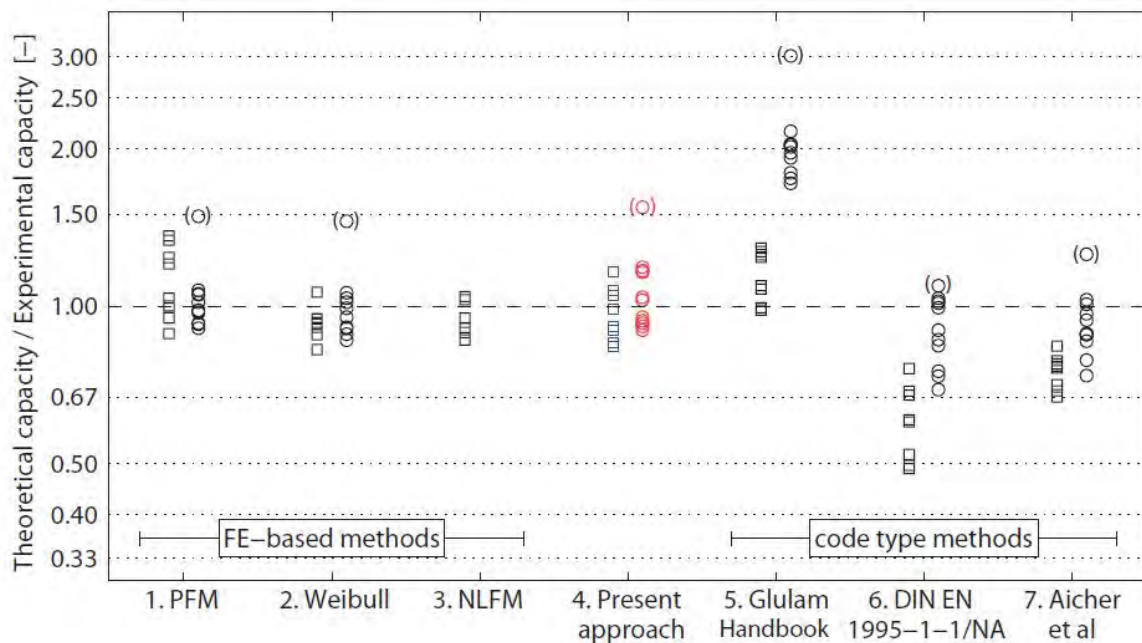


Figure 87 Comparison between different strength analyses, presented by Danielsson and Gustafsson (2015).

Empirical Research

In the last decades, researchers gained knowledge through numerous physical experiments. The direct proximity of the openings is of interest since at an early stage during testing cracks develop which eventually lead to failure. For buildings, it is recommended to consider the first crack as the “ultimate” load a beam can resist. The difference between the first crack and the maximum load is used as a buffer to deal with loading effects as a result of shrinkage and swelling due to temperature and moisture content fluctuations (Stora-Enso, 2020).

Although the exact location of cracking depends on many geometrical and material factors, research has created a clear view of the origin and progression of cracks, as can be seen in *Figure 88* for a circular hole in an LVL beam subject to shear and bending (Ardalany et al., 2013). Literature and codes/handbooks make a distinction between circular holes and rectangular openings – with prescribed rounded corners – as significant differences occur in their failure behavior. For rectangular openings, Danielsson and Gustafsson (2015), assumed that the crack location is approximately at an angle of 45 degrees measured from the center of the corner's radius (*Figure 89*, left) and that inside a rectangle with rounded holes a perfect rectangular opening can be assumed with a height decreased with two times $0.3 \cdot r$ (*Figure 89*, right).

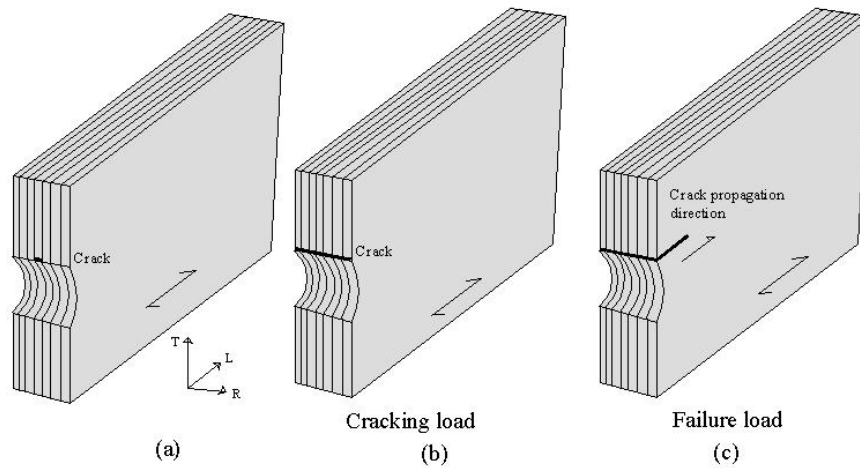


Figure 88 Crack initiation and propagation in case of a circular hole in an LVL member: (a) crack starts in the center of the beam in the round corner (45° angle), (b) crack develops towards the beam's surface, and (c) crack continuous along the beam's outer surface until failure (Ardalany et al., 2013).

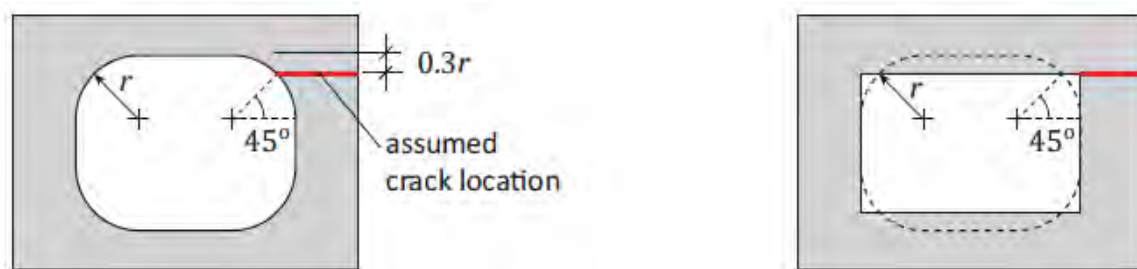


Figure 89 Location of cracking for a rectangular opening with rounded corners (Danielsson and Gustafsson, 2015).

Stress Peaks

The theoretical explanation for the location of the cracks is the presence of high stress peaks in the vicinity of the opening's corner. The reason for these stress peaks is the disturbing stress flow in which normal and shear stresses have to be diverted around the opening. *Figure 90* displays the resulting peaks in shear stress (left) and tensile stress perpendicular to the grain (right) in the corners of a rectangular opening (Aicher and Tapia, 2018). Both stress peaks can lead to brittle failure of the structure and are frequently governing in the design. It's generally assumed that shear stress peaks only depend on the presence of shear forces (a), and that tensile stress peaks perpendicular to the grain can be caused by both shear and moment forces (b).

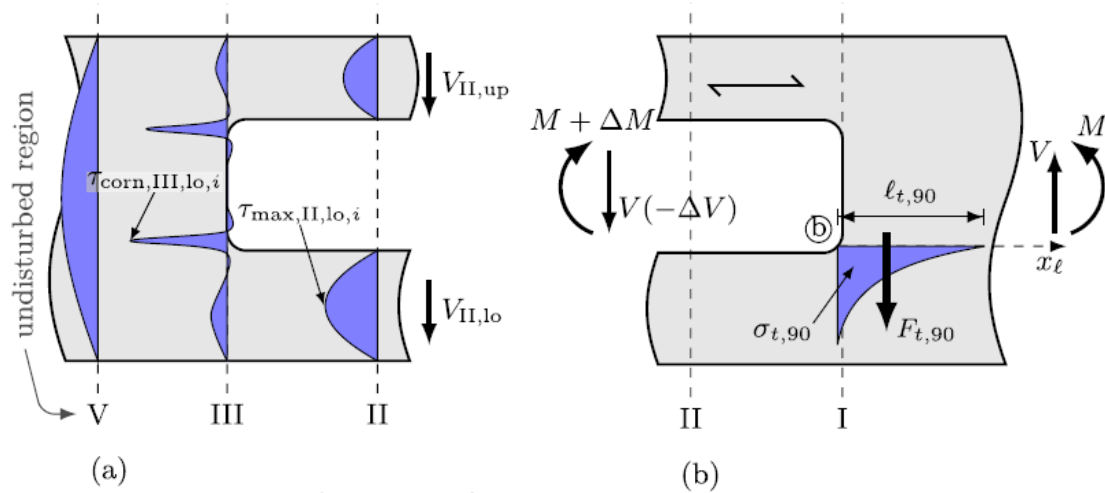


Figure 90 Shear stresses and tensile stresses perpendicular to the grain.

Figure 91 shows the stresses perpendicular to the grain for a circular opening in the case of shear (left) pure moment (middle) and axial tension (right) (Danielsson and Gustafsson, 2015). It has to be noted that for visualization reasons the stress values in the case of a pure moment had to be multiplied by a factor of 10. One can conclude therefore that, for a circular opening, shear forces are critical in the formation of tensile stress perpendicular to the grain.

Axial Stress

Most of the codes and handbooks only consider shear and moment forces acting on beams. This can be explained due to the industry's specific interest in openings in horizontal beams and the fact that much of the physical experiments have been done using standard beam tests, during which normal forces are absent. For Notre Dame's rafters, axial forces are clearly present and should, when possible, be taken into account in the design equations, even though the axial forces are mainly compression and will probably lead to minor shear or perpendicular tensile stresses. Danielsson and Gustafsson (2015), who use a *beam theory fracture mechanics approach*, are one of the few authors found that include axial forces in their derivations.

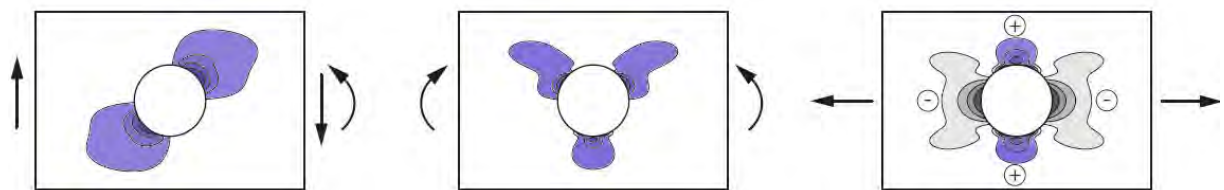


Figure 91 Stresses perpendicular to the grain (y-axis is vertical) in numerical analysis of a circular hole: left: shear, middle: moment forces, right: axial tension.

Theories For Brittle Failure

For the brittle failure of materials, there are a few theories to describe and predict the event of failure. In the quest for consensus on how to design beams with openings, authors have approached the problem through different perspectives which result in a range of verification methods.

Generally, the dominating verification method for timber structures can be referred to as *conventional stress analysis* (CSA), which compares the stress in an assumed ideally brittle and homogeneous material with the material strength. Several design methods are based on the Weibull theory which presumes that “A chain exposed to tensile stress is as strong as its weakest link.” Weibull (1939) described the stress distribution in ideally brittle heterogeneous materials. The Weibull theory is widely used for the verifications of brittle failure modes in timber structures, which for example can be seen in the volume effect formulas (Blaß and Sandhaas, 2017). The downside of both CSA and Weibull's theory is that the verifications can not be used for elements with significant stress peaks and singularities (Danielsson and Gustafsson, 2011). The methods are therefore often combined with the *mean stress method* in which the stress peak is assumed to spread over a presumed area.

Fracture mechanics, the science of crack propagation in ideally brittle materials, can be an alternative point of departure to analyze brittle failure modes in LVL. To determine the crack load, different failure modes can be distinguished (Figure 92). Due to the anisotropic behavior of wood, two types of failure mechanisms are assumed to govern the design: tension perpendicular to the grain (mode 1: opening), and longitudinal shear (mode 2 sliding). For timber structures, it is assumed that the plastic zone at the tip of the crack is relatively small compared to the length of the crack and that therefore linear-elastic principles can be applied. The resulting theory is better known as Linear Elastic Fracture Mechanics (LEFM).

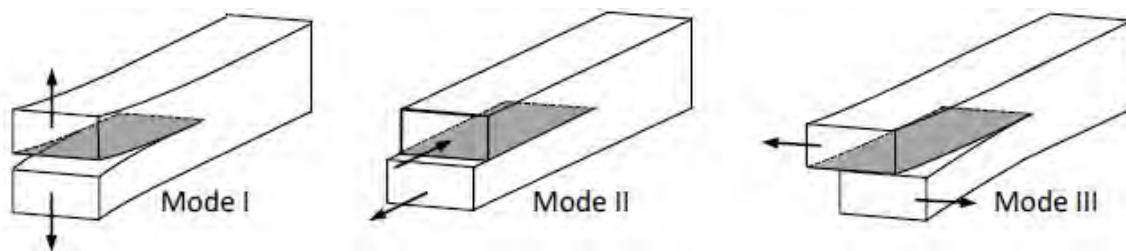


Figure 92 Fracture Mechanics' opening modes. 1. opening, 2. sliding, 3. tearing

To apply LEFM to a structural design problem, first the release-energy G at the fracture surface has to be determined. It is then compared with the material's critical strain energy release rate, better known as fracture energy, which depends on the relevant opening mode. For Notre Dame's LVL rafters the following fracture energy values are used for Baubuche Q panels including cross-plyies $G_1 = 0.7 \text{ N/mm}$, $G_2 = 1.2 \text{ N/mm}$ (Ardalany, 2012). It is important to mention that in a pure LEFM analysis the material properties that determine the resistance to crack propagation are only the modulus of elasticity E and fracture energy G_c , and that material strength values are not required to determine the maximum load. For an easy application though, many codes and handbooks provide equations to translate the material strength values into fracture energy values, which are also included in this chapter's appendix.

RAFTER AS IMAGINARY BEAM WITH A LARGE RECTANGULAR HOLE

Different Material / Different Geometry

The presented verification equations and the accompanying design checks give an insight into the possible local stresses that occur in Notre Dame's rafters. Extra attention should be given to the applied type of LVL and the geometry, especially since the shape of Notre Dame's rafters falls outside many of the numerically and physically tested specimens. The presence of cross-plyies in Baubuche Q panels is taken into account through a significantly higher material strength in the perpendicular direction (8 N/mm²) and by using fracture energy values which are provided specifically for LVL panels with cross-plyies. The sensitivity of the formulas for the geometry of the design is an important factor. To take the deviating geometry of Notre Dame's rafter into account, each verification method is adjusted as well as possible.

Definition Of Forces

Analytical verification using the "beam-with-rectangular-holes" approach requires a translation of the forces inside Notre Dame's rafters into "beam forces". The forces in the relatively complex geometry of Notre Dame's rafters are checked with Scia Engineer by assuming that the rafter behaves like a Vierendeel truss with rigid connections. The forces are then projected on an imaginary beam running through the middle of the truss, as displayed in Figure 93. The maximum normal- shear- and moment forces are given by Table 4 (in connecting members of intermediate members (B100 - B104), which will be used in the verification in the next sections. An overview of all the forces acting in the rafters is given in the appendix.

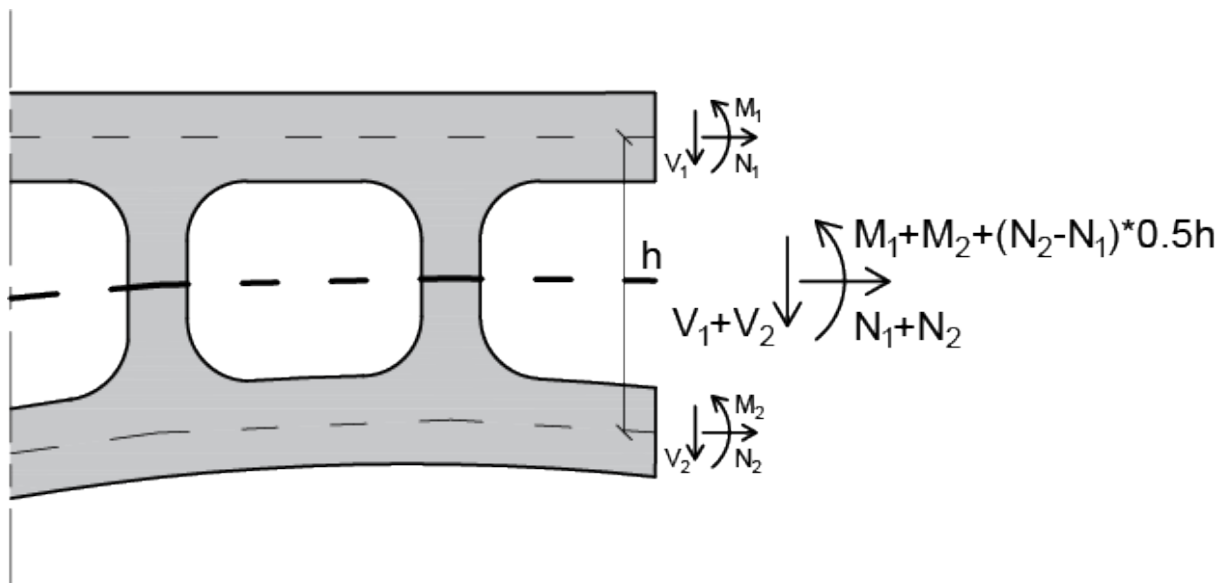


Figure 93 Translation of Vierendeel truss forces into single-beam forces.

Table 4 Maximum forces in top and bottom members connected to intermediate members B100, B101, B102, B103, and B104 (SCIA Engineer) translated into single beam forces

Maximum forces projected in single beam	
N _{beam.d.max}	-250 kN (compression)
V _{z.beam.d.max}	30 kN
M _{y.beam.d.max}	110 kNm

TENSION FORCE PERPENDICULAR TO GRAIN PEAK

Two verification methods will be used to check the tension forces perpendicular to the grain at the corners of a beam's opening, which result from a combination of shear forces and moment forces in the beam. The first method is developed by Ardalany (2013) and is partly based on fracture mechanics. It is among others recommended in the handbook of LVL producer Stora Enso. The second method is more frequently used by for example LVL producers Pollmeier and Metsä Wood. It is based on the integration of the normal- and shear forces that can't be transferred over the area of the hole and to compare them with the resistance of a specific crack length area right next to the hole.

Approach Of Ardalany

Ardalany (2013) uses a method to determine the tensile force perpendicular to the grain ($F_{t,90}$) by calculating the resistance of the entire beam against shear and moment forces. To determine the contribution of shear only, he created a structural model with the assumptions that after cracking the beam can be modeled as a beam on an elastic foundation from which both parts are connected by Winkler springs (Figure 94). The model only holds if the upper beam is much smaller and therefore has a much lower stiffness. The Winkler springs work as the resistance of LVL against cracking which is defined by both the tensile strength perpendicular to the grain and the fracture energy. The stress just ahead of the crack tip can then be captured in an equation to determine the shear force resistance which for the given geometry and material results in a critical tensile force perpendicular to the grain of $F_{t,cr,90} = 32.08$ kN per corner, and a total critical shear force for the complete beam's cross-section of **$F_{t,cr,90} = 64.16$ kN**.

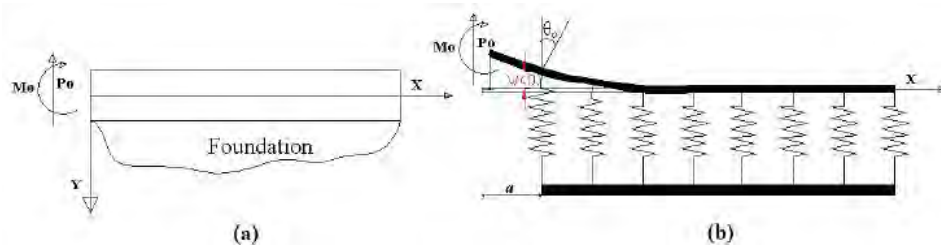


Figure 94 Crack model of Ardalany (2012) consisting out of a beam on an elastic foundation connected with Winkler springs.

To determine the contribution of a pure bending moment, Ardalany makes a difference between a local moment in the upper section of the beam on an elastic foundation and a global moment that will lead to axial stress in the part of the beam above the opening. With the given formulas the local moments are estimated on **$M_{t,cr,90} = 14.07$ kNm** per corner. Using elastic finite element analysis on LVL beams Ardalany schematizes the distribution of stresses due to global bending moments for a beam with a circular hole according to Figure 95. The critical moment for the entire beam yields **$M_{t,cr,90} = 875.92$ kNm**. The unity check for tensile stresses perpendicular to the grain according to Ardalany yield **$UC = 0.48$** from which the moment force contribution is less than 5%.

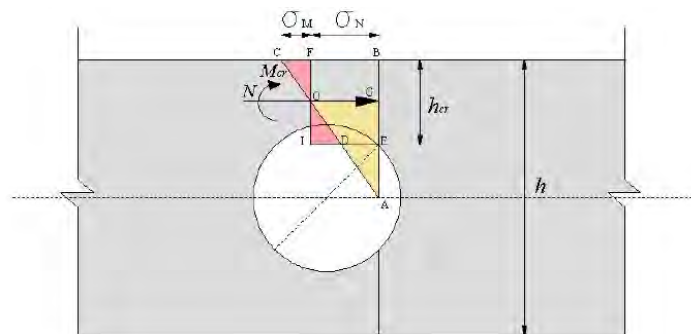


Figure 95 Stress distribution due to global bending moment acting on the entire cross-section.

Integrating Of Stresses Vs. Crack Length Area

For the second method, integration of shear- and moment stresses is used to arrive at a resulting tensile force ($F_{t,90} = F_{t,v} + F_{t,M}$), which is then compared with the tensile force capacity (resistance $R_{t,90}$). The shear force component $F_{t,v}$ is derived through integration over the hatched area in Figure 96 (Blaß and Sandhaas, 2017), which yields for the specified forces and design of Notre Dame $F_{t,v} = 7.80 \text{ kN}$. The moment force component, $F_{t,M}$, used by many codes and handbooks was empirically derived by Kolb and Eppe (1985), which for de design yields $F_{t,M} = 2.93 \text{ kN}$.

The tensile force resistance $R_{t,90}$ is derived by multiplying the resistant area ($R_{t,90,A} = 0.5 * b * l_{t,90}$) with the material strength perpendicular to the grain ($f_{t,90}$). It is then corrected with a Weibull-type size-dependent degradation in strength for LVL with cross-plyies ($k_{t,90}$). Mainly the load distribution length $l_{t,90}$ requires some extra attention (Figure 97). The formulas to determine $l_{t,90}$ differ between different codes and authors and show many discrepancies with the experimentally obtained crack lengths, which are generally shorter (Aicher and Tapia, 2018). For Notre Dame's rafters, the equations yield a length ranging from 850 mm to 950 mm. The limited inter-opening space of the rafters though limits the load distribution length to 200 mm. For two closely located openings, it's assumed that only half of that length can be used per opening, yielding $l_{t,90} = 100 \text{ mm}$ and a tensile force resistance of $R_{t,90} = 22.07 \text{ kN}$, with a resulting unity check of $UC = 0.47$.

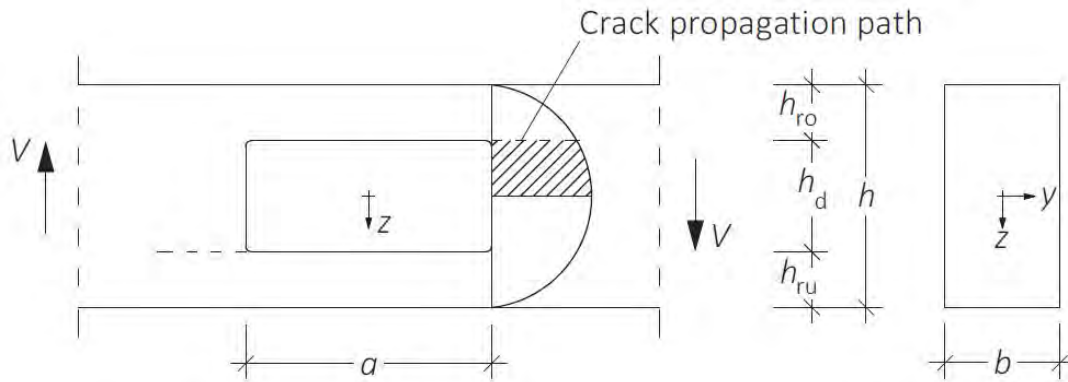


Figure 96 Integration area of shear forces (hatched) for a rectangular opening in a beam.

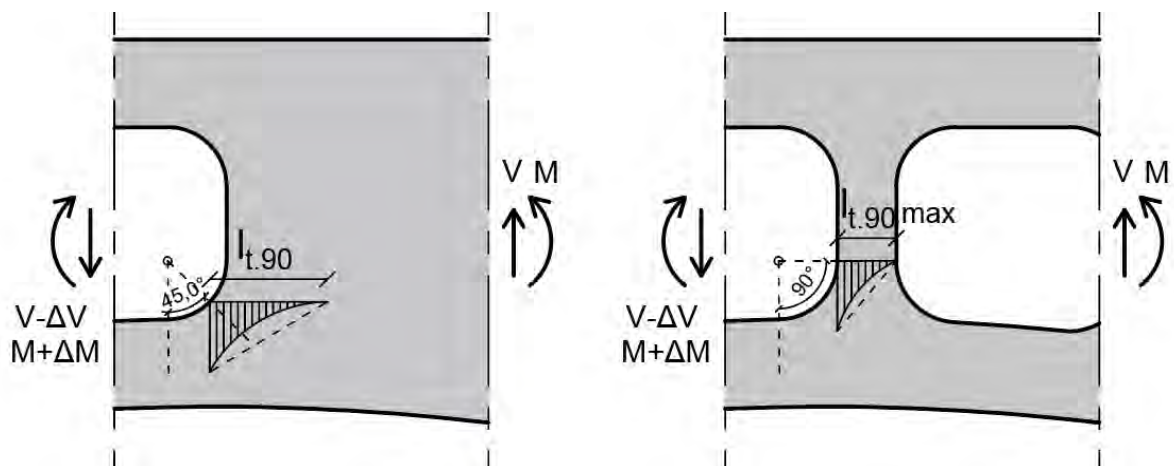


Figure 97 Load distribution length in case of big inter-opening spacing (left) and with small inter-opening spacing (right).

Ardalany (2012) also proposed an alternative verification method to predict the tensile load in possible reinforcement around holes in LVL beams (Ardalany, 2012). Although the eventual purpose was to apply the theory on steel reinforcement bars around the opening, the assumptions and derivations can be used to determine the resulting tensile force $F_{t,90}$.

The equations are derived using a hypothetical truss model around a circular opening that diverts the shear and moment forces around the opening (Figure 98). The shear forces that can't be transferred due to the hole are integrated over the opening area and then converted into a tensile force perpendicular to the grain by using the equilibrium of node A. To calculate the bending moment's contribution to the perpendicular tensile force, again node A is used to derive equilibrium equations. The assumed linear stress distribution is shown in Figure 98.

The resulting tensile forces due to shear and moment forces are respectively $F_{t,V} = 7.53 \text{ kN}$ and $F_{t,M} = 7.88 \text{ kN}$. Compare to the tensile force resistance $R_{t,90} = 22.07 \text{ kN}$ the resulting unity check yields $UC = 0.70$.

The resulting unity checks are provided in Table 5.

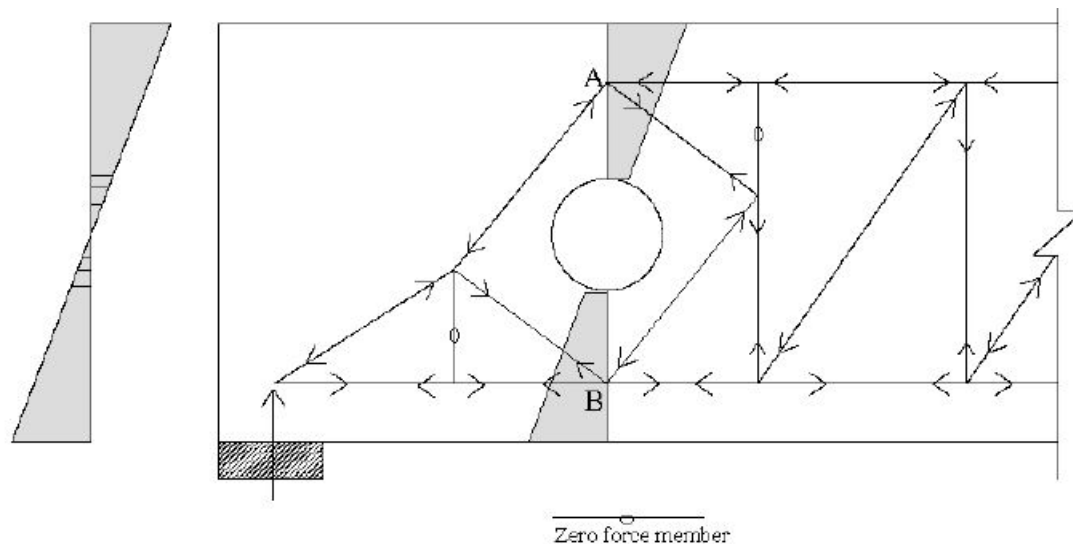


Figure 98 Truss model around a circular opening, used to derive equilibrium equations in node A to determine the perpendicular tensile forces.

Table 5 Tension perpendicular to the grain stress peak in the corner of rectangular opening according to different verification methods.

TENSILE STRESS PERPENDICULAR TO GRAIN PEAK	UC _{shear contrib.}	UC _{moment contrib.}	UC _{total}
1.) Approach of Ardalany (Store Enso)	0.47	0.01	0.48
2.2) Aicher/DIN 1052 (2008)	0.35	0.12	0.47
2.2) Ardalany (2012)	0.34	0.36	0.70

SHEAR STRESS PEAK

The first step to determine the shear stress peaks in the opening of a large rectangular opening is to define the maximum shear stress acting in the part of the beam above and below the opening. The maximum shear force location is taken from the computer model (member B87). For a shear force of $V_{\text{beam.d.max}} = 28 \text{ kN}$, this results in shear stress of $\tau = 1.75 \text{ N/mm}^2$. For the verifications, the geometry of that part of the rafter is taken into consideration. Next, two strategies can be identified to take the shear stress peak into account: decreasing the resistance of the material or increasing the shear stress by a correction factor.

Adapting Material Resistance

The former is sometimes based on an end-notched beam model (Figure 99). Although the end-notched beam calculations are described in the Eurocode and advised by LVL producers (Pollmeier and Stora Enso), the conversion of a beam with an opening into an end-notched beam didn't make it further than the Eurocode's draft since experimental results showed the method was non-conservative (Aicher et al., 2002). Pollmeier prescribes an end-notched beam approach including LEFM aspects. A geometrical correction factor is applied to lower the material strength. For LVL panels without cross plies the geometry of Notre Dame's rafters would result in a factor of $k_v = 0.15$, yielding a unity check of **UC = 1.75**. The handbook describes that due to the presence of cross plies this correction factor doesn't have to be applied on Baubuche Q, which yields **UC = 0.28**. The Swiss code applies the end-notched beam approach by correcting the shear force with a geometry-dependent factor $k_{\text{red}} = 0.14$, yielding a unity check of **UC = 2.04**.

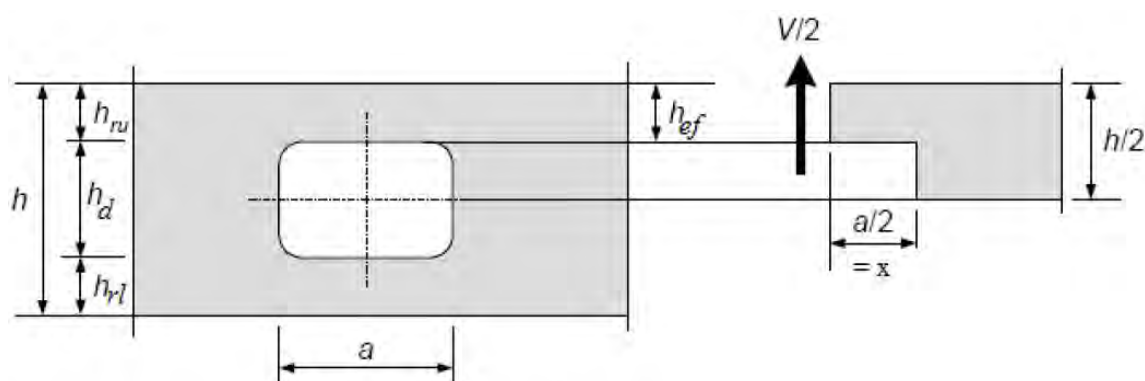


Figure 99 Transformation from a beam with a rectangular opening into a beam with end-notch (Swiss code).

Adapting Shear Stress

The option of an increased shear stress factor is preferred by LVL suppliers Stora Enso and Metsä Wood, who recommend increasing the shear stress with k_r , depending on the height, width, and shape of the opening (Figure 90, right). The $k_r = 2.41$ factor is highly sensitive for the shape of the corner, for which it can be generally stated that smoother holes lead to lower stress peaks. The unity check yields: **UC = 0.68**.

The results of the unity checks for the different verification methods are given in Table 6. The unity checks' validity is discussed in chapter 10.

Table 6 Shear stress peak in the corner of rectangular opening according to different verification methods.

SHEAR STRESS PEAK		UC
1.) Pollmeier for end-notched beams		0.28
2.) Swiss code		2.04
3.) Store Enso and Metsä Wood		0.68

RAFTER AS A VIERENDEEL TRUSS

The following section will treat the different parts of Notre Dame's rafters as individual members with rigid connections, better known as a Vierendeel truss. Literature suggests that the area of a timber beam near an opening can be treated as individual structural members, which is especially valid for beams with large openings (Chan et al., 1974; Pirzada et al. 2008). Compared to a solid beam, the openings in a Vierendeel truss cause a relatively small reduction in bending resistance, but the shear transfer gets significantly disturbed. Shear transfer across openings results in specific behavior which is often described as Vierendeel or 'four corner' bending. It is expected that the truss has comparable failure mechanisms as beams with large openings, where shear forces are also frequently governing the design.

This section focuses specifically on the stresses acting in the nodes between the individual members. The regular checks for the field forces in the structural members are not expected to govern the design as moment forces reach their maximum value at the connections. For the members on the bottom of the truss, it will be important to include the direction of the stresses relative to the wood grain direction - which in the worst-case scenario reaches a 26 ° angle -. The Vierendeel truss will be analyzed using a *conventional stress analysis* (CSA) and a 'beam theory fracture mechanics' approach. The former is already presented in the main body of the thesis. The latter method, developed by Danielsson and Gustafsson (2015), is not commonly used but has the potential to derive accurate stress verifications which also include normal forces.

'Beam Theory Fracture Mechanics' Approach

This design approach by Danielsson and Gustafsson (2015) is based on a beam model according to Timoshenko beam theory which is applied in combination with LEFM for which both failure mode 1 (opening) and 2 (sliding) are taken into account. The approach is general in the sense that it allows for any cross-sectional forces to be included and that the authors claim that the accuracy of the method is not limited by the opening size. The location of the check is assumed to be known: in the corner of the opening at an angle of 45° degrees to the central axis (Figure 89).

The structural model used is displayed in Figure 100 (left) assumes that the two sides of the hole remain plane at loading. Distance L is assumed slightly longer than the opening according to $L = a + 0.5 \cdot h_1$. The authors provide equations to determine the forces at the crack location (Figure 100, right). Due to the complex geometry of the rafters, it is decided to use the ULS forces of the members between the top and bottom part of the rafter from the SCIA Engineer model (Figure 101, with the governing members, marked red) ($N = -10.56 \text{ kN}$, $V_z = 12.82 \text{ kN}$, $M_y = 5.88 \text{ kNm}$).

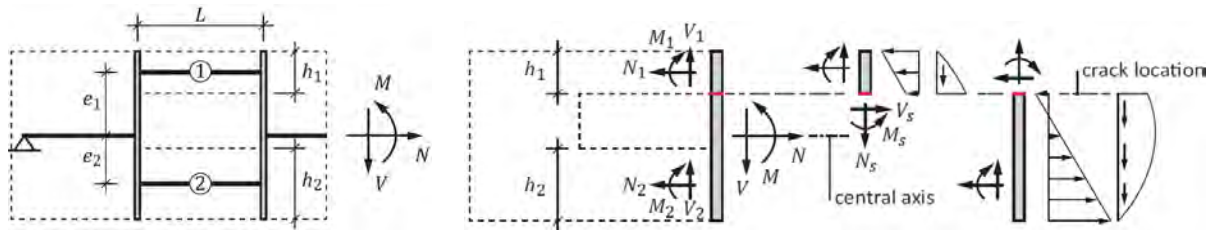


Figure 100 Left: structural model around a rectangular opening. Right: possible translation of global forces (N, V, M) into forces acting on the crack (N_s, V_s, M_s) (Danielsson and Gustafsson, 2015).

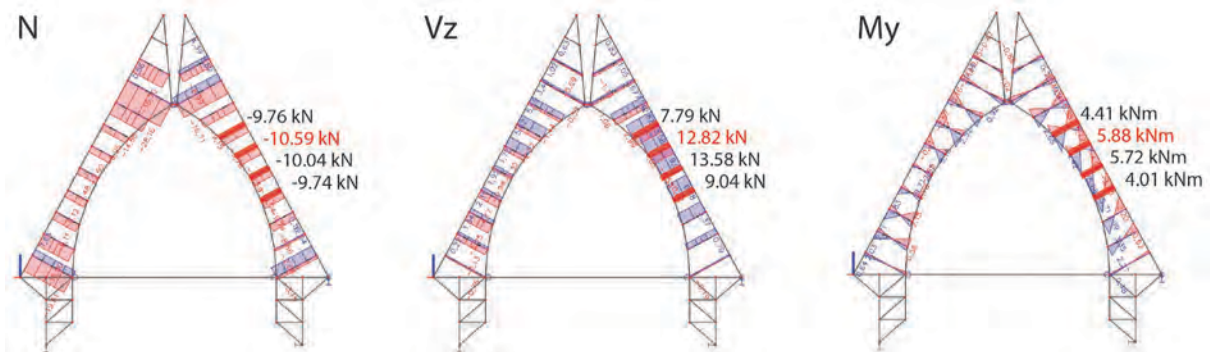


Figure 101 Member forces of the connecting members between top and bottom part rafter. From left to right: axial force N , shear force V_z , and moment force M_y . Red marked the governing member which is assessed.

The forces acting on the crack are then related to the relative displacements u, v , and relative rotation θ through a compliance matrix C which can be determined using kinematic assumptions according to Timoshenko beam theory (Figure 102). The total work W to close the crack can then be determined and compared with the critical energy release rate for both failure modes 1 and 2. The interaction criterion resulted in a **UC = 0.78**.

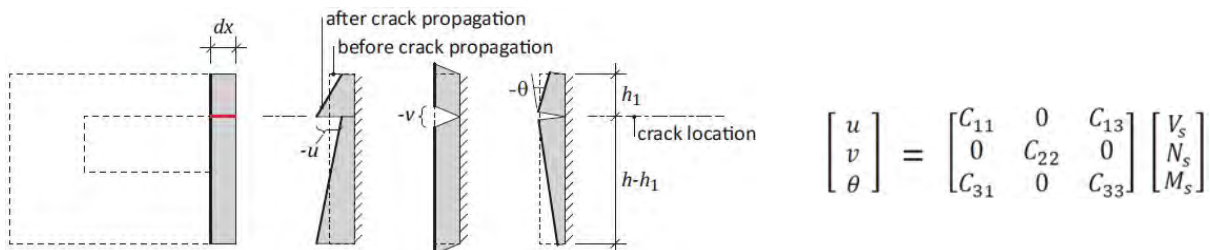


Figure 102 Relative displacements and the resulting system of equations.

VERIFICATION OF TENSION STRESS PEAK PERPENDICULAR TO GRAIN

total shear and moment $V_{d,w} := 30 \text{ kN}$ $M_{d,y,w} := 110 \text{ kN} \cdot \text{m}$ $f_{Q,t,90,d} := k_{mod} \cdot \frac{f_{Q,t,90,k}}{\gamma_m} = 6.769 \frac{\text{N}}{\text{mm}^2}$

1.0) approach of Ardalany

1.1) DUE TO SHEAR

fracture energy in mode 1 (opening)

$$G_{I,f} := 0.635 \frac{\text{N}}{\text{mm}} \quad G_{I,f,d} := \frac{k_{mod} \cdot G_{I,f}}{\gamma_m} = 0.537 \frac{\text{N}}{\text{mm}}$$

shear modulus

$$G_{mean} := 820 \frac{\text{N}}{\text{mm}^2} \quad E_{0,mean} := 12800 \frac{\text{N}}{\text{mm}^2}$$

shear area

$$h_{cr} := 300 \text{ mm} \quad A_{cr} := b \cdot h_{cr} = (2.4 \cdot 10^4) \text{ mm}^2$$

$$W_{cr} := \frac{b \cdot h_{cr}^2}{6} = (1.2 \cdot 10^6) \text{ mm}^3 \quad I_{cr} := \frac{b \cdot h_{cr}^3}{12} = (1.8 \cdot 10^8) \text{ mm}^4$$

spring stiffness beam on elastic foundation

$$K := \frac{1}{2} \cdot \frac{f_{Q,t,90,d}^2}{G_{I,f,d}} = (4.264 \cdot 10^{10}) \frac{\text{kg}}{\text{m}^2 \cdot \text{s}^2}$$

parameter beam on elastic foundation

$$\lambda := \sqrt[4]{\frac{K \cdot b}{4 \cdot E_{0,mean} \cdot I_{cr}}} = 4.386 \frac{1}{\text{m}}$$

tension resistance of beam for pure shear in one corner

$$F_{cr,t,90,V} := \frac{b \cdot f_{Q,t,90,d}}{\sqrt{4 \cdot \lambda^2 + \frac{6}{5} \cdot \frac{K \cdot b}{G_{mean} \cdot A_{cr}}}} = 32.08 \text{ kN}$$

total shear resistance of the hole

$$F_{cr,V} := 2 \cdot F_{cr,t,90,V} = 64.16 \text{ kN}$$

UC for only shear

$$\frac{V_{d,w}}{F_{cr,V}} = 0.468$$

1.2) DUE TO BENDING

local moment in upper part crack at crack initiation

$$M_{cr,t,90} := \frac{b}{2 \cdot \lambda^2} \cdot f_{Q,t,90,d} = 14.074 \text{ kN} \cdot \text{m} \quad \sigma_{cr,t,90,M} := \frac{M_{cr,t,90}}{W_{cr}} = 11.728 \frac{\text{N}}{\text{mm}^2}$$

critical moment for rectangular holes

$$M_{cr,M} := 2 \cdot M_{cr,t,90} + b \cdot \sigma_{cr,t,90,M} \cdot (h_{cr} + h_d) \cdot (h - h_{cr}) = 874.924 \text{ kN} \cdot \text{m}$$

UC of shear and bending

$$\left(\frac{V_{d,w}}{F_{cr,V}} \right) + \left(\frac{M_{d,y,w}}{M_{cr,M}} \right)^2 = 0.483$$

2.) integration of forces around the opening

2.1) ACTION

with for round holes $\kappa := 0.706$

tension force perpendicular to grain (Aicher/DIN 1052(2008))

$$F_{t,v} := \frac{V_{d,w} \cdot \kappa \cdot h_d}{4 \cdot h} \cdot \left(3 - \frac{(\kappa \cdot h_d)^2}{h^2} \right) = 7.889 \text{ kN}$$

$$F_{t,M} := 0.008 \cdot \frac{M_{d,y,w}}{h_r} = 2.933 \text{ kN}$$

tension force perpendicular to grain (Ardalany, 2012)

$$F_{t,v} := \frac{\sqrt{2} \cdot V_{d,w} \cdot h_d \cdot (3 \cdot h^2 - h_d^2)}{8 \cdot h^3} = 7.527 \text{ kN}$$

$$F_{t,M} := \left(\frac{3}{4} \right) \cdot \frac{M_{d,y,w} \cdot h_d^3 \cdot (h_d + h)}{h^3 \cdot (h \cdot h_d + h^2 + h_d^2)} = 7.879 \text{ kN}$$

$$F_{t,90,d,Aich} := \frac{\sqrt{2} \cdot V_{d,w} \cdot h_d \cdot (3 \cdot h^2 - h_d^2)}{8 \cdot h^3} + 0.008 \cdot \frac{M_{d,y,w}}{h_r} = 10.461 \text{ kN}$$

$$F_{t,90,d,Ard} := \frac{\sqrt{2} \cdot V_{d,w} \cdot h_d \cdot (3 \cdot h^2 - h_d^2)}{8 \cdot h^3} + \left(\frac{3}{4} \right) \cdot \frac{M_{d,y,w} \cdot h_d^3 \cdot (h_d + h)}{h^3 \cdot (h \cdot h_d + h^2 + h_d^2)} = 15.406 \text{ kN}$$

2.2) RESISTANCE

total triangular resistance area:

$$R_{A,t,90} := 0.5 \cdot l_{t,90} \cdot b \cdot k_{t,90}$$

shape parameter

size effect for LVL with cross plies (EN-1995-1-1 recommends 5)

$$m_s := 5 \quad k_{t,90} := \min \left(1, \left(\frac{450 \text{ mm}}{h} \right)^{\frac{1}{m_s}} \right) = 0.815$$

load distribution length:

- Pollmeier
- Store Enso and Metsä Wood
- Aicher
- but due to small inter-hole distance

$$l_{t,90} := 0.353 \cdot h_d + 0.58 \cdot h = 954.45 \text{ mm}$$

$$l_{t,90} := 0.5 \cdot (h_d + h) = 950 \text{ mm}$$

$$l_{t,90} := 0.5 \cdot (\kappa \cdot h_d + h) = 854.45 \text{ mm}$$

$$l_{t,90} := \frac{z}{2} = 100 \text{ mm}$$

resistance area

$$R_{A,t,90} := 0.5 \cdot k_{t,90} \cdot l_{t,90} \cdot b = (3.261 \cdot 10^3) \text{ mm}^2$$

resistance force

$$R_{t,90} := R_{A,t,90} \cdot f_{t,90,d} = 22.073 \text{ kN}$$

mean tension stress perpendicular

$$\sigma_{t,90,d} := \frac{F_{t,90,d}}{R_{A,t,90}} = 4.164 \frac{\text{N}}{\text{mm}^2}$$

UC

$$\frac{F_{t,90,d,Aich}}{R_{t,90}} = 0.474$$

$$\frac{F_{t,90,d,Ard}}{R_{t,90}} = 0.698$$

HEAR STRESS PEAK IN CORNER OF OPENING ACCORDING TO DIFFERENT CODES/SUPPLIERS

geomtry beam where highest shear peaks are expected:

$$\begin{aligned} b &:= 80 \text{ mm} & h_{ro} &:= 300 \text{ mm} \\ h &:= 2000 \text{ mm} & h_{ru} &:= 300 \text{ mm} \\ h_d &:= 1400 \text{ mm} & l_z &:= 200 \text{ mm} \\ a &:= 800 \text{ mm} \end{aligned}$$

max shear force in rafter

$$V_{d,w,ru} := 28 \text{ kN} \quad \tau_{d,ru} := \left(\frac{3}{2} \right) \frac{V_{d,w,ru}}{b \cdot h_{ru}} = 1.75 \frac{\text{N}}{\text{mm}^2}$$

Pollmeier adds $k_{h,v}$ for Baubuche-Q

$$k_{h,v} := \left(\frac{600 \text{ mm}}{0.5 \cdot h} \right)^{0.13} = 0.936 \quad f_{Q,v,k} := 7.8 \frac{\text{N}}{\text{mm}^2} \quad f_{Q,v,d} := k_{h,v} \cdot k_{mod} \cdot \frac{f_{Q,v,k}}{\gamma_m} = 6.176 \frac{\text{N}}{\text{mm}^2}$$

1.) Pollmeier for end-notched beams, based on LEFM

for LVL

notch inclination, for rectangle = 0

height of beam above and below opening

$$\begin{aligned} k_n &:= 4.5 \\ i &:= 0 \\ h_{ef} &:= h_{ru} = 300 \text{ mm} \end{aligned}$$

half of beam height

$$h_{0.5} := \frac{h}{2} = (1 \cdot 10^3) \text{ mm}$$

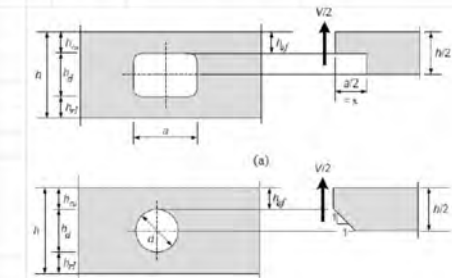
half of opening width

$$x := \frac{a}{2} = 400 \text{ mm}$$

proportion of notch/beam

$$\alpha := \frac{h_{ef}}{\left(\frac{h}{2} \right)} = 0.3$$

$$k_v := \min \left(1, \frac{4.5}{\sqrt{1000} \cdot \left(\sqrt{0.3 \cdot (1 - 0.3)} + 0.8 \cdot \frac{400}{1000} \cdot \sqrt{\frac{1}{0.3} - 0.286^2} \right)} \right) = 0.148$$



$$k_v = \min \left(\frac{1.0}{k_n \left(1 + \frac{1.1 \cdot i^{1.5}}{\sqrt{h}} \right)}, \frac{1}{\sqrt{h} \left[\sqrt{\alpha(1 - \alpha)} + 0.8 \frac{x}{h} \sqrt{\frac{1}{\alpha} - \alpha^2} \right]} \right) \quad (6.62)$$

UC

S without cross-plyes

$$\frac{\tau_{d,ru}}{k_v \cdot f_{S,v,d}} = 1.745$$

Q with cross-plyes

$$\frac{\tau_{d,ru}}{f_{Q,v,d}} = 0.283$$

2.) Swiss code uses strategy:

geomtrical properties

value defined by code

opening height

$$\begin{aligned} h_{ef} &:= h_{ru} = 300 \text{ mm} \\ \Delta h_0 &:= 45 \text{ mm} \\ h_d &:= (1.4 \cdot 10^3) \text{ mm} \end{aligned}$$

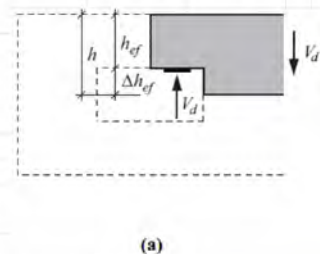
$$\Delta h_{ef} := \frac{h_d}{2} = 700 \text{ mm}$$

$k_{red} = 1$ for corner compression perp. to grain

$$k_{red} := \sqrt{\frac{h_{ef}}{0.5 \cdot h} \cdot \left(\frac{\Delta h_0}{\Delta h_{ef}} \right)} = 0.139$$

UC

$$\frac{\tau_{d,ru}}{k_{red} \cdot f_{Q,v,d}} = 2.04$$



3.) Store Enso and Metsä Wood & Aicher (2018)

stress concentration factor

$$k_r := 1.85 \cdot \left(1 + \frac{a}{h} \right) \cdot \left(\frac{h_d}{h} \right)^{0.2} = 2.412$$

UC

$$\frac{k_r \cdot \tau_{d,ru}}{f_{Q,v,d}} = 0.683$$

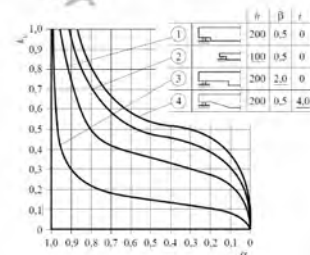


Figure D5-6 Factor k_r depending on x/h for solid timber with various h/b and i (STEP 1995 Article B5)

Wind loads according to EN 1991-1-4:2005 + wind loads on Notre Dame.

FLAT ROOFS $\alpha < 5^\circ$

Load case with overpressure $c_{pi} = +0.2$

Zone F: $c_{f,pe} (-1.8) - c_{pi} (+0.2) = -2.0$

Zone G: $c_{g,pe} (-1.2) - c_{pi} (+0.2) = -1.4$

Zone H: $c_{h,pe} (-0.7) - c_{pi} (+0.2) = -0.9$

Zone I: $c_{i,pe} (+0.2) - c_{pi} (+0.2) = 0.0$

FLAT ROOFS $\alpha < 5^\circ$

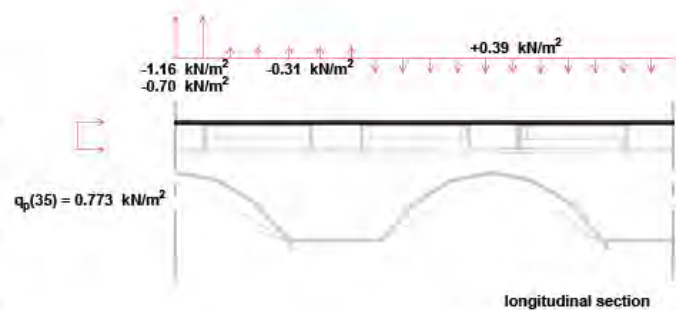
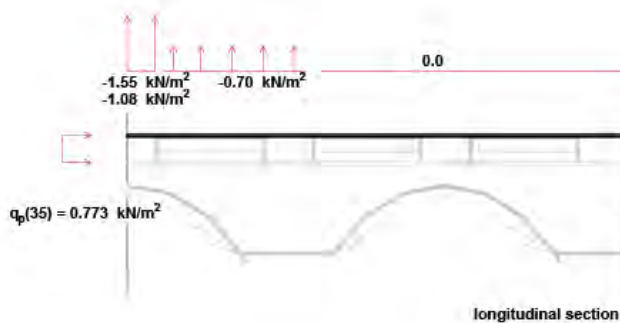
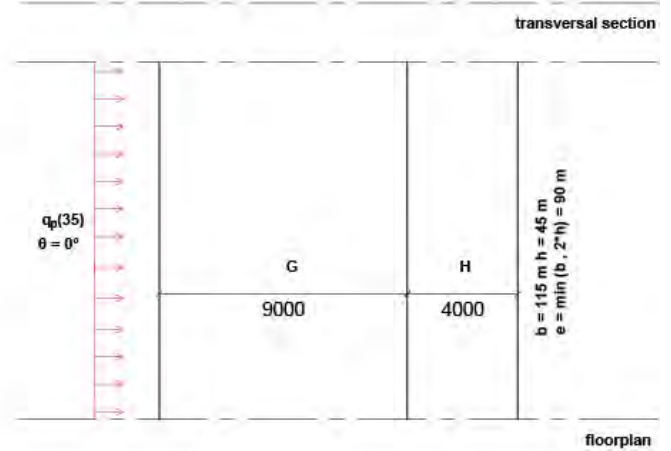
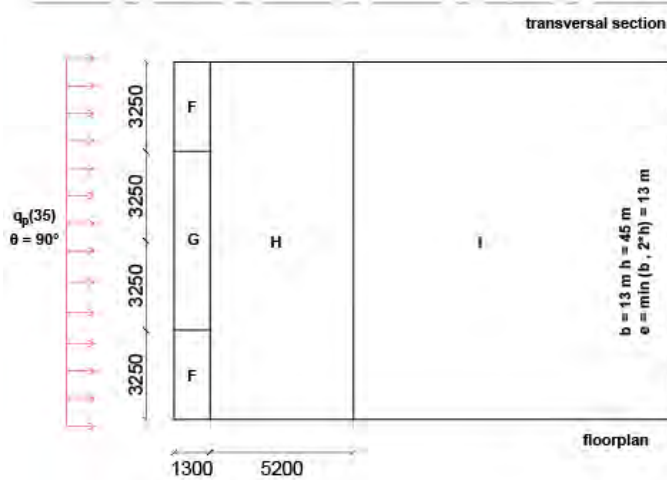
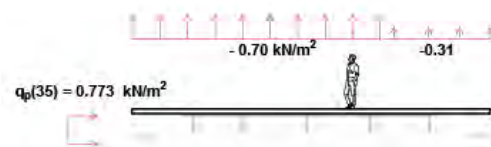
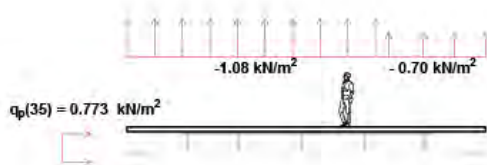
Load case with underpressure $c_{pi} = -0.3$

Zone F: $c_{f,pe} (-1.8) - c_{pi} (-0.3) = -1.5$

Zone G: $c_{g,pe} (-1.2) - c_{pi} (-0.3) = -0.9$

Zone H: $c_{h,pe} (-0.7) - c_{pi} (-0.3) = -0.4$

Zone I: $c_{i,pe} (+0.2) - c_{pi} (-0.3) = 0.5$



GABLE ROOFS $\alpha = 60^\circ$

Load case with overpressure $c_{pi} = +0.2$

Zone F: $c_{f,pe} (+0.7) - c_{pi} (+0.2) = 0.5$

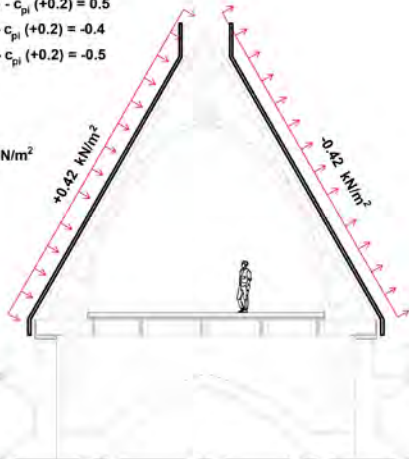
Zone G: $c_{g,pe} (+0.7) - c_{pi} (+0.2) = 0.5$

Zone H: $c_{h,pe} (+0.7) - c_{pi} (+0.2) = 0.5$

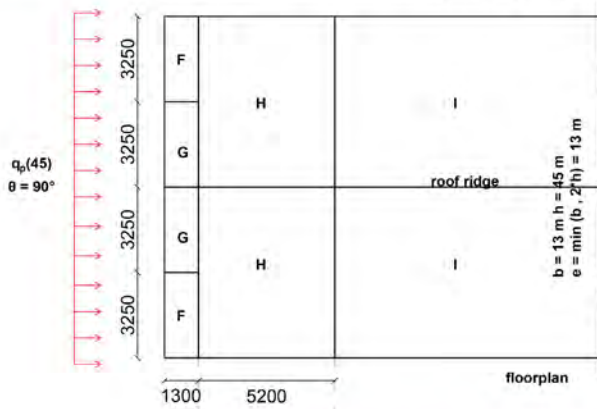
Zone I: $c_{i,pe} (-0.2) - c_{pi} (+0.2) = -0.4$

Zone j: $c_{j,pe} (-0.3) - c_{pi} (+0.2) = -0.5$

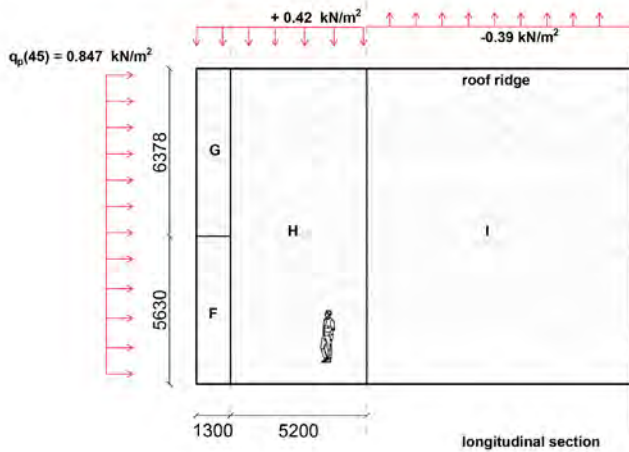
$q_p(45) = 0.847 \text{ kN/m}^2$



transversal section



floorplan



longitudinal section

GABLE ROOFS $\alpha = 60^\circ$

Load case with underpressure $c_{pi} = -0.3$

Zone F: $c_{f,pe} (+0.7) - c_{pi} (-0.3) = 1.0$

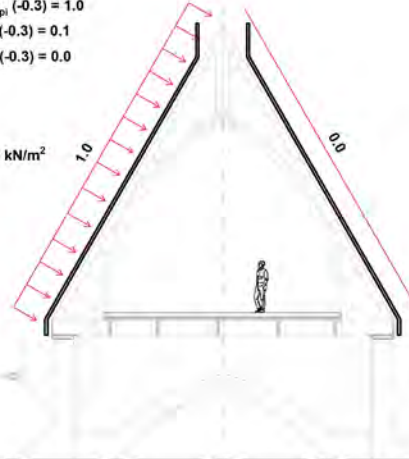
Zone G: $c_{g,pe} (+0.7) - c_{pi} (-0.3) = 1.0$

Zone H: $c_{h,pe} (+0.7) - c_{pi} (-0.3) = 1.0$

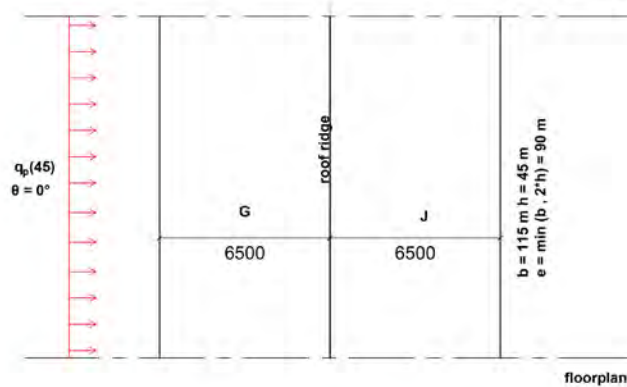
Zone I: $c_{i,pe} (-0.2) - c_{pi} (-0.3) = 0.1$

Zone j: $c_{j,pe} (-0.3) - c_{pi} (-0.3) = 0.0$

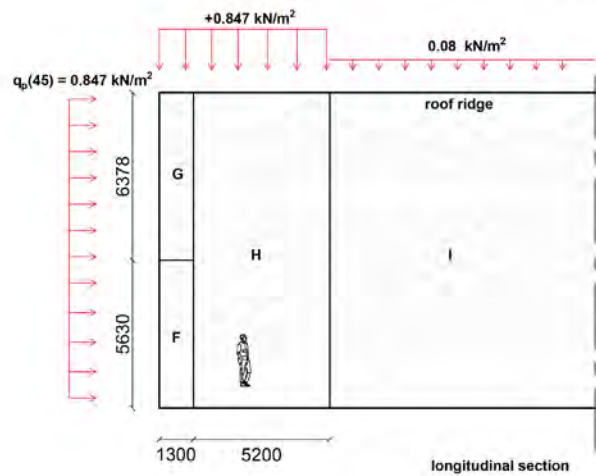
$q_p(45) = 0.847 \text{ kN/m}^2$



transversal section



floorplan



longitudinal section

4 WIND VELOCITY AND VELOCITY PRESSURE, EN 1991-1-4:2005

4.2 BASIC VALUES

basic wind velocity in Paris:
(assumed region III in NEN-EN 1991-1-4:2011)
direction factor
season factor
basic wind velocity from Expression (4.1)

$$v_{b,0} := 24.5 \frac{m}{s}$$

$$c_{dir} := 1.0$$

$$c_{season} := 1.0$$

$$v_b := v_{b,0} \cdot c_{dir} \cdot c_{season} = 24.5 \frac{m}{s}$$

4.3 MEAN WIND

roughness length area II
minimum height according to Table 4.1 area IV
maximum height
orography factor (4.3.3)
roughness length area IV

$$z_{0,2} := 0.05 \text{ m}$$

$$z_{min} := 10 \text{ m}$$

$$z_{max} := 200 \text{ m}$$

$$c_0 := 1.0$$

$$z_0 := 1 \text{ m}$$

$$k_r := 0.19 \cdot \left(\frac{z_0}{z_{0,2}} \right)^{0.07} = 0.234$$

reference heights
 $b < h \leq 2b$

flat roofs (7.2.3)
FAST FIX
 $z_{35} := 35 \text{ m}$

duopitch roofs (7.2.5)
FIELD FACTORY
 $z_{45} := 45 \text{ m}$

terrain roughness factor (4.4)

$$c_{r35} := k_r \cdot \ln \left(\frac{z_{35}}{z_0} \right) = 0.833$$

$$c_{r45} := k_r \cdot \ln \left(\frac{z_{45}}{z_0} \right) = 0.892$$

mean wind velocity from Expression (4.3)

$$v_{m35} := c_{r35} \cdot c_0 \cdot v_b = 20.411 \frac{m}{s}$$

$$v_{m45} := c_{r45} \cdot c_0 \cdot v_b = 21.854 \frac{m}{s}$$

4.4 WIND TURBULENCE

turbulence factor (recommended 1.0)
turbulence intensity (4.7)

$$k_l := 1.0$$

$$l_{v35} := \frac{k_l}{c_0 \cdot \ln \left(\frac{z_{35}}{z_0} \right)} = 0.281$$

$$l_{v45} := \frac{k_l}{c_0 \cdot \ln \left(\frac{z_{45}}{z_0} \right)} = 0.263$$

4.5 PEAK VELOCITY PRESSURE

ρ air density according to NA: 1.25 kg/m³)

$$\rho := 1.25 \frac{kg}{m^3}$$

peak velocity pressure at height (4.8)
 $z = 35$ meters for the **FAST FIX**

$$q_{p,35} := (1 + 7 \cdot l_{v35}) \cdot 0.5 \cdot \rho \cdot v_{m35}^2 = 0.773 \frac{kN}{m^2}$$

peak velocity pressure at height (4.8)
 $z = 45$ meters for the **FIELD FACTORY**

$$q_{p,45} := (1 + 7 \cdot l_{v45}) \cdot 0.5 \cdot \rho \cdot v_{m45}^2 = 0.847 \frac{kN}{m^2}$$

5 WIND ACTIONS

$$c_{pi} := 1$$

$$c_{pe} := 1$$

5.2 WIND PRESSURE ON SURFACES

wind pressure external surfaces
at reference height:

FAST FIX

$$w_{e.35} := q_{p.35} \cdot c_{pe}$$

FIELD FACTORY

$$w_{e.45} := q_{p.45} \cdot c_{pe}$$

$c_{pe.10}$ for global structure
 $c_{pe.1}$ used for small element with surface 1m²

wind pressure on internal surfaces:

$$w_{i.45} := c_{pi} \cdot q_{p.35}$$

$$w_{i.45} := c_{pi} \cdot q_{p.45}$$

7 PRESSURE AND FORCE COEFFICIENTS

7.2.3 FLAT ROOFS

for flat roof use: (table 7.2)
with parapets assumed $h_p/h=0.025$

$$c_{Fpe.10} := -1.6$$

$$c_{Fpe.1} := -2.2$$

$$c_{Gpe.10} := -1.1$$

$$c_{Gpe.1} := -1.8$$

$$c_{Hpe.10} := -0.7$$

$$c_{Hpe.1} := -1.2$$

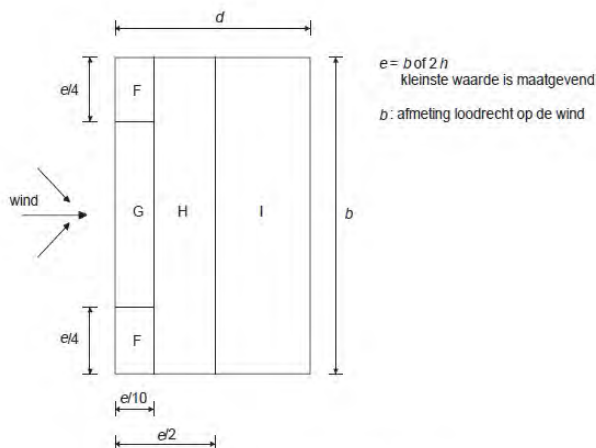
$$c_{Ipe.10} := -0.2$$

$$c_{Ipe.1} := -0.2$$

or

$$c_{Ipe.10} := 0.2$$

$$c_{Ipe.1} := 0.2$$



Figuur 7.6 — Zones bij platte daken

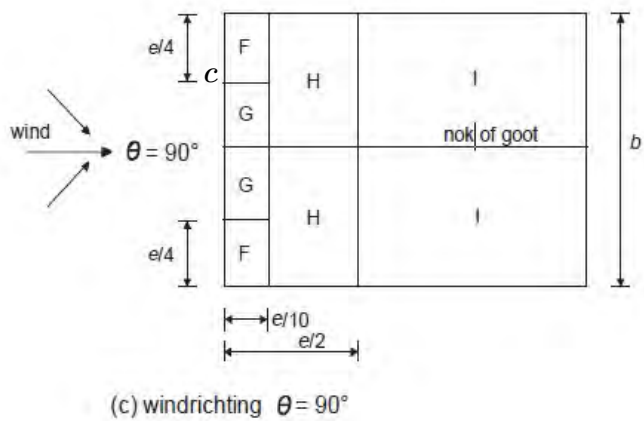
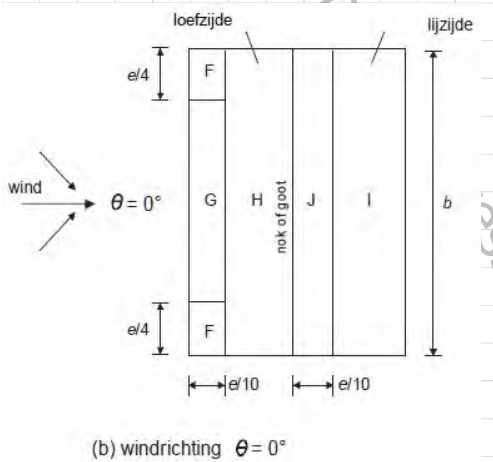
7.2.5 DUOPITCH ROOFS

total width structure
total height structure
length of zone e
depth structure in wind direction

$b := 115 \text{ m}$
 $h := 45 \text{ m}$
 $e := \min(b, 2 \cdot h) = 90 \text{ m}$
 $d := 13 \text{ m}$

external pressure coefficients for duopitch roofs
 $\alpha = 60$ degrees (table 7.4a)

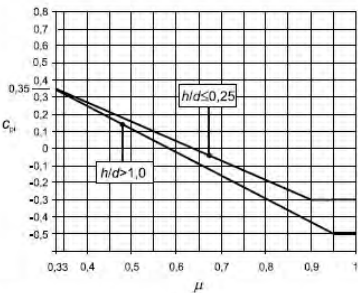
$c_{Fpe.10} := 0.7$	$c_{Fpe.1} := c_{Fpe.10}$
$c_{Gpe.10} := 0.7$	$c_{Gpe.1} := c_{Gpe.10}$
$c_{Hpe.10} := 0.7$	$c_{Hpe.1} := c_{Hpe.10}$
$c_{Ipe.10} := -0.2$	$c_{Ipe.1} := c_{Ipe.10}$
$c_{Jpe.10} := -0.3$	$c_{Jpe.1} := c_{Jpe.10}$



7.2.9 INTERNAL PRESSURE

for building without dominant face
 c_{pi} given by figure 7.13 with:

$\frac{h}{d} > 1 \quad \mu := 0.5 \quad c_{pi} := 0.12$



NOTE For values between $h/d = 0,25$ and $h/d = 1,0$ linear interpolation may be used.

Figure 7.13 — Internal pressure coefficients for uniformly distributed openings

in case c_{pi} between $-0.3 < c_{pi} < 0.2$ both underpressure
and overpressure has to be taken into account:

$c_{pi.overpressure} := 0.20 \quad c_{pi.underpressure} := -0.3$

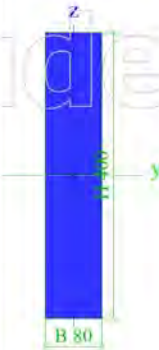
1. FAST FIX Engineering report printing

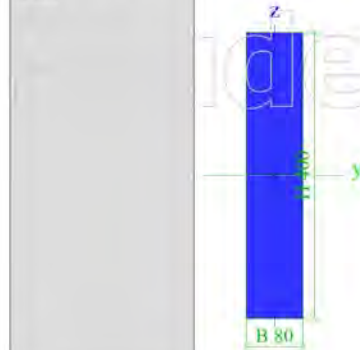
2. Materials

Timber ECS

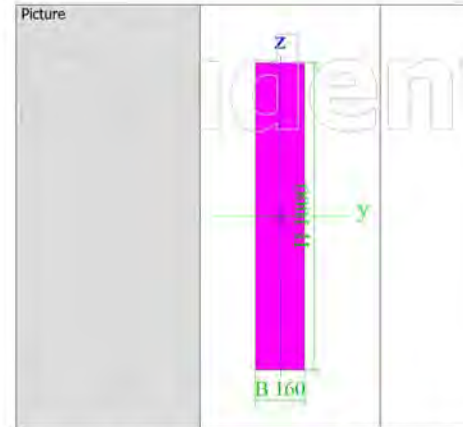
Name	Type of Timber	μ	E_{mod} [MPa]	$f_{m,k}$ [MPa]	$f_{t,0,k}$ [MPa]	$f_{t,90,k}$ [MPa]	$f_{c,0,k}$ [MPa]	$f_{c,90,k}$ [MPa]	$f_{t,k}$ [MPa]	Colour
	ρ [kg/m ³]	α [m/mK]	G_{mod} [MPa]							
Baubuche S (edge wist)	Glued, laminated 730.0	0 0.00	1.6800e+04 7.6000e+02	75.0	60.0	1.5	57.5	11.7	8.0	

3. Cross-sections

CS2		
Type	RECT	
Detailed	80; 400	
Shape type	Thick-walled	
Item material	Baubuche S (edge wist)	
Fabrication	timber	
Colour		
A [m ²]	3.2000e-02	
A _y [m ²], A _z [m ²]	2.6783e-02	2.6671e-02
A _L [m ² /m], A _D [m ² /m]	9.6000e-01	9.6000e-01
c _{y,UCS} [mm], c _{z,UCS} [mm]	40	200
α [deg]	0.00	
I _y [m ⁴], I _z [m ⁴]	4.2667e-04	1.7067e-05
I _y [mm], I _z [mm]	115	23
W _{el,y} [m ³], W _{el,z} [m ³]	2.1333e-03	4.2667e-04
W _{pl,y} [m ³], W _{pl,z} [m ³]	3.2681e-03	6.5362e-04
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	1.88e+05	1.88e+05
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	3.76e+04	3.76e+04
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	5.9501e-05	1.8901e-07
β_y [mm], β_z [mm]	0	0
Picture		



CS3		
Type	RECT	
Detailed	160; 1000	
Shape type	Thick-walled	
Item material	Baubuche S (edge wist)	
Fabrication	timber	
Colour		
A [m ²]	1.6000e-01	
A _y [m ²], A _z [m ²]	1.3426e-01	1.3336e-01
A _L [m ² /m], A _D [m ² /m]	2.3200e+00	2.3200e+00
c _{y,UCS} [mm], c _{z,UCS} [mm]	80	500
α [deg]	0.00	
I _y [m ⁴], I _z [m ⁴]	1.3333e-02	3.4133e-04
I _y [mm], I _z [mm]	289	46
W _{el,y} [m ³], W _{el,z} [m ³]	2.6667e-02	4.2667e-03
W _{pl,y} [m ³], W _{pl,z} [m ³]	4.0851e-02	6.5362e-03
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	2.35e+06	2.35e+05
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	3.76e+05	3.76e+05
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	1.2223e-03	2.4756e-05
β_y [mm], β_z [mm]	0	0



Explanations of symbols	
A	Area
A _y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A _z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
A _L	Circumference per unit length
A _D	Drying surface per unit length
c _{y,UCS}	Centroid coordinate in Y-direction of Input axis system
c _{z,UCS}	Centroid coordinate in Z-direction of Input axis system
I _{y,UCS}	Second moment of area about the YLCS axis
I _{z,UCS}	Second moment of area about the ZLCS axis
I _{yz,UCS}	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I _y	Second moment of area about the principal y-axis
I _z	Second moment of area about the principal z-axis
I _y	Radius of gyration about the principal y-axis
I _z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el,y}$	Elastic section modulus about the principal y-axis
$W_{el,z}$	Elastic section modulus about the principal z-axis
$W_{pl,y}$	Plastic section modulus about the principal y-axis
$W_{pl,z}$	Plastic section modulus about the principal z-axis
$M_{pl,y,+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl,y,-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl,z,+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl,z,-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

4. Load cases

Name	Description	Action type	Load group	Direction	Duration	Master load case
Spec		Load type				
Self Weight 1	Self weight LVL cassettes	Permanent	Self Weight	-Z		
Self Weight 2	Floor	Self weight Permanent Standard	Self Weight			
Variable Floor Load	Variable Floor cat. C	Variable	Variable cat. C		Medium	None
Wind	Wind Suction Flat Roofs <5° Static wind	Variable Static	Wind			None
Snow	Snow Snow	Variable Static	Snow			None

5. Combinations

Name	Description	Type	Load cases	Coeff. [-]
ULS-Set B (auto)		EN-ULS (STR/GEO) Set B	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow Wind - Wind Suction Flat Roofs <5* Self Weight 2 - Floor	1.00 1.00 1.00 1.00 1.00
SLS-Char (auto)		EN-SLS Characteristic	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow Wind - Wind Suction Flat Roofs <5* Self Weight 2 - Floor	1.00 1.00 1.00 1.00 1.00
SLS-Char (auto)1		Envelope - serviceability	Self Weight 1 - Self weight LVL cassettes	1.00
SLS-Char (auto)2		Envelope - serviceability	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C	1.00 1.00
SLS-Char (auto)3		Envelope - serviceability	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C	1.00 0.60
SLS-Char (auto)4		Envelope - serviceability	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow Wind - Wind Suction Flat Roofs <5*	1.00 1.00 0.00 0.00
SLS-Char (auto)5		Envelope - serviceability	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow Wind - Wind Suction Flat Roofs <5*	1.00 0.60 1.00 0.00
SLS-Char (auto)6		Envelope - serviceability	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow Wind - Wind Suction Flat Roofs <5*	1.00 0.60 0.00 1.00
ULS-Set B (auto)1		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes	1.35
ULS-Set B (auto)2		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes	0.90
ULS-Set B (auto)3		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes	1.20
ULS-Set B (auto)4		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C	1.35 0.90
ULS-Set B (auto)5		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C	0.90 0.90
ULS-Set B (auto)6		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C	1.20 1.50
ULS-Set B (auto)7		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C	0.90 1.50
ULS-Set B (auto)8		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow	1.20 0.90 1.50
ULS-Set B (auto)9		Envelope - ultimate	Self Weight 1 - Self weight	0.90

Name	Description	Type	Load cases	Coeff. [-]
			LVL cassettes Variable Floor Load - Variable Floor cat. C Snow - Snow	0.90 1.50
ULS-Set B (auto)10		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Wind - Wind Suction Flat Roofs <5*	1.20 0.90 1.50
ULS-Set B (auto)11		Envelope - ultimate	Self Weight 1 - Self weight LVL cassettes Variable Floor Load - Variable Floor cat. C Wind - Wind Suction Flat Roofs <5*	0.90 0.90 1.50

6. Nodes

Name	Coord X [m]	Coord Y [m]	Coord Z [m]
N32	8.000	2.500	0.000
N33	12.000	2.500	0.000
N34	8.000	4.500	0.000
N35	12.000	4.500	0.000
N36	8.000	6.500	0.000
N37	12.000	6.500	0.000

Name	Coord X [m]	Coord Y [m]	Coord Z [m]
N38	8.000	8.500	0.000
N39	12.000	8.500	0.000
N40	8.000	10.500	0.000
N41	12.000	10.500	0.000
N42	8.000	0.000	0.000
N43	8.000	13.000	0.000

Name	Coord X [m]	Coord Y [m]	Coord Z [m]
N44	12.000	0.000	0.000
N45	12.000	13.000	0.000
N63	7.150	0.500	0.000
N64	12.850	0.500	0.000
N65	12.850	12.500	0.000
N66	7.150	12.500	0.000

7. Members

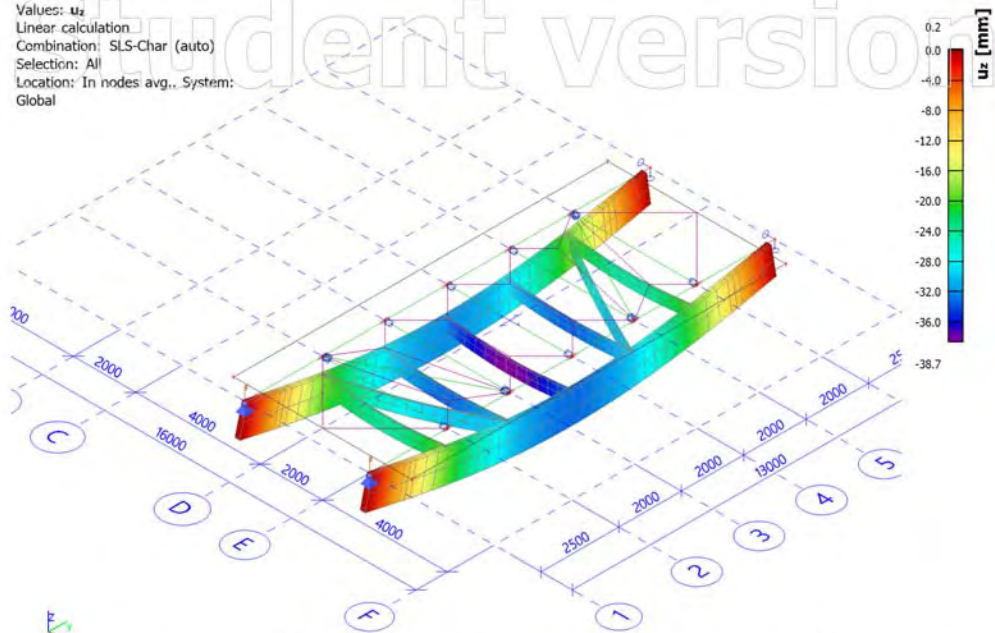
Name	Cross-section	Material	Length [m]	Beg. node	End node	Type
B9	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.000	N32	N33	beam (80)
B10	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.000	N34	N35	beam (80)
B11	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.000	N36	N37	beam (80)
B12	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.000	N38	N39	beam (80)
B13	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.000	N40	N41	beam (80)
B14	CS3 - RECT (160; 1000)	Baubuche S (edge wist)	13.000	N42	N43	beam (80)
B15	CS3 - RECT (160; 1000)	Baubuche S (edge wist)	13.000	N44	N45	beam (80)
B18	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.472	N32	N35	beam (80)
B19	CS2 - RECT (80; 400)	Baubuche S (edge wist)	4.472	N40	N39	beam (80)

8. Nodal supports

Name	Node	System	Type	X	Y	Z	Rx	Ry	Rz
Sn5	N42	GCS	Standard	Rigid	Rigid	Rigid	Free	Free	Free
Sn6	N43	GCS	Standard	Rigid	Free	Rigid	Free	Free	Free
Sn7	N44	GCS	Standard	Rigid	Rigid	Rigid	Free	Free	Free
Sn8	N45	GCS	Standard	Rigid	Free	Rigid	Free	Free	Free

9. 3D displacement; u_z

Values: u_z
Linear calculation
Combination: SLS-Char (auto)
Selection: All
Location: In nodes avg., System:
Global

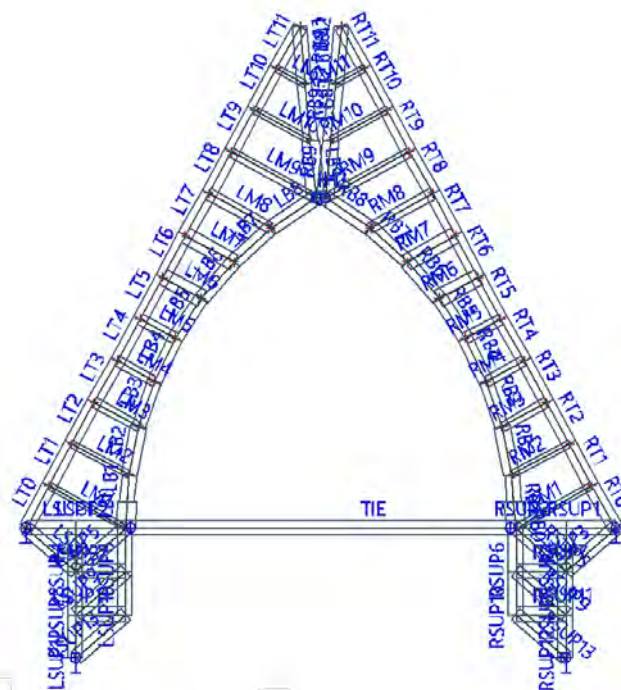


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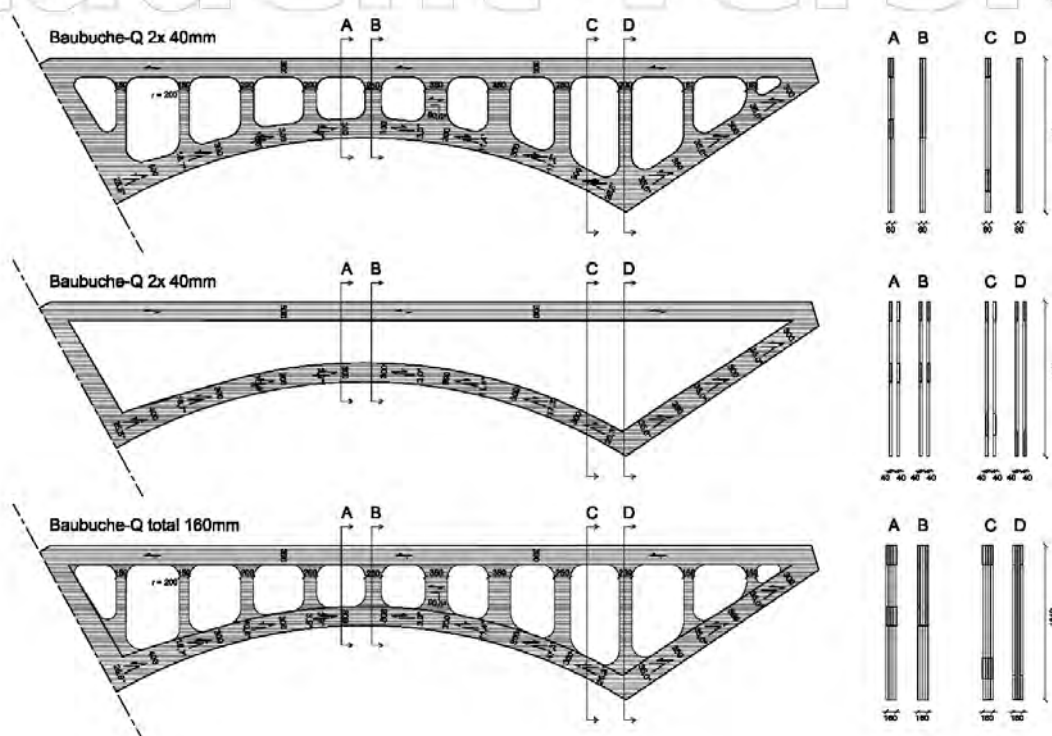
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2. GEOMETRY

2.1. Analysis model



2.2. Top part rafters



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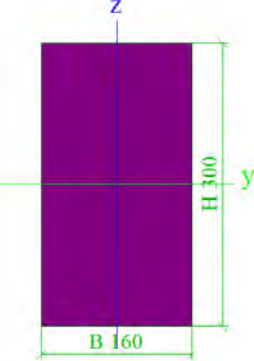
2.3. Materials

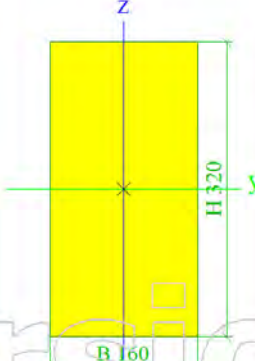
Timber EC5

Name	Type of timber	μ	E_{mod} [MPa]	$f_{m,k}$ [MPa]	$f_{t,0,k}$ [MPa]	$f_{t,90,k}$ [MPa]	$f_{c,0,k}$ [MPa]	$f_{c,90,k}$ [MPa]	$f_{v,k}$ [MPa]	Colour
	ρ [kg/m ³]	α [m/mK]	G_{mod} [MPa]							
Baubuche Q edgewise 0*	Glued, laminated 730,0	0 0,00	1,2800e+04 8,2000e+02	59,0	49,0	8,0	62,0	22,0	8,0	
Baubuche Q edgewise 90*	Glued, laminated 730,0	0 0,00	2,0000e+03 8,2000e+02	9,0	8,0	49,0	22,0	62,0	8,0	

2.4. Cross-sections

Bottom And Top Members 160 x 300

Type	RECT	
Detailed	160; 300	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 0*	
Fabrication	timber	
Colour		
A [m ²]	4,8000e-02	
A _y [m ²], A _z [m ²]	4,0098e-02	4,0028e-02
A _L [m ² /m], A _D [m ² /m]	9,2000e-01	9,2000e-01
c _{y,ucs} [mm], c _{z,ucs} [mm]	80	150
α [deg]	0,00	
I _y [m ⁴], I _z [m ⁴]	3,6000e-04	1,0240e-04
i _y [mm], i _z [mm]	87	46
W _{el,y} [m ³], W _{el,z} [m ³]	2,4000e-03	1,2800e-03
W _{pl,y} [m ³], W _{pl,z} [m ³]	3,1784e-03	1,6951e-03
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	1,97e+05	1,97e+05
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	1,05e+05	1,05e+05
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	2,7215e-04	2,4074e-07
β_y [mm], β_z [mm]	0	0
Picture		

M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	1,12e+05	1,12e+05
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	2,9917e-04	3,3631e-07
β_y [mm], β_z [mm]	0	0
Picture		

Bottom Member 160 x 350

Type	RECT	
Detailed	160; 350	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 0*	
Fabrication	timber	
Colour		
A [m ²]	5,6000e-02	
A _y [m ²], A _z [m ²]	4,6667e-02	4,6667e-02
A _L [m ² /m], A _D [m ² /m]	1,0200e+00	1,0200e+00
c _{y,ucs} [mm], c _{z,ucs} [mm]	80	175
α [deg]	0,00	
I _y [m ⁴], I _z [m ⁴]	5,7167e-04	1,1947e-04
i _y [mm], i _z [mm]	101	46
W _{el,y} [m ³], W _{el,z} [m ³]	3,2667e-03	1,4933e-03
W _{pl,y} [m ³], W _{pl,z} [m ³]	4,3261e-03	1,9777e-03
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	2,68e+05	2,68e+05
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	1,23e+05	1,23e+05
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	3,4053e-04	0,0000e+00
β_y [mm], β_z [mm]	0	0

Bottom Member 160 x 320

Type	RECT	
Detailed	160; 320	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 0*	
Fabrication	timber	
Colour		
A [m ²]	5,1200e-02	
A _y [m ²], A _z [m ²]	4,2771e-02	4,2693e-02
A _L [m ² /m], A _D [m ² /m]	9,6000e-01	9,6000e-01
c _{y,ucs} [mm], c _{z,ucs} [mm]	80	160
α [deg]	0,00	
I _y [m ⁴], I _z [m ⁴]	4,3691e-04	1,0923e-04
i _y [mm], i _z [mm]	92	46
W _{el,y} [m ³], W _{el,z} [m ³]	2,7307e-03	1,3653e-03
W _{pl,y} [m ³], W _{pl,z} [m ³]	3,6163e-03	1,8081e-03
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	2,24e+05	2,24e+05

A diagram of a rectangular cross-section. The vertical axis is labeled Z and the horizontal axis is labeled Y . The width of the rectangle is labeled $B = 160$ and the height is labeled $H = 350$.

Type	RECT	
Detailed	160; 400	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 0*	
Fabrication	timber	
Colour		
A [m ²]	6,4000e-02	
A _y [m ²], A _z [m ²]	5,3464e-02	5,3354e-02
A _L [m ² /m], A _D [m ² /m]	1,1200e+00	1,1200e+00
C _{y,ucs} [mm], C _{z,ucs} [mm]	80	200
α [deg]	0,00	
I _y [m ⁴], I _z [m ⁴]	8,5333e-04	1,3653e-04
i _y [mm], i _z [mm]	115	46
W _{el,y} [m ³], W _{el,z} [m ³]	4,2667e-03	1,7067e-03
W _{pl,y} [m ³], W _{pl,z} [m ³]	5,6505e-03	2,2602e-03
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	3,50e+05	3,50e+05
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	1,40e+05	1,40e+05
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	4,0787e-04	9,4395e-07
β _y [mm], β _z [mm]	0	0
Picture		

A diagram of a rectangular plate with a width $B = 160$ and a height $H = 400$. A vertical Z -axis is shown passing through the center of the plate, and a horizontal Y -axis is shown passing through the center of the plate.

Type	RECT		
Detailed	80; 150		
Shape type	Thick-walled		
Item material	Baubuche Q edgewise 90*		
Fabrication	timber		
Colour			
A [m ²]	1,2000e-02		
A _y [m ²], A _z [m ²]	1,0022e-02	1,0006e-02	
A _L [m ² /m], A _D [m ² /m]	4,6000e-01	4,6000e-01	
c _{y,UCS} [mm], c _{z,UCS} [mm]	40		75
α [deg]	0,00		

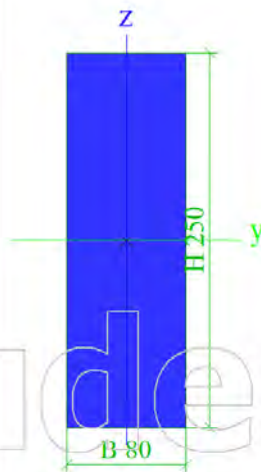
Type	RECT	
Detailed	80; 200	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 90*	
Fabrication	timber	
Colour		
A [m ²]	1,6000e-02	
A _y [m ²], A _z [m ²]	1,3370e-02	1,3339e-02
A _L [m ² /m], A _D [m ² /m]	5,6000e-01	5,6000e-01
C _{y,ucs} [mm], C _{z,ucs} [mm]	40	100
α [deg]	0,00	
I _y [m ⁴], I _z [m ⁴]	5,3333e-05	8,5333e-06
i _y [mm], i _z [mm]	58	23
W _{el,y} [m ³], W _{el,z} [m ³]	5,3333e-04	2,1333e-04
W _{pl,y} [m ³], W _{pl,z} [m ³]	4,2667e-04	1,7067e-04
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	9,39e+03	9,39e+03
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	3,75e+03	3,75e+03
d _y [mm], d _z [mm]	0	0
I _t [m ⁴], I _w [m ⁶]	2,5484e-05	1,4719e-08
β _y [mm], β _z [mm]	0	0
Picture		

A diagram of a rectangular plate with a width labeled B and a height labeled H . The width B is 80 and the height H is 200. A vertical Z -axis and a horizontal Y -axis intersect at the center of the plate, with the origin O at the intersection point.

Type	RECT
Detailed	80; 250
Shape type	Thick-walled

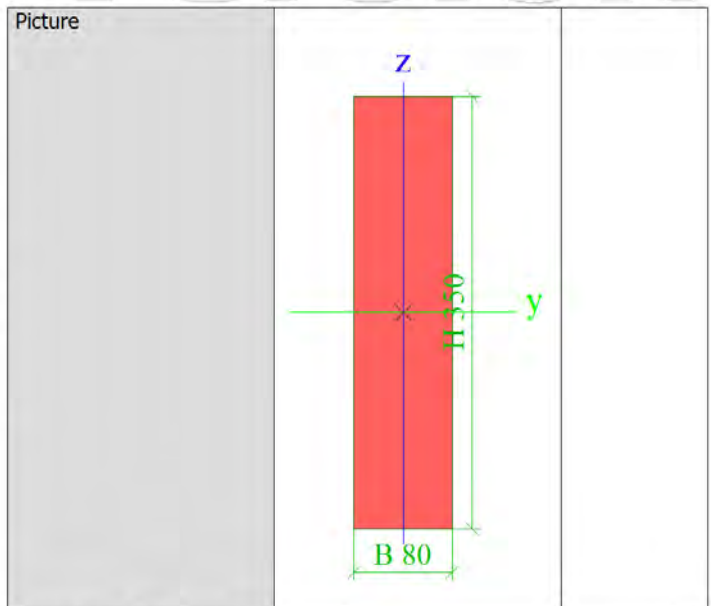
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Item material	Baubuche Q edgewise 90*	
Fabrication	timber	
Colour		
A [m²]	2,0000e-02	
A _y [m²], A _z [m²]	1,6724e-02	1,6673e-02
A _L [m²/m], A _D [m²/m]	6,6000e-01	6,6000e-01
C _{y,ucs} [mm], C _{z,ucs} [mm]	40	125
α [deg]	0,00	
I _y [m⁴], I _z [m⁴]	1,0417e-04	1,0667e-05
i _y [mm], i _z [mm]	72	23
W _{el,y} [m³], W _{el,z} [m³]	8,3333e-04	2,6667e-04
W _{pl,y} [m³], W _{pl,z} [m³]	6,6667e-04	2,1333e-04
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	1,47e+04	1,47e+04
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	4,69e+03	4,69e+03
d _y [mm], d _z [mm]	0	0
I _t [m⁴], I _w [m⁶]	3,3983e-05	3,6050e-08
β _y [mm], β _z [mm]	0	0
Picture		



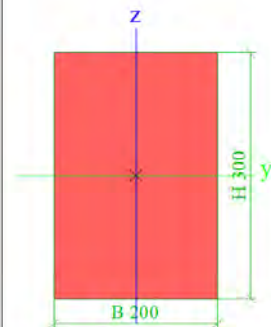
Intermediate Member 80 x 350

Type	RECT	
Detailed	80; 350	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 90*	
Fabrication	timber	
Colour		
A [m²]	2,8000e-02	
A _y [m²], A _z [m²]	2,3435e-02	2,3339e-02
A _L [m²/m], A _D [m²/m]	8,6000e-01	8,6000e-01
C _{y,ucs} [mm], C _{z,ucs} [mm]	40	175
α [deg]	0,00	
I _y [m⁴], I _z [m⁴]	2,8583e-04	1,4933e-05
i _y [mm], i _z [mm]	101	23
W _{el,y} [m³], W _{el,z} [m³]	1,6333e-03	3,7333e-04
W _{pl,y} [m³], W _{pl,z} [m³]	1,3067e-03	2,9867e-04
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	2,87e+04	2,87e+04
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	6,57e+03	6,57e+03
d _y [mm], d _z [mm]	0	0
I _t [m⁴], I _w [m⁶]	5,0988e-05	1,2048e-07
β _y [mm], β _z [mm]	0	0



Imaginary Tie-Beam

Type	RECT	
Detailed	200; 300	
Shape type	Thick-walled	
Item material	Baubuche Q edgewise 0*	
Fabrication	timber	
Colour		
A [m²]	6,0000e-02	
A _y [m²], A _z [m²]	5,0078e-02	5,0035e-02
A _L [m²/m], A _D [m²/m]	1,0000e+00	1,0000e+00
C _{y,ucs} [mm], C _{z,ucs} [mm]	100	150
α [deg]	0,00	
I _y [m⁴], I _z [m⁴]	4,5000e-04	2,0000e-04
i _y [mm], i _z [mm]	87	58
W _{el,y} [m³], W _{el,z} [m³]	3,0000e-03	2,0000e-03
W _{pl,y} [m³], W _{pl,z} [m³]	3,9730e-03	2,6486e-03
M _{pl,y,+} [Nm], M _{pl,y,-} [Nm]	2,46e+05	2,46e+05
M _{pl,z,+} [Nm], M _{pl,z,-} [Nm]	1,64e+05	1,64e+05
d _y [mm], d _z [mm]	0	0
I _t [m⁴], I _w [m⁶]	4,6913e-04	2,3929e-07
β _y [mm], β _z [mm]	0	0
Picture		



Explanations of symbols

A	Area
A _y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A _z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
A _L	Circumference per unit length
A _D	Drying surface per unit length
C _{y,ucs}	Centroid coordinate in Y-direction of

Explanations of symbols

C _{z,ucs}	Input axis system
C _{z,ucs}	Centroid coordinate in Z-direction of Input axis system
I _{y,lcs}	Second moment of area about the YLCS axis
I _{z,lcs}	Second moment of area about the ZLCS axis
I _{yz,lcs}	Product moment of area in the LCS

Project Notre Dame Field Factory**Explanations of symbols**

	system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis
i_z	Radius of gyration about the principal z-axis
$W_{el,y}$	Elastic section modulus about the principal y-axis
$W_{el,z}$	Elastic section modulus about the principal z-axis
$W_{pl,y}$	Plastic section modulus about the principal y-axis
$W_{pl,z}$	Plastic section modulus about the principal z-axis

Explanations of symbols

$M_{pl,y,+}$	Plastic moment about the principal y-axis for a positive M_y moment
$M_{pl,y,-}$	Plastic moment about the principal y-axis for a negative M_y moment
$M_{pl,z,+}$	Plastic moment about the principal z-axis for a positive M_z moment
$M_{pl,z,-}$	Plastic moment about the principal z-axis for a negative M_z moment
d_y	Shear center coordinate in principal y-direction measured from the centroid - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

2.5. Nodes

Name	Coord X [m]	Coord Y [m]	Coord Z [m]
HINGE	2,850	6,130	6,870
N-LB1	2,850	2,070	0,000
N-LB2	2,850	2,150	1,100
N-LB3	2,850	2,350	2,100
N-LB4	2,850	2,680	3,050
N-LB5	2,850	3,120	3,950
N-LB6	2,850	3,660	4,780
N-LB7	2,850	4,330	5,540
N-LB8	2,850	5,100	6,250
N-LB9	2,850	6,030	6,870
N-LB10	2,850	5,920	8,060
N-LB11	2,850	5,810	9,250
N-LSUP1	2,850	1,035	0,000
N-LSUP2	2,850	1,035	-0,900
N-LSUP3	2,850	2,070	-0,900
N-LSUP4	2,850	1,035	-1,800
N-LSUP5	2,850	2,070	-1,800
N-LSUP6	2,850	1,035	-2,700
N-LT0	2,850	0,000	0,000
N-LT1	2,850	0,475	0,870
N-LT2	2,850	0,950	1,740
N-LT3	2,850	1,425	2,610
N-LT4	2,850	1,900	3,480
N-LT5	2,850	2,375	4,350
N-LT6	2,850	2,850	5,220
N-LT7	2,850	3,325	6,090
N-LT8	2,850	3,800	6,960
N-LT9	2,850	4,275	7,830
N-LT10	2,850	4,750	8,700
N-LT11	2,850	5,225	9,570
N-LT12	2,850	5,700	10,440
N-RB1	2,850	10,190	0,000

Name	Coord X [m]	Coord Y [m]	Coord Z [m]
N-RB2	2,850	10,110	1,100
N-RB3	2,850	9,910	2,100
N-RB4	2,850	9,580	3,050
N-RB5	2,850	9,140	3,950
N-RB6	2,850	8,600	4,780
N-RB7	2,850	7,930	5,540
N-RB8	2,850	7,160	6,250
N-RB9	2,850	6,230	6,870
N-RB10	2,850	6,340	8,060
N-RB11	2,850	6,450	9,250
N-RSUP1	2,850	11,225	0,000
N-RSUP2	2,850	11,225	-0,900
N-RSUP3	2,850	10,190	-0,900
N-RSUP4	2,850	11,225	-1,800
N-RSUP5	2,850	10,190	-1,800
N-RSUP6	2,850	11,225	-2,700
N-RSUP7	2,850	2,110	0,550
N-RSUP8	2,850	10,150	0,550
N-RT0	2,850	12,260	0,000
N-RT1	2,850	11,785	0,870
N-RT2	2,850	11,310	1,740
N-RT3	2,850	10,835	2,610
N-RT4	2,850	10,360	3,480
N-RT5	2,850	9,885	4,350
N-RT6	2,850	9,410	5,220
N-RT7	2,850	8,935	6,090
N-RT8	2,850	8,460	6,960
N-RT9	2,850	7,985	7,830
N-RT10	2,850	7,510	8,700
N-RT11	2,850	7,035	9,570
N-RT12	2,850	6,560	10,440

2.6. Members

Name	Cross-section	Material	Length [m]	Beg. node	End node
H1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,100	N-LB9	HINGE
H2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,100	HINGE	N-RB9
LB0	Bottom Member 160 x 400 - RECT (160; 400)	Baubuche Q edgewise 0*	0,551	N-LB1	N-RSUP7
LB1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,551	N-RSUP7	N-LB2
LB2	Bottom Member 160 x 320 - RECT (160; 320)	Baubuche Q edgewise 0*	1,020	N-LB2	N-LB3
LB3	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,006	N-LB3	N-LB4
LB4	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,002	N-LB4	N-LB5
LB5	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,990	N-LB5	N-LB6
LB6	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,013	N-LB6	N-LB7
LB7	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,047	N-LB7	N-LB8
LB8	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,118	N-LB8	N-LB9
LB9	Bottom Member 160 x 350 - RECT (160; 350)	Baubuche Q edgewise 0*	1,195	N-LB9	N-LB10
LB9.1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,195	N-LB10	N-LB11
LB9.2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,195	N-LB11	N-LT12
LM1	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	1,817	N-LT1	N-LB1
LM2	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	1,360	N-LT2	N-LB2
LM3	Intermediate Member 80 x 200 - RECT (80; 200)	Baubuche Q edgewise 90*	1,056	N-LT3	N-LB3
LM4	Intermediate Member 80 x 200 - RECT (80; 200)	Baubuche Q edgewise 90*	0,891	N-LT4	N-LB4
LM5	Intermediate Member 80 x 250 - RECT (80; 250)	Baubuche Q edgewise 90*	0,846	N-LT5	N-LB5
LM6	Intermediate Member 80 x 350 - RECT (80; 350)	Baubuche Q edgewise 90*	0,922	N-LT6	N-LB6
LM7	Intermediate Member 80 x 350 - RECT (80; 350)	Baubuche Q edgewise 90*	1,146	N-LT7	N-LB7
LM8	Intermediate Member 80 x 250 - RECT (80; 250)	Baubuche Q edgewise 90*	1,481	N-LT8	N-LB8
LM9	Intermediate Member 80 x 200 - RECT (80; 200)	Baubuche Q edgewise 90*	2,000	N-LT9	N-LB9
LM10	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	1,334	N-LT10	N-LB10
LM11	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	0,667	N-LT11	N-LB11
LSUP1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-LT0	N-LSUP1
LSUP2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-LB1	N-LSUP1
LSUP3	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-LT0	N-LSUP2
LSUP4	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-LSUP1	N-LSUP2
LSUP5	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-LB1	N-LSUP2
LSUP6	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-LB1	N-LSUP3
LSUP7	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-LSUP3	N-LSUP2
LSUP8	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-LSUP2	N-LSUP4
LSUP9	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-LSUP3	N-LSUP4
LSUP10	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-LSUP3	N-LSUP5
LSUP11	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-LSUP5	N-LSUP4
LSUP12	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-LSUP4	N-LSUP6
LSUP13	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-LSUP5	N-LSUP6
LT0	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT0	N-LT1
LT1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT1	N-LT2
LT2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT2	N-LT3
LT3	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT3	N-LT4
LT4	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT4	N-LT5
LT5	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT5	N-LT6
LT6	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT6	N-LT7
LT7	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT7	N-LT8
LT8	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT8	N-LT9
LT9	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT9	N-LT10
LT10	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT10	N-LT11
LT11	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-LT11	N-LT12
RB0	Bottom Member 160 x 400 - RECT (160; 400)	Baubuche Q edgewise 0*	0,551	N-RSUP8	N-RB1
RB1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,551	N-RB2	N-RSUP8
RB2	Bottom Member 160 x 320 - RECT (160; 320)	Baubuche Q edgewise 0*	1,020	N-RB2	N-RB3
RB3	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,006	N-RB3	N-RB4
RB4	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,002	N-RB4	N-RB5
RB5	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,990	N-RB5	N-RB6
RB6	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,013	N-RB6	N-RB7
RB7	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,047	N-RB7	N-RB8
RB8	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,118	N-RB8	N-RB9
RB9	Bottom Member 160 x 350 - RECT (160; 350)	Baubuche Q edgewise 0*	1,195	N-RB9	N-RB10
RB9.1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,195	N-RB10	N-RB11
RB9.2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,195	N-RB11	N-RT12
RM1	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	1,817	N-RT1	N-RB1
RM2	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	1,360	N-RT2	N-RB2

Name	Cross-section	Material	Length [m]	Beg. node	End node
RM3	Intermediate Member 80 x 200 - RECT (80; 200)	Baubuche Q edgewise 90*	1,056	N-RT3	N-RB3
RM4	Intermediate Member 80 x 200 - RECT (80; 200)	Baubuche Q edgewise 90*	0,891	N-RT4	N-RB4
RM5	Intermediate Member 80 x 250 - RECT (80; 250)	Baubuche Q edgewise 90*	0,846	N-RT5	N-RB5
RM6	Intermediate Member 80 x 350 - RECT (80; 350)	Baubuche Q edgewise 90*	0,922	N-RT6	N-RB6
RM7	Intermediate Member 80 x 350 - RECT (80; 350)	Baubuche Q edgewise 90*	1,146	N-RT7	N-RB7
RM8	Intermediate Member 80 x 250 - RECT (80; 250)	Baubuche Q edgewise 90*	1,481	N-RT8	N-RB8
RM9	Intermediate Member 80 x 200 - RECT (80; 200)	Baubuche Q edgewise 90*	2,000	N-RT9	N-RB9
RM10	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	1,334	N-RT10	N-RB10
RM11	Intermediate Member 80 x 150 - RECT (80; 150)	Baubuche Q edgewise 90*	0,667	N-RT11	N-RB11
RSUP1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-RT0	N-RSUP1
RSUP2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-RSUP1	N-RB1
RSUP3	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-RT0	N-RSUP2
RSUP4	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-RSUP1	N-RSUP2
RSUP5	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-RB1	N-RSUP2
RSUP6	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-RB1	N-RSUP3
RSUP7	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-RSUP3	N-RSUP2
RSUP8	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-RSUP2	N-RSUP4
RSUP9	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-RSUP3	N-RSUP4
RSUP10	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-RSUP3	N-RSUP5
RSUP11	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,035	N-RSUP5	N-RSUP4
RSUP12	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,900	N-RSUP4	N-RSUP6
RSUP13	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	1,372	N-RSUP5	N-RSUP6
RT0	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT0	N-RT1
RT1	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT1	N-RT2
RT2	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT2	N-RT3
RT3	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT3	N-RT4
RT4	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT4	N-RT5
RT5	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT5	N-RT6
RT6	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT6	N-RT7
RT7	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT7	N-RT8
RT8	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT8	N-RT9
RT9	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT9	N-RT10
RT10	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT10	N-RT11
RT11	Bottom And Top Members 160 x 300 - RECT (160; 300)	Baubuche Q edgewise 0*	0,991	N-RT11	N-RT12
TIE	Imaginary Tie-Beam - RECT (200; 300)	Baubuche Q edgewise 0*	8,120	N-RB1	N-LB1

2.7. Nodal supports

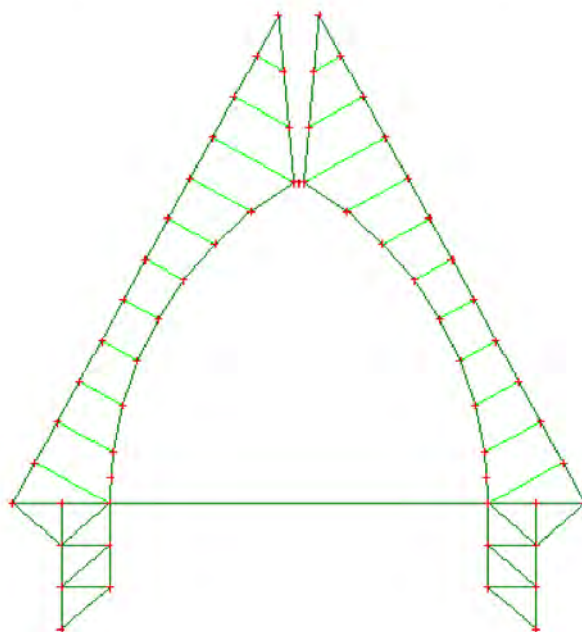
Name	Node	System	Type	X	Y	Z	Rx	Ry	Rz
Sn5	N-LB9	GCS	Standard	Rigid	Free	Free	Free	Free	Free
Sn6	N-RB9	GCS	Standard	Rigid	Free	Free	Free	Free	Free
Sn8	N-RT0	GCS	Standard	Rigid	Free	Rigid press only	Free	Free	Free
Sn81	N-LSUP6	GCS	Standard	Rigid	Rigid press only	Rigid press only	Free	Free	Free
Sn82	N-RSUP6	GCS	Standard	Rigid	Rigid tension only	Rigid press only	Free	Free	Free
Sn83	N-LT0	GCS	Standard	Rigid	Free	Rigid press only	Free	Free	Free

3. LOADS

3.1. Load cases

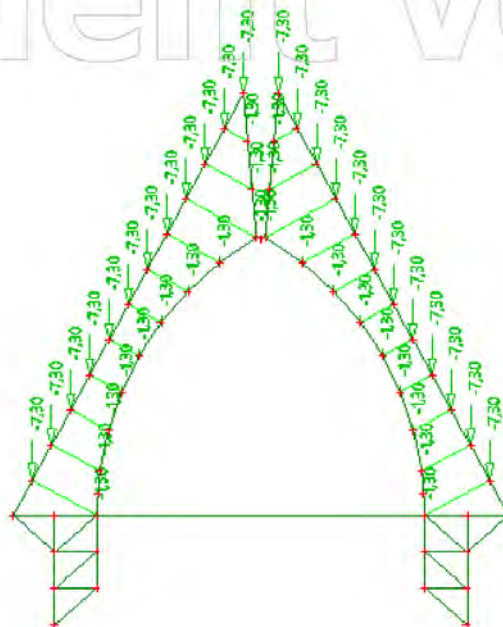
3.1.1. Load cases - LC1

Name	Description	Action type	Load group	Load type	Direction
LC1	Self-weight rafters	Permanent	LG1	Self weight	-Z



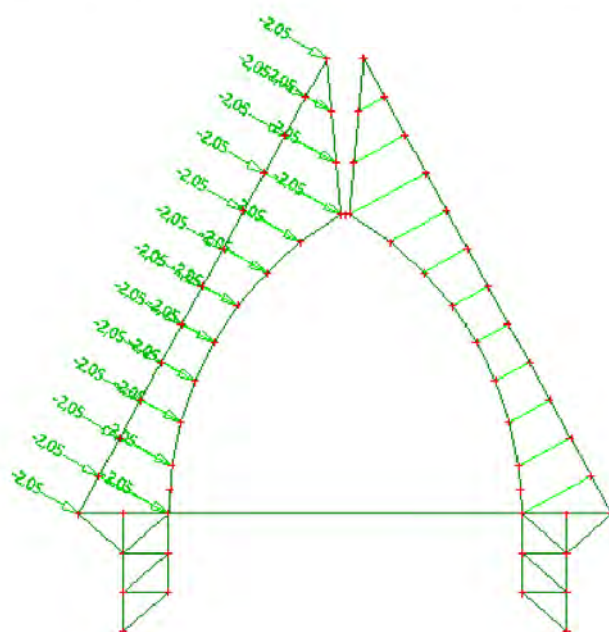
3.1.2. Load cases - LC2

Name	Description	Action type	Load group	Load type
LC2	Self-weight roof	Permanent	LG1	Standard



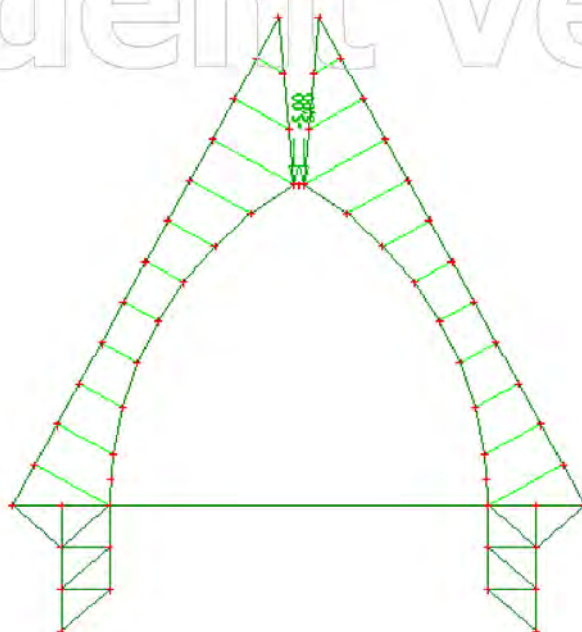
3.1.3. Load cases - LC3

Name	Description	Action type	Load group	Load type	Spec	Duration	Master load case
LC3	Wind	Variable	LG2	Static	Standard	Instantaneous	None



3.1.4. Load cases - LC4

Name	Description	Action type	Load group	Load type	Spec	Duration	Master load case
LC4	Crane	Variable	LG3	Static	Standard	Short	None



3.2. Nonlinear combinations

Name	Type	Load cases	Coeff. [-]
LC.1	Ultimate	LC1 - Self-weight rafters	1,00
LC.2	Ultimate	LC2 - Self-weight roof	1,00
LC.3 (incl. LC.1)	Ultimate	LC2 - Self-weight roof	1,00
		LC3 - Wind	1,00
LC.4	Ultimate	LC4 - Crane	1,00

Project Notre Dame Field Factory

Name	Type	Load cases	Coeff. [-]
ULS 1.0	Ultimate	LC1 - Self-weight rafters	1,35
ULS 1.1	Ultimate	LC1 - Self-weight rafters	1,00
ULS 1.2	Ultimate	LC1 - Self-weight rafters	1,15
ULS 1.3	Ultimate	LC1 - Self-weight rafters	1,00
		LC3 - Wind	1,50
ULS 1.4	Ultimate	LC1 - Self-weight rafters	1,15
		LC3 - Wind	1,50
ULS 1.5	Ultimate	LC1 - Self-weight rafters	1,35
		LC4 - Crane	1,50
ULS 1.6	Ultimate	LC1 - Self-weight rafters	1,00
		LC4 - Crane	1,50
ULS 1.7	Ultimate	LC1 - Self-weight rafters	1,15
		LC4 - Crane	1,50
ULS 1.8	Ultimate	LC1 - Self-weight rafters	1,00
		LC3 - Wind	1,50
		LC4 - Crane	1,50
ULS 1.9	Ultimate	LC1 - Self-weight rafters	1,15
		LC3 - Wind	1,50
		LC4 - Crane	1,50
ULS 2.0	Ultimate	LC1 - Self-weight rafters	1,35
		LC2 - Self-weight roof	1,35
ULS 2.1	Ultimate	LC1 - Self-weight rafters	1,00
		LC2 - Self-weight roof	1,00
ULS 2.2	Ultimate	LC1 - Self-weight rafters	1,15
		LC2 - Self-weight roof	1,15
ULS 2.3	Ultimate	LC1 - Self-weight rafters	1,00
		LC2 - Self-weight roof	1,00
		LC3 - Wind	1,50
ULS 2.4	Ultimate	LC1 - Self-weight rafters	1,15
		LC2 - Self-weight roof	1,15
		LC3 - Wind	1,50
ULS 2.5	Ultimate	LC1 - Self-weight rafters	1,35
		LC2 - Self-weight roof	1,35
		LC4 - Crane	1,50
ULS 2.6	Ultimate	LC1 - Self-weight rafters	1,00
		LC2 - Self-weight roof	1,00
		LC4 - Crane	1,50
ULS 2.7	Ultimate	LC1 - Self-weight rafters	1,15
		LC2 - Self-weight roof	1,15
		LC4 - Crane	1,50
ULS 2.8	Ultimate	LC1 - Self-weight rafters	1,00
		LC2 - Self-weight roof	1,00
		LC3 - Wind	1,50
		LC4 - Crane	1,50
ULS 2.9	Ultimate	LC1 - Self-weight rafters	1,15
		LC2 - Self-weight roof	1,15
		LC3 - Wind	1,50
		LC4 - Crane	1,50
SLS 1.1	Serviceability	LC1 - Self-weight rafters	1,00
SLS 1.2	Serviceability	LC1 - Self-weight rafters	1,00
		LC3 - Wind	1,00
		LC4 - Crane	1,00
SLS 2.1	Serviceability	LC1 - Self-weight rafters	1,00
		LC2 - Self-weight roof	1,00
SLS 2.2	Serviceability	LC1 - Self-weight rafters	1,00
		LC2 - Self-weight roof	1,00
		LC3 - Wind	1,00
		LC4 - Crane	1,00

3.3. REACTION FORCES PER LOAD CASE

3.3.1. Reactions; R_y ; R_z

Values: R_z , R_y

Nonlinear calculation

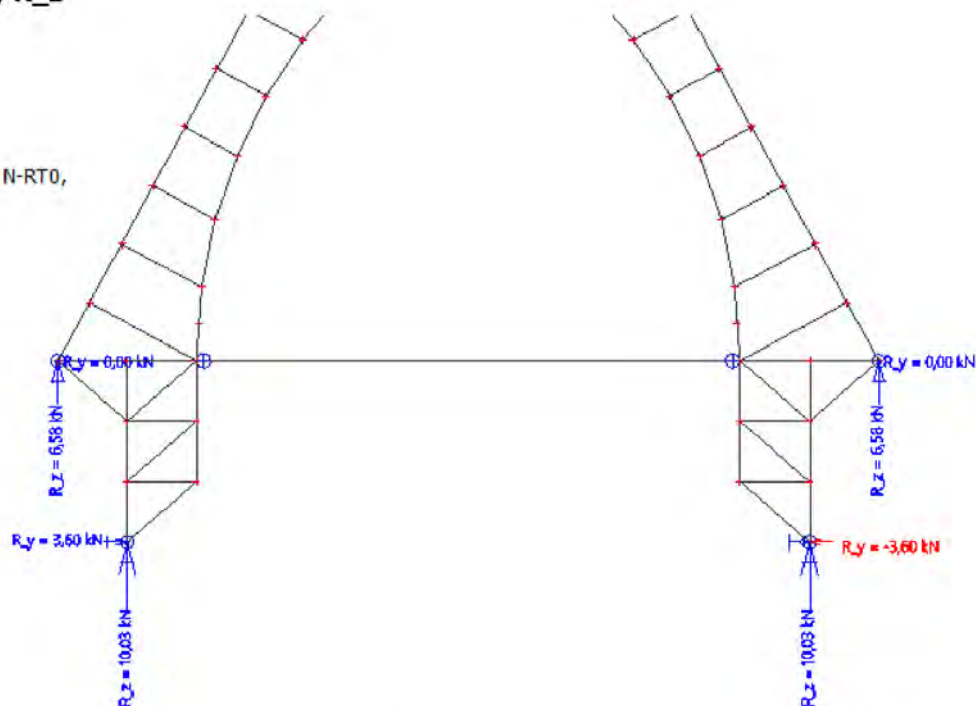
NonLinear Combi: LC.1

System: Global

Extreme: Member

Selection: Sn8, Sn81..Sn83, N-RT0,

N-LT0, N-LSUP6, N-RSUP6



3.3.2. Reactions; R_y ; R_z

Values: R_z , R_y

Nonlinear calculation

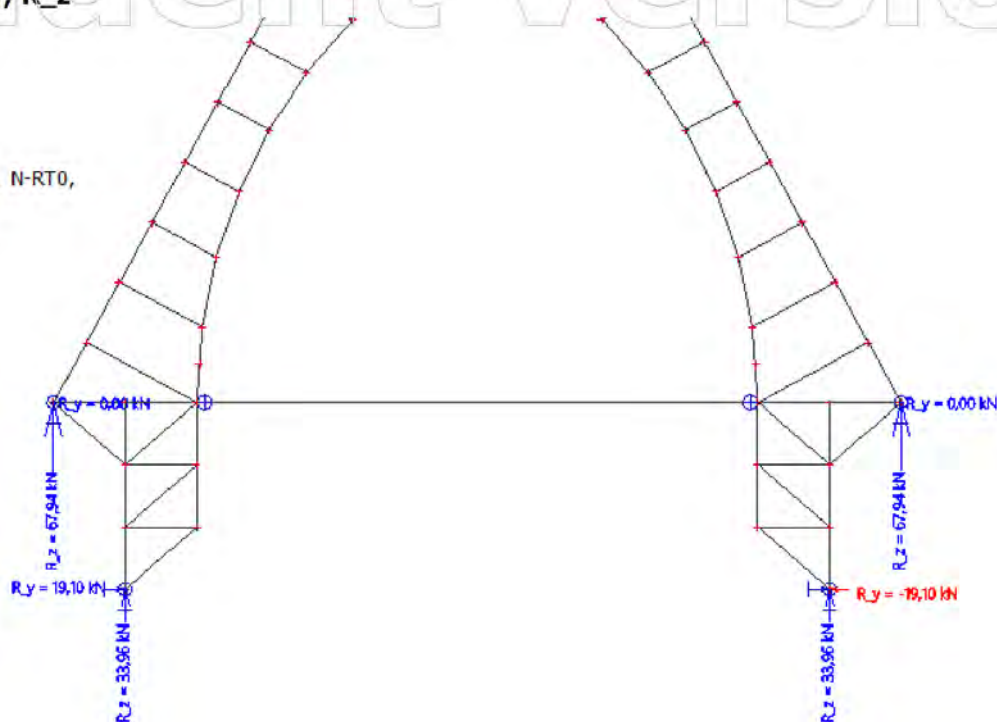
NonLinear Combi: LC.2

System: Global

Extreme: Member

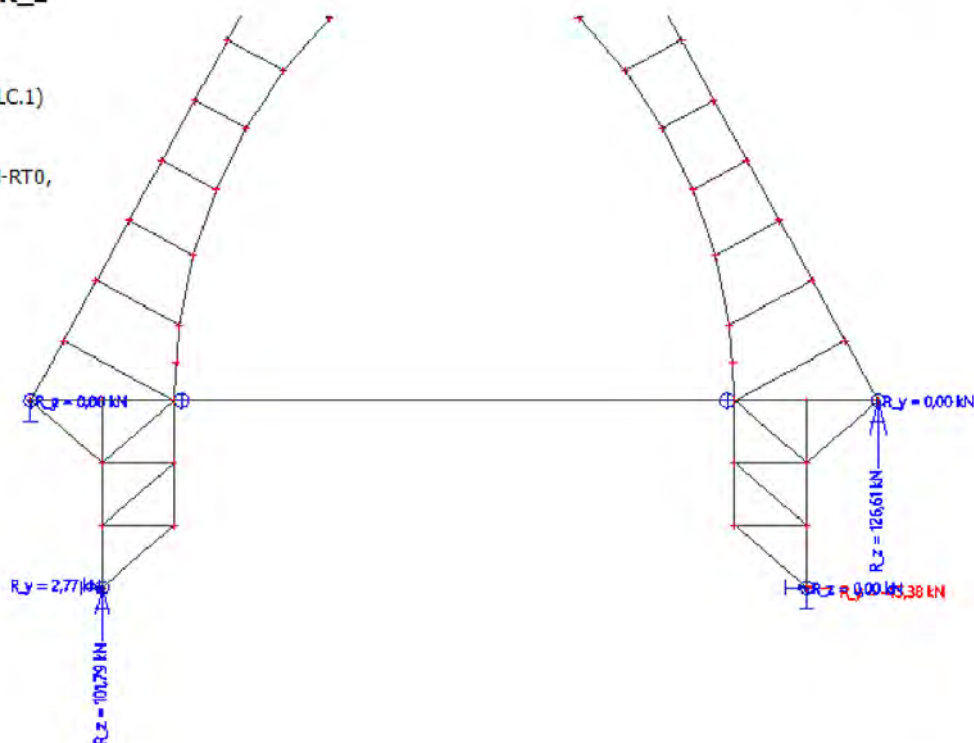
Selection: Sn8, Sn81..Sn83, N-RT0,

N-LT0, N-LSUP6, N-RSUP6



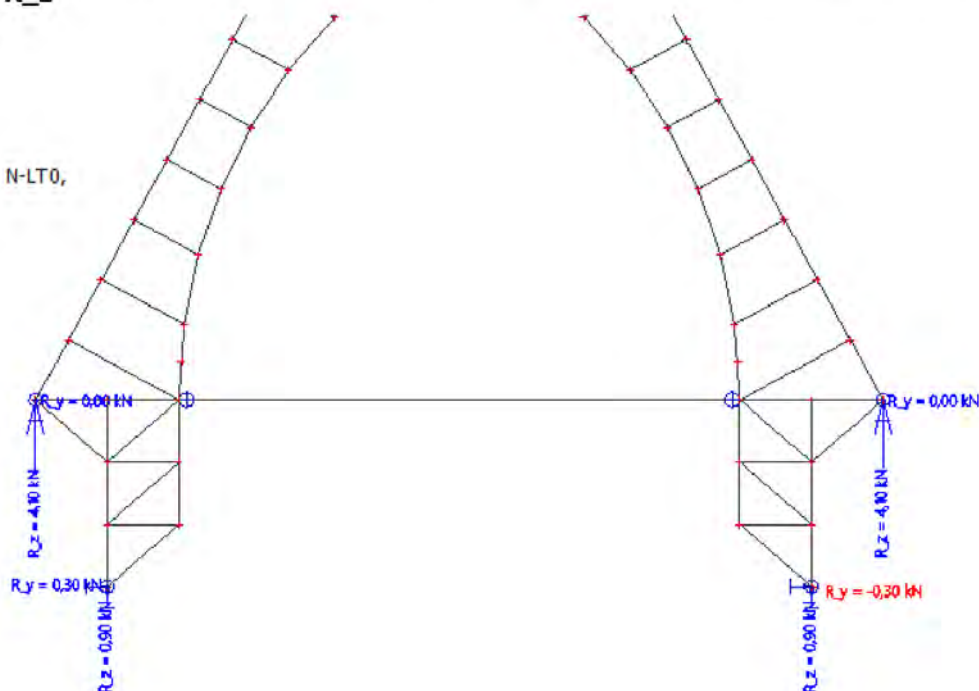
3.3.3. Reactions; R_y ; R_z

Values: R_z , R_y
Nonlinear calculation
NonLinear Combi: LC.3 (incl. LC.1)
System: Global
Extreme: Member
Selection: Sn8, Sn81..Sn83, N-RT0,
N-LT0, N-LSUP6, N-RSUP6



3.3.4. Reactions; R_y ; R_z

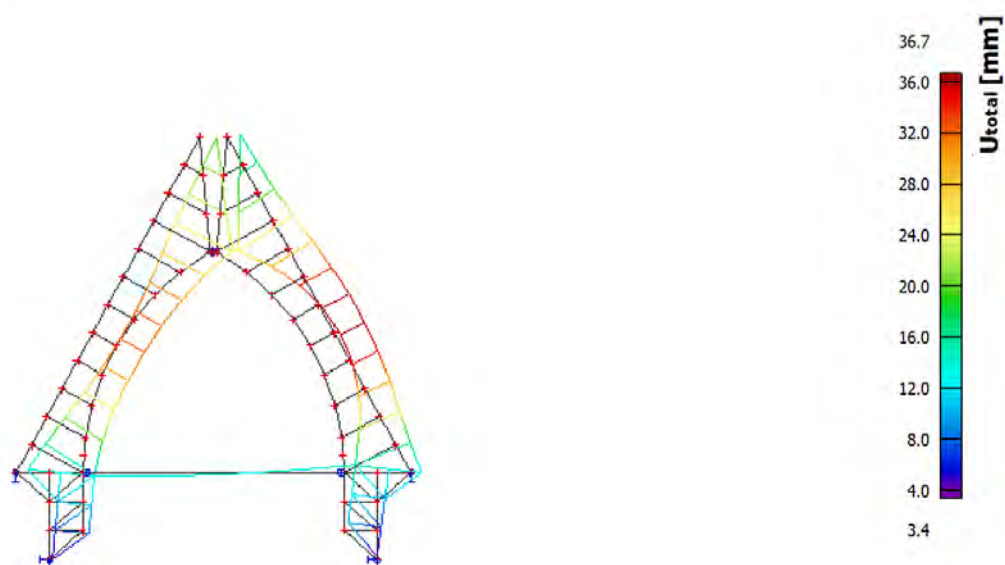
Values: R_z , R_y
Nonlinear calculation
NonLinear Combi: LC.4
System: Global
Extreme: Member
Selection: Sn8, Sn83, N-RT0, N-LT0,
N-LSUP6, N-RSUP6



4. SLS 1.2 DEFORMATIONS

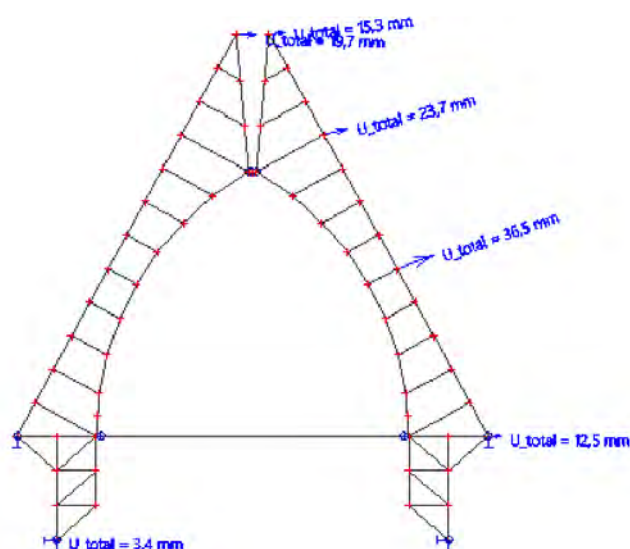
4.1. 3D displacement; U_total

Values: U_{total}
Nonlinear calculation
NonLinear Combi: SLS 1.2
Selection: All
Location: In nodes avg.. System:
Global



4.2. Displacement of nodes; U_total

Values: U_{total}
Nonlinear calculation
NonLinear Combi: SLS 1.2
Extreme: Node
Selection: N-RT0, N-LT12, N-RT5,
N-RT9, N-RT12, N-LSUP6



4.3. GOVERNING DEFLECTIONS AND CREEP

LARGEST DEFLECTIONS DUE TO FINAL DEFORMATION LC.1 + INITIAL ELASTIC DEFORMATION LC.3 TOTAL: 19.85 MM UC: 0.441
GOVERNING CHECK INITIAL ELASTIC DEFORMATION LC.1 + LC.3 TOTAL: 19.58 MM, UC: 0.653

According to Eurocode 5 - Table 7.2
 And Polimeter Handbook

$$l_{ref} = 9000 \text{ mm}$$

$$k_{def} = 0.8 \quad \psi_{2,1} = 0$$

Initial elastic deformation

$$u_{int,ref} = \frac{l_{ref}}{300} = 30 \text{ mm}$$

Unity Checks

self-weight deformation

$$u_{int,G} = 0.28 \text{ mm}$$

wind load

$$u_{int,Q} = 16.04 \text{ mm}$$

total initial deformation

$$u_{int} = u_{int,G} + u_{int,Q} = 16.32 \text{ mm}$$

(x Vierendeel factor 1.2)

$$u_{int,s} = (u_{int,G} + u_{int,Q}) \cdot 1.2 = 19.584 \text{ mm}$$

$$\eta = \frac{u_{int,s}}{u_{int,ref}} = 0.653$$

Final deformation

$$u_{fin,ref} = \frac{l_{ref}}{200} = 45 \text{ mm}$$

self-weight deformation

$$u_{fin,G} = u_{int,G} \cdot (1 + k_{def}) = 0.504 \text{ mm}$$

wind load

$$u_{fin,Q} = u_{int,Q} \cdot (1 + \psi_{2,1} \cdot k_{def}) = 16.04 \text{ mm}$$

total final deformation

$$u_{fin,tot} = u_{fin,G} + u_{fin,Q} = 16.544 \text{ mm}$$

(x Vierendeel factor 1.2)

$$u_{fin,s} = (u_{fin,G} + u_{fin,Q}) \cdot 1.2 = 19.853 \text{ mm}$$

$$\eta = \frac{u_{fin,s}}{u_{fin,ref}} = 0.441$$

Total final deformation

$$u_{tot,fin,ref} = \frac{l_{ref}}{300} = 30 \text{ mm}$$

self-weight deformation

$$u_{tot,fin,G} = u_{int,G} \cdot (1 + k_{def}) = 0.504 \text{ mm}$$

wind load

$$u_{tot,fin,Q} = u_{int,Q} \cdot \psi_{2,1} \cdot (1 + k_{def}) = 0 \text{ mm}$$

total final deformation

$$u_{tot,fin} = u_{tot,fin,G} + u_{tot,fin,Q} = 0.504 \text{ mm}$$

(x Vierendeel factor 1.2)

$$u_{tot,fin,s} = (u_{tot,fin,G} + u_{tot,fin,Q}) \cdot 1.2 = 0.605 \text{ mm}$$

$$\eta = \frac{u_{tot,fin,s}}{u_{tot,fin,ref}} = 0.02$$

SLS 1.1 SELF-WEIGHT FIELD FACTORY

Name	Case	Ux [mm]	Uy [mm]	Uz [mm]	Φx [mrad]	Φy [mrad]	Φz [mrad]	Utotal [mm]
N-RT0	SLS 1.1	0	0	0	0	0	0	0
N-RT5	SLS 1.1	0	0.2	0.1	0	0	0	0.3
N-RT9	SLS 1.1	0	-0.2	-0.1	0.2	0	0	0.2

		Uy*sin(30)	Uz*cos(30)	Utotal
U PERPENDICULAR TO ROOF SURFACE		Uy*p [mm]	Uz*p [mm]	Utot*p [mm]
N-RT0	SLS 1.2 BOTTOM	0	0	0
N-RT5	SLS 1.2 MIDDLE	0.1	0.0866	0.1866
N-RT9	SLS 1.2 TOP	-0.1	-0.0866	-0.1866
		CORRECTED U		0.2799

SLS 1.2 SELF-WEIGHT FIELD FACTORY + WIND + CRANE

Name	Case	Ux [mm]	Uy [mm]	Uz [mm]	Φx [mrad]	Φy [mrad]	Φz [mrad]	Utotal [mm]
N-RT0	SLS 1.2	0	12.5	0	-4.2	0	0	12.5
N-RT5	SLS 1.2	0	34.4	12.2	-1	0	0	36.5
N-RT9	SLS 1.2	0	22.9	6	4.7	0	0	23.7

		Uy*sin(30)	Uz*cos(30)	Utotal
U PERPENDICULAR TO ROOF SURFACE		Uy*p [mm]	Uz*p [mm]	Utot*p [mm]
N-RT0	SLS 1.2 BOTTOM	6.25	0	6.25
N-RT5	SLS 1.2 MIDDLE	17.2	10.5652	27.7652
N-RT9	SLS 1.2 TOP	11.45	5.196	16.646
		CORRECTED U		16.3172

SLS 2.1 SELF-WEIGHT FORMER FINISHING

Name	Case	Ux [mm]	Uy [mm]	Uz [mm]	Φx [mrad]	Φy [mrad]	Φz [mrad]	Utotal [mm]
N-RT0	SLS 2.1	0	0.1	0	0.1	0	0	0.1
N-RT5	SLS 2.1	0	0.8	-0.1	0.1	0	0	0.9
N-RT9	SLS 2.1	0	-0.6	-1.1	0.7	0	0	1.3

		Uy*sin(30)	Uz*cos(30)	Utotal
U PERPENDICULAR TO ROOF SURFACE		Uy*p [mm]	Uz*p [mm]	Utot*p [mm]
N-RT0	SLS 1.2 BOTTOM	0.05	0	0.05
N-RT5	SLS 1.2 MIDDLE	0.4	-0.0866	0.3134
N-RT9	SLS 1.2 TOP	-0.3	-0.9526	-1.2526
		CORRECTED U		0.9147

SLS 2.2 SELF-WEIGHT FORMER FINISHING + WIND + CRANE

Name	Case	Ux [mm]	Uy [mm]	Uz [mm]	Φx [mrad]	Φy [mrad]	Φz [mrad]	Utotal [mm]
N-RT0	SLS 2.2	0	6.1	0	-1.6	0	0	6.1
N-RT5	SLS 2.2	0	19.9	7	-0.9	0	0	21.1
N-RT9	SLS 2.2	0	12.1	2.6	3.7	0	0	12.4

		Uy*sin(30)	Uz*cos(30)	Utotal
U PERPENDICULAR TO ROOF SURFACE		Uy*p [mm]	Uz*p [mm]	Utot*p [mm]
N-RT0	SLS 1.2 BOTTOM	3.05	0	3.05
N-RT5	SLS 1.2 MIDDLE	9.95	6.062	16.012
N-RT9	SLS 1.2 TOP	6.05	2.2516	8.3016
		CORRECTED U		10.3362

5. ULS 1.3 INTERMEDIATE MEMBERS

5.1. 1D internal forces; N

Values: N

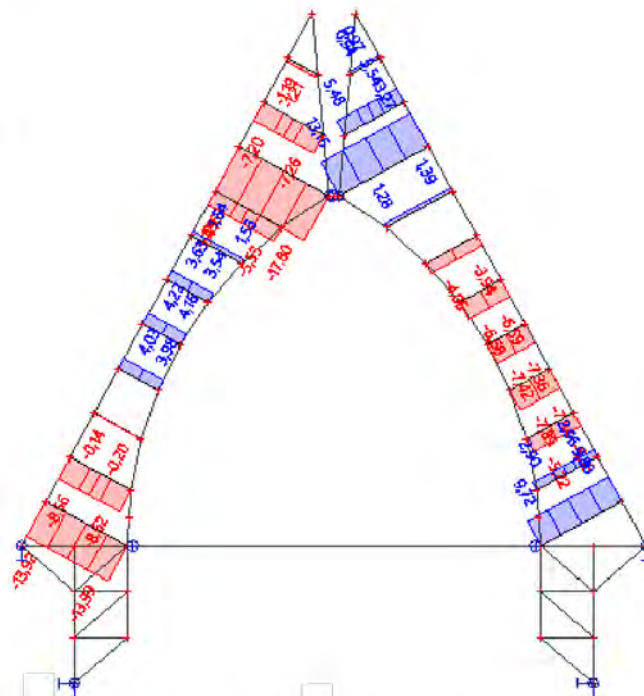
Nonlinear calculation

NonLinear Combi: ULS 1.3

Coordinate system: Member

Extreme 1D: Member

Selection: LM1..LM11, RM1..RM11



5.2. 1D internal forces; V_z

Values: V_z

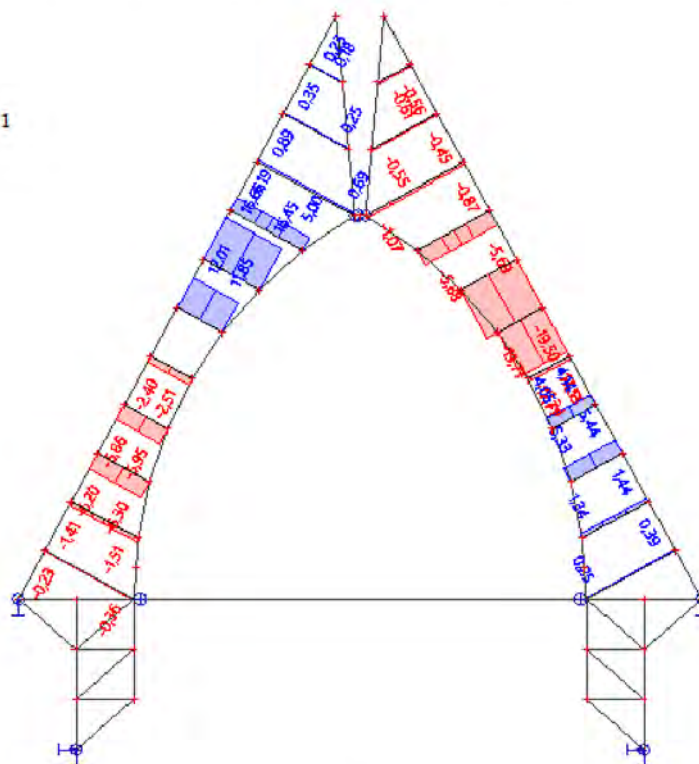
Nonlinear calculation

NonLinear Combi: ULS 1.3

Coordinate system: Member

Extreme 1D: Member

Selection: LM1..LM11, RM1..RM11



5.3. 1D internal forces; M_y

Values: M_y

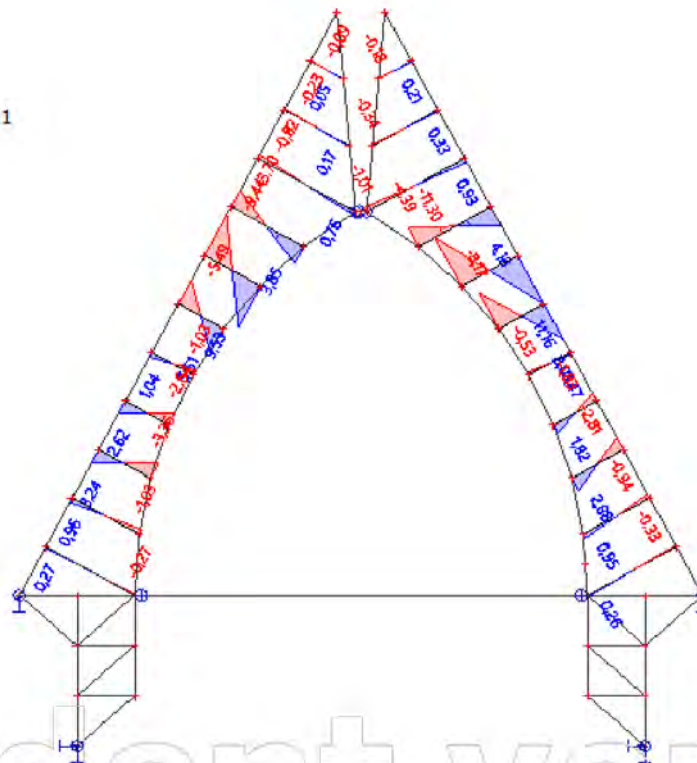
Nonlinear calculation

NonLinear Combi: ULS 1.3

Coordinate system: Member

Extreme 1D: Member

Selection: LM1..LM11, RM1..RM11



5.4. EXAMPLE CALCULATION INTERMEDIATE MEMBER RM7

EXAMPLE CALCULATION MEMBER RM7

GEOMETRICAL AND MATERIAL PROPERTIES

length
width
height
cross-section properties
kmod instantaneous (ULS 1.3)
safety factor LVL

$$\begin{aligned} l &:= 1150 \text{ mm} \\ b &:= 80 \text{ mm} \\ h &:= 350 \text{ mm} \\ A &:= b \cdot h = (2.8 \cdot 10^4) \text{ mm}^2 \\ W &:= \left(\frac{1}{6}\right) \cdot b \cdot h^2 = (1.633 \cdot 10^6) \text{ mm}^3 \\ k_{mod} &:= 1.1 \\ \gamma_m &:= 1.3 \end{aligned}$$

BAUBUCHE-Q PANELS 90° ANGLE TO GRAIN IN MEMBERS:

$$\begin{aligned} f_{Q.m.90.k} &:= 9 \frac{N}{\text{mm}^2} \\ f_{Q.t.90.k} &:= 8 \frac{N}{\text{mm}^2} \\ f_{Q.c.90.k} &:= 22 \frac{N}{\text{mm}^2} \\ f_{Q.v.k} &:= 7.8 \frac{N}{\text{mm}^2} \end{aligned}$$

$$\begin{aligned} f_{Q.m.90.d} &:= k_{mod} \cdot \frac{f_{Q.m.90.k}}{\gamma_m} = 7.615 \frac{N}{\text{mm}^2} \\ f_{Q.t.90.d} &:= k_{mod} \cdot \frac{f_{Q.t.90.k}}{\gamma_m} = 6.769 \frac{N}{\text{mm}^2} \\ f_{Q.c.90.d} &:= k_{mod} \cdot \frac{f_{Q.c.90.k}}{\gamma_m} = 18.615 \frac{N}{\text{mm}^2} \\ f_{Q.v.d} &:= k_{mod} \cdot \frac{f_{Q.v.k}}{\gamma_m} = 6.6 \frac{N}{\text{mm}^2} \end{aligned}$$

FORCES IN INTERMEDIATE MEMBER

$$N_d := -4.05 \text{ kN} \quad V_{z.d} := -19.71 \text{ kN} \quad M_d := 11.3 \text{ kN} \cdot \text{m}$$

COMPRESSION AT AN ANGLE 90° CHECK:

stress $\sigma_{c.90.d} := \frac{N_d}{A} = -0.145 \frac{N}{\text{mm}^2}$

resistance $f_{Q.c.90.d} = 18.615 \frac{N}{\text{mm}^2}$

UC $\left| \frac{\sigma_{c.90.d}}{f_{Q.c.90.d}} \right| = 0.008$

OK!

SHEAR AT AN ANGLE 90° CHECK:

stress $\tau_{max} := \left(\frac{3}{2}\right) \cdot \frac{V_{z.d}}{A} = -1.056 \frac{N}{\text{mm}^2}$

resistance $f_{Q.v.d} = 6.6 \frac{N}{\text{mm}^2}$

UC $\left| \frac{\tau_{max}}{f_{Q.v.d}} \right| = 0.16$

OK!

BENDING AT AN ANGLE 90° CHECK:

stress $\sigma_{m.90.d} := \frac{M_d}{W} = 6.918 \frac{N}{\text{mm}^2}$

resistance $f_{Q.m.90.d} = 7.615 \frac{N}{\text{mm}^2}$

UC $\left| \frac{\sigma_{m.90.d}}{f_{Q.m.90.d}} \right| = 0.908$

OK!

COMBINED COMPRESSION AND BENDING 26°

UC $\left| \left(\frac{\sigma_{c.90.d}}{f_{Q.c.90.d}} \right)^2 + \frac{\sigma_{m.90.d}}{f_{Q.m.90.d}} \right| \leq 1 = 1$

OK!

$\left(\frac{\sigma_{c.90.d}}{f_{Q.c.90.d}} \right)^2 = 6.037 \cdot 10^{-5} \frac{\sigma_{m.90.d}}{f_{Q.m.90.d}} = 0.908$

OVERVIEW ULS CHECKS FOR ALL INTERMEDIATE MEMBER

[illegible]

Name	lat [deg]	lon [deg]	Case	N ₁ [km]	N ₂ [km]	N ₃ [km]	N ₄ [km]	N ₅ [km]	N ₆ [km]	N ₇ [km]	N ₈ [km]	N ₉ [km]	N ₁₀ [km]	N ₁₁ [km]	N ₁₂ [km]	N ₁₃ [km]	N ₁₄ [km]	N ₁₅ [km]	N ₁₆ [km]	N ₁₇ [km]	N ₁₈ [km]	N ₁₉ [km]	N ₂₀ [km]	N ₂₁ [km]	N ₂₂ [km]	N ₂₃ [km]	N ₂₄ [km]	N ₂₅ [km]	N ₂₆ [km]	N ₂₇ [km]	N ₂₈ [km]	N ₂₉ [km]	N ₃₀ [km]	N ₃₁ [km]	N ₃₂ [km]	N ₃₃ [km]	N ₃₄ [km]	N ₃₅ [km]	N ₃₆ [km]	N ₃₇ [km]	N ₃₈ [km]	N ₃₉ [km]	N ₄₀ [km]	N ₄₁ [km]	N ₄₂ [km]	N ₄₃ [km]	N ₄₄ [km]	N ₄₅ [km]	N ₄₆ [km]	N ₄₇ [km]	N ₄₈ [km]	N ₄₉ [km]	N ₅₀ [km]	N ₅₁ [km]	N ₅₂ [km]	N ₅₃ [km]	N ₅₄ [km]	N ₅₅ [km]	N ₅₆ [km]	N ₅₇ [km]	N ₅₈ [km]	N ₅₉ [km]	N ₆₀ [km]	N ₆₁ [km]	N ₆₂ [km]	N ₆₃ [km]	N ₆₄ [km]	N ₆₅ [km]	N ₆₆ [km]	N ₆₇ [km]	N ₆₈ [km]	N ₆₉ [km]	N ₇₀ [km]	N ₇₁ [km]	N ₇₂ [km]	N ₇₃ [km]	N ₇₄ [km]	N ₇₅ [km]	N ₇₆ [km]	N ₇₇ [km]	N ₇₈ [km]	N ₇₉ [km]	N ₈₀ [km]	N ₈₁ [km]	N ₈₂ [km]	N ₈₃ [km]	N ₈₄ [km]	N ₈₅ [km]	N ₈₆ [km]	N ₈₇ [km]	N ₈₈ [km]	N ₈₉ [km]	N ₉₀ [km]	N ₉₁ [km]	N ₉₂ [km]	N ₉₃ [km]	N ₉₄ [km]	N ₉₅ [km]	N ₉₆ [km]	N ₉₇ [km]	N ₉₈ [km]	N ₉₉ [km]	N ₁₀₀ [km]	N ₁₀₁ [km]	N ₁₀₂ [km]	N ₁₀₃ [km]	N ₁₀₄ [km]	N ₁₀₅ [km]	N ₁₀₆ [km]	N ₁₀₇ [km]	N ₁₀₈ [km]	N ₁₀₉ [km]	N ₁₁₀ [km]	N ₁₁₁ [km]	N ₁₁₂ [km]	N ₁₁₃ [km]	N ₁₁₄ [km]	N ₁₁₅ [km]	N ₁₁₆ [km]	N ₁₁₇ [km]	N ₁₁₈ [km]	N ₁₁₉ [km]	N ₁₂₀ [km]	N ₁₂₁ [km]	N ₁₂₂ [km]	N ₁₂₃ [km]	N ₁₂₄ [km]	N ₁₂₅ [km]	N ₁₂₆ [km]	N ₁₂₇ [km]	N ₁₂₈ [km]	N ₁₂₉ [km]	N ₁₃₀ [km]	N ₁₃₁ [km]	N ₁₃₂ [km]	N ₁₃₃ [km]	N ₁₃₄ [km]	N ₁₃₅ [km]	N ₁₃₆ [km]	N ₁₃₇ [km]	N ₁₃₈ [km]	N ₁₃₉ [km]	N ₁₄₀ [km]	N ₁₄₁ [km]	N ₁₄₂ [km]	N ₁₄₃ [km]	N ₁₄₄ [km]	N ₁₄₅ [km]	N ₁₄₆ [km]	N ₁₄₇ [km]	N ₁₄₈ [km]	N ₁₄₉ [km]	N ₁₅₀ [km]	N ₁₅₁ [km]	N ₁₅₂ [km]	N ₁₅₃ [km]	N ₁₅₄ [km]	N ₁₅₅ [km]	N ₁₅₆ [km]	N ₁₅₇ [km]	N ₁₅₈ [km]	N ₁₅₉ [km]	N ₁₆₀ [km]	N ₁₆₁ [km]	N ₁₆₂ [km]	N ₁₆₃ [km]	N ₁₆₄ [km]	N ₁₆₅ [km]	N ₁₆₆ [km]	N ₁₆₇ [km]	N ₁₆₈ [km]	N ₁₆₉ [km]	N ₁₇₀ [km]	N ₁₇₁ [km]	N ₁₇₂ [km]	N ₁₇₃ [km]	N ₁₇₄ [km]	N ₁₇₅ [km]	N ₁₇₆ [km]	N ₁₇₇ [km]	N ₁₇₈ [km]	N ₁₇₉ [km]	N ₁₈₀ [km]	N ₁₈₁ [km]	N ₁₈₂ [km]	N ₁₈₃ [km]	N ₁₈₄ [km]	N ₁₈₅ [km]	N ₁₈₆ [km]	N ₁₈₇ [km]	N ₁₈₈ [km]	N ₁₈₉ [km]	N ₁₉₀ [km]	N ₁₉₁ [km]	N ₁₉₂ [km]	N ₁₉₃ [km]	N ₁₉₄ [km]	N ₁₉₅ [km]	N ₁₉₆ [km]	N ₁₉₇ [km]	N ₁₉₈ [km]	N ₁₉₉ [km]	N ₂₀₀ [km]	N ₂₀₁ [km]	N ₂₀₂ [km]	N ₂₀₃ [km]	N ₂₀₄ [km]	N ₂₀₅ [km]	N ₂₀₆ [km]	N ₂₀₇ [km]	N ₂₀₈ [km]	N ₂₀₉ [km]	N ₂₁₀ [km]	N ₂₁₁ [km]	N ₂₁₂ [km]	N ₂₁₃ [km]	N ₂₁₄ [km]	N ₂₁₅ [km]	N ₂₁₆ [km]	N ₂₁₇ [km]	N ₂₁₈ [km]	N ₂₁₉ [km]	N ₂₂₀ [km]	N ₂₂₁ [km]	N ₂₂₂ [km]	N ₂₂₃ [km]	N ₂₂₄ [km]	N ₂₂₅ [km]	N ₂₂₆ [km]	N ₂₂₇ [km]	N ₂₂₈ [km]	N ₂₂₉ [km]	N ₂₃₀ [km]	N ₂₃₁ [km]	N ₂₃₂ [km]	N ₂₃₃ [km]	N ₂₃₄ [km]	N ₂₃₅ [km]	N ₂₃₆ [km]	N ₂₃₇ [km]	N ₂₃₈ [km]	N ₂₃₉ [km]	N ₂₄₀ [km]	N ₂₄₁ [km]	N ₂₄₂ [km]	N ₂₄₃ [km]	N ₂₄₄ [km]	N ₂₄₅ [km]	N ₂₄₆ [km]	N ₂₄₇ [km]	N ₂₄₈ [km]	N ₂₄₉ [km]	N ₂₅₀ [km]	N ₂₅₁ [km]	N ₂₅₂ [km]	N ₂₅₃ [km]	N ₂₅₄ [km]	N ₂₅₅ [km]	N ₂₅₆ [km]	N ₂₅₇ [km]	N ₂₅₈ [km]	N ₂₅₉ [km]	N ₂₆₀ [km]	N
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[illegible][illegible]

OVERVIEW ULS CHECKS FOR ALL INTERMEDIATE MEMBERS

[illegible]

Name	dx [m]	Cover	W ₁ [m]	W ₂ [m]	W ₃ [m]	W ₄ [m]	W ₅ [m]	W ₆ [m]	W ₇ [m]	W ₈ [m]	W ₉ [m]	W ₁₀ [m]	W ₁₁ [m]	W ₁₂ [m]	W ₁₃ [m]	W ₁₄ [m]	W ₁₅ [m]	W ₁₆ [m]	W ₁₇ [m]	W ₁₈ [m]	W ₁₉ [m]	W ₂₀ [m]	W ₂₁ [m]	W ₂₂ [m]	W ₂₃ [m]	W ₂₄ [m]	W ₂₅ [m]	W ₂₆ [m]	W ₂₇ [m]	W ₂₈ [m]	W ₂₉ [m]	W ₃₀ [m]	W ₃₁ [m]	W ₃₂ [m]	W ₃₃ [m]	W ₃₄ [m]	W ₃₅ [m]	W ₃₆ [m]	W ₃₇ [m]	W ₃₈ [m]	W ₃₉ [m]	W ₄₀ [m]	W ₄₁ [m]	W ₄₂ [m]	W ₄₃ [m]	W ₄₄ [m]	W ₄₅ [m]	W ₄₆ [m]	W ₄₇ [m]	W ₄₈ [m]	W ₄₉ [m]	W ₅₀ [m]	W ₅₁ [m]	W ₅₂ [m]	W ₅₃ [m]	W ₅₄ [m]	W ₅₅ [m]	W ₅₆ [m]	W ₅₇ [m]	W ₅₈ [m]	W ₅₉ [m]	W ₆₀ [m]	W ₆₁ [m]	W ₆₂ [m]	W ₆₃ [m]	W ₆₄ [m]	W ₆₅ [m]	W ₆₆ [m]	W ₆₇ [m]	W ₆₈ [m]	W ₆₉ [m]	W ₇₀ [m]	W ₇₁ [m]	W ₇₂ [m]	W ₇₃ [m]	W ₇₄ [m]	W ₇₅ [m]	W ₇₆ [m]	W ₇₇ [m]	W ₇₈ [m]	W ₇₉ [m]	W ₈₀ [m]	W ₈₁ [m]	W ₈₂ [m]	W ₈₃ [m]	W ₈₄ [m]	W ₈₅ [m]	W ₈₆ [m]	W ₈₇ [m]	W ₈₈ [m]	W ₈₉ [m]	W ₉₀ [m]	W ₉₁ [m]	W ₉₂ [m]	W ₉₃ [m]	W ₉₄ [m]	W ₉₅ [m]	W ₉₆ [m]	W ₉₇ [m]	W ₉₈ [m]	W ₉₉ [m]	W ₁₀₀ [m]	W ₁₀₁ [m]	W ₁₀₂ [m]	W ₁₀₃ [m]	W ₁₀₄ [m]	W ₁₀₅ [m]	W ₁₀₆ [m]	W ₁₀₇ [m]	W ₁₀₈ [m]	W ₁₀₉ [m]	W ₁₁₀ [m]	W ₁₁₁ [m]	W ₁₁₂ [m]	W ₁₁₃ [m]	W ₁₁₄ [m]	W ₁₁₅ [m]	W ₁₁₆ [m]	W ₁₁₇ [m]	W ₁₁₈ [m]	W ₁₁₉ [m]	W ₁₂₀ [m]	W ₁₂₁ [m]	W ₁₂₂ [m]	W ₁₂₃ [m]	W ₁₂₄ [m]	W ₁₂₅ [m]	W ₁₂₆ [m]	W ₁₂₇ [m]	W ₁₂₈ [m]	W ₁₂₉ [m]	W ₁₃₀ [m]	W ₁₃₁ [m]	W ₁₃₂ [m]	W ₁₃₃ [m]	W ₁₃₄ [m]	W ₁₃₅ [m]	W ₁₃₆ [m]	W ₁₃₇ [m]	W ₁₃₈ [m]	W ₁₃₉ [m]	W ₁₄₀ [m]	W ₁₄₁ [m]	W ₁₄₂ [m]	W ₁₄₃ [m]	W ₁₄₄ [m]	W ₁₄₅ [m]	W ₁₄₆ [m]	W ₁₄₇ [m]	W ₁₄₈ [m]	W ₁₄₉ [m]	W ₁₅₀ [m]	W ₁₅₁ [m]	W ₁₅₂ [m]	W ₁₅₃ [m]	W ₁₅₄ [m]	W ₁₅₅ [m]	W ₁₅₆ [m]	W ₁₅₇ [m]	W ₁₅₈ [m]	W ₁₅₉ [m]	W ₁₆₀ [m]	W ₁₆₁ [m]	W ₁₆₂ [m]	W ₁₆₃ [m]	W ₁₆₄ [m]	W ₁₆₅ [m]	W ₁₆₆ [m]	W ₁₆₇ [m]	W ₁₆₈ [m]	W ₁₆₉ [m]	W ₁₇₀ [m]	W ₁₇₁ [m]	W ₁₇₂ [m]	W ₁₇₃ [m]	W ₁₇₄ [m]	W ₁₇₅ [m]	W ₁₇₆ [m]	W ₁₇₇ [m]	W ₁₇₈ [m]	W ₁₇₉ [m]	W ₁₈₀ [m]	W ₁₈₁ [m]	W ₁₈₂ [m]	W ₁₈₃ [m]	W ₁₈₄ [m]	W ₁₈₅ [m]	W ₁₈₆ [m]	W ₁₈₇ [m]	W ₁₈₈ [m]	W ₁₈₉ [m]	W ₁₉₀ [m]	W ₁₉₁ [m]	W ₁₉₂ [m]	W ₁₉₃ [m]	W ₁₉₄ [m]	W ₁₉₅ [m]	W ₁₉₆ [m]	W ₁₉₇ [m]	W ₁₉₈ [m]	W ₁₉₉ [m]	W ₂₀₀ [m]	W ₂₀₁ [m]	W ₂₀₂ [m]	W ₂₀₃ [m]	W ₂₀₄ [m]	W ₂₀₅ [m]	W ₂₀₆ [m]	W ₂₀₇ [m]	W ₂₀₈ [m]	W ₂₀₉ [m]	W ₂₁₀ [m]	W ₂₁₁ [m]	W ₂₁₂ [m]	W ₂₁₃ [m]	W ₂₁₄ [m]	W ₂₁₅ [m]	W ₂₁₆ [m]	W ₂₁₇ [m]	W ₂₁₈ [m]	W ₂₁₉ [m]	W ₂₂₀ [m]	W ₂₂₁ [m]	W ₂₂₂ [m]	W ₂₂₃ [m]	W ₂₂₄ [m]	W ₂₂₅ [m]	W ₂₂₆ [m]	W ₂₂₇ [m]	W ₂₂₈ [m]	W ₂₂₉ [m]	W ₂₃₀ [m]	W ₂₃₁ [m]	W ₂₃₂ [m]	W ₂₃₃ [m]	W ₂₃₄ [m]	W ₂₃₅ [m]	W ₂₃₆ [m]	W ₂₃₇ [m]	W ₂₃₈ [m]	W ₂₃₉ [m]	W ₂₄₀ [m]	W ₂₄₁ [m]	W ₂₄₂ [m]	W ₂₄₃ [m]	W ₂₄₄ [m]	W ₂₄₅ [m]	W ₂₄
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OVERVIEW ULS CHECKS FOR ALL INTERMEDIATE MEMBER

[illegible]

OVERVIEW ULS CHECKS FOR ALL INTERMEDIATE MEMBER

[illegible]

6. ULS 1.8 BOTTOM MEMBERS**6.1. 1D internal forces; N**

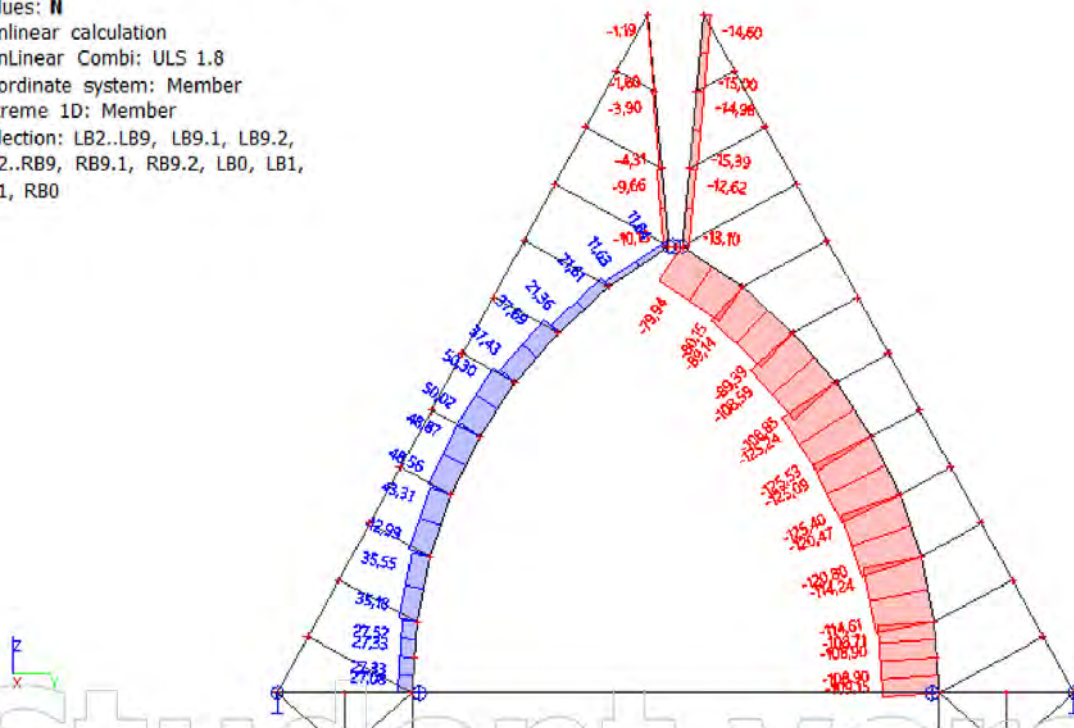
Values: N

Nonlinear calculation

NonLinear Combi: ULS 1.8

Coordinate system: Member

Extreme 1D: Member

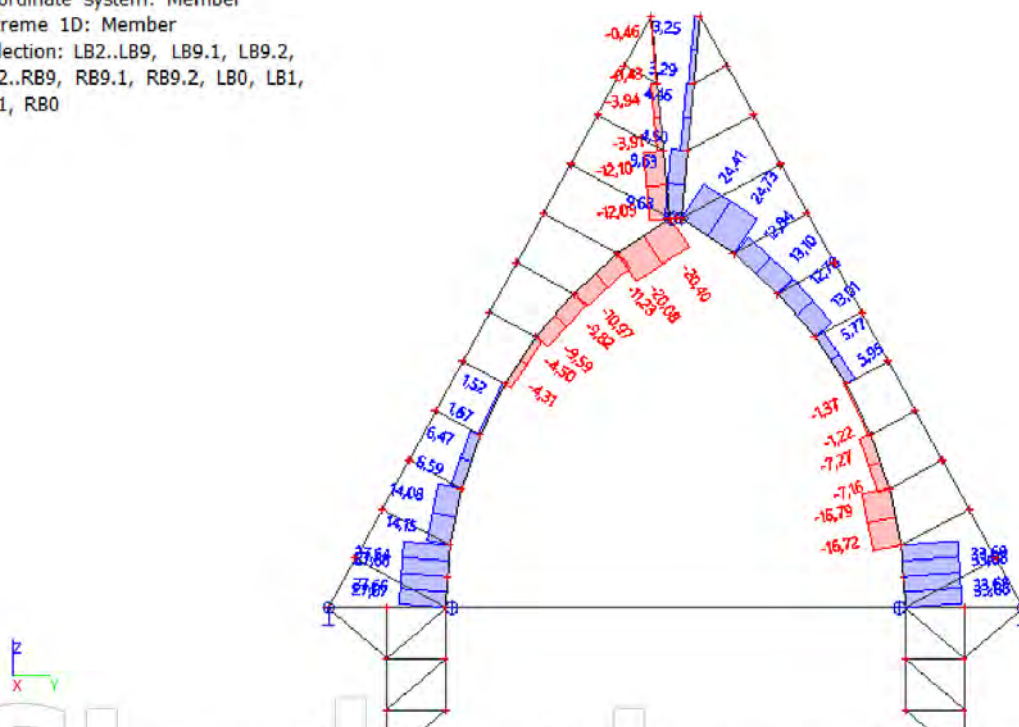
Selection: LB2..LB9, LB9.1, LB9.2,
RB2..RB9, RB9.1, RB9.2, LB0, LB1,
RB1, RB0**6.2. 1D internal forces; V_z**Values: V_z

Nonlinear calculation

NonLinear Combi: ULS 1.8

Coordinate system: Member

Extreme 1D: Member

Selection: LB2..LB9, LB9.1, LB9.2,
RB2..RB9, RB9.1, RB9.2, LB0, LB1,
RB1, RB0

6.3. 1D internal forces; M_y

Values: M_y

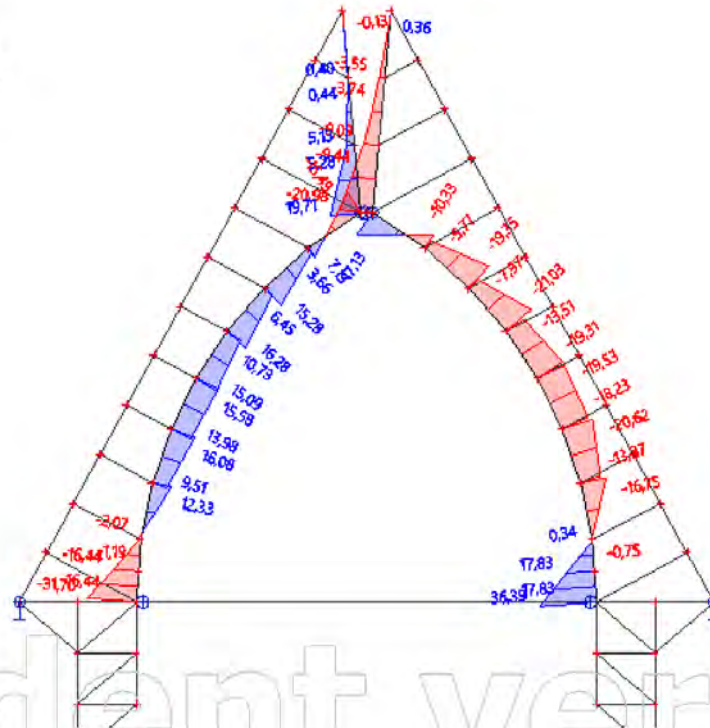
Nonlinear calculation

NonLinear Combi: ULS 1.8

Coordinate system: Member

Extreme 1D: Member

Selection: LB2..LB9, LB9.1, LB9.2,
RB2..RB9, RB9.1, RB9.2, LB0, LB1,
RB1, RB0



6.4. EXAMPLE CALCULATION BOTTOM MEMBER RB0

EXAMPLE CALCULATION MEMBER RB0 (grain angle 25.8°)

GEOMETRICAL AND MATERIAL PROPERTIES

width
height
cross-section propertieskmod instantaneous (ULS 1.8)
safety factor LVL

$$\alpha := 25.8 \cdot \left(\frac{\pi}{180} \right) = 0.45$$

$$b := 160 \text{ mm}$$

$$h := 400 \text{ mm}$$

$$A := b \cdot h = (6.4 \cdot 10^4) \text{ mm}^2$$

$$W := \left(\frac{1}{6} \right) \cdot b \cdot h^2 = (4.267 \cdot 10^6) \text{ mm}^3$$

$$k_{mod} := 1.1$$

$$\gamma_m := 1.3$$

BAUBUCHE-Q PANELS 25.8° ANGLE TO GRAIN IN MEMBERS:

$$f_{Q.m.0.k} := 59 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.m.90.k} := 9 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.c.0.k} := 62 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.c.90.k} := 22 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.v.k} := 7.8 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.m.0.d} := k_{mod} \cdot \frac{f_{Q.m.0.k}}{\gamma_m} = 49.923 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.m.90.d} := k_{mod} \cdot \frac{f_{Q.m.90.k}}{\gamma_m} = 7.615 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.c.0.d} := k_{mod} \cdot \frac{f_{Q.c.0.k}}{\gamma_m} = 52.462 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.c.90.d} := k_{mod} \cdot \frac{f_{Q.c.90.k}}{\gamma_m} = 18.615 \frac{\text{N}}{\text{mm}^2}$$

$$f_{Q.v.d} := k_{mod} \cdot \frac{f_{Q.v.k}}{\gamma_m} = 6.6 \frac{\text{N}}{\text{mm}^2}$$

FORCES IN INTERMEDIATE MEMBER

$$N_d := -109 \text{ kN} \quad V_{z,d} := -33.7 \text{ kN} \quad M_d := 36.39 \text{ kN} \cdot \text{m}$$

COMPRESSION AT AN ANGLE 25.8° CHECK: $\sigma_{c,\alpha,d} \leq k_{\alpha,c} \cdot f_{Q.c.0,d}$

$$\text{stress} \quad \sigma_{c,\alpha,d} := \frac{N_d}{A} = -1.703 \frac{\text{N}}{\text{mm}^2}$$

$$\text{resistance} \quad k_{\alpha,c} := \frac{1}{\left(\frac{f_{Q.c.0,d}}{f_{Q.c.90,d}} \right) \cdot \sin(\alpha)^2 + \cos(\alpha)^2} = 0.744 \quad k_{\alpha,c} \cdot f_{Q.c.0,d} = 39.022 \frac{\text{N}}{\text{mm}^2}$$

$$\text{UC} \quad \left| \frac{\sigma_{c,\alpha,d}}{k_{\alpha,c} \cdot f_{Q.c.0,d}} \right| = 0.044$$

$$\text{OK!}$$

BENDING AT AN ANGLE 25.8° CHECK: $\sigma_{m,\alpha,d} \leq k_{\alpha,m} \cdot f_{Q.m.0,d}$

$$\text{stress} \quad \sigma_{m,\alpha,d} := \frac{M_d}{W} = 8.529 \frac{\text{N}}{\text{mm}^2}$$

$$\text{resistance} \quad k_{\alpha,m} := \frac{1}{\frac{f_{Q.m.0,d}}{f_{Q.m.90,d}} \cdot \sin(\alpha)^2 + \frac{f_{Q.m.0,d}}{f_{Q.v,d}} \cdot \sin(\alpha) \cdot \cos(\alpha) + \cos(\alpha)^2} = 0.199$$

$$\text{UC} \quad \left| \frac{\sigma_{m,\alpha,d}}{k_{\alpha,m} \cdot f_{Q.m.0,d}} \right| = 0.857$$

$$\text{OK!} \quad k_{\alpha,m} \cdot f_{Q.m.0,d} = 9.952 \frac{\text{N}}{\text{mm}^2}$$

COMBINED COMPRESSION AND BENDING 25.8°

$$\text{OK!} \quad \left(\frac{\sigma_{c,\alpha,d}}{k_{\alpha,c} \cdot f_{Q.c.0,k}} \right)^2 + \frac{\sigma_{m,\alpha,d}}{k_{\alpha,m} \cdot f_{Q.m.0,d}} \leq 1 = 1$$

OVERVIEW ULS CHECKS FOR ALL BOTTOM MEMBERS

										STRESSES				ANGLE FACTOR		MATERIAL STRENGTH				CHECKS			COMBINATIONS		
BOTTOM MEMBERS LEFT	N	Vz	My	b [m]	h [m]	A [m²]	Wp[m³]	Iy [m⁴]	σ.c.d	σ.t.d	σ.m.d	t.max	α	k.a.c	k.a.t	k.a.m	f _{0.2,c.d} k.a.c	f _{0.2,t.d} k.a.t	f _{0.2,m.d} k.a.m	compression	tension	bending	comp.&bend.	tension&bend.	
L80	U.S.1.3	35.52	31.85	-19.48	0.16	0.4	0.064	0.004267	0.000853333	0.00	0.55	-4.87	0.75	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.06	0.87	0.87	0.93
L81	U.S.1.3	35.77	31.87	-37.05	0.16	0.4	0.064	0.004267	0.000853333	0.00	0.74	-0.77	0.99	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.08	0.08	0.08	0.16
L82	U.S.1.3	35.52	31.85	-19.48	0.16	0.3	0.048	0.0024	0.00036	0.00	0.74	-8.12	1.00	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.08	0.82	0.82	0.90
L83	U.S.1.3	44.68	16.34	13.76	0.16	0.3	0.048	0.0024	0.00036	0.00	0.93	5.73	0.51	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.08	0.44	0.44	0.52
L84	U.S.1.3	44.32	16.41	-13.76	0.16	0.3	0.048	0.0024	0.00036	0.00	0.92	-5.73	0.51	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.08	0.44	0.44	0.52
L85	U.S.1.3	53.65	7.82	18.32	0.16	0.3	0.048	0.0024	0.00036	0.00	1.12	7.63	0.24	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.07	0.40	0.40	0.46
L86	U.S.1.3	53.32	7.94	10.4	0.16	0.3	0.048	0.0024	0.00036	0.00	1.11	4.33	0.25	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.07	0.22	0.22	0.29
L87	U.S.1.3	60.46	2.37	18.14	0.16	0.3	0.048	0.0024	0.00036	0.00	1.26	7.56	0.07	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.05	0.23	0.23	0.28
L88	U.S.1.3	60.16	2.52	15.08	0.16	0.3	0.048	0.0024	0.00036	0.00	1.25	6.53	0.08	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.05	0.20	0.20	0.25
L89	U.S.1.3	62.37	3.53	12.91	0.16	0.3	0.048	0.0024	0.00036	0.00	1.32	5.38	-0.14	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.00	0.05	0.15	0.15	0.20
L89.1	U.S.1.3	62.99	-4.14	17.1	0.16	0.3	0.048	0.0024	0.00036	0.00	1.31	7.13	-0.13	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.00	0.05	0.20	0.20	0.25
L89.2	U.S.1.3	50.87	-10.62	7.77	0.16	0.3	0.048	0.0024	0.00036	0.00	1.06	3.24	-0.33	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.00	0.06	0.17	0.17	0.24
L89.3	U.S.1.3	50.6	-10.39	18.42	0.16	0.3	0.048	0.0024	0.00036	0.00	1.05	7.68	-0.32	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.00	0.06	0.41	0.41	0.48
L87	U.S.1.3	33.72	-13.21	4.54	0.16	0.3	0.048	0.0024	0.00036	0.00	0.70	1.89	-0.38	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.00	0.06	0.14	0.14	0.19
L87	U.S.1.3	33.48	-12.05	17.3	0.16	0.3	0.048	0.0024	0.00036	0.00	0.70	7.21	-0.38	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.00	0.06	0.53	0.53	0.58
L88	U.S.1.3	23.24	-23.08	-17.24	0.16	0.3	0.048	0.0024	0.00036	0.00	0.48	-7.18	-0.72	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.00	0.05	0.73	0.73	0.79
L88	U.S.1.3	23.02	-22.77	8.39	0.16	0.3	0.048	0.0024	0.00036	0.00	0.48	3.50	-0.71	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.00	0.05	0.36	0.36	0.41
L89	U.S.1.3	-6.76	-12.93	6.02	0.16	0.35	0.056	0.003267	0.000571667	-0.16	0.00	1.84	-0.35	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.04	0.24	0.24	0.24
L89.1	U.S.1.3	-9.24	-12.88	21.44	0.16	0.35	0.056	0.003267	0.000571667	-0.17	0.00	6.56	-0.35	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.84	0.84	0.84
L89.2	U.S.1.3	-2.73	-4.3	0.73	0.16	0.3	0.048	0.0024	0.00036	-0.06	0.00	0.30	-0.13	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.04	0.04	0.04	0.04
L89.3	U.S.1.3	-3.14	-4.26	5.84	0.16	0.3	0.048	0.0024	0.00036	-0.06	0.00	2.43	-0.13	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.01	0.31	0.31	0.31
L89.2	U.S.1.3	-0.02	-0.72	-0.16	0.16	0.3	0.048	0.0024	0.00036	0.00	0.00	-0.07	-0.02	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.01	0.01	0.01
L89.2	U.S.1.3	-0.42	-0.68	0.68	0.16	0.3	0.048	0.0024	0.00036	-0.01	0.00	0.28	-0.02	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.04	0.04	0.04	0.04
R80	U.S.1.3	-105.9	30.63	32.09	0.16	0.4	0.064	0.004267	0.000853333	-1.65	0.00	7.52	0.72	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.06	0.76	0.76	0.76
R80	U.S.1.3	-105.65	30.65	15.2	0.16	0.4	0.064	0.004267	0.000853333	-1.65	0.00	3.56	0.72	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.06	0.36	0.36	0.36
R81	U.S.1.3	-105.65	30.65	15.2	0.16	0.3	0.048	0.0024	0.00036	-2.00	0.00	0.63	0.96	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.06	0.04	0.64	0.64	0.64
R82	U.S.1.3	-105.46	30.67	-1.71	0.16	0.3	0.048	0.0024	0.00036	-2.20	0.00	0.71	0.96	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.06	0.07	0.07	0.07	0.07
R83	U.S.1.3	-110.21	-14.98	-16	0.16	0.3	0.048	0.0024	0.00036	-2.30	0.00	-6.67	-0.47	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.05	0.02	0.52	0.52	0.52
R83	U.S.1.3	-110.58	-14.91	-0.76	0.16	0.3	0.048	0.0024	0.00036	-2.30	0.00	-0.32	-0.47	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.05	0.00	0.02	0.02	0.02
R83	U.S.1.3	-115.27	-5.96	-19.05	0.16	0.3	0.048	0.0024	0.00036	-2.40	0.00	-7.94	-0.19	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.05	0.04	0.41	0.41	0.41
R83	U.S.1.3	-115.6	-5.84	-13.12	0.16	0.3	0.048	0.0024	0.00036	-2.41	0.00	-5.47	-0.18	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.05	0.00	0.28	0.28	0.28
R84	U.S.1.3	-118.51	-0.3	-17.46	0.16	0.3	0.048	0.0024	0.00036	-2.47	0.00	-7.28	-0.01	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.02	0.22	0.22	0.22
R84	U.S.1.3	-118.82	-0.15	-17.24	0.16	0.3	0.048	0.0024	0.00036	-2.48	0.00	-7.18	0.00	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.05	0.02	0.22	0.22	0.22
R85	U.S.1.3	-116.77	6.45	-11.51	0.16	0.3	0.048	0.0024	0.00036	-2.43	0.00	-4.80	0.20	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.05	0.04	0.14	0.14	0.14
R85	U.S.1.3	-117.06	6.44	-11.99	0.16	0.3	0.048	0.0024	0.00036	-2.44	0.00	-7.50	0.21	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.05	0.02	0.21	0.21	0.21
R86	U.S.1.3	-98.56	12.65	-6.75	0.16	0.3	0.048	0.0024	0.00036	-2.05	0.00	-2.81	0.40	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.04	0.00	0.15	0.15	0.15
R86	U.S.1.3	-98.82	12.88	-19.68	0.16	0.3	0.048	0.0024	0.00036	-2.06	0.00	-8.20	0.40	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.04	0.00	0.44	0.44	0.44
R87	U.S.1.3	-79.1	12.36	-4.97	0.16	0.3	0.048	0.0024	0.00036	-1.65	0.00	-0.27	0.39	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.04	0.05	0.15	0.15	0.15
R87	U.S.1.3	-79.35	12.62	-18.05	0.16	0.3	0.048	0.0024	0.00036	-1.65	0.00	-7.52	0.39	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.04	0.05	0.15	0.15	0.15
R88	U.S.1.3	-70.19	22.65	16.14	0.16	0.3	0.048	0.0024	0.00036	-1.46	0.00	6.73	0.71	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.04	0.09	0.69	0.69	0.69
R88	U.S.1.3	-70.04	22.97	-9.36	0.16	0.3	0.048	0.0024	0.00036	-1.47	0.00	-3.90	0.72	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.04	0.00	0.40	0.40	0.40
R89	U.S.1.3	-12.23	9.18	-0.99	0.16	0.35	0.056	0.003267	0.000571667	-0.22	0.00	-2.78	0.25	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.36	0.36	0.36
R89.1	U.S.1.3	-12.71	9.23	-20.09	0.16	0.35	0.056	0.003267	0.000571667	-0.23	0.00	-6.15	0.25	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.78	0.78	0.78
R89.1	U.S.1.3	-14.42	4.29	-3.61	0.16	0.3	0.048	0.0024	0.00036	-0.30	0.00	-1.50	0.13	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.19	0.19	0.19
R89.2	U.S.1.3	-14.83	4.33	-8.76	0.16	0.3	0.048	0.0024	0.00036	-0.31	0.00	-3.65	0.14	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.47	0.47	0.47
R89.2	U.S.1.3	-14.03	3.13	0.34	0.16	0.3	0.048	0.0024	0.00036	-0.29															

OVERVIEW ULS CHECKS FOR ALL BOTTOM MEMBERS

												STRESSES				ANGLE FACTOR				MATERIAL STRENGTH				CHECKS				COMBINATIONS				
BOTTOM MEMBERS LEFT																																
N	Vz	My	b [m]	h [m]	A [m²]	Wip[3]	yy [m⁴]	σ.c.d	σ.t.d	σ.m.d	t.max	α	k.a.c	k.a.t	k.a.m	f0.σ.c⁴k.a.c	f0.σ.c⁴k.a.t	f0.σ.m⁴k.a.m	compression	tension	bending	comp.& bend.	tension & bend.									
L80	U5.1.6	-10.16	-4.37	3.42	0.16	0.4	0.064	0.004267	0.000853333	-0.16	0.00	0.80	-0.10	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.08	0.08	0.14	0.00	9.19	9.93	0.01	0.04	0.04	0.14	
L80	U5.1.6	-10.43	-4.37	5.83	0.16	0.4	0.064	0.004267	0.000853333	-0.16	0.00	1.37	-0.10	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.08	0.08	0.14	0.00	9.19	9.93	0.01	0.04	0.04	0.14	
L81	U5.1.6	-9.88	-4.4	1	0.16	0.3	0.048	0.0024	0.00036	-0.21	0.00	0.42	-0.14	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.01	0.04	0.04	0.04	0.00	9.19	9.93	0.01	0.04	0.04	0.04	
L81	U5.1.6	-10.17	-4.39	3.42	0.16	0.3	0.048	0.0024	0.00036	-0.21	0.00	1.43	-0.14	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.01	0.04	0.04	0.04	0.00	9.19	9.93	0.01	0.04	0.04	0.14	
L82	U5.1.6	-10.52	-2.43	-1.29	0.16	0.3	0.048	0.0024	0.00036	-0.22	0.00	-0.54	-0.08	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.04	0.04	0.04	0.00	11.77	12.93	0.01	0.04	0.04	0.04	
L82	U5.1.6	-10.89	-2.36	1.15	0.16	0.3	0.048	0.0024	0.00036	-0.23	0.00	-0.48	-0.07	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.01	0.04	0.04	0.04	0.00	11.77	12.93	0.01	0.04	0.04	0.04	
L83	U5.1.6	-11.51	-1.61	-2.22	0.16	0.3	0.048	0.0024	0.00036	-0.24	0.00	-0.93	-0.05	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.05	0.05	0.05	0.00	17.09	19.27	0.01	0.05	0.05	0.05	
L83	U5.1.6	-11.84	-1.5	-0.66	0.16	0.3	0.048	0.0024	0.00036	-0.25	0.00	-0.28	-0.05	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.01	0.01	0.01	0.01	0.00	17.09	19.27	0.01	0.01	0.01	0.01	
L84	U5.1.6	-12.77	-1.23	-2.69	0.16	0.3	0.048	0.0024	0.00036	-0.27	0.00	-1.12	-0.04	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.01	0.03	0.03	0.03	0.00	26.88	32.45	0.01	0.03	0.03	0.03	
L84	U5.1.6	-13.08	-1.08	-1.54	0.16	0.3	0.048	0.0024	0.00036	-0.27	0.00	-0.64	-0.03	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.01	0.02	0.02	0.02	0.00	26.88	32.45	0.01	0.02	0.02	0.02	
L85	U5.1.6	-14.55	-0.68	-2.89	0.16	0.3	0.048	0.0024	0.00036	-0.30	0.00	-1.02	-0.02	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.01	0.03	0.03	0.03	0.00	28.90	35.40	0.01	0.03	0.03	0.03	
L85	U5.1.6	-14.84	-0.5	-1.86	0.16	0.3	0.048	0.0024	0.00036	-0.31	0.00	-0.78	-0.02	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.01	0.02	0.02	0.02	0.00	28.90	35.40	0.01	0.02	0.02	0.02	
L86	U5.1.6	-15.69	0.32	-1.42	0.16	0.3	0.048	0.0024	0.00036	-0.33	0.00	-0.59	0.01	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.01	0.03	0.03	0.03	0.00	16.55	18.62	0.01	0.03	0.03	0.03	
L86	U5.1.6	-15.96	0.55	-1.86	0.16	0.3	0.048	0.0024	0.00036	-0.33	0.00	-0.78	0.02	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.01	0.04	0.04	0.04	0.00	16.55	18.62	0.01	0.04	0.04	0.04	
L87	U5.1.6	-15.13	0.58	-0.95	0.16	0.3	0.048	0.0024	0.00036	-0.32	0.00	-0.40	0.02	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.01	0.03	0.03	0.03	0.00	12.49	13.73	0.01	0.03	0.03	0.03	
L87	U5.1.6	-15.37	0.84	-1.69	0.16	0.3	0.048	0.0024	0.00036	-0.32	0.00	-0.70	0.03	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.01	0.05	0.05	0.05	0.00	12.49	13.73	0.01	0.05	0.05	0.05	
L88	U5.1.6	-14.5	2.02	1.23	0.16	0.3	0.048	0.0024	0.00036	-0.30	0.00	0.51	0.06	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.00	0.05	0.05	0.05	0.00	9.07	9.78	0.00	0.05	0.05	0.05	
L88	U5.1.6	-14.71	2.34	-1.21	0.16	0.3	0.048	0.0024	0.00036	-0.31	0.00	-0.50	0.07	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.01	0.05	0.05	0.05	0.00	9.07	9.78	0.01	0.05	0.05	0.05	
L89	U5.1.6	-1.91	0.47	-0.55	0.16	0.35	0.056	0.003267	0.000571667	-0.03	0.00	-0.17	0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.02	0.02	0.02	0.00	7.29	7.84	0.00	0.02	0.02	0.02	
L89	U5.1.6	-2.39	0.52	-1.14	0.16	0.35	0.056	0.003267	0.000571667	-0.04	0.00	-0.35	0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.04	0.04	0.04	0.00	7.29	7.84	0.00	0.04	0.04	0.04	
L89.1	U5.1.6	-1.56	0.2	-0.25	0.16	0.3	0.048	0.0024	0.00036	-0.03	0.00	-0.10	0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.01	0.01	0.01	0.00	7.29	7.84	0.00	0.01	0.01	0.01	
L89.1	U5.1.6	-1.97	0.24	-0.51	0.16	0.3	0.048	0.0024	0.00036	-0.04	0.00	-0.21	0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.03	0.03	0.03	0.00	7.29	7.84	0.00	0.03	0.03	0.03	
L89.2	U5.1.6	0	1.06	0.18	0	0.16	0.3	0.048	0.0024	0.00036	-0.02	0.00	0.00	0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.00	0.00	0.00	7.29	7.84	0.00	0.00	0.00	0.00
L89.2	U5.1.6	-1.47	0.22	-0.24	0.16	0.3	0.048	0.0024	0.00036	-0.03	0.00	-0.10	0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.01	0.01	0.01	0.00	7.29	7.84	0.00	0.01	0.01	0.01	
R80	U5.1.6	-10.43	4.37	5.83	0.16	0.4	0.064	0.004267	0.000853333	-0.16	0.00	1.37	0.10	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.14	0.14	0.14	0.00	9.19	9.93	0.00	0.14	0.14	0.14	
R80	U5.1.6	-10.17	4.39	3.42	0.16	0.4	0.064	0.004267	0.000853333	-0.16	0.00	0.80	0.10	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.08	0.08	0.08	0.00	9.19	9.93	0.00	0.08	0.08	0.08	
R81	U5.1.7	-10.16	4.39	3.42	0.16	0.3	0.048	0.0024	0.00036	-0.21	0.00	1.40	-0.11	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.14	0.14	0.14	0.00	9.19	9.93	0.00	0.14	0.14	0.14	
R81	U5.1.6	-9.88	4.4	1	0.16	0.3	0.048	0.0024	0.00036	-0.21	0.00	0.42	-0.14	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.01	0.04	0.04	0.04	0.00	9.19	9.93	0.01	0.04	0.04	0.04	
R82	U5.1.6	-10.52	-2.43	-1.29	0.16	0.3	0.048	0.0024	0.00036	-0.22	0.00	-0.54	-0.08	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.04	0.04	0.04	0.00	11.77	12.93	0.01	0.04	0.04	0.04	
R82	U5.1.6	-10.89	-2.36	1.15	0.16	0.3	0.048	0.0024	0.00036	-0.23	0.00	-0.48	-0.07	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.01	0.04	0.04	0.04	0.00	11.77	12.93	0.01	0.04	0.04	0.04	
R82	U5.1.6	-11.51	-1.61	-2.22	0.16	0.3	0.048	0.0024	0.00036	-0.24	0.00	-0.93	-0.05	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.05	0.05	0.05	0.00	17.09	19.27	0.01	0.05	0.05	0.05	
R83	U5.1.6	-11.84	-1.5	-0.66	0.16	0.3	0.048	0.0024	0.00036	-0.25	0.00	-0.28	-0.05	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.01	0.01	0.01	0.01	0.00	17.09	19.27	0.01	0.01	0.01	0.01	
R84	U5.1.6	-12.77	-1.23	-2.69	0.16																											

OVERVIEW ULS CHECKS FOR ALL BOTTOM MEMBERS

												STRESSES		ANGLE FACTOR		MATERIAL STRENGTH				CHECKS		COMBINATIONS		
BOTTOM MEMBERS LEFT	N	Vz	My	b [m]	h [m]	A [mm ²]	Wp [m ³]	Iyy [mm ⁴]	e.c.d	e.t.d	e.m.d	t.max	a	k.a.c	k.a.t	k.a.m	f _{0.6} σ _d *k.a.c	f _{0.6} σ _d *k.a.t	f _{0.6} σ _d *k.a.m	compression	tension	bending	comp. & bend.	tension & bend.
L80 ULS1.9	26.52	27.4	-16.23	0.16	0.4	0.064	0.004267	0.000853333	0.00	0.41	-3.80	0.64	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.05	0.38	0.38	0.43
L80 ULS1.9	-26.52	27.42	-31.35	0.16	0.4	0.064	0.004267	0.000853333	-0.43	0.00	-5.12	0.51	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.04	0.74	0.74	0.92
L81 ULS1.9	26.74	27.42	-1.13	0.16	0.3	0.048	0.0024	0.00036	0.00	0.50	-0.47	0.86	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.05	0.20	0.05	0.18
L81 ULS1.9	26.52	27.4	-16.23	0.16	0.3	0.048	0.0024	0.00036	0.00	0.55	-6.76	0.86	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.06	0.68	0.68	0.68
L82 ULS1.9	34.78	13.95	12.27	0.16	0.3	0.048	0.0024	0.00036	0.00	0.72	5.11	0.44	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.06	0.40	0.40	0.46
L82 ULS1.9	-34.78	14.01	-12.27	0.16	0.3	0.048	0.0024	0.00036	-0.00	0.72	-5.11	0.44	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.06	0.06	0.06	0.13
L83 ULS1.9	42.53	6.39	15.98	0.16	0.3	0.048	0.0024	0.00036	0.00	0.89	6.66	0.20	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.05	0.35	0.35	0.40
L83 ULS1.9	42.16	6.52	9.49	0.16	0.3	0.048	0.0024	0.00036	0.00	0.88	3.95	0.20	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.05	0.21	0.21	0.26
L84 ULS1.9	48.06	1.44	15.45	0.16	0.3	0.048	0.0024	0.00036	0.00	1.00	6.44	0.05	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.04	0.20	0.20	0.24
L84 ULS1.9	47.7	1.62	13.92	0.16	0.3	0.048	0.0024	0.00036	0.00	0.99	5.80	0.05	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.04	0.18	0.18	0.22
L85 ULS1.9	49.41	0.56	10.51	0.16	0.3	0.048	0.0024	0.00036	0.00	1.03	4.42	-0.14	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.00	0.04	0.12	0.12	0.16
L85 ULS1.9	49.08	-4.35	15.02	0.16	0.3	0.048	0.0024	0.00036	0.00	1.02	6.26	-0.14	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.00	0.04	0.18	0.18	0.21
L86 ULS1.9	36.72	-9.83	6.39	0.16	0.3	0.048	0.0024	0.00036	0.00	0.77	2.66	-0.31	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.00	0.05	0.14	0.14	0.19
L86 ULS1.9	36.42	-9.57	16.21	0.16	0.3	0.048	0.0024	0.00036	0.00	0.76	6.75	-0.30	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.00	0.05	0.36	0.36	0.41
L87 ULS1.9	20.65	-11.25	3.61	0.16	0.3	0.048	0.0024	0.00036	0.00	0.43	1.50	-0.35	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.00	0.03	0.11	0.11	0.14
L87 ULS1.9	20.37	-10.94	15.23	0.16	0.3	0.048	0.0024	0.00036	0.00	0.42	6.35	-0.34	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.00	0.03	0.46	0.46	0.50
L88 ULS1.9	10.91	-20.37	-15.48	0.16	0.3	0.048	0.0024	0.00036	0.00	0.23	-6.45	-0.64	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.00	0.03	0.66	0.66	0.68
L88 ULS1.9	10.67	-20	7.08	0.16	0.3	0.048	0.0024	0.00036	0.00	0.22	2.95	-0.63	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.00	0.02	0.30	0.30	0.33
L89 ULS1.9	-9.87	-1.21	5.26	0.16	0.35	0.056	0.003267	0.000571667	-0.18	0.00	1.61	-0.32	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.01	0.01	0.01
L89 ULS1.9	-10.42	-12.05	19.68	0.16	0.35	0.056	0.003267	0.000571667	-0.19	0.00	6.02	-0.32	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.77	0.77	0.77
L89.1 ULS1.9	-0.02	-3.94	0.42	0.16	0.3	0.048	0.0024	0.00036	-0.08	0.00	0.18	-0.12	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.02	0.02	0.02
L89.1 ULS1.9	-4.5	-3.9	5.11	0.16	0.3	0.048	0.0024	0.00036	-0.09	0.00	2.13	-0.12	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.27	0.27	0.27
L89.2 ULS1.9	-0.25	-0.46	-0.13	0.16	0.3	0.048	0.0024	0.00036	-0.03	0.00	-0.05	-0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.01	0.01	0.01
L89.2 ULS1.9	-1.72	-0.41	0.39	0.16	0.3	0.048	0.0024	0.00036	-0.04	0.00	0.16	-0.01	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.02	0.02	0.02
R80 ULS1.9	-108.36	33.52	36.39	0.16	0.4	0.064	0.004267	0.000853333	-1.69	0.00	8.53	0.79	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.04	0.00	0.86	0.86	0.93
R80 ULS1.9	-108.07	33.55	17.9	0.16	0.4	0.064	0.004267	0.000853333	-1.69	0.00	4.20	0.79	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.04	0.02	0.42	0.42	0.47
R81 ULS1.9	-108.07	33.55	17.9	0.16	0.3	0.048	0.0024	0.00036	-2.25	0.00	7.46	1.05	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.06	0.05	0.75	0.75	0.75
R81 ULS1.9	-107.85	33.56	-0.16	0.16	0.3	0.048	0.0024	0.00036	-1.00	0.00	-0.25	0.86	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.00	0.04	0.05	0.05	0.05
R82 ULS1.9	-113.32	-16.77	-16.58	0.16	0.3	0.048	0.0024	0.00036	-2.36	0.00	-6.91	-0.52	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.00	0.04	0.53	0.53	0.53
R82 ULS1.9	-113.74	-16.69	0.48	0.16	0.3	0.048	0.0024	0.00036	-2.37	0.00	0.20	-0.52	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.05	0.00	0.02	0.02	0.04
R83 ULS1.9	-119.55	-7.35	-20.15	0.16	0.3	0.048	0.0024	0.00036	-2.49	0.00	-0.54	-0.23	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.04	0.29	0.29	0.33
R83 ULS1.9	-119.92	-7.22	-13.17	0.16	0.3	0.048	0.0024	0.00036	-2.50	0.00	-5.49	-0.20	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.00	0.04	0.31	0.31	0.38
R84 ULS1.9	-124.24	-1.52	-19.5	0.16	0.3	0.048	0.0024	0.00036	-2.59	0.00	-8.13	-0.05	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.05	0.25	0.25	0.25
R84 ULS1.9	-124.6	1.35	-18.06	0.16	0.3	0.048	0.0024	0.00036	-2.60	0.00	-7.53	-0.04	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.05	0.00	0.23	0.23	0.23
R85 ULS1.9	-124.64	5.54	-13.57	0.16	0.3	0.048	0.0024	0.00036	-2.60	0.00	-5.65	0.17	3.0	1.00	0.76	0.71	-52.04	28.90	35.40	0.05	0.16	0.16	0.16	0.16
R85 ULS1.9	-124.97	5.56	-10.51	0.16	0.3	0.048	0.0024	0.00036	-2.60	0.00	-7.88	0.18	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.05	0.16	0.12	0.12	0.12
R86 ULS1.9	-108.52	12.54	-8	0.16	0.3	0.048	0.0024	0.00036	-2.26	0.00	-3.33	0.39	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.05	0.00	0.18	0.18	0.18
R86 ULS1.9	-108.82	12.18	-20.84	0.16	0.3	0.048	0.0024	0.00036	-2.27	0.00	-8.68	0.40	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.05	0.00	0.47	0.47	0.47
R87 ULS1.9	-89.41	12.63	-5.79	0.16	0.3	0.048	0.0024	0.00036	-1.86	0.00	-2.41	0.39	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.04	0.00	0.18	0.18	0.18
R87 ULS1.9	-89.69	12.99	-19.17	0.16	0.3	0.048	0.0024	0.00036	-1.87	0.00	-0.99	-0.40	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.04	0.00	0.58	0.58	0.59
R88 ULS1.9	-80.32	24.13	16.89	0.16	0.3	0.048	0.0024	0.00036	-1.67	0.00	7.04	0.75	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.04	0.00	0.72	0.72	0.72
R88 ULS1.9	-80.57	24.5	-10.28	0.16	0.3	0.048	0.0024	0.00036	-1.68	0.00	-4.27	0.63	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.04	0.00	0.44	0.44	0.44
R89 ULS1.9	-12.67	9.51	-9.33	0.16	0.35	0.056	0.003267	0.000571667	-0.20	0.00	-2.86	0.25	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.36	0.36	0.36
R89 ULS1.9	-12.22	9.56	-20.72	0.16	0.35	0.056	0.003267	0.000571667	-0.24	0.00	-6.34	0.26	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.81	0.81	0.81
R89.1 ULS1.9	-14.92	4.4	-3.7	0.16	0.3	0.048	0.0024	0.00036	-0.31	0.00	-1.54	0.14	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.00	0.00	0.20	0.20	0.20
R89.1 ULS1.9	-15.39	4.44	-8.98	0.16	0.3	0.048	0.0024	0.00036	-0.32	0.00	-3.74	0.14	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.48	0.48	0.48
R89.2 ULS1.9	-14.45	3.21	0.35	0.16	0.3	0.048	0.0024	0.00036	-0.30	0.00	0.15	0.10	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.02	0.02	0.02
R89.2 ULS1.9	-14.92	3.25	-3.51	0.16	0.3	0.048	0.0024	0.00036	-0.31	0.00	-1.46	0.10	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.19	0.19	0.19

BOTTOM MEMBERS LEFT	N	Vz	My	b [m]	h [m]	A [mm ²]	Wp [m ³]	Iyy [mm ⁴]	e.c.d	e.t.d	e.m.d	t.max	a	k.a.c	k.a.t	k.a.m	f _{0.6} σ _d *k
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OVERVIEW ULS CHECKS FOR ALL BOTTOM MEMBERS

										STRESSES				ANGLE FACTOR				MATERIAL STRENGTH				CHECKS				COMBINATIONS									
BOTTOM MEMBERS LEFT										N	Vz	My	b [m]	h [m]	A [m ²]	W [m ³]	Iy [m ⁴]	σ.c.d	σ.t.d	σ.m.d	t.max	α	k.a.c	k.a.t	k.a.m	f _{0.2.c.d} k.a.c	f _{0.2.d} k.a.t	f _{0.2.m.d} k.a.m	compression	tension	bending	comp. & bend.	tension & bend.		
L80	U.S.2.5	-68.11	-8.45	5.54	0.16	0.4	0.064	0.004267	0.000853333	-1.06	0.00	1.30	0.20	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.03	0.00	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13			
L80	U.S.2.5	-68.85	-8.42	10.19	0.16	0.4	0.064	0.004267	0.000853333	-1.07	0.00	2.39	0.20	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.03	0.00	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13			
L81	U.S.2.5	-67.86	-8.46	0.88	0.16	0.3	0.048	0.0024	0.00036	-1.41	0.00	0.37	-0.22	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04		
L81	U.S.2.5	-67.86	-8.46	0.88	0.16	0.3	0.048	0.0024	0.00036	-1.41	0.00	0.37	-0.22	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04		
L82	U.S.2.5	-65.53	-3.46	-2.38	0.16	0.3	0.048	0.0024	0.00036	-1.37	0.00	-0.99	-0.11	18.7	0.84	0.31	0.26	-44.23	0.11	17.93	0.03	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08		
L82	U.S.2.5	-65.53	-3.46	-2.38	0.16	0.3	0.048	0.0024	0.00036	-1.37	0.00	-0.99	-0.11	18.7	0.84	0.31	0.26	-44.23	0.11	17.93	0.03	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08		
L83	U.S.2.5	-63.73	-1.79	-3.26	0.16	0.3	0.048	0.0024	0.00036	-1.33	0.00	-1.36	-0.06	10.8	0.94	0.45	0.39	-49.31	0.19	17.93	0.03	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07		
L83	U.S.2.5	-63.73	-1.79	-3.26	0.16	0.3	0.048	0.0024	0.00036	-1.33	0.00	-1.36	-0.06	10.8	0.94	0.45	0.39	-49.31	0.19	17.93	0.03	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07		
L84	U.S.2.5	-63.01	-1.23	-3.57	0.16	0.3	0.048	0.0024	0.00036	-1.31	0.00	-1.49	-0.04	3.9	0.99	0.71	0.65	-52.04	0.19	17.93	0.03	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07		
L84	U.S.2.5	-63.01	-1.23	-3.57	0.16	0.3	0.048	0.0024	0.00036	-1.31	0.00	-1.49	-0.04	3.9	0.99	0.71	0.65	-52.04	0.19	17.93	0.03	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07		
L85	U.S.2.5	-63.25	-0.53	-3.1	0.16	0.3	0.048	0.0024	0.00036	-1.32	0.00	-1.29	-0.02	3.0	1.00	0.76	0.71	-52.20	0.19	17.93	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04		
L85	U.S.2.5	-63.25	-0.53	-3.1	0.16	0.3	0.048	0.0024	0.00036	-1.32	0.00	-1.29	-0.02	3.0	1.00	0.76	0.71	-52.20	0.19	17.93	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	
L86	U.S.2.5	-62.99	-1.17	-1.5	0.16	0.3	0.048	0.0024	0.00036	-1.31	0.00	-0.63	0.04	11.4	0.93	0.44	0.37	-49.00	0.19	17.93	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05		
L86	U.S.2.5	-62.99	-1.17	-1.5	0.16	0.3	0.048	0.0024	0.00036	-1.31	0.00	-0.63	0.04	11.4	0.93	0.44	0.37	-49.00	0.19	17.93	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	
L87	U.S.2.5	-61.45	0.26	-1.78	0.16	0.3	0.048	0.0024	0.00036	-1.28	0.00	-0.74	0.01	17.3	0.86	0.33	0.28	-45.17	0.19	17.93	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05		
L87	U.S.2.5	-61.45	0.26	-1.78	0.16	0.3	0.048	0.0024	0.00036	-1.28	0.00	-0.74	0.01	17.3	0.86	0.33	0.28	-45.17	0.19	17.93	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	
L88	U.S.2.5	-61.78	0.62	-2.24	0.16	0.3	0.048	0.0024	0.00036	-1.29	0.00	-0.93	0.02	17.3	0.86	0.33	0.28	-45.17	0.19	17.93	0.03	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07		
L88	U.S.2.5	-61.78	0.62	-2.24	0.16	0.3	0.048	0.0024	0.00036	-1.29	0.00	-0.93	0.02	17.3	0.86	0.33	0.28	-45.17	0.19	17.93	0.03	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07		
L89	U.S.2.5	-24.43	-2.08	-0.06	0.16	0.35	0.056	0.003267	0.000571667	-0.44	0.00	-0.02	-0.06	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10		
L89	U.S.2.5	-24.43	-2.08	-0.06	0.16	0.35	0.056	0.003267	0.000571667	-0.44	0.00	-0.02	-0.06	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	
L89.1	U.S.2.5	-19.55	0.11	-2.28	0.16	0.3	0.048	0.0024	0.00036	-0.41	0.00	-0.95	0.00	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	
L89.1	U.S.2.5	-19.55	0.11	-2.28	0.16	0.3	0.048	0.0024	0.00036	-0.41	0.00	-0.95	0.00	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	
L89.2	U.S.2.5	-15.65	1.7	-0.12	0.16	0.3	0.048	0.0024	0.00036	-0.33	0.00	-0.05	0.05	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	
L89.2	U.S.2.5	-15.65	1.7	-0.12	0.16	0.3	0.048	0.0024	0.00036	-0.33	0.00	-0.05	0.05	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
L89.2	U.S.2.5	-16.2	1.75	-2.18	0.16	0.3	0.048	0.0024	0.00036	-0.34	0.00	-0.91	0.05	34.9	0.63	0.19	0.16	-32.89	0.19	17.93	0.03	0.01	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	
R80	U.S.2.5	-68.45	8.42	10.19	0.16	0.4	0.064	0.004267	0.000853333	-1.07	0.00	2.39	0.20	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.03	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24		
R80	U.S.2.5	-68.45	8.42	10.19	0.16	0.4	0.064	0.004267	0.000853333	-1.07	0.00	2.39	0.20	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.03	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
R81	U.S.2.5	-68.11	8.45	5.54	0.16	0.3	0.048	0.0024	0.00036	-1.42	0.00	2.31	0.26	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.03	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23		
R81	U.S.2.5	-68.11	8.45	5.54	0.16	0.3	0.048	0.0024	0.00036	-1.42	0.00	2.31	0.26	25.8	0.74	0.24	0.20	-39.03	0.19	9.93	0.03	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	
R82	U.S.2.5	-67.86	-8.46	0.88	0.16	0.3	0.048	0.0024	0.00036	-1.41	0.00	0.37	-0.22	25.8	0.74																				

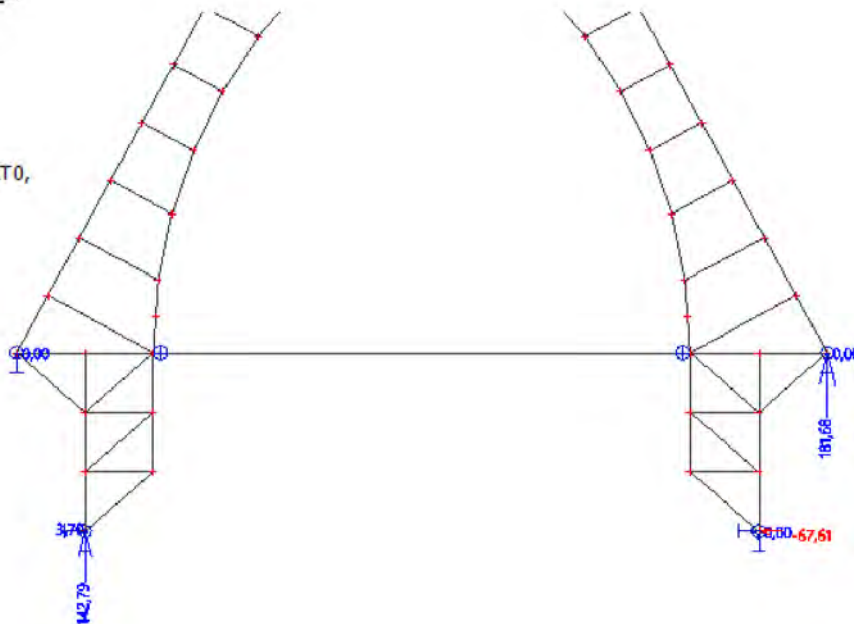
OVERVIEW ULS CHECKS FOR ALL BOTTOM MEMBERS

										STRESSES				ANGLE FACTOR				MATERIAL STRENGTH				CHECKS				COMBINATIONS		
BOTTOM MEMBERS LEFT										e.c.d	a.t.d	a.m.d	t.max	a	k.a.c	k.a.t	k.a.m	f _{0.6} .d ³ .k.a.c	f _{0.6} .d ³ .k.a.t	f _{0.6} .m.d ³ .k.a.m	compression	tension	bending	comp. & bend.	tension & bend.			
L80	ULS 2.8	-34.94	17.74	-10.98	0.16	0.4	0.064	0.004267	0.0008533333	-0.55	0.00	-2.57	0.42	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.01	0.00	0.49	0.49	0.49			
L80	ULS 2.8	-35.19	17.76	-20.77	0.16	0.4	0.064	0.004267	0.0008533333	-0.55	0.00	-4.87	0.42	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.02	0.00	0.05	0.05	0.05			
L81	ULS 2.8	-34.75	17.73	-1.2	0.16	0.3	0.048	0.00204	0.00036	-0.72	0.00	-0.50	0.55	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.02	0.00	0.05	0.05	0.05			
L81	ULS 2.8	-34.94	17.74	-10.98	0.16	0.3	0.048	0.00204	0.00036	-0.73	0.00	-4.58	0.55	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.02	0.00	0.46	0.46	0.46			
L82	ULS 2.8	-26.07	10.43	8.78	0.16	0.3	0.048	0.00204	0.00036	-0.54	0.00	3.66	0.33	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.01	0.00	0.28	0.28	0.28			
L82	ULS 2.8	-26.44	10.5	-1.9	0.16	0.3	0.048	0.00204	0.00036	-0.55	0.00	-0.79	0.33	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.01	0.00	0.06	0.06	0.06			
L83	ULS 2.8	-17.12	5.43	12.03	0.16	0.3	0.048	0.00204	0.00036	-0.36	0.00	5.01	0.17	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.01	0.00	0.26	0.26	0.26			
L83	ULS 2.8	-17.44	5.54	6.52	0.16	0.3	0.048	0.00204	0.00036	-0.36	0.00	2.72	0.17	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.01	0.00	0.14	0.14	0.14			
L84	ULS 2.8	-10.46	1.7	11.99	0.16	0.3	0.048	0.00204	0.00036	-0.22	0.00	5.00	0.05	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.00	0.15	0.15	0.15			
L84	ULS 2.8	-10.77	1.85	10.22	0.16	0.3	0.048	0.00204	0.00036	-0.22	0.00	4.26	0.06	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.00	0.13	0.13	0.13			
L85	ULS 2.8	-7.16	-2.88	8.42	0.16	0.3	0.048	0.00204	0.00036	-0.15	0.00	3.51	-0.09	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.00	0.00	0.10	0.10	0.10			
L85	ULS 2.8	-7.45	-2.7	11.18	0.16	0.3	0.048	0.00204	0.00036	-0.16	0.00	4.66	-0.08	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.00	0.00	0.13	0.13	0.13			
L86	ULS 2.8	-14.51	-6.57	5.28	0.16	0.3	0.048	0.00204	0.00036	-0.30	0.00	2.20	-0.21	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.01	0.00	0.12	0.12	0.12			
L86	ULS 2.8	-14.77	-6.34	11.81	0.16	0.3	0.048	0.00204	0.00036	-0.31	0.00	4.92	-0.20	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.01	0.00	0.26	0.26	0.26			
L87	ULS 2.8	-25.89	-8.92	2.19	0.16	0.3	0.048	0.00204	0.00036	-0.54	0.00	0.91	-0.28	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.01	0.00	0.07	0.07	0.07			
L87	ULS 2.8	-26.13	-8.65	11.39	0.16	0.3	0.048	0.00204	0.00036	-0.54	0.00	4.75	-0.27	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.01	0.00	0.35	0.35	0.35			
L88	ULS 2.8	-34.52	-15.84	-12.92	0.16	0.3	0.048	0.00204	0.00036	-0.72	0.00	-5.38	-0.50	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.02	0.00	0.55	0.55	0.55			
L88	ULS 2.8	-34.73	-15.52	4.61	0.16	0.3	0.048	0.00204	0.00036	-0.72	0.00	1.92	-0.49	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.02	0.00	0.20	0.20	0.20			
L89	ULS 2.8	-17.71	-12.16	1.97	0.16	0.35	0.056	0.003267	0.000571667	-0.49	0.00	0.60	-0.33	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.08	0.08	0.08			
L89	ULS 2.8	-28.19	-12.11	16.47	0.16	0.35	0.056	0.003267	0.000571667	-0.50	0.00	5.04	-0.32	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.02	0.00	0.64	0.64	0.64			
L89.1	ULS 2.8	-19.76	-3.13	-1.76	0.16	0.3	0.048	0.00204	0.00036	-0.47	0.00	-0.73	-0.10	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.09	0.09	0.09			
L89.1	ULS 2.8	-20.17	-3.1	1.96	0.16	0.3	0.048	0.00204	0.00036	-0.41	0.00	0.82	-0.10	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.10	0.10	0.10			
L89.2	ULS 2.8	-14.68	1.28	-0.14	0.16	0.3	0.048	0.00204	0.00036	-0.31	0.00	-0.06	0.04	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.01	0.01	0.01			
L89.2	ULS 2.8	-15.09	1.32	-1.7	0.16	0.3	0.048	0.00204	0.00036	-0.31	0.00	-0.71	0.04	34.9	0.63	0.19	0.16	-32.89	7.29	7.84	0.01	0.00	0.09	0.09	0.09			
R80	ULS 2.8	-104.35	27.72	33.04	0.16	0.4	0.064	0.004267	0.0008533333	-1.63	0.00	7.74	0.65	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.04	0.00	0.78	0.78	0.78			
R80	ULS 2.8	-104.1	27.74	17.75	0.16	0.4	0.064	0.004267	0.0008533333	-1.63	0.00	4.16	0.65	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.04	0.00	0.42	0.42	0.42			
R80	ULS 2.8	-104.1	27.74	17.75	0.16	0.4	0.064	0.004267	0.0008533333	-1.63	0.00	7.40	0.67	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.06	0.00	0.74	0.74	0.74			
R81	ULS 2.8	-103.91	27.75	2.45	0.16	0.3	0.048	0.00204	0.00036	-2.16	0.00	1.02	0.87	25.8	0.74	0.24	0.20	-39.03	9.19	9.93	0.06	0.00	0.10	0.11	0.11			
R82	ULS 2.8	-106.55	-14.31	-11.12	0.16	0.3	0.048	0.00204	0.00036	-2.22	0.00	-4.63	-0.45	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.05	0.00	0.36	0.36	0.36			
R82	ULS 2.8	-106.91	-14.24	3.44	0.16	0.3	0.048	0.00204	0.00036	-2.23	0.00	1.43	-0.45	18.7	0.84	0.31	0.26	-44.23	11.77	12.93	0.05	0.00	0.11	0.11	0.11			
R83	ULS 2.8	-111.13	-7.83	-15.5	0.16	0.3	0.048	0.00204	0.00036	-2.32	0.00	-4.46	-0.24	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.05	0.00	0.34	0.34	0.34			
R83	ULS 2.8	-111.45	-7.71	-7.69	0.16	0.3	0.048	0.00204	0.00036	-2.32	0.00	-3.20	-0.24	10.8	0.94	0.45	0.39	-49.31	17.09	19.27	0.05	0.00	0.17	0.17	0.17			
R84	ULS 2.8	-116.34	-4.22	-16.52	0.16	0.3	0.048	0.00204	0.00036	-2.42	0.00	-6.88	-0.13	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.00	0.00	0.21	0.21	0.21			
R84	ULS 2.8	-116.65	-4.07	-12.38	0.16	0.3	0.048	0.00204	0.00036	-2.43	0.00	-5.16	-0.13	3.9	0.99	0.71	0.65	-52.04	26.88	32.45	0.05	0.00	0.16	0.16	0.16			
R85	ULS 2.8	-121.62	0.42	-13.37	0.16	0.3	0.048	0.00204	0.00036	-2.53	0.00	-5.57	0.01	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.05	0.00	0.16	0.16	0.16			
R85	ULS 2.8	-121.9	0.61	-13.82	0.16	0.3	0.048	0.00204	0.00036	-2.54	0.00	-5.78	0.02	3.0	1.00	0.76	0.71	-52.20	28.90	35.40	0.05	0.00	0.16	0.16	0.16			
R86	ULS 2.8	-118.1	6.81	-7.82	0.16	0.3	0.048	0.00204	0.00036	-2.46	0.00	-3.26	0.21	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.05	0.00	0.17	0.18	0.17			
R86	ULS 2.8	-118.36	7.04	-14.83	0.16	0.3	0.048	0.00204	0.00036	-2.47	0.00	-6.18	0.22	11.4	0.93	0.44	0.37	-49.00	16.55	18.62	0.05	0.00	0.33	0.33	0.33			
R87	ULS 2.8	-108.49	7.01	-6.16	0.16	0.3	0.048	0.00204	0.00036	-2.26	0.00	-2.57	0.22	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.05	0.00	0.19	0.19	0.19			
R87	ULS 2.8	-108.74	7.27	-13.9	0.16	0.3	0.048	0.00204	0.00036	-2.27	0.00	-5.68	0.23	17.3	0.86	0.33	0.28	-45.17	12.49	13.73	0.05	0.00	0.41	0.41	0.41			
R88	ULS 2.8	-104.03	16.37	9.49	0.16	0.3	0.048	0.00204	0.00036	-2.17	0.00	3.95	0.51	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.06	0.00	0.40	0.41	0.40			
R88	ULS 2.8	-104.25	16.69	-8.98	0.16	0.3	0.048	0.00204	0.00036	-2.17	0.00	-3.74	0.52	26.3	0.74	0.24	0.20	-38.66	9.07	9.78	0.06	0.00	0.38	0.39	0.38</			

7. ULS 2.9 MAXIMUM REACTION FORCES

7.1. Reactions; R_y ; R_z

Values: R_y , R_z
Nonlinear calculation
NonLinear Combi: ULS 2.9
System: Global
Extreme: Member
Selection: Sn8, Sn83, N-RT0, N-LT0,
N-LSUP6, N-RSUP6



ROOF RIDGE JOINT CALCULATIONS

axial force (compression)

shear force + LC.4

eccentricity and moment

$$e := 340 \text{ mm}$$

ULS 2.4 $N_d := 70 \text{ kN}$

$$V_{zd} := 40 \text{ kN} + 15 \text{ kN} = 55 \text{ kN}$$

$$M_{yd} := V_{zd} \cdot e = 18.7 \text{ kN} \cdot \text{m}$$

ULS 1.4 $N_d := 27 \text{ kN}$

$$V_{zd} := 50 \text{ kN} + 15 \text{ kN} = 65 \text{ kN}$$

$$M_{yd} := V_{zd} \cdot e = 22.1 \text{ kN} \cdot \text{m}$$

(ULS 1.4 governing load case: LC.1 self-weight rafter + LC.3 wind + LC.4 crane)

kmod factor

$$k_{mod} := 1.1$$

(EN 1995-1-1: 3 instantaneous)

partial safety factor LVL

$$\gamma_m := 1.2$$

(EN 1995-1-1: 2.4.1)

GEOMETRY

dowel diameter:

$$d := 12 \text{ mm}$$

effective number of fasteners in Baubuche-Q

$$n := 12$$

due to cross-plies no reduction required

$$n_{ef} := n$$

governing LVL thickness per shear plane

$$t_1 := 70 \text{ mm}$$

steel plate thickness:

$$t_{steel} := 20 \text{ mm}$$

thick steelplate since $t_{steel} > d$

grain angle with most

conservative spacing :

$$\alpha := 0 \cdot \left(\frac{\pi}{180} \right) = 0$$

spacing a1 and a3.t taken as conservative distances for design:

spacing of **84 mm from the edge** of the structure and **60 mm between dowels**.

Distances for steel-to-timber connections in Baubuche with dowel $d = 12 \text{ mm}$

spacing a1

$$0 - \alpha - 360$$

$$a_1 := (3 + 2 \cdot |\cos(\alpha)|) \cdot d = 60 \text{ mm}$$

spacing a2

$$0 - \alpha - 360$$

$$a_2 := 3 \cdot d = 36 \text{ mm}$$

loaded end a3.t

$$-90 - \alpha - 90$$

$$a_{3,t} := \max(7 \cdot d, 80 \text{ mm}) = 84 \text{ mm}$$

unloaded end a3.c

$$90 - \alpha - 150$$

$$a_{3,c} := \max(a_{3,t} \cdot |\sin(\alpha)|, 3 \cdot d) = 36 \text{ mm}$$

$$150 - \alpha - 210$$

$$a_{3,c} := 3 \cdot d = 36 \text{ mm}$$

$$210 - \alpha - 270$$

$$a_{3,c} := \max(a_{3,t} \cdot |\sin(\alpha)|, 3 \cdot d) = 36 \text{ mm}$$

loaded edge a4.t

$$0 - \alpha - 180$$

$$a_{4,t} := \max((2 + 2 \cdot \sin(\alpha)) \cdot d, 3 \cdot d) = 36 \text{ mm}$$

unloaded edge a4.c

$$180 - \alpha - 360$$

$$a_{4,c} := 3 \cdot d = 36 \text{ mm}$$

Spacing and distances	Minimum values Bolts	Minimum values Tight-fitting bolts/dowels
Spacing a_1 (parallel to the grain)	$(4 + \cos \alpha) \cdot d$	$(3 + 2 \cdot \cos \alpha) \cdot d$
Spacing a_2 (perpendicular to the grain)	$4 \cdot d$	$3 \cdot d$
Distance $a_{3,t}$ (loaded end)	$\max(7 \cdot d; 80 \text{ mm})$	$\max(7 \cdot d; 80 \text{ mm})$
Distance $a_{3,c}$ (unloaded end)	$(1 + 6 \cdot \sin \alpha) \cdot d$	$\max(a_{3,t} \cdot \sin \alpha ; 3 \cdot d)$
	$4 \cdot d$	$3 \cdot d$
	$(1 + 6 \cdot \sin \alpha) \cdot d$	$\max(a_{3,t} \cdot \sin \alpha ; 3 \cdot d)$
Distance $a_{4,t}$ (loaded edge)	$\max((2 + 2 \cdot \sin \alpha) \cdot d; 3 \cdot d)$	$\max((2 + 2 \cdot \sin \alpha) \cdot d; 3 \cdot d)$
Distance $a_{4,c}$ (unloaded edge)	$3 \cdot d$	$3 \cdot d$

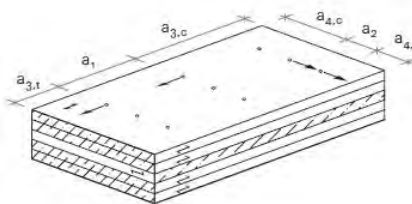
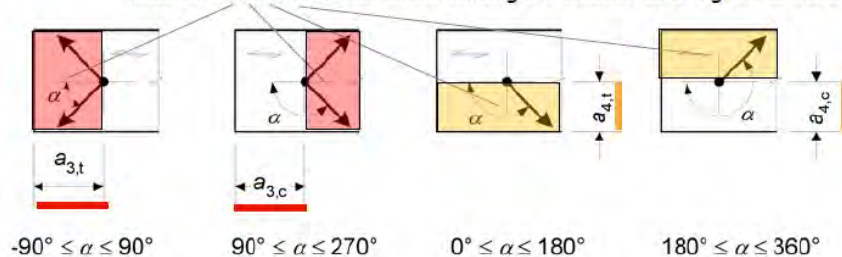


Figure 17:
Definition of
fastener spacing
and distances

fastener loads the timber under an angle α relative to the grain direction



Key:

- (1) Loaded end
- (2) Unloaded end
- (3) Loaded edge
- (4) Unloaded edge

[illegible]
$$f_{u.k} := 430 \frac{N}{mm^2}$$
$$M_{y.Rk} := 0.3 \cdot f_{u.k} \cdot \left(\frac{d}{mm} \right)^{2.6} \cdot mm^3 = (8.25 \cdot 10^4) \, N \cdot mm$$
$$\rho_k := 730 \frac{\text{kg}}{\text{m}^3}$$
$$f_{h,k} := \left(0.082 \cdot 730 \cdot \left(1 - 0.01 \cdot \frac{d}{mm} \right) \right) \frac{N}{mm^2} = 52.677 \frac{N}{mm^2}$$

$$k_{90} := 0.90 + 0.015 \cdot \frac{d}{mm} = 1.08$$
$$\alpha := 90 \cdot \left(\frac{\pi}{180} \right) = 1.571$$
$$f_{h,\alpha,k} := \frac{f_{h,k}}{k_{90} \cdot \sin^2(\alpha) + \cos^2(\alpha)} = 48.775 \frac{N}{mm^2}$$
[illegible]

GOVERNING LOAD ON DOWEL

Table 34 shows that grain angle has minor effect on the embedding strength. For dowels of 12 mm the maximum strength loss due the grain angle is -7.4%, which consequently means that the coinciding direction of the normal, shear and moment forces appeared more critical than the direction of the force relatively to the main grain angle. The governing dowel therefore is the one at the right bottom, as shown in the image below.

amount and distances of
dowels to rotation centre

$$n_1 := 8 \quad r_1 := 0.135 \text{ m}$$

$$n_2 := 4 \quad r_2 := 0.097 \text{ m}$$

normal/shear force uniformly distributed

$$F_N := \frac{N_d}{n} = 2.25 \text{ kN} \quad F_V := \frac{V_{zd}}{n} = 5.417 \text{ kN}$$

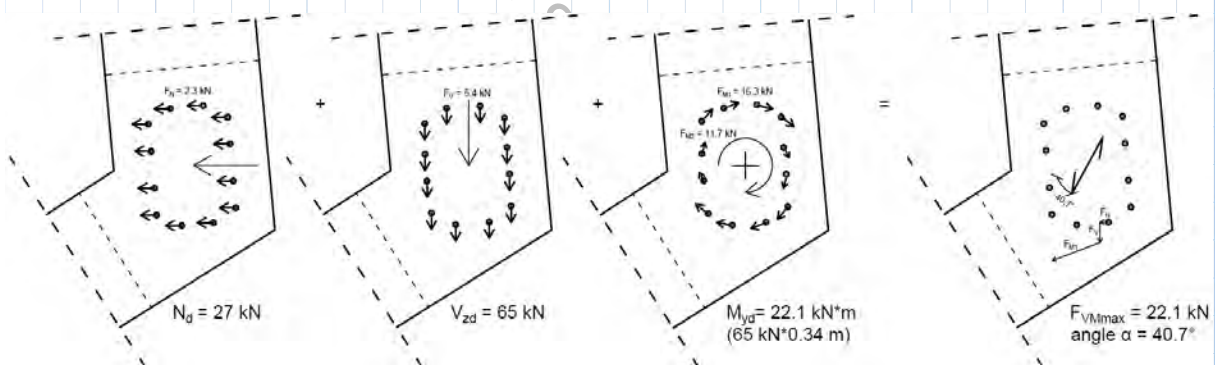
governing force due to moment in fastener
furthest from rotation point (distance r_1)

$$F_{M1} := \frac{M_{yd} \cdot r_1}{n_1 \cdot r_1^2 + n_2 \cdot r_2^2} = 16.265 \text{ kN}$$

total load on fastener peaks where normal
and shear force act in the same direction as
moment force

$$F_{VM,max} := F_{M1} + \sqrt{F_N^2 + F_V^2} = 22.13 \text{ kN}$$

load distribution over dowels



load-carrying capacity per shear plane f, g & h:
(no rope effect can be used for dowels)

$$F_{v.Rk.f} := f_{h,\alpha,k} \cdot t_1 \cdot d = 40.971 \text{ kN}$$

$$F_{v.Rk.g} := f_{h,\alpha,k} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + \frac{4 \cdot M_{y,Rk}}{f_{h,\alpha,k} \cdot t_1^2 \cdot d}} - 1 \right) = 18.614 \text{ kN}$$

$$F_{v.Rk.h} := 2.3 \cdot \sqrt{M_{y,Rk} \cdot f_{h,\alpha,k} \cdot d} = 15.983 \text{ kN}$$

$$F_{v.Rk} := \min \left(f_{h,\alpha,k} \cdot t_1 \cdot d, f_{h,\alpha,k} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + \frac{4 \cdot M_{y,Rk}}{f_{h,\alpha,k} \cdot t_1^2 \cdot d}} - 1 \right), 2.3 \cdot \sqrt{M_{y,Rk} \cdot f_{h,\alpha,k} \cdot d} \right) = 15.983 \text{ kN}$$

load-carrying capacity

per dowel (2 shear planes)

$$F_{v.Rd} := 2 \cdot \frac{k_{mod}}{\gamma_m} \cdot F_{v.Rk} = 29.301 \text{ kN}$$

CHECK
OK!

$$\frac{F_{VM,max}}{F_{v.Rd}} = 0.755$$

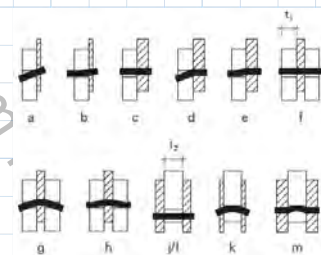


Figure 11. Failure modes for steel-to-timber connections