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Wooden Architecture & Constructions in the Future

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Block 2
Where is the Nordic construction market in five years?

Where will the Finnish building market be in five years?

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2 Where will the Finnish building market be in five years? | P. Heino

Where will the Finnish building market be in five years?

Abstract

Bauhaus Earth (1) is an initiative for accelerating the transformation of the built environment towards sustainability and inclusiveness. One of their projects, ReBuilt, seeks to advance the novel concept of a regenerative built environment and demonstrate how it can be implemented in practice. The ReBuilt project focuses on five themes of transformation: the shift to bio-based materials; circular building practices; polycentric and resource-efficient urban systems; people-centred and inclusive approaches; and an enabling environment for systems change.
This kind of holistic thinking is rare when looking at the activities of governments and businesses, also in Finland. By solving problems we create a new set of problems. Therefore, you can predict the future to be not that much different from today, except more climate extremes.

1. Carbon neutrality of the nation is the political driving factor

According to the government programme, the Finnish Government is committed to meeting emission reduction targets and moving towards carbon neutrality in 2035 followed by carbon negativity. The new Climate Change Act entered into force in 2022. The Act set emission reductions targets for 2030, 2040 and 2050 and laid down the target of carbon neutrality in 2035.

Emissions are at the moment at abt. 40 million tonnes of carbon dioxide equivalent. By 2035 emissions and the net sink from the land-use sector should be at about 21 million tonnes, providing the carbon neutrality of the nation. During the previous government, 13 industries prepared their own low-carbon roadmaps, coordinated by the Ministry of Economic Affairs and Employment of Finland. This year the roadmaps were updated in accordance. They support national policy preparation and promote the internal dialogue of industries on emission reduction measures.

Finland has an ambitious goal to become carbon neutral soonish. But relying on natural sinks has proven to be problematic. Roadmaps related to energy production and consumption are progressing positively, but overall consumption based emissions are lacking behind and not directly under current policies.

2. Ongoing work to make the future brighter for the Finnish construction sector

Aalto University and 21 companies have started a process to describe the future of the Finnish construction sector. The work ongoing within the Building 2030 -project is built on international benchmarks, adopting best practices from other fields and the transformation of operations brought on by digitalisation and new technologies.

«Building 2030 is a consortium that develops a vision for the Finnish construction sector in the year 2030 and to work toward implementing it. This is undertaken to prepare for future changes in the field of construction – digitalisation, rapid urbanization, climate change and more – and to ensure a sustainable foundation for business in the sector going forward». The core of the Building 2030 vision is built around five themes, which are reliable, user-centric, sustainable development, productivity, and inspiring. (2) The work is focusing on building process management.

The sector specific carbon roadmap for the building industry was prepared by the Confederation of Finnish Construction Industries RT.(3). It recognizes several actions potential to contribute to the emission reduction of the construction sector and providing business potential for the entrepreneurs:

- Fossil-free steel – production and usage
- Production and usage of low-carbon concrete and cement
- Production and usage of low-carbon asphalt
- Reuse of structural elements and other building products
- Wood construction
- Extension of the lifetime of buildings or structures (incl. renovation)
- Increment of low carbon building sites
- Prioritizing emission-free transports
- Management of aggregates and soils in infrastructure projects
- HVAC-improvement efficiency activities
- Building specific heat systems
- Utilisation of dynamic energy prices in consumption

Building 2030 vision works on the building level and building process level for more efficiency to produce suitable homes with less defects and more sustainably. The Confederation of the construction industry is an organisation of the cement, concrete, steel and plastic industry. Since the major housebuilders are also members of the Confederation of the construction industry, there is very little discussion of the building technologies replacing the precast concrete system. Companies producing wooden houses or elements are members of the Federation of the Finnish Woodworking industries. In the building sector, mitigating climate change is not a priority.

3. Building market in Finland today

The Government's construction sector's business cycle group Raksu estimates in a recent report that construction will still decrease during the current year, but the growth situation will turn to growth in 2025. The reasons for this are confidence in the future in falling interest rates, a slowdown in inflation and a picking up economy. (4)

According to the report, self-financed housing production is currently very little. New apartment starts will pick up slowly, because the number of unsold apartments is large and selling takes time. Next year the new construction will accelerate. This is supported by population growth, immigration and migration to growth centres. The situation in business premises construction has been better than in residential construction. The report predicts that the growth in the construction of new business premises will start already this year.

According to the report, the slowdown in price growth and the drop in interest rates will also speed up the start of deferred repair work by building societies. It is estimated that the renovation of apartments will turn to a small increase at the end of the year and will continue during the next year. Repairing public buildings accelerates the repair construction of business premises. Rail investments will lead to growth in civil engineering next year.

The total construction market size is about 41 billion € in 2022, divided into new construction (18,8 bn €.), renovations (15,4 bn €) and infra and water construction (7,4 bn €). The reports do not pay any attention to the climate change mitigation, and the possible interconnections between the economical situation, growth possibilities, sustainable development of the sector, and specific Finnish possibilities. The adaptation is mentioned as a challenge of the future.

4. Current use of wood in construction in Finland

Because wood construction solutions already have a strong position in the Finnish building market, use of wood is widely accepted. There is the culture, knowledge, industry and acceptance. Wood has a dominant position in the detached house construction market as well as in the recreational home market, and also among agricultural buildings. The market

share for wood is about 28 %, calculated by building volumes of the building permits. In blocks of flats concrete is the predominant material counting over 90 %-m3 of starts, majority being precast element buildings. Wood has a market share of 4,5 % in 2023 in multi-storey buildings. The greatest potential for increasing the use of wood products lies in the multi-storey apartment and office buildings as well as in larger public buildings.

The housing starts are at a very low level of 17 000 units at the moment of which the majority over 12 000 are in a multistorey building. Single-family and townhouses encompass only abt. 5000 units. Numbers are expected to rise up to 23 500 by 2025. The city planning ideologies support the dense urban models, which do not support the wood usage by current industries.

There are some sectors that have gained progress regarding the use of wood. In general off-site prefabrication is more utilized. In 2020 the Finnish government, as a part of the National Wood Building program, set up targets for the share of wood in public buildings in 2025 to be 45 % (share of built volumes). In 2023 the public sector is building less and the municipalities have economic challenges. Especially schools, daycare centers and assembly are more often built with wood. The share of wood has not increased as a whole, but the pre-fab mass timber solutions are more common and the project sizes are bigger.

The emergency of fighting climate change is not visible in the Finnish construction market.

5. Discussion

The role of the forest and product biomass in the climate commitments is going to have an effect on the availability of wood for the industry. A possibility is that internationally Finnish forests are seen either as a sustainable source for biomass or a growing carbon sink. These things have an opposite effect for the utilization and the development of the industry and the construction sector.

The Finnish forestry sector is now in a wicked position. Biodiversity challenges and the ambition for annual carbon sink in the forests, a significant component of the EU LULUCF (Land use and land use change and forestry) -regulation, overrule the potential of the sector in communication and policies. The national climate neutrality is linked to the annual sink, which now seems impossible to reach. At the same time Finnish companies like UPM (5) and StoraEnso (6) are receiving recognition in the international sustainability assessments.

Where is the construction market in Finland in five years? At the moment the confederation of the building sector and the government are relying on developing technologically and materially neutral solutions for growth and decarbonisation. Fossil-free energy for heating and product manufacturing is seen as the answer. The new building law will require a climate declaration on all new construction by the beginning of 2026. From then on, also limit values for carbon footprint are applied. The law also requires the calculation of a carbon handprint, which describes the positive effects of the building projects, such as the carbon storage in and on buildings. The national law together with EU-regulation such as EPBD will change also the Finnish market in the favor of proven low-carbon solutions. The use of recycled materials will be a big challenge for the building sector. Availability of raw materials, virgin and recycled, with rising prices, will affect the relative cost competitiveness of materials used.

The Finnish wood products industry is investing in sawmilling technology, and less in the further processing. Foreign investments in sawmilling have increased in the last ten years. The driving factors are the good quality and relative secure sourcing of wood. LVL production is expanding. There are very few investments for the capacity of industrialized wood construction for the domestic market, nor for the export market.

Overall, the Finnish construction sector is expected to become more innovative, sustainable, and technologically advanced over the next five years. But not much, I would predict.

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Block 3
How to develop a sustainable world

Real estate development for a sustainable world

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2 Real estate development for a sustainable world | A. Ervast Oeberg

Real estate development for a sustainable world

Abstract

As a developer, you are responsible for choosing suitable building products with assessed properties for your project.
In the work of developing economically sustainable projects with the best possible characteristics that meet requirements, building standards, politics and a market demand, positioning and future-proofing are critical factors. Large-scale wooden construction for housing in an urban context contains all the opportunities to streamline the construction industry to accommodate all these factors and take responsibility for contributing to a climate change of the industry that today consumes the largest part of the world's carbon dioxide budget and extraction of virgin raw material.

1. The challenges

1.1. Climate breakdown

The built environment is responsible for approximately 50% of all extracted materials and 35% of the EU's total waste generation. The production of cement and steel alone account for about 15% of global greenhouse gas emissions.

1.2 We are in the grip of a global housing crisis

Around 1.6 billion people worldwide lack adequate housing, according to a United Nations report – and experts say this could rise to 3 billion by 2030.

In order to house those 3 billion, the world will need to build 96,000 new affordable homes every day, UN-Habitat says.

2. The power of the developer

2.1. The company that decided only to build in wood

Folkhem is a company that puts the climate issue at the top of the agenda, which is why more than ten years ago they made the decision to only build houses with wooden frames. The decision was made based on the realization that the construction industry is one of the most emission-intensive industries and that switching to wood in the frame instead of concrete reduces the carbon footprint of each house by at least 50%. Effect of carbon dioxide storage not included. The prerequisites for the technology to build high and fireproof and the industry's production of products for this were in place even then, albeit rather untested.
With that decision at the back, Folkhem has had the opportunity to work dedicatedly in step with the growth of industry, knowledge and demand in building to develop its portfolio, optimizing its projects and developing its own processes to control and follow up the climate impact in all its projects, from the first concept stage, to the erected building.

Folkhem works actively with LCA for each project, from the first sketch to the finished house and sets demands on themselves and their contractors to optimize the projects in terms of climate impact. By following the project through the entire journey, conscious decisions can be made regarding architecture, choice of materials and form.
Through a 360-degree perspective on the material's inherent properties, Folkhem will demonstrate that wooden construction is beautiful, profitable and sustainable. A burden of proof to drive development, demand and competitiveness in wood construction.

Folkhem is today active in Stockholm and Uppsala with a portfolio containing about 10 different housing projects, all in an urban context, and all developed to be built in wood. The first project to be realized in the purely wooden house portfolio is the symbolic project

Cederhusen, Stockholm's first solid wood project and one of the largest in the world, just completed in Hagastaden and visited by more than a thousand people from all continents of the world. Cederhusen contains 234 homes in two inner city blocks built with wooden frames and wooden facades in a challenging location over three tunnels.
Folkhem aims to be an international role model that drives the development of large scale wooden construction by building sustainable, beautiful houses with outstanding architecture and high ambition, which make a difference - for people, the city and the planet.
In the future, there are plans to expand in other metropolitan regions in both Sweden and Norway.

2.2 Bridging between research and reality to accelerate development

With an expertise in wood construction combined with a creative and innovative way of working, Folkhem works based on needs and continuously develops its processes. Each project must contribute to moving the positions forward, by giving them a specified task and making the projects in the portfolio available for different types of research projects in order to create a bridge between academia and reality and thus the conditions for an accelerated turnover. An established work-culture that the experience and knowledge from the now completed project Cederhusen in Hagastaden shows the inherent potential and possibilities.

Through e.g. the «4 steps to GTIN» development project, the insight raised into the critical value of a quality-assured and unbroken information chain, a common digital language between all parties throughout the project's process, has been established. This as the very basic condition, the DNA, to create the possibility of circular flows and reuse.

Seeing through the material properties of wood hosting the optimal conditions for reuse was investigated in the Swedish research project «Design of the Future» where Folkhem was a partner and contributed with one of its projects as a demonstrator.

This led to involvement as a partner in «Woodcircles», a project that develops solutions for circular use of wood construction materials, to reduce greenhouse gas emissions, waste generation, and materials consumption.

Woodcircles is a four-year innovation project co-funded by the European Union that aims to develop circular and sustainable solutions for wood in construction. Its overall goal is to accelerate the green transition of the construction sector. Woodcircles is a multi-stakeholder collaboration between 20 leading European partners across the wood construction value chain.

Folkhem's unique advantage is the experience-based expertise in large-scale wooden construction together with its architectural ambition. Thanks to the solid knowledge of the material's inherent properties, the industry's development and its own agile processes, Folkhem can contribute to being an inspirer and work for climate change in the entire construction industry.

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Stora Enso head office
The story behind

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Stora Enso head office
The story behind

Abstract

This is the story of how innovation in mass timber architecture is contributing to Helsinki's net zero goals while sensitively regenerating a historic harbourside townscape – and creating a new home for Stora Enso. Katajanokan Laituri is a modern mixed-use building, owned and developed by Varma Mutual Pension Insurance Company. Building facilitates the new head office of Stora Enso and a hotel and serve as a meeting place for city dwellers, tourists, and service providers in one of Helsinki's finest locations. Background, vision and motivation of the project as well as the main design drivers are discussed in this paper.

1. New landmark of wooden construction

1.1. About Stora Enso

Part of the global bioeconomy, Stora Enso is a business-to-business company and a leading provider of renewable products in packaging, biomaterials, and wooden construction, and one of the largest private forest owners in the world.
The Wood Products division of Stora Enso is Europe's largest sawn timber producer and a leading provider of sustainable wood-based solutions for the global construction industry. Additionally, it offers window and door components, and co-products such as pellets made from wood residuals.

1.2 Background of the new head office

Stora Enso's old head office in Kanavaranta, Helsinki was designed by one the most recognized Finnish architect Alvar Aalto. Building was completed in 1961 and since then it has been an iconic landmark in Helsinki's cityscape. Stora Enso sold Kanavaranta office in 2008 and continued as tenant for more than ten years in Kanavaranta premises until announced plans to move to a new office. In November 2019 Varma Mutual Pension Insurance Company, one of the largest investors and real estate developers in Finland, and Stora Enso as the future anchor tenant, announced plans regarding new wooden building at the South Harbor in Helsinki.

1.3. Ambitious targets for the new building

Varma Mutual Pension Insurance Company had secured a planning permission for a site in the South Harbour area of Helsinki to build a new head office for Stora Enso as well as a hotel. The gross area of new building should be approximately 22,000 sqm. Varma values successful long-term investments and seeks to build symbiotic relationships with both tenants and wider society. With this approach, the company has succeeded in generating considerable financial as well as social value. Stora Enso is one of the oldest companies in the world, rooted in the Nordic forests. For its success, Stora Enso does not rely only on the richness of its heritage but on strong values and a commitment to innovation. Today, Stora Enso provides low-carbon alternatives across many industries, including construction. Wooden buildings store carbon over their lifespan, help to reduce energy use in buildings and have a low carbon footprint. Stora Enso is leading the way in transition from a fossil-dependent world with finite resources into a renewable, sustainable world. As the renewable materials company Stora Enso wanted the new head office to be located in a wooden office building. As a visionary statement the new building should reflect the ecological diversity, resilience and most importantly the beautiful sensitivity of the forest. The new building should present the versatile relationship between the Nordic people and their forests, accomplishing this in a uniquely insightful way through a harmonius synthesis of the forest and surrounding urban environments. The look and feel of the building should

be defined by a commitment to biophilic design. Aesthetically, the new building shall be attractive and inspiring to staff, but also support the employees' wellbeing and their ability to maintain a work-life balance.

The new building should be a low-carbon building based on extensive use of wood, energy efficient solutions and by using energy from renewable sources. Building is also aiming to the highest levels of sustainability in terms of environmental certificate and aims for LEED Platinum certificate, which takes a wide range of sustainability aspects into consideration.

1.4. Early stages of the development including architectural competition

The South Harbour area is directly connected to the historical centre and the downtown of Helsinki. The harbour is the origin and the roots of Helsinki. A village has grown into a town and town into a city. At the same the relation of the city and the harbour has changed. The City of Helsinki has a strategic goal to vitalize further the old harbour area and the neighbouring historical centre. The new building is situated in a nationally significant cultural environment.

As the anchor tenant of the new office, it was agreed to use Stora Enso's massive wood elements in construction. The new seafront building should also take its shape based on the Office Building Concept by Stora Enso, which was published in 2019. Katajanokan Laituri will also include a hotel and serves as a meeting place for city dwellers, tourists, and service providers in one of Helsinki's finest locations.

Architectural design competition was defined as the best way to secure the functionality, sustainability and aesthetic requirements of the new building in this kind of a unique and valuable site. Architectural design competition was arranged by Varma, Stora Enso, and the City of Helsinki. Six architect offices were invited to participate in a design competition in early 2020. Each of the selected architect offices brought their own unique set of strengths and experience to the competition. Aim of the competition was to create a high standard and sustainable premises that coincide with nationally significant cultural environment, and which also consider the opportunity for people to enjoy the seaside location. Architect offices were invited from Finland, Sweden, Norway, and Japan, including ALA Architects (FIN), Anttinen Oiva Architects (FIN), PES-Architects (FIN), White Arkitekter (SWE), Snøhetta (NOR), and Shigeru Ban (JPN).

The winner of the competition was announced in June 2020. All competition entries were evaluated anonymously, without knowing which architect office was behind of each proposal. Winner was Anttinen Oiva Architects with their winning proposal "Spring":





Pictures 1 and 2 by Anttinen Oiva Architects: Winning architectural proposal "Spring"

"Strongly grounded in its site and surroundings, Spring looks towards the future and sets an example in sustainable growth and the possibilities of wood construction in a sensitive urban environment. Both urban and natural, it is strong but serene on the outside, while its interior and materiality evoke the dense but tranquil atmosphere of a Finnish forest. Its architecture embraces and elevates its surroundings, extending and reiterating the city's grid and silhouette in harmony with the old. Through a powerful, but open and inviting gesture and presence, it re-activates its surroundings and the city's waterfront.

Building represents sustainable architecture in every meaning of the word: flexible structural solution capable of accommodating a range of different uses during the building's lifespan, adaptable spatial organization combined with form and materiality that is true to its values – architecture with a true human approach. It showcases low-carbon construction with innovative and creative use of standard Stora Enso massive wood products and intelligent technical systems. Its spatial solutions allow a diverse and flexible interaction between people and the different functions of the building.

Inventive and inspiring, Spring represents dynamic and modern, but timeless and firm architecture that fits and elevates its surroundings while following biophilic design principles. The building and its formal solutions create a clear identity that stems from its historic urban context as much as a Nordic forest and relationship to nature. Spring offers proud and strong urban presence with the warmth, smell and feeling of a Nordic forest after the rain" (Invited Architectural Design Competition – Jury Evaluation Report, 2020)

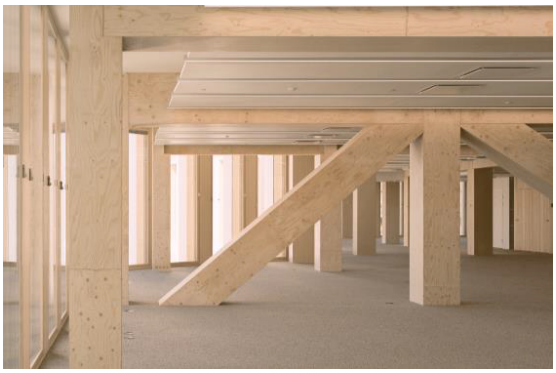
1.5. Construction of the new wooden building

Before any new construction started, an old warehouse dating back to 1960 was standing on the site in South Harbour. One of the preconditions for new development was to get approval for new detailed city plan. New detailed city plan was approved by the City of Helsinki in June 2021. Old warehouse was demolished, and materials were recycled as far as possible. The building was no longer in use for its original purpose and had no special architectural or aesthetic value. Eventually 99,5% of all materials demolished from the warehouse were able to be recycled. Construction of the new building began with groundworks in January 2022. One of the main challenges in this site was to ensure water tightness of trench walls used around the excavation. Excavation achieved level -4.5m below sea level in August 2022, and first sections of base slab for the new building were casted in September 2022 by using green concrete to mitigate climate impacts of structures below the ground. Manufacturing of first wooden elements started in October 2022 at Stora Enso LVL mill in Varkaus, Finland. Majority of all load-bearing structures above the ground are made of wood by using CLT and LVL by Stora Enso. Installations of the wooden frame began at site in March 2023. From the ground level to the topping out of the roof, including the intallation of pre-fabricated façade elements, the entire wooden frame was completed in just seven months and perfectly on schedule. The Sylva prefabricated kit of parts was delivered using 171 just-in-time deliveries which greatly reduced the need for storage and on-site labour. This level of accuracy and speed demonstrates that building with mass timber is a not only a viable alternative to steel and concrete, it's also now one of the best architectural examples in the world that building with mass timber can develop utterly unique and complex structures from a prefabricated and standardized kit of parts and achieve outstanding, first-time-right results.

The goal of minimizing the climate impact during the whole life cycle of the building guided the design from the beginning. Climate change mitigation is one of key goals of Varma who aims to be a pioneer in reducing the carbon footprint of its properties. Construction from Stora Enso's massive wooden elements help to aim for these climate goals. In Katajanokan Laituri sustainability is considered from much wider perspective than just as an environmental certificate. Katajanokan Laituri is developed as nature-respecting, energy-efficient, and intelligent building made of wood. The building is a great example to showcase how to use wood in sustainable urban construction.



Picture 3: Stora Enso / ©Puurakentajat Group Oy



Picture 4: Stora Enso/©Kalle Kouhia

1.6. Architectural vision

From the outset, there was a commitment to inspire wellbeing and let only resource-efficient methods and materials inform the design. Adapting to the the marine climate and accounting for inevitable rising sea levels, the end result is an astounding landscraper in both its visual appeal and engineering.

The sense of wonder and awe Katajanokan Laituri evokes is best experienced in person to fully appreciate the acoustics, sensory touch, and resinous smell of natural wood. After only a few moments of being inside the building, visitors remark on the sense of calm and wellness all around them. The space is nothing short of a modern day tribute to Finland's finest artisans and craftspeople and an ode to what is possible when there is an unwavering commitment to climate-aligned design.

Katajanokan Laituri is a remarkable 23,000 m² mass timber building, the largest of its kind in Finland and a new low-carbon marvel capturing attention across the architectural world. The original vision was to let the most resource-efficient methods and materials shape the building's design and construction. The result is a landmark in low-carbon construction.

1.7. Low-carbon materials

The structural frame consists of over 7,600 cubic meters (m³) of wood, prefabricated into over 2,000 bespoke load-bearing elements. Each individual piece was manufactured with millimeter precision and delivered as a kit of parts based on Sylva™ by Stora Enso. The wood for the structure originated in sustainably managed Nordic forests that use third-party traceability certifications.

The load-bearing laminated veneer lumber (LVL) columns and beams were further processed by Finnish timber specialist, Stora Enso’s subcontractor, Punkaharjun Puutaito Oy (PPT). Stora Enso manufactured LVL at Varkaus mill, Finland, and delivered LVL panels to PPT where panels were cut to shape and block-glued to their final dimensions before sanding and treating columns and beams with UV and moisture protection for long-term durability.

The second mass timber product, based on Sylva kit of parts by Stora Enso, is cross-laminated timber (CLT). CLT is used on Floors, Walls and Roof elements manufactured at Stora Enso mill in Gruvön, Sweden. Even the elevator shafts and stairs of Katajanokan Laituri are made with millimeter precise CLT elements by Stora Enso.

When entering the building, visitors are intuitively invited to rest on a signature circular seating area that encapsulates the concept of the infinite potential of renewable materials. The round-shaped lobby bench is made entirely from LVL and is a tribute to Finnish craftmanship. As your eyes naturally gaze upwards into the curved central atrium, you will find yourself immediately reaching for your camera to capture the incredible sculptured curvatures of the CLT. Both features, the round-shaped LVL bench and the curved CLT lamellas around the atrium, were crafted by Finnish timber specialist, Raision Puusepät Oy.

In Katajanokan Laituri also the window components are wooden. The 700 wrap-around windows offering beautiful views of Helsinki that bathe occupants in Nordic light are supported by engineered wood components made with Effex® Dura by Stora Enso manufactured in Honkalahti mill, Finland.

1.8. Sustainability

Design is led from the start by a commitment to minimise climate impacts and demonstrate the remarkable possibilities of wooden construction. Compared with a concrete-based building, specifying wood for the structure resulted in an impressive 35% reduction in before use greenhouse gases. On top of having lower carbon footprint than a benchmark concrete building, wood also stores carbon. No other commercially available building material can do that.

This isn't just about today – the mixed-use building concept ensures the building can adapt to future needs, reinforcing a sustainable approach to urban development and construction. The wooden elements used in the construction captured 6,000 tonnes of carbon dioxide (CO_2) during the trees' growth and are now storing it in the building for at least the next 100 years. This carbon storage is remarkable when we consider it will take the next 50 years heating and cooling the building and other operational activities to produce the same amount of CO_2 emissions.

In addition to its sustainability benefits, the building's biophilic design – a concept that integrates architecture and materials to foster a closer connection to nature – includes green roofs and an outdoor forest. These natural elements, combined with the use of organic materials and thoughtfully designed lighting inside, create an environment that enhances well-being and fosters a sense of harmony for all who visit this remarkable space. With its mass timber structure, Katajanokan Laituri is designed to reach the highest level of LEED certification – LEED Platinum status.

2. A celebration of nature and renewal

Katajanokan Laituri is more than a building; it's a cultural hub and a living testament to the power of sustainable architecture. It is designed as a multi-sensory experience, with nature at its core.

In the lobby, you'll find an iconic circular bench which is the embodiment of the continuous cycle of renewable material and magnifies the true potential of wood. The Stora Enso story and history is told through this original piece of art themed "Hidden Treasures".

Made with LVL from our mill in Varkaus, sourced from the tough and densely grown spruce of the Savo region's forests, and designed by Anttinen Oiva Arkkitehdit, the bench has further come to life in the hands of Finnish craftmanship and an artistic vision of what Stora Enso promise and create.

Glass pieces atop the wooden bench integrate copper, forest, wood chips, lignin, NeoLigno, wood foam (Papira), and TreeToTextile fibers, to convey our transformation and evolution, our heritage from copper mining and forestry to the infinite possibilities of renewable materials.

The bench and glass pieces are a moving manifestation of Stora Enso's resilience and adaptability to change. Literally. With each step you take around the bench, the reflections in the glass pieces change, one moment a reflection of the dome ceiling, another of a person standing next to you, or another of beaming light.

This historic site in Katajanokka, Helsinki, once a bustling shipyard, has been reimagined into a beacon of modern, circular design. Woven into Helsinki's harbor and coastline, this architectural masterpiece not only highlights Finnish craft but embodies our deep commitment to renewable materials.

We invite you to explore Katajanokan Laituri, where world-class architecture meets vibrant public spaces. Discover a dynamic mix of amenities, including a hotel, conference centre, green roof, alongside thoughtfully designed public areas – all reflecting the innovative spirit of Stora Enso. Welcome to Stora Enso's new head office, the largest mass timber building in Finland and a landmark of sustainable, low-carbon construction.





Pictures 5 and 6: Stora Enso/©Kalle Kouhia, ©Tuomas Uusheimo

Key Partners

Developer
Varma Mutual Pension Insurance Company

Architect
Anttinen Oiva Architects

Anchor tenants
Stora Enso Oyj - Head Office
Solo Sokos Hotel Pier 4

Structural Engineer
Sweco Rakennetekniikka Oy

Project Management
Haahtela Oy

Structural mass timber and window components manufacturer
Stora Enso Oyj

Specialist Timber Contractors and Subcontractors
Puurakentajat Rakennus Oy (structural frame installation)
Haka Pks Oy (prefabricated facade elements)
Raision Puusepät Oy (round-shaped LVL bench and curved CLT lamellas around the atrium)
Punkaharjun Puutaito Oy (further processing of LVL columns and beams)
Timberpoint Oy (further processing of LVL facade structures)

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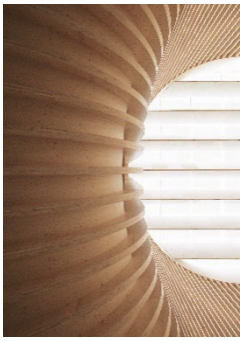
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Stora Enso Head Office- About us | Stora Enso

Stora Enso Reference Library - Katajanokan Laituri | Stora Enso Head Office (Sep 2024)
Katajanokan Laituri | Stora Enso Head Office

Block 4A
Large Buildings

Katajanokan Laituri – urban scale wood architecture in stone city

Anttinen Oiva Architects
Helsinki, Finland



Katajanokan Laituri – urban scale wood architecture in stone city



Picture 1: Katajanokan Laituri, view from sea. (Photograph: Kalle Kouhia)

Katajanokan Laituri (Katajanokka pier) is an office and hotel building with a solid wood structure located in Helsinki´s historic waterfront. As the first new building on the formerly industrial Katajanokka waterfront that directly connects to Helsinki's iconic cityscape opening towards the sea, it spearheads the transition of this formerly closed-off part of the embankment into an open urban space accessible to all. The building is the result of an invitation-only international architectural competition held in 2020.

Katajanokan Laituri building was designed as the new headquarters for Stora Enso, a leading Nordic forest industry company, whose previous, iconic headquarters designed by Alvar Aalto are located nextdoor to the site. Half of the building was designed as a hotel and is occupied by Solo Sokos Hotel Pier 4. The building is owned by the Finnish Varma Mutual Pension Insurance Company. Construction of the project was managed by Haahtela.

The building has four floors above the ground, a publicly accessible green rooftop terrace, and a basement that houses technical facilities and parking. The street-level spaces feature a restaurant, a café, conference spaces and other services that all open directly towards the surrounding city. A large foyer space that connects the city to the seafront is shared by the different operators of the building and acts as an open space for encounter.

The building continues the iconic, uniform, neo-classical silhouette of Helsinki along its southern waterfront. On the other side, it connects to the diverse and historically layered building stock of the Katajanokka peninsula, which features industrial harbour buildings as well as housing from different periods, including famous turn-of-the-century Jugendstil

housing blocks. The aim of the design is to create a peaceful and harmonic presence in the classical waterfront cityscape and a multi-faceted and animated experience for the pedestrian that resonates with the different scales, styles and shapes of the surrounding Katajanokka neighbourhood. The free-form and flowing shapes of the facades break the scale of the large, elongated block structure dictated by the harbour´s historic storage facilities. They also create sheltered enclaves for the outdoor spaces and open up variable views to the surrounding landscape.

Katajanokan Laituri sets an example for the use of wood for the sustainable development of cities. The entire project has from its outset been directed by the objective of minimising climate impacts over a long lifecycle. The aim has been to maximise a long lifespan, flexibility for conversion, versatility of use, and energy efficiency. The structural system and the design of the spaces allow for a maximal adaptability of the interior for possible future needs. The wood elements used in the building store 6 000 tonnes of carbon dioxide (CO_2) that the trees have bound during their growth. An efficient use of the underground spaces has been optimised by a robotic parking system.

The building´s above-ground structures utilise standard, industrial, prefabricated Stora Enso massive wood products. The post-beam-slab frame and the structure for the façade are made of Laminated Veneer Lumber (LVL) produced at Stora Enso's factory in Varkaus, Finland. The inner walls and lift- and staircase shafts that stiffen the structure, as well as the floor- and roof structures, are made of Cross Laminated Timber (CLT) produced at Stora Enso´s factory in Gruvön, Sweden. The trees used are from within 100 km of the production facilities. The entire frame consists of 7600 m^3 of spruce and 2500 wood elements. Inside the building, this structural wood has been left exposed as much as possible.

The façade of the building continues the clear, harmonious and uniform classical edifice of the city that begins from the Esplanadi Park and extends through the Market Square to Katajanokka with Aalto's white former Stora Enso Headquarters. The façade solution for this both technically difficult and historically sensitive site is based on a double structure. The outermost protective layer of glass, natural stone and aluminium connects it to the surrounding, predominantly stone surroundings. Behind this visually unified layer, the deep warmth of wood variates in several different layers. The outer character of the building is the result of the interplay of the two layers, the appearance of which alternates with light conditions throughout the day as well as the four seasons.



Picture 2: Katajanokan Laituri, main foyer. (Photograph: Kalle Kouhia)

The interior of the building utilises wood in many forms and formats. On the street-level, hefty LVL columns and the sculptural CLT covered opening towards the sky form a distinct and impressive public space. The colours of the building are deliberately natural. Changing shades and patterns of spruce alternate with ash detailing and exhibit the versatility and flexibility of wood as a material. The design of the interior has been guided by a preference for materials that are natural, sustainable and will age beautifully. The Viitasaari grey granite, used on the facades and on the pavement outside, continues inside on the floor of the street-level public spaces. The furniture for the foyer exhibits the best of Finnish design, while the office spaces have been furnished using largely recycled elements and furniture.



Picture 3: Katajanokan Laituri, office space with exposed LVL structures. (Photograph: Kalle Kouhia)

The outdoor spaces of the building are inspired by northern nature. Terraces that open above the ground floor to different directions as well as the interior courtyard all display natural habitats appropriate for their environments. On the narrow, courtyard the canopy of a birch grow protects a multi-spiece forest floor below. The terraces feature natural stones and species typical for the rough conditions of the archipelago, while a northern coastal meadow takes over the rooftop terrace. In the midst of the meadow, there are solar panels, a restaurant terrace, and areas for leisure and workout. The versatile outdoor spaces of the building create a small enclave of nature in the midst of the city – for human beings as well as other species.

The building's close proximity to the sea means that extensive flood protection measures have been implemented as part of the building project. As the first block redeveloped on the Katajanokka waterfront, this site spearheads the reinforcement and raising up of the entire embankment. Until the larger reconstruction of the entire embankment takes place, the surrounding area is reinforced and organised with temporary solutions by the City of Helsinki. They pave the way for a future permanent reconstruction of the embankment as a continuous public space from the Market Square to the tip of Katajanokka.

Katajanokan Laituri – project information

Location: Helsinki, Finland

Timeline: Invitational competition 2020, completion 2024

Area: gross ~23,000 m^2, building legal floor area ~16,400 m^2

Client: Mutual Pension Insurance Company Varma

Project management: Haahtela-rakennuttaminen

Main users: Stora Enso Oyj, Sokos hotels / Solo Sokos Hotel Pier 4

Architect: Anttinen Oiva Architects

Interior design / Stora Enso premises and main lobby: Anttinen Oiva Architects

Interior design / Pier 4 hotel facilities: Franz

Structural design: Sweco Rakennetekniikka

Fire engineering planning: KK-Palokonsultti

Landscape design: Nomaji

Acoustic design: Akukon

HVAC, Electrical and Life Cycle Planning: Granlund

GEO planning: Sitowise

Block 4B
Barriers and harmonization of standardization

Eurocode Highlight – Safe rules for brittle failures

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Faculty of Science and Technology
NMBU, Norwegian University of Life Science
Ås, Norway



Eurocode Highlight – Safe rules for brittle failures

1. Introduction

1.1. Evolution of EN 1995

Eurocode 5 (EN 1995-1-1 [1]) is the European standard governing the design and construction of timber structures, encompassing a wide range of wood-based products used in buildings and civil engineering works. This includes materials such as solid timber (sawn and planed wood), glued laminated timber, and wood-based panels assembled using adhesives or mechanical fasteners. Initially developed during the 1980s and 1990s as part of the first generation of Eurocodes, Eurocode 5 has been a key framework for ensuring safe and efficient timber construction across Europe. Over time, as advancements in engineering knowledge and construction practices have emerged, the code has undergone significant updates in its second generation to address earlier shortcomings, align with modern regulations, and reflect current best practices.

The second generation of Eurocode 5 introduces important updates aimed at enhancing the design and construction of timber structures. These revisions represent the forefront of timber engineering, ensuring the code remains relevant and effective in meeting contemporary construction challenges. Key changes include the incorporation of new materials, revised design procedures, and enhanced fire design rules.

Additionally, the updated code now includes provisions for ductility, robustness, and brittle failure analysis, ensuring timber structures can withstand various stresses and loads.

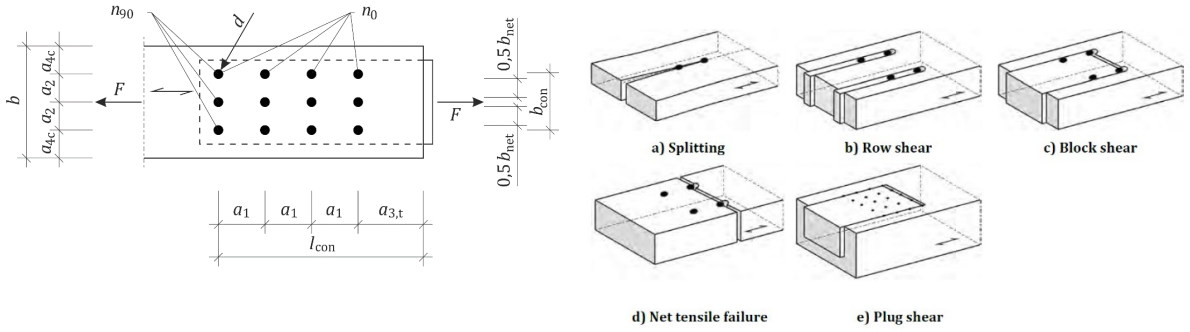
Currently, the complete set of European design standards is transitioning from the first to the second generation of Eurocodes. This shift brings a substantial increase in complexity and page count. For example, prEN 1995-1-1 [2] has expanded from 123 to 440 pages. While the new codes offer greater precision and address topics absent from the first generation, timber must still often be treated as a brittle material, and the fundamental characteristics of the material remain unchanged. As a result, expertise and competence in design will continue to be essential for safe and effective timber construction.

1.2. Brittle failure modes

Chapter 11.5 of the latest draft of Eurocode 5 outlines design rules for brittle failure modes of connections with dowel-type fasteners [2]. When multiple fasteners are used in steel-to-timber and timber-to-timber connections, the potential for brittle failures caused by splitting, row shear, block shear, plug shear, and net tensile failure must be considered, see Figure 1 II.

In the current version of Eurocode 5 (EN 1995-1-1:2004) not all these brittle failure modes were explicitly addressed in the main text. Instead, prescriptive limitations were provided, focusing on minimum spacings (a_1, and a_2), edge and end distances (a_3, and a_4), and effective numbers of connectors in the grain direction n_{ef}. These prescription limitations aimed to mitigate the risk of splitting (a), without directly addressing the other failure modes. Annex A, however, does provide rules for block shear and plug shear failure, but only for multiple dowel-type steel-to-timber connections. These rules were informative, based on a plastic definition of effective thickness, which could overestimate the strength of the resistant plane if not compensated in the design procedure [3].

The latest draft of Eurocode 5, prEN1995 [2] introduces a complete and updated design procedure for all brittle failure modes in connections with dowel-type fasteners, as outlined in Figure 1 II. This new approach heavily incorporates the model developed by Yurrita and Cabrero, which is based on extensive experimental investigations [4], [5], [6].



(I) (II)

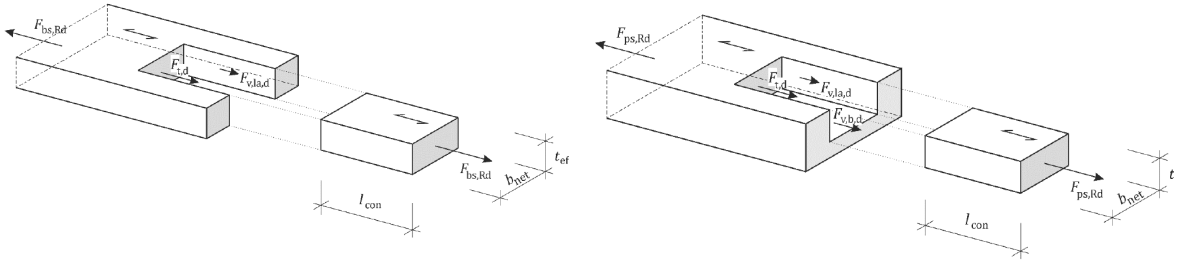
Figure 1: (I) Basic geometry of a generic multiple fastener connection; (II) Brittle failure modes in connections with dowel type fasteners as depicted in Eurocode 5 draft prEN1995-1-1 [2]: splitting (a), row shear with full penetration (b), block shear or tear-out (c), net tensile failure (d) plug shear (e)

The splitting (a) and net failure (d) modes are managed using the same approach as in the current Eurocode 5 version. Splitting is addressed through the effective number of fasteners, while net failure modes are managed via explicit tension verification of the net section in the connected members.

The other failures modes row shear (b), block shear (c) and plug shear (e) are now explicitly calculated by modelling the design resistances of individual failure planes:

- side shear planes,
- bottom shear plane
- head tensile plane in block shear.

In Figure 2 are reported the description of the individual failure planes for block and plug shear failures modes.



c) Block shear failure surfaces e) Plug shear failure surfaces

Figure 2: Brittle shear failure modes c and d

The equations for side shear planes and head tensile plane are based on the concept of an effective thickness (see Figure 2) which differs from the approach proposed in Annex A of the current Eurocode. Specifically, the correct definition of effective thickness must account for the fact that brittle modes occur before the full plasticization of the fasteners [7].

Brittle failure modes of connections may be neglected provided that both the following conditions apply.

- First Condition: in the governing shear failure mode of the dowel type connectors all the plastic hinges in the steel dowels are formed, and the characteristic strength of per dowel and shear plane are given by the following formula:

$$F_{D,k} = 1.15 \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2\, M_{y,k}\, f_{h,1,k}\, d} \qquad (1)$$

- Second condition: the minimum spacings are met and the spacings a_1, a_2 and loaded end distance $a_{3,t}$ are increased by the factor k_{br} higher than 1 given in [2].

It's important to note that this simplification can lead to oversized connections, making it a less favourable approach. Another method for preventing splitting failure is the use of adequate reinforcement perpendicular to the grain, as shown in Figure 3. In that case the effective number of fasteners may be taken as $n_{ef} = n_0$.

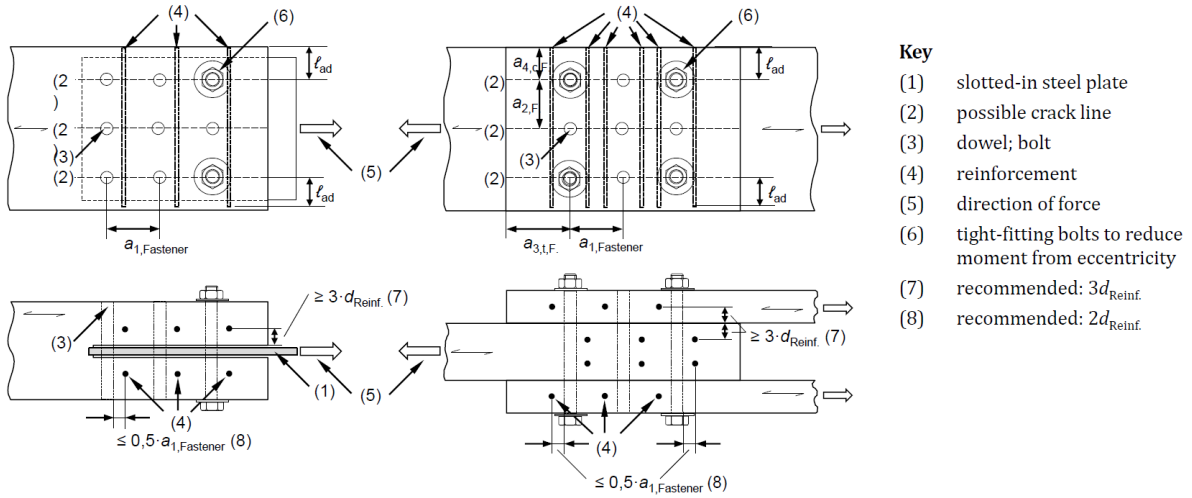


Figure 3: Reinforcement of dowel-type fastener connections

Lateral reinforcement can influence also the failure mode c in Figure 1 II. The block shear failure mode is in general resisted by the block net tension head area or the net shear area at the sides of the block. In general, one or the other of these two resistance effects is the strongest and is giving the design resistance. These resistance effects are generally not additive, as combining them would require identical deformation patterns at the point of fracture. However, with adequate reinforcement, prEN1995-1-1 proposes an additive model. For small head tension areas, tensile properties limit the capacity. In larger blocks, the shear capacity may govern the overall strength.

2. Case study: Tretten bridge

2.1. Introduction

Tretten bridge, a two-lane combined steel and timber truss bridge, was finalized in 2012. The bridge concept may briefly be explained as U-shaped moment rigid frames in the transversal direction formed by the verticals and the crossbeams in steel. All diagonals and upper- and lower chords are in wood (glulam), except the noses for landing the trusswork at the abutments, which are in steel. The deck, resting on the crossbeams, is a stress-laminated continuous timber deck with asphalt topping. The two bridge trusses were manufactured in 6 subframes each and connected by installation of the dowel in the joints on site. Both the upper and lower chords are continuous, except at the 5 splicing locations. More details about the bridge is present in a conference paper, see [8].



Figure 4: Finished bridge in 2012.

The Norwegian Safety Investigation Authority (NSIA), together with an external expert group, have carried out the technical investigations with respect to the collapse. The investigation is documented by the report [9].

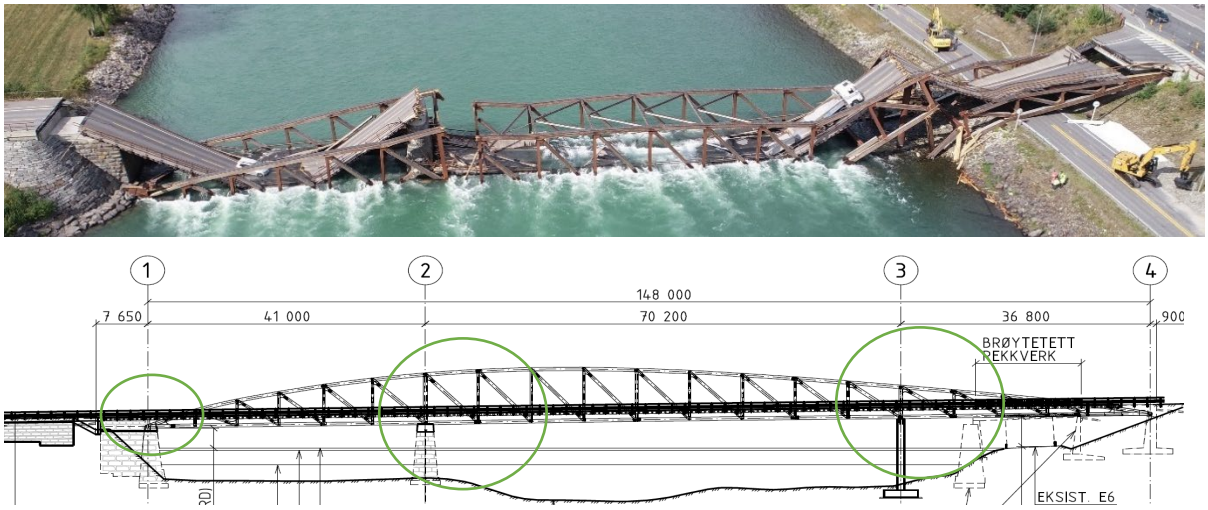


Figure 5: Collapsed bridge and bridge zones with top priority (marked with circles).

2.2. Bridge collapse and investigation

On the morning of 15 August 2022, Tretten Bridge in Norway collapsed and fell into the river Gudbrandsdalslågen and onto the E6 highway. The bridge collapsed without warning, for traffic loads much smaller than the design load. Luckily no-one was seriously injured, both drivers on the bridge at the collapse were safely rescued.

Shortly after the collapse, the local police closed the actual parts of the connected roads and started to explore the situation. The traffic situation on highway E6 required removal of the bridge parts blocking the highway as soon as possible. This work was challenging due to the large dimensions of the bridge components, rough stream in the river and a difficult situation with respect to the workers HSE conditions. Based on videos of the collapsed bridge, it was decided to put the emphasis on the zones marked with circles in Figure 5. The majority of the parts within the circles were carefully collected and made available for inspections.

2.3. Structural modelling for collapse sequence scenarios

As a part of the investigation, the bridge was modelled with different levels of detailing using several 3D programs in addition to a 2D program. The resulting axial force distribution indicates that the most utilized zones are close to axes 2 and 3.

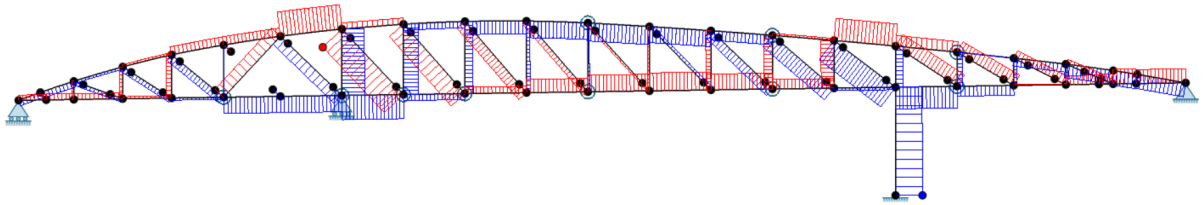


Figure 6: Numerical simulation of the loading immediately prior to collapse. Upper: Vehicle on critical locationfor zone around axes 2. Lower: Axial force distribution, resulting critical zones close to axes 2 and 3.

Several initial locations were explored, but it turned out that only failure initiation in diagonals 6 or 7 were consistent with the observed damages from the collapse, and had the potential to propagate the failure into a full collapse for the given load scenario with a single lorry of 490 kN on the bridge. However, an initial failure in diagonal 7 did not give immediate failure propagation, but merely additional damage to the neighbouring elements like diagonal 6. When a failure in diagonal 6 occurs, the failure will propagate. The potential elements for failure initiation are visualized on Figure 7.

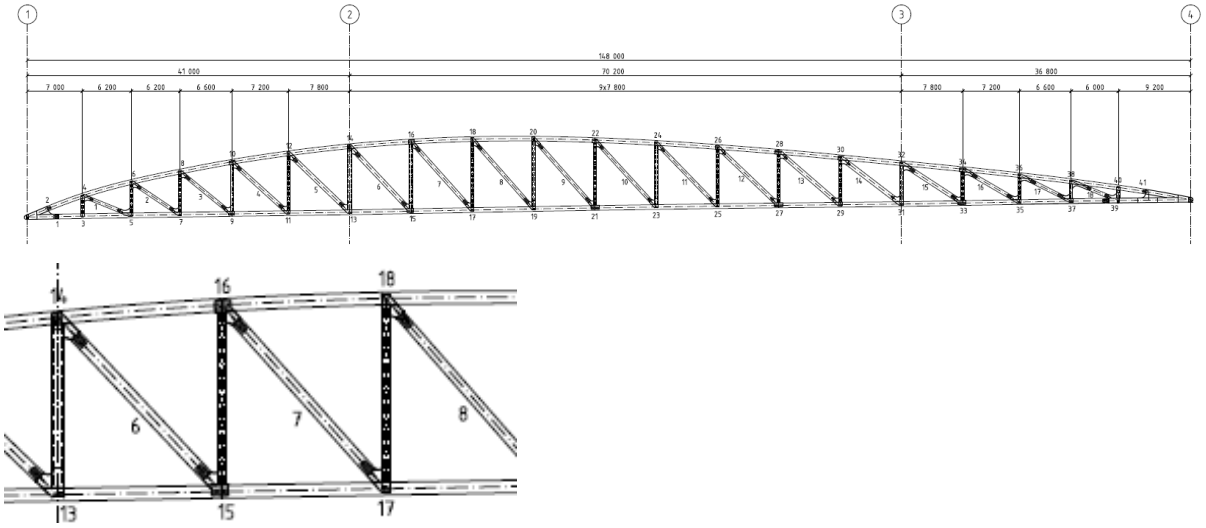


Figure 7: Potential elements for initial failure, diagonal 6 and 7.

The failure sequence is summarised as follows and visualized on Figure 8.

- The failure is initiated in diagonals 6 or 7.
- Left part axes 1-2 moves like a rigid body, the deck acts as a membran and pulls off the cross beams from the supports at axes 1 and 2.
- The splice and diagonals to the right of axis 3 become overloaded.
- The bolts at the crossbeam in the U-frame in axis 3 are clipped by the rotational movement of the centre part of the bridge and trusses fall.

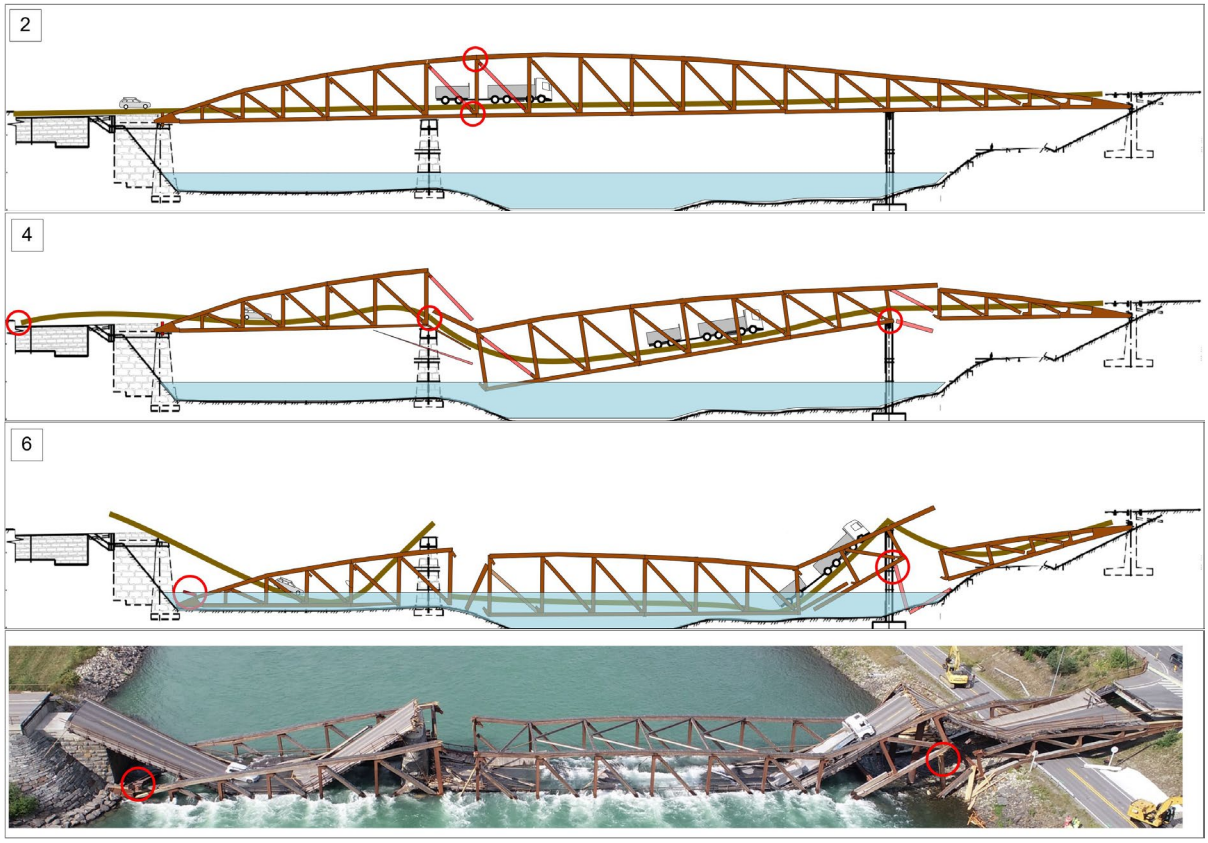


Figure 8: Visualization of the stages of progressive collapse.Failure mode and strength

The diagonals 6A, 6B, 7A and 7B all had large tensile forces (A = upstream truss, B=down-stream truss). The moments at the joints of the trusses were not large but might have effect on the strength of the connections. During the progressive collapse the moments in some joints became large and led to moment failure. Most of the failed joints in diagonals 6A, 6B, 7A and 7B showed variants of block (shear) pull-out failure, consider Figure 9.



Figure 9: Block (shear) pull-out failure joint 14B in diagonal 6B.

The material in the diagonals was GL30c, i.e. the inner lamellas (T14.5) had lower strength. The strength of the connections was determined to be:

- Diagonal 6: 2630 kN
- Diagonal 7: 1890 kN

These strengths are consistent with those in modern timber regulations like [1, p. 5], [10], but with mean values and all factors related to statistical distribution and safety removed.

2.4. Cause of failure

The sequence of the collapse from the physical evidence and the numerical simulations showed no contradictions and hence the failure probably started in one of the diagonals 6A, 6B, 7A and 7B. The mean tensile strength of the material of these diagonals was

estimated by tests and calculations, and based on this knowledge, the loading scenarios which might lead to the failure were investigated. In short there are only two possibilities, either overloading by too many or too heavy vehicles on the bridge simultaneously, or damage accumulated over time leading to reduced strength.

2.5. Extreme loading

The bridge collapsed with an ordinary truck passing the bridge in direction axes 1 to 4 (load scenario 1). The weight of the truck was about 50 ton or 490 kN. This loading is far less than the design load. The following loading scenarios for overloading were computed by the numerical FE models, both in 2D and 3D and the corresponding forces in the diagonals were compared to the mean strength.

1. Weight of truck at collapse: 50 ton (500 kN)
   Force diagonal 6: 1776 kN (Capacity =2630 kN)
   Force diagonal 7: 1400 kN (Capacity =1890 kN)
2. Largest special vehicle in 2015: 97 ton (970 kN)
   Force diagonal 6: 1969 kN (Capacity =2630 kN)
   Force diagonal 7: 1516 kN (Capacity =1890 kN)
3. Two 50 tons trucks are meeting on worst locations, or two 50 tons trucks are in a row in the same lane, or whole midspan filled with 25 kN/m in one lane (timber transport 60t per vehicle in a row without separation) give maximum forces:
   Force diagonal 6: 2215 kN (Capacity =2630 kN)
   Force diagonal 7: 1561 kN (Capacity =1890 kN)

From the above loading scenarios on the bridge, it is not likely that the bridge becomes overloaded by too much simultaneous traffic. From comparison of the special transport in 2015 (load scenario 2) to the load on the bridge at collapse (load scenario 1), it is obvious that the damage was not created by the truck on the bridge at collapse, but it must have been present in advance. No damage was reported after load scenario 2 in 2015, and no serious damage has been reported after the regular inspections of the bridge.
There is a traffic station which register the traffic passing the bridge, and only a regular traffic pattern has been recorded in the time before the collapse. There is therefore no trace indicating that overloading was the direct cause of the bridge collapse.

2.6. Fatigue loading

The heavy traffic on the bridge was put together based on the traffic registration and on the regular traffic to and from the dairy plant of Tine. Tine is not included in the traffic registration due to their entrance being located just after the bridge abutment in ax1s 1, which is just before the traffic registration location. The overview is an estimation of the heavy traffic which has passed the bridge during its lifetime of ten years.
It was judged that lorry number 3 specified in fatigue load model 4 (FML4) in EN 1991-2 [3] was quite representative with respect to load and load distribution of much of the traffic, so FML4-lorry 3 was used throughout the fatigue computations. The loading was then sorted after weight in load cases.

The fatigue computations considered all failure modes in the connections and most of the connections showed no vulnerability to fatigue damage. However, some of the connections were extremely exposed to fatigue damage, especially the connection in diagonal 7, which has passed its lifetime to an extreme degree with a Miner-Palmgren sum of more than 200. The next connection is the one in diagonal 6 with Miner-Palmgren sum more than 9, hence this diagonal was also well above its expected lifetime. Also, connections in diagonal 16 close to axis 3 showed too much fatigue damage. The fatigue damages were all due to tensile block (shear) pull-out failure mode.

2.7. Summary of Tretten bridge collapse

Tretten bridge had reduced strength prior to the collapse. There was no trace of traffic overload and the most likely cause of the collapse is damage accumulated over time i.e. fatigue. The failure mode was block (shear) pull-out failure. The bridge was designed

according to regulations [11] from the national road authorities and the Norwegian national standard for timber structures at that time (NS 3470-1) [12], which did not have any regulations regarding block (shear) pull-out failure. Diagonal 7 was much more exposed to fatigue than diagonal 6, and most probably failure initiation took place in diagonal 7. However, also a failure in diagonal 6 was necessary for propagation. Wood has large variability in strength, and by assuming that diagonal 7 is 20% stronger than diagonal 6, the order of failure initiation may be switched between diagonals 6 and 7. The fatigue is caused by too high stress concentrations in the head tensile area resulting from narrow dowel groups.

3. Enhancing the Robustness

The Norwegian Safety Investigation Authority (NSIA) conclusions after the Tretten bridge collapse are collected in two sub-reports, and the following quoting is from Sub-report 2.

Sub-report 2 focuses on the management and follow-up of Tretten Bridge throughout its life cycle. This includes design, control and approval of the bridge. It also describes how the bridge, since its construction, has been followed up by the Norwegian Public Roads Administration (NPRA) (2012–2020).
Tretten bridge was an architect-designed truss bridge in glulam and steel, and was constructed by the NPRA in 2012 as part of the project E6 highway Øyer-Tretten. The investigation has shown that inadequate caution was exercised in the planning, design, inspection and approval of Tretten Bridge, with respect to the risk factors linked to its unconventional design. A short construction period, span lengths and reuse of existing foundation were framework conditions for the planning of the bridge. These framework conditions, combined with the choice of material and a strong focus on aesthetics, contributed to the structure not being robust.
Tretten Bridge was designed during a transitional period between old and new regulations. According to the transitional provisions, the project had the opportunity to relate to the old regulations. In retrospect, all the involved parties can be criticized for selecting and accepting regulations that proved to have a serious shortcoming (block shear failure). However, the transition from national standards to European standards for structural design (Eurocodes) was extensive and complex.

In the following a brief overview on possible actions to increase the robustness of timber bridges with Tretten bridge as a case is discussed.

3.1. Design choices, Trusswork and Diagonals

Wood fibers are generally stronger in tension than in compressions, but the risk for knots, grain deviations and other irregularities reduce the characteristic strength in tension dramatically and for most species the design tensile strength is considerable less than the compressive strength.
Slender members in compression can be vulnerable to buckling, which are not the case for tensile member, but apart from the buckling issue, compressive members have seldom dramatic failures. Therefore, it is usually more robust to have highly stressed non-slender diagonals in truss works in compression rather than in tension. For the Tretten bridge this could have been achieved by letting the diagonals to the right of axes 2 and 3 be pointing with the bottom ends towards the foundation supports.

Mechanical connections in timber structures are usually made by making slots for steel-plates and holes for dowels, screws and bolts. Consequently, wood material is removed at the connections, making them the weakest part of the member and therefore also the most vulnerable parts. Furthermore, the strength of a connection is seldom stronger than half of the strength of the connected member, so a connection will always be a vulnerable part in a structure. Furthermore, for tensile members, the effective net section area is reduced due to the connections and usually no other load carrying effect can be utilized. For compression members stresses can often be transferred by contact between mating parts if the capacity of a connections is insufficient.

3.2. Redundancy and part removals

For the Tretten bridge, the top and bottom chords are too long to be manufactured in a single component and therefore they must be spliced at several locations. This reduces the load carrying capacities in the chords significantly at the locations of the connections.

In Figure 10, loading, axial and moment distribution for one of the critical locations for traffic loading is presented for Tretten bridge.

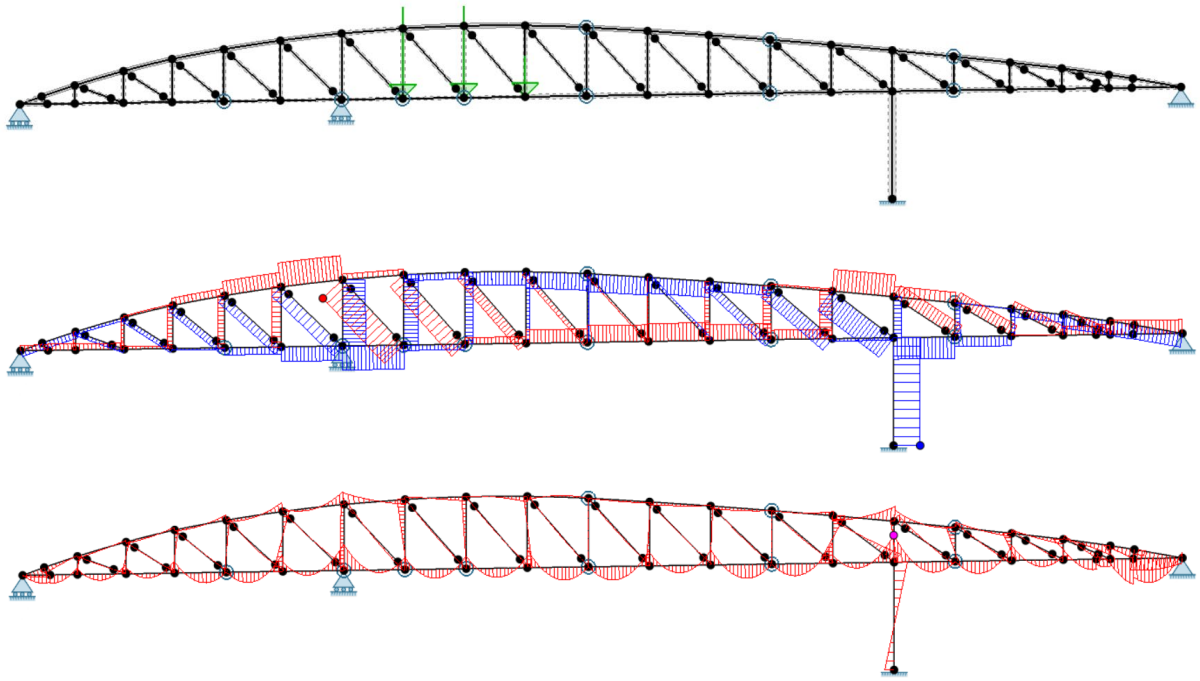


Figure 10: Numerical model of Tretten bridge, loading (top), axial force distribution (middle) and moment distribution (bottom).

Considering the axial force distribution in the middle plot; to the right of axes 2, there are two, maybe three diagonals which have large tensile forces. The same may be observed to the right of axes 3. These diagonals might be critical elements, since a tensile failure is brittle and has little strength reserve.

To explore the robustness of a structure critical elements or connections can be removed one at a time to evaluate if there is a risk for propagation of a possible failure, or if alternative load paths might be activated. This method is also proposed in prEN1995-1-1 Annex A, 2024. Theoretically, an instantaneous load application of a dead load will increase the impact with a dynamic factor of two. However, in this evaluation also the effect on strength from the duration of the load may be taken into account. Mainly two alternative computational methods might be used; one alternative is to perform quasi-static analyses of the effect of part removals. The other alternative is to perform dynamic analysis of the part removals, but it might be quite complicated as also the traffic loads are moving on bridges.

Although it might be quite common to model dowel connections in FEM software as hinged connections, they will probably be exposed to considerable moment actions due to their size and extent both in the axial and transversal directions. The dowelled connections will probably have smaller stiffness then their attached members, but the moments should not be neglected as the dowel connections are weak for moment exposure.

Considering the moment distribution (bottom of Figure 10), the largest moment are prone to occur at the joints in the truss work. This is an argument for to avoid splicing of the chords at the joints, it leads to less moment exposure if the locations for splicing are chosen in the middle of the element between the joints.

The vulnerability of splicing at the joints of the truss becomes very clear after an analysis with diagonal number 6 is removed, see Figure 11. The moments in the chords become very large, largest above the support in axis 2 (joints 13, 14), but also far above the capacity at the chord splices in the joints to the right of axes 2 (joints 15, 16). This give probably failure at the splice at joints 15, 16 first, as much material is removed due to the splice, but the capacity is also highly exceeded at joints 13, 14. This leads to a progressive collapse as no other possible load path can be found for the bridge structure. The chords should be designed to carry these extraordinary moments to compensate for the failed diagonal 6 in order to have a robust structure.

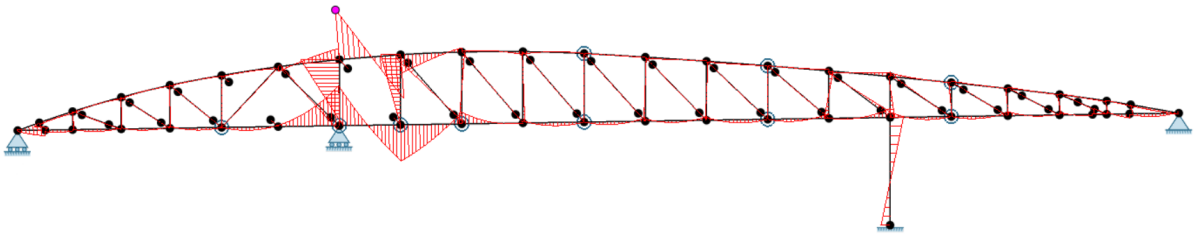


Figure 11: Diagonal 6 removed on Tretten bridge. Moment distribution

3.3. Dowel Connections and Ductility

Ductility in materials and connections is a mean of increasing the robustness of structures by letting overloading increase the deformations in the materials without significant increase in strength. In this way an overloading might be visible on the structure and action to prevent failures or collapse may be taken. Another effect of high ductility is that increased load actions might be redistributed to less critical parts of the structure and thereby the load actions on critical region limited. A common way to quantify ductility is to relate the non-liner deformation to the elastic deformation without significant loss of resistance.

For laterally loaded dowels installed in wood, crushing of wood in front of the dowels might take place, consider Figure 12 a). This is a kind of nonlinear deformation which might give some ductility. From experiments this is often observed, but mostly for connections having few dowels.

Only the wood material properties and geometry govern this type of failure, and their characteristic capacity per dowel and shear plane is calculated by

$$F_{D,k} = min\begin{cases} f_{h,1,k}\ t_{h1}\ d \\ f_{h,2,k}\ t_{h2}\ d \end{cases} \qquad (2)$$

where $f_{h,1,k}$ and $f_{h,2,k}$ are the embedment strengths in timber parts 1 or 2, and the thicknesses are t_{h1} and t_{h2}, respectively. The dowel diameter is d.

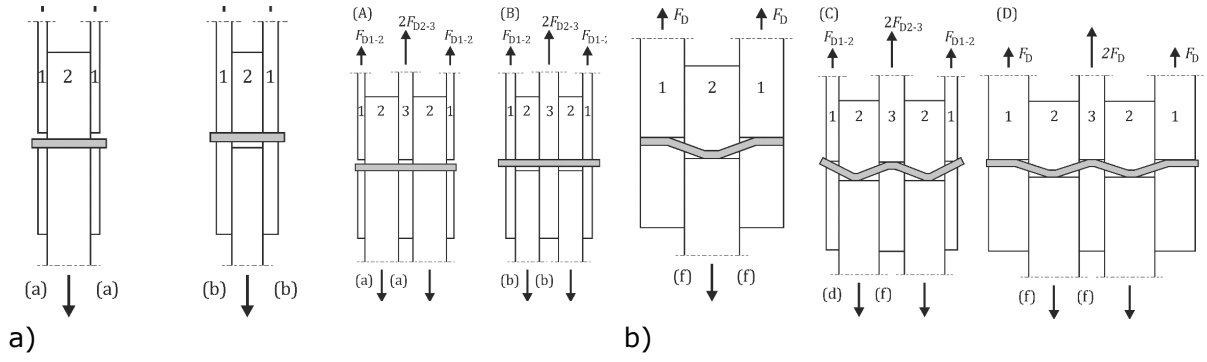


Figure 12: Dowel type connections with crushing of wood and yielding of the fasteners

For dowels having several rows and several dowels in each row, the wood crushing in front of dowels may easily lead to splitting as in Figure 1 II.

The general way to avoid splitting in group of fasteners is to have sufficient spacing between fasteners along grains a_1, and perpendicular to grains a_2, as well as end $a_{3,t}$ and edge $a_{4,c}$ distances, confer Figure 1 I.

If there are several dowels in rows, the effective capacity is smaller than the sum of individual dowels, which is taken care of by using an effective number of dowels per row $n_{ef} < n_0$. This is a way to limit the risk of splitting. An improvement regarding risk of splitting of dowel type connections is to install reinforcement perpendicular to grains by using e.g. fully threaded screws. Then the effective number of dowels can be taken as $n_{ef} = n_0$.

The dowels are usually made out of steel and as some steel qualities have large nonlinear deformations before failure, a possible way to introduce ductility in dowel connections is to make the steel dowels the weakest part in the connections and letting them get yielding for the design loading. The optimal way is to let the steel dowels be embedded and clamped in the timber parts and thereby develop yielding zones in the vicinity of the shear planes as visualized Figure 12 b).

In this case both the yielding properties of the steel dowels as well as the embedding strength of the timber parts govern the characteristic strength of per dowel and shear plane, as given in equation (1).

It is considerable uncertainty regarding ductility due to bending of steel dowels. Steel dowels are usually delivered with guaranteed minimum yield and strength properties, but their real properties might be considerable higher than the guaranteed properties. In that case the dowel bending modes might lead to higher resistance than the crushing/splitting mode and no ductility is achieved.

Note that for fatigue exposed connections slender dowels and weak bending properties of steel dowels might lead to fatigue in the steel dowels as the bending stresses are prone to give fatigue failure.

For wide blocks or high tensile capacity, it is a risk that row shear failure occurs, i.e. the wood between the fasteners fails in shear and in these cases, it is only the shear properties which is involved in the failure, see Figure 1 II.

Finally, as the installation of fasteners and connectors usually remove wood material from the timber element, a check of the net tension area is mandatory, to assure the necessary tensile capacity, confer Figure 1 II. Tension failures are very brittle.

3.4. Fatigue

Fatigue is always a brittle failure mode and the development of fatigue damage is very hard to discover prior to failures. The driving forces behind a fatigue failure is the level of the stresses and the number of cycles, and it is possible to estimate the risk of fatigue failure by using data and methods given in current EN1995-2 or prEN1995-1-1. The fatigue phenomenon in timber structures is sparsely investigated and more test results should be provided in order to enhance the reliability to the design rules.

3.5. Concluding Remarks

The call for enhanced ductility might be contradictory to the need for fatigue strength, especially in road bridges or other fatigue exposed timber structures. However, there is no contradiction between robustness and fatigue strength. Robustness of timber structures should be obtained by overstrength of critical parts or increased redundancy.

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Eurocode 5 – 2nd Generation – Design of Cross-Laminated Timber

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Eurocode 5 – 2nd Generation – Design of Cross-Laminated Timber

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Summary

The development of cross-laminated timber (CLT) over the past 20 years represents a significant progress for the timber construction sector illustrated e.g. by the tall timber movement. Consequently, it is evident to introduce this now commonly established timber product into European design standards. This contribution features the key contents and background information on the new developed sections for the design of *cross-laminated-timber* for the next generation of Eurocode 5. It also elucidates changes compared to previously applied design methods.

Key words: Eurocode 5; Cross-laminated timber; Reinforcement; Self-tapping screws; Wall diaphragms; Concentrated loads; Shear failure mechanisms;

1. Introduction

Since the first pilot projects and technical approvals in the mid-1990s, cross-laminated timber (CLT) has undergone impressive development [1]. The continuously growing number of production facilities and general technical approvals (abZ) or European Technical Assessments (ETA) clearly demonstrate the increasing acceptance and attractiveness of this now-established construction product. The industrial production of CLT enables a high degree of prefabrication; large-format wall, ceiling, and roof elements, including openings and conduit routing, can easily be manufactured. On the construction site, these elements can be assembled quickly and easily, for example, using fully-threaded screws [2]. CLT elements are predominantly used in the construction of single and multi-family homes, multi-story residential buildings, schools, and commercial structures.

The current version of the European design standard for timber structures, Eurocode 5, [3] does not include specific provisions for the design of cross-laminated timber. The code-compliant application of cross-laminated timber has only been made possible in a few European countries through non-contradictory complementary information to Eurocode 5 (NCCI) in the national annexes (NA) [4], [5]. To address the evident gap in Eurocode 5, the design of *Cross-Laminated Timber* has been prioritized on the list of work items for its revision [6].

2. Method

As illustrated in Figure 1, standardization is the consequence of successful research and development, which has found positive application and acceptance in practice. According to the European position on standardization, harmonized technical rules for common design cases should be developed, containing only rules based on widely accepted research results that have been validated through sufficient practical application and related experiences. The target group for such design rules is structural engineers with at least 3 years of experience in the respective field — qualified professionals capable of working independently in their field [7].

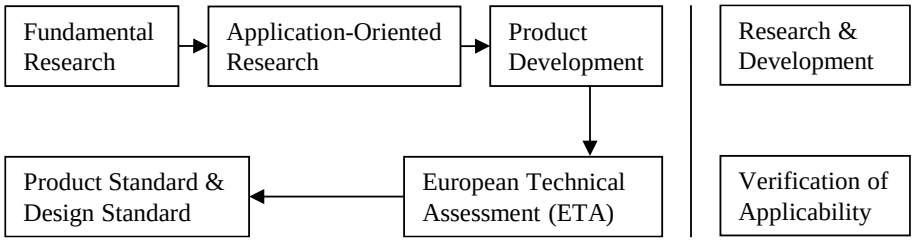


Figure 1: Development of products and the path to their applicability in practice

The development of design approaches for construction products requires standardized testing procedures and a product standard or a European Technical Assessment (ETA) to be applicable. The work on European design standards is structured as illustrated in Figure 2 and [8]. The European standardization committee CEN/TC 250 has a mandate to further develop design rules for the construction industry. This committee is divided into 11 sub-committees; CEN/TC 250/SC 5, one of these sub-committees, is responsible for all parts of Eurocode 5 (EN 1995). National standardization bodies (e.g., DIN) send delegates to these sub-committees through their mirror committees. To facilitate the technical work, Sub-Committee SC 5 is supported by 11 Working Groups (WGs); WG 1 is responsible for cross-laminated timber. National mirror committees send experts to the working groups. The working groups are responsible for their specific work programs, engaging in technical discussions that lead to proposals for standardization.

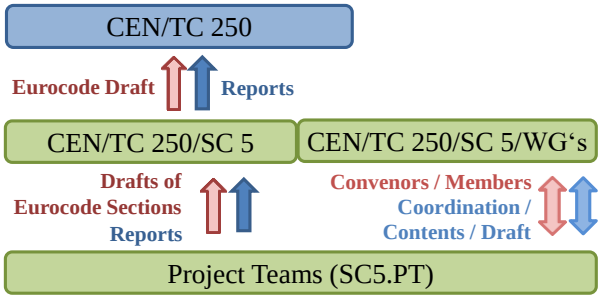


Figure 2: Workflow of standardization

The development of a draft clause, including drawings, is the responsibility of so-called Project Teams (PTs). Within a specified timeframe, the PTs must create a standard draft clause, including harmonized terminology and symbols, considering the principles of "ease-of-use" [7]. Additionally, the PTs must develop background documents that describe the technical justifications and scientific backgrounds for the proposed clauses within the standardization. During the drafting phase, the PTs' drafts go through three phases of commenting by the national standardization committees. The Project Team SC5.T1, responsible for the drafts on the design of *cross-laminated timber* and *reinforcement*, submitted its final drafts at the end of April 2018. The draft on CLT was further discussed, commented on, and adjusted in Working Group WG 1. In October 2021, the two drafts became part of prEN 1995-1-1, the consolidated version of Eurocode 5-1 for the purpose of an internal enquiry in CEN/TC 250/SC 5. After incorporating the comments received during the internal enquiry, the draft document for the official formal enquiry was prepared. The official enquiry on prEN 1995-1-1:2023 was finalized in January 2024. These comments will be discussed and incorporated in the draft version for formal vote until mid-2024. This manuscript presents the most important content and background information of prEN 1995-1-1:2023 for formal enquiry on the subject of *cross-laminated timber*, including changes during the course of standardization work. It also explains the improvements compared to previous application documents.

3. Cross-laminated timber

3.1. Fundamentals of structural design and material properties

General

Due to the orthotropic arrangement of individual layers, the load-bearing behavior of cross-laminated timber elements is characterized by designed orthotropic properties, as illustrated in Figure 3.

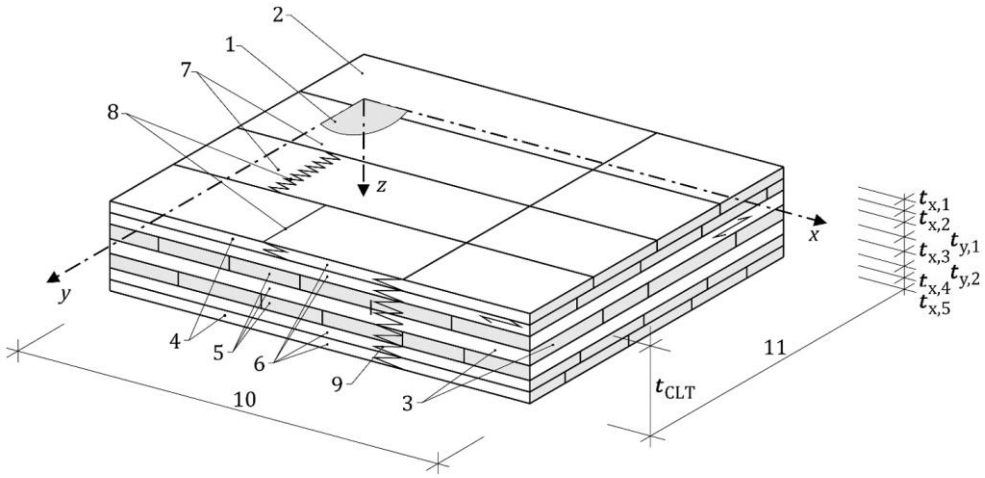


Figure 3: Cross-laminated timber; 1) plane of the member; 2) wide face; 3) narrow face; 4) outer layer; 5) inner layer; 6) parallel outer laminations; 7) lamination; 8) finger joint in lamination; 9) large finger joint; 10) usually the length; 11) usually the width b_{CLT} of a plate or height h_{CLT} of a beam.

For stress and deformation calculations, it is crucial to use correct cross-sectional and stiffness values. In determining normal stresses, only the layers oriented in the direction of stress are considered, while the cross layers are neglected in determining cross-sectional values. However, it is essential to note that these cross layers are subjected to rolling shear. Rolling shear strength is very low. Due to the simultaneously very low rolling shear modulus ($G_{r,mean} \approx 0.15\ G_{mean}$), significantly higher shear deformations occur in the cross layers compared to the longitudinal layers. As a result, the Bernoulli hypothesis of a planar cross-section is no longer fulfilled, and the distribution of bending stresses deviates from the Bernoulli beam theory, as shown in Figure 4.

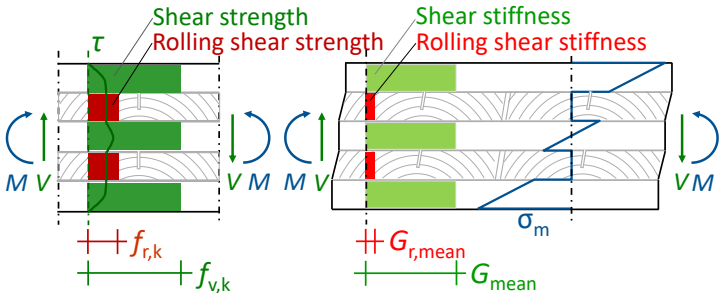


Figure 4: Cross-laminated Timber: distribution of shear and bending stresses, deformation figure, shear strength and shear stiffness subject to layer direction.

In the draft clauses on *Cross-Laminated Timber*, the material is defined as a load-bearing timber product consisting of at least three layers bonded together, always including board laminations but also allowing for wood-based panels. At least one layer is arranged perpendicular to the adjacent layers. The application is limited to service classes 1 and 2.

Partial safety factors, k_{mod}, k_{def}

Despite the specific characteristics of cross-laminated timber that have enabled a broad application of large, solid wood components, some analogies can be drawn in regards to glued laminated timber (GL). Both products typically consist of strength-graded softwood lamellas, bonded with adhesives from the same adhesive families. The minimum requirements for manufacturing, initial testing, and in-house product control are comparable, and the reference test specimens for CLT and GL (according to EN 14080 [9]) have similar dimensions and a similar number of lamellas with the grain parallel to the load direction. This level of similarities justifies [10] why the same partial safety factor is recommended for glued laminated timber and cross-laminated timber:

$$\gamma_M = 1{,}25 \tag{1}$$

The modification factor k_{mod} for cross-laminated timber is also recommended to have identical values as for solid wood and glue-laminated timber, as shown in Tab. 1.

Table 1: Values of k_{mod} for cross-laminated timber

Service class	Load-duration class				
	Permanent	Long-term	Medium term	Short-term	Instantaneous
1	0,6	0,7	0,8	0,9	1,1
2	0,6	0,7	0,8	0,9	1,1

When considering the deformation factors k_{def}, it is essential to take into account that the cross layers of cross-laminated timber are subjected to rolling shear, as illustrated in Figure 4. Significant creep deformation occurs in these layers [11]. Therefore, it is recommended to use the same deformation factors k_{def} as for plywood, as shown in Table 2. These values differ from those in the German National Annex to Eurocode 5 [4].

Table 2: Values of k_{def} for cross-laminated timber

Material	Service-class	
	1	2
Cross-Laminated Timber	0,80	1,00

Strength and stiffness properties

Cross-laminated timber shall be manufactured according to a national product standard, general technical approval or an ETA based on the EAD [13]. The specified strength and stiffness properties typically refer to the properties of the boards of the layers with grain parallel to the direction of the considered stress. The system effects known from abZ/ETAs or [3] shall not be applied; they are already integrated into the underlying strength model. Minor cross-sectional weaknesses resulting from gaps and relief grooves may be disregarded for the verification of the structural member. Table 3 presents a proposal for a strength class CL24. The values given in this table represent lower-bound values of strength and stiffness properties given in current ETAs. Specific ETAs might feature higher single values.

Numerous comprehensive studies have been realized to enable the determination of strength and stiffness properties of a CLT element from the properties of the layers. Such a model might be included in a future revision of EN 16351. Interested readers are referred to [14] as well as the information und literature references in [15].

Table 3: Characteristic strength and stiffness properties and density for cross-laminated timber with layers from solid wood.

Property		Symbol	Unit	CL24
Bending strength	Out of plane loading (slab)	$f_{m,k}$	N/mm²	24
	In plane loading (diaphragm)	$f_{m,edge,k}$	N/mm²	20,5 [a]
Tension strength	In plane loading	$f_{t,0,k}$	N/mm²	14
	Perpendicular to plane loading	$f_{t,90,k}$	N/mm²	0,12
Compressive strength	In plane loading	$f_{c,0,k}$	N/mm²	21,0
	Perpendicular to plane loading	$f_{c,90,k}$	N/mm²	2,5
Shear strength	Out of plane loading	$f_{v,k}$	N/mm²	3,5
	Rolling shear	$f_{r,k}$	N/mm²	0,7
	In plane loading	$f_{v,xy,k}$	N/mm²	5,5 [a]
		$f_{tor,node}$	N/mm²	2,5 [a]
Modulus of elasticity	In and out of plane loading	E_{mean}	N/mm²	11.000
	Perpendicular to plane loading	$E_{90,mean}$	N/mm²	370
Shear modulus	Out of plane loading	G_{mean}	N/mm²	650
	Rolling shear	$G_{r,mean}$	N/mm²	50
	In-plane loading	$G_{v,xy,mean}$	N/mm²	250
Density		ρ_k	kg/m³	385

[a] This value is usually not declared within DoP. The given value is taken from EN 16351:2021 which is not a harmonized standard. If a different value is declared by the manufacturer, that value applies. National regulations may apply.

For cross-laminated timber elements subjected to out of plane bending (perpendicular loading to the plane) of the panel (including buckling), with widths b_{CLT} smaller than the element thickness t_{CLT}, the strength properties are to be set to zero. For element widths $t_{CLT} \leq b_{CLT} \leq 600$ mm, the bending, tensile, and compressive strengths parallel to the grain should be reduced by $k_{red,b}$:

$$k_{\mathrm{red,b}} = \frac{b_{CLT}[mm]}{1200} + 0{,}5 \qquad (2)$$

For cross-laminated timber elements subjected to in plane bending, system factors (increase factors) are provided for components with more than one layer with grain orientation parallel to the associated stress (a conservative assumption in the strength model) and for component heights 150 mm $\leq h_{CLT} \leq$ 600 mm. For cross-laminated timber elements subjected to out of plane bending of the panel, no additional volume factor k_h is specified due to the relatively low number of layers and the associated brittle tensile failure of the outer layers.

The swelling and shrinkage factors of cross-laminated timber made of softwood can be assumed as followed: Perpendicular to the element plane, it is 0.24% per percentage change in wood moisture content, and in the element plane, it is 0.02% - 0.04% per percentage change in wood moisture content. The swelling and shrinking dimensions in plane depend on the layup. For typical structures, the smaller value refers to the x-direction (direction of the outer layers), and the larger value refers to the y-direction (direction of the cross layers). In some cases, the swelling and shrinking dimensions may be higher [16].

3.2. Ultimate limit state (ULS)

The draft clauses on the design of *Cross-Laminated Timber* do not provide specific information on the application of particular methods for the calculation of cross-laminated timber elements under out-of-plane loading. The choice of analytical calculation approaches (e.g., γ-method [3], shear analogy method [4], Timoshenko) is left to the structural engineer. A comparative overview of applicable methods can be found in [17] and [18].

The stress verifications are to be realized with the effective cross-sections of the layers oriented in the direction of the stresses under consideration, as shown in Figure 5, unless they are explicitly referred to the gross cross-section of the cross-laminated timber element. The designation of the direction of moment loading in Figure 5 deviates from the classical nomenclature according to plate theory. This divergence is necessary to avoid confusion between two different bending stresses (from the bending of the plate or slab), which would otherwise share the same designation.

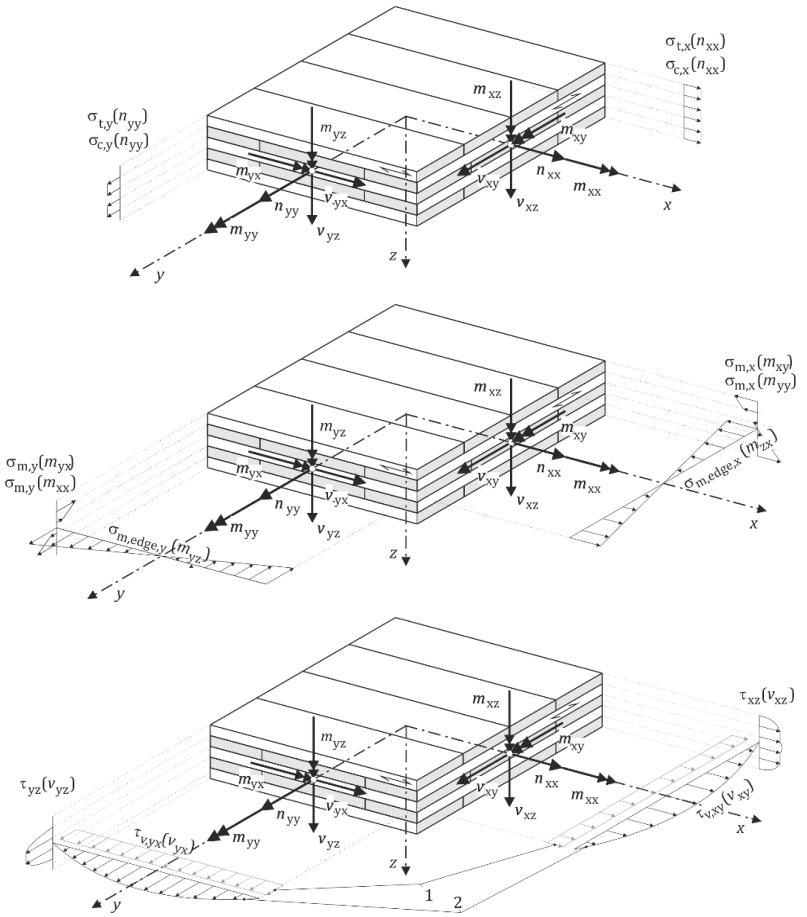


Figure 5: Designation of stresses, forces and moments for an exemplary stress distribution in a cross-laminated timber element (above: normal stresses; middle: bending stresses; below: shear stresses)

Bending

The influence of shear deformation on bending stresses may be neglected for slender single-span beams (ℓ/t_{CLT} or $\ell/h_{CLT} \geq 30$). The bending strengths provided in Table 3 already account for the combination of bending and tensile stresses in the layers, hence verification of bending stresses at the outer edges is sufficient. The need for a combined verification of tension stresses at the center of gravity of the outer laminations and the bending stresses at the outer edges of the CLT panel is omitted, except for stocky cross-laminated timber elements ($\ell/h_{CLT} < 5$) subjected to in-plane bending.

Shear

Depending on the loading conditions, up to five different failure modes in shear need to be considered for cross-laminated timber: shear and rolling shear in the case of out-of-plane loading, as illustrated in Figure 4, as well as gross shear, net shear, and torsional shear at the intersections of two laminations in the case of in-plane loading of the plate, as shown in Figure 6.

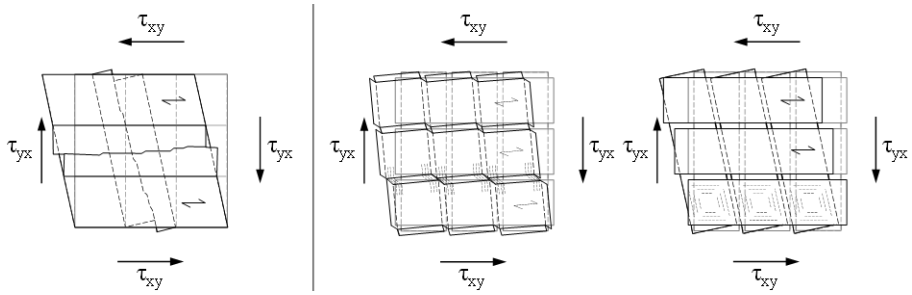


Figure 6: Failure mechanisms gross shear, net shear and torsion, from [20]

In comparison to the information in the general abZ (German national technical approval) and ETAs (European Technical Assessments), the verification methods and strength parameters considered in the *Cross-Laminated Timber Draft* have been harmonized. In cases of out-of-plane loading, the rolling shear stresses are usually governing. Rolling shear is primarily governed by the ratio between lamination width or the distance between gaps/grooves to the layer or lamination thickness, b_l/t_l, see also [19]. The crack factor can be set to k_{cr} = 1,0. When the CLT plate is subjected to in-plane loading, typically only the verification of net shear (with the associated strength $f_{v,xy,k} = f_{v,yx,k}$ = 5,5 N/mm²) is governed. When determining the decisive effective net cross-section (sum of the weaker layers), the thickness t_l of the outer layers shall be reduced by 20%, as outlined in [20]. In-plane loading on net shear of the plate also results in torsional stresses in the intersections between two laminations (and vice versa). The corresponding verification of torsional stresses in the bonded surfaces of orthogonally glued laminations is currently required only for cross-laminated timber elements where the ratio is $b_l/t_l \leq 4$ [21]. In these cases:

$$\tau_{\mathrm{tor,node,d}} = \frac{3}{2} \cdot \tau_{\mathrm{v,xy,d}} \cdot \left(\frac{t_l}{b_l}\right) \leq f_{\mathrm{tor,node,d}} \qquad (3)$$

The properties of cross-laminated timber under gross shear are not considered, as they would require a load-bearing bonding of the narrow edges of the laminations and nearly crack-free performance over the service life of the structure.

The shear design for beams made of cross-laminated timber under in-plane bending (edgewise bending) is also addressed in the draft clauses on the design of *Cross-Laminated Timber*. The design approach closely follows the information provided by Flaig [22].

Stability verification

In the verification for buckling using the equivalent length method, the imperfection factor may be set to β_c = 0.1, similar to glued laminated timber. In the stress calculation, shear deformations may be neglected, as the risk of buckling increases with higher slenderness, while the influence of shear deformations on bending stresses decreases.

3.3. Serviceability limit state (SLS)

In typical design situations, deformation verifications are usually governing for cross-laminated timber elements supported as single-span beams with spans ℓ < 4 m, while vibration verifications typically govern for spans ℓ > 4 m. The share of shear deformation w_v in the total deformation w is typically below 10 % when the span-to-thickness ratio is $\ell/t_{CLT} \geq 30$, in these cases, the additional deformation from shear may be neglected.

Within the framework of vibration verifications according to the currently applicable version of Eurocode 5 [3], verifications of natural frequency, deflection under a single load (stiffness), and acceleration are conducted. Recent developments in floor constructions such as cross-laminated timber and timber-concrete composite constructions are challenging to verify with these criteria (e.g. natural frequency $f_1 > 8$ Hz). Accordingly, the new Eurocode draft also includes *Cross-Laminated Timber* floors (typical heavier floors $g_k \geq 50$ kg/m^3) in the design rules for the vibration verification, also including recommendations of *Hamm* [23]. These suggestions were discussed and further processed in the responsible Project Team SC5.T3 and then incorporated into the consolidated version prEN 1995-1-1:2021. The approach layed out in the current draft document prEN1995-1-1:2023 is shown in Table. 4. Six floor performance levels are given (of which levels I and II are limits typically aimed at for most floor structures). The performance levels are linked to limits of the response factor and the maximum deflection under a single load. Using these elements, verification of natural frequency, minimum stiffness, acceleration and velocity are required. Table 5 provides values for the modal damping ratio of cross-laminated timber floors given in prEN1995-1-1:2023.

Table 4: Floor vibration criteria according to the floor performance level

	Floor performance levels					
Criteria	I	II	III	IV	V	VI
Response factor R	4	8	12	24	36	48
Upper deflection limit $w_{lim,max}$ in mm	0,25		0,5	1,0	1,5	2,0
Stiffness criteria for all floors	$w_{1kN} \leq w_{lim}$ mm					
Frequency criteria for all floors	$f_1 \geq 4{,}5$ Hz					
Acceleration criteria for resonant vibration design situations ($f_1 < f_{1,lim}$)	$a_{rms} \leq 0{,}005\ R$ m/s^2					
Velocity criteria for all floors	$v_{rms} \leq 0{,}0001\ R$ m/s					

Table 5: Values for the modal damping ratio ζ *for cross-laminated timber* floors

Floor lay-up	Modal damping ratio ζ
CLT-floor with floating floor layer	0,04
CLT-floor	0,025

3.4. Members

Wall diaphragms

For cross-laminated timber elements used as wall diaphragms, construction rules are provided, which are given in the following in excerpts. A structural wall diaphragm should have a height-to-width ratio of $h/\ell \leq 4$. Wall diaphragms can be realized as monolithic or segmented wall diaphragms. The connections between parts of a segmented wall can be designed for a constant shear flow. Stresses from horizontal loads in window or door lintels should be considered, if the lifting of adjacent walls is not prevented. When calculating stresses and deformation, the deformation contributions of the connections should be considered. In contrast to the information in general abZ / ETAs, the shear stiffness of a wall diaphragm made of CLT may be determined using the shear modulus for in-plane shear, $G_{xy,mean} = G_{yx,mean}$ and assuming the gross cross-section.

Concentrated loads in plane

When joists or purlins are supported by CLT walls, high loads are locally transferred into the structural element, at the supporting area. Additionally, compression stresses in the contact area, the stability of the member and, if necessary, compression stresses in the

member underneath the CLT wall should be verified. Information on the internal load transfer is provided for this purpose, by the new Eurocode draft. For cross-laminated timber elements within the following limits, this information is significantly simplified:

- The ratio of the sum of the layer thicknesses in the direction of the load Σt_x to the total thickness t_{CLT} is $\Sigma t_x / t_{CLT} \geq 0{,}75$
- The ratio of the width of the load introduction to the member height h is $b/h \leq 0{,}15$

In these cases, the effective width at half the height of the element $b_{ef}(h/2)$ can simply be calculated assuming a load dispersion angle of 15°. This load dispersion shall only be considered over half of the member height h. In the case of different lengths of load introduction, b_{ef} shall be calculated based on the smaller length.

Compression perpendicular to grain

For cross-laminated timber elements with a ratio between maximum and minimum layer thickness $t_{l,max}/t_{l,min} \leq 2$, the stress spreading factor $k_{c,90,CLT}$ can be calculated as follows:

- $$k_{c,90,CLT} = \sqrt{\frac{b_{ef} \cdot l_{ef}}{b \cdot l}} \leq 4 \qquad (4)$$

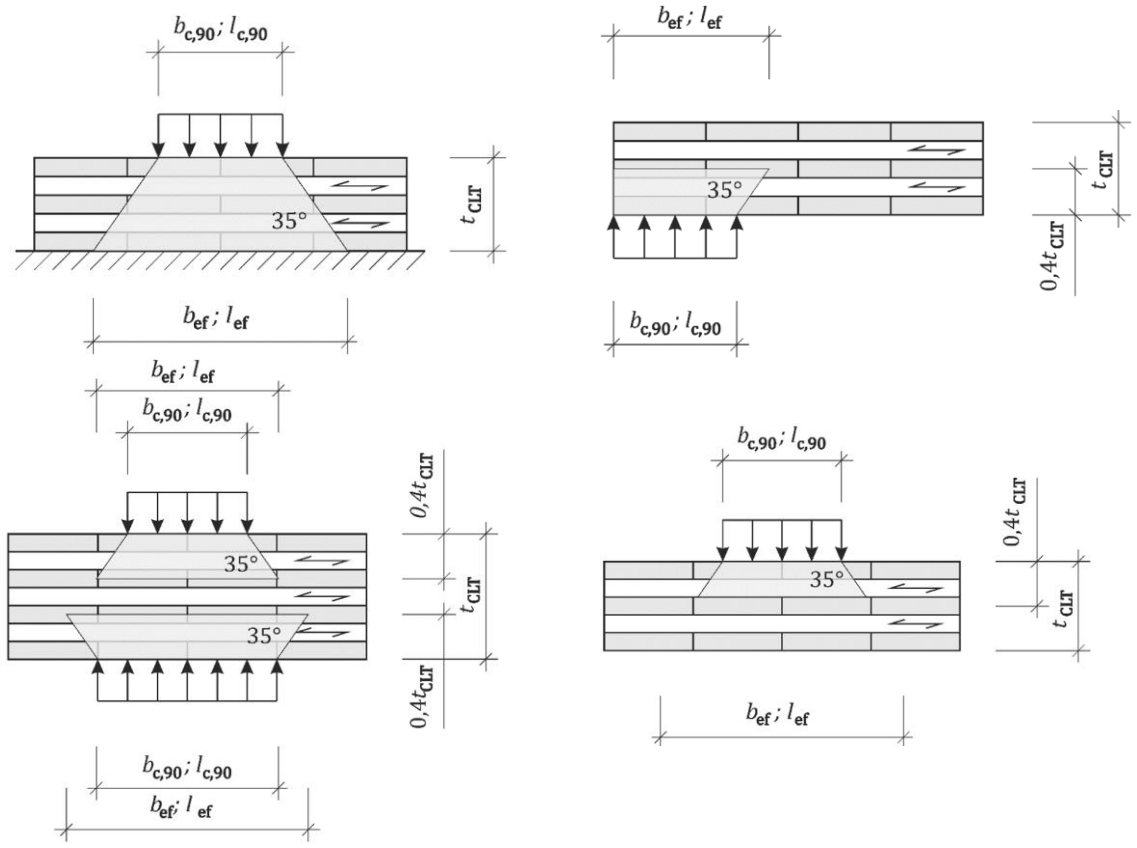


Figure 7: Force distribution width and stress spreading length in CLT subjected to compression perpendicular to the plane

The distribution width b_{ef} or length ℓ_{ef} is based on the assumption of an average stress distribution angle of 35° (longitudinal layers: 45°, transverse layers: 15°), as shown in Figure 7. In many ETAs of CLT products, the compressive strength perpendicular to the grain of CLT made of solid wood laminations is $f_{c,90,k}$ = 3,0 N/mm^2. This is higher compared to glued laminated timber. The same applies to the elastic modulus perpendicular to the grain. In both cases, this is due to the restraining effect of the cross layers [24].

Concentrated loads perpendicular to plane

For the "punching shear verification" under concentrated loads perpendicular to the plane of cross-laminated timber panels, the governing factor is usually the rolling shear strength. This situation usually applies for point supported CLT floor elements. Due to a positively acting stress interaction from compression perpendicular to the grain and rolling shear, as well as nonlinear effects in the area of load concentration [25], for cross-laminated timber consisting exclusively of solid wood layers, a 60% higher rolling shear strength can be assumed [26]. Therefore, a verification of rolling shear stresses, applying an increase factor $k_{r,pu}$, has to be conducted. The rolling shear stresses in the area of concentrated loads may be determined by assuming an effective width. For this purpose, a perimeter under an angle of 35° up to the decisive location can be assumed, as shown in Figure 8 and [25].

The verification is then to be carried out as follows:

$$\tau_{r,d} \leq k_{r,pu} \cdot f_{r,d} \tag{5}$$

with: $k_{r,pu}$ = 1,60 for CLT exclusively made of timber layers

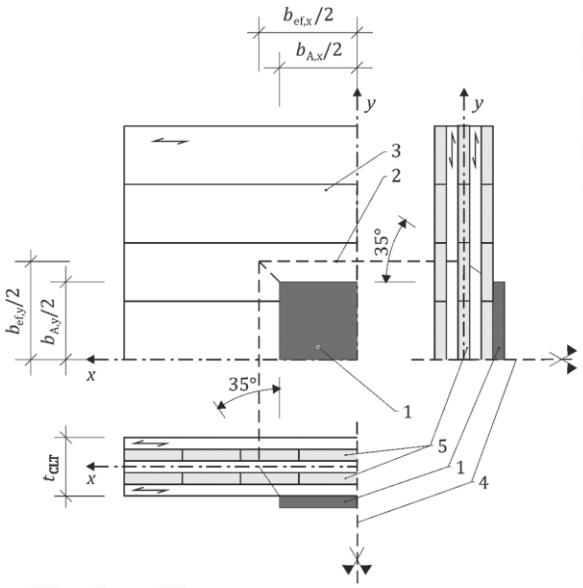


Figure 8: CLT under concentrated load (1 – area of load introduction; 2 – perimeter for stress verification; 3 – CLT; 4 – member edge or axis of symmetry; 5 – decisive layer)

Reinforcement to cover high support loads or high point loads are also addressed in the draft clauses on the design of *Cross-Laminated Timber*. The design approach closely follows the information provided by *Mestek* [25].

Ribbed plates

The draft for Cross-Laminated Timber (CLT) also includes information on the design of ribbed panels made from CLT panels, bonded with, for example, a glued laminated timber cross-section.

4. Acknowledgements

The number of authors is a result of the collaborative effort involved in creating the draft *Cross-Laminated Timber* (and draft *Reinforcement* not presented in this manuscript). Philipp Dietsch, Gerhard Schickhofer, Alfons Brunauer†, Roberto Tomasi, and Tobias Wiegand (lead) contributed as members of the Project Team (PT). Additionally, Ulrich Hübner, Harald Krenn, Peter Mestek, and Thomas Moosbrugger played leading roles as members of CEN/TC 250/SC 5/WG 1 in the development of both drafts.

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6. Appendix

G_{mean} *mean shear modulus;*

$G_{r,mean}$ *mean rolling shear modulus;*

b_{ef} *effective width;*

$f_{r,d}$ *design rolling shear strength*

$f_{tor,node,k}$ *characteristic torsional shear strength of the glued area of crosswise bonded lamellae;*

$k_{r,pu}$ *factor to account for the non-linear behaviour and the strength combination;*

$k_{red,b}$ *factor for reduced width of CLT members;*

$k_{c,90,CLT}$ *stress spreading factor accounting for the distribution of compressive stresses in the member and the layup of the CLT element*

ℓ_{ef} *spreading length;*

$t_{l,min}$ *minimum lamination thickness within a layup;*

$t_{l,max}$ *maximum lamination thickness within a layup;*

t_x *sum of thicknesses of layers in x-direction;*

β_c *imperfection factor for flexural buckling about y- or z-axis;*

ζ *modal damping ratio;*

$\tau_{r,d}$ *design rolling shear stress;*

$\tau_{tor,node,d}$ *design torsional shear stress of glued area of crosswise bonded lamellae.*

Standardization and fire

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Standardization and fire

Abstract

The revision work of the new EN 1995-1-2 was performed by a Project Team (PT), which consists of Andrea Frangi (chair, ETH), Alar Just (TalTech), Norman Werther (TU Munich), Jouni Hakkarainen (Eurofins), Joachim Schmid (ETH), and additional members from 2021 – Harald Krenn (KLH), Renaud Blondeau (Stora Enso) and Gordian Stapf (Henkel).
Basis for the drafting work are extensive documents, reports and publications with the update state-of-the-art with regard to the structural fire behaviour and fire design of timber structures, e.g. the European Technical Guideline «Fire safety in timber buildings» or the reports prepared in the frame of the recently concluded COST Action FP1404. The PT started his work on June 2018 and regularly reports to the Working Group WG4 of CEN/TC250/SC5, which is responsible for the revision of EN 1995-1-2. During the last 3 years, the PT prepared three drafts, which were reviewed by the WG4 and commented by the national standardisation bodies. The first draft (May 2019, 75 pages) received 265 comments, the second draft (Mai 2020, 134 pages) 624 comments and the third draft (November 2020, 138 pages) 364 comments. The final draft of EN 1995-1-2 was submitted at the end of August 2021 and received 396 comments. Formal Enquiry of the document was held from September 2023 to January 2024 and received 856 comments.
The following list gives some highlights of the new EN 1995-1-2.

1. Structure of the new EN 1995-1-2

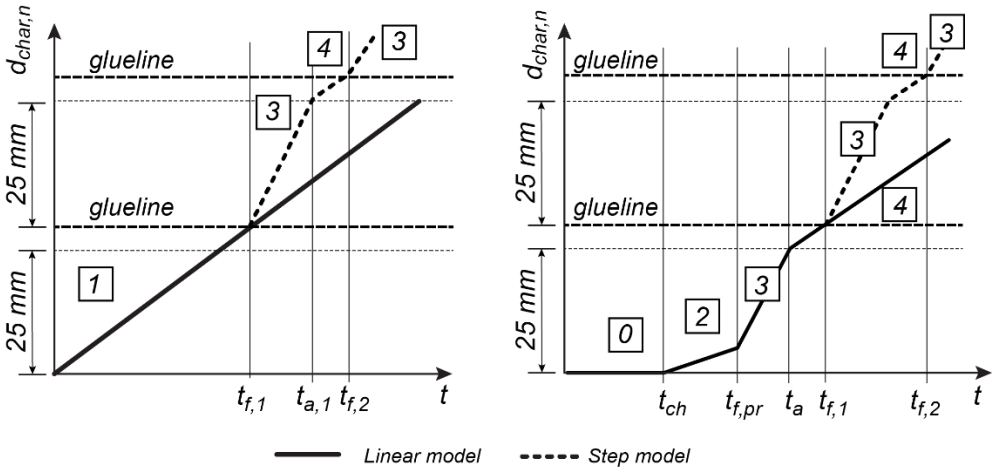
Structure of the new EN 1995-1-2 follows the harmonised logic of all fire parts of the Eurocodes. The structure and also the first four chapters for all fire parts of the Eurocodes are written by TC250 «Horizontal Group Fire». This has resulted in harmonized structure, which contributes to user-friendliness. The principle of a three-stage possibility of verification levels with different levels of complexity and accuracy, which is already known from other Eurocodes, is now also fully established for timber construction. Thus, in the future there are

- tabulated design data (Chapter 6),
- simplified design methods (Chapter 7) and
- advanced design methods (Chapter 8)

In addition to the principles for simplified design models already known in the current EN 1995-1-2 and the basics for numerical simulation models, Chapter 6 in the new EN 1995-1-2 lists for the first time proven construction structures or predefined characteristic values for some design parameters. In this way the user is given a very simple and efficient way of verifying fire resistance. Despite the increased scope of regulations and the expansion of the scope of application, the structure adapted in this way will enable simple application.

2. European Charring Model

Charring has extensively been dealt and the current model (in the future renamed as the European Charring Model) has been generalised considering the different phases of protection and the different modification factors for charring in a more systematic way. Further, the charring model clearly distinguishes two cases (bond line integrity maintained or not maintained during the fire exposure). See Figure 1. Supplementary guidance for the assessment of the bond line integrity in fire has been included in the new Annex B of this standard.



(a) Initially unprotected sides of timber members *(b) Initially protected sides of timber members*

Key:

0	Encapsulated phase (Phase 0)
1	Normal charring phase (Phase 1)
2	Protected charring phase (Phase 2)
3	Post-protected charring phase (Phase 3)
4	Consolidated charring phase (Phase 4)

$d_{char,n}$ - notional charring depth
t - time
t_{ch} - start time of charring
$t_{f,pr}$ - failure time of the fire protection system
t_a - consolidation time

Figure 1: Phases for timber members according to the European charring model.

To evaluate the performance characteristic of the bonding in case of fire, two test methods are introduced in Annex B. For face bonds, the charring rate of a tested CLT or glulam specimen is compared to a solid wood reference in 120 minutes standard fire test in a horizontal position. The bond line integrity is assessed as maintained if the difference in charring rate of test specimen do not exceed 5% of the charring rate of solid wood reference.

For the assessment of finger-joints in the flanges of I-joists in case of fire, a test method was introduced in Annex B with loaded small test specimens that are exposed to constant heat flux. Based on the test results, the finger joints are divided into three performance classes, which are taken into account in the design for the fire case.

For the European Charring Model, time limits between phases for initially protected timber members are defined by start time of charring t_{ch}, failure time (or fall-off time) of the fire protection system $t_{f,pr}$ and consolidation time t_a. For structures with bond line integrity not maintained, the fall-off time of charred layers $t_{f,i}$ is considered.

Start time of charring t_{ch} will be designed according to Separating Function Method.

Fall-off time $t_{f,pr}$ is one of the most important parameters influencing the fire resistance of initially protected timber structures, especially concerning timber members with small cross-section. In the new Eurocode the failure times (defined as fall-off times) of different panels, including gypsum plasterboard Type A and F and gypsum fibreboards are given with simplified equations based on a large data base of fire tests. Further, for the fire design it is possible to use failure times based on full-scale fire tests performed according to EN 13381-7.

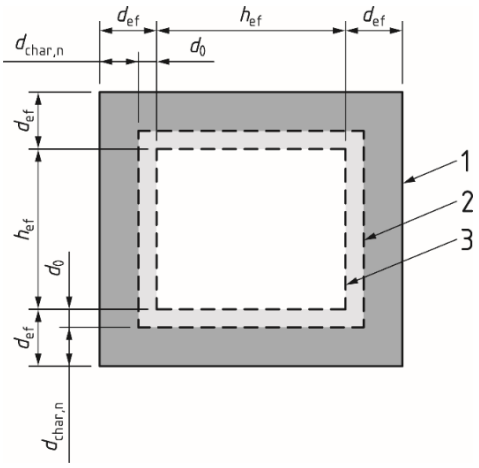
Notional design charring rates in different phases and for different types of members will be calculated using applicable modification factors.

In the EN 1995-1-2:2025 the basic design charring rates β_0 are given for European softwood (Table B.2 in EN 14081-1), some hardwood species (beech, oak, ash) and also panels made of LVL, OSB, fibre boards, particle boards and plywood. Compared to EN 1995-1-2:2004 the basic design charring rates of some wood based board products are decreased.

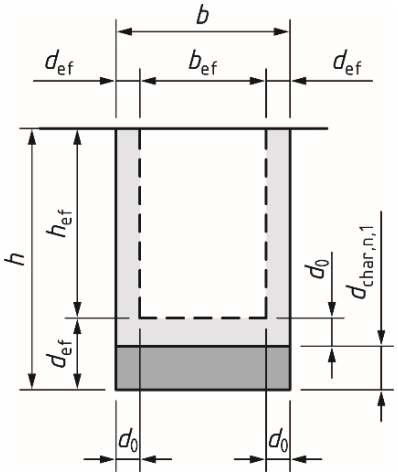
The basic design charring rates for engineered wood (glulam, CLT) should be chosen according to the wood species that lamellae are made of.
The basic design charring rates for species and wood-based boards not listed in the EN 1995-1-2 should be determined using the assessment method at Annex C.

3. Effective Cross-Section Method

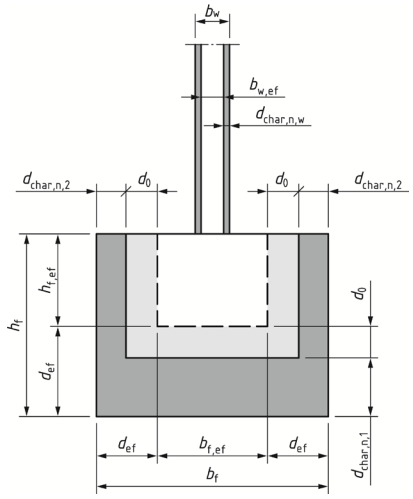
The Effective Cross-section Method has extensively been revised and its use extended to all common structural timber members, including cross-laminated timber panel (CLT), timber frame assemblies and timber-concrete-composite elements (TCC).



a) Linear member exposed from 4 sides



b) Member of a timber frame assembly exposed from 1 side



c) I-shaped member of timber frame assembly

Key

1 fire exposed side(s) or fire exposed perimeter
2 border-line of the residual cross-section
3 border-line of the effective cross-section

d_{ef} - effective charring depth

$d_{char,n}$ - notional charring depth

d_0 - zero-strength layer depth

b_{ef} - width of the effective cross-section'

h_{ef} - depth of the effective cross-section

Figure 2: Examples of the effective cross-sections.

Many fire test results from the last decades show that zero strength layer depth 7 mm given in the current Eurocode 5 leads often to unconservative results in the design. In the 2nd generation of Eurocode 5 the zero-strength layer depths are increased to 10 mm for bending and tension members and 14 mm for compression members.

The design model for timber frame assemblies with filled cavities is based on the Effective Cross-section Method and allows considering the performance of different types of insulation (mineral wool, cellulose, wood fibre, etc.). The performance of the insulation can be evaluated with small-scale fire tests and assessed in 3 different protection level according to the new Annex D.

Calculation of the effective cross-section is given for cross-laminated timber, taking into account the load-bearing and non-load bearing layers.

4. Separating Function Method

The current Annex E (Component additive method) for the verification of the separating function has extensively been improved and moved to the main part of the EN 1995-1-2. The revised content is normative and the design method for separating function has been extended to 120 minutes fire resistance.

Heat path through the timber members or other materials as panels and insulations maybe calculated by taking the contribution of each layer into account starting from the fire exposed side. Layers can be combined and changed easily. Total insulation time of an assembly is the sum of contributions of all layers. Generic values for protection times are given for wood-based products, gypsum boards, clay and lime plasters, mineral wool and cellulose based insulation materials and as well as cement based screed. The method is open to include new materials or products using the procedure in the new Annex G.

5. Connections

Calculation method for connections is extended and enhanced. Improved rules for the fire design of connections up to 120 minutes fire resistance are given based on extensive experimental and numerical analysis. Further, tabulated design data have been included allowing a simple fire design of connections.

Detailing rules shall be satisfied when simplified design methods or tabulated design parameters are used.

The rules for detailing were extended and enhanced compared to current version of EN 1995-1-2. In addition to rules for panels and insulation (dimensions, spacings, fixation, etc), rules for joints in and between elements and to other adjacent components as well as rules for penetrations and openings are given.

6. Design for physically based fires

For the design of timber structures exposed to physically based design fires improved rules and design methods have been developed and given in the new Annex A.

Design methods for any temperature-time curve including the parametric fires are given. According to the new Eurocode timber structures can be designed until burnout including the contribution of structural fire load to the total fire load. The calculation of structural fire load is performed according to Annex H of a new EN 1991-1-2.

The standardized methods for physical based fires create new possibilities for design of fire resistance of large and tall timber buildings for a full fire scenario according to Eurocode 5.

7. Summary

It is expected that the second generation of EN 1995-1-2 will fill most gaps of the current EN 1995-1-2 and will allow a safe and economic design of timber structures in fire.

The scope of the standard has been growing due to the necessary consideration of new products made of wood, the demand for more accurate design and the increase of available design approaches. Despite this, a central focus is on maintaining and even increasing the ease-of use through restructuring, homogenisation and simplified methods. Nevertheless, similar to the changeover to the first generation of EN 1995-1-2, an additional learning and training process will be necessary, which should start before the final publication.

Block 5A
Digitalization and the construction industry

Revealing hidden simplicity in complex builds

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Revealing hidden simplicity in complex builds

Abstract

We need to change how we model geometry. Why? Our conventional digital modelling approach emulates analogue construction methods and does not exploit digital super-powers. This is a theoretical teaser for a much more visual and fun talk I'll be having at the conference. Keywords: Abstractions, relations and finding repetitions.

1. The tricks

We're still in a state where we have fancier production planning and manufacturing tools than what we're able to exploit. We have robots, we have AI, we have BIM and CNC. Still, we are doing a lot of stuff manually. Either analog manually or emulating analogue operations digitally. Why is that? Why hasn't the building industry come further?

Complexity. Or seemingly complexity. The ever challenge with the building industry is that we tend to make one-off buildings – or at least we like to think that we make one-off buildings. Even house-manufacturers building hundreds of houses a year are struggling with getting a satisfying efficiency in production planning and manufacturing. But just to be clear: I view a conventional stud frame wall to be a very complex geometry – it doesn't have to doubly curved and made by starchitects!

But does it have to be that difficult? Don't we all have a gut feeling that planning and building processes can become more efficient? I will not claim that I have the answer in this talk, but through working with relatively complex projects and a small budget – and being involved in both design, detailing, manufacturing and building, I believe I've found some hidden tricks on how to build stuff more efficiently.

Abstractions, Relations and Finding repetitions. Three connected concepts that can be hidden in most types of buildings and even products. After explaining these, I will exemplify how these principles are used in real building projects.

2. Abstractions

A wall. The classic Norwegian wall. Cladding, battens, wind-barrier, insulated structural stud frame, vapor barrier, another layer of insulation and then interior cladding. Then windows, flashings, shafts, and other stuff. Only in one single wall, one can have 10-20 products and hundreds of components. Conventionally in digital planning, all these components are grouped into layers, then modelled as simple boxes. Alternatively, a wall is just modeled a simple box. So, in a way, abstractions are very common. However, If the wall is modelled as a box, it's impossible for a robot to put on the cladding boards in correct positions and order. But if a wall is modelled as components, then getting a wall area can become a complex operation. The answer is that we need both. But that's not the trick. The simple trick is to model abstractions in the right order (preferably algorithmically, but not necessarily).
A trivial but relevant example: It's tedious to model the centerline from a complex profile cladding board. But it is very easy to model a complex cladding board from a simple centerline. If putting a wall creation into a chain from very abstracted geometry to detailed components, it can look like this:

Wall bottom/top curve+ openings + metadata ***GENERATES*** a wall surface + metadata
Wall surface + metadata ***GENERATES*** wall layers as surfaces + metadata
Wall layer surface + metadata ***GENERATES*** component's centerlines + metadata
Component centerline + metadata ***GENERATES*** Component cross section + metadata
Component cross section + metadata ***GENERATES*** Solid building component

The result is a multi-scalar model, that essentially can be regenerated to a fully detailed model by adjusting the bottom/top curve or any of the intermediate steps. Also, by having all these abstractions, one can also generate other types of outputs. For a robot, it's not enough to have the cladding board. You also need (among other information) the gripper position. By then having the centerline of the cladding and the orientation and thickness stored in metadata, one can easily generate a gripper plane.

Points, planes, curves and surfaces does not exist in the real world, but abstracting geometry into these simple concepts, can help a lot. Such abstractions turn two very different walls into the same logic, then drastically reducing complexity and possibly also modelling time.

3. Relations

Relations are the most underutilized weapon in digital planning. As a BIM-modeler, you know that a wall belongs to the living room, but you also know that the same wall belongs to the bedroom. If modelled well, the BIM-model itself is also aware of this. But this is as far as it goes. A Column in a building does not have a clue if it is holding up a living room or is just hanging around in space. A cladding might know that it is in a wall and a certain wall layer but will have big problems telling you that the board is southwest facing, is outside a living room, is placed on the second floor and is partly trimmed by window X. But does this matter? Yes.

Relations to other components are key to determining the properties of any component in a building. Still, these relations are seldom well modelled – the relations are just human tacit knowledge but can evidently not be utilized by a computer.

Geometric relationships can be simplified into 3 categories. Child-parent relations, sibling relations and abstraction types. Going back to the wall, a wall layer is a child of the wall itself, and a stud is a child of a wall layer. Further, one can say that two studs are siblings, cause they are in the same layer and have the same parent. Abstraction types relate to the multi-scalar modelling approach. A stud can be expressed both as a simple line, a simplified volume or a complex mesh. They all represent the same component, but they have different abstractions.

All relationships can then be described as a graph network. And by this graph, you get superpowers!

4. Finding repetitions

By finding repetitions and similarities, one can speed up both planning and production. In complex projects, these repetitions can be hard to find. However, by combining the above tricks about using abstractions and modelling relations, many similarities appear!

Me and you look probably different. We have different hair color, different nose size, different height and different shape of our hands. But! Me and you and the rest of the human species, share the same logical skeletal. The skeletal is a model of a human being and consists of abstracted components (bones) and joints (relations). By this, one can say that all human beings are the same. In fact, many mammals have skeletons quite like our skeletons.

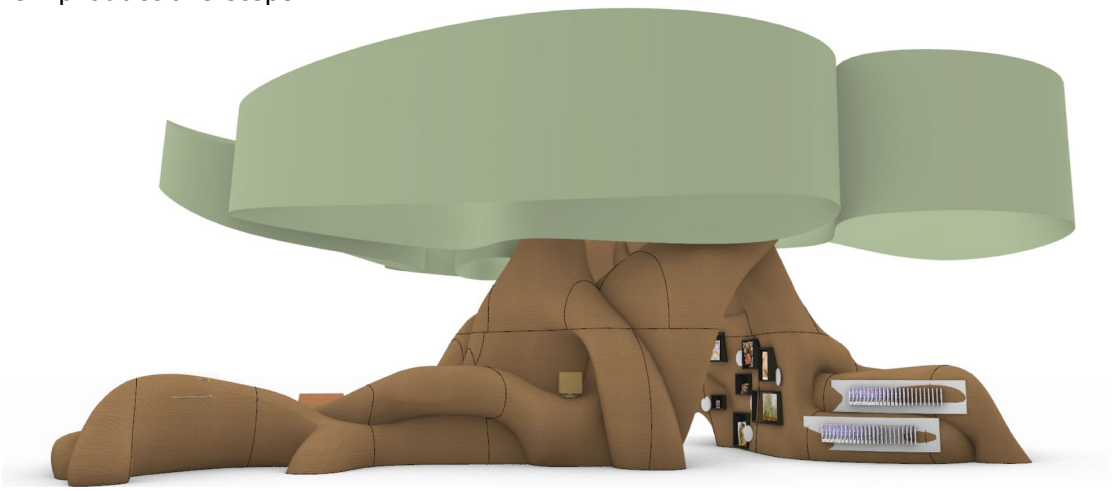
But then you think. Well, bears are walking on four legs. But topologically they are not different than ours- we can also stand on four legs.

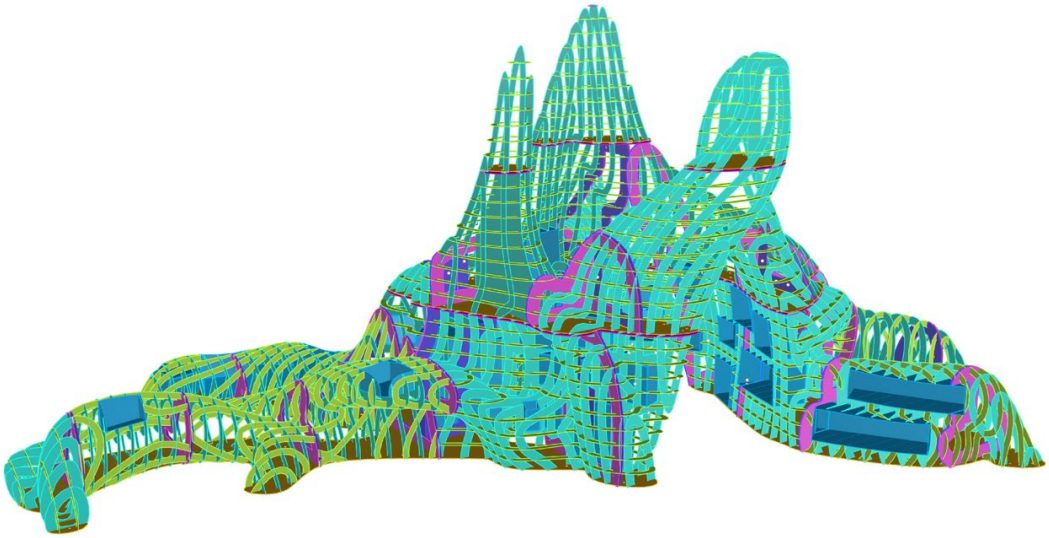
The same goes for any buildings. If we are able to describe buildings as relations and abstracted geometry with attached building components, we can drastically speed up any planning and building processes, cause then suddenly there are many logical similarities.

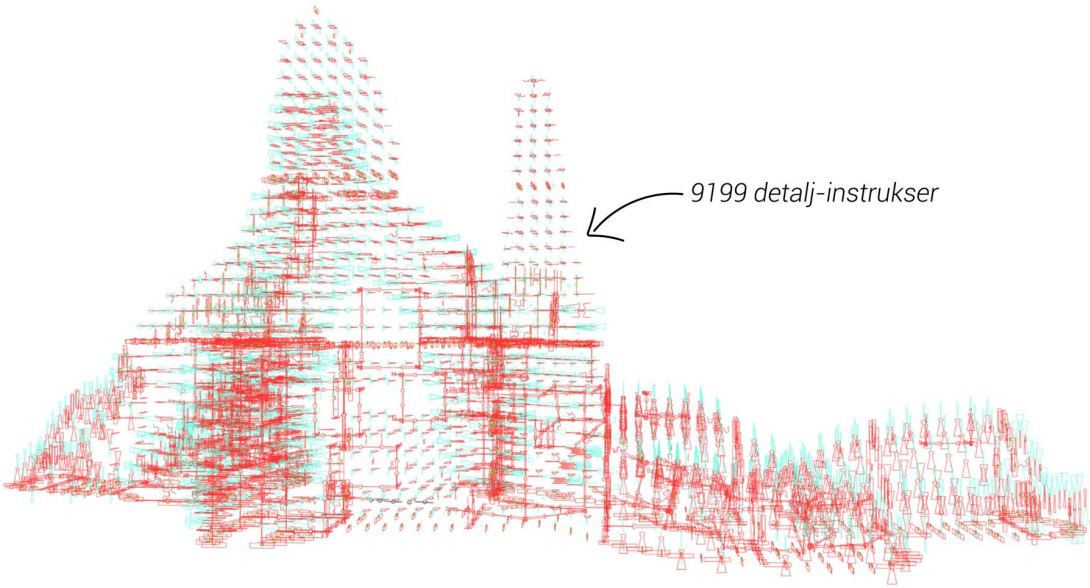
For the wall example, then shape does not matter. How many windows or the thickness of the wall does not matter. All walls can be described by a surface shape and metadata. All walls can be described as a list of wall layers (a list can have one member), and all walls can be described as building components having sibling and child-parent relation.

5. Examples

In the talk I will show different examples that are built based on the principles of abstractions, relations and finding relations. Below are pictures of a tree – or a very big modular sofa. Looks complex, but boils down to repetitive geometry with repetitive relations and a few productions steps.







Block 5B
Architecture and health

Healthy energy-efficient urban home ventilation

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Healthy energy-efficient urban home ventilation

Abstract

More wooden buildings are now being built in Nordic cities than before. Wood is a hygroscopic material, and it can have a positive impact on the indoor climate in the building. Moisture is absorbed by the wood and released into the indoor air depending on the relative humidity in the indoor environment. The relative humidity in the indoor environment of a wooden building will thus be higher and more stable than in houses made of non-hygroscopic material. This will have a positive effect on health problems and illness in residents. The use of wood in the indoor environment, together with a ventilation system, will be able to reduce energy consumption in the wooden building. A 5-30% reduction has been calculated compared to similar buildings made of non-hygroscopic material.

1. Hygroscopic properties in wood house in Nordic countries

Urban homes in Norway, Sweden, and Finland are made of different building materials, such as timber, wood, concrete, and brick. Central parts of major cities have traditionally more houses made of concrete and brick than of wood. This is in change, larger sustainable areas with cross-laminated wood (CL-wood) are built or will be built in Nordic cities. Examples of this are *The Modern Wooden Town Project (1999 – 2010) in Finland* (Karjalainen, M. and Koiso-Kanttila, J. (2005) and Stockholm Wood City. In Stockholm, construction will start in 2025, and 7000 new offices and 2000 homes will be built.



Figure 1: Wood city, Helsinki (Photo Credit: Sami Kero / HS)

In this article, the focus will be on hygroscopic properties in the wood, energy efficiency, ventilation, indoor air pollution and health effects as well as possible actions to facilitate a good indoor climate in terms of health.

1.1. Hygroscopic properties in the wood

Wood is a hygroscopic material (Glass and Zelinka, 2010). In a study by Nore et al**,** 2017, the hygroscopic properties of wood were explored in two timber tests houses. They revealed that a substantial effect on wood surface temperature were associated with indoor relative humidity. In addition, the results were in good agreement with hygrothermal numerical simulations. Further simulation used two bathroom models, one covered with wood and the other covered with tiles. The buffering effect from the wood surfaces keep the RH within a close interval compared to the simulation with the tiles.

In a study (Skulberg et al, 2022) at Hanstad school in Inland, Norway two classrooms built with CL-wood were used, one with untreated walls and ceiling while the other with walls and ceiling threated with transparent varnish.



Figure 2a and 2b: Exterior and interior picture from Hanstad school, Elverum, Norway

During 4 weeks in autumn 2021 measurements of moisture in the wooden walls and relative humidity in the indoor environment were measured. A strong positive association between moisture in the wood and RH in the room with untreated wooden walls was found. In the room with varnish a weak positive association was found. These associations were reduced by the ventilation. In addition, the wood moisture in the untreated classroom was slightly reduced during the study period. probably because of cold outdoor temperature in the last part of the study.

1.2. Energy-efficient urban ventilation

Ventilation is an important part of any building. Ventilation may be achieved by opening windows or by ventilation systems (Wargocki, 2021). It brings fresh air in and remove polluted air from inside to outside. It will then provide a good indoor air quality. The ventilation is usually controlled by the amount of CO_2 in the home. The ventilation systems can be mechanical or balanced systems that are controlled automatically.

Increased surface temperature of the wood will follow after moisture absorption in the wood. Thus, energy savings may happen by regulating the temperature in a HVAC system into lower levels, but levels will be experienced as good indoor temperatures (Nore et al, 2017). Zhang et al (2017) found that the potential energy saving rate in buildings with hygroscopic materials could be up to 25–30% in temperate climates and semi-arid climates. Another study has found a 5-30 % reduction of energy consumption for heating and cooling by using hygroscopic materials (Osanyintola & Simonson, 2006).

2. Pollutants in the indoor environment of urban homes and associated health effects

Pollutants in the indoor environment in urban housing can be of different types and negative health effects will depend on the concentration of the pollution in the indoor environment. Examples of these pollutants are particles from traffic, wood burning indoors or in neighbourhoods, other particles, and gases (chemical compounds).

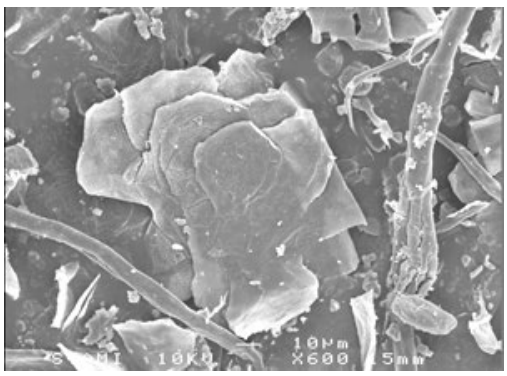


Figure 3: Particles found in indoor air (STAMIs partikkelatlas)

Figure 3 shows different particles, scales, textile fibres and combustion particles from an indoor environment. Chemical gases in the indoor environment may be from different sources: evaporation from painting or from wood walls, cleaning products, deodorants, gases or particles during cooking and frying. In addition, high concentrations of water vapour in the home, due to frequent and prolonged showering or drying of clothes inside a small apartment. In relation to the particle pollution in urban areas, it has been shown that concentrations of particles in the indoor environment will be approximately 50% of the outdoor concentration. In large cities, this exposure can therefore have a negative impact on health.

Negative health effects associated with pollutants in the indoor environment are irritation from the eyes, nose and throat, dry or irritated skin and general ailments such as headache, fatigue and reduced concentration (Skyberg et al, 2003, Eriksson and Stenberg, 2006). In a longitudinal study by Sahlberg et al, 2012, 452 persons were followed. Indoors environment in their dwellings and symptoms were examined. Following environmental factors were associated with mucosal irritation symptoms: Dampness or indoor moulds, indoor painting, and biomarkers of allergy and inflammation.

Scientific papers have shown that the combustion particles can lead to inflammatory reactions in cells (Schwarze et al, 2013). Recent research has shown that exposure to ambient air pollution on humans is associated with various respiratory diseases such as asthma, chronic obstructive pulmonary disease and acute respiratory infections (Li et al, 2020).

In a laboratory study by Skulberg et al, 2019, the emissions of volatile organic compounds from fresh pine (intervention) and old spruce (control) wood were measured during exposure in a laboratory chamber. The gases methanol, acetaldehyde, formic acid, acetone/propanal, acetic acid, monoterpenes and TVOC were measured. Mean exposure of monoterpenes were 172 ppb during intervention exposure and 1 ppb during control exposure. The corresponding exposure of TVOCs were 288 ppb and 35 ppb. Human participants were examined before and after exposure. Mucosal irritation effects, air flow measurements, neuropsychological tests were conducted. No negative health effects were revealed.

3. Efforts that may influence on health in urban wooden homes

There are factors, other than the ventilation system, which may influence on the indoor environment and health.

Many persons in homes experience a dry indoor air during the winter season. Experience shows that use of humidifier in indoor environment will add more moisture to the home. However, after long-term use and poor cleaning of the humidifier growth of bacteria in the water will be experienced. Therefore, it is not recommended to use humidification with a humidifier in an indoor climate (Lee JH, Ahn KH, Yu IJ, 2012)

Indoor moisture problems can occur in homes with many residents and small area for the living space. Other reasons for moisture problems can be frequent drying of laundry in small apartments or frequent and long showers in apartments with poor ventilation or no ventilation at all.

Other efforts that can improve the indoor climate are a good filter in the air intake, good cleaning routines in the home (Skulberg et al, 2004), an efficient kitchen fan (Jutulstad et al, 2022) and the use of air purifiers in the indoor environment (Skulberg et al, 2005).

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Healing architecture with a children's perspective – Queen Silvia Children's Hospital in Gothenburg, Sweden

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Healing architecture with a children's perspective – Queen Silvia Chidren's hospital in Gothenburg, Sweden

Abstract

Care for the patient, family and relatives is the guidning principle in the design of Queen Silvia Children's hospital. A combination of a reassuring playfulness and medical professionalism is combined for optimal care and recovery, always with the child in focus. Research on the impact of healthcare environments and nature on recovery has emphasized natural materials, such as wood, and greenery (Ulrich 1984, Centrum för vårdens arkitektur 2021). A colorful building that both destigmatizes children's encounters with healthcare and conveys strength, well-being, and security for children, their families, and staff.

1. Introduction

The new building for Queen Silvia Children's Hospital has been the first step in the development of the hospital campus Sahlgrenska University Hospital «Östra Hospital» in Gothenburg. The hospital area, which was built based on the ideals of the 60's and 70's as a spacious and freestanding buildings in a park concept, is evolving into a more urban setting with streets, squares, and avenues. The new children's hospital is standing adjacent to the existing. The buildings are interconnected through a common main entrance, connecting corridors, and service tunnels. One significant challenge has been to combine high-tech functions with high demands on equipment, hygiene, and efficiency, while also meeting the softer environmental requirements posed by children and their families. The average length of stay is relatively short, 2½ days, but several children spend a long time at the hospital, sometimes up to six months or more.

2. The building

The new children's hospital is located in the southwestern part of the hospital area. This provides an open location with views and connection to walking paths, nature, and play areas for parents and siblings. The main entrance is situated in the north, adjacent to an entrance square. Surrounding the building are areas designated for various activities. To the east, there is a play garden for the play therapy department, to the south, there are training and rehabilitation spaces for physiotherapy and occupational therapy. To the west, there is an outdoor space for the staff. The children's hospital is designed as a U-shaped building surrounding a glass-covered courtyard. The lower floors have a wider building depth combined with high ceilings to accommodate technology-intensive operations and rehabilitation facilities. The entrance level houses Play Therapy, Physiotherapy, and Occupational Therapy, while floor 1 has the Surgery and Intensive Care Unit (ICU). The upper floors are designed for patient wards with slightly lower ceiling heights and narrower building depths to provide sunlight and good daylight conditions. Technical rooms for ventilation are located on an interstitial floor directly above the Surgery and the ICU units. The care is highly specialized, treating patients from Sweden and the Nordics.

Departments:
- Surgery department with theatres
- Intensive Care Unit
- Patient wards
- Play Therapy
- Hospital School
- Physiotherapy with a rehabilitation pool and gymnasium
- Occupational Therapy
- Sterile Center shared with the entire hospital campus
- Helipad

3. Architecture and design

To repeat, the hospital campus was planned and built in the 60s and 70s, consisting of up to 10-story tall buildings with facades in concrete and green metal panels. The new children's hospital introduces a lower scale with colorful facades and elements of natural wood in both interiors and exteriors. The new building relate to the existing hospital in colour and geometry, at te same time it destigmatizes children's encounter with healthcare and is easy to recognize.

Parallel to being a hospital for children and their families, it is also a workplace that should signal professionalism and highly advanced medical services. The balance is illustrated by the project's three keywords: playfulness, feeling safe, and professionalism.

3.1. Playfulness

Play is central to children's development and health. Despite disabilities and illness, recovery and health should always be encouraged, and children heal and learn through play. Play is an important part of recovery. The environment offers and encourages various types of play and provides opportunities for meaningful activities, stimulation, and creativity. At the same time, there is also space for calm and peaceful moments. Playfulness adds something important to the large-scale, rational healthcare environments that make them human and cozy. The materials and colour scheme in areas for play encourages children to play and interact with the building.

3.2. Feeling safe

Proximity between children, staff, and relatives is crucial for feeling of safe. Division into smaller teams (5-6 patients) with open team stations for healthcare staff contributes to short distances. The rooms are designed so that children, parents, siblings and friends can spend a lot of time together and feel safe and cared for.

In the hospital's most public areas, such as the courtyard, entrance pathways, lounges, ward entrances, and dayrooms, a safe environment is created by using wood on walls and ceilings. By balconies and terraces, large glass panels from floor to ceiling open the wooden interiors to the surroundings, as warm and inviting cuts in the building volume.

3.3. Professionalism

Queen Silvia Children's Hospital provides highly specialized world-class care with very high demands on technical solutions, safety, and hygiene. The glazing on the façade has an almost technical expression, and staff areas are planned for both high focus work and interactions.

3.4. Art

Five art commissions with a child's perspective as the theme are integrated into the building and emphasize play, interaction and exploration. Play is a key part of the rehabilitation process, where art can contribute to recovery. In various ways, it is possible to interact with all art.

4. Innovation

To be able to answer to the set goals and aims of the project, extra attention was put into two different fields: the family perspective and the use of wood.

4.1. Family Perspective

Families and relatives are at the center of the children's hospital. The presence of relatives is essential for both the children's health and a participation in the care process and recovery. Their presence provides children with a sense of security and calm, while also relieving the healthcare staff of some responsibilities.

Significant attention has been given to environments for relatives, with family kitchens for cooking familiar food from home or lounge areas where siblings and friends can relax and spend time. In each patient room, there is space for two relatives to stay overnight, equipped with a sofa bed and a bed that can be rolled up to the same level as the patient's bed. Each patient room has storage for relatives and a workspace to do school work or for remote working. Moreover, some rooms have terraces similar to small townhouse gardens. The intensive care unit (ICU) also has sleeping alcoves that enable overnight stays directly adjacent to the patient's bed, as well as a separate family unit. The green indoor courtyard provides a place for breaks and relaxation in a peaceful and calming environment.

4.2. Wood in Healthcare

Sustainability and climate neutrality have brought wood materials to the forefront of the construction industry. It is known that natural materials can have positive effects on stress levels (Bernard, Kutnar 2015) and rehabilitation (Nyrud 2013, 2017). In the children's hospital, natural wood is used in spaces where patients, visitors and staff spend time. It is a material that creates a calm atmosphere, relates to the nature areas outside the hospital and gives a home-like, relaxing and pleasant feeling. We know that the hospital and caring environment is a key part in the process of healing and overall well being as it supports our restorative processes (Wijk 2014). Using wood is one of the answers to create a healing environment that can be welcoming, playful and give a feeling of safety.

In Queen Silvia Children's hospital wood is found both in public spaces, as well as in spaces for patients' recovery. It is used in the exposed structure of the south facade of the courtyard, exterior façade on terraces and balconies, the courtyard's façade cladding, glass partitions, as well as walls and ceilings in public interior spaces. Additionally, wood is a material that conveys a warmth and is nice to touch, people tend to appreciate the pleasant feeling of warmth and calmness that wood transmits, making them feel more calm and safe (Verma et al. 2016, make it wood 2016). Moreover, wood can withstand abrasion and scratch marks, still having a pleasant appearance.

In addition, wood is primarily used in spaces with lower demands from infection control. In spaces where there are no specific demands from infection control, the wood has been treated with a hard wax oil, in other areas, where the requirements are higher, it is coated with a lacquer.

To increase the amounts of used wood in treatment spaces, more research in both requirements as well as in product development is needed. There is an ongoing research project, «*Wood for Health*», that is investigating surface treatments of wood in environments with high demands on infection control.

5. Process

In this project, both healthcare staff and users have been involved from early conceptual phases to final construction documents. The result is a building adapted to the latest findings regarding both work processes and medical treatments. The UN Convention on the Rights of the Child says that children's rights to participate is a key aspect in decision making concerning them. Therefore, having inclusive planning processes with children and their families and design in a child friendly way has been of great importance.

An open process with the client and contractor has allowed for late adaptations in terms of new work methods that had an impact on the layout and design. An example is the shortage of nurses which led to pharmacists working full-time in the patient wards, with additional requirements for their working environments.

In addition, an important discussion was the idea of single patient rooms. This has been addressed in discussions on the functions and size of patient rooms, the placement of hygiene rooms, doors between patient rooms, and whether a solution with only single-patient rooms is overall the best for healthcare. Several aspects have been considered, including hygiene, infection control, monitoring capabilities, staff efficiency, and the children's feelings of loneliness (especially teenagers). The chosen solution includes occasional double-patient rooms in the ICU but majority of single patient rooms in wards.

6. Sustainability

A children's hospital places unique demands on ecological sustainability. Small children often play on the floor and like to explore surfaces with their hands and mouths. Therefore, using environmentally friendly and healthy natural materials such as wood and rubber flooring instead of plastic floors are of great importance. These materials contributes to the calm atmosphere, they are tactile and reminiscent of nature, while also having a smaller environmental impact.

Energy-saving measures have been prioritized in the project, such as using surplus heat from the sterilization center to heat the inner courtyard. Another example are solar panels on the roof.

Balancing economics, medical outcomes, and human perspectives is always a crucial (and challenging) aspect of hospital planning. A key to economic sustainability is finding positive incentives in the planning process for long-term economic viability, such as shorter hospital stays and conditions that facilitate efficient work methods for staff. Designing a caring and healing environment, is therefore of great importance.

7. Staff

The patient always comes first. However, there is also an increased focus on good staff facilities and a good working environment. It is about providing the best conditions for a full workday but also to attract the best employees in a competitive and sometimes over-heated job market. The hospital is planned with areas for staff where patients do not have access, to provide staff with their own environments have which have a completely different character than the clinical areas.

8. Key Contributors

- Client: Region Västra Götaland (administration for building and property management, former Västfastigheter)
- User: Drottning Silvias barnsjukhus, (Queen Silvia Children's hopsital) Sahlgrenska Universitetssjukhuset (Sahlgrenska University Hospital)
- Main Contractor: NCC
- Architect: White Arkitekter
- Landscape Architect: Landskapsgruppen
- Interior Designer: White Arkitekter
- Sustainability Consultant: Bengt Dahlgren
- Structural Engineer: VBK
- Electrical Engineer: WSP
- HVAC Engineer: WSP
- Fire Safety Consultant: Bengt Dahlgren
- Project Management: White Arkitekter
- Cost Estimation: White Arkitekter

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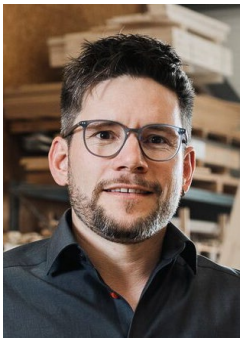
Friday 04 October

Block 6A
New opportunities

Wisdome Stockholm

A brave combination of 18th century-craftmanship and high-tech Free Form Timber construction

Martin Looser-Frey
Blumer Lehmann AG
Gossau, Switzerland



A brave combination of 18th century-craftmanship and high-tech Free Form Timber construction

Abstract

The combination of traditional, proven construction methods and state-of-the-art manufacturing methods in timber construction is increasingly being rediscovered. This combination in turn creates new opportunities to recognize and achieve previously unattainable goals.

Traditional dowelled beams or even toothed beams which has been used to build bridges as early as the 18th century, can be found in new, contemporary construction projects.

The enthusiasm for combining the advantages of tradition and high-tech is fascinating and drives us to constantly develop new ideas. For example dowelled beams are used to build huge, self-supporting roofs, always combined with the latest construction methods and techniques.

1. Wisdome; A Dome under a Gridshell

1.1. Pioneering Timber Design

The striking design by Elding Oscarson and Florian Kosche emerged as the winner of an architecture competition. The heavily curved Free Form timber roof connects the inside and outside areas of the National Museum of Science and Technology and creates a spectacular interior for the dome structure of Wisdome Stockholm. The roof geometry spans a footprint of 25 x 48 meters, without columns. On three sides of the building, a projecting roof supplements the roof support structure and brings the curvature of the roof surface level with the eaves line. The Free Form structure is based on a grid system of LVL beams. The dome below the vaulted roof is made from cross-laminated timber. A requirement of the architecture competition was that timber be used in the construction, in particular cross-laminated timber (CLT) and laminated veneer lumber (LVL).



Figure 1: Scaled Model of Wisdome Stockholm / Elding Oscarson Architects

1.2. Dovelled or toothed beams used for wooden bridges

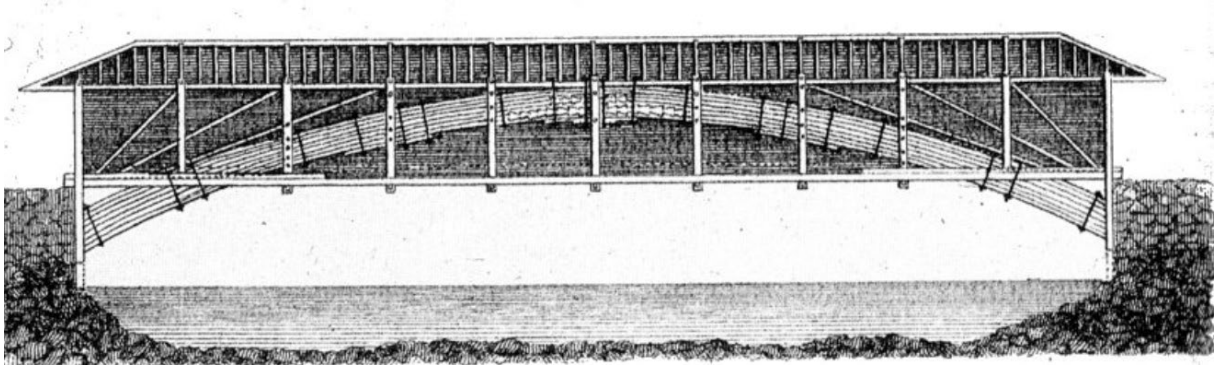
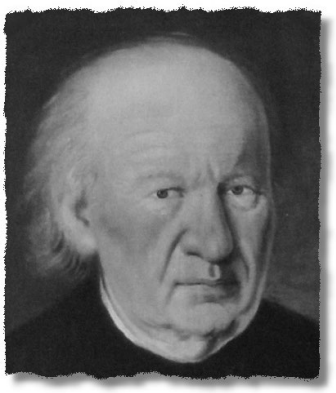


Figure 2: Bridge over the Limmat River in Wettingen 60meters span 1766 / Drawing from John Soane ca. 1780



More than 250 years bevor the invention of Glued laminated timber (Glulam/GLT), carpenter and builder Hans Ulrich Grubenmann built wide span wooden bridges over rivers and deep gorges. He mastered spans most often longer than the height of a tree with interlocking wooden beams. For a long time, toothed beam or dowelled beam technology was almost forgotten.

This system was intended to be used now for the gridshell roof of the wisdome project. So we did some Research with an wood experienced Team together to find out the right solution.

Figure 3: Johann Ulrich Grubenmann from Teufen, Switzerland, 1709-1783 / Zeughaus Teufen / Wikipedia

1.3. Partnerships in every phase

One of the main partners of the Wisdome Stockholm project is Stora Enso, one of the world's largest forestry firms. The Finnish-Swedish company also supplied all the timber construction material for the project. The flagship building marks a milestone in sustainable, climate-friendly construction. It pushes existing technical boundaries and is intended to showcase what is possible with wood as a climate-friendly construction material.

With its experience in timber construction and its technical expertise in project planning and the implementation of sophisticated Free Form geometries, we as Blumer Lehmann were the right partner to execute this ambitious project.
Supported by our partners from many previous Free Form projects, the timber construction engineers from SJB Kempter Fitze with Hermann Blumer, and the parametric planners at Design-to-Production, the Blumer Lehmann team got to work planning the highly complex roof construction in detail.

1.4. 1:1 Mock-ups for the roof construction

Two mock-ups provided the information needed for the construction design. «This allowed us to instil confidence in the client,» says Martin Looser-Frey, looking back.
The grid-shell roof support structure consists of three transverse layers of LVL beams and two longitudinal layers of beams. Unlike constructions made of laminated timber, which are bent and milled in the factory beforehand and delivered to the construction site as complete components, for construction of the main roof only the lowest panel strip layer was laminated in the required curvature and delivered as a finished component. This layer served as a scaffold and structural aid for the complex assembly process. The remaining four beam layers were only bent and dowelled during assembly on site. All the connections are designed with dowels and peg connections, which are milled from the same LVL material.

The actual dome inside the building is made entirely of cross-laminated timber (CLT). It was produced back in the Stora Enso factory in Sweden and then assembled on site underneath an assembly tent. The Blumer Lehmann team from Switzerland also carried out the parametric design, the engineering and the detail planning for the dome construction.



Figure 4: 1:1 Mock up Main Roof / Blumer Lehmann AG

1.5. Roof structure with dovelled LVL Panel strips

«Our main job was to develop a concept for the supporting structure that would allow the chosen architectural design to be implemented», remembers Martin Looser-Frey, who serves as Free Form Division Manager and is responsible for international sales at Blumer Lehmann.

For the main roof, the timber construction engineers at SJB Kempter Fitze opted for a lattice-shell structure with criss-crossing LVL beams. With interlocking dowel joints and the help of cutting-edge planning tools, these were connected to form a double-curved grid. The differently vaulted roof construction is supported around the edge by 24 solid columns that are made from block-laminated LVL with dimensions of 60 x 80 and 60 x 60 centimetres. The columns are connected rigidly to the approximately 1.20-metre-high concrete base. To limit the horizontal deformation of the roof support structure, tensioning rods are integrated into the wooden columns, to which a considerable pretensioning force is applied after assembly. A solid edge beam is arranged on the column heads to enclose the Free Form construction all the way round.

Due to the very ambitious time schedule of the project they cooperated with various wood construction partners for the production, which also have an affinity for high precision work.



Figure 5: Installation grid shell roof with dowelled girders / National Musseum of Science and Technology

1.6. Installation phase

The installation of the dome and the grid shell structure was completed in 6 months only. Due to the complete site covering with a giant tent, which had integrated the construction crane into itself, it was possible to work steadily regardless of weather conditions and to ensure an optimal construction process.



Figure 6: Finished timber construction work / National Musseum of Science and Technology

2. Construction details

Project:
Wisdome Stockholm at the National Museum of Science and Technology (Tekniska museet)

Address:
Museivägen 7, 115 27 Stockholm, Sweden

Project type:
Art and culture, museum construction

Client:
Tekniska Museet, Stockholm

Architecture:
Elding Oscarson Architects, Stockholm

Main Engineering / Supporting structure:
Florian Kosche AS, Oslo, Norway

General Contractor:
Oljibe AB, Sweden

Timber supplier and main partner:
Stora Enso, Sweden

Planning and execution Free Form timber construction:
Blumer Lehmann AG, Switzerland

Timber construction engineers:
SJB Kempter Fitze, Switzerland, with Hermann Blumer

Parametric design:
Design-to-Production, Erlenbach, Switzerland

Construction type:
Free Form Timber Structure

Execution:
Start of planning 5/2021, start of construction 7/2022, handover 12/2022

Challenges and Opportunities in Overseas Markets

Anton Wanas
RUBNER Holzbau GmbH
Ober-Grafendorf, Austria



Challenges and Opportunities in Overseas markets

Abstract

Engineered timber structures are facing a growing demand. Not only in Europe where the century old practice of using timber has evolved to the point of re-introducing timber structures into the city landscape, into multi story buildings and even mid- and high-rise buildings. What has been achieved in Europe over the last few decades is being recognized in many parts of the world.
The US, Canada, the Middle East, Asia and Australia show a very keen interest in this technology and multiple examples are proving the fact that timber has arrived in these markets. What looks quite like a perfect match at first glance, is worth having a closer look, in order to understand the drivers for such a growing demand and the challenges which all parties involved might face.

1. «Can you do it in timber?»



Picture 1: Mactan Cebu Airport, Rubner Holzbau © Christopher Colinares

«Can you do it in timber?» This is – from my personal experience – in most cases the starting point for many timber projects in regions outside of Europe.

«Why do you want to have it in timber?» I ask then, and the answers are not surprisingly very similar:

We want to bring sustainability to our projects – and show it.
- We want to apply for a green building certificate.
- We want to use renewable materials.
- We want to reduce the carbon footprint of our projects.

Overall: Engineered timber structures are related to sustainable construction like no other material.

1.1. Main Drivers for looking into timber

Sustainability aspect
Renewable material
Carbon footprint reduction
Green Building certificates
Architectural language displaying social topics of society

Prefabrication aspect
Speed of construction
Precision of fabrication
No. of on-site personnel

Creative aspect
Opportunities of forms and shapes

Project Management topics
Reliable component

2. Working in countries overseas

What like to work overseas?
Considering the highly diverse teams which are usually involved in a construction project, one can image that the level of experience when it comes to working with timber can be equally «diverse». From architects and consultants of different disciplines to main contractors and project owners – the expertise how to drive a timber project will show many different shades.

During the realization of projects, one can also experience the impact of the cultural environment in any given country. The way people communicate and interact is a mirror of culture and tradition. Especially countries in Asia require a deep immersion into local habits and customs. For Europeans it is sometimes difficult to understand why issues can't be addressed openly or why decisions can't be discussed openly and consequently corrected.

Especially the Asian societies with their young population (e.g. Philippines median age of 25,7 years vs EU with 44,5 years) are passionate and future-driven in a way we don't often see any more in Europe.
Combining this kind of passion with an intense sustainability mindset often results in the willingness to build everything in timber.
However, in order to make a project succeed, we see a necessity for mindful discussions with the project stakeholders about:
- Where does a timber structure make sense? And where not?
- What is a perfect (useful) design for using timber? And what is not?
- Does it work to simply transfer a concrete design into timber?
  (Spoiler alert: of course, not)

Very frequently asked questions:
- Climate
- Termites
- Fire

3. Challenges

With all the excitement around building timber constructions which can be really overwhelming sometimes, we shall never forget the challenges to be addressed. I believe, it is our responsibility as members of the European and global timber industry to help the market grow in a healthy and sustainable manner in the long run.

We must bear in mind that the level of detailed timber expertise is often not there, and careful advice to project teams is needed to avoid long term problems maybe due to a lack of understanding the material.

For the technical part it starts for me with a very «simple» topic: We need to make sure that the timber structures we are asked to build are fit and suitable for the local environment.



Picture 2: Pyramidenkogel, Keutschach am See, Rubner Holzbau © Rubner

Not every climate in the world is suitable for timber structures, especially fully exposed ones. Not every timber structure is suitable for getting exposed.

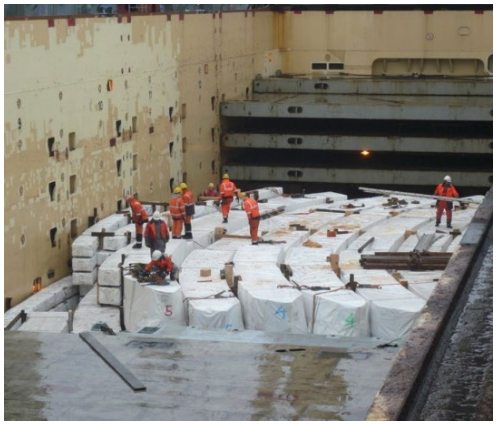
Challenge – Timber is proposed when not suitable
Fully exposed in tropical climate
Concrete designed transferred into timber

Challenge – Local Authorities and Building departments
Building codes not accepting timber
Fire design not accepted
Extra high fire resistance is required

Challenge – Adapting design and fabrication process
Design and fabrication process must match the overall program
Timber process coordinated with on-site works

Challenge – Local companies
Local companies need to gain expertise and experience
Local companies need to find their position in the supply chain

Challenge – Transportation & Logistics
Transportation and logistics are key for the overall project
Including all related topics such as insurance, wrapping, handling, storage



Picture 3: Break Bulk Cargo © Rubner

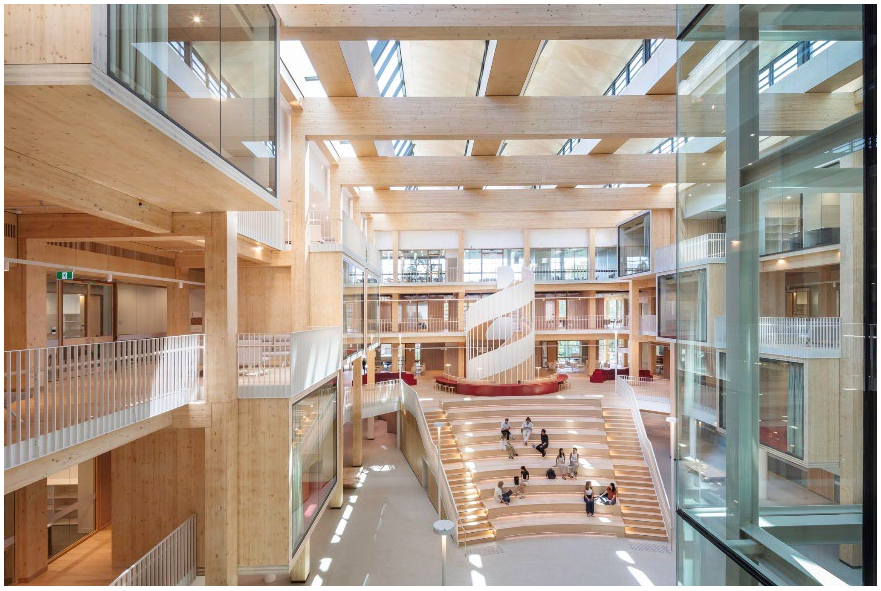


Picture 4: Cargo Vessel © Rubner

Challenge – Commercial & Legal
Most underrated challenge for all international (overseas) projects
Contractual model, customs, payment terms

Summing up, there are challenges in overseas markets. Challenges which need a lot of attention and preparation.

Most of them are solvable – and most of them are solvable on a personal basis by understanding local needs, local habits and getting involved with local culture.

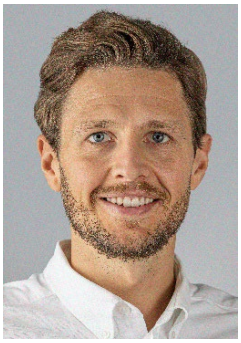


Picture 5: Macquarie University Law School, Rubner Holzbau © Fretwell Photography

Block 6B
Market demands –
as seen from the contractors

The interaction between engineer and contractor

Sondre Dolsvåg
Degree of Freedom
Oslo, Norway



The interaction between engineer and contractor

Abstract

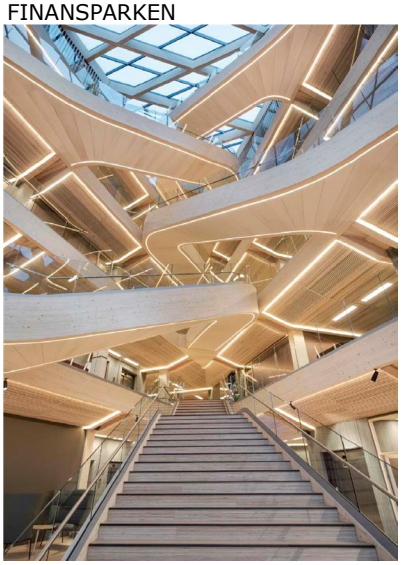
The interaction between engineers and contractors is pivotal in the successful delivery of timber projects. The role of a structural engineer can differ significantly whether hired by the General Contractor (GC) or the Timber Contractor (TC). One offer streamlined communication and coordination across disciplines, and the other offer early access to crucial timber information. The structural design approach also influences the collaboration between engineer and contractor. The engineer designs a structure from top to bottom and the contractor build the structure from bottom-up. Many times, this creates a timing conflict. To achieve the benefits of prefabricating timber components the design must be finalized before off-site manufacturing begins. This pushes the interdisciplinary coordination to an earlier stage than traditional designing. Two success criteria we have found to lead to successful projects are having a single structural engineering firm handle all phases, including the production model and drawings, and involving the TC early in the project. This approach allows for earlier decision-making and optimization, which in turn enables an earlier start on site for the GC.

1. Introduction

At Degree of Freedom we have a long history of working on timber projects, both standard buildings and complex, award winning timber structures. We are structural engineers, and we are often involved from the early stages, such as the architectural competition or initial sketches, through preliminary and detailed design, developing production models, creating production and erection drawings, and following up on-site during construction.

Example projects:

FINANSPARKEN



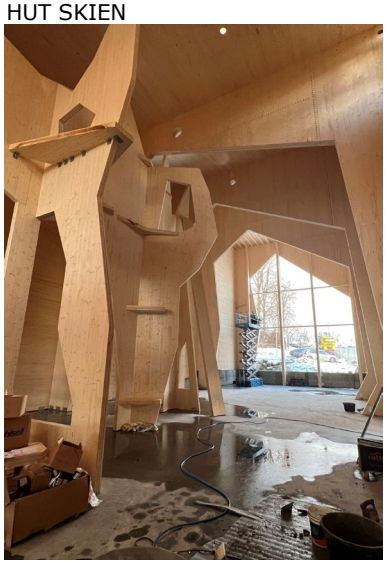
© SINDRE ELLINGSEN

SPOR X



© ZINC/THOMAS MELLBYE

HUT SKIEN



© KAI-OTTO MELAU_Høyt & Lavt

There is a long tradition of building with prefabricated steel and concrete structures. In these cases, it is common for the structural engineer to share the loads on elements and forces between them to the fabricator, who then designs the elements accordingly. However, we have observed that many in the construction industry do not realize that this approach is not suitable for timber structures. Applying this method to timber structures can lead to overly conservative results, which diminishes the competitiveness of timber compared to concrete and steel. That is why we always create our own FEM models of structures and applying loads rather than forces, to optimize the design of elements and connectors.

We have worked in various constellations in different projects:
- For the main client in the early stages, developing the concept and leaving the detailed design to the General Contractor's (GC) structural engineer.
- For the GC as the traditional structural engineer, handling the detailed design in a design-build contract, while the Timber Contractor's (TC) structural engineer takes on the elaboration of production models, etc.
- Directly for the TC, designing the timber structure and preparing the production files.

In many projects, we have undertaken all three tasks–either working for the same client throughout all phases or transitioning from the main client to the GC and finally to the TC as the project progresses. Our experience shows that having one engineering firm with the competence and capability to manage all phases typically yields the best results for timber projects.

This paper focuses on the interaction between the structural engineer and the contractor in timber building projects seen from an engineer's point of view.

2. The difference of working for the General Contractor or a Subcontractor

In design build contracts the structural engineer might be hired either by the GC or by the Subcontractor for the timber structure – the TC. The roles, the responsibilities, the incentives and the scope of work can vary to some extent. Working for the GC usually means to work in an integrated design team together with other disciplines. This collaboration allows for seamless integration of design and construction, where the structural engineer works closely with the GC and other disciplines to ensure that the designs are practical, buildable, cost-effective and in accordance with the architectural requirements. The GC coordinates the design team, and they have the possibility to influence the designs to reduce risks and to optimize construction efficiency. The main advantage of being directly hired by the GC is the streamlined communication and coordination with other disciplines in both the design team and the construction team. The main disadvantage is the lack of input information regarding the timber design. The design process can even start before the GC has contracted the TC, hence before the specifics of the timber products that will finally be applied in the project it is concluded.

Designing timber structures require knowledge of the timber properties, products design principles, codes and standards, timber types and grades, environmental considerations, construction practices, and the use of design software. A structural engineer for timber will select the appropriate type of material for a project, based on factors such as strength, durability, availability, cost, sustainability, and whether it will be exposed (visible) or not. This decision-making process ensures that the chosen material aligns with the vision of the client, the contractor, the architect and considers environmental factors.

The main advantage of working for the Timber Contractor is that this information is accessible/available from the beginning of the design process. This makes a huge difference when it comes to selecting the structural components correctly and providing accurate inputs and requirements to other disciplines. The timber engineer, working for the TC, can be part of the design team managed by the GC, or be working separately under the TC. This makes a difference when it comes to interdisciplinary coordination. When the access to information and responsibilities are fragmented, it's difficult to ensure that all components work seamlessly together. This is the main disadvantage in this role.

3. The design approach and the Contractor's needs

When designing a building the structural engineer prefers a systematic top-down approach. Designing each level of the structure in sequence, starting with the roof and working downward to the foundation. This approach ensures that load paths are properly established, elements are well-coordinated, and the building is structurally sound. If the structural engineering role is divided between concrete and timber, the concrete engineer must wait until the timber engineer has made significant progress with their design before starting their own work. The contractor on the other hand starts to build from the bottom-up and needs information about excavations and foundations first. This often creates a difficult situation for collaboration between the engineer and the contractor. The engineer wants to finish the design, and the contractor wants the information to prepare, plan and order products for the construction.

Construction projects develop in design phases. Normally it can be divided into Concept Design, Preliminary Design, Detailed Design and Production Design. Each phase builds upon the previous one, ensuring that the design progresses from an idea to a fully detailed and constructible project. For timber engineering the distribution of work, time and effort is not close to evenly distributed between the phases of a project. When a detailed design is well advanced it is easy to believe that the design process is close to finished, but it still requires a lot of time and effort to complete a timber project. Ergodomus states that the Pareto principle (also known as the 80/20 rule) is reflected in timber design (Ergodomus, 2023). They believe that only 20 % of the timber engineering is done when a project is 80 % developed. This means that the timber engineer needs to perform 80 % of the work in the last 20 % of a project's development. Detailing, clash solving, preparing production files, assembly plans, and logistics are all aspects that hugely impacts the outcome of a timber project.

Table 1: The percentage distribution of several factors between the first 80 % and the last 20 % of the work

Project Development	**80 %**	**20 %**
Time	20 %	80 %
Complexity and effort	20 %	80 %
Problems to solve	20 %	80 %
Risk	20 %	80 %

When the contractor considers the first 80 % to be enough to go on-site and completing the last 20% during the building process, the GC increase the risk of running into unnecessary costly mistakes, delays, and dangerous project changes grows exponentially.

4. Prefabrication in timber projects

Contractors in industrial timber projects benefit from increased efficiency and speed on-site compared to other types of construction. This is much due to a high quality and precise prefabrication of timber components. When components are manufactured and pre-assembled off-site, and delivered ready for assembly, it reduces the number of activities on-site drastically. This is not only beneficial to reduce time and on-site costs, but it can also reduce the complexity, risk, potential errors as well as increasing the site safety. Working in a controlled environment makes it easier to manage factors such as weather and site conditions, and to perform quality control. To minimize the downtime in the construction process, the off-site work with timber components is normally executed while the GC execute the concrete structure on-site.

To take advantage of this benefit the design must be frozen at a certain stage to avoid continuous changes, such as modifications of structural element's positions and holes. This will disrupt the workflow of the timber engineer and delay the production design. Even though this stage is planned from the beginning of a project, it's normally hard for all disciplines to develop their design to this point (Gosselin et al., 2018). Fire, acoustical, electrical, and ventilation engineers and architects all affect the timber design. For that

reason, an effective communication and coordination among these professionals are essential to achieving success. The phrase "What You Draw Is What You Get" underscores the principle of the accuracy and clarity of design directly influence the outcome of a project. This is highly applicable in timber projects.

5. References

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Engineering and innovation

Flemming Højbjerre Sørensen
Bollinger+Grohmann
Copenhagen, Denmark



Engineering and innovation

Abstract

Timber structures have a long tradition, and combined with modern design technology new possibilities for innovative structures occur. This paper deals with the design process using parametric design tools to design modern innovative interpretations of timber structures using traditional building methods. The articles present two projects with their own unique structural system.

1. Introduction

Digital design tools make it possible to design complex geometries which was difficult to develop without these tools. These complex geometries create new challenges not only for the engineer to design, but also for the contractor to build. By combining traditional low-tech timber solutions and high-tech parametric geometry models it is possible to create innovative structures. The two projects are exampling this approach to obtain modern structures.

2. Project example – Service station Torghatten, Norway

2.1. Project description

The project 'Service station Torghatten' is a collaboration between the architecture studio Atelier Oslo AS and Bollinger+Grohmann on behalf of the road authority 'Statens vegvesen' of Norway.
Three volumes where the cantilevered roof structure rests offer shelter and space for toilets and exhibition areas. The covered outdoor area below the canopy roof creates open spaces equipped with bicycle parking spaces and seating. Except for the foundation, the whole building consists of solid timber.

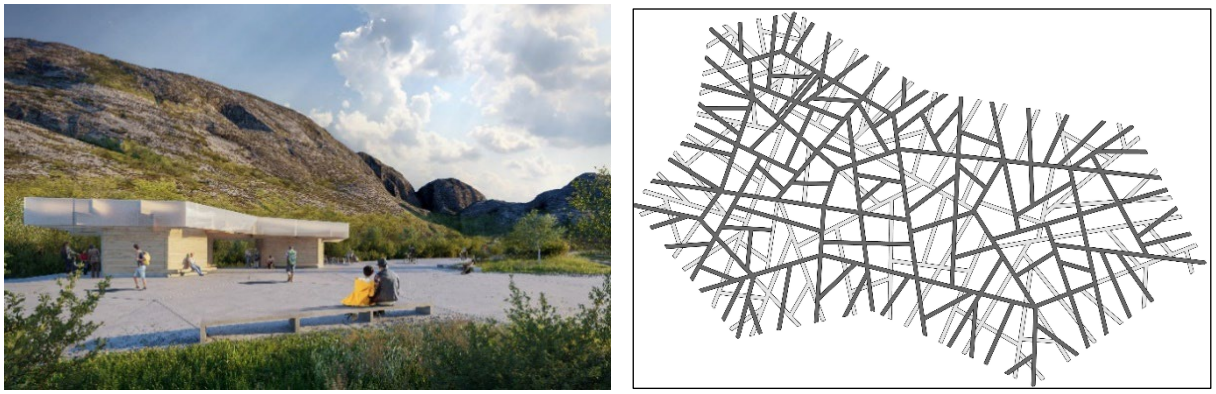


Figure 1: Left: Visualization of the service station by Atelier Oslo. Right: Plan of roof beams

2.2. Structural concept for roof

The roof structure follows the principle of beams arranged in layers to create a log structure, which has a long tradition in Norway and is known as 'lafteverk'. The construction method with its traditional roots represents a low-tech solution common in rural areas and differs from many high-tech solutions in urban construction. Each beam consists of three solid timber members, that are screwed together to create one stiffer member. The simple screw connections and the renunciation of glued timber connections allow the building to be easily dismantled at the end of its lifetime.
The roof beam is placed in two levels which on its own would be unstable, but in combination the two levels create a stable system where the acting forces travel up and down in the levels and end at the cores.

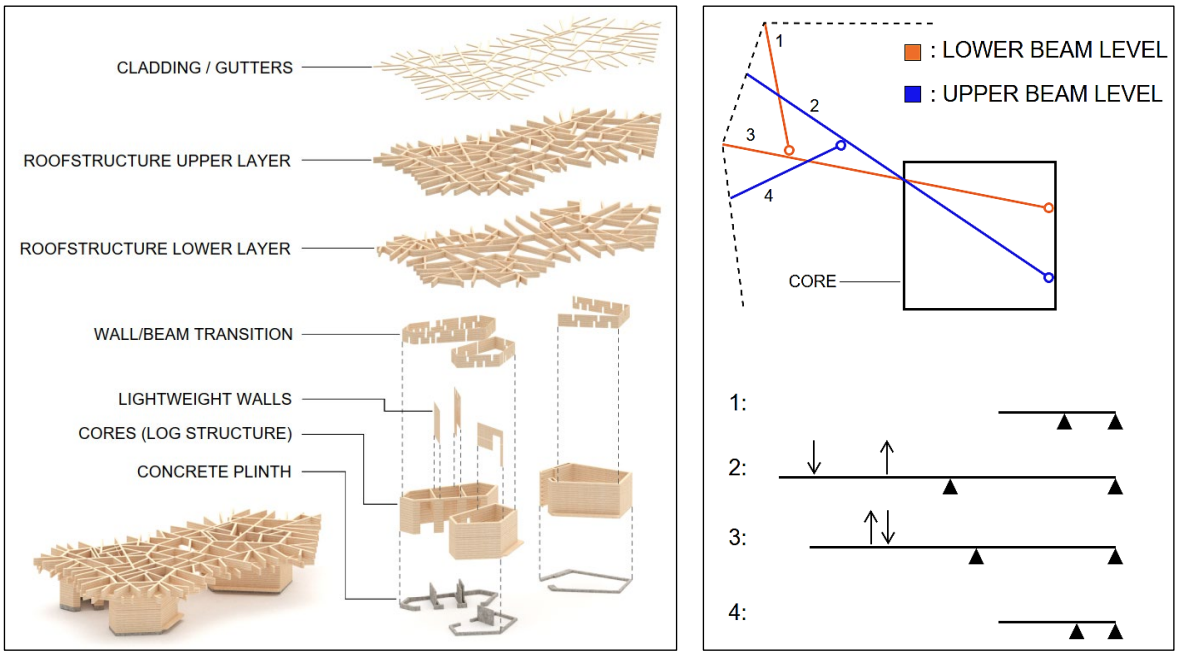


Figure 2: Left: Illustration of the service building components by Atelier Oslo (translated).
Right: Illustration of the structural concept of the roof.

2.3. Design process with parametric tools

The architectural design showed the roof geometry as a disordered interaction of beams. The task was, therefore, to optimise the beam layout in a way that combines the architectural concept and an efficient structural design. A parametric workflow was created based on the architect's geometry model by using the Grasshopper application Karamba in combination with a multi-objective optimization using the application Octopus. The results of the optimization were rated according to the three variation criteria «deformation», «deviation of the wall geometry» and «beam diversity». The architects used the optimization variants (examples shown in Figure 3) as the basis for discussion and further development of the roof geometry. The last adjustments, which led to the final geometry, were made in collaboration between Bollinger+Grohmann and Atelier Oslo based on more detailed FE calculation models using Dlubal RFEM.

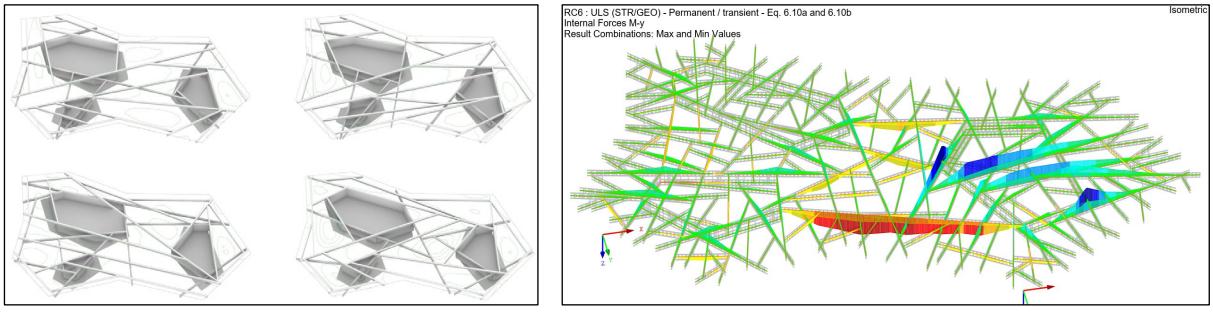


Figure 3: Left: Selection of results of the form-finding process. Right: Calculation model – moment diagram of the result beams

The parametric model was used to perform a collision check of the complex geometry to ensure constructability. By choosing a parametric workflow for this project for all elements of the structure, it was possible to automate almost the entire shop drawing process.

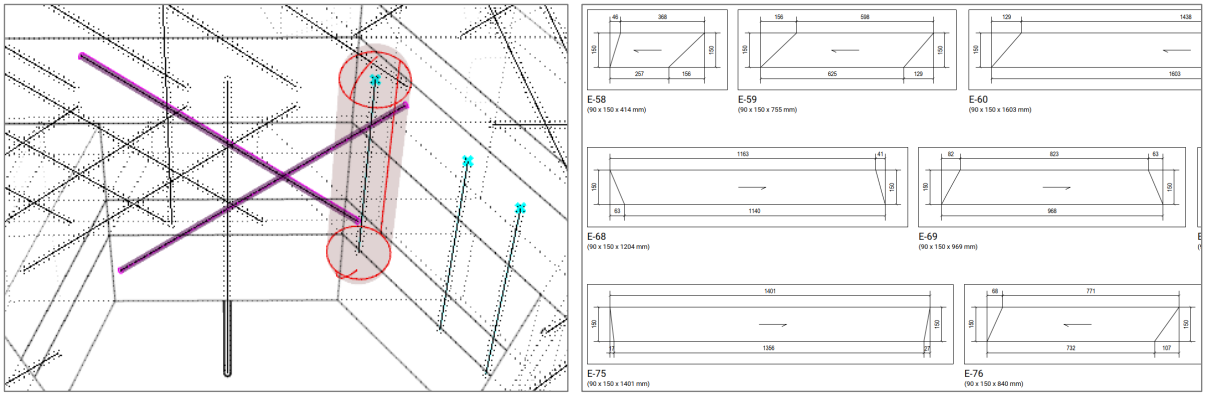


Figure 4: Left: Showcasing a collision check example. Right: Extract from the cutting drawing.

2.4. Construction of the structure

The three cores were built in a factory and transported to site by truck. The roof structure was built on site using a scaffolding as temporary support. The construction work on site started in February 2023 and the building was inaugurated in June 2024.



Figure 5: Impressions from construction site. Left: Roof structure ©Nils Petter Dale.
Right: Mounting of glass ©Idar Flostrand.



Figure 6: Impressions of completed structure.

3. Project example – Timber pavilion, Belgium

3.1. Project description

A wooden pavilion is currently being designed for a private client and contains mainly a gallery of art exhibition where the roof should be a part of the exhibition, a work of art itself. Therefore, the roof structure will visible inside and some part outside. The Oslo-based architectural firm Snøhetta designed an organic shaped roof with flat, double-curved

surfaces. Thus, a structural system needs to be created which sets not only high ambitions on aesthetics but also on the structural efficiency, costs, and environmental impacts. Consequently, the Zollinger system was chosen as a basis for designing a structural grid shell made of mostly straight beams with the help of parametrical design tools.

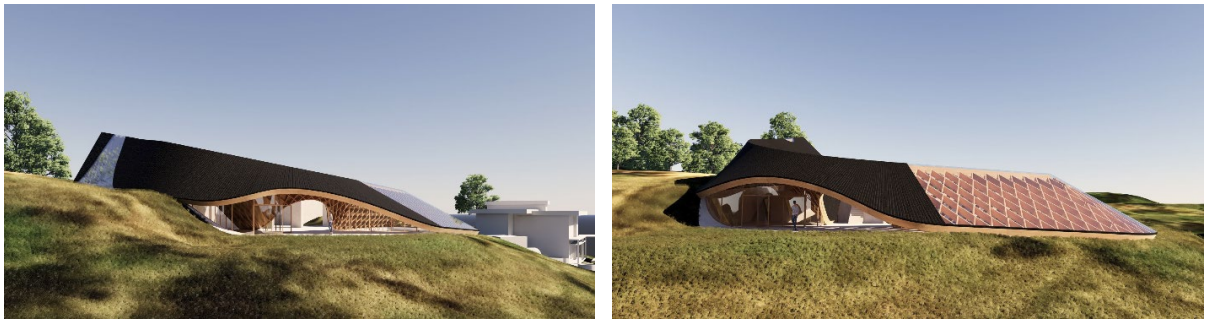


Figure 7: Exterior views ©Snøhetta

3.2. Structural concept

The Zollinger system is named after the German architect Friedrich Zollinger who patented the structure during the recession in the 1920s. Rhombical arranged lamellas form a grid shell that can span large spans, using timber beams with a limited length and simple connection details. Both factors make prefabrication possible and an easy installation process on site.
The main pattern of a Zollinger lamella roof is shown in Figure 8. Beams are arranged at angles to each other so that two lamellas meet in the middle of one continuous lamella, forming a rhombus. The pattern enables the lamellas to be connected without moment transfer and still provides a stiff system. While the forces are transferred in several directions, the Zollinger system allows for large openings. The Zollinger system is not limited to the structures mentioned above, but it also allows to form double-curved structures out of straight beams while keeping the connection details simple. The system's disadvantages are that the installation process takes longer due to the many details.

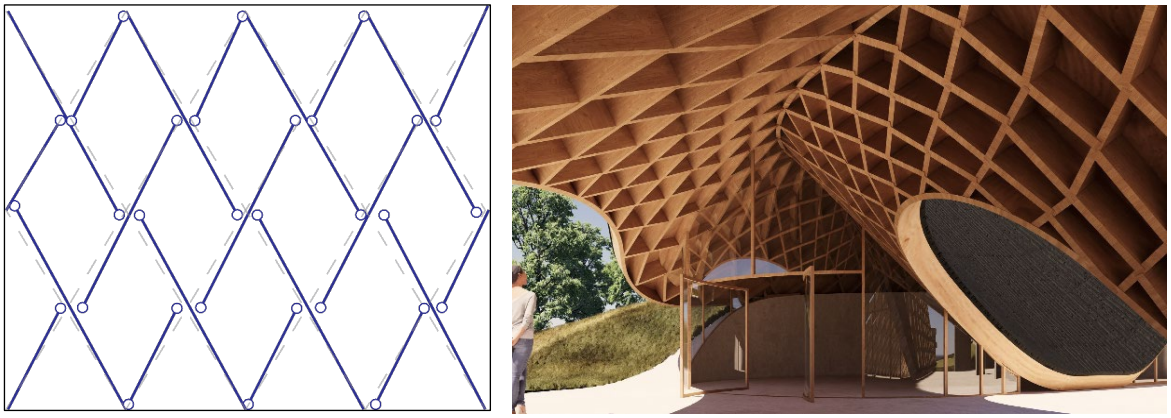


Figure 8: Left: Principal pattern of the Zollinger lamella structure. Right: Interior view ©Snøhetta

The roof of the pavilion is a composition of double and single-curved surfaces; therefore, a definition for the grid must have been found, which allows a smooth transition between the different types of surfaces, see Figure 9.

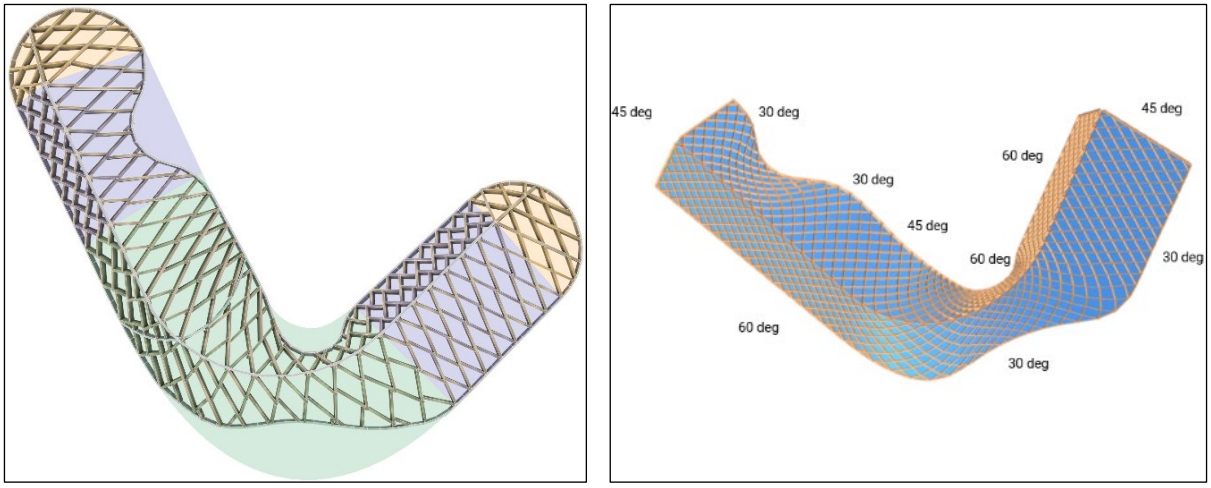


Figure 9: Different types of surfaces (left) and roof angles (right). Left: Flat surface (blue), double-curved surface (green) and double-curved, ellipsoid (orange).

3.3. Parametric design process

Parametric design tools are used to establish the mesh's geometry with the specifications and requirements e.g. straight beams, limited number of beam types and simple connection design. Based on the experience from the Zollinger roof of the Frans Masereel Centrum in Kasterlee in Belgium (engineered by Bollinger+Grohmann), different connection details were evaluated, and it has been determined to use a screwed connection for typical connections.

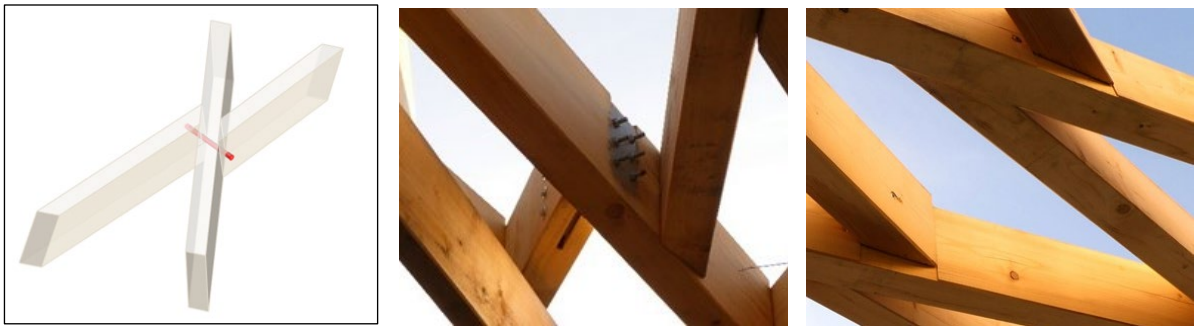


Figure 10: Different types of connection details. Left: Zollinger pin. Middle: Detail with slotted-in steel plates from Frans Masereel Centrum in Kasterlee, Belgium. Right: Screwed connection from Frans Masereel Centrum in Kasterlee, Belgium.

To define the geometry Grasshopper was used. Once the boundary conditions were defined, an algorithm was set up that easily allows to adjust of the mesh size. It efficiently and quickly contributes to the study and identification of geometrical problems, such as collisions between the Zollinger roof and the facade line or the evenness of the roof membrane. The geometry defined in Grasshopper was used to set up a calculation model in Dlubal RFEM to analyze the structural performance.

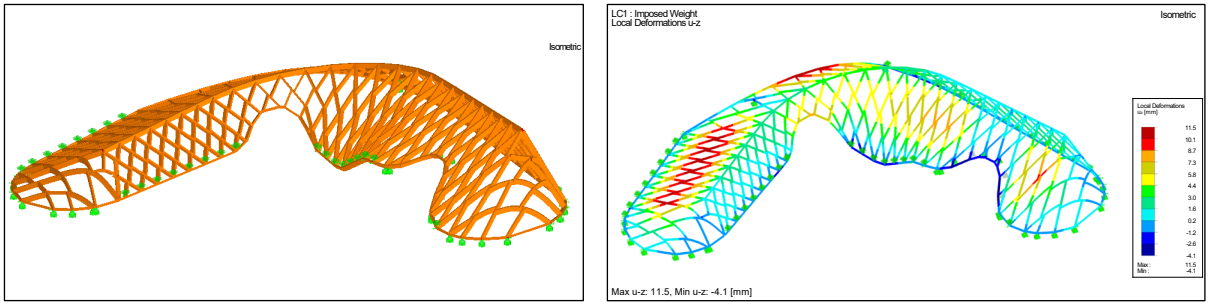


Figure 11: Calculation model in Dlubal RFEM. Left: Structural model. Right: Deformation.

Block 7A
New innovative products

The story behind a new connector

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The story behind a new connector

Abstract

Building owners often want that new buildings shall have open and flexible architecture to allow flexible use and future changes. A way to improve timber buildings in this direction is to increase the stiffness of the connections between horizontal and vertical members of the structural systems. This paper presents a new connector for moment resisting timber frames with good performance both with respect to stiffness and ductility. The paper shows the details of the parts of the connection, but the history of the development is only given at corresponding the oral presentation. The properties of the connection are published in scientific journals (see references) and is not repeated herein, though some experimental results for prototypes are given. The moment resisting connection consists of inclined threaded rods installed in predrilled holes and coupled with friction couplings.

1. Introduction

Wood is a relatively light and soft material and serviceability issues like deformations, vibrations and acoustics are the major challenges in multistory timber buildings, together with fire safety. For serviceability, small vibrations around deformations due to static loading might be problematic, as most popular timber connections have or develop decreased incremental stiffness due to slips and clearance around holes and fasteners. This can in turn give rise to vibrations perceptible to building users and cause discomfort. Connections based on gluing, glued-in rods and screwed-in rods show little sensitivity to this potential problem. Therefore, these fastening techniques are well suited for moment resisting connections intended for high rigidity with respect to the serviceability issues.

For the present connection, screwed-in rods have been chosen as these can be quickly installed, do not require curing, do not have the strong dependency to temperature as glue, and are not sensitive to the installation procedures. However, typically screwed-in rods require longer embedment length in the timber than their glued-in counterparts.

In principle, a connection in timber structures, joining a timber element to another, consists of a fastener to the first timber component and a fastener to the second component and a connector joining the two fasteners. The fasteners and the connector can be formed by one physical part, e.g. a long self-tapping screw. In that case the connection must be installed at the building site and the parts cannot be easily disassembled. By separating the fasteners from the connector, the fasteners can be installed during production at the factory and only the mounting of the parts are performed at the building site.

The moment resisting connection (MRC) presented herein is developed at NTNU, Norway through many years of research on screws and connections. This activity has engaged many MSc and PhD students as well as colleagues at NTNU, with considerable financial support from Norwegian Research Council. Review and references to other national or international work and authors on this subject are not given herein but can be found in our publications. The development tools have been a combination of numerical and analytical considerations together with experiments for validation of the structural behavior. The rotational stiffness and capacity in moment resisting timber frames combined with ductile behavior have been the governing issues for the development.

2. Beam-Column Connections

2.1. Structural Properties and Multistory Buildings

The stiffness of softwood in the grain direction is about 15 to 30 times the stiffness in the transverse direction. Obviously, to avoid excessive vertical deformation in multistory timber buildings, the grain direction of vertically loaded structural elements should be vertical. An intermediate deck of 0.5 m height with horizontal grain direction, gives much more vertical deformation than that caused by a full story height with vertical columns.

Therefore, multistory buildings based on columns and beams should have continuous columns where the beams are attached in between the columns to the side faces.

It is usually more challenging to achieve sufficient serviceability properties for multistory timber buildings than the necessary strength. The horizontal sway should not exceed H/500 to H/300 where H is the building height. Moreover, horizontal peak accelerations, usually due to fluctuating wind forces, should be limited to the order of 0.05 m/s2, depending on the actual use of the building. Furthermore, the floors should have fundamental frequencies above 8 Hz and smaller deformation due to human footfall than roughly 1 mm. These values are only indicative; it is up to the coming building owner to specify these requirements depending on the planned use of the building.

2.2. Horizontal Stabilization

The horizontal stabilization of beam and column types of multistory timber buildings is most commonly by truss work or diagonals in steel or timber, by shear walls based on wood panels, or by use of the stair/lift houses often in reinforced concrete. In the latter two cases, the horizontal forces are transferred to the ground by one or a few discrete structures, and it is hence necessary that horizontal structures like the decks, can transfer the forces to these stabilizing structures.

By use of moment resisting and stiff connections between beams and columns, it is possible to reduce the need for separate stabilizing elements like diagonals and more architectural freedom is achieved. Introduction of rigid connections also gives distributed stiffness in the building and thereby increase the robustness and reduces or eliminates the need for large force transfer between the horizontal deck elements and e.g. lift houses. Furthermore, moment resisting connections (MRC) have often large residual capacity for vertical shear forces after the moment capacity is passed.

Horizontal stabilization based on MRC has limitations and is not suited as the only stabilizing structural elements for high-rise building, say above ten stories. However, most urban buildings are between four and eight stories, and for this domain of buildings, it is an interesting feature. The connection will be more complicated than in simply pinned connections, and utilization of MRC in both longitudinal and transverse directions of the building may not be feasible. It is usually preferred to have distributed stiffness in the transverse direction of the building.

The necessary stiffness in the connections is dependent on the building layout, number of stories and the number of rigid connections. For multistory timber building less than nine stories with usual spacing between columns, the stiffness requirement will probably be in the order of 5 – 20 MNm/rad [5], [10]. It is essential for serviceability issues that no slips occur in the connections for small load variations around the static deformation.

2.3. Grain Direction and Orientation of Screw Axes

Threaded rods (long, large screws) have diameter so large that it is necessary to install them in predrilled holes in wood. Most self-tapping timber screws and threaded rods are long and slender. Due to their slenderness, they are best suited for axial loading. The stiffness of wood along the grain is in the order of 15 – 30 times the stiffness in the transversal directions, so to utilize the high material stiffness of wood in the grain direction, the preferred orientation of the rod axes should not be in transverse directions to grain. Furthermore, installation of long metal pieces with distributed stress interaction with the surrounding wood in the transverse direction, will prohibit the dimensional change of wood and give risk of cracking along the grain due to moisture induced tensile stresses normal grain. On the other hand, screws oriented along the grain might also be vulnerable to cracks, since a single crack along the grain might lead to considerable loss in strength if the crack and rod coincide. Therefore, screwed-in rods should have a minimum angle to the grain, but orientations normal to grain should be avoided. However, inclined rods give some practical implications regarding the installations and coupling of rods in a connection, as the inclination makes all coupling parts more complicated. Based on experience from experimental testing, a good choice of angle between rod axes and grain is in the range 5 to 70 degrees for these applications.

2.4. Screw Location and Stiffness

A simplified model of the rotational stiffness of a connection is obtained by the usual equilibrium considerations at the interface between the beam and the column, leading to an expression like

$$K\theta = \sum n1\ Ksizi2 \qquad (1)$$

where Ksi is the combined stiffness of all elements attached to a screw at the local coordinate zi. Often it is sufficient to model this as a series of springs including the elements in the column, the elements in the beam and possibly a coupling element connecting the rods in the beam to the rods in the column. The stiffness is proportional to the squared distance between the rods at the interface, so this distance should be as large as possible.

3. Moment resistant connection

3.1. General

The forces in the present moment resistant connection are picked up by screws, preferably in screw pairs, with timber threads in the timber members and are transferred through brackets at the tip of each screw, confer Figure 1. The brackets are connected to a metallic plate by slip resistant friction connections. From the metallic plate the forces are redistributed in a similar manner to other connected timber members according to their relative stiffnesses. The connection is suitable for transfer of any action between timber members, but it is especially suitable for rigid transfer of moment actions. Various layouts of the connection are described in [1].

A possible layout of a moment rigid friction connection between a vertical column and a horizontal beam is shown in principle on Figure 1. The column marked (14) and the beam marked (12) are both made of wood. The loads in the beam are picked up by the inclined screws in the beam (22) and transferred to a steel plate (20) by slip resistant point connections, each consisting of two brackets (24) and a pretension bolt (26) passing through holes in the steel plate (20) and the brackets. Furthermore, the forces are transferred from the steel plate to the column in the same way and distributed to the column by the inclined screws (28). The slip resistance between the steel plate and the brackets are due to friction between the mating surfaces.

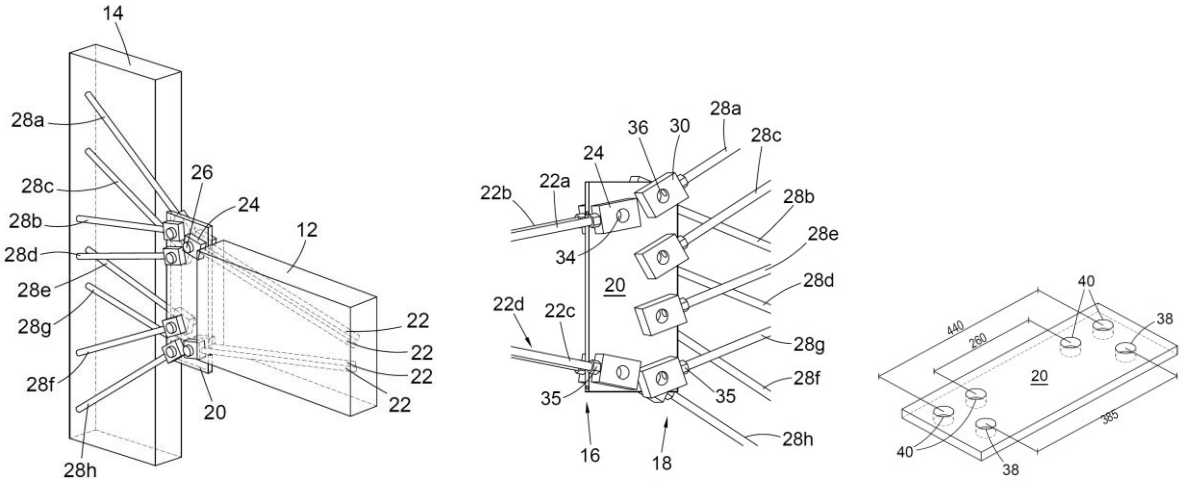


Figure 1: Moment rigid friction connector between a column (14) and a beam (12).

Figure 1 (middle) illustrates the positioning of the screws (22,28) with brackets (24,30) to the matching holes (38,40) in a metal plate (20) (Figure 1 right). The holes in the metal plate are usually oversized to accommodate for the production and mounting tolerances, and to give slip deformation in the friction coupling for load exceeding the design load. The mounting of the connection is made by letting high strength bolts (Figure 1, 26) pass through the matching holes both in the brackets (34,36) and in the metal plate (38,40). The oversized holes enable slips in the frictional coupling of the brackets onto the metal

plate, and the holes can be tuned to give the desired ductility of the connection. In the following the properties, parts and installation of the connection is described.

3.2. Properties

The long and large screws, installed inclined relative to the grain direction of the timber components, give a strong and stiff embedment to the wood in the connected members. The large screws are connected to a metal (steel) plate by a slipresistant friction connection, which is well proven and standardized connection in design codes for metallic structures like steel. The screws properties can be calculated by the design regulations in prEN 1995-1-1 [2], while the design rules for the steel parts are given in [3].

There is no initial slip or slack in this connection even for alternating load exposures. Therefore, it is well suited for horizontal stabilization of buildings due to wind or earthquake exposures. It shows very high stiffnesses especially for rotations and axial deformations. The long and large screws embedded into the timber play a similar role as reinforcement bars do in concrete. The rotationally stiffness can be made sufficiently large and thereby enables moment rigid frames in wood for up to about 8 story buildings.

The design idea of the connection is to make a very stiff connection without brittle failure modes following ordinary design principles. This is achieved by always letting the frictional resistance in the slip resistant point connection have lower resistance and give larger deformation, than all other failure modes. The key issue is here the oversized holes in the metal plate, allowing large deformation for approximately constant action and resistance. If the slip clearance is exceeded, the resistance in the connection will start to increase until an ultimate failure mode finally occurs for external actions considerable larger than the slip resistance. This approach gives a robust rigid connection with control of ductility and strength to avoid any brittle failures in timber structures.

3.3. Tested example

The mode of operation of the connection is tested in several full-scale tests, see e.g. the pictures from a connection test between a column and a beam in glulam shown on Figure 2.

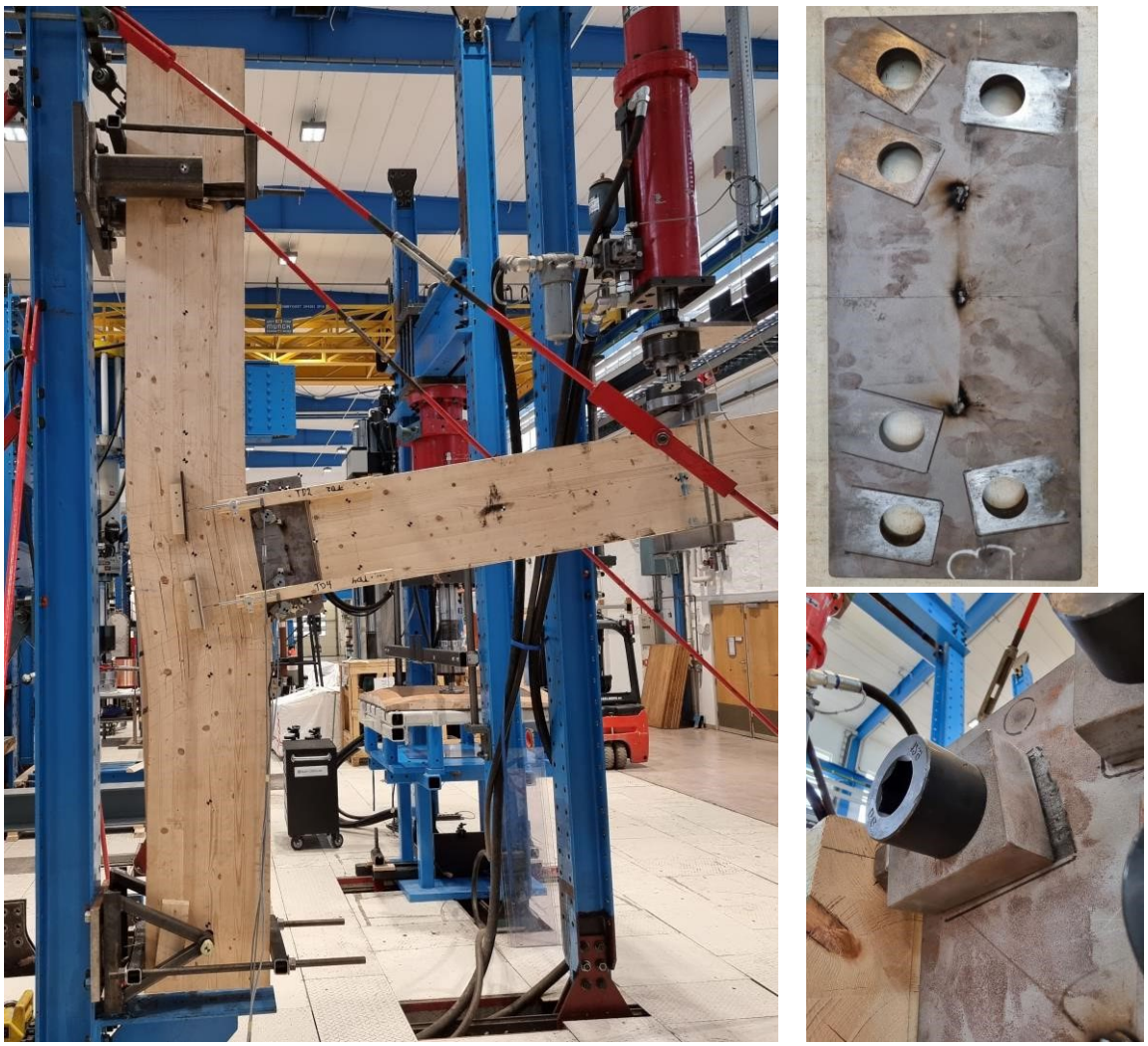


Figure 2: Full scale prototype test of moment rigid connection between glulam column and glulam beam (left), the steel plate with brackets removed after test (top right) and bracket fastening point.

The test set-up presented here was glulam GL30c with 140 mm x 450 mm and 140 mm x 405 cross sections in the column and beam, respectively. Note that this is a very small cross-section for a column, as the column size often will be governed by the fire requirements.

The connection was designed to allow maximum rotation of the beam relatively to the column of about 1.7 degrees due to the clearance in the oversized holes in the steel plate. This inclination is so large that it will give a highly visible deformations in buildings. For loads exceeding the slip resistance, the slip takes place between the brackets and the steel plate, confer the pictures to the right of Figure 2. Furthermore, the test was performed with cyclic loading at the tip of the beam with increasing cyclic deformations until the displacement capacity of the loading actuator was reached (±250 mm). The maximum loading was much higher than the design loading. In the test the maximum moment capacity exceeded 85 kNm and a ductility ratio above 5 was achieved. The ultimate strength was limited by shear cracking and splitting in the column in the connection fastening zone, confer Figure 2, which reduced the bending capacity in the column. However, the column maintained its axial load capacity. The measured rotational stiffness of the connection between the column and the beam was 10 MNm/rad.

4. Parts of each point in the slip resistant connection

In the following each part of the connection system will be described in detail. Each slip resistant fastening «point» consist of the following parts, confer Figure 1.

- Two large screws, each with wood threads and a small part with metric threads and a nut
- Two brackets
- Two optional shims (thin plates with special friction properties)
- One bolt for pretension with nut and two washers
- One (common) steel plate with oversized holes for connecting all screws in a group.

4.1. Screws

The wood screws used herein have nominally 22 mm diameter and are currently available in two different lengths, the long screws have 1000 mm wood threads, while the shorter ones have 600 mm wood screw threads. Both lengths have a metric threaded part of 65 mm length for fastening of brackets.

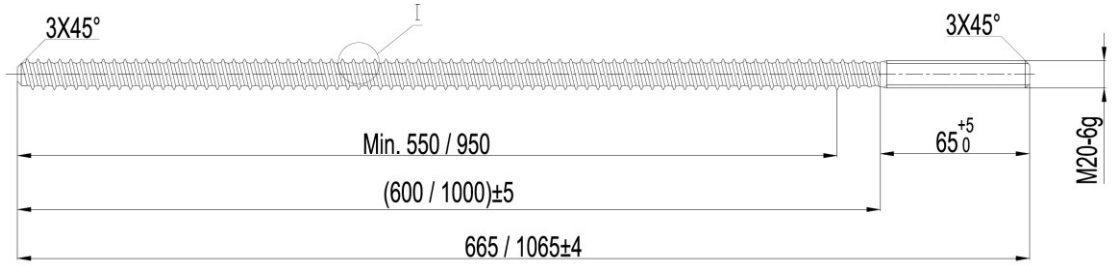


Figure 3: Wood screws with a small metric thread part.

The screws satisfy the requirements and calculation methods in prEN 1995-1-1 [2]. The material properties of the screw steel material are nominally 8.8. i.e. nominal yield stress is 640 MPa and ultimate stress is 800 MPa. The certificates of the present screws give fy = 820 and fu = 925 MPa. The nominal yield strength of the screws for axial loading is hence 147 kN, and ultimate nominal strength is 184 kN. Obtained test values for axial loading on the present screws are 190 kN and 213 kN for yield and ultimate strength, respectively.

Typical axial stiffness of this large, long screws with 22 mm diameter is roughly 100 kN/mm, dependent on wood properties and the angle to grain.

4.2. Brackets

The bracket is fabricated in structural steel. The dimensions and tolerances for M30 pre-tension bolts are given in Figure 4 a.

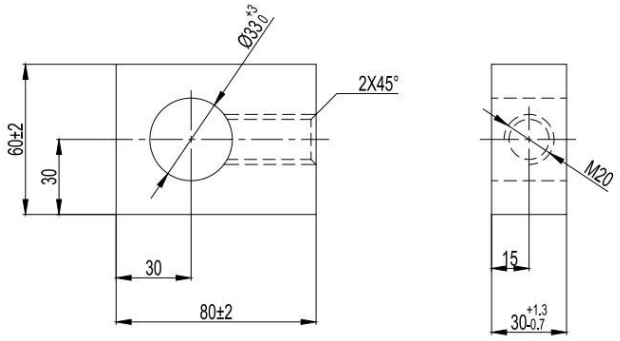


Figure 4: Brackets for fastening of screws onto a steel plate

The end with the metric M20 end of the screws is screwed into the hole with internal M20 threads and locked with a M20 nut to the bracket at the time of the assembly. The brackets are equipped with this slightly oversized hole to allow tolerances for the installation of the screws. In the normal direction of the screws the tolerances are given by the oversized holes with nominal diameter of 33 mm in addition to the oversized hole in the plate.

4.3. Plate

A metallic plate is connecting all bolts and transfers the forces between them. The force transfer is by friction in slip-resistant connections with pretension bolts passing through holes in the plate and the brackets. The thickness of the transfer plate is dependent on the required spacing between parallel planes with screws and the necessary strength and stiffness. The standard default transfer plate is a S355 steel plate with thickness 20 mm. The transfer plate shall be equipped with oversized holes for a slip resistant connection between the brackets and the plate. The oversized holes can be circular, elliptical, slotted, or of any shape. It is the oversizing of the holes in the plates which introduces the ductility in the connection by allowing slip for forces larger than the frictional resistance. The end and edge distance of holes in the transfer plate shall satisfy the minimum distances in EN19931-8 , and the clearance requirements in EN1090-2. The spacing between the center of the holes should be at least 70 mm in any direction, or the space between mating surfaces of two neighboring brackets should not be less than 10 mm to allow slip displacements. An example of a plate satisfying the minimum requirements is given in Figure 5.

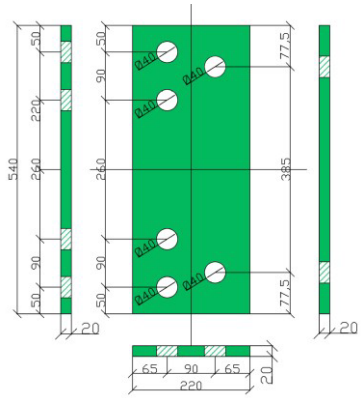


Figure 5: An example of a steel plate satisfying the minimum requirements with 40 mm holes designed for M30 bolts.

4.4. Shims

The optional thin shims are used to get accurate control of the frictional properties between the brackets and the plate. The shims can also be used to accommodate for tolerances normal to the screw axes, and they are available in a wide range of different thicknesses (0.7 – 12 mm). The standard default thickness is $tshims$ = 3 mm. The recommended width and length of the shims is the same as the brackets and is given on the drawing in Figure 4.

4.5. Bolts

All parts of the slip resistant system at a point are assembled using a high strength bolt passing through the holes in the plate with diameter larger than the bolts. The bolts and procedures for pretension should be in accordance with EN 1993-18 [3], and the Execution standard EN 1090-2 [13]. The standard default length of the bolts (M27 or M30) is 130 mm and allows the use of shims (optional) for better control of friction properties.

5. Installation

The connection requires that the holes in the metal plate, the entrance holes of the screw on the wooden surface of the timber component and the angles of the screws embedded in the timber component are all matching the layout of the planned connection.

5.1. Drilling and screwing of the wood screws

The screws can be mounted in any angles, but along the grain is not recommended due to risk of splitting along grain direction due to moisture variations or other stresses normal to grains. A minimum angle to grain of at least 5 degrees should be used, 10 degrees or more is recommended.

The pre-drilling diameter is 16.5 mm for the 22 mm screws. The screws shall be screwed into the softwood in the predrilled holes. It is recommended that all holes are predrilled before any screws are mounted.

5.2. Screws with bracket

The protruding part of a screw after installation into the wood has metric threads (M20). First a M20 nut shall be screwed on to the M20 male threads of the screw. Thereafter a bracket is screwed onto the screws (similar to the nut). The bracket is also equipped with female metric M20 threads, confer the picture on Figure 6 Finally, the nut is tightened to the bracket to avoid any initial slip between the bracket and the screw, shown on the picture on Figure 6.

5.3. Assembly and mounting of the slip resistant connection

As much as possible should be assembled prior to final erection and mounting on the building site. If not all connection can be pre-assembled, it is proposed that the side of the connection having most fastening points are pre-assembled, and the other side is used for joining of the structures at the building site. A finished assembly is shown down to the right on the picture on Figure 6.

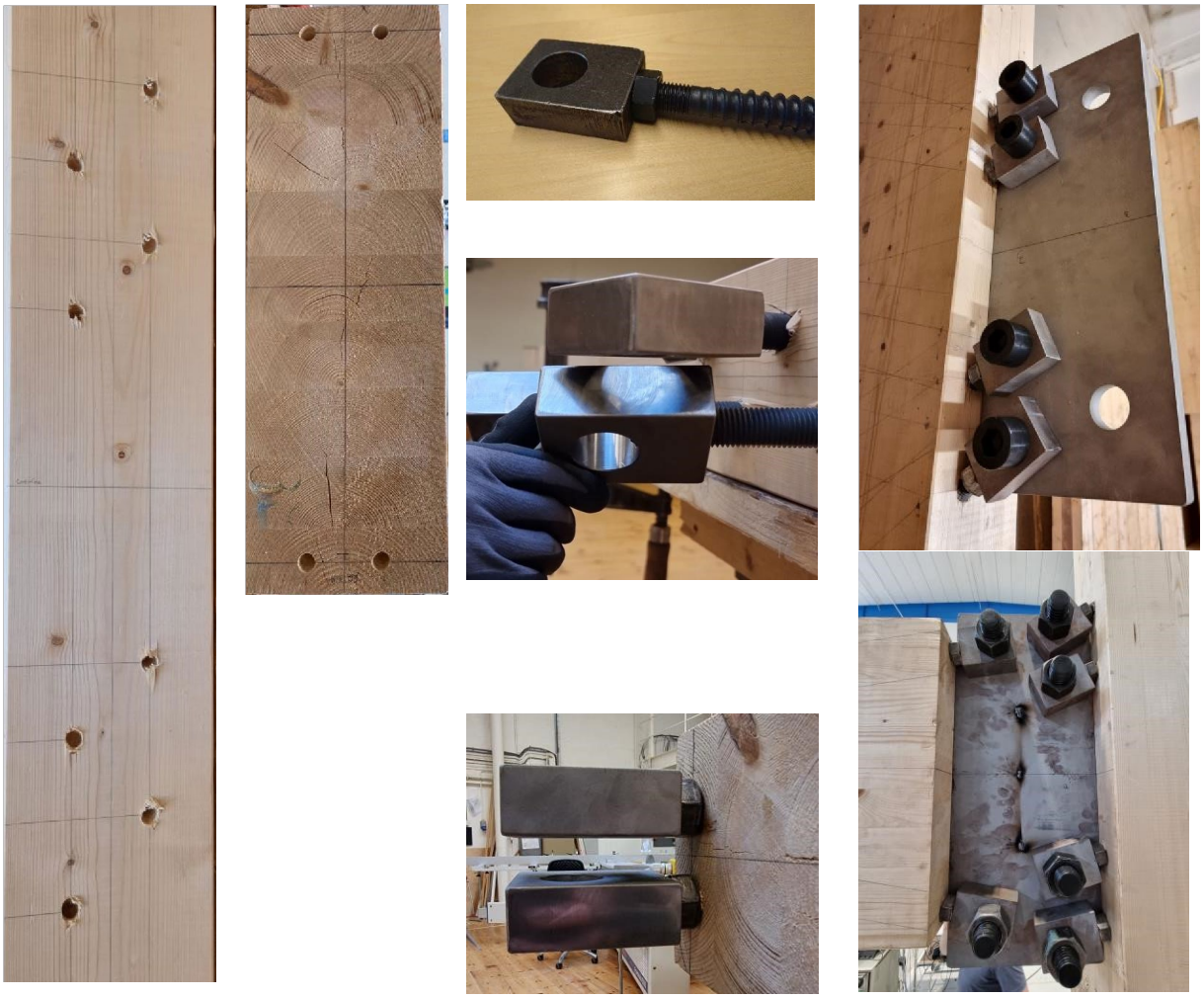


Figure 6: Predrilled holes in side-face and end grain face (left), and right mounting of brackets on screws and plate.

6. Final Remarks

Open and flexible architecture allows flexible use and future changes of buildings. A way to improve timber buildings in that direction is to have more rotational stiffness in the connections between the horizontal and vertical members of the structural system. It has been demonstrated by experiments that rotational stiffness by use of pairs or groups of inclined rods a rotational stiffness of the order of 10 MNm/rad or higher can be achieved

even for small cros-sections of glulam. Hence, it seems possible to use this type of technology for horizontal stabilization of medium rise timber buildings together with contribution from stairs/lift towers, possibly also timber based.

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Wood based products for reuse

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Wood based products for reuse

1. Introduction

1.1. The Sirktre project

SirkTRE is a Green Platform project that focuses on establishing circular value chains for timber in Norway [1]. By reusing and recycling wood materials, SirkTRE aims to significantly reduce waste and contribute to Norway's carbon reduction commitments, potentially accounting for 8% of the country's targets.

The project involves various stakeholders, including forest owners, waste management companies, the timber industry, technology firms, public building owners, architects, and industry organizations. Their collaborative efforts are geared towards creating sustainable solutions and promoting green growth through research and innovation.

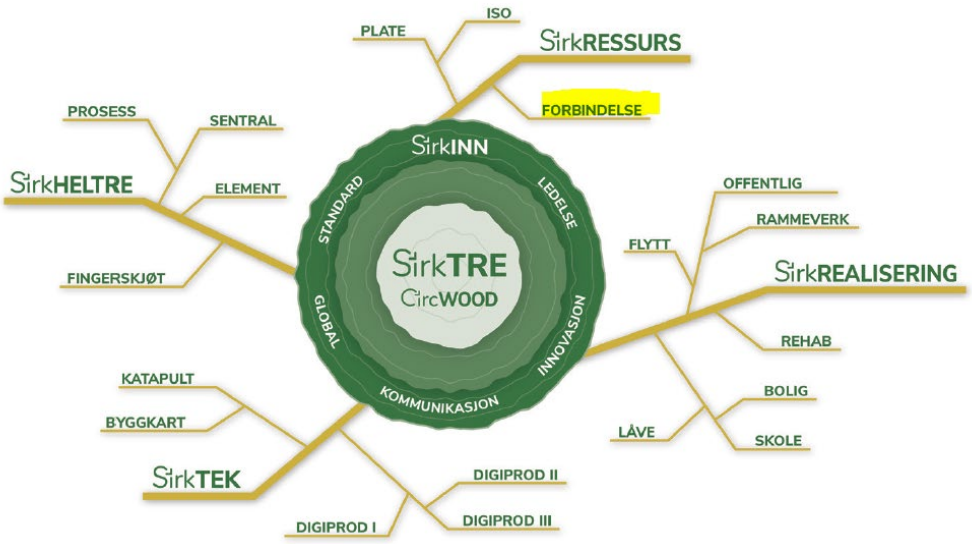


Figure 1: Mind-Map of SirTre project [1], with the subproject "Forbindelser"

1.2. The SirkTre's subproject «Forbindelser»

SirkRESSURS, a branch of SirkTre, comprises three subprojects: Plate, Iso, and Forbindelser. The Forbindelser subproject, spearheaded by Aanesland Treindustri, is in the process of devising a structural unit focused on the creation of wood-to-wood connections. This approach aims to reduce metal usage, streamline the repurposing of timber structures, and extend their lifespan.

Forbindelser is dedicated to the development of solid wood connections for industrial use. The primary goal is to reduce the reliance on steel, which enables the reutilization and recycling of timber structures. Through the standardization of wood-to wood connections, the aim is to enhance competitiveness when compared to conventional steel and concrete structures.

Our milestone involves the design of connections, testing, modeling, and the creation of prototypes. Our deliverables include the development of methodologies and solutions for design, as well as the standardization of connections.

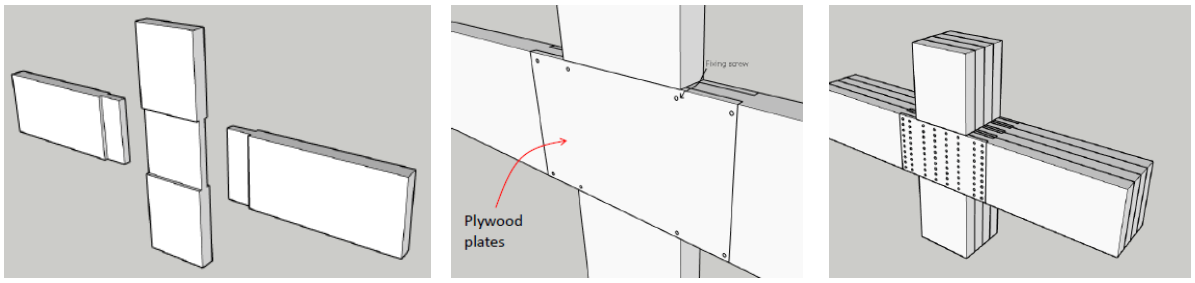
Currently, we are developing methodologies and solutions for design. We have made progress in the testing of this technology on a laboratory scale, with partial success. New research projects are being identified to further test and refine this technology, and we are actively seeking effective methods for calculations. Nonetheless, challenges persist, including the balancing of practical requirements like dimensional constraints and aesthetics with structural demands. Optimizing various aspects remains a concern, and we continue to address practical execution on production sites, including precision, moisture conditions, and timber types. Ensuring cost-effectiveness in this context is essential, as are finding pilot customers willing to shoulder some risk.

Additionally, production processes and customer satisfaction are aspects that require continuous attention, as does marketing the innovation to a wider audience.
In summary, while we have partially tested the technology at the laboratory scale and are actively exploring further research opportunities and improved calculation methods, several practical and logistical challenges remain to be addressed as we move towards full-scale implementation and standardization.

2. Connections for post and beam glulam structures using birch plywood plates and screw

Typically, the major challenge in timber structures – in order to make these sufficiently reliable, strong and stiff – is the choice of the connection.
For multi-storey timber buildings with post and beam type load bearing system, a typically adopted connection is that which make use of slotted-in steel plates and dowels.

In this type of connection, both steel plates and timber need to be predrilled before they are delivered to the building site. Then, steel plates are fitted into pre-cut slots in the glulam, and then fixed into place with steel dowels. This type of connection requires a very high degree of accuracy. In case of erroneous placement of the holes – which is not a very uncommon event – the connection needs to be adjusted at the building site, typically by re-drilling a number of holes or by enlarging their size, thus affecting the load-carrying capacity of the connection. A way of enhancing this typical connection, both in terms of ease of installation and of environmental impact, is to adopt gusset plates of high-strength wood based products instead of the steel plates.
The main idea is to build up the wanted cross section by a number of «packages» with small widths as in Figure 2.



a) b) c)

Figure 2: Phase of join production a) Mill the glulam locally to leave space for plywood, b) Insert plywood plates and fix the to the glulam by a few small screws, c) Put the packages together. Predrill holes. Insert the screws to the entire width.

Not many studies have been conducted on timber connections with gusset plates made of timber material, the investigations on connections with timber slotted-in plates being even rarer. There has not been performed much research on using birch plywood as gusset plates in beam-column connections but using plywood in timber truss joints is studied.

Douglas-fir plywood gusset plates for timber trusses have been studied by [2], [3], [4]. More recently, a truss system consisting of structural timber members connected one to another by LVL gusset plates, the so called «Ariane system», have been introduced by Jean-Luc Sandoz [5]. This truss system has the capability of reaching spans up to 60 meters.

Lately, experiments have been performed on timber truss members joined by means of gusset plates, of both LVL and plywood, subjected to tensile load [6], [7].

Post-beam connection consisting of slotted-in plywood plates in combination with either full-threaded self-tapping screws or small-diameter dowels has been objected of a comprehensive experimental investigation on the load bearing capacity [8].

This investigation also includes a study on the influence of the plywood plate's orientation in relation to the direction of load. Three different failure modes in the plywood gusset plates were observed during testing, namely: tensile failure, compression failure and shear

failure. A configurations with a screwed column connection failed first in compression, at the top part of the plywood gusset plate (see Figure 3 a) and eventually in tension, at the bottom of the plywood plate (see Figure 3 b and c). The stresses that caused the failure developed as a result of bending of the plate.

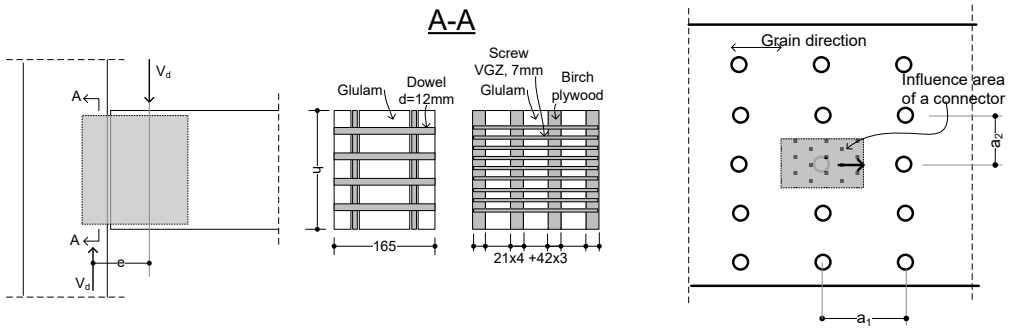


a) b) c)

Figure 3: Failure modes observed experimentally: a) Compression failure of the top part of the plywood gusset plate due to bending; b) tension failure mode in case of 0-degree oriented plywood; c) tension failure mode in case of 45-degree oriented plywood.

The local compression failure on the top part of the plywood gusset plates occurred approximately at the same location, regardless the orientation of the face grain,see Figure 3 a. The final failure, however, was affected by the face grain angle of the plywood. The fracture of the 45°-oriented plywood plate showed a «zig-zag» pattern, while the 0°–oriented plywood displayed a nearly vertical fracture. For the 0°–oriented plywood plate, all layers orientated parallel to the tension forces tore in bundles on different points of the layers. The layers orientated perpendicular to the face grain separated between the fibres. For the 45°-oriented plywood plate, the situation was different; in this case, the failure was a combination of bundle rupture and failure along the fibres.

Regarding the use of birch plywood as a replacement for steel plates in beam-column connections, it can be concluded based on this investigation that beam-column connections with gusset plates of birch plywood and screws could reach sufficient capacities for realistic design in the Ultimate Limit State (ULS).



a) b)

Figure 4: The two analysed solutions for the connection between a beam and a column, left: i) (total dimension 165 mm) of and right: ii) (total dimension of 168 mm). Definition of the influence area of a connector

In order to study the feasibility of the connection investigated in this paper in practical applications, a simplified comparison with a (well-proven) traditional connection is performed. Figure 4shows a schematic sketch of a connection between a beam and a column, both made of glulam with grade GL30c, in a multi-story residential building. The connection is designed so that, at the column part, the number of connectors is limited to the minimum necessary to resist the shear force, in order to reduce the bending stiffness at this part, thus drawing a very limited amount of bending moment here. (The eccentricity moment $V_d \cdot e$ is assumed to be taken by the connection at the beam side).

Table 1 gives a general depiction of the strength and of the efficiency of the different types of connectors. The efficiency Is defined as the strength of a fastener divided by its influence area.

Table 1: Influence area, strength and efficiency of dowels dd=12, S355 and self-tapping screws for the connection shown in Figure 13 and Figure 14.

Type of connector	Type of connection	Influence area $a_1.a_2$ [mm^2]	Strength of a single fastener [kN]	Efficiency [N/mm^2]
Dowel d_d=12, S355	i	2160	49	~23
Screw, d_s=7mm, non-predrilled	ii	1225	25	~20
Screw, d_s=7mm, predrilled	ii	784	25	~32

As it can be noticed, the efficiency of connections type i) (with steel slotted-in plates and dowels d_d=12 mm) is very similar to that of connections type ii) (with birch plywood and self-tapping screw d_s=7mm), when the screws are not predrilled. In case the screws are predrilled, connection type ii) is approximately 40% more efficient than connection type i).

3. Connections for post and beam glulam structures using birch plywood plates and wooden dowel

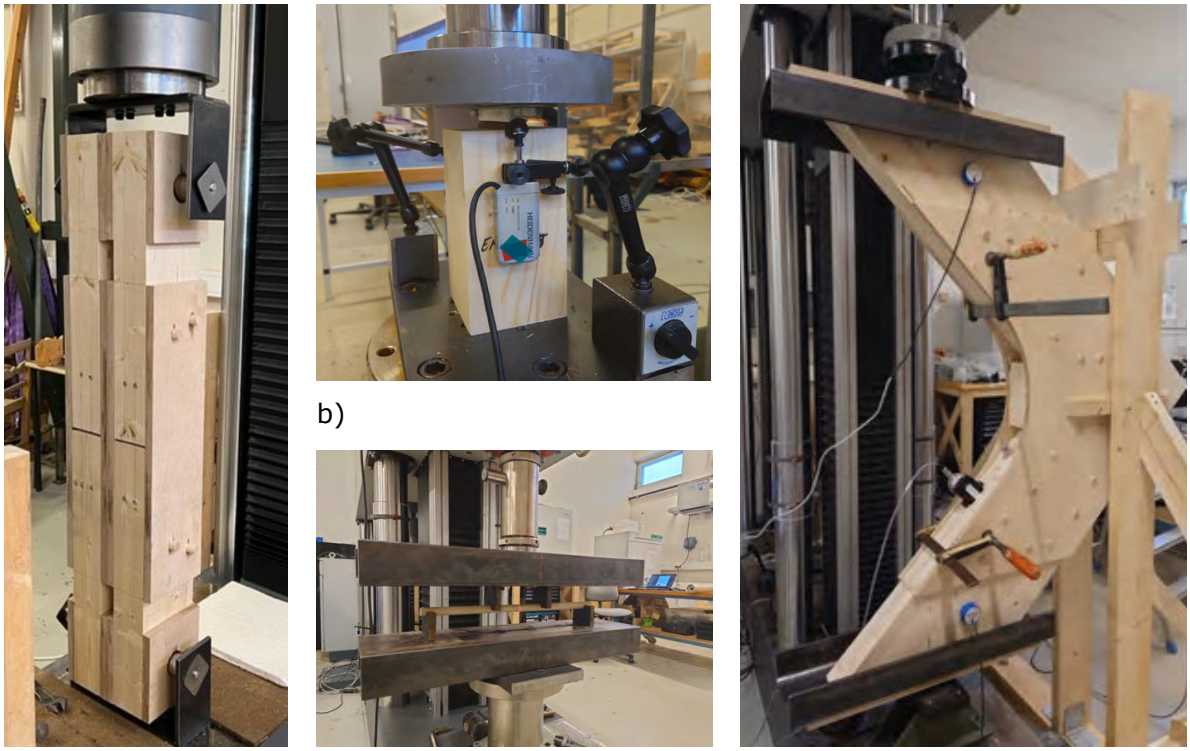
Within Sirktre Project, a new connection is proposed with the aim to reduce the disadvantages of the traditional connection with slotted-in steel plates and steel dowels are addressed by substituting steel components with wood products, i.e. birch plywood gusset plates and wood dowels.
Several investigations available in the literature were conducted on the properties of wooden dowels connectors [9], [10]. Some studies proposed new design methods for predicting both strength, stiffness and other mechanical properties of a connection consisting of wood as the only material.
What is missing is an extended database of experiments allowing to validate the mechanical models presented in literature. Some master thesis works performed at the Norwegian University of Life Science tried to cover these gaps with a series of experiments with birch plywood and different dowel type connectors: beech dowels, birch dowels, laminated Densified Wood dowel and full-threaded screw [11], [12].

In a preliminary study at NMBU experimental investigations of pure wood connections were conducted by substituting the steel plates and fasteners with gusset plates of birch plywood and laminated densified wood dowels [11]. Previous studies have investigated laminated densified wood, which proved that the material possesses impressive capacities. This further study confirmed that laminated densified wood had potential as a fastener in a timber-to-timber multiple shear connection. For the sake of comparison, two additional test groups were made, one with steel screws, and one with birch dowels. A tensile load was applied to the connections, and the load-deformation behavior was recorded with the setup illustrated in Figure 5 a). One of the results was that a 20 mm densified wood dowel showed similar short-term capacity compared to 9 mm steel screws.

In a subsequent Master thesis at NMBU ad-hoc tests on the isolated wooden connector wewre performed to determine the properties to be used in the analytical models [12].
The embedment strength of a connector is a crucial characteristic property used in the European Yield Model. For that purpose embedment tests have been performed. For this purpose, the half hole procedure was used to determine the embedment strength [13], see Figure 5 b). The primary goal of the embedment tests was to determine the embedment strength of glulam and plywood and verify the adjusted formulas given in the new proposed standard. LDW dowels resulted in strengths of 26,2 MPa for glulam parallel, 13.2 MPa for glulam perpendicular, and 51,6 MPa for plywood. This closely matched the analytical material estimates, except for an underestimating glulam perpendicular.

It was found thatthe predictions according to the current draft of Eurocode 5for wood dowel embedment strength deviated significantly from experimental values. Conducting embedment tests with birch also proved to be difficult, as the dowels underwent substantial deformations.

Four-point bending tests according to EN408 [14] were then performed to find the shear and bending properties of the birch and laminated densified wood dowels see Figure 5 c). The bending and shear tests aimed to determine the bending strength of birch and LDW dowels. The shear strength was determined by modifying the traditional four-point bending tests by shortening the distance between outer and inner supports.
The bending strength ended up being an impressive 101,3 MPa for birch and 168,9 MPa for LDW. The adopted test setup to determine the shear strength of the dowels, on the other hand, proved to be ineffective as the observed failure was due to bending also when the span between outer and inner supports was reduced. Therefore, these tests need to be run again with a new setup.



a) b) c) d)

Figure 5: Experimental activity on wooden dowels connections a) ;b) ; c); d)

In order to validate the analytical approach based on the tested material properties, eight frame corner joints were constructed and tested in this study with the setup according to Wang Y. et al. [15], see Figure 5 d).

The analytical estimations of pure-wood, timber-to-timber connection were generally accurate. The birch estimation had the best fit. LDW exerted 30% higher resistance than anticipated from the current Eurocode 5 draft. The difference may be due to the yield moment calculation method and/or to therope effect, which was neglected in the analytical model. Overall, the embedment-, bending- and shear tests revealed that LDW dowels are significantly stronger and stiffer than birch dowels. The final timber connection endured a moment of 36,4 kNm for birch and 63,3 kNm for LDW. Birch had a rotational stiffness of 19 kNm/deg, and LDW had 33,6 kNm/deg. The latter is slightly higher than the similar study performed by Wang Y. et al. [15] using metal fasteners.

4. Conclusion

It can be stated that timber connections entirely made of wood products can be a plausible solution for more environmentally friendly and circular timber structures. The strength and the ductility of moment stiff connections utilizing birch dowels with diameter d=20mm are comparable to the ones ofsimilar connections that utilizes full-threaded screws with diameter d=9mm. It was observed that in general dowel of densified wood, although stronger and stiffer, showsignificantly lower ductility than similar birch dowel.

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Standard framework for new products

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Standard framework for new products

Abstract

Within the past couple of years, the Eurocode 5 has been reviewed and also the chapter «Connections» was redrafted and updated. The content in the 2004 edition of Eurocode 5 was updated regarding harmonisation, reduction of number of National Determined Parameters (NDP's), simplification, effective material usage (positive effect on climate change) and safety. A special focus was put to the ease of use while new design rules and new material parameters were integrated into the chapter. The new content includes design rules for bonded-in rods, brittle failure modes such as row shear, shear plug and block shear as well as modern carpentry connections, and others. In addition, it allows an easier integration of new materials and material paraemters and, hence, supports innovation and opens up to new areas of applications. In this paper an overview and insight in the developments of the new chapter «Connections» in Eurocode 5 is be given.

1. Introduction

Within the project to review and update Eurocode 5 chapter «Connections», the CEN committee TC250/SC5 has installed Working Group 5, where the different European countries are represented, and technical discussions are carried out. A Project Team SC5.T5, which was formed from the authors of [1], has drafted the main content of the Connections chapter of prEN 1995-1-1:2023 [2]. Based on the existing content in the 2004 edition of Eurocode 5 [3] the Project Team worked towards harmonisation, reduction of number of National Determined Parameters (NDP's), simplification, effective material usage (positive effect on climate change) and safety. The following, more specific tasks were given in addition:

- Reduction in number of National Choices (NDPs)
- Enhanced ease of use
- Bonded in rods
- Redraft design rules with non-continuous (i.e. –stepped) output
- Row shear, shear plug and block shear
- Design rules for modern carpentry connections
- Connection geometry (spacing, end-, edge distances, etc.)
- Lateral load-carrying capacity, European-yield-model (EYM)
- Fatigue

The completion of the specified tasks forms the basis for the updated chapter «Connections» in Eurocode 5 (see also Figure 1). In addition, the general development of connection technology and new typologies of connections and fasteners were implemented into the new chapter. In contrast with the 2004 version of the Eurocode 5 new clauses are accompanied by background documentation.
In addition to the specific tasks described above, handled by the Project Team, Working Group 5 in CEN TC250/ SC5 included the latest development and state-of-the-art in connection technology in the revision of the Eurocode 5, such as modern self-tapping screws, connections with inclined fasteners (screws), connections with interlayers, design assisted by testing, and others.

2. Overview of the connections chapter

2.1. General

Easy design of connections is essential for increasing the use of timber structures and thereby take advantage of the low environmental impact. Presently section 8 of the 2004 edition of EN 1995-1-1 «Connection with metal fasteners» only covers a few of the combinations of fastener types and wood products used in practise. There are also non-physical jumps in the design rules. To facilitate the ease of use the clauses in the updated chapter «Connections» in Eurocode 5, are restructured.

The new structure allows an easier understanding of the design of connections also by engineers not working with timber structure on a daily routine. The new structure as well as other proposals for the update of EC5 were developed within the work of COST Action FP1402 [4]. One of the challenges in this restructuring was the question of the size and page count of the revised version of Eurocode 5. Due to the large number of new topics, approaches, and technologies, the size of the new EC5 increased considerably compared to the previous version. The relative size of the different chapters of prEN 1995-1-1:2023 are shown in Figure 1. However, more emphasis is placed on clarity and completeness than on pure brevity, and therefore the increase in size is also justified in view of the many new contents. Some details on the content of the chapter Connections of the new Eurocode 5 is presented in the following.

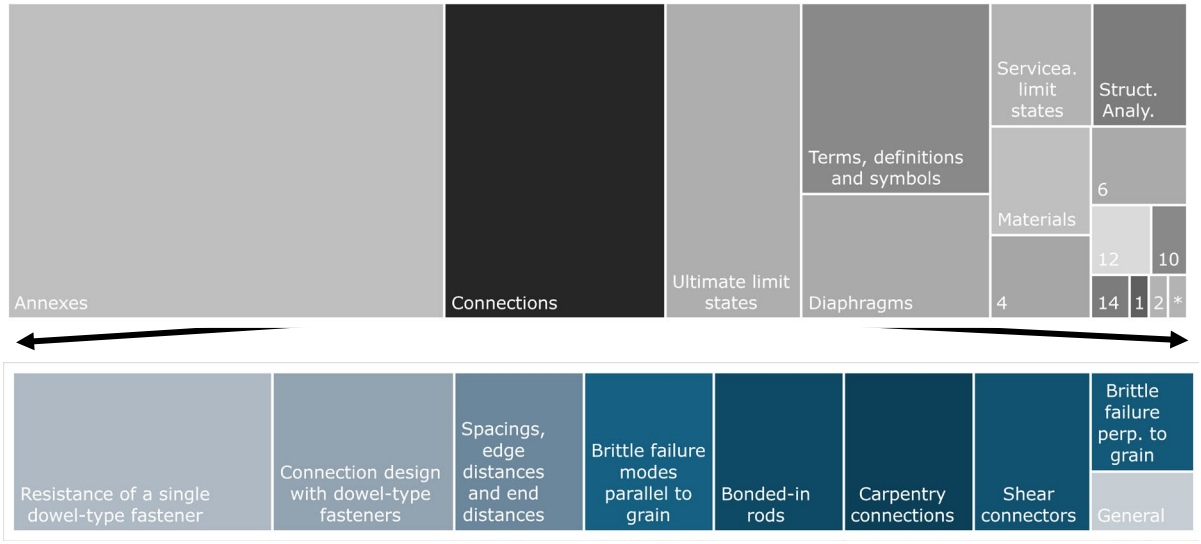


Figure 1: Content and amount of prEN 1995-1-1:2023 and Chapter 11: Connections.
Legend: 1 Scope, 2 Normative references, 4 Basis of design, 6 Durability, 10 Fatigue, 12 Mechanically jointed and glued webbed or flanged beams, 14 Timber foundation piles, * Bibliography

2.2. Axial resistance of a fastener

Modern fasteners such as self-tapping screws show highest performance when loaded in axial direction. For the axial resistance three different failure modes are distinguished:

- Head pull-through resistance $F_{pull,k}$ of the fastener head on the timber surface
- Withdrawal resistance $F_{w,k}$ of the thread of the fastener in the timber member
- Tensile resistance $F_{t,k}$ of the fastener itself.

The design axial tensile resistance $F_{ax,t,d}$ of a connection with axially-loaded fasteners can be written as follows:

$$F_{ax,t,d} = n_{ef} \min \begin{cases} \frac{k_{mod}}{\gamma_R} \max \begin{cases} F_{pull,k} \\ F_{w,k} \end{cases} \\ \frac{F_{t,k}}{\gamma_{M2}} \end{cases} \quad (1)$$

The two first failure modes describe failure in the timber whereas the third failure mode describes a pure steel failure. The corresponding partial factors γ_R and γ_{M2} are applied accordingly. In a connection between two timber members, the head pull-through and withdrawal resistance has to be checked in both members and the tensile resistance of the fasteners has to be satisfied. In case the fasteners exhibit both head pull-through and withdrawal resistance in one member, the higher of the two may be used for the design. For partially threaded screws or screws with double thread and without head, of course only the respective resistance is used directly. The effective number of fasteners n_{ef} in the connection depends amongst others on the insertion angle, type of connection, and the number of the fasteners in the group.

For the determination of withdrawal resistance, the former withdrawal parameter f_{ax}, which is related to the withdrawal length and diameter of the fastener, was replaced by a new

withdrawal strength f_w, which is based on the withdrawal length and circumferences of the fastener. This new withdrawal strength f_w is now a real "strength" based on the cylindric circumference area of the thread and not only a parameter. This new definition creates a ratio of π between the old withdrawal parameter and the new withdrawal strength. However, it is considered as a more general and consistent expression which is applicable to all dowel-type fasteners according to EN 14592 as well as to bonded-in rods and large diameter screws and threaded rods. In addition, it is in line with the general us of small letter f for strength values instead of parameters.
The values of withdrawal strength for the calculation of the withdrawal resistance and the formulas for the calculation of head-pull-through resistance are given in clear tables for the different fasteners and material combinations. New materials such as CLT were added, and values are updated to most recent knowledge. This allows a quicker design and direct comparison of different fastener types and material performances, and also the addition of parameters for new products, such as new timber species used in Europe or even globally.

2.3. Lateral resistance of a fastener

General

One aspect that was criticized by engineers in a survey on their satisfaction with EC5 [4] is the complexity of the existing equations for the determination of the dowel-type fastener capacity in EC5. These equations commonly referred to as the European Yield Model (EYM) are the basis for the design of connections in all types of layouts as timber-to-timber or steel-to-timber in single or double shear. In the updated chapter the equations of the EYM were revised and simplified in order to reduce the number of different cases to consider in the different layouts. This resulted in only a single set of equations which means a considerable reduction in length and complexity of the chapter. In addition to this reduction the ease of navigation and guidance for an optimal design of connection is achieved by giving equations for recommended thicknesses in order to achieve the desired beneficial failure modes.
The lateral resistance $F_{v,k}$ of a dowel-type fastener is established by a dowel-effect contribution $F_{D,k}$ and a contribution from the rope-effect $F_{rp,k}$ as specified in Equation (2). In the so-called European Yield Model (EYM) in the 2004 version of Eurocode 5 both these contributions are merged into the same equation. In the new chapter for the revised Eurocode 5 the dowel-effect contribution and the rope-effect contribution are distinguished more clearly and calculated separately. The rope effect contribution may be considered for the failure modes with an inclination of the fastener or when hinges develop in the fastener. However, it must not be considered in the failure modes with pure embedment failure of the fastener in the timber.

$$F_{v,k} = F_{D,k} + F_{rp,k} \quad (2)$$

Dowel-effect contribution of a fastener in single shear

Instead of distinguishing a variety of different configurations of timber-to-timber and steel-to-timber connections, in the new chapter it was focussed on the clear definition of the basic formulas and an explanation and definition of parameters for a more universal use. For a connection between two members with a fastener in single shear, the 6 well known failure modes can be defined as shown in Figure 2 (top). The equation for the determination of the load-carrying capacity is the same as for timber-to-timber (or wood-based panel) in the 2004 version of Eurocode 5. For connections with fasteners in double shear the resistance can be determined from the same formula by omitting the failure modes that cannot develop in this configuration as shown in Figure 2 (bottom).
There are several advantages of the new formulation of the EYM approach and by using only the very general equation. One of them is that only one set of formulae are necessary to cover all combinations for one, two, or even multiple shear planes. The new approach is applicable for a wide range of materials used in modern connections, such as softwood and hardwood materials, panels made of Engineered wood products, aluminium, high strength steel, and others. When being able to define the embedment strength for all these materials, the resistance for connections between members made of all these materials can be determined. This applies also to connections between timber and steel members, by considering a sufficiently high embedment strength of the steel plate. For steel plates

the embedment strength is typically three times the yield strength and hence, the assumption of an embedment strength of 600 MPa is adequate. This way, the possible embedment failure modes in the steel plate is directly considered, which is particularly important for internal steel plates.
With the new formulation the differentiation of «thin» or «thick» steel plate is automatically considered and can be evaluated in detail, that means whether the fastener rotates in the respective member or not. Extreme combinations of material parameters can be chosen e.g. for aluminium plates and high-grade steel dowels or high strength steel plates and low-grade fasteners.

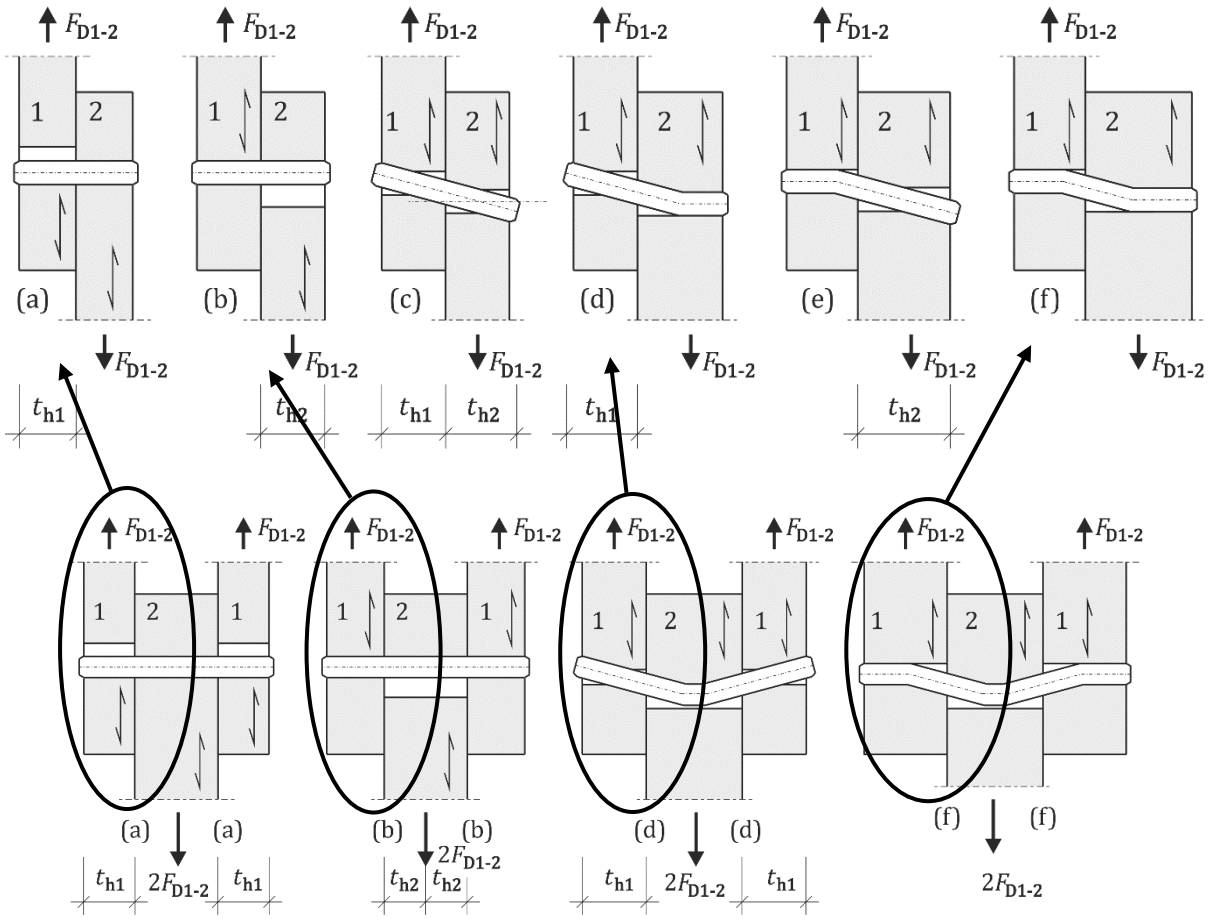


Figure 2: Failure modes according to the EYM for a connection with a laterally loaded fasteners in single shear (top) and double shear (bottom).

Material parameters
In the formulas for the determination of the dowel effect contribution the characteristic embedment strength and the characteristic yield moment are considered as basic material parameters. These values are summarized in clear tables for the different materials and fastener types. New materials or fasteners can be easily added to these tables to allow for future innovation or wider application around the globe.

Rope-effect contribution
In a connection with laterally loaded dowel-type fasteners, the load-carrying capacity of the entire connection in shear consists of the dowel-effect contribution and a part arising from friction between the timber members as well as from the axial load-carrying capacity of the deformed fastener contribute. This latter contribution from the axial force in the deformed fastener to the shear capacity of the connection is referred to as rope effect. In general, the rope effect is 25% of the axial resistance of the fastener. For coated staples and rink shank nails a higher contribution is possible. The rope effect is limited in relation to the dowel-effect contribution for the different fastener types. In the updated chapter the rope effect contribution is more clearly addressed and all the values for the different

fastener types are tabulated, hence, new values for other fastener and material combinations can be added.

2.4. Annexes

Besides the common connection types specified in the main body of prEN 1995-1-1:2023 also recommendations for some additional connection types are given in the Annexes.

- Connections with punched metal plate fasteners (PMPF)
- Connections with three-dimensional connectors
- Connections with expanded tube fasteners
- Connections with interlayers

These recommendations address some more industrially used connection types such as the PMPF, but provide also information about novel and more specialised connections such as the expanded tube fasteners offering high ductility. The specifications for connection with interlayers provide extensions for the EYM for other configurations with stiff or flexible interlayers, that can occur in connections with intermediate insulation materials, connections through sheathing on studs, or reinforced connections with densified plywood. The design equations are quite lengthy that is why proper programming in spreadsheets is recommended for the design.

3. Conclusion

The revision and updating of the connections chapter for Eurocode 5 by PT5 and WG5 of CEN TC250/SC5 has resulting in a new chapter considering all the aspects of modern connection in timber structures. By including updated values of material and fastener properties, the chapter provides the basis for an efficient design of modern timber structures. New types of fasteners but also traditional connections not yet considered in Eurocode will provide the full range of connection technology to the engineer. The possibility to update and include new property values for new materials or fasteners in the tables, will facilitate keeping the standard up to date and will enable also the use of the chapter in other parts of the world outside Europe.

4. Acknowledgements

This article is based on the contributions [1], [5] & [6] at WCTE 2021, IHF 2022, and WBB 2024, respectively.

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Block 7B
Global market trends –
as seen from the industries

Global market Trends in Timber Construction

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Global market Trends in Timber Construction



Source: Generated by ChatGPT

Abstract

We live in a very dynamic time span where, in addition to all the trends, humans must again be the focus of attention in the coming years. With a critical eye and a high affinity for recognising trends, the following three global market developments in the wood industry and in timber construction, among others, will be in the global focus in the coming years:

Growing focus on sustainability and circular economy
Demand for sustainable building materials is growing worldwide, with wood at the centre of it. There is a trend towards increased use of certified wood from sustainable sources to minimise the environmental footprint. In addition, circular economy concepts are gaining importance, in which wood products are recycled or reused at the end of their life cycle to conserve resources and reduce waste.

Technological innovation and digitalisation in wood processing
The use of technologies such as Building Information Modelling (BIM), robotics and artificial intelligence is transforming the timber construction industry. These innovations enable greater precision, efficiency and cost-effectiveness, from planning to production and assembly. Modular timber construction and prefabricated components are becoming increasingly popular as they reduce construction times and improve quality.

Urbanisation and the rise of mass timber in high-rise construction
In urban centres around the world, interest in timber as a building material for multi-storey buildings is growing. Mass timber, in particular cross-laminated timber (CLT) and other engineered timber products, is seen as a sustainable alternative to steel and concrete, enabling the construction of environmentally friendly and low-carbon high-rise buildings. This development is being driven by municipal governments and architects looking for environmentally conscious construction methods to meet the growing demand for living and working space in cities.

The following content has been generated by the perspective of an innovator and networker, CEO / Head of Business Development at n`H International AG and as a board and committee member of Lignum Holzwirtschaft Zentralschweiz. These various disciplines are also the recipe for success of my activities in the national and international market.

n'H International is a specialist in timber engineering. We are a partner and a link between timber contractors, architects, engineers, investors and builders.
Our aim is to optimise the use of timber as building material. We stand for quality, innovation and reliability. The mother company neue Holzbau AG has been developing timber solutions for more than 40 years.

The research and development for practical applications has also provided, for example, what is probably the best connecting component for sophisticated and high-performance timber construction: The GSA Technology

1. Global Market Insights

Growth in timber construction:
The engineered wood market (e.g. glulam) is expected to grow by 6.52% annually until 2030. This market is expected to grow from USD 276.8 billion in 2021 to around USD 458.8 billion in 2030. The market for cross-laminated timber (CLT) was valued at USD 1.11 billion in 2021 and is expected to expand at a compound annual of 14.54% over the next few years.

Strong growth countries:
North America and Europe are leading the way in timber construction, with countries such as Canada, Germany and Sweden being particularly important players. Canada is the largest exporter of wood products worldwide, with exports worth USD 29.7 billion per year. China and the USA are the largest importers of wood, which underlines the strong demand for wood products in these countries. The USA imports USD 35.3 billion worth of wood annually.

Increasing importance of solid timber:
Solid timber construction, in particular the use of cross-laminated timber (CLT), is gaining importance worldwide, not only in traditional timber construction markets, but also in countries such as Japan and the UK.

Sustainability and the environment:
Using wood in construction can reduce a building's carbon footprint by up to 69%, which continues to drive the trend towards sustainable building materials. Wood also stores significant amounts of carbon, making it an important material in the fight against climate change.



Various media releases on the trend towards timber construction Source: Research n`H International

2. Business opportunities

The future of global timber construction offers enormous opportunities through sustainable building practices that reduce the carbon footprint and provide an environmentally friendly alternative to conventional building materials.

In addition, technological innovations such as prefabricated timber modules and modern engineered woods promote efficient construction processes and enable the construction of state-of-the-art, energy-efficient buildings worldwide.

As a result, there are business opportunities – here are 5 of them (excerpt/incomplete):

- Sustainable timber construction for climate protection
- Prefabricated timber modules and modular construction
- Development and distribution of modern wood-based materials
- Timber technology for energy efficiency in construction
- Wood recycling and circular economy

3. Challenges



Source: Research n`H International / Bigstock / Woodness

Are we on the right track to tackle these (and other) challenges? Are we considering the individual country and project specifics?

How do we deal with climate change, and what influence will this continue to have on our economy? What influence do the ambitious climate targets have? Do we have enough wood? Or: do we use the wood efficiently and optimally - in the best possible cascade use? What influence do new legal (partly obscure) new regulations have on our work, our global trade relations? How do we solve the problem of the lack of skilled workers?

As an industry, we have to respond to this very soon, today, tomorrow at the very latest, try to evaluate and rethink solutions.

4. Current situation / Downsides

From the perspective of the processing industry, we see the following international (especially in Europe) situations and downsides from our position in Switzerland: (Incomplete list)

- Timber construction must be planned as a timber construction – this is still not clear to the mainstream
- The early involvement of a timber specialist in a project is the key to success
- Many architects and engineers from the conventional construction industry are showing interest – and think they can do timber construction. Unfortunately, this has a damaging effect on the image... without any timber specialist
- Everyone is shouting about DFMA / BIM in connection with timber construction – there are very few good examples

- In the whole timber construction trend, the upstream stakeholders also seem to have forgotten how to plan -the delays are becoming more and more significant and have more impacts on larger project volumes
- Wood can burn - and it seems to burn differently all over the world
- Wood can get wet and cause problems

5. Outlook / Summary



Source: Bigstock / Woodness

Three global market developments in the wood industry and timber construction:
- Growing focus on sustainability and circular economy
- Technological innovation and digitalisation in wood processing
- Urbanisation and the rise of mass timber in high-rise construction

The global timber industry is set to enjoy a bright future characterised by innovation, sustainability and growth. With a growing focus on the circular economy and sustainable construction practices, timber companies have a unique opportunity to significantly reduce the carbon footprint of the construction sector while laying the foundation for a greener future. Technological innovations and digitalisation in the timber processing sector are also revolutionising the efficiency and precision of construction projects. Finally, urbanisation, combined with the rise of solid timber in high-rise construction, is opening up new markets and making visionary construction projects a reality.



Source: Bigstock / Woodness

Now is the time to embrace the strength of wood and put the timber industry at the leading edge of global transformation!

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